ENGINEERING HANDBOOK

LIBRARY

R- 107

hydrology

HYDRO LIBRARY

Hydro Lib

LIBRARY

SUPPLEMENT A

FLOOD CONTROL DISTRICT
OF
MARICOPA COUNTY
3335 W DURANGO
PHOENIX ARIZONA 85009

FLOOD CONTROL DISTRICT
OF
MARICOPA COUNTY
3335 W DURANGO
PHOENIX, ARIZONA 85009

section

4

Errata Corrections made.

U.S. DEPARTMENT OF AGRICULTURE

SOIL CONSERVATION SERVICE

80.3-00-1- 00/00

PREFACE

This revision of the Hydrology Guide For Use in Watershed Planning contains five parts. Part 1 presents a general introduction to hydrologic requirements in the Service. Part 2 gives flow charts and brief discussions of principal hydrologic tasks. Part 3 gives working details and examples for the various steps shown in Part 2. Part 4 shows the application of the procedures of Parts 2 and 3 to specific projects. Part 5 contains definitions and conversions. The parts are further divided into sections on major subjects.

The first version of the Guide was distributed to Service hydrologists in November 1954 for use and testing. Their suggestions for revisions were sent to the Staff Hydrologist of the Engineering Division. A two-week meeting was held in Washington, D. C. during September 1955, in which the Director of the Engineering Division, the Staff Hydrologist, the Staff Specialist in Infiltration, a member of the Planning Division, hydrologists of the Central Technical Unit, and hydrologists of the Engineering and Watershed Planning Units participated, together with hydrologists of the Forest Service. Proposed revisions were discussed in general meetings. Committees were formed to outline specific revisions, and the Committee reports were reviewed in a final general session. Agreements on general recommendations were reached and referred to the Staff Hydrologist for use in revising the Guide.

The revised Guide was prepared by Victor Mockus, hydrologist, Central Technical Unit, under the general direction of H. O. Ogrosky, Staff Hydrologist, Engineering Division. Each of the following contributed to the development of the Guide:

Andrews, R. G.	Hydrologist, E&WP Unit, SCS,	Fort Worth, Texas
Carmichael, E. F.	" " " "	Spartanburg, S. C.
Cowan, W. L.	Head, Central Technical Unit, SCS,	Beltsville, Md.
Dawes, J. H.	Hydrologist, E&WP Unit, SCS,	Upper Darby, Pa.
Dorroh, J. H., Jr.	" " " "	Portland, Ore.; (field office, Riverside, Calif.)
Erichsen, F. P.	" " " "	Milwaukee, Wisc.
Holtan, H. N.	" Central Technical Unit, SCS	Beltsville, Md.
Kent, K. M.	" E&WP Unit, SCS,	Lincoln, Neb.
Mockus, Victor	" Central Technical Unit, SCS	Beltsville, Md.
Morey, H. F.	Forester, Forest Service,	Washington, D. C.

Musgrave, G. W.	Staff Specialist (Infiltration),	SCS,	Washington,D.C.
Rosa, J. M.	Hydrologist,	Forest Service,	Ogden, Utah
Sharp, A. L.	"	E&WP Unit, SCS	Portland, Ore.
Whelan, D. E.	"	Forest Service,	Upper Darby, Pa.

Many others in Federal and State agencies have contributed useful material or helpful suggestions. These contributions are acknowledged in the text where possible.

Although portions of this Guide are in a tentative stage, the Guide is for official use by Soil Conservation Service hydrologists. Field personnel and other users are encouraged to submit comments and suggestions for improvements. Revisions of individual parts, sections, or pages will be made available as they are prepared.

For additional information on the contents of this Guide, refer to H. O. Ogrosky, Staff Hydrologist, Engineering Division, Soil Conservation Service, Washington 25, D. C.

TABLE OF CONTENTS

NATIONAL ENGINEERING HANDBOOK, SECTION 4, SUPPLEMENT A

The Hydrology Guide For Use in Watershed Planning

	Page
PART 1. INTRODUCTION	1.1-1
1.1 General	1.1-1
Purpose	1.1-1
Scope	1.1-1
1.2 Responsibility of the hydrologist in the selection of methods	1.2-1
General	1.2-1
Range of problems	1.2-1
Selection of method	1.2-1
1.3 Other technical guides	1.3-1
PART 2. PROCEDURES	2.1-1
2.1 Watershed conditions	2.1-1
2.2 Data conditions	2.2-1
2.3 With streamflow and rainfall data available	2.3-1
2.4 With streamflow data only	2.4-1
2.5 With rainfall data only	2.5-1
2.6 With no rainfall or runoff data on the watershed	2.6-1
2.7 Concordant flow	2.7-1
2.8 Regional analysis	2.8-1
2.9 Snowmelt	2.9-1
Degree-day method	2.9-1
Rosa's snowmelt method	2.9-1
2.10 Watershed yield	2.10-1
2.11 Design hydrographs	2.11-1

	Page
PART 3. WORKING DETAILS	3.1-1
3.1 Reconnaissance	3.1-1
3.2 Base map	3.2-1
3.3 Evaluation series	3.3-1
Purpose	3.3-1
Types	3.3-1
Selection	3.3-1
3.4 Precipitation data	3.4-1
Purpose	3.4-1
Instrumentation conditions	3.4-1
Sources of data	3.4-1
Historical storm series	3.4-2
Synthetic storm series	3.4-2
Area rainfall	3.4-2
Point rainfall. Expansion to area.	3.4-2
Storm transposition	3.4-2
Orographic influences	3.4-3
Time distribution	3.4-3
Antecedent rainfall	3.4-3
Storm duration	3.4-5
3.5 Streamflow data	3.5-1
Purpose	3.5-1
Sources of data	3.5-1
Instrumentation conditions	3.5-1
Historical and synthetic flood series	3.5-2
Peak rates of flow	3.5-2
Flood volumes	3.5-3
Antecedent flood conditions	3.5-3
Runoff transposition	3.5-5
3.6 Stream reaches and cross sections	3.6-1
Purpose	3.6-1
Locations for reaches	3.6-1
Process of locating reaches	3.6-2
Alluvial fans	3.6-2
Location of cross sections	3.6-3

		Page
3.7	Hydrologic soil groups	3.7-1
	Purpose	3.7-1
	Determination of hydrologic soil groups	3.7-1
	Soil arrays	3.7-2
3.8	Land use and treatment classes	3.8-1
	Purpose	3.8-1
	Crop rotations	3.8-1
	Native pasture and range	3.8-1
	Farm woodlots	3.8-1
	Commercial forest	3.8-1
	Miscellaneous	3.8-2
	Straight-row farming	3.8-2
	Contouring	3.8-2
	Contour trenches	3.8-2
	Terracing	3.8-2
	Determination of land use and treatment classes	3.8-2
3.9	Hydrologic soil-cover complexes	3.9-1
	Purpose	3.9-1
	Table 3.9-1	3.9-1
	Forest Service procedure	3.9-3
	Determination of curve numbers for mixed areas	3.9-3
3.10	Estimation of direct runoff from rainfall	3.10-1
	Purpose	3.10-1
	General	3.10-1
	Runoff equation	3.10-1
	Significance of S	3.10-2
	Significance of I_a	3.10-3
	System of curve numbering	3.10-3
	Antecedent conditions	3.10-3
	Process of estimation	3.10-4
	Special watershed conditions	3.10-8
3.11	Estimation of runoff from snowmelt	3.11-1
	Purpose	3.11-1
	Significance of snowmelt floods	3.11-1
	Regional analysis	3.11-1
	Degree-day method, ungaged watersheds	3.11-1
	Degree-day method, gaged watersheds	3.11-6
	Adjustment of temperatures for altitude	3.11-6
	K factors	3.11-6
	Concordant flow method	3.11-7
	Other methods	3.11-7

		Page
3.12	Hydrologic effects of land use and treatment	3.12-1
	Volume effects	3.12-1
	Lag effects	3.12-1
	Determination of volume effects	3.12-3
	Determination of lag effects	3.12-3
	Determinations for snowmelt runoff	3.12-6
	Determination of surface storage effects	3.12-7
3.13	Area-inundated relations	3.13-1
	Purpose	3.13-1
	Stage vs. area inundated, simple cases	3.13-1
	Stage vs. area inundated, complex cases	3.13-1
	Stage vs. area inundated. Planimetering method	3.13-6
	Flood peak or volume vs. area inundated	3.13-8
	Frequency vs. area inundated	3.13-9
3.14	Stage-discharge relations	3.14-1
	Purpose	3.14-1
	Direct measurement	3.14-1
	Manning's formula	3.14-1
	Isolated cross sections	3.14-2
	Concordant flow	3.14-2
	Slope-area estimates	3.14-3
	Water surface profiles	3.14-7
	Doubt's method	3.14-7
	Escoffier's method	3.14-13
	Leach's method	3.14-17
	Step method	3.14-18
	Summary of water surface elevations for Six Mile Creek	3.14-21
	Starting elevations	3.14-21
	Selection of method	3.14-22
	Road and bridge crossings	3.14-24
	Discussion	3.14-35
	Overtopping of embankment	3.14-36
3.15	Time of concentration and lag	3.15- 1
	Purpose	3.15-1
	Physical significance	3.15-1
	Variability of T_c	3.15-1
	T_c from hydrographs	3.15-2
	T_c from flood observations	3.15-3
	T_c from stream hydraulics	3.15-3
	T_c from watershed approximate slope	3.15-5
	T_c as a function of area	3.15-5

	Page
3.16 Hydrographs	3.16-1
Purpose	3.16-1
Types of hydrographs	3.16-1
Elements of a hydrograph	3.16-1
General relations between variables	3.16-3
Dimensionless hydrograph	3.16-4
Comparison of hydrographs	3.16-4
Triangular hydrographs	3.16-4
Composite hydrographs	3.16-8
Effect of watershed drainage pattern	3.16-8
Effect of uneven runoff distribution	3.16-9
Effect of storm duration	3.16-9
Effect of a variable storm	3.16-10
Summary	3.16-10
Snowmelt hydrographs	3.16-11
Snowmelt and rainfall hydrographs	3.16-11
Design hydrographs	3.16-11
3.17 Flood routing	3.17- 1
Purpose	3.17- 1
Types	3.17-1
Basic equation for storage type of routing	3.17-1
Storage-indication method	3.17-2
Summary	3.17-2
Storm reach example	3.17-2
Comments	3.17-6
Routing upstream	3.17-8
Local inflow	3.17-9
Watershed example	3.17-9
Reservoir example	3.17-9
Shortcuts	3.17-14
Goodrich-Wisler method	3.17-16
Wilson method	3.17-19
Approximate routing methods	3.17-20
Peak equations	3.17-20
General equations	3.17-22
Watershed examples	3.17-22
3.18 Frequency methods	3.18-1
Purpose	3.18-1
Types of frequency series	3.18-1
Time units	3.18-1
Probability paper	3.18-1
Frequency lines. Annual series.	3.18-2
Plotting method	3.18-2
Skewed frequency lines	3.18-7
Length of record	3.18-9

		Page
3.18	Frequency methods (cont'd)	
	Frequency lines. Annual series	3.18-11
	Computing method	3.18-11
	Use of logarithms	3.18-13
	Frequency lines. Partial duration series	3.18-14
	Synthetic series	3.18-17
3.19	Transmission losses	3.19-1
	Purpose	3.19-1
	Effect on project evaluation	3.19-1
	Types of procedures	3.19-2
	Estimation of transmission losses	3.19-3
	Discussion	3.19-4
3.20	Watershed yield	3.20-1
	Purpose	3.20-1
	Summary of problems	3.20-1
	Methods of estimating yields	3.20-2
	Regional analysis	3.20-2
	Water accounting	3.20-3
	Direct runoff method	3.20-5
	Climatic and geographic factors	3.20-6
	Discussion	3.20-6
3.21	Design hydrographs	3.21-1
	Purpose	3.21-1
	Summary of method	3.21-1
	Storm criteria	3.21-2
	Example. Ordinary case	3.21-3
	Complex watersheds	3.21-7
	Large watersheds	3.21-7
PART 4.	APPLICATIONS IN NATURAL WATERSHEDS	4.0-1
4.1	Water surface profiles; variation of Leach's method	4.1-1
4.2	Concordant flow	4.2-1
PART 5.	DEFINITIONS AND CONVERSIONS	5.1-1
5.1	Definitions	5.2-1
5.2	Conversions FACTORS	5.2-1

PART 1. INTRODUCTION

1.1 General

Purpose. The Hydrology Guide is intended for the use of Soil Conservation Service technicians and engineers. It presents material needed for fulfilling national Service responsibilities in the field of soil and water conservation. Other technicians and engineers may find portions of the Guide useful in their own work, but they should recognize that the Guide attempts first to fill Service needs and requirements.

The methods and examples of the Guide are for use in developing the hydrology of watersheds; in the solution of special hydrologic problems that arise in the planning of watershed protection and flood prevention projects; in the preparation of working tools for use in the planning and design of structures for water use and disposal; and in training personnel newly assigned to activities involving hydrology.

Scope. The Guide contains some new techniques that are not generally available. Most of these were developed by Service personnel specifically for Service hydrologic needs. Standard techniques that are included illustrate applications in watershed project planning, evaluation, and design.

A minimum of hydrologic theory is given. This usually is done in showing the development of a method that is not readily available elsewhere. References to standard hydrologic literature are given where possible. These, and hydrologic textbooks, should be consulted for general hydrologic theory. The Engineering and Watershed Planning Unit hydrologist can recommend textbooks suitable for use in the area his Unit serves.

Each major type of hydrologic problem is discussed and, where possible, more than one method for the solution is given. The relative merits of alternate methods are briefly considered. Only the hydrologic phases of Watershed Protection and Flood Prevention Program planning problems are discussed. Descriptive material, as such, is avoided and an attempt is made to keep the Guide concise, factual, clear, and useful.

1.2 Responsibility of the hydrologist in the selection of methods

General. Memoranda from the Director of the Engineering Division define the official technical duties and responsibilities of the hydrologist. This section discusses the related responsibility of determining the suitability of hydrologic methods for use in a given problem.

Range of problems. Service engineering projects that require some application of hydrology may range in construction cost from about one hundred dollars to several million dollars. A hydrologic method suitable at one end of this range is usually unsuitable at the other end. Two projects of about the same potential cost may require widely different hydrologic methods because of differences in available hydrologic data, topography, location of benefits, or climatic and other physical factors. In some cases the choice of hydrologic method may be limited by available survey time or funds. Where such limitations apply, the results obtained should be regarded as tentative pending an investigation of adequate scope. The important point is that the hydrologic approach should be adequate to establish sound conclusions in terms of the conditions, objectives, and functions of the project.

Selection of method. The advice of the E&WP hydrologist should be sought in cases where there is doubt as to the suitability of a method.

Since all possible field conditions cannot be considered here, only a general rule for the selection of hydrologic methods will be given. The hydrologist will learn more about details by personal observation and comparison of recent actual work plans for projects in his vicinity.

The hydrologist should be familiar with the practical working details of several methods before he chooses one for a given problem. Many methods that appear difficult to use are often only difficult to explain, and they may be both simpler and faster in actual use than methods that appear simple and fast at first sight, but which require lengthy and difficult computations that are not readily apparent.

Potential cost of a project and the hazard involved in the event of failure determine the scope of investigation and the selection of hydrologic methods. Exceptions arise in practice,

especially where the difficulty of project evaluation or design varies from the average. The major steps in the selection of hydrologic methods for a typical project evaluation are as follows:

1. Estimate the locations and relative importance of potential project benefits.

2. Determine the types and amounts of available and required hydrologic data.

3. Estimate the required hydrologic work as well as possible, and fit the work into the time and manpower schedule for the entire investigation. Some E&WP Units have schedule forms that may be used. The schedule should show the sequence of work so that "bottlenecks" may be detected.

Under some conditions, the hydrology in step 3 requires emphasis. For example, where the project consists of channel improvements, sufficient field data must be obtained for a preliminary design, the studies of channel hydraulics must be of high quality, and detailed flood routings must be made. These steps may delay completion of a work plan, but since the channel changes will usually result in larger flood flows at downstream sections, it is vital to the successful operation of the project that these steps be taken. Similar situations may arise with other types of projects.

Where hydrology does not require emphasis, the methods in step 3 may be altered to eliminate or reduce delaying items.

A high degree of accuracy may not be needed as a basis for benefit determinations on projects with high benefit-cost ratios, but limited or inaccurate hydrologic determinations will almost always adversely affect the project development and operation.

The method to be used for any project can be selected by a similar procedure. In making the selection, it is important for the hydrologist to be familiar with the work and needs of the economist, design engineer, geologist, and others who will use the results of the hydrologic studies.

1.3 Other technical guides

The Service hydrologist should have or be familiar with other national guides and handbooks used in the Service; in particular, with the following:

1. Interim Watershed Protection Handbook.

2. Economics Guide for Watershed Protection and Flood Prevention.

3. Hydraulics Handbook, (NEH-~~4~~5) including Supplement A (Water Surface Profiles), ES-97 (Flow in Circular Conduits), and others as available.

4. Drainage Handbook (NEH-16).

5. Irrigation Handbook (NEH-15).

6. Sedimentation Handbook (NEH-3).

7. Technical Releases on subjects related to hydrology or which use hydrologic results.

The appropriate State SCS handbooks should also be studied.

Handbooks, manuals and other in-Service publications of State and other Federal agencies (such as the Bureau of Reclamation and the Corps of Engineers) should be examined when possible. In cooperative work with a State or Federal agency, it is important to be familiar with the hydrologic methods those agencies use. In some cases it is desirable to evaluate a project by using both Service methods and those of the cooperating agency in order to meet as nearly as possible the requirements of both agencies. However, Service methods will be used for Service purposes.

The State publications giving criteria that affect the hydrology of a project should be studied. It is Service policy that such State criteria be met or exceeded--that is, where State criteria are more strict in their requirements than Service criteria, the State criteria will govern; otherwise, Service criteria will govern.

Technical papers on hydrology should be read, especially those appearing in Transactions of the American Geophysical Union; Agricultural Engineering, which is the journal of the American Society of Agricultural Engineers; Civil Engineering, which is the magazine of the American Society of Civil Engineers, and also the society's Transactions. Papers on hydrology or on important subjects related to hydrology often appear in the Journal of Soil and Water Conservation; Journal of the American Society of Agronomy; Soil Science, which is the magazine of the Soil Science Society of America, and also that society's Proceedings; and the American Journal of Forestry.

PART 2. PROCEDURES

2.1 Watershed conditions

It is Service practice to evaluate watersheds under the three following conditions:

1. Present condition. This is the watershed condition at the time of the survey, and it is the base to which a proposed project is added.

2. With future land treatment measures. (Proposed land treatment measures superimposed on the present condition.) The types of measures considered are described in the Interim Watershed Protection Handbook.

3. With future land treatment and structural measures. (Watershed-protection and flood-prevention structural measures superimposed on condition 2.) These measures consist of one or more of the structural measures described in the Interim Watershed Protection Handbook.

The three watershed conditions are always evaluated in the order given above. This not only is Service practice, but it is the logical order for evaluating proposed watershed projects, since the measures that have the first effect on runoff should be evaluated first and those having the last effect should be evaluated last. In some cases the second or third conditions may be further subdivided in order to evaluate a particular group of measures. Here again, measures having the first effect on runoff are evaluated first.

Some watersheds may already have a complete conservation program applied. For such a watershed, the first two conditions are identical, and only one evaluation is needed.

2.2 Data conditions

In general, two types of data are used in project planning and evaluation: (1) that which can be obtained by field measurements during the time of the study, such as channel and valley cross sections and profiles, surveys of potential structure sites and location maps; and (2) that which usually must accumulate before the study, such as precipitation data, or flood and other streamflow data.

Data of the first type are always obtainable, and the required degree of planning accuracy is the principal criterion in obtaining such data. (See Section 1.2). Field survey requirements are considered in Sections 3.2, 3.6, 3.7, and 3.8.

Data of the second type may not be available, but data from nearby areas may be used with such methods as regional analysis (Section 2.8), storm transposition (Section 3.4), and runoff transposition (Section 3.5). Hydrologic instrumentation conditions are also discussed in Sections 3.4 and 3.5.

Data such as the present condition of land treatment on the watershed are obtained during the time of study. However, the land treatment conditions of previous years (the years of an evaluation series) are generally only vaguely known, and the condition at the time of study is used for the previous years except in special analyses.

The hydrologist's first concern is the availability of hydrologic data. The procedures to be used in the hydrologic study of watersheds are considered here in terms of that availability.

2.3 With streamflow and rainfall data available

Figure 2.3-1 shows the general flow of hydrologic work for a watershed project evaluation. Note on this and following charts that some items (such as Base map) are shown for the sake of completeness, and that the hydrologist may not have full responsibility for obtaining or determining such items.

All three watershed conditions of Section 2.1 can be evaluated using the process of figure 2.3-1, omitting those steps or boxes that do not apply. For example, the "structural measures data" may not be used in evaluating the present condition, and once having made the step of "Location of reaches" the step is used for remaining watershed conditions.

The streamflow data are also used for checking (1) the computed rainfall-runoff relation, and (2) the entire flood-routing process. The first check is discussed in Section 3.10 in connection with table 3.10-2. The second check is a comparison of routed hydrographs versus actual hydrographs for selected floods. A comparison of hydrograph peak rates alone is not a sufficient check, and a comparison of hydrograph shapes and time positions should be made. When the routed hydrographs compare poorly with the actual hydrographs, a review of hydrologic and hydraulic factors is made. For example, if a routed peak occurs in 50 percent of the time at which the actual peak occurred, the discrepancy may be due to improper estimates of runoff distribution in time and location, or it may be due to improper hydraulic estimates, or to other similar factors. Since the reason for the discrepancy may be difficult to find, it is important to keep hydrologic and hydraulic estimates on as high a level as practical during the project evaluation.

426963 O - 57 - 2

FLOW CHART— General process, hydrology of watershed project evaluation

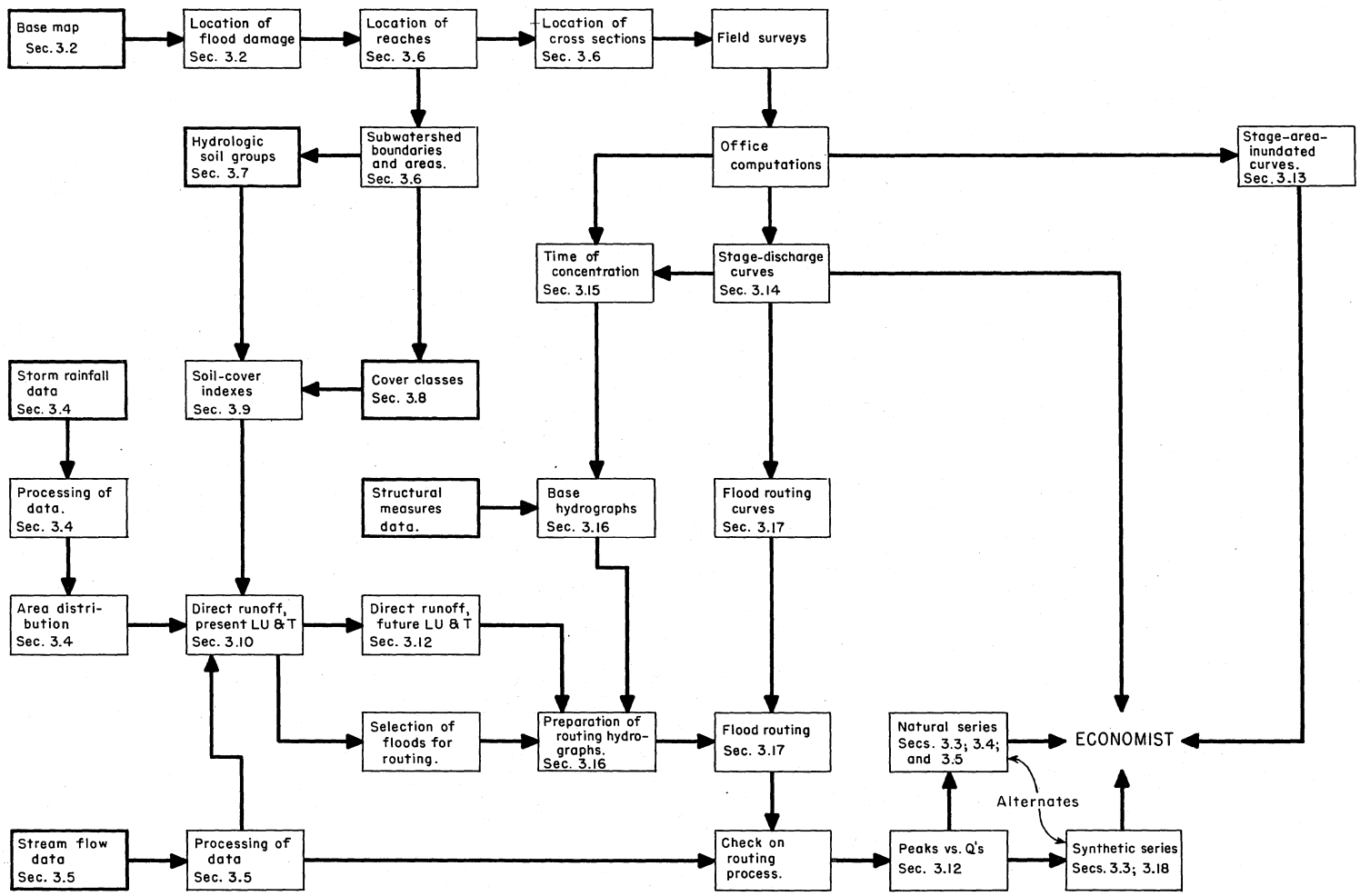


Figure 2.3-1

2.4 With streamflow data only

Figure 2.4-1 shows the general flow of hydrologic work for the special case when adequate streamflow data are available. Note that precipitation data are not used.

FLOW CHART — Hydrology of watershed project evaluation.

With adequate stream flow data available

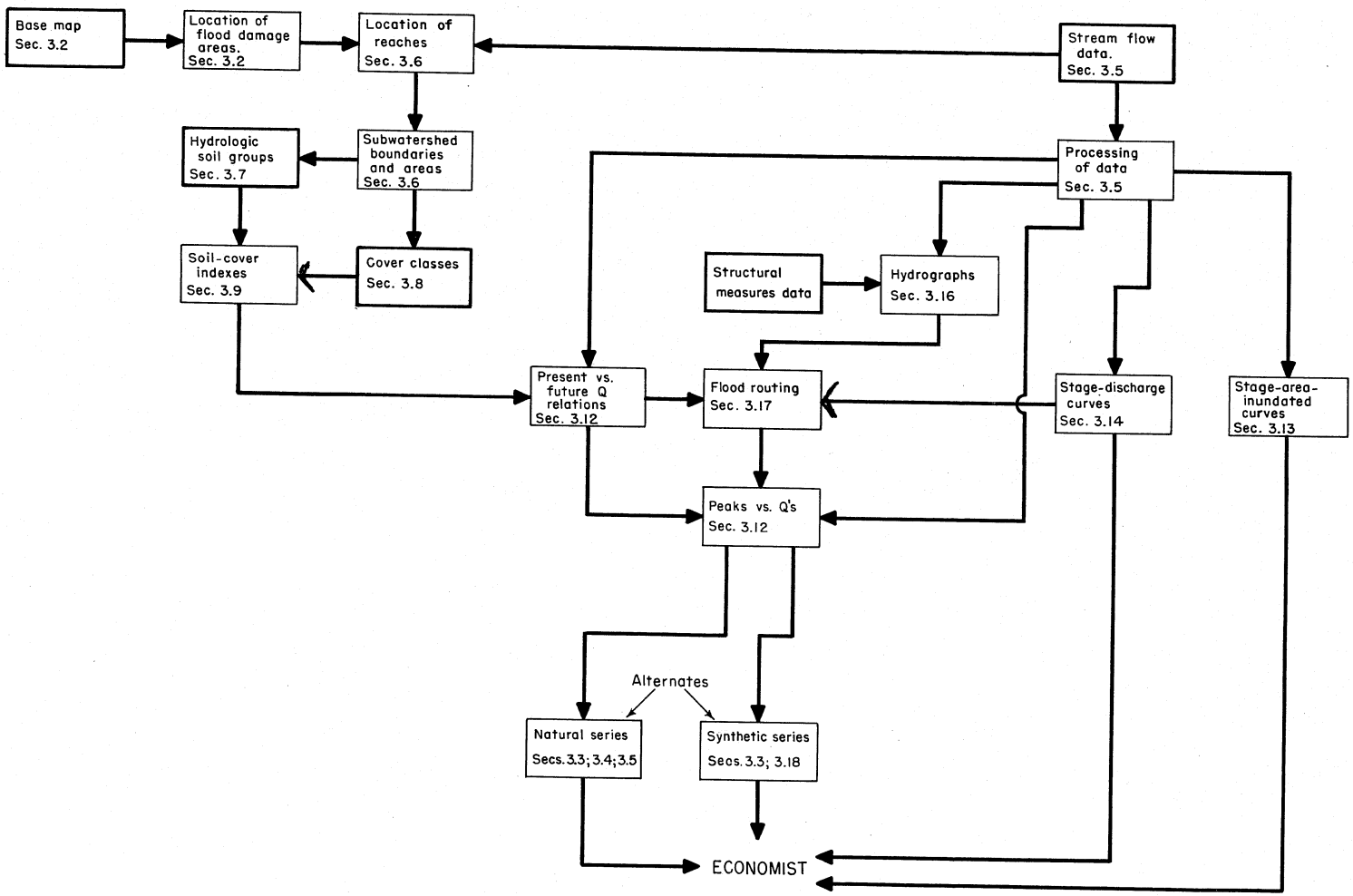


Figure 2.4-1

2.5 With rainfall data only

When no streamflow data are available, the general process is as given on figure 2.3-1, but omitting the steps involving actual streamflow data. No direct checks on the runoff estimates can be made. High-water marks can be used as an over-all check of the combined runoff and routing work, or to adjust one specific item such as a runoff estimate by assuming other work to be sufficiently accurate. An example of the use of high-water marks to adjust Manning's n values is given in Section 2.7, Concordant flow. The "Selection of floods for routing" is made directly after the "Computation of direct runoff" and is based on a selection of representative storms in the series. After "Flood routing" the check is omitted, the next box or step being "Plot peaks versus Q's".

2.6 With no rainfall or runoff data on the watershed

When neither precipitation nor streamflow data are available for the watershed, then either (1) regional analysis is used (Section 2.8), or (2) data transposition is used (Sections 3.4 and 3.5). When rainfall data are transposed, the work follows the process given in Section 2.5. When runoff data are transposed, the work follows the process given in Section 2.4.

2.7 Concordant flow

This method uses a recent, well-defined, large flood as the basis of proportions for evaluating present and future hydrologic conditions on a watershed. The flow chart, figure 2.7-1, shows the steps in the method. The text below, which gives additional information, is adapted from "Concordant Flow Routing Used in Evaluating Hydrologic Effects of Measures on Flood Flows", submitted by R. G. Andrews.

The method is generally applicable under the following conditions:

1. No channel improvements, levees, or floodwater diversions are to be evaluated.

2. There is a relatively small amount of concentration of floodwater damages to urban, industrial, or high-value agricultural developments in the watershed.

3. Existing or proposed structural measures to be evaluated are about uniformly distributed over the watershed.

4. The derived proportions and estimates are used only for evaluations of projects in the watershed for which the derivations were made.

5. The flood-producing storms are generally about uniformly distributed over the watershed.

6. Times of concentration in the watershed are proportional (or nearly so) to watershed size. (See figure 3.15-4).

Step procedure

1. Refer to Sections 3.1, 3.2, and 3.6 for field and office work. Locations and elevations of high-water marks (HWM) for a recent well-defined, large flood must be obtained in the field survey.

2. Using the field estimates of "Manning's n" values, compute the discharge at each HWM. (Section 3.14, Manning's formula). See table 4.2-3, Section 4.2.

3. Make an isohyetal map for the rainfall of the recent, well-defined, large flood. (NEH-4, 4.5.3, Isohyetal method)

4. Determine hydrologic soil-cover complexes for drainage areas between cross sections. (Sections 3.7, 3.8, and 3.9)

5. Prepare the table for weighting the rainfall (table 4.2-1, Section 4.2, "Concordant flow") and make the computations using the isohyetal map of Step 3.

6. Estimate weighted runoff (last column of table 4.2-1, Section 4.2) by the method of Section 3.10, "Estimation of direct runoff from rainfall".

7. Prepare the table for adjusting preliminary discharge estimates (table 4.2-2, Section 4.2).

8. Plot adjusted discharges (last column of table 4.2-2, Section 4.2) versus drainage area at respective cross sections on log paper (figure 4.2-1, Section 4.2).

9. Fit a straight line to the plotted points (figure 4.2-1, Section 4.2). Compute the slope of the line for later use. The scatter of points may require the fitting of two or more straight lines. In such cases each line is treated separately.

10. For each plotted point that is well away from the line, assume the discharge at the line to be correct, work backward through table 4.2-2 of Section 4.2, and arrive at an "adjusted Manning's n" for Step 2.

11. If the adjusted "n" values are not unreasonably high or low, use them to complete stage-discharge curves at each cross section (table 4.2-3, Section 4.2).

The procedure of Steps 2 through 11 may be repeated for other HWM or for bankfull discharges, and "n" value adjustments made throughout the stage-discharge curve.

12. Compute area inundated at each cross section (table 4.2-3, Section 4.2). A check on the areas inundated at the HWM level is given in Section 3.13, "Area-inundated relations".

13. Plot stage-discharge and stage-area inundated curves (figure 4.2-2, Section 4.2) on cross section paper. (Sections 3.13 and 3.14).

14. Find the minimum amount of rainfall it takes to begin flooding by working backward from the bankfull discharge for the lowermost cross section, to the amount of runoff (table 4.2-1, Section 4.2) and thence to the amount of rainfall (table 4.2-1, Section 4.2).

15. Reduce the number of storms in the rainfall series (Section 3.4) by deleting those with less rainfall than the amount determined in Step 14.

16. Determine the direct runoff for each storm in the series for present conditions, as shown in table 4.2-4, Section 4.2 (Section 3.10, "Estimation of direct runoff from rainfall").

17. Prepare area-inundated versus direct runoff graphs by computing table 4.2-5, Section 4.2, for selected amounts of direct runoff ranging from the highest to the lowest of Step 16. Plot the total area-inundated versus direct runoff amounts as shown on figure 4.2-4, Section 4.2.

18. Plot the area-inundated by depth increments versus runoff as shown on figure 4.2-4, Section 4.2. Use the stage-area inundated curves of Step 13 to compute increments of area.

19. With the present runoff amounts of table 4.2-4, Section 4.2, and with figure 4.2-4, Section 4.2, obtain the area inundated for each storm in the series of Step 15. Tabulate for use by the economist.

20. Determine the runoff curve number for the watershed conditions assuming future land use and treatment measures in place. (Sections 3.8, 3.9, and 3.10. Also see Step 4.)

21. Repeat Step 16 assuming future land use and treatment measures in place. Tabulate runoffs as shown on table 4.2-4, Section 4.2.

22. Repeat Step 19 with the data of Step 21.

23. Prepare table 4.2-6, Section 4.2, for each runoff amount of Step 17.

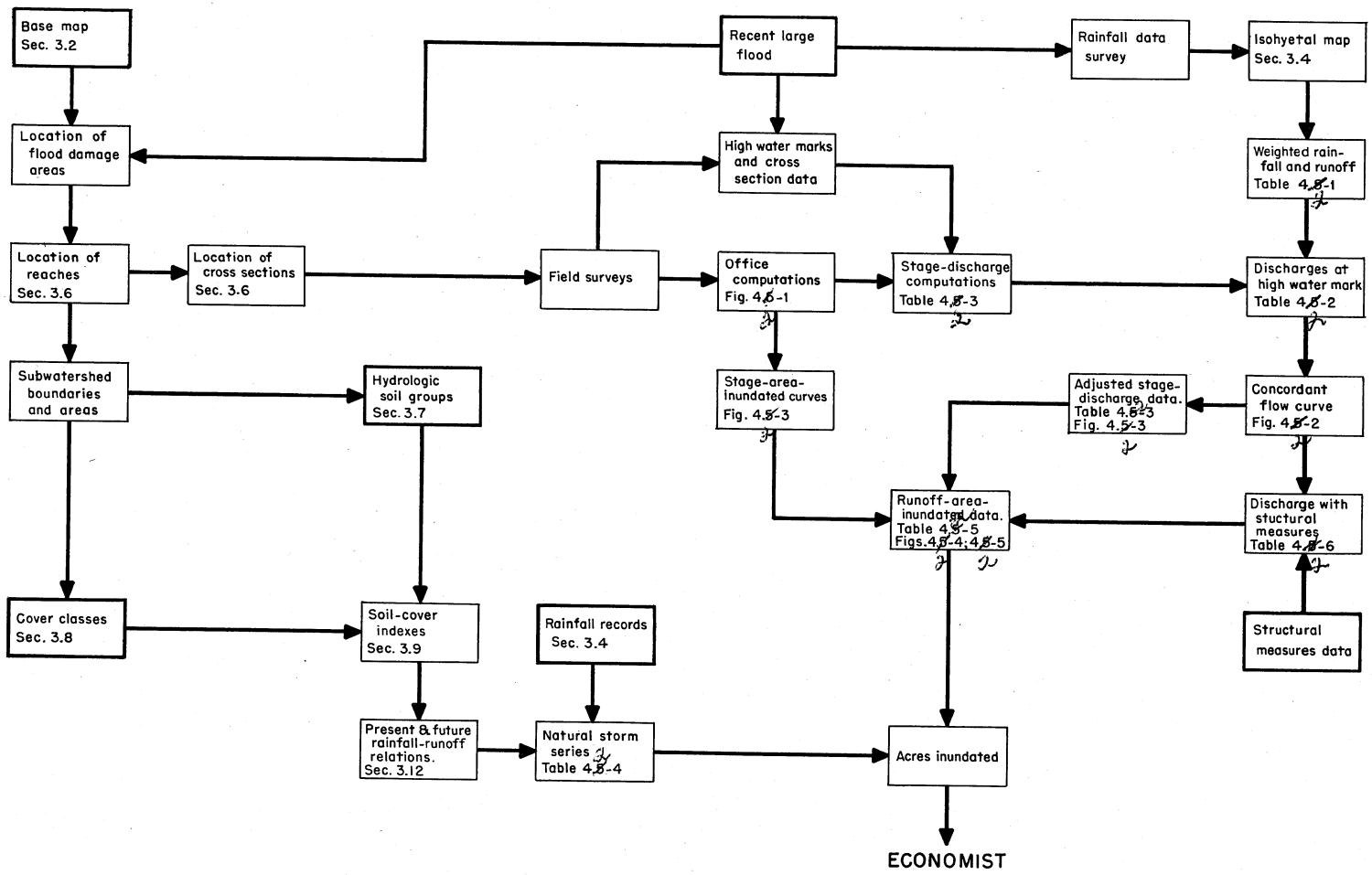
24. Repeat Step 18, using table 4.2-6 data for each runoff amount.

25. Repeat Step 19, using the future land use and treatment measures assumed in place.

See Section 4.2 for an application of this method to a watershed having both land treatment and structural measures under consideration.

FLOW CHART— Hydrology of watershed project evaluation

Concordant flow method



REFERENCE

Andrews, R.G., Unpublished material

Figure 2.7-1

2.8 Regional analysis

This method uses runoff frequency lines for nearby gaged watersheds as a basis for estimating a runoff frequency line for a given ungaged watershed or gaged watershed with a short period of record. The method can be used with flood peaks, flood volumes, annual runoff, monthly runoff, and similar data.

Step procedure

1. Select gaged watersheds near the ungaged watershed. The selected watersheds should be as like the ungaged watershed as possible in all but one of many factors such as climate, soils, cover, topography, and drainage area. Usually drainage area is the variable factor used with flood peaks. Some other factor is ordinarily used with flood or annual volumes. (See discussion following these steps.)

2. Prepare annual frequency lines for the selected watersheds (Section 3.18, "Frequency lines, Plotting method"). Draw the frequency lines parallel if at all possible.

3. Check the frequency lines for length of record, and either discard the too-short records or extend them (see Section 3.18, "Length of Record").

4. Plot the 2-year and 100-year frequencies of the adequate records versus drainage area or time of concentration, on log paper, if flood peaks are being studied. Plot the frequencies on the log scale of semi-log paper if other variables are used. (See discussion following these steps.)

5. Draw lines through points of equal frequency. These lines should be straight and parallel unless the plotting shows a very strong curvature for the points of a given frequency.

6. Enter the graph of Step 5 with the drainage area (or other factor used as the principal variable) of a stream reach in the problem watershed, and find the 2-year and 100-year frequencies.

7. Plot the 2-year and 100-year frequencies on log-normal paper and draw the frequency line.

Once the frequency lines have been drawn they may be used in any one of several ways. Figure 2.8-1 is a flow chart showing the use of regional analysis in one method of watershed project evaluation. When the frequencies of either runoff peaks or volumes have been determined, the effect of land treatment measures is easily found.

(Section 3.12. Also see equation 9a in Section 3.17). The effects of structural measures must be determined using one of the approximate routing methods given in Section 3.17, such as that shown on figure 3.17-5(b). In addition, the flood damages (or project benefits) must be determined on a frequency basis as discussed in Section 3.13 "Frequency vs. area inundated" and Section 3.18 "Synthetic series".

Further discussions of the use of regional analysis are given in Section 3.20.

The principal factor chosen in Step 1 is the one for which the 2-year and 100-year frequencies will have the greatest variation among the selected watersheds. Although drainage area is customarily used with flood peaks, time of concentration may also be used alone or in combination with area (as A/T_c, for example). Other factors may also be used alone or in combinations, but unless they can be shown to have a strong influence on the results, they should be neglected. Some factors require special systems of plotting, which limits their use in combinations. Watershed aspect or position facing a particular direction, for example, may be a strong influence on the runoff amounts of mountainous watersheds. The azimuth or direction of the main stem of a watershed may be used as an index of aspect. The 2-year and 100-year frequencies are plotted on the log scale (vertical) of semi-log paper, while the index is plotted on the arithmetic scale (horizontal) and a pair of about equidistant curves is obtained if aspect is a strong influence. Nearly horizontal plotting indicates no influence. However, few other important watershed or climatic factors (mountain barriers may sometimes be an exception) will require such a directional scale, so that aspect must generally be evaluated alone.

Regional analysis is essentially a regression method (see statistical texts for further information). It has not only the advantages of that method, but also the disadvantages. Interpretation of the results of a regional analysis must be carefully made. For example, an index for one watershed or climatic factor may turn out to be an index for other factors more important than the chosen factor. If the regional analysis is satisfactory, it will then appear that the frequency values are related to the chosen factor, but they may actually be unrelated to that factor. Direct factors, like drainage area, time of concentration, temperature, and precipitation may be safely used. Indirect factors like aspect, elevation, storm barriers, and latitude tend to be indexes or require indexes in their use, and should be used with caution. Results from their use should be localized as much as possible, and no interpretation of physical significance should be made.

FLOW CHART— Hydrology of watershed project evaluation

Method of "Regional Analysis"

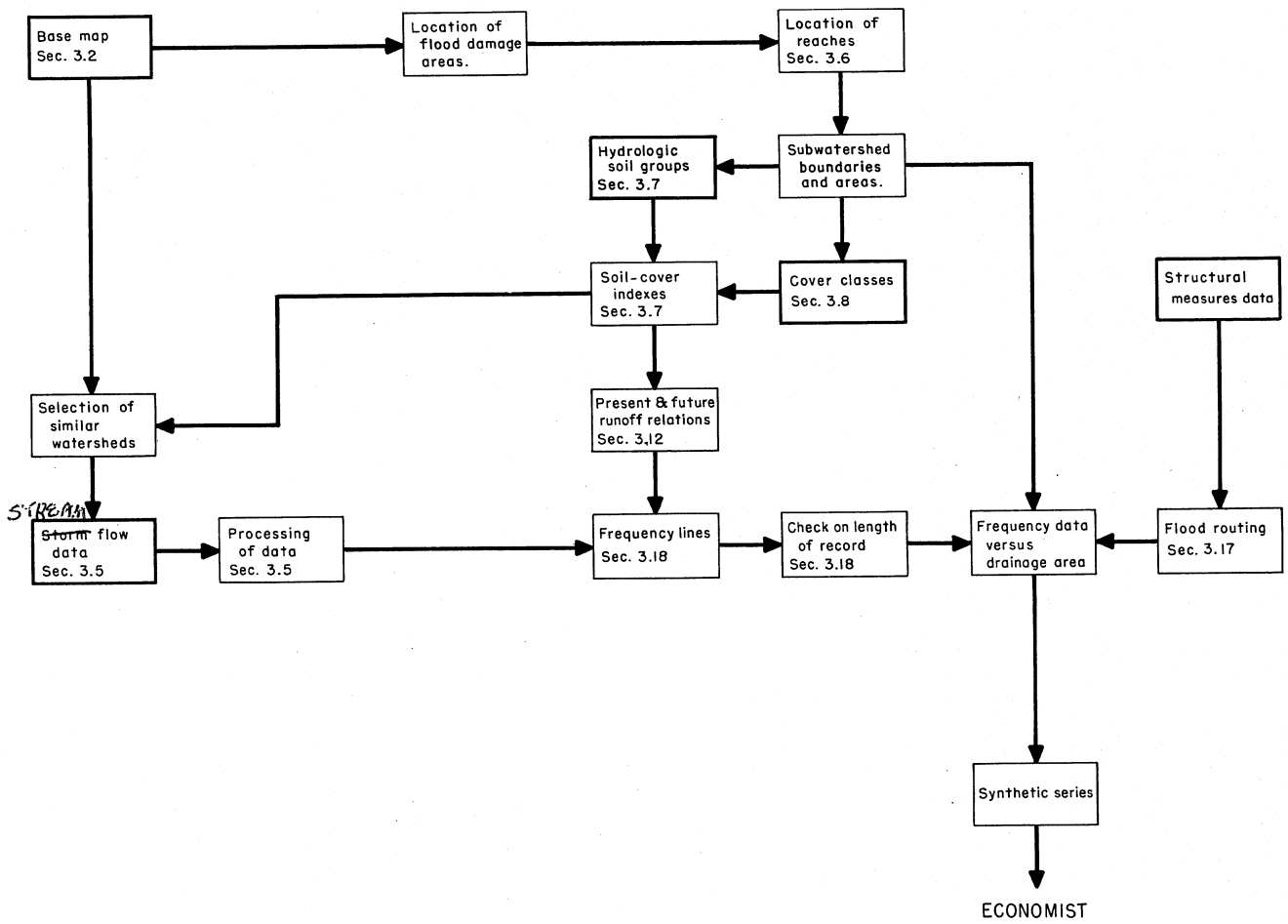


Figure 2.8-1

2.9 Snowmelt

Degree-day method. Details of this method are given in Section 3.11. The flow chart, figure 2.9-1, shows an adaptation for watershed project evaluation.

Rosa's snowmelt method. Figure 2.9-2 gives a flow chart based on a method described by J. M. Rosa in "Forest Snowmelt and Spring Floods", Journal of Forestry, April 1956. The flow chart, figure 2.9-3, shows an adaptation for watershed project evaluation. Use of the Rosa method is limited to the states for which the required relations of snow storage and melt in forest and open land (see Rosa's article) have been developed.

FLOW CHART— Hydrology of watershed project evaluation

SNOWMELT— Degree-day method adapted to watershed project evaluation.

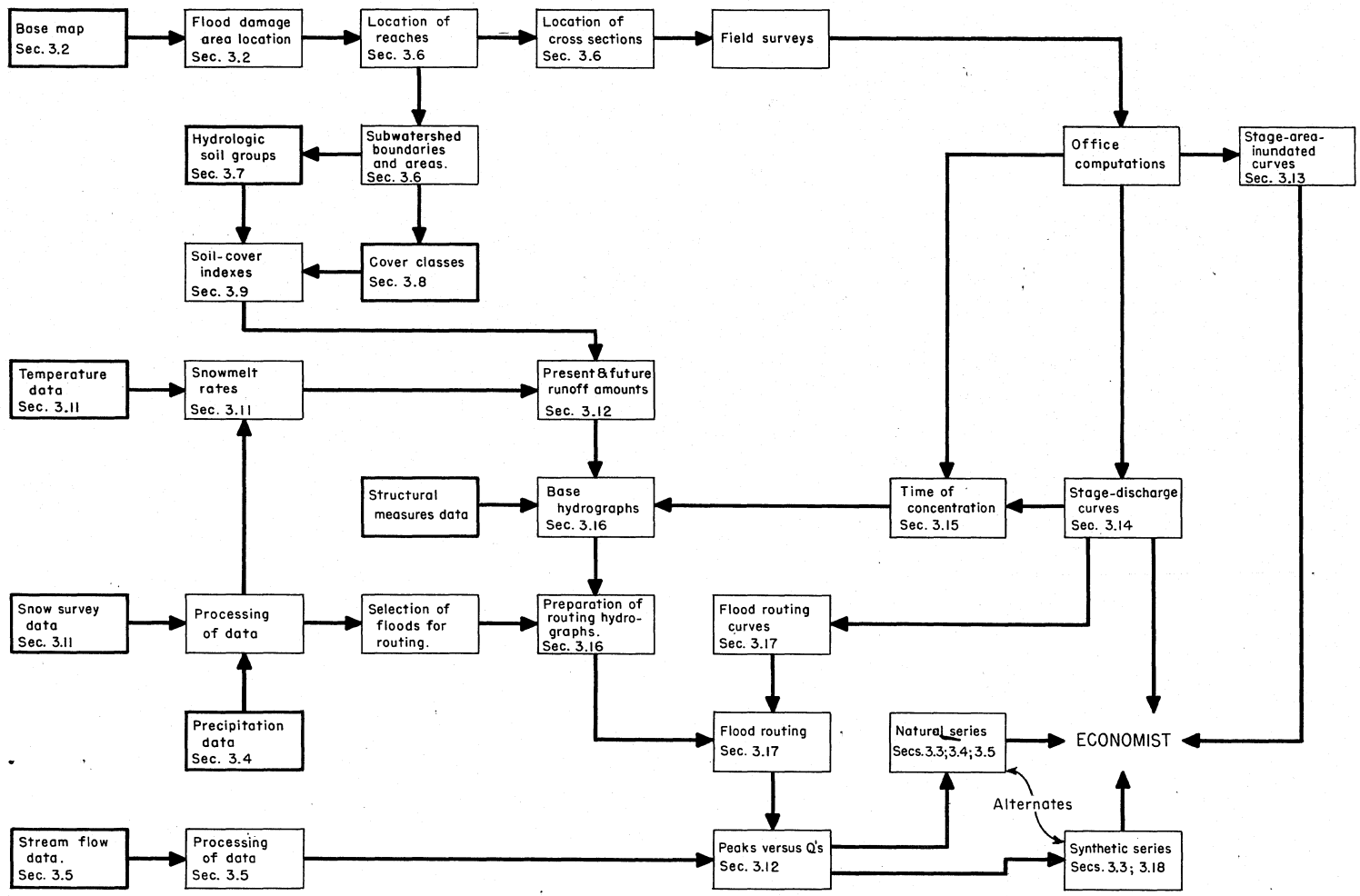
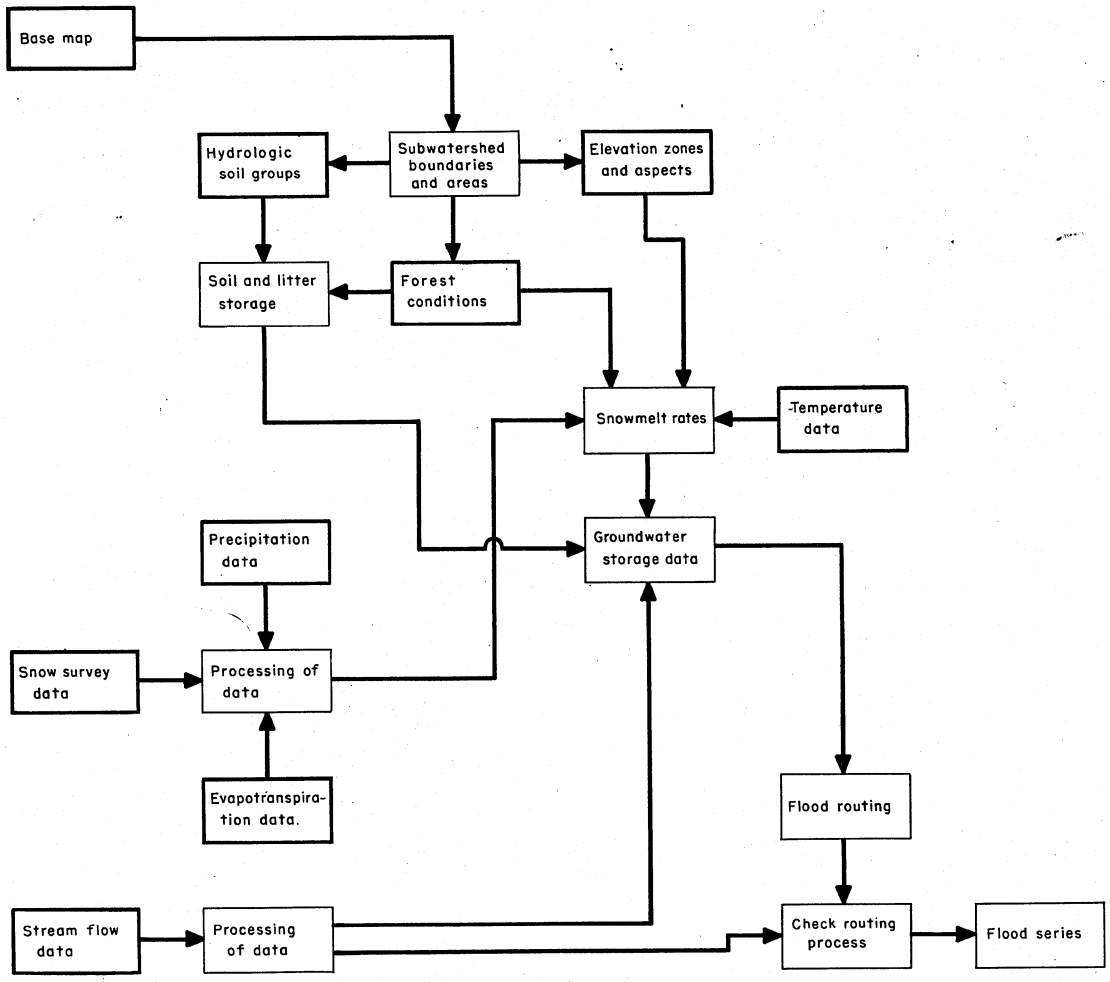


Figure 2.9-1

FLOW CHART— Hydrology of watershed project evaluation

SNOWMELT— Rosa's method



REFERENCE

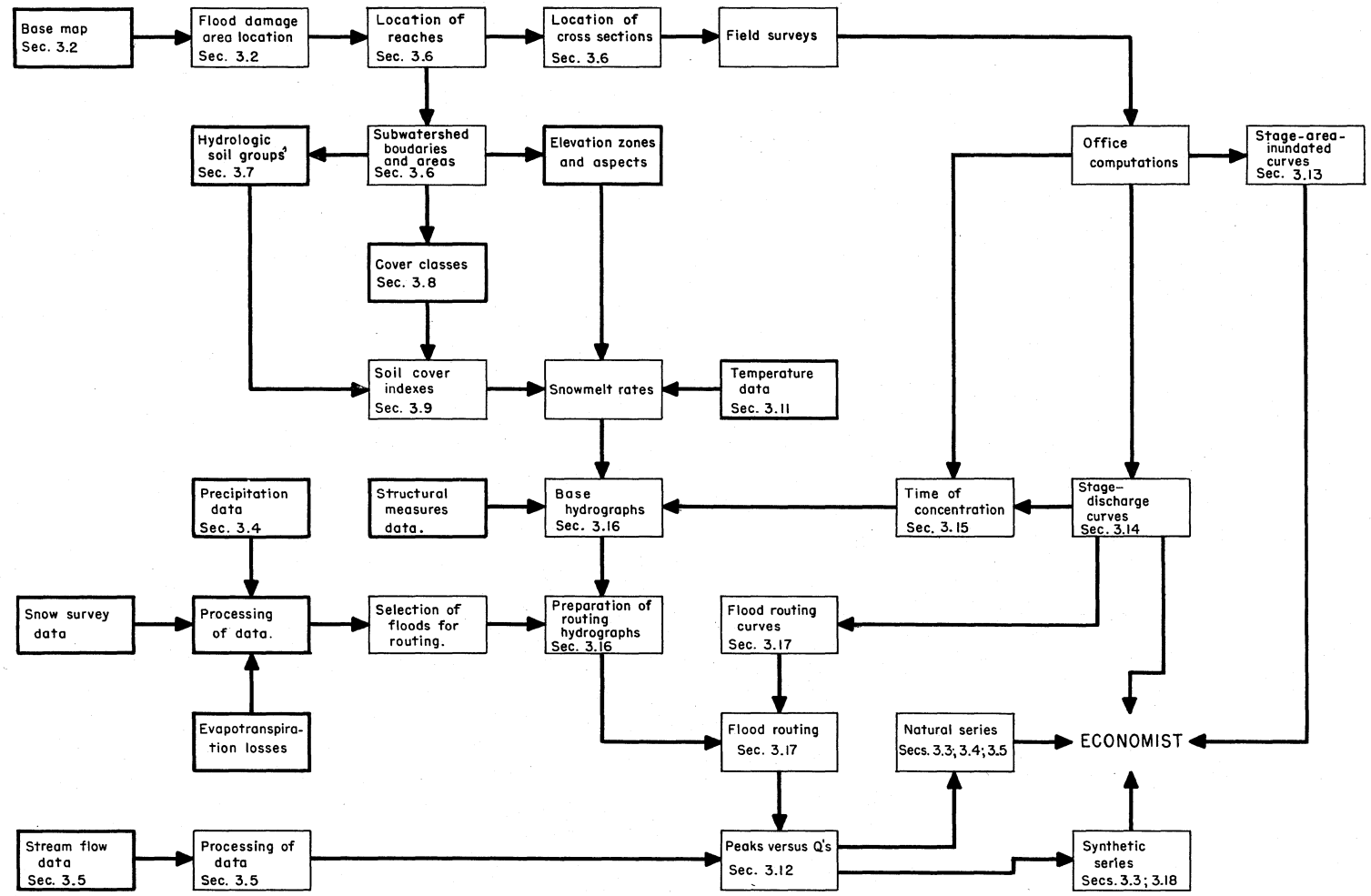
Rosa, J.M., Forest snowmelt and spring floods: Journal of Forestry, vol. 54, no. 4, April 1956.

Figure 2.9-2

426963 O - 57 - 3

FLOW CHART— Hydrology of watershed project evaluation

SNOWMELT— Rosa's method adapted for watershed project evauation.



REFERENCE

Rosa, J.M., Forest snowmelt and spring floods: Journal of Forestry, vol. 54, no. 4, April 1956.

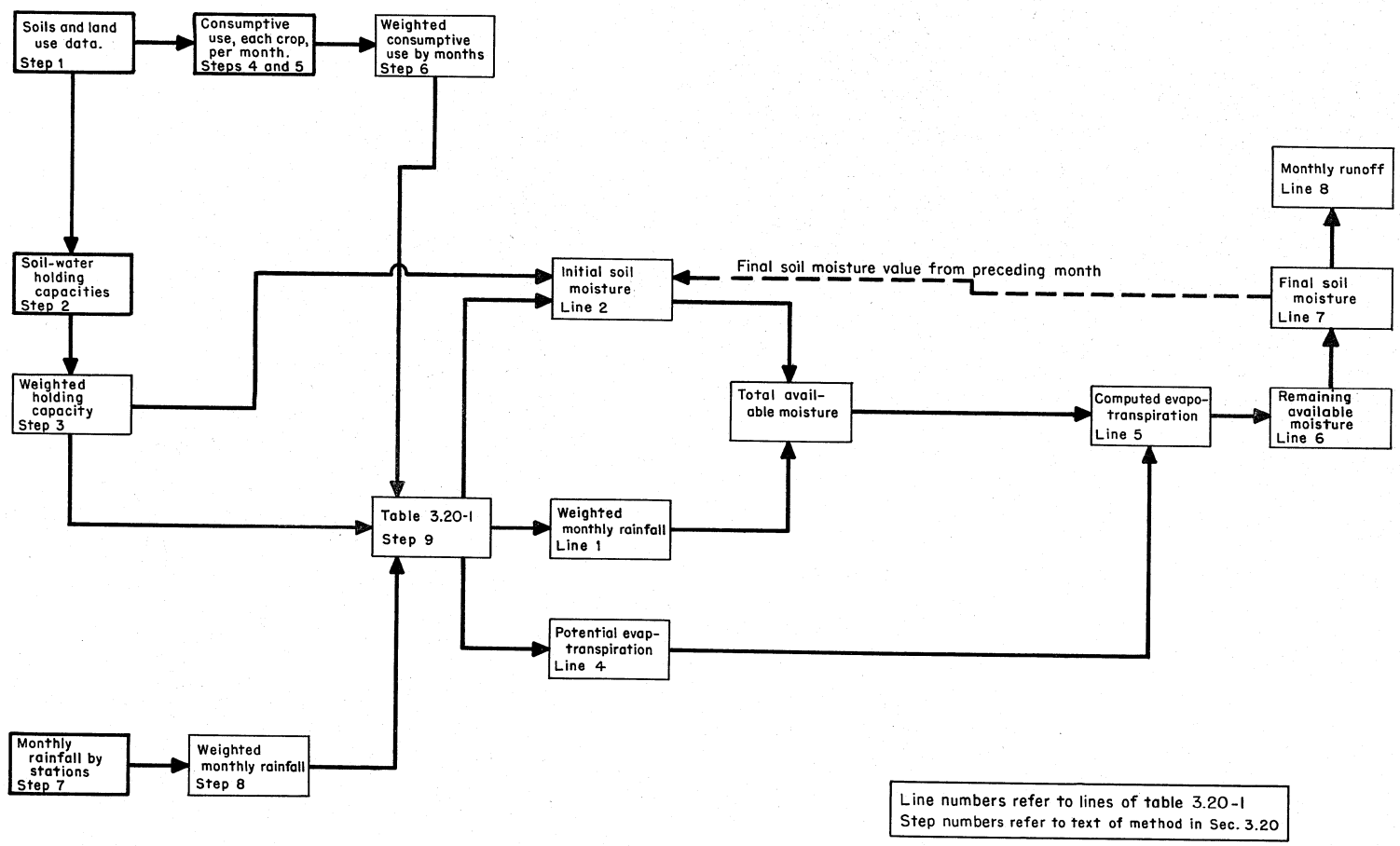
Figure 2.9-3

2.10 Watershed yield

Section 3.20 gives a brief discussion of the yield problem, and presents several methods for estimating yield. The flow chart, figure 2.10-1, shows the step-by-step process of the water accounting method, which is further described in Section 3.20.

FLOW CHART— Hydrology of watershed project evaluation

WATERSHED YIELD— by the water accounting method



REFERENCE

Sharp, A.L., Unpublished material

Figure 2.10-1

2.11 Design hydrographs

The flow chart, figure 2.11-1, shows the general process of obtaining design hydrographs to meet the needs of Engineering Memorandum No. 3. Details of the process are given in Section 3.21.

FLOW CHART — Hydrology

DESIGN HYDROGRAPHS — General method

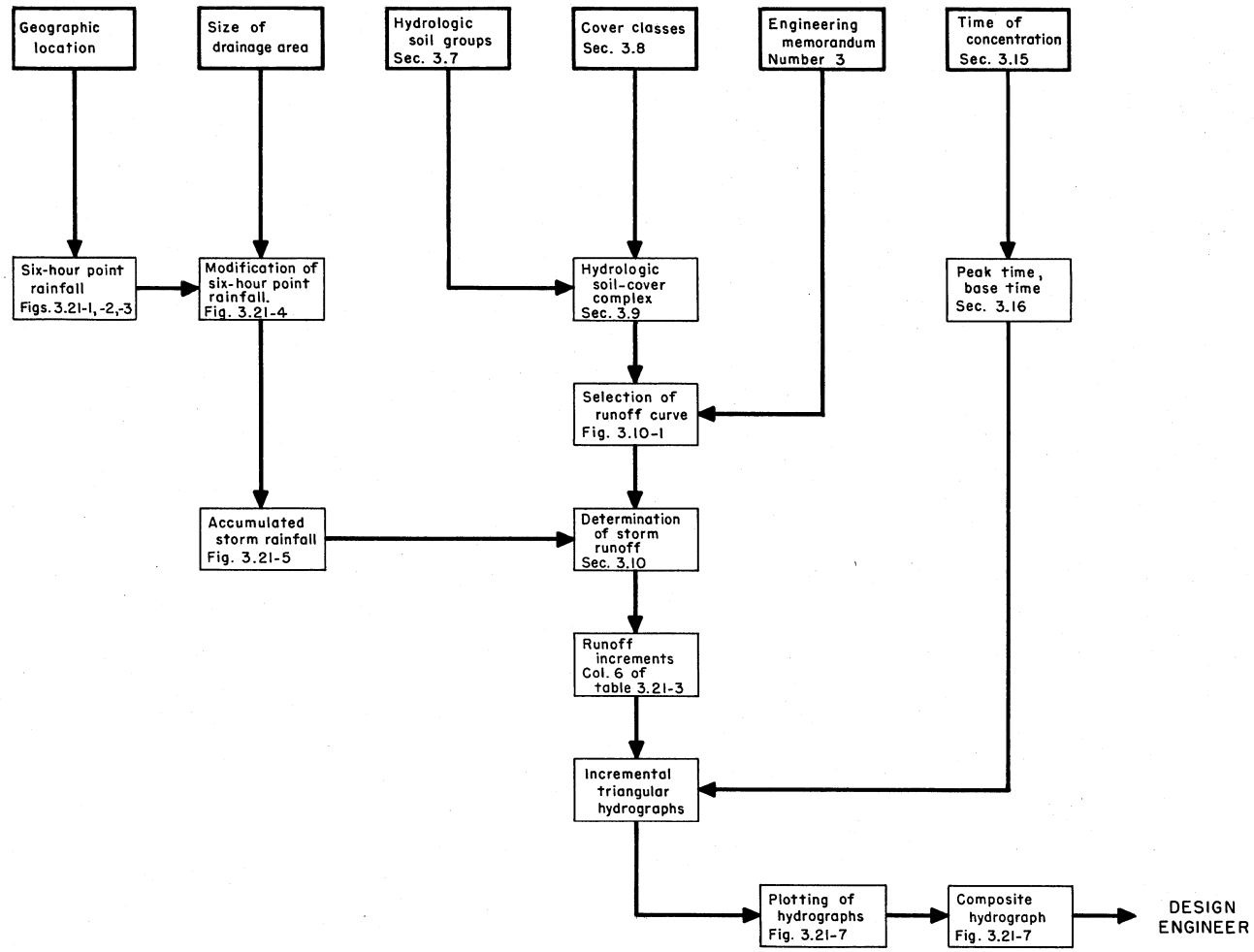


Figure 2.11-1

PART 3. WORKING DETAILS

3.1 Reconnaissance

Probably the most important single step in the hydrologic evaluation of a watershed is a field trip through the watershed. This should be taken early in the study. Field observations are valuable in engineering phases of the study, such as in locating reaches, cross sections, and structural sites, and in any job that uses physical evidence remaining after an event, such as in the selection of highwater marks after a flood. However, field observations for hydrologic purposes should be supported by instrumental measurements or by analyses involving measured data.

The conduct of a reconnaissance follows the instructions given in the Interim Watershed Protection Handbook. In general, during the reconnaissance, possible remedial measures are tentatively located in relation to watershed problem areas, preliminary estimates of the work load are made, and a visual impression of the watershed is obtained that is valuable later when working with the watershed map.

3.2 Base map

It is desirable to have a base map of the watershed as early as possible in the study--preferably before the reconnaissance, since it can then be used in making preliminary field estimates of damages and remedial measures.

When adequate published maps are not at hand, the Cartographic Unit prepares the base maps according to the work plan party's specifications. A line drawing of a controlled mosaic of aerial photographs is sometimes desirable, but in many cases a line drawing using an uncontrolled mosaic is adequate. A composite aerial photo map can be quickly prepared and may be sufficient for survey needs, especially on the smaller watersheds.

A base map should show the stream system wherever significant floodwater damage is likely to exist and wherever good structural sites may be found. The road system should be shown for convenience both in travel and in the location of cross sections. Mines, quarries, oil fields, gas and oil lines, power transmission lines, and similar items should be shown on the map when they are likely to affect either the evaluation of damages or the location of structural measures. Towns and cities should be shown with the minimum detail needed in the damage evaluation.

3.3 Evaluation series

Purpose. An evaluation series of floods or storms is used in obtaining flood damages for a representative or "normal" climatic period. In evaluating the future effects of a proposed project, the same series is used again, assuming it to be equally representative of the future climatic period.

Types. Every type of evaluation series should be tested for adequacy of length of record by the method given in Section 3.18. Too short a series may not provide a period that is "normal", whereas too long a series will add work without proportionally increasing accuracy of the estimates of average annual benefits.

The events used in evaluation may be in a

storm series, in which storm rainfall data are obtained first, with runoff being computed later; or in a

flood series, in which peak rates or volumes of flood runoff are directly obtained.

A series may be

historical by consisting of all actual storms (or floods) that caused flood damage in the watershed in a given period of years, with the date of each storm or flood being known; or

synthetic by consisting of storms or floods calculated by applying frequency methods to actual data. One storm or flood per unit of time is used, but no specific date within that unit is specified. Generally the time unit is a calendar year, though water years, seasons, or calendar months are sometimes used.

The above definitions can be combined to give four types of series:

1. Historical storm
2. Historical flood
3. Synthetic storm
4. Synthetic flood

Selection. The choice of a type of an evaluation series rests primarily on the survey party leader. The hydrologist should be familiar with the hydrologic operations for each type, since there are advantages and disadvantages in each type according to the kinds of data that are available.

For additional information on the preparation of a series see Section 3.4 on the historical and synthetic storm series; Section 3.5 on the historical and synthetic flood series; and Section 3.18 on the synthetic storm and flood series. Examples of the testing of series for adequacy of length of record are given in Section 3.18.

3.4 Precipitation data

Purpose. Precipitation data are used with established rainfall-runoff relations to estimate present or future runoff.

This section presents material not yet included in the National Engineering Handbook, Section 4, Hydrology. References will be made to that source where possible (using the abbreviation NEH-4) in order to present a more complete outline of methods of working with precipitation data.

Instrumentation conditions: One of four conditions of raingage instrumentation is found on a watershed:

1. Several gages spaced regularly near and within the watershed, so that the watershed may be thought of as being well and uniformly gaged.

2. Several gages in or near the watershed, with irregular spacing so that uniform coverage of the watershed is not obtained.

3. One gage in or near the watershed.

4. No gages in or near the watershed.

The determination of watershed rainfall for condition 1 is discussed below under "Area rainfall". Condition 2 may be considered similar but less accurate than condition 1. For condition 3 see below, "Point rainfall, expansion to area", and also NEH-4, 4.5.4 Depth-Area curves. The procedure for condition 4 is discussed below in "Storm transposition". Also see the instrumentation conditions described in Section 3.5.

Sources of data. (See NEH-4, Table 4.4-1, Sources of precipitation data.) Rainfall amounts are usually compiled from "Climatological Data" (see NEH-4, Table 4.4-1). Daily amounts for non-recording gages (standard gages) and hourly amounts for recording gages are given. Earlier "Climatological Data" varied from this practice. The times of measurement of standard gages may vary (as described in footnotes), and a storm rainfall amount measured at one gage may be tabulated one day later than other gages in the watershed.

So-called "bucket survey" data are sometimes available for unusually large storms. Such data consist of rainfall catches in watering troughs , buckets, pop bottles, and similar items. Ordinarily the data may assist in extending isohyetal lines that are mainly based on standard or recording gage data. When the catch of a "bucket gage" exceeds the catch of the nearest standard or recording gage by more than about 30 percent, the "bucket gage" catch should be used cautiously, if at all.

Narrow bore tubes of the type popularly used as raingages have been experimentally shown to give results almost equal to those from standard gages. Exposure and servicing of the tube gage must be made properly in order to obtain such results.

The data for each gage should be tested for length of record (Section 3.18). The double mass method (NEH-4, 4.3.3 Double mass analyses for adjusting, interpolating, or extending data) may be used to extend certain types of records. Other types may be extended by using regression methods (see a statistical text for details).

Historical series. Flood dates for which storm rainfall amounts need to be compiled are obtained from USGS Water-Supply Papers or by interviews with watershed residents. In the latter case the economist may obtain the dates. (See Economics Guide, Chapter 2.)

Synthetic storm series. The process for this series is like that for the historical series except that only one historical storm per year is used. The data are used to prepare frequency lines. See Section 3.18 Synthetic series, for other details.

Area rainfall. For discussions and methods of obtaining average rainfall over areas of various sizes, see NEH-4, 4.5.1 Average of precipitation amounts for all stations in the area; 4.5.2 Thiessen method; and 4.5.3 Isohyetal method.

Point rainfall, expansion to area. Relations of point rainfall to area are discussed in NEH-4, 4.5.4 Depth-area curves and 4.6 Depth-area-duration of storm rainfall. Such relations are useful for instrumentation condition 3 (see above) to expand the data at one gage to areas of various sizes. Figure 3.4-1 shows depth-area-duration curves suitable for ordinary use, while figure 3.21-4 shows depth-area curves for design storm purposes.

Storm transposition. This process consists of selecting a storm that occurred over one area and using it over a second area that is believed to be climatically homogeneous with the first.

An evaluation series may be prepared by this method for watersheds without raingages in or near them. The pattern of gages in the nearest area with a similar climate is assumed to exist in the watershed. The records are tabulated and used as for instrumentation conditions 1 and 2 (see above). Sometimes an already completed set of storm computations may be transferred from a nearby watershed on which a hydrologic study has been made.

Transposition of storm data from one hilly watershed to another may involve the orographic influences discussed below. Storm transposition in mountainous areas is less often practiced, since it is difficult in such cases to detect and adjust for the differences in physiographic variables.

Orographic influences. In hilly or mountainous country, rainfall will be influenced by physiographic variables. Walter T. Wilson, of the Weather Bureau, in a communication to H. O. Ogrosky has named the most commonly used variables as (1) elevation, (2) local slope, (3) orientation, (4) degree of exposure, (5) distance from moisture source, and (6) topographic barriers to incoming moisture.

Service hydrologists will seldom find it possible to determine the influences of all these variables in specific cases. Where special studies are needed for important projects, arrangements can be made through the Washington staff for cooperative studies by the Weather Bureau.

In the simpler cases of orographic influences, one variable (usually elevation) is used together with local measured rainfall, to obtain a relation that is later used for developing an isohyetal map based on the local data. Even in these simple cases, some effort is made to keep other variables constant--that is, (1) the area is divided into zones in which all variables except elevation are about constant; (2) a rainfall-elevation relation is prepared for each zone using data within that zone; and (3) the relation is used with elevations to develop an isohyetal map for that zone, using the measured catches as firm data. Figure 3.4-2 shows an example of an elevation (or altitude) versus storm rainfall relation that was used in the construction of an isohyetal map. Note that orientation (desert side; coast side) was used as the principal additional variable in defining zones.

Time distribution. A variety of ways in which storm patterns can be developed is illustrated in NEH-4, 4.4 Time distribution of storm rainfall, and 4.6 Depth-area-duration of storm rainfall.

Antecedent rainfall. This is the amount of rainfall in a selected period preceding a given storm. The rainfall in a period of from 5 to 30 or more days has been used in various methods to obtain an index of antecedent rainfall.

In general, the heavier the antecedent rainfall, the greater the direct runoff that occurs in a given storm. The effects of infiltration and evapotranspiration during the antecedent period are also important, and may increase or lessen the effect of antecedent rainfall.

The procedure of Section 3.10 "Estimation of direct runoff from rainfall", recognizes the difficulties of determining antecedent and storm conditions from normally available data. The conditions are therefore reduced to the three cases described here:

Condition I. An optimum condition of watershed soils, where the soils are dry but not to the wilting point, and when satisfactory plowing or cultivation takes place.

Condition II. The average case for annual floods.

Condition III. When heavy rainfall, or light rainfall and low temperatures have occurred during the 5 days previous to the given storm.

The three conditions may be estimated from rainfall records or by the use of streamflow records (see Section 3.5 "Antecedent flood conditions").

The recommended method of estimation using rainfall is to obtain total rainfall for five days preceding each storm; next, to arrange the storms into three groups corresponding to the three antecedent conditions on the basis of the antecedent total rainfall amounts. Table 3.4-1 shows rainfall groups that can be used on an annual basis. Table 3.4-2 shows rainfall groups that can be used on a seasonal basis. In general, the seasonal grouping is best. The rainfall amounts shown in both tables should vary slightly for different soil-cover complexes, but it is usually adequate to use the groupings as shown.

Experience and some soil studies have indicated that a five-day period is a minimum for estimating antecedent conditions. Longer periods, such as two weeks, are sometimes desirable but the additional work does not always produce additional accuracy in the runoff estimates.

Table 3.4-1. Rainfall groups for estimating antecedent conditions on annual basis.

Condition	5-day antecedent rainfall, in inches
I	Less than 0.5
II	0.5 to 1.5
III	Over 1.5

Table 3.4-2. Rainfall groups for estimating antecedent conditions by major seasons.

Condition	5-day antecedent rainfall, in inches	
	Dormant season	Growing season
I	Less than 0.5	Less than 1.4
II	0.5 to 1.1	1.4 to 2.1
III	Over 1.1	Over 2.1

Storm duration. The hourly data given in "Climatological Data" can be used to obtain storm durations to the nearest whole hour. The duration of the runoff-producing part of the storm is needed in runoff work, but since this can seldom be known exactly, the hourly data can be used to get an effective duration, which is an estimate of the duration of excess rainfall. Usually the storm total must be reduced when using an effective duration, since hours with low amounts at beginning and ending periods are not used. Table 3.4-3 shows schematic examples that can be used as a guide in estimating effective durations and the rainfall totals that go with them. The last six storm sequences could be used in other ways than are shown. For instance, the last sequence could be considered three storms for a small watershed or one storm for a large watershed. This illustrates the difficulty in defining a universal D for a given storm pattern. In practice, on watersheds less than 400 square miles in size, the D is often selected for the whole watershed and used without change for its subwatersheds.

Table 3.4-3 uses 0.21" as a lower limit of significant rainfall. Values less than 0.21" are not added to the storm total when those values are initial or final amounts. Various types of studies may be made of this limit, but the results should be exhibited in terms of significant changes in hydrographs, for hydrographs (or their peak rates) are the end results of these calculations. In practice it will be found that a constant limit, as used in table 3.4-3, is adequate. Hourly rainfall amounts less than 0.21" occurring at the beginning of a storm should be used in estimating the antecedent condition, as described above.

Daily rainfall at standard gages, as given in "Climatological Data" can generally be given a duration determined for the nearest recording gage. When no recording gage data are available, figure 3.4-3 will be of assistance in estimating a typical duration for use in a storm series. For example, the daily rainfalls for a storm series are selected from "Climatological Data" for a watershed having an annual rainfall of 32 inches; then the typical duration of storms in the series is 5 hours (rounded from 5.1 hours). This duration is used with all the storms in the series. Figure 3.4-3 is based on data from the areas shown, and is primarily for convection-type storms. However, brief studies have shown it useful for other storm types in southeastern and northwestern United States. A similar graph should be developed for other storm types when necessary, but it is important to note that the duration shown is an effective duration, that is, the duration of the runoff-producing part of a storm. Therefore, only runoff-producing storms should be studied in preparing such a graph.

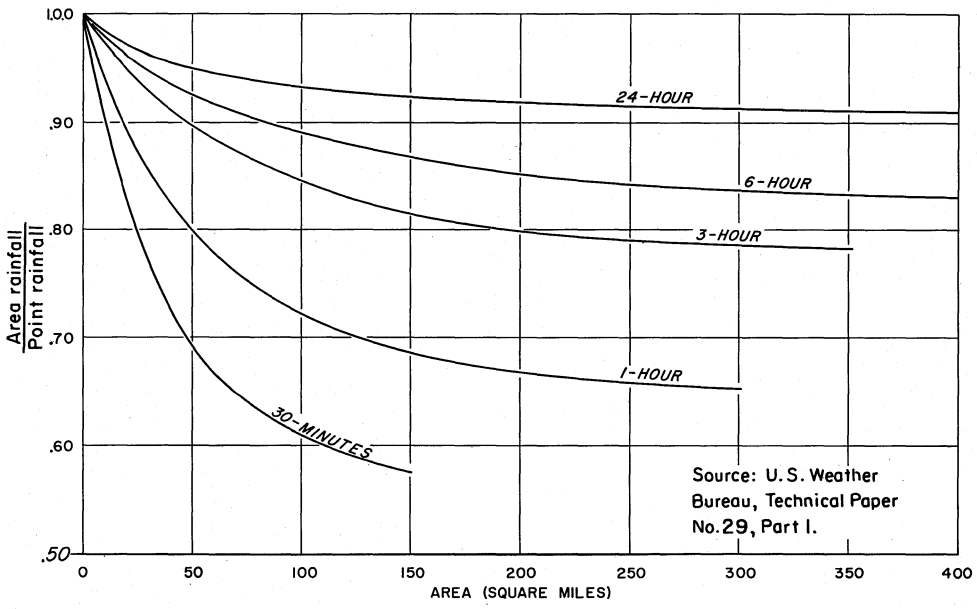
When recording gage charts are readily available, they will assist in determining the storm duration. Quite often, since rain does not fall as conveniently as could be wished, the duration will be a compromise value. Table 3.4-3 can be used as a guide for such cases.

Table 3.4-3. Examples of use of hourly data to define effective duration and accompanying rainfall.

Using 0.21" as lower limit

Sequence of hourly storm rainfall by inches in each hour						Duration		Rainfall	
						Given:	Use:	Given:	Use
Hour: 1	2	3	4	5	6				
(Inches)						(Hours)	(Hours)	(Inches)	(Inches)
			0.80			1	0.50	0.80	0.80
			.80	0.20		2	.75	1.00	.80
			.80	.20	.20	3	.75	1.20	.80
		0.20	.80			2	.75	1.00	.80
	0.20	.20	.80			3	.75	1.20	.80
		.20	.80	.20		3	1.00	1.20	.80
		.80	.80			2	1.00	1.60	1.60
		.80	.80	.20		3	1.50	1.80	1.60
	,20	.80	.80			3	1.50	1.80	1.60
	.20	.80	.80	.20		4	2.00	2.00	1.60
	.80	.80	.80			3	2.00	2.40	2.40
	.80	.80	.80	.80		4	3.00	3.20	3.20
	.80	.80	.20	.80		4	3.00	2.60	2.60
0.20	.80	.80	.20	.80		5	3.50	2.80	2.60
.20	.80	.80	.20	.80	.20	6	4.00	3.00	2.60
.80	.80	.20	.20	.80	.80	6	5.00	3.60	3.60
.80	.20	.20	.20	.80	.80	6	5.00	3.00	3.00
.80	.20	.80	.20	.80	.20	6	4.50	3.00	2.80

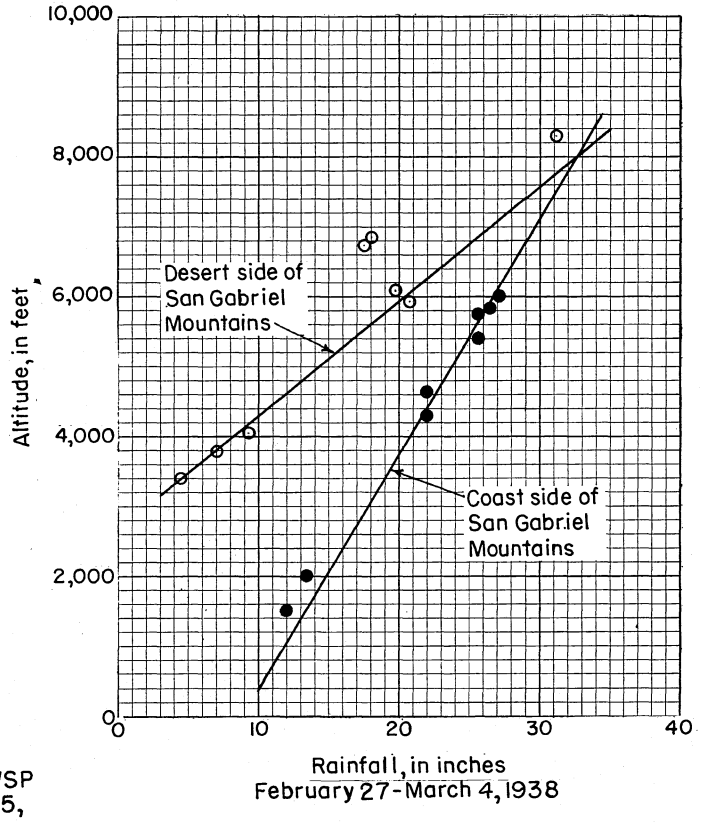
Note: Rainfall amounts shown are schematic values for indicating the parts of the given storm that have runoff-producing rainfall. In the first line, where 0.80 inch is given for one hour, the rain could have fallen in anywhere from about 1 to 60 minutes, with a probable average time of about 0.50 hour. In the second line, the 0.80 for the first hour is followed by 0.20 in the second, so that there is some (but not complete) assurance that the duration is greater than 0.50 hour, and is probably about 0.75 hour. Similar reasoning is used with succeeding lines.



RAINFALL TAKEN FROM USWB TP #29, ON YARNELL'S BULLETIN (USDA MISC. PUBL. #204).

This graph is for use with ~~gages whose catch is less than the maximum rainfall depth of the storm. If the gage has caught the maximum~~, use Figure 3.21-4 when DETERMINING AREAL RAINFALL OF SPILLWAY DESIGN STORMS TO MEET ENGR. MEMO. #3 REQUIREMENTS.

Figure 3.4-1



Source: USGS WSP No. 844, Figure 5, page 48.

Figure 3.4-2

426963 O - 57 - 4

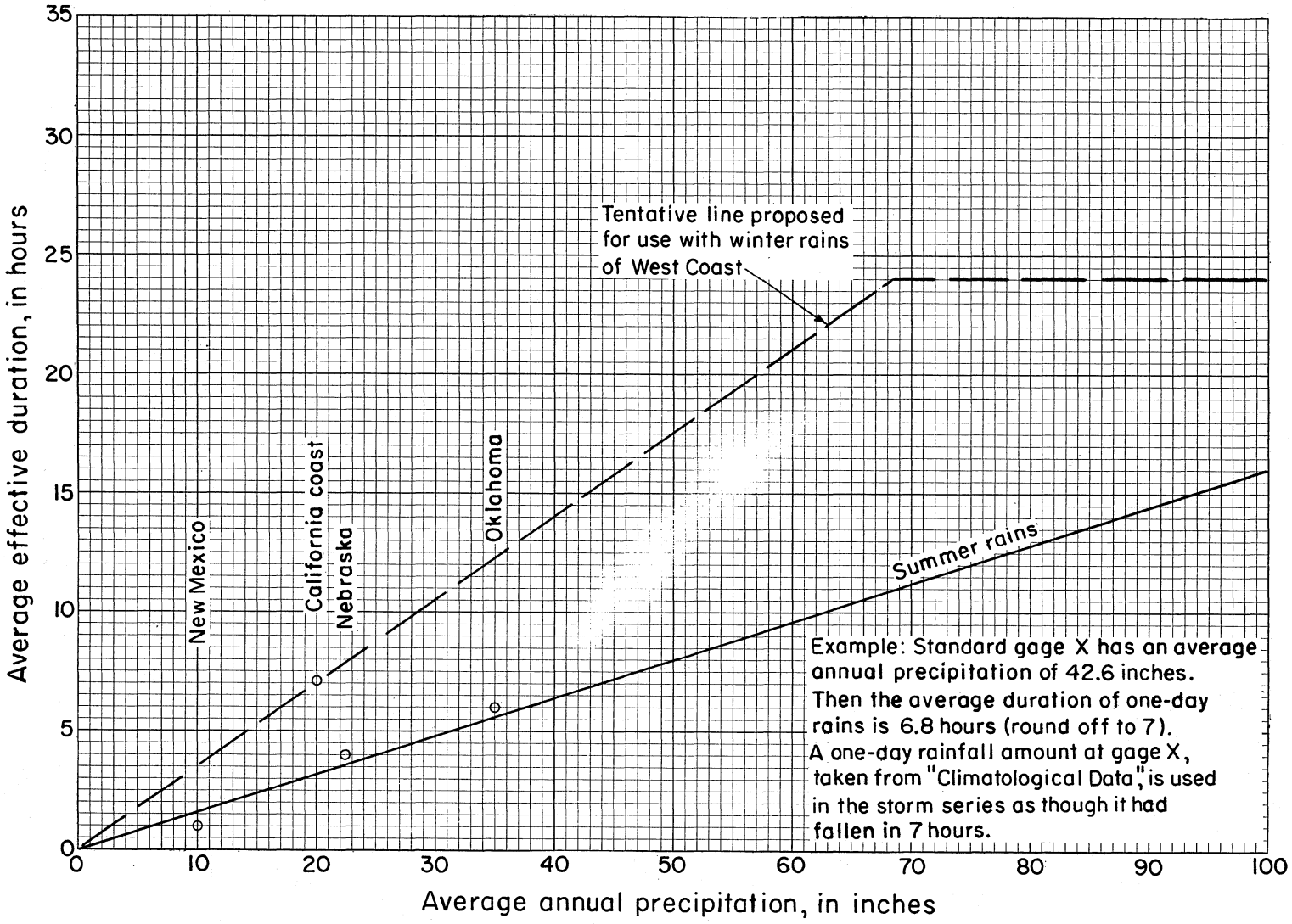

Average effective duration, in hours
35
30
25
20
15
10
5
0
0
10
20
30
40
50
60
70
80
90
100
Average annual precipitation, in inches
New Mexico
California coast
Nebraska
Oklahoma
Tentative line proposed for use with winter rains of West Coast
Summer rains
Example: Standard gage X has an average annual precipitation of 42.6 inches. Then the average duration of one-day rains is 6.8 hours (round off to 7). A one-day rainfall amount at gage X, taken from "Climatological Data", is used in the storm series as though it had fallen in 7 hours.

Figure 3.4-3

3.5 Streamflow data

Purpose. Streamflow data are used to determine present hydrologic conditions of a watershed (thus indirectly to determine present damage conditions) and to check procedures used to estimate future hydrologic conditions.

Sources of data. Streamflow data may be found in the files or publications of Federal agencies, State agencies, water boards, universities and colleges, reclamation districts, irrigation districts, flood control districts, municipalities, and private water users.

The chief sources of streamflow data are:

U. S. Geological Survey. Water-supply papers (WSP) and circulars are issued regularly.

USDA Soil Conservation Service. Streamflow data were formerly published in the TP, the Technical Bulletin, and the Hydrologic Bulletin series. Research stations formerly operated by SCS are now operated by ARS (see below). Records from pilot watersheds are being published in cooperation with the USGS.

USDA Forest Service. Streamflow data published in technical bulletins and papers.

USDA Agricultural Research Service. Streamflow data collected primarily from plots and small watersheds. Publications will be available soon.

It is difficult to build up a complete file of hydrologic data, since many publications are out of print or not available for distribution. For general information, refer to summary publications, and for details, use the files of the SCS E&WP Units, USGS district offices, and public or university libraries, offices of the Bureau of Reclamation, Corps of Engineers, TVA, and other Federal agencies, and other State and municipal water resource agencies.

Instrumentation conditions. One of four stream-gaging station conditions is found for a watershed:

1. One or more stream-gaging stations are located within the watershed, so that all important hydrologic soil-cover complexes may be thought of as gaged.

2. The watershed itself is not gaged, or is only partly gaged, but one or more stream-gaging stations are located in nearby areas of comparable hydrologic soil-cover complexes, so that the streamflow data may be transposed, and all important complexes in the watershed may be thought of as gaged.

3. Stream-gaging stations in or near the watershed gage some but not all of the important complexes.

4. None of the watershed's important complexes are gaged in or near the watershed.

The above classification of conditions applies to runoff volumes. It is very difficult to find a watershed that is gaged well enough to eliminate the need for hydrograph or peak rate estimates of one kind or another (see section 3.16).

When condition 1 exists on a watershed, storm rainfall data are not needed if the runoff records are of adequate length. Present watershed conditions are evaluated by use of the existing records. Conditions with future land use and treatment are evaluated using a graph of present versus future runoff (see "Determination of volume effects" in Section 3.12). Conditions with both future land use and treatment and structures are evaluated using flood routing (Section 3.17). Condition 2 is similarly treated. Condition 3 requires the use of rainfall records for the estimation of direct runoff for the ungaged complexes. Once the estimates of present runoff amounts for all complexes are made, the evaluation proceeds as before. Condition 4 requires the use of rainfall records on all of the complexes, again only through the evaluation of present watershed conditions.

Historical and synthetic flood series. Both types of series are based on experienced floods. In the historical series, the dates and peak rates of flow for every over-bank flow are compiled. In the synthetic series, the peak rates of flow for the annual floods are compiled, regardless of whether the flow was over-bank or not, and the annual floods are used to obtain a frequency line (see Section 3.18). In many cases it is better to work with flood volumes rather than peak rates of flow. The flood volumes for every damaging or over-bank flood are compiled for the historical series and the volumes for the annual floods are compiled for the synthetic series.

Peak rates of flow. These are generally listed in the data publications. Early USGS Water-Supply Papers may give only the annual flood peak in cfs. Peaks for other floods in a historical series may be obtained from the appropriate USGS district office. However, it should be kept in mind that peak rates of flow that were indirectly determined (slope-area method, contracted-opening method, extension of rating curve, weir formula, critical depth method) are subject to revision.

Flood volumes. (Also refer to NEH-4, Section 6, Storm Runoff.) The volume of flood runoff (direct runoff) must sometimes be separated from total flow that includes ground water (base flow). In other cases the flood flow must be separated from flow occurring from a previous flood. Both types of separation are discussed in NEH-4, Section 6. An approximate procedure for determining flood volumes directly from tables of mean daily discharges is shown in table 3.5-1.

Antecedent flood conditions. Base flow discharges from gaged watersheds are sometimes used as an index of storm runoff potential. For example, a high base flow indicates a high runoff potential, and a low base flow a low runoff potential (Figure 3.5-1b).

Such a relation between prior flow and storm runoff potential is more useful for flood forecasting than for a damage evaluation of a historical or synthetic series of floods. In general, if a record of flood flows is available for use in a historical or synthetic series, the antecedent conditions are not needed. In cases where records from a nearby watershed with certain soil-cover complexes are used to indicate antecedent conditions on an ungaged watershed with other soil-cover complexes, the following procedure may be useful:

1. Tabulate storm rainfall and total direct runoff for each flood on the gaged watershed, and for each date of a known flood on the ungaged watershed.

2. Plot the storm rainfall versus total direct runoff as shown in figure 3.5-1a. The flood date is given at each plotted point.

3. Fit a II-curve for the watershed to the points, using figure 3.10-1.

4. Curves for moisture conditions I and III are obtained using table 3.10-1, and plotted using figure 3.10-1.

5. Curves halfway between the I and II, and the II and III are drawn in as shown. The area between the halfway curves is shaded on figure 3.5-1a.

6. Flood dates for points falling in the shaded area are classed in the II condition; above the shaded area in the III condition; and below the shaded area in the I condition.

Figure 3.5-1b shows the relation between antecedent flow and the curve number of each point. This illustrates the type of plotting generally obtained. Flood runoff from storms with unusually high intensities of rainfall or from rainfall on frozen ground may fall considerably above the average line. (Also see definition of Moisture Condition III in Section 3.4.)

Table 3.5-1 Computation of runoff volume

Watershed: Wabash River, near New Corydon, Indiana

Drainage area: 258 sq. mi.

Source of data: USGS WSP 1275, p. 459

The maximum discharge for the 1953 water year is 4360 cfs on March 4, 1953. The flood volume for this annual flood can be determined directly from the tabulation of mean daily discharge, using a calculator. The data are:

Date	Mean daily discharge in cfs	Remarks
Feb. 27	156	Flow from previous rise.
28	136	
Mar. 1	126	
2	105	Low point.
3	222	Rise begins.
4	3630	Peak date.
5	1730	Flood receding.
6	558	
7	320	
8	191	
9	146	Approx. equal to low point.
10	206	New rise begins.
11	355	

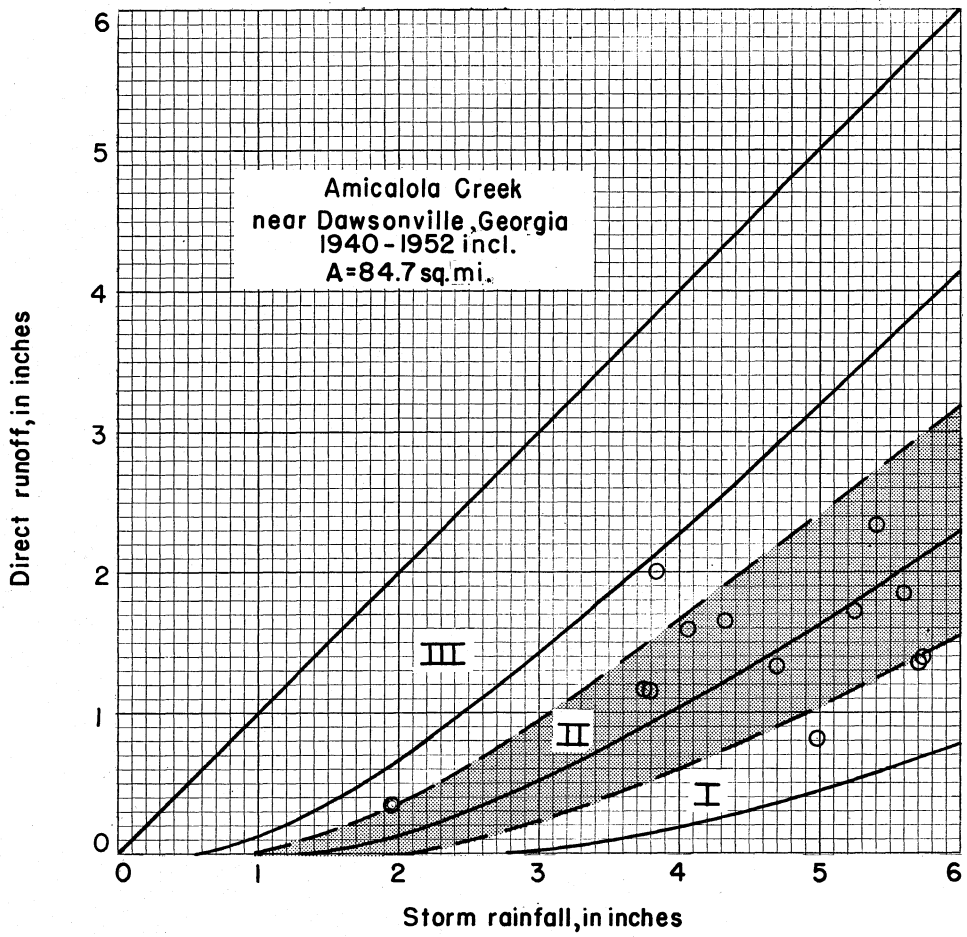
The underlined mean dailies from the low point before the flood through the approximately equal point on the flood recession are totaled and converted to runoff in inches, using the conversion from table 5.2-1:

$$\frac{\text{cfs days}}{\text{square miles}} (0.03719) = \text{inches depth} = Q''$$

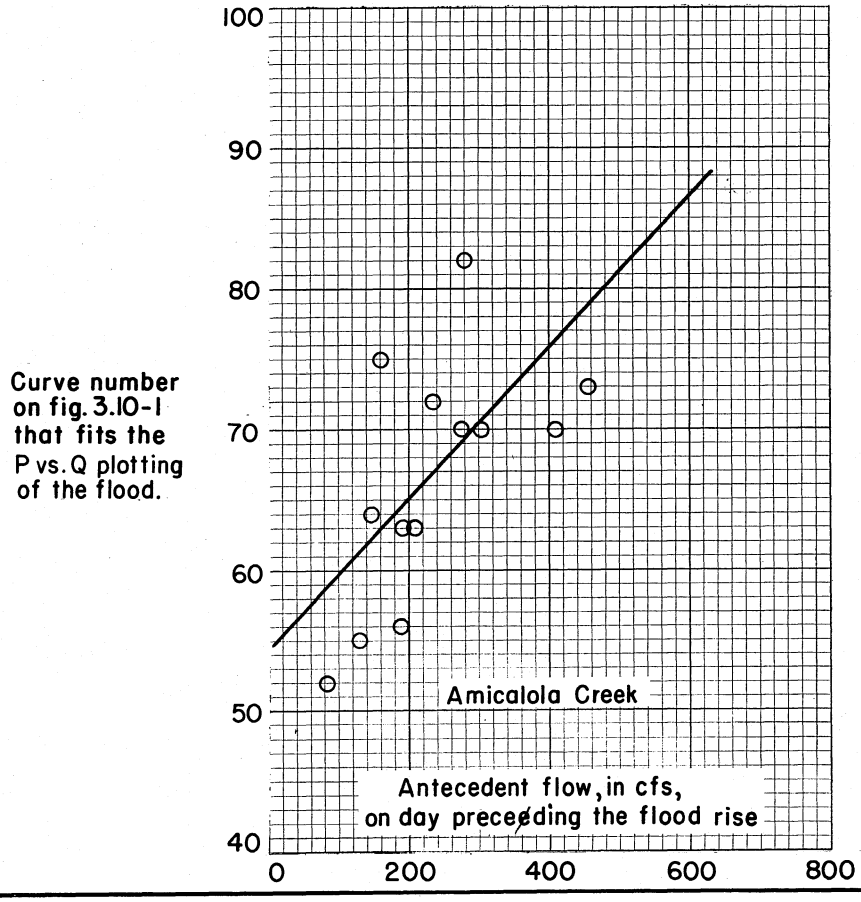
In this case: $\frac{6797}{258} (0.03719) = 0.9796''$. Use $0.98''$.

Runoff transposition. The practice of transposition is reasonably satisfactory with storm runoff, but less so with annual runoff. Usually the transposition consists of assuming that storm runoff for a given ungaged soil-cover complex for a given storm is the same as the storm runoff for a nearby gaged watershed with a closely similar soil-cover complex.

When storm rainfall varies much in amount and distribution, it is necessary to use figure 3.10-1. For example, a gaged watershed having P = 7.2" and Q = 3.2" would locate a point on the 64 curve of figure 3.10-1. An ungaged area with a similar soil-cover complex, a similar antecedent moisture condition, and having P = 4.1" would then have Q = 1.05" (about), using curve 64.



(a)



(b)

Figure 3.5-1

3.6 Stream reaches and cross sections

Purpose. Stream reaches and cross sections are used for establishing the relation between stage and inundation, or stage and discharge, for flood routing purposes, and for other related studies in the evaluation and operation of a system of watershed measures. All damages in a reach are usually related to the stage at one cross section in the reach.

Reaches solely for hydraulic purposes such as flood routing are selected to maintain a probable level of accuracy in the hydraulic computations.

Floodwater damages on an alluvial fan (also called talus, debris slope, debris fan, etc.) are usually related to the actual or estimated total direct runoffs, or to the peak rates, at an upstream gage or cross section.

Locations for reaches. The head (upstream end) and foot (downstream end) of a reach is usually at one of the following places:

a. Upstream and downstream end of floodplain areas where flood damage occurs.

b. Where a major tributary enters the main stem.

c. Where a major change in type of agricultural damage occurs. For example, within a reach where flood damage occurs, a high damage section of irrigated land may be found, and the upstream and downstream limits of these lands could be locations for heads of reaches.

d. Upstream and downstream limits of a town, oil-storage field, or any other area of high potential flood damage near which levees or other structural measures may be proposed.

e. At regular distances in the undamaged sections of the main stem and major tributaries, in order to keep flood-routing reaches from being too long and the combination of hydrographs from being inexact.

f. At each streamflow station in the watershed.

g. At each definite hydraulic control section, such as a weir, or a culvert in a high road fill.

h. At probable locations of major structures.

i. At sections where shape and characteristics of channel or valley change materially.

Process of locating reaches. The work plan party as a whole generally determines how many reaches are needed for a particular survey and where they should be located. Information obtained during the reconnaissance is used in preliminary location of reaches on a base map. Final agreement on the locations is generally made during the course of the instrument survey.

It is important to know the floodplain areas with probably significant floodwater damage, and to have these areas plotted on a base map before the selection and location of reaches. Quite often much survey time can be saved by reducing or eliminating cross-sectioning in areas with no important floodwater damage.

The floodwater damages are usually related to a cross section about in the middle of the reach, or at the lower end of the reach, or to all cross sections as shown in Section 4.2. The survey party decides which cross section locations are suitable. Once the probable locations of these cross sections are known, the drainage area above each such cross section is determined.

The subwatersheds formed by the boundaries are convenient area divisions for later use. Determinations of hydrologic soil groups (Section 3.7) are best made on a subwatershed basis. Similarly, hydrologic soil-cover complexes are most efficiently used when prepared on a subwatershed basis (Section 3.9). Table 3.6-1 shows a sample arrangement of watershed data for the watershed of figure 3.6-1. In this sample the floodwater damages in a reach are related to the cross section at the foot of the reach. The arrangement of data in table 3.6-1 is similar when the damages are related to a cross section near the middle of a reach.

Alluvial fans. Flood and debris damages on alluvial fans are usually evaluated directly in terms of frequency. A summary of the process is as follows: (1) Obtain information on the damage for each known flood by interview or from historical sources.
(2) Obtain the volume of flood runoff for each flood from streamflow records or estimate using rainfall data (see Section 3.10).
(3) Establish relation between flood runoff amounts and damages.
(4) Determine the frequency (Section 3.18) of flood damage.
(5) Plot a damage-frequency curve on arithmetic graph paper.
(6) Obtain the average annual damage by planimetering.
(7) Determine the effects of proposed programs on the flood amounts.
(8) Use the amount-damage relation to estimate remaining damages for given frequencies.
(9) Finally, derive new damage-frequency curves.

This summary shows that the hydrologist is less involved than when the stage-damage method previously described is used. He may not be involved at all in the selection or location of areas for estimating damages, since the hydrology can often be developed with reference only to the upstream gage or cross section.

Location of cross sections. One or more cross sections are located in each damage reach so that the average shape and the hydraulic characteristics of the channel and valley will be adequately defined. This means that generally the cross sections in a reach will be located at:

1. The head and foot of the reach.

2. At every major change in valley shape.

3. At several locations upstream and downstream from constrictions that may cause backwater, such as bridges or culverts. Cross sections along center lines of the roads may also be necessary. The requirements vary according to field conditions and type of project.

4. At regular intervals in a long, uniform reach if channel changes are to be evaluated.

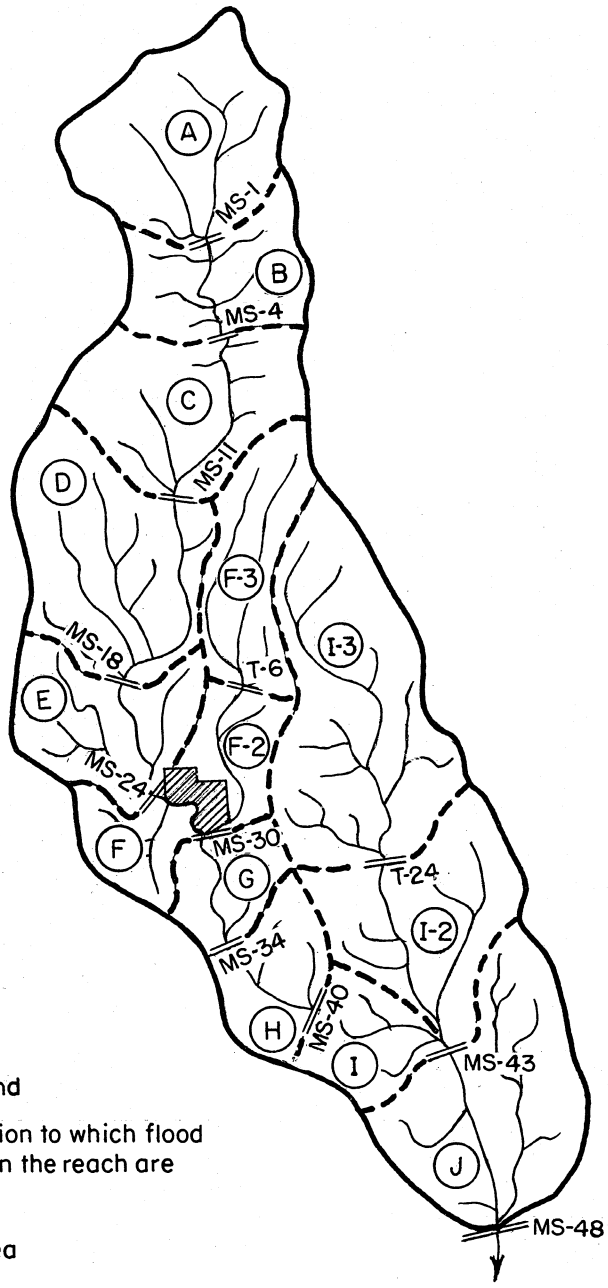
The cross sections in a given reach are usually located after examination of aerial photos of the flood areas. Stereoscopic pairs are sometimes used to find preliminary locations. Final locations are made in the field by the instrument survey party. The required number of cross sections will vary with the type of hydrologic evaluation and the complexity of the reach. One cross section per reach is often sufficient for small projects and even some larger ones. However, when channel improvements are to be proposed, many more cross sections per reach may be needed. The E&WP Unit hydrologist should be consulted for guidance in specific cases.

Usually the cross sections are referenced to a common datum, such as USGS bench marks, so that a stream profile can be drawn. In some cases the cross sections are independent of each other, as described in Section 3.14.

Table 3.6-1. Sample arrangement of watershed data. (Watershed shown on figure 3.6-1)

1	2	3	4	5	6	7	8	9	10	11	12
Reach number	Major cross-section 1/ number	Sub-water-shed number	Area: Sub-water-shed (sq.mi.)	Area: Accum-ulated (sq.mi.)	Time of concentration: Sub-watershed 2/ (hrs.)	Time of concentration: Total area 3/ (hrs.)	Subwater-shed soil group 5/	Soil-cover complex numbers: Present: Sub-water-shed	Soil-cover complex numbers: Present: Total area 6/	Soil-cover complex numbers: Future: Sub-water-shed	Soil-cover complex numbers: Future: Total area 6/
							TRIBUTARIES				
5-T	T-6	F-3	16.0	16.0	6.3	6.3	B	75			
	MS-30	F-2	6.5	22.5	2.5	7.8	C	80			
	MS-30	F	10.5	33.0	3.2	9.8	C	80			
8-T	T-24	I-3	37.6	37.6	10.8	10.8	B	76			
	MS-43	I-2	10.4	48.0	3.2	12.2	C	79			
	MS-43	I	24.0	72.0	5.4	15.8	D	85			
							MAIN STEM				
1	MS-1	A	28.0	28.0	8.9	8.9	B	68			
2	MS-4	B	16.5	44.5	4.3	11.7	B	72			
3	MS-11	C	23.5	68.0	5.3	15.1	B	74			
4	MS-18	D	27.8	95.8	5.9	18.5	C	78			
5	MS-24	E	17.2	113.0	4.4	20.4	C	78			
6	MS-30	F	33.0	146.0	9.8 4/	23.8	C	77.6			
7	MS-34	G	9.3	155.3	3.0	24.8	C	80			
8	MS-40	H	10.7	166.0	3.3	25.8	D	85			
9	MS-43	I	72.0	238.0	15.8 4/	32.0	D	79.4			
	MS-48	J	30.0	268.0	6.2	34.4	D	85			

1/ Cross section to which damages in the reach are related.
2/ T_c for a subwatershed computed as shown for subwatershed B of figure 3.16-8(a) and on table 3.16-4.
3/ T_c for the total area is not an accumulation of subwatershed T_c values except for a headwaters subwatershed (F-3, I-3, A).
4/ From column 7 of the tributary breakdown given above.
5/ See Section 3.7.
6/ Soil-cover complex numbers for total area above the foot of the reach, obtained by weighting. See details in Section 3.9, table 3.9-4.



Legend

Cross section to which flood damages in the reach are related.

Urban area

A,F-2 Subwatershed designation

MS-4,T-6 Cross section designation

Figure 3.6-1

3.7 Hydrologic soil groups

The following text was largely prepared by G. W. Musgrave, Staff Specialist (Infiltration):

Purpose. Watershed soil determinations are used in the preparation of hydrologic soil-cover complexes (Section 3.9), which in turn are used in estimating direct runoff. Four major soil groups are used. The soils are classified on the basis of intake of water at the end of long-duration storms occurring after prior wetting and opportunity for swelling, and without the protective effects of vegetation.

The major soil groups are:

A. (Lowest runoff potential). Includes deep sands with very little silt and clay, also deep, rapidly permeable loess.

B. Mostly sandy soils less deep than A, and loess less deep or less aggregated than A, but the group as a whole has above-average infiltration after thorough wetting.

C. Comprises shallow soils and soils containing considerable clay and colloid, though less than those of group D. The group has below-average infiltration after pre-saturation.

D. (Highest runoff potential). Includes mostly clays of high swelling percent, but the group also includes some shallow soils with nearly impermeable subhorizons near the surface.

Determination of hydrologic soil groups. This determination is made on a subwatershed basis. The subwatersheds have been previously decided on (Section 3.6). Each subwatershed is classified as being in one of the four major groups given above. The soil array, table 3.7-1, is used to find the average classification of the soils. For example, a given subwatershed has 80 percent of its area in the B soil group and 20 percent in the C. The C soils are interspersed with the B soils. The subwatershed is classed as a B soil group area. The C soil group cannot be handled separately unless it is a compact area--in which case, the subwatershed is divided into two parts, which are handled as individual subwatersheds during the estimations of direct runoff volumes. In general, however, the original subwatershed areas should be used as units, since the original divisions considered the soil groups (Section 3.6).

The determinations are made by the hydrologist in consultation with a designated Work Unit, Area, or State soil scientist.

Soil arrays. The relative hydrologic response of different soils or soil groups is an essential item in many engineering determinations. The following list of major soils of the United States (i.e., either of major importance locally or of major extent) is a guide to the relative base rating of any soil. A local soil not included in the list can be compared with those that are included and its relative position thus determined.

The array is based on the premise that soils of similar profile characteristics (particularly depth, texture, organic matter content, structure, and degree of swelling when saturated) will respond in essentially similar manner under a long storm of appreciable intensity. In making comparisons it is assumed that the soils have minimum cover (bare); maximum swelling has taken place; and the applied rainfall exceeds potential infiltration. Since different types of infiltrometers are known to give different magnitudes of f_c for the same soils, it is necessary to avoid using data from diverse techniques of measurement in developing the array. One soil cannot be placed on the basis of one technique of measurement and another soil placed according to a different method. All types of information are justifiably used in placing a soil in its proper relative position among other soils.

The measurement of infiltration rates through the artificial application of water, as by "rainfall simulators" or by one of several procedures for flooding the soil surface without rain impact, has been a common procedure for comparing soils or the effects of vegetation. These procedures are valuable for the relative comparisons they provide but they are not satisfactory as a means of expressing infiltration rates quantitatively.

Extensive comparisons under well replicated and controlled conditions show:

(1) Rainfall simulators providing rainfall impact and turbidity of surface water give lower rates for the same soils and vegetation than do flooding types. (Thus Type F and FA infiltrometers give lower rates than do tubes or rings.)

(2) Infiltrometers of larger ground area give lower rates than do those covering a smaller area where the proportion of border effects is greater.

(3) Generally infiltrometers also tend to give larger rates than those derived from watersheds.

Conversion factors for adjusting the results from different techniques to an equivalent base have been found to be impractical.

To overcome these difficulties, use is made of the extensive knowledge of soil profile characteristics possessed by the soil scientists. On this basis the major soils of the country are placed in an array, certain members of which already have been rated by watershed studies.

When the major soils of the United States are arranged in proper relative order, the range of f_c will begin with tight clays--essentially zero rates under these conditions--and extend to the maximum rates of deep, well aggregated silts or to those of the deep sands such as are found in sandhill areas.

The curve of the array is essentially that of fig. 3.7-1. A possible normal range of variation due to variation within a given soil by depth, strauctura, or texture is expressed by the dash line--25 percent above and 25 percent below the mean. The mean is indicated by the heavy line.

On this curve a number of points have already been established and additional ones will be forthcoming from evaluation of watersheds, analyses of existing research data, and new experimental work. Thus the magnitude of the curve can be fixed on the basis of minimum watershed performance. The problem of converting different kinds of infiltrometer data to a watershed basis is thus solved.

It is clear that the curve presenting this array of United States soils can be regarded as a base or "floor" upon which may be superimposed the effects, for example, of lesser soil moisture, varying kinds of vegetation, and the accumulation with time of organic matter derived from better vegetation. Supplementary information on the divergent effects of soil moisture on sands, clays, or laterites, of the aggregating effects of organic matter on these different kinds of soil, and the resultant effect on infiltration, will aid in the practical application of the basic soils rating.

The major soils in each of four hydrologic groups are listed in table 3.7-1. The area or areas in which each has been reported are indicated by the numeral or numerals following each soil name and which are keyed to the map, fig. 3.7-2.

Acknowledgments are due the numerous Soil Scientists and Soil Correlators who have so freely provided basic information on the physical properties of these soil profiles. It is their detailed knowledge that has made possible this array of about 2,000 major soils of continental U. S. A.

426963 O - 57 - 5

Table 3.7-1. Hydrologic soil groups 1/

GROUP A

(Includes deep sands with very little silt and clay; also deep, rapidly permeable loess)

Soil	2/ Area or areas reported
Adams	1
Alton	1
Americus	2
Ankeny	3
Arion loamy fine sand	3
Arkport	1
Arnold sandy loam	6
Arredondo	2
Ashley	6
Aspen	6
Balls loamy fine sand	5
Barnston gravelly sandy loam	6
Barth	2
Bartleson	6
Beebe	6
Bellevue	2
Berrien	3
Billingsley fine sand	4
Bingen	4
Blanton	2
Boone fine sandy loam	3
Braham	3
Brazita	4
Broward	2
Brownfield sand and loamy sand	4
Bruno sand and loamy sand	2, 4
Buckner loamy sands and sands	3
3/Carlisle (New York)	1
Carver	1
Cashmere	6
Central	3
Chelsea sand and loamy sand	3
Chenango	1
Chetek	3
Chiefland	2
Chute	3
Cinibar silt loam	6
Cispus pumicy sandy loam	6
Clack	2
Coloma	3
Colonie	1
Colosse	1
Colton gravelly sandy loam	1
Crevasse	2
Croghan	1
Derby fine sand and loamy fine sand	4
Deschutes	6
Dolph sandy loam	6
Duelm	3
Dukes	1
Dune sand	1,2, 5
Dwyer fine sand	5
Eatontown	1
Elliber	1
Elmira	6
Emerson	6
Enterprise loamy fine sand	4
Ephrata loamy sand and sandy loam	6
Estherville loamy sand	3
Eufala fine sand	4
Eustis	2,,4
Everett gravelly sandy loam	6
Fitch gravelly sandy loam	6
Flasher fine sand	5
Fordney loamy sand	6
Fort Meade	2
Garrison	6
Gaviota sandy loam	6
Gearhart sands	6
Geer	6
Giles very fine sandy loam	6
Glenbar	4
Goldridge fine sandy loam	6
Grayling	3
Greenwater sandy loam	6
Grimstad loamy fine sand	3
Gudrid loamy fine sand	3
Guin	2, 4

Soil	2/ Area or areas reported
Hampden	1
Haskill fine sand	5
Hesseltine gravelly sandy loam	6
Hinckley	1
Hiwood loamy fine sand	3
Holden	6
Hoodsport gravelly loam	6
Hoosic	1
Hubbard fine sand and loamy fine sand	3
Huckabee	2
Immokalee	2
Independence	2
Indianola sandy loam	6
Isanti loamy fine sand	3
Ivie	6
Izard loamy sand	4
Jaffrey	1
Jonesville	2
Kalkaska	3
Kenney loamy fine sand	4
Kershaw	2
Kilbourne	3
Klaus gravelly loam	6
Klej	2
Kootenai	6
Lakehurst	1
Lakeland fine sand	2, 4
Lakeville gravelly loam and sandy loam	3
Lakewood	1
Lakin loamy sand	1
Lauren sandy loam	6
Leavenworth sandy loam	6
Lihen loamy fine sand	5
Lincoln fine sand and loamy fine sand	4
Lynden fine sandy loam	6
Lynndyl	6
Lystair loamy sand	6
Manchester	1
Marble sand	6
Marenisco	3
Marina sand	6
Marquette gravelly loamy sand	3
Medio fine sand	4
Menahga	3
Monteola clay	4
Moorefield	4
Morocco	3
Moro Cojo loamy sand	6
Nashville	4
Nekoosa	3
Nodaway sand	3
Nueces fine sand	4
Oakville	3
Ochlockonee loamy fine sand	4
Orelia clay loam	4
Orem	6
Orlando	2
Otisville	1
Ottawa	1, 3
Ottokee	3
Palm Beach	2
Payette	6
Pend Oreille sandy loam	6
Pentura	6
Perks	3
Peshastin stony fine sandy loam	6
Petosky	1
Plainfield	3
Plummer	2
Plymouth	1
Pound	6
Preston	6
Quincy sand and loamy sand	6
Quonset	1

Soil	2/ Area or areas reported
Ragner fine sandy loam	6
Ratlum	1
Ravola	6
Riffe	6
Rodman	3
Roosville sand	5
Roscommon	3
Rubicon	3
Rudd	4
Rupert sand	6
Saffell	2, 4
St. Lucie	2
Santaquin	6
Sarpy loamy sand and sand	3
Selle	6
Sheppard	6
Sioux loamy sand	3
Skykomish gravelly sandy loam	6
Snoqualmie gravelly loam	6
Spanaway gravelly sandy loam	6
Spinks	3
Springdale	6
Springer loamy sands	4
Stillman	6
Syracuse	6
Tedrow	3
Thornwood gravelly loam	6
Tivoli sand and loamy sands	4, 5
Tombigbee	2
Touhey sandy loam	6
Toutle loamy sand	6
Tujunga gravelly sand	6
Tunkhannock	1
Ulen loamy fine sand	3
Valentine fine sand	5
Vebar fine sand	5
Victoria clay	4
Vilas	3
Vinton	6
Vona	5
Wardboro	6
Wasatch	6
Westport sands	6
Winchester sands	6
Wind River gravelly loam	6
Windsor	1
Yonkers	4
Zimmerman fine sand and loamy fine sand	3

1/ Since this is the first time a hydrologic array of this kind has been prepared, this grouping is tentative and subject to change. Adjustments will be made as discrepancies are discovered and as more information and experience are gained through usage. The soil series name only is given where the textural range is narrow and the soil occurs in only one group.

2/ See areas shown on fig. 3.7-2.

3/ Carlisle muck in Ohio is in Group C.

GROUP B

(Mostly sandy soils less deep than A, and loess less deep or less aggregated than A, but the group as a whole has above-average infiltration after thorough wetting)

Soil	1/ Area or areas reported
Abajo	6
Abernathy	2
Ackmen	6
Acton	1
Acworth	1
Agawam	1
Aiken clay loam	6
Akron	2
Albemarle	2
Albia	1
Albien	5
Alcoa	2
Alderwood gravelly loam	6
Alexandria	3
Alicel silt loam	6
Allagash	1
Allen	2, 4
Allenwood	1
Allison	3
Alvin	3
Amarillo fine sandy loam	4
Amenia	1
Amite	2, 4
Ammon	6
Amsterdam silt loam	5
Annabella	6
Annandale	1
Anthony	4
Appling	2
Arapien	6
Arenzville	1
Arkansas fine sandy loam	4
Aroostook	1
Arveson	3
Asbury	1
Ashe	2
Ashton	1
Astoria loam	6
Athena silt loam	6
Atterberry	3
Attleboro	1
Atwood	2
Aura	1
Avery	2
Badrock gravelly loam	5
Balfour	2
Bancroft	6
Bangor	1
Bannock	6
Barbourville	2
Barnes	3
Barnstead	1
Barrington	1
Bates	4, 5
Battrick stony loam	5
Baxter	2, 4
Beatty	1
Beaver	3
Beaverton gravelly loam	5
Becket	1
Bedford	2
Beechy	2
Benedicta	1
Benewah	6
Benson	1
Berkshire	1
Bermudian	2
Berthoud silt loam	4
Bertie	2
Bertrand silt loam	3
Beulah	4
Beverly	6
Bewleyville	2
Bickleton	6
Bienville	2
Biggs	3
Biggsville	3
Billett	3
Bingham	6
Birchwood	1
Birkbeck	3
Biscay	3
Blackfoot	6
Blakely	2
Blandford	1
Blanding	6
Blodgett	5
Bodine	2
Bold	3
Bolton	2
Bonner	6
Bowie	2, 4
Bowman	1
Boxelder	6
Boyer	3
Boynton	1
Bradbury	1

Soil	1/ Area or areas reported
Brady	3
Brandon	2, 4
Branford	1
Brassua	1
Bratton	3
Breece	5
Brennan loamy fine sand	4
Brenton	3
Bridgehampton	1
Bridgeville	1
Brimfield	1
Broadbrook	1
Brookfield	1
Brookston	3
Brownfield loamy fine sand	4
Brownlee	6
Brunswick	2
Buckland	1
Budger	6
Buncombe	4
Burke	6
Burnham	1
Burnsville	3
Burton	2
Butte stony loam	6
Bybee	2
Cabot	1
Cahaba	2, 4
Calais	1
Caldwell	6
Caleta	6
Calouse	6
Camas	6
Cameron	4
Canadian	4
Canyon	5
Capron	3
Capshaw	2
Carey	4
Caribou	1
Carmi	3
Carnegie	2
Carrington	3
Carstairs	6
Casa	4
Cascade	6
Casco	3
Castana	3
Castana-Napier complex	3
Catherine	6
Cattaraugus	1
Caylor	2
Cecil	2
Centerton	4
Chagrin	1, 3
Chalker	1
Chamokane	6
Charlton	1
Chaseburg	3
Chaseburg-Nodaway complex	3
Chattahoochee	2
Chehalis clay loam, loam, and silt loam	6
Chemawa	6
Cheshire	1
Chester	1, 2
Cheyenne	5
Chickasha	4
Chicopee	1
Chili	3
Chilmark	1
Chittenden	1
Choccolocco	2
Choctaw	4
Cisco	4
Clsiborne	2
Clallam	6
Clarion	3
Clarion-Dickinson complex	3
Clarion-Nicollet complex	3
Clarksville	2, 4
Clary	3
Clearfield	3
Cleburne	2, 4
Cle Elum	6
Cleora	4
Clifton	2
Clinton	3
Cloquet	3
Clyde-Floyd complex	3
Clymer	1
Cobb	4
Cokesbury	1
Colburn	6
Colby	5
Colebrook	1
Colfax	2
Collbran	6

Soil	1/ Area or areas reported
Collenston	6
Collins	2, 4
Colrain	1
Colton sandy loam	1
Conant	1
Condon	6
Conestoga	1
Congaree	2, 4
Cookeville	2
Cooper	3
Coosa	2
Copake	1
Cornwall	1
Cossayuna	1
Cotaco	2
Courtrock	6
Cowiche	6
Creedmoor	2
Creston	5
Crider	2
Crossville	2
Culleoka	2
Culpeper	2
Cumberland	2, 4
Cushman	5
Dade	2
Dakota	3
Dalhart fine sandy loam	4, 5
Danforth	1
Darnell stony fine sandy loam	4, 5
Davidson	2
Decatur	2, 4
Declo	6
Decorra	3
Dekalb	1
Dekoven	2
Delphi	6
Dewey	2
Dexter	2
Dickinson	3
Dickson	2
Dierks	2, 4
Dill	4
Disco	3
Dixie	6
Dixmont	1
Dodds	2
Dodgeville	3
Dolph loam	6
Dorchester	3
Doty	6
Dougherty	4
Douglas	6
Dover	1
Downey	6
Downs	3
Doyle	1
Dragston	2
Drake	4
Draper	6
Dresden	3
Drummer	3
Dubbs	4
Dubuque	3
Ducker	2
Duffield	1
Dunbar	2
Duplin	2
Durham	2
Dutchess	1
Duval fine sandy loam, deep phase	4
Easton	1
Ebbs	6
Edgemont	1, 2
Edneyville	2
Elco	3
El Dorado	4
Elioak	2
Elk	2, 4
Ellisforde	6
Ellison	3
Elmore	6
Elmwood	1
Emmet	3
Emory	1, 4
Endicott	6
Enfield	1
Ennis	2
Ensley	3
Enterprise very fine sandy loam	4
Essex	1
Estherville silt loam	3
Etowah	2
Eulonia	2
Ewingville	1

1/ See areas shown on fig. 3.7-2.

Group B (Cont'd)

Soil	1/ Area or areas reported
Faceville	2
Fairhaven	3
Faison	2
Falk	6
Fall	3
Fallbrook	6
Fallsington	2
Fannin	2
Farland	5
Farmington	1
Faunsdale	2
Fauquier	2
Fayette	3
Flanagan	3
Flathead	5
Flom	3
Florham	1
Floyd loam and silt loam	3
Forbes (undifferentiated)	3
Fossum	3
Fox	3
Foxon	1
Frankstown	1
Frederick	1
Fredon	1
Freeland	2, 4
Fruita	6
Fullerton	2
Gainesville	2
Gallion	4
Galva	3
Geary	5
Genesee	1, 3
Genola	6
Georgeville	2, 4
Gilbert	3
Gilcrest	5
Gilead	2
Gilford	3
Glendive	5
Glenelg	1, 2
Glenwood	3
Gloucester	1
Glover	1
Goldsboro	2
Goodloe	2
Grafton	1
Granby	3
Grant	4, 5
Grantsdale	5
Grantsville	6
Granville	2
Gravity	3
Green Bluff	6
Greenriver	6
Greensboro	1
Greenville	2, 4
Grimstad fine sandy loam	3
Grover	2
Groveton	1
Grygla	3
Gudrid fine sandy loam	3
Habersham	2, 4
Hackers	1
Hackettstown	1
Haddam	1
Hadley	1
Hagener	3
Hagerstown	1, 2
Halewood	2
Half Moon	5
Halsey	1
Hamblen	2
Hamburg	3
Hamilton	5
Hanceville	2, 4
Hannahatchee	4
Hartford	1
Hartland	1
Hartleton	1
Hartsells	2
Hayden	3
Hayesville	2
Haynie	3
Hayter	1, 2
Hazel	2
Hector	4
Heisler	6
Hembre	6
Hermitage	2
Hermon (Gloucester)	1
Herndon	2, 4
Hero	1
Hidalgo	4
Highland	6
Hiko Springs	6
Hillsboro	6
Hillsdale	3
Hiwassee	2, 4
Hixton	3
Hobbs	6
Hodgenville	2

Soil	1/ Area or areas reported
Hoffman	2
Holdrege	5
Holland	6
Hollis	1
Holston	1, 2, 3, 4
Holyoke	1
Honeoye	1
Hopper	3
Houlton	1
Housatonic	1
Hovenweep	6
Howard	1
Hoye	6
Hubbard sandy loam	3
Hublersville	1
Hugo sandy loam (Calif.)	6
Humphreys	2
Hunters	6
Huntington	1, 2, 3, 4
Huntsville	3
Hyattsville	2
Hyde	2
Hymon	2
3/ Hyrum (Utah)	6
Ida	3
Ina	2
Iona	3
Iron River	3
Irvington	2
Jackson	3
Jefferson	2, 4
Jenness	6
Johnston	2
Joy	3
Juliaetta	6
Kalamazoo	3
Kalmia	2, 4
Kanosh	6
Kars	1
Katama	1
Kato	3
Kaysville	6
Keith	5
Kempsville	2
Kennebec	3
Kennedy	6
Kenton	2
Kerby	6
Keyport	2
Kilburn	6
Kittitas	6
Knappa	6
Knutsen	6
Lackawanna	1
Ladoga group	3
Laidig	1
Lamont	3
Lancaster	5
Landes	3
Lanesville	1
Lansdale	1
Lansing	1
Lapine	6
Laredo	4
Larimer	5
Larkin	6
Lawrence	2
Leadvale	2
Lee	2
Leetonia	2
Lehew	1, 2
Lehew-Ashby complex	1
Leicester	1
Lempster	1
Lenox	1
Leon	2
LeSeur	3
Letort	1
Lewisberry	1
Lexington	2, 4
Lihen fine sandy loam	5
Limerick	1
Lincroft	1
Linker	2, 4
Linneus	1
Lintonia	4
Littleton	3
Lobelville	2
Lodi	2
Lolalita	6
Lolo	5
Lonoke	4
Loradale	2
Lorenzo	3
Louisa	2
Louisburg	2
Lyman	1
Lynchburg	2
Lyons	1

Soil	1/ Area or areas reported
Machias	1
Madison	2
Madras	6
Madrid	1
Magnolia	2
Mahaska	3
Mallory	2
Manitou	2
Manor	1
Mansfield	1
Marlboro	2
Marlow	1
Marquette sandy loam	3
Marshall	3, 5
Masada	2
Masardis	1
Massilon	3
Matapeake	2
Mattapex	2
Maumee	3
Maury	2
May	4
Mayfield	6
Mayodan	2
McBeth	6
McBride	3
McCammon	6
McKenna	6
McPaul (Hornick)	3
Meda	6
Mellenthin	6
Melrose	1
Memphis	2, 4
Mendon	6
Merrimac	1
Mess	6
Metea	3
Methow	6
Middlefield	1
Milaca	3
Miles	4
Millard	6
Mill Creek	3
Minco	4
Mindensilt	3
Minidoka	6
Minvale	2
Moffat	6
Monarda	1
Monona	3
Montalto	1
Monticello	6
Montville	1
Moody	3
Moody-Crofton-Arion-Thurman complex	3
Morrison	1
Morton	5
Moscow	6
Moshannon	1
Moulton	6
Mt. Carroll	3
Mountview	2
Murrill	1
Muscatine	3
Muse	2
Musinia	6
Muskingum 2/	1, 2, 4
Musselshell	5
Myersville	2
Myton	6
Nantucket	1
Napier	3
Naples	6
Narcisse	6
Narragansett	1
Nashua	1
Nason	2
Nassau	1
Nebish	3
Negley	3
Nehalem	6
Nesika	6
Newberg	6
Newfield	1
Newmarket	1
Newport	1
Newton	3
Newtonia	4
Nicholson	2
Nicollet	3
Ninigret	1
Nixon	1
Noble	4
Nodaway silt loam	3
Nodaway-Napier-Colo	3
Nolichucky	2
Norfolk	2, 4
Norwood	4
Nova	6

1/ See areas shown on fig. 3.7-2.

2/ Muskingum is in Group C in Ohio.

3/ Group C in Idaho.

Group B (Cont'd)

Soil	Area or areas reported
Oasis	6
Obion	2
Ochlockonee fine sandy loam	2, 4
Ockley	3
Okenee	2
Olequa	6
Olmitz	3
Olympic silty clay loam	6
Ona	2
Onamia	3
Onaway	3
Ondawa	1
O'Neill	3
Onslow	2
Ontario	1
Onyx	6
Ooltewah	2
Ora	2, 4
Orangeburg	2, 4
Orchard	6
Oshtemo	3
Otero	5
Othello	2
Otley	3
Ozark	4
Pace	2
Pack	6
Paden	2
Paiso	4
Palisade	6
Palmyra	1
Palouse	6
Paraloma	4
Parker	1
Parleys	6
Pasquotank	2
Patit Creek	6
Pawlet	1
Paxton	1
Peacham	1
Pegram	6
Pekay group	3
Pembroke	2
Penasco	4
Pennington	1
Penwood	1
Peone	6
Pequanoc	1
Perkinsville	2
Peterboro	1
Philby	3
Philo	2, 4
Pickwick	2
Pierce	3
Pinson	2, 4
Pisgah	2
Pittsfield (Nellis)	1
Pittsford	1
Pittstown	1
Plaisted	1
Pleasant Grove	6
Pocomoke	2
Podunk	1
Poe	6
Pomfret	1
Pontotoc	4
Pope	1, 2, 3, 4
Poquonock	1
Portales	4
Port Byron	3
Porters	2
Portneuf	6
Portsmouth	2
Potlatch	6
Poultney	1
Presque Isle	1
Primghar group	3
Proctor	3
Puget	6
Purgatory	6
Puyallup	6
Quakertown	1
Quandahl	3
Rabun	2
Racine	3
Rains	2
Ralston	6
Ramsey	2
Rapidan	2
Red Bay	2
Redfield	6
Red Hook	1
Red Rock	6
Reeves	4
Renslow	6
Reynolds	3
Riddle	4
Ridgebury	1
Rimer	3
Ringwood	3
Ritzville	6
Roane	2, 4
Robinsonville	2, 4
Rockaway	1
Rockingham	1
Rockmart	2
Rockton	3
Rockton-Dodgeville (chert phase)	3
Rockwell	3
Roseburg	6
Roseville	3
Royalton	1
Rumford	2
Rumney	1
Russell	3
Ruston	2, 4
Ryders	1
Sable	3
Sac	3
Saco	1
St. Agatha	1
St. Albans	1
St. Charles	3
St. George	6
St. Joe	6
St. Johns	2
St. Mary's	6
Salem	6
Salix	3
Sanpete	6
Sassafras	2
Saugatuck	1, 3
Sawmill (Illinois)	3
Saybrook	3
Scarboro	1
Schnorbush	6
Schumacher	6
Sciotoville	2
Scituate	1
Scobey	5
Scorup	6
Scranton	2
Seaton	3
Selma	3
Seneca	2
Sequatchie	1, 2
Sevy	6
Shannon	2, 4
Shapleigh	1
Sharon	3
Sharpsburg	3
Sheffield	1
Shelburne	1
Shelbyville	2
Shelton	6
Sheridan	6
Sidell	3
Sifton	6
Sigurd	6
Sinclair	6
Sioux sandy loam	3
Siskiyou	6
Sisson	3
Skerry	1
Skiyou	6
Snohomish	6
Snow	6
Sogn	5
Squapan	1
Squires	1
Starr	2
Staser	2
State	2, 4
Statesville	2
Steinsburg	2
Stephenville	4
Sterling	6
Stetson	1
Stevens	6
Stissing	1
Stockbridge	1
Storden loam	3
Strasburg	1
Stronghurst	3
Stupel	6
Sudbury	1
Sultan	6
Sumner	3
Sunset	6
Surprise	6
Sutton	1
Sverup	3
Swanton	1
Swauk	6
Tabiona	6
Talcott	3
Tallula	3
Tama	3
Tanberg	3
Tatum	2
Teller	4
Tellico	2
Terrill	3
Terry	5
Thorndike	1
Thurman-Clarion complex	3
Thurman-Marshall complex	3
Thurman-Tama complex	3
Thurmont	2
Tifton	2
Tilden	2, 4
Timpanogos	6
Timula	3
Tioga	1
Tirzah	2
Tisbury	1
Todd	3
Tokul	6
Tollgate	6
Townsbury	1
Tridell	6
Tusquitee	2
Umpqua	6
Unadilla	1
Uncompahgre	6
Valois	1
Van Buren	1
Vassar	6
Vaucluse	2
Vicksburg	2, 4
View	6
Vista	6
Volinia	3
Volney	3
Wadena	3
Wadesboro	2
Wahee	2
Waits	6
Walden	1
Wallagras	1
Walla Walla	6
Walpole	1
Warm Springs	6
Warsaw	3
Warwick (Stratham)	1
Washburn	1
Washington	1
Wassaic	1
Watauga	2
Waukegan	3
Waukesha	3
Waumbek	1
Wauseon	3
Waynesboro	1, 2, 4
Weaver	2
Weeksville	2
Welby	6
Wenham	1
Westland loam	3
Westminster	1
Wethersfield	1
Whately	1
Wheeler	6
Wheeling	1,2, 3
Whitefish	5
Whiteford	2
Whitman	1
Whitwell	2
Wickham	2, 4
Wilkinson	6
Willamette	6
Williams	5
Wilmington	3
Wiltshire	1
Winooski	1
Witt	4
Woodbridge	1
Woodside	5
Woodstock (rocky)	1
Woodstown	2
Woodward very fine sandy loam	4
Wooster	1, 3
Woostern	1
Worthen	3
Worthington	1
Yadkin	2
Zahl	5
Zane	6
Zaneis fine sandy loam	4

GROUP C

(Comprises shallow soils and soils containing considerable clay and colloid, though less than those of Group D. The group has below-average infiltration after pre-saturation.)

Soil	Area or areas reported
Abilene fine sandy loam	4
Abington	3
Acme clay loam	4
Ada	6
Addison	1
Afton	3
Agency	6
Agnos	4
Alamance	2, 4
Aldino	1
Algiers	3
Almo	2, 4
Altamont	6
Altavista	2, 4
Alvira	1
Amarillo clay loam	4
Ambraw	3
Ames	3
Amity	6
Andres	3
Angie	2
Antelope Springs	6
Applegate	6
Ark	4
Armagh	1
Armuchee	2
Asa	4
Ashby	1
Ashkum	3
Ashwood	2
Atkins	1, 2, 3, 4
Auburn	6
Augusta	2, 4
Austin	4
Ava	3
Avalon	6
Baca	4
Baker	6
Baldock	6
Bark River	3
Barrows	3
Bath	1
Bayside	6
Beasley	2
Beaucoup	3
Beauregard	4
Belinda	3
Belmont	1
Bennington	3
Bentonville	3
Berks	1
Berlin	1
Bernardston	1
Berwick	3
Beryl	6
Bethany	4
Bibb	2, 4
Biddeford	1
Bieber	6
Big Timber	5
Billings	4, 6
Binnsville	4
Bissell	6
Blacklock	6
Blockton	3
Blount	3
Bluford	3
Bogota	3
Bolivar	4
Bono	3
Boomer	6
Bothwell	6
Bowdre	4
Brackett	4
Bradley	2
Brashear	2
Brecknock	1
Bremer	3
Brenner	6
Bridport	1
Briggsdale	5
Brinkerton	1
Brinsburg	4
Brittain	4
Brood	6
Brookside	1
Buchanan	1
Buckhannon	1
Bullion	6
Burchard	5
Burgin	2
Burmester	6
Burrell	3
Buse	3
Butler	5
Buxton	1
Byars	2
Byington	2

Soil	Area or areas reported
Cabinet	5, 6
Cacapon	1
Calloway	2, 4
Calvin	1
Canadice	1
Canfield	1, 3
Carbo	2
Cardington	3
Carlisle muck (Ohio) 1/	3
Carlton	6
Catalpa clay loam	4
Cathcart	6
Catron	6
Cavode	1
Celina	3
Canterfield	6
Chance	6
Chandler	4
Chariton	3
Chehalis silty clay loam	6
Chesterfield	2
Chewacla	2, 4
Chilo	3
Chippewa	3
Churchill	6
Clackamas	6
Clareville	4
Clarksburg	1
Clayton	6
Clebit	4
Cloquallum	6
Clyde	3
Costicook	1
Cocalalla	6
Colo	3
Colville	6
Colwood	3
Colyer	2
Commerce	4
Conda	6
Conowingo	1
Conway	4
Cookport	1
Coral	3
Corley	3
Cornutt	6
Coston	6
Cottonwood	4
Cougar	6
Couse	6
Covington	1
Cresco	3
Crete	5
Crofton	3
Crosby	3
Croton	1
Culvers	1
Curran	3
Cut Bank	6
Cuthbert	2, 4
Dalhena	6
Dendridge	2
Danvers	5
Deary	6
Delfina	4
Delp	6
Dennis	4, 5
Denton	4
Deseret	6
Dewart	1
Dixon	4
Dixonville	6
Dulac	2, 4
Dundas	3
Dundee	4
Dunkirk	1
Dunmore	1, 2
Ecleto	4
Ector	4
Edalgo	5
Eddy	4
Eden	2
Edgington	3
Edina	3
Egam	2
Elliott	3
Ellsberry	3
Emmons	6
Enders	2, 4
Erie	1
Ernest	1
Escalante	6
Escondido	6
Esto	2
Fairview	4
Falaya	2, 4
Fallsburg	3

Soil	Area or areas reported
Fay	3
Ferron	6
Fiander	6
Fincastle	3
Fingal	6
Fitchville	3
Flint	2, 4
Florence	5
Flowell	6
Floyd (plastic till variant)	3
Floyd (thin surface variant)	3
Fort Pierce	6
Freesoil	3
Fremont	1
Frio	4
Fulton loam	1, 3
Galvin	6
Gara	3
Garwin	3
Garwin-like	3
Gem	6
Gerald	4
Gilpin	1
Glasgow	6
Glencoe	3
Glenford	3
Glenville	1, 2
Goessel	5
Goldston	4
Gooch	6
Gooding	6
Goose Creek	6
Gordon	6
Gosport	3
Grail	5
Grantsburg	3
Greendale	2, 4
Grenada	2, 4
Gresham	1
Groseclose	2
Grundy	3, 5
Guelph	3
Guernsey	1
Hack	6
Haig	3
Hansel	6
Harley	4
Harpster	3
Harwood	6
Hastings	5
Hatchie	2
Heath	5
Heflin	4
Helena	2
Helmer	6
Herrick	3
Hesson	6
Hockley	4
Hollister silt loam and clay loam	4
Hollywood	2
Hosmer	3
Houlka	2
Huckleberry	6
Hudson	1
Hugo silt loam (Oregon)	6
Humeston	3
2/ Hyrum (Idaho)	6
Idana	5
Inkom	6
Inman	2
Isabella	3
Ivins	6
Izagora	2, 4
Johnsburg	2, 4
Josephine	6
Jura	6
Kasson, thick A_2 variant	3
Kasson-like, nearly level variant	3
Kasson-like silt loam	3
Katy	4
Keene (Coshocton, Ohio Station)	3
Kelso	6
Kendaia	1
Kettleman	6
Kilchus	6
Kings	3
Kirkham	6
Kirvin	2, 4
Kistler	1
Kitsap	6
Klamath	6
Koster	6
Krum	4
Labette	5

1/ Carlisle is in Group A in New York.
2/ Group B in Utah.

Group C (Cont'd)

Soil	Area or areas reported
La Luz	4
Lamonta	6
Lanark	6
Langford	1
Lassen	6
Latah	6
Laughlin	6
Leeper	2
Lehigh	1
Leland	6
Leshe	4
Lewisville	4
1/ Lickdale (W. Va. and Southeast)	1, 2
Lima	1
Lindley	3
Lindside	2, 3, 4
Lisbon	3
Little Timber	5
Litz	1, 2
Livingston	1
Lobdell	3
Lockerby	6
Logan	6
Lorain	3
Lordstown	1
Loring	2, 4
Los Osos	6
Loudonville	3
Lucas	1
Lumberton	3
Luray	3
Luverne	2, 4
Macon	2
Madalin	1
Manassa	6
Manastash	6
Mansker	4
Mantachie	2, 4
Marcus	3
Mardin	1
Marengo	3
Marias	5
Marion	3
Marshan	3
Massie	3
Mayhew	4
Maytown	6
Mazeppa	1
McCornick	6
McKey	6
Medio loamy fine sand	4
Meigs	1, 3
Melbourne	6
Melvin	1, 2, 4
Mench	1
Mercer	2
Meriba	4
Metuchen	1
Mhoon	2, 4
Midnay	5
Millbrook	3
Miller clay loam and silt loam	4
Mimbres	4
Mission	6
Modale	3
Modoc	6
Monongahela	1, 2, 3, 4
Monroeville	3
Montevallo	2
Montgomery	3
Montwel	6
Morley	3
Moro Bay	2, 4
Morris	1
Morrow	6
Munising	3
2/ Muskingum	3
Muskogee	4
Nacimiento	6
Natchitoches	2
Negrett	4
Newark	4
Newell	6
Nez Perce	6
Nimrod	4
Nixa	4
Nolo	1
Norma	6
Norwich	1
Nunn	5
O'Brien	3
Oconee	3
Odessa	1
O'Fallon	3
Okoboji	3
Olivier	4
Olmstead	3
Olympic clay loam	6
Ombey	1
Oquaga	1
Orange	2
Orrville	3
Orwell	1

1/ Group D in Arkansas.
2/ Group B in areas 1, 2, 4.

Soil	Area or areas reported
Panton	1
Papakating	3
Parsippany	1
Passaic	1
Patrick	4
Paul	6
Pearman	2
Pearson	4
Pedernales	4
Penn	1, 2
Pershing	3
Petrolia	3
Pewamo	3
Pharo	6
Pheba	2, 4
Pilot Rock	6
Pinoyer	6
Poganesb	6
Pond Creek	4
Potter	4
Powell	6
Power	6
Prentiss	2, 4
Prior	5
Providence	2, 4
Pullman	4
Rahway	1
Ramona	6
Rankin	3
Ravenna	3
Rayne	1
Readington	1
Reagan	4
Reaville	1
Red Bayou	4
Redmond	6
Regent	5
Renfro	4
Rhinebeck	1
Richfield	5
Richland	4
Richview	3
Ridgecrest	6
Roellen	2
Rolfe	3
Rossmoyne	3
Ruckles	6
Sagemore	6
St. Paul	4
Salkum	6
Sango	2
Santa	6
Santa Lucia	6
Sauvie	6
Savage	5
Savannah	2, 4
Sawyer	4
Saxon	6
Scantic	1
3/ Schapville (Iowa)	3
Schoharie	1
Selah	6
Sequoia	2
Seymour	3
Shawnee	4
Shay	6
Shelby	3, 5
Shelmadine	1
Shelocta	1
Shiloh	3
Shirley	1
Shoals	3
Shooks	3
Shoreham	1
Shubuta	2, 4
Shumaker	6
Sierra	6
Simas	6
Sims	3
Sites	6
Skanee	3
Skumpah	6
Skyberg	3
Sleath	3
Snake	6
Sodus	1
Southwick	6
Sperry	3
Spur	4
Staley	6
Standfield	6
Starks	3
Steinauer	5
Stidham	4
Storden clay	3
Stough	2, 4
Stowbridge	4
Stoy	3
Suffield	1
Summit	4, 5
Sumter	2, 4
Sutherlin	6
4/ Sweeney clay loam (Calif.)	6
Sweet	6

3/ Group D in Illinois.

Soil	Area or areas reported
Tabor	4
Taft	2, 4
Taintor	3
Talladega	2, 4
Tarrant	4
Tate	2, 4
Taylorsflat	6
Taylorsville	6
Teas	1, 2
Thackery	3
Thatcher	6
Thatuna	6
Thomasville	4
Tilsit	1, 2, 3, 4
Tippah	2
Tisch	6
Tishomingo	4
Toledo	1, 3
Tonawanda	1
Trappist	2
Trask	6
Trexler	1
Troy	1
Trumbull	1
Tupelo	2
Tuscumbia	2
Tyler	1, 2, 3, 4
Ucolo	6
Upshur	1, 2
Uveda	6
Uvalde	4
Valier	5
Vallecitos	6
Vance	2
Vandalia	1
Varna	3
Vega	6
Veneta	6
Vergennes	1
Vian	4
Virden	3
Virgil	3
Volusia	1
Waha	6
Waldock	6
Wardwell	6
Warne	2
Watchung	1
Waterville	6
Watogua	4
Webb	4
Webster	3
Webster-Nicollet complex	3
Weikert	1
Weinbach	3
Weld	5
Weller	3
Westland (heavier than loam - Ohio)	3
Westmoreland	1, 2, 3
Weston	2
Westville	3
Westwater	6
Weymouth	4
Wharton	1
Wheelon	6
Whippany	1
Wilkes	2
Windthorst fine sandy loam	4
Wingville	6
Winterset	3
Wolftever	2
Woodrow	6
Woodscross	6
Woodward clay loam	4
Woolper	2
Worsham	2
Wyatt	1
Xenia	3
Zaneis silt loam	4
Zapata	4
Zoar	1

4/ Group D in Oregon.

GROUP D

(Includes mostly clays of high swelling percent; but the group also includes some shallow soils with nearly impermeable subhorizons near the surface)

Soil	Area or areas reported
Abbott	6
Abilene silty clay loam	4
Acme silty clay loam	6
Airport	6
Albaton	3
Alligator	4
Almont	4
Arno	4
Arvada	5
Avonburg	3
Badlands	5
Battlecreek	6
Bear Lake	6
Beaumont	4
Bell	4
Bellingham	6
Benjamin	6
Bergland	3
Bernard	4
Bethel	3
Black Canyon	6
Blago	4
Blencoe	3
Bow	6
Bramwell	6
Brennan fine sandy loam	4
Brewer	4
Brooklyn	3
Bryce	3
Bude	4
Calumet	4
Carroll	4
Catalpa clay	4
Cayucos	6
Chamber	6
Chastain	2, 4
Cherokee	4, 5
Cleveland	4
Chilcott	6
Chipeta	6
Christianburg	6
Churchill	6
Cisne	3
Clarence	3
Clarinda	3
Clatsop	6
Clermont	3
Climax	6
Colbert	2, 4
Coldwater	3
Colp	3
Colt	4
Concord	6
Condit	3
Conover	3
Corydon	1, 4
Courtney	6
Cove	6
Crawford	4
Crockett	4
Crowley	4
Cusick	6
Darwin	3
Day	6
Dayton	6
Defiance	3
Delmita	4
Denmark	6
Denny	3
Denrock	3
DeSoto	3
Diablo	6
Dowellton	2
Drein	6
Drummond	4
Duggins	6
Dunning	2, 4
1/ Duval fine sandy loam	4
Earl	4
Edge	4
Edna	4
Ellsworth	3
Erath	4
Eutaw	4
Fargo	3
Flora	3
Foard	4
Foley (Wynne)	4
Forestdale	4
Fulton silty clay	3
Garfield	6
Garrett	4

1/ Duval fine sandy loam (deep phase) is in Group B.

Soil	Area or areas reported
Gasconade	4
Gay	3
Gee	6
Geiger	4
Ginat	3
Gore	4
Grande Ronde	6
Gunnison	6
Guthrie	4
Harding	6
Hardy	6
Harrisburg	6
Henneke	6
Henry	2, 3, 4
Holcomb	6
Hollister silty clay loam	4
Hortman	4
Houston	2, 4
Huey	3
Hunt	2, 4
Iredell	2
Irving	4
Jacob	3
Kaufman	4
Kent	3
Kipling	4
Kirkland	5
Kopiah	6
Kosmos	6
Lacamas	6
Ladysmith	5
Lafe	4
Lagonda	3
Lahonton	6
Lake Charles	4
Leaf	2, 4
Lebanon	4
Le Flore	4
Lela	4
Letha	6
2/Lickdale (Arkansas)	4
Lightning	4
Lismas	5
Lucien	4
Lufkin	4
Lukin	3
Luton	3
Mahoning	3
Manila	6
McKamie	4
Mellor	6
Menefee	6
Meskell	6
Middle	6
Miguel	4
Miller clay and silty clay loam	4
Milroy	3
Modena	6
Moenkopie	6
Monee	3
Montara	6
Montoya	4
Morse	4
Mullins	4
Myatt	2, 4
Napa (Luton) clay (black alkali phase)	3
Napanee	3
Naturita	6
Navajo	6
Navasota	4
Neola	6
Neosha	5
Nester	3
Nevada	4
Niota	3
Northdale	6
Odin	6
Okaw	3
Oktibbeha	2, 4
Onawa	3
Ontonagon	3
Orson	3
Osage	5
Osceola	3
Page	4
Pariette	6
Parsons	4, 5
Pauchuta	4
Paulding	3

2/ Group C in Southeast.
3/ Group C in Iowa.
4/ Group C in California

Soil	Area or areas reported
Pavant	6
Pawnee	5
Payne	4
Payson	6
Pecos	4
Penjur	4
Perry	4
Persayo	4, 6
Phillips	5
Pickford	3
Pierre	5
Placentia	6
Pledger	4
Portland	4
Post	5
Poteau	4
Pottsville	2
Prescott	4
Racoon	3
Rafael	6
Randall	4
Rantoul	3
Ravalli	5
Reardan	6
Reed	6
Rhoades	5
Richmond	6
Rinard	3
Rittman	3
Roanoke	2, 4
Robertsville	2, 4
Roselms	3
St. Clair	3
Saltair	6
San Antonio	4
San Joaquin	6
San Saba	4
Sawmill silty clay loam (Iowa)	3
3/Schapville (Illinois)	3
Sebring	3
Selkirk	3
Sharkey clay	2, 4
Shavano	6
Slights	6
Snowville	6
Souva	4
Spur	4
Susquehanna	2, 4
4/Sweeney (Oregon)	6
Tabler	4
Talbott	2, 4
Taloka	4
Taylor	3
Telchina	4
Terminal	6
Tillman	4
Timpahute	6
Tortugas	6
Tower	6
Trenton	6
Trinity	4
Trumbull	3
Tunica	4
Una	2, 4
Vaiden	2, 4
Valara	4
Vanderdasson	6
Verhalen	4
Vernon	4
Veyo	6
Virgin River	6
Wabash	3
Wade	5
Wadsworth	3
Wapato	6
Watsonville	6
Waverly	2, 4
Weehadkee	4
Weir	3
West Point	4
Whatcom	6
Whiteson	6
White Store	2
Whitson	3
Wilcox	4
Wilson	4
Windthorst clay loam	4
Wing (valley fill)	4
Winslow	6
Wyemouth	4
Wrightsville	4
Wynoose	3
Yoncalla	6
Zaca	6
Zook	3

USDA-SCS-BELTSVILLE, MD. 1957

Table 3.7-2. Hydrologic soil groups - Puerto Rico

Group A 1/

Ciales
Pandura
Teja
Utuado

Group B 2/

Alonso
Catalina
Cayagua
Descalabrado
Guayama
Jayuya
Juana Diaz
Juncos
Malaya
Mariana
Mucara
Nipe
Plata
Sabana

Group C 3/

Aguilita
Cialitos
Colinas
Daguao
Ensenada
Jacana
Lajas
Naranjito
Picacho
Rio Piedras
Rosario
San German
Soller
Tanama
Yunes

Group D 4/

Los Guineos
Rough Stony

1/ Includes deep sands with very little silt and clay; also deep, rapidly permeable loess.

2/ Mostly sandy soils less deep than A, and loess less deep or less aggregated than A, but the group as a whole has above-average infiltration after thorough wetting.

3/ Comprises shallow soils and soils containing considerable clay and colloid, though less than those of Group D. The group has below-average infiltration after pre-saturation.

4/ Includes mostly clays of high swelling percent; but the group also includes some shallow soils with nearly impermeable subhorizons near the surface.

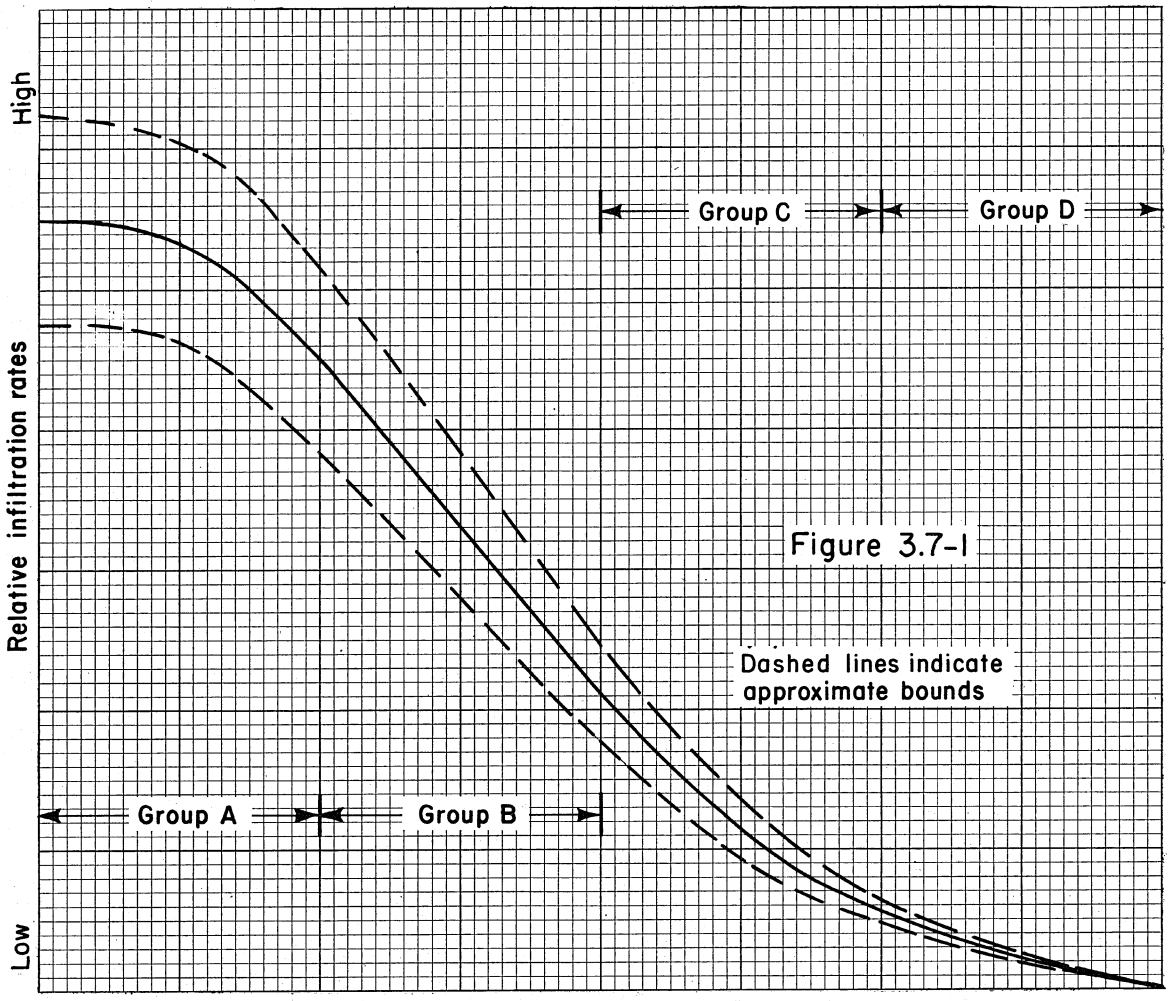
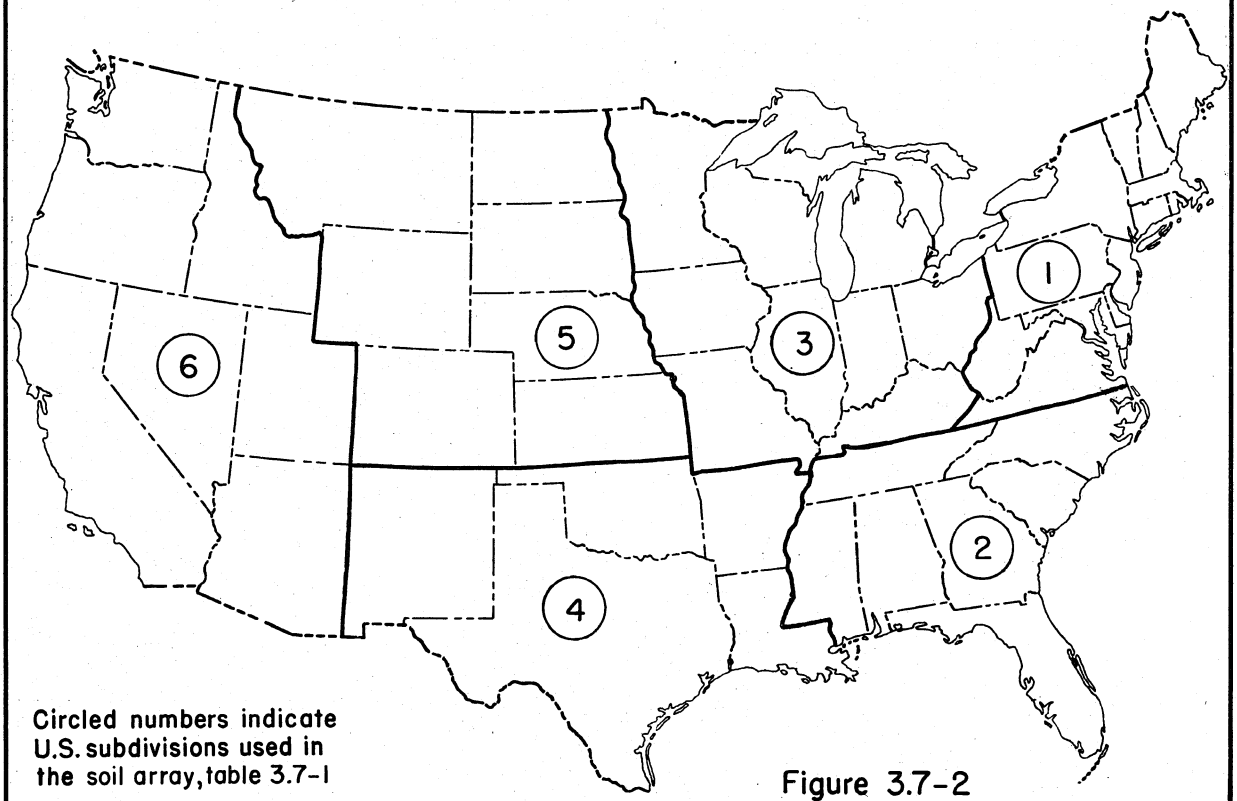


Figure 3.7-1



Circled numbers indicate U.S. subdivisions used in the soil array, table 3.7-1

Figure 3.7-2

3.8 Land use and treatment classes

Purpose. These classes are used in the preparation of hydrologic soil-cover complexes, (Section 3.9), which in turn are used in estimating direct runoff. Types of land use and treatment are classified on a flood runoff-producing basis. The greater the ability of a given land use or treatment to increase total retention, the lower it is on a flood runoff-production scale. Land use or treatment types not described here may be classified by interpolation, as discussed in Section 3.9. Combined classes, such as contour stripcropping, are obtained by weighting, a process described in Section 3.9. See Section 5.1, Definitions, for brief definitions of fallow, row crop, small grain, etc.

Crop rotations. The sequence of crops on a watershed must be evaluated on the basis of its hydrologic effects. Rotations range from poor (or weak) to good (or strong) largely in proportion to the amount of dense vegetation in the rotation. Poor rotations are those in which a row crop or small grain is planted in the same field year after year. A poor rotation may combine row crops, small grains, or fallow, in various ways. Good rotations will contain alfalfa, or other close-seeded legumes or grasses, to improve tilth and increase infiltration. For example, a 2-year rotation of wheat and fallow may be a good rotation for crop production where low annual rainfall is a limiting factor, but hydrologically it is a poor rotation.

Native pasture and range. Three conditions are used, based on hydrologic considerations, not on forage production. Poor pasture or range is heavily grazed, has no mulch, or has plant cover on less than about 50 percent of the area. Fair pasture or range has between about 50 and 75 percent of the area with plant cover and is not heavily grazed. Good pasture or range has more than about 75 percent of the area with plant cover, and is lightly grazed.

Farm woodlots. The classes are based on hydrologic factors, not on timber production. Poor woodlots are heavily grazed and regularly burned in a manner that destroys litter, small trees, and brush. Fair woodlots are grazed but not burned. These woodlots may have some litter, but usually these woods are not protected. Good woodlots are protected from grazing so that litter and shrubs cover the soil.

Commercial forest. The hydrologic condition classes are determined on the basis of depth and quality of litter, humus, and compactness of humus. The Forest Service procedure for determining the classes is given in Section 3.9. In areas where commercial or national forest is a significant part of the watershed cover, the SCS hydrologist is guided by arrangements made in accordance with the joint memorandum between the Forest Service and the Soil Conservation Service.

Miscellaneous. Usually only very small parts of a watershed are in farmsteads, roads, and urban areas. When this is so, the areas may be included with one of the other land use cover types (such as fallow or small grain) in the computation of runoff (see Section 3.9.)

Provision is made in table 3.9-1 (Section 3.9) for farmsteads and roads. These land uses are generalized, since they vary so much. Where it is necessary to work with more detail (as sometimes in a very small watershed, or with superhighway, airport, or urban areas) the impervious areas are considered an individual class with 100 percent runoff, and the remaining land uses are handled as usual (see Section 3.9).

Straight-row farming. This class includes up-and-down and cross-slope farming in straight rows. In areas of 1 or 2 percent slopes, farming cross-slope in straight rows is almost the same as contour farming. Where the proportion of cross-slope farming is believed to be significant, it may be classed halfway between straight-row and contour farming in the table 3.9-1.

Contouring. Contour furrows used with small grains and legumes are made while planting, are generally small, and tend to disappear due to climatic action. Contour furrows, and beds on the contour, as used with row crops are generally large. They may be made in planting and later reduced in size by cultivation, or they may be insignificant after planting and become large from cultivation. Average conditions are used in table 3.9-1.

Surface runoff reductions due to contour farming are greater as land slopes decrease. The curve numbers for contouring shown in table 3.9-1 were obtained using data from experimental watersheds having 3 percent to 8 percent slopes.

Contour furrows in pasture or range land are usually of the permanent type. Their dimensions and spacing generally vary with climate and topography. Table 3.9-1 considers average conditions in the Great Plains.

Terracing. Terraces may be graded, open-end level, or closed-end level. The effects of graded and open-end level terraces are considered in table 3.9-1, and the effects of both contouring and the grass waterway outlets are included.

Closed-end level terraces should be handled like contour furrows in Section 3.12.

Determination of land use and treatment classes. The hydrologist should determine the classes in consultation with the economist, Work Unit or Area Conservationist, or other designated person. The determinations should be made on a subwatershed basis wherever possible, and should be made concurrently for both present and expected future watershed conditions.

3.9 Hydrologic soil-cover complexes

Purpose. Table 3.9-1 combines soil groups and land use and treatment classes into hydrologic soil-cover complexes. The numbers show the relative value of the complexes as direct runoff-producers (see Section 3.10). The higher the number, the greater the amount of direct runoff to be expected from a storm.

Table 3.9-1. The table was prepared using data from gaged watersheds with known soils and cover. Storm rainfall was plotted versus direct runoff for annual floods and other significant floods. The curve of figure 3.10-1 best fitting the plotted points was determined, and its number was used to obtain an average curve number (II-curve) for table 3.9-1. Related numbers for above-average (III-curve) and below-average (I-curve) points were similarly developed.

Curve numbers for several soil-cover complexes were estimated or computed from relations developed in the work, since actual hydrologic data were not available for all given complexes.

The curve numbers on table 3.9-1 are subject to revision as additional hydrologic data become available.

Table 3.9-1. Runoff curve numbers for hydrologic soil-cover complexes.

For watershed condition II, and $I_a = 0.2(S)$

Land use or cover	Treatment or practice	Hydrologic condition	Hydrologic soil group A	B	C	D
Fallow	Straight row	- - -	77	86	91	94
Row crops	"	Poor	72	81	88	91
	"	Good	67	78	85	89
	Contoured	Poor	70	79	84	88
	"	Good	65	75	82	86
	"and terraced	Poor	66	74	80	82
	" " "	Good	62	71	78	81
Small grain	Straight row	Poor	65	76	84	88
		Good	63	75	83	87
	Contoured	Poor	63	74	82	85
		Good	61	73	81	84
	"and terraced	Poor	61	72	79	82
		Good	59	70	78	81
Close-seeded legumes 1/ or rotation meadow	Straight row	Poor	66	77	85	89
	" "	Good	58	72	81	85
	Contoured	Poor	64	75	83	85
	"	Good	55	69	78	83
	"and terraced	Poor	63	73	80	83
	"and terraced	Good	51	67	76	80
Pasture or range		Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
	Contoured	Poor	47	67	81	88
	"	Fair	25	59	75	83
	"	Good	6	35	70	79
Meadow (permanent)		Good	30	58	71	78
Woods (farm woodlots)		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	25	55	70	77
Farmsteads		---	59	74	82	86
Roads (dirt) 2/		---	72	82	87	89
(hard surface) 2/		---	74	84	90	92

1/ Close-drilled or broadcast.
2/ Including right-of-way.

Forest Service procedure. Table 3.9-2 shows curve numbers developed by the Forest Service. These numbers are used in hydrologic evaluations of commercial or national forest. Figure 3.9-1 gives a nomograph by which the hydrologic condition class of forest is estimated. The following definitions are used with figure 3.9-1:

Litter. This includes the fermentation layer. Consists of undecomposed dead vegetal material including grasses, forbs, leaves, needles, twigs, bark, etc. It varies in depth with season, being thinnest in the late winter. The fermentation or F layer consists of partly decomposed litter, with the origin still recognizable.

Humus. Includes either the H layer of mor (also known as duff or raw humus), or the A layer (otherwise called mull), in which the organic matter is incorporated in the mineral soil.

When the condition class is obtained on figure 3.9-1, it is used only with part I of table 3.9-2. Part II of that table gives tentative special values prepared by the Forest Service for certain forest-range areas in the western United States.

Determination of curve numbers for mixed areas. Table 3.9-3 shows the process by which a weighted soil-cover complex number is obtained for areas having several soil-cover complexes. The example area is in a B soils group. The weighted number can be more easily obtained by accumulative multiplication on a calculator.

Table 3.9-2. Runoff curve numbers for hydrologic soil-cover complexes.

I. Commercial or national forest, for watershed condition II, and $I_a = 0.2\ S$

Hydrologic condition class		Hydrologic soil group A	B	C	D
I	Poorest	56	75	86	91
II	Poor	46	68	78	84
III	Medium	36	60	70	76
IV	Good	26	52	62	69
V	Best	15	44	54	61

II. Forest-range areas in western United States, for watershed condition III, and $I_a = 0.2\ S$

Cover	Condition	Soil Groups A	B	C	D
Herbaceous	Poor		90	94	97
	Fair		84	92	95
	Good		77	86	93
Sagebrush	Poor		81	90	
	Fair		66	83	
	Good		55	66	
Oak-Aspen	Poor		80	86	
	Fair		60	73	
	Good		50	60	
Juniper	Poor		87	93	
	Fair		73	85	
	Good		60	77	

(Note that this table is for condition III.)

Data supplied by Forest Service, June 1956.

Table 3.9-3. Sample computation, weighting of hydrologic soil-cover complex numbers.

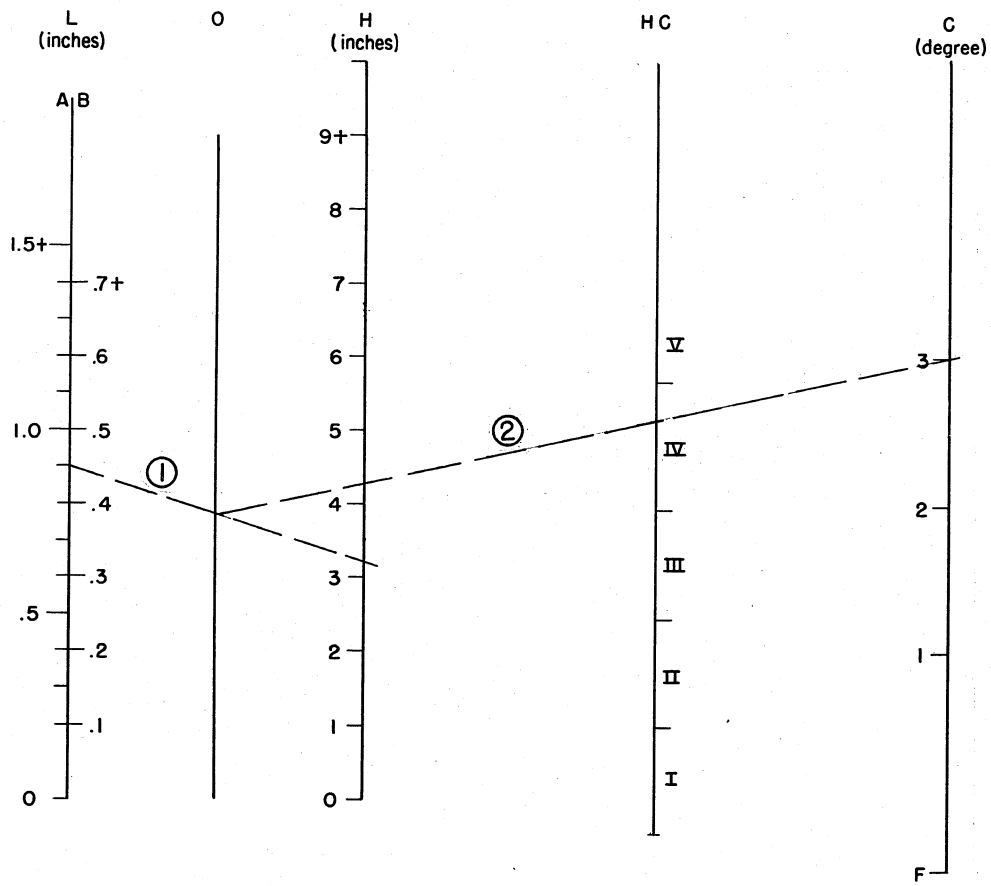
Complex	Curve number	Percent of area	Number times percent
Row crop, straight row, good rotation	78	56.2	4384
Legumes, contoured, good rotation	69	37.5	2588
Meadow, permanent	58	6.3	365
	Totals	100	7337

$$\text{Weighted number} = \frac{7337}{100} = 73.37$$

Round off to 73.

426963 O - 57 - 6

CHART FOR DETERMINING HYDROLOGIC CONDITION OF FOREST AND WOODLAND



LEGEND

L — Litter depth

A — Litter before compaction

B — Litter after compaction

H — Humus depth

HC — Hydrologic condition class

C — Compactness

- F — Frozen, refers to "concrete" frost.
- 1 — Compact, tight
- 2 — Moderately compact
- 3 — Loose, not compact, friable.

O — Turning axis

EXAMPLE

Given:

L = 0.9" (before compaction)

H = 3.2"

C = 3.0

Solution:

1. Draw line ① connecting L = 0.9 and H = 3.2.
2. Draw line ② connecting intersection of line ① and O scale, and C = 3.
3. On HC scale, read HC = IV

REFERENCE

Morey, H.F., Alignment chart: Forest Service, April 21, 1955.

Figure 3.9-1

3.10 Estimation of direct runoff from rainfall

Purpose. A method for determining the relation of rainfall to runoff for a given watershed condition is needed for evaluating the effects of a proposed project that contains land treatment measures. Such a method is also required for use on watersheds with stream gaging stations, since the flow records can show only the effects of the present watershed condition, and an estimate of the effects of the future condition is also needed in the evaluation of a proposed project.

General. The method uses three variables in estimating runoff: Rainfall, antecedent moisture condition, and the hydrologic soil-cover complex. For previous Service work with similar methods, see the following:

Andrews, R. G. The Use of Relative Infiltration Indices in Computing Runoff. Fort Worth, Texas. Processed.

Mockus, V. Estimation of Total (and Peak Rates of) Surface Runoff for Individual Storms. Lincoln, Nebr. Mimeographed.

Sharp, A. L., Holtan, H. N., and Musgrave, G. W. Infiltration in Relation to Runoff on Small Watersheds. SCS-TP-81. Washington, D. C.

Runoff equation. The curves of figure 3.10-1 are obtained using the equation:

$$Q = \frac{(P - 0.2\ S)^2}{(P + 0.8\ S)} \qquad (1)$$

where Q = direct runoff, in inches;

P = storm rainfall, in inches; and

S = maximum potential difference between P and Q, in inches, at time of storm's beginning.

Equation 1 is derived by starting with the proportion:

$$\frac{P - Q}{S} = \frac{Q}{P} \tag{2}$$

where

(P-Q)/S is visualized as the ratio of actual to potential MAXIMUM difference between P and Q, and Q/P is visualized as the ratio of actual to potential MAXIMUM runoff.

Solving for Q gives:

$$Q = \frac{P^2}{P + S} \tag{3}$$

Equation 3 is useful under conditions where there is a possibility of runoff whenever there is rainfall. For the condition that Q = 0 at a value of P greater than zero, use of an initial abstraction (I_a) is required (see the diagram on figure 3.10-1). With the condition that I_a cannot be greater than P, equation 2 then becomes:

$$\frac{(P - I_a) - Q}{S} = \frac{Q}{(P - I_a)} \tag{4}$$

and solving for Q gives:

$$Q = \frac{(P - I_a)^2}{(P - I_a + S)} \tag{5}$$

Since S includes I_a, an empirical relation between the variables can be developed to simplify equation 5. Data from watersheds in various parts of the country give:

$$I_a = 0.2\ S \tag{6}$$

Substituting (0.2 S) for I_a in equation 5 gives equation 1.

Equation 5 can be further expanded to recognize other given conditions or factors. However, such expansions in themselves do not mean greater accuracy in runoff estimating, since this would require that the terms expressing additional factors be of a high order of accuracy.

<u>Significance of S</u>. Plottings of direct runoff (Q) versus storm rainfall (P) on natural watersheds show that Q approaches P as P continues to increase in the storm. The same data show that (P - Q) approaches a constant as P continues to increase. The quantities can be shown together as in equation 2, above, and it is apparent that the constant S is the maximum difference (P - Q) that could occur for the given storm and watershed conditions. The proportion can be made more complex (as discussed above) but not all the additional factors can be related to S.

The variable S is therefore a maximum potential (P - Q). During a storm, the actual (P - Q) that occurs is limited by either soil-water storage or an infiltration rate as P increases. The maximum potential (P - Q) or S, therefore is dependent on soil-water storage and the infiltration rates of a watershed.

The value of S can be obtained for any gaged watershed by plotting P versus Q. For ungaged watersheds, S values must be estimated from table 3.9-1.

Significance of I_a. The insert on figure 3.10-1 shows that I_a is equal to the rainfall that occurs before runoff starts. Physically I_a consists principally of interception, infiltration, and surface storage. Equation 6, which relates I_a to S, is based on data from large and small watersheds in various parts of the country. Further refinement of equation 6 is not recommended, since the data needed to break I_a into components of interception, infiltration, and surface storage are seldom available on a watershed basis. For the same reason, adjustment of the coefficient (0.2) in equation 6 is not recommended.

System of curve numbering. For convenience in interpolation, the curves of figure 3.10-1 are numbered from 100 to zero. The numbers are related to S as follows:

$$\text{Curve number} = \frac{1000}{10 + S} \qquad (7)$$

A curve for the case $I_a = 0$, (equation 3), is displaced to the right for the case $I_a = 0.2\,S$, (equation 1), by the amount of 0.2 S. Therefore, the curve numbers given in table 3.9-1 should be used only with figure 3.10-1 or with equation 1.

Antecedent conditions. The numbers on table 3.9-1 are for the average watershed condition (Condition II) before floods from storm rainfall occur. Section 3.4, Antecedent rainfall, and Section 3.5, Antecedent flood conditions, give more details on the determination of antecedent conditions.

Curve numbers for antecedent conditions I and III are obtained from table 3.10-1. For example, the computation given in table 3.9-2 gives a condition II curve number of 73. By interpolation in columns 2 and 3 of table 3.10-1, the number for condition I is 55, and 89 for condition III.

Process of estimation. More than one process can be used, but the one most generally applicable is given. On a subwatershed basis, determine:

1. Storm rainfall volumes for which direct runoff estimates are needed (Section 3.4);

2. Antecedent condition for each storm (Section 3.4);

3. Hydrologic soil-cover complex number for watershed antecedent condition II (Section 3.9);

4. Complex numbers for antecedent conditions I and III, from table 3.10-1; and

5. Direct runoff, by entering figure 3.10-1 with the storm rainfall and reading the direct runoff at the appropriate curve for the complex and antecedent condition.

Note that this is on a subwatershed basis. Theoretically, a better process is one in which runoff is first estimated for each soil-cover complex area and the runoff for the subwatershed obtained by a weighted average. However, the added calculations will generally not improve significantly on estimates made using the first process, except when a large proportion of the watershed is impervious (has 100% runoff).

Table 3.10-2 shows the data and the estimates for a small watershed near Waco, Texas. (Source of data: "Summary of rainfall and runoff, 1940-1951, at Blacklands Experimental Watershed, Waco, Texas", May 1952, USDA Soil Conservation Service, in cooperation with Texas Agri. Expt. Station.) The soil is Houston black clay (D group) and the cover (average for years 1940 and 1951) was:

	Percent
Row crop, straight row, poor rotation	58
Small grain, straight row, poor rotation	25
Pasture or hay, fair condition	15
Farmsteads, etc.	2

The average slope of the watershed is 2.2 percent, and possibly where farmed cross-slope the area could have been considered contoured. In this case it was considered straight row.

Table 3.10-1. Conversions and constants.

For the case $I_a = 0.2\ S$

1	2	3	4	5
Curve number for Condition II	Corresponding curve numbers for: Condition I	Condition III	S values*	Curve* originates where P =
100	100	100	0	0
95	87	99	.526	.10
90	78	98	1.11	.22
85	70	97	1.76	.35
80	63	94	2.50	.50
75	57	91	3.33	.67
70	51	87	4.29	.86
65	45	83	5.38	1.08
60	40	79	6.67	1.33
55	35	75	8.18	1.64
50	31	70	10.00	2.00
45	27	65	12.2	2.44
40	23	60	15.0	3.00
35	19	55	18.6	3.72
30	15	50	23.3	4.66
25	12	45	30.0	6.00
20	9	39	40.0	8.00
15	7	33	56.7	11.34
10	4	26	90.0	18.00
5	2	17	190.0	38.00
0	0	0	infinity	infinity

*For curve number in column 1.

However, the cover given above was used as shown to give the following hydrologic soil-cover complex numbers:

Condition II = 89 (See Section 3.9)

Condition I = 77 (See Table 3.10-1)

Condition III = 98 (See Table 3.10-1)

Note the following about the sample computations in table 3.10-2:

a. For a multiple-day storm, the estimate is made by days.

b. Antecedent moisture condition (AMC) was estimated using table 3.4-1.

c. The second and succeeding days of a multiple-day storm always fall into the condition III class.

d. Though more detailed rainfall records are available, it was assumed here that those data were available only on a daily basis--the usual field condition.

e. The estimates are made to the nearest tenth of an inch. Generally, this is all the refinement justified in the estimate.

f. The selected storms in this sample are those in which the annual flood peak occurred.

The computed results in this case can be compared with actual runoff (last column). The comparison is generally good except for one point (Sept. 7-9, 1942). The differences between actual and estimated runoff can be attributed to any one of many factors. Where actual runoff is known, such a difference can be studied in detail. In the normal case, where estimates are made for ungaged watersheds, the estimates must be accepted at face value.

The purpose of estimating runoff volumes for a gaged watershed is either to demonstrate a process or to show that since the process works well on a gaged area (by a comparison of actual vs. computed) it is very likely that the process works equally well for ungaged areas.

Table 3.10-2. Sample computation. Direct runoff estimates, annual floods. Watershed W-1, Waco, Texas.

Year	Month	Day	Storm rainfall	AMC 1/	Est. runoff by days	Est. runoff total	Total actual runoff 2/
1940	Nov.	22	4.74	I	2.4		
		23	2.20	III	2.0		
		24	2.03	III	1.9		
		25	.38	III	.3	6.6	6.00
1941	June	10	2.39	III	2.2	2.2	2.04
1942	Sept.	7	3.89	I	1.8		
		8	3.36	III	3.2		
		9	.78	III	.6	5.6	2.83
1943	June	5	1.58	I	.2	.2	.51
1944	April	29	3.63	I	1.5		
		30	2.64	III	2.4		
	May	1	6.37	III	6.2		
		2	1.10	III	.9	11.0	10.76
1945	March	2	.77	I	.0		
		3	2.50	III	2.3	2.3	2.38
1946	May	12	2.90	III	2.7		
		13	.95	III	.8	3.5	2.95
1947	March	18	1.74	I	.3	.3	.85
1948	April	25	3.10	I	1.2	1.2	1.17
1949	July	4	2.86	I	1.0	1.0	1.07
1950	Feb.	12	1.94	III	1.7	1.7	1.09
1951	June	16	1.64	II	.7	.7	.19

1/ Antecedent moisture condition, using Table 3.4-1.
2/ For comparative purposes. See text.

Special watershed conditions. The process described above generalizes the effect of cover, practices, season, and other factors influencing runoff. For example, the effect of straight-row, poor-rotation corn is assumed to be uniform throughout the year.

In any case where more specific watershed conditions are to be evaluated, the runoff estimates can be made in the same manner, but using hydrologic soil-cover complex numbers that are obtained by interpolation or adjustment on table 3.9-1. This table is in a sense an index table of plant density, root density, time of planting, etc. For example, a watershed with a B soil and with a yearly average of straight-row, poor-rotation corn (maize) could be classed in Iowa as a fallow watershed in May; as a straight-row, good-rotation row crop in July; and as the equivalent of fair pasture in October, with the II curve numbers being 86 in May, 78 in July, and 69 in October.

HYDROLOGY: SOLUTION OF RUNOFF EQUATION $Q = \frac{(P-0.2S)^2}{P+0.8S}$

P = 0 to 12 inches
Q = 0 to 8 inches

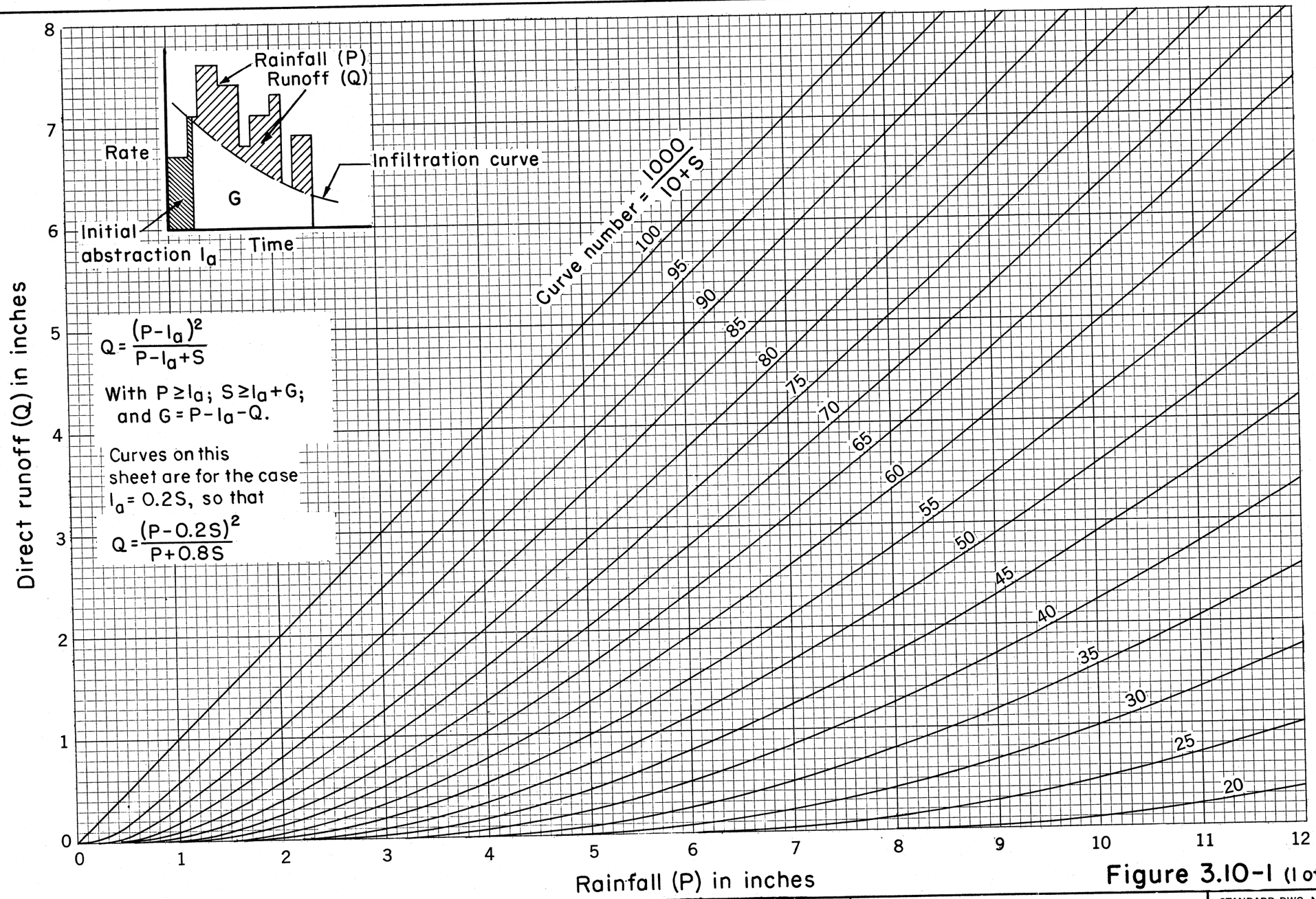


Figure 3.10-1 (1 of 2)

REFERENCE

Mockus, Victor; Estimating direct runoff amounts from storm rainfall: Central Technical Unit, October 1955.

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - CENTRAL TECHNICAL UNIT

STANDARD DWG. NO.
ES-1001
SHEET 1 OF 2
DATE 6-29-56

426963 O-57 (Face p. 90)

HYDROLOGY: SOLUTION OF RUNOFF EQUATION $Q = \frac{(P-0.2S)^2}{P+0.8S}$

P = 8 to 40 inches
Q = 0. to 40 inches

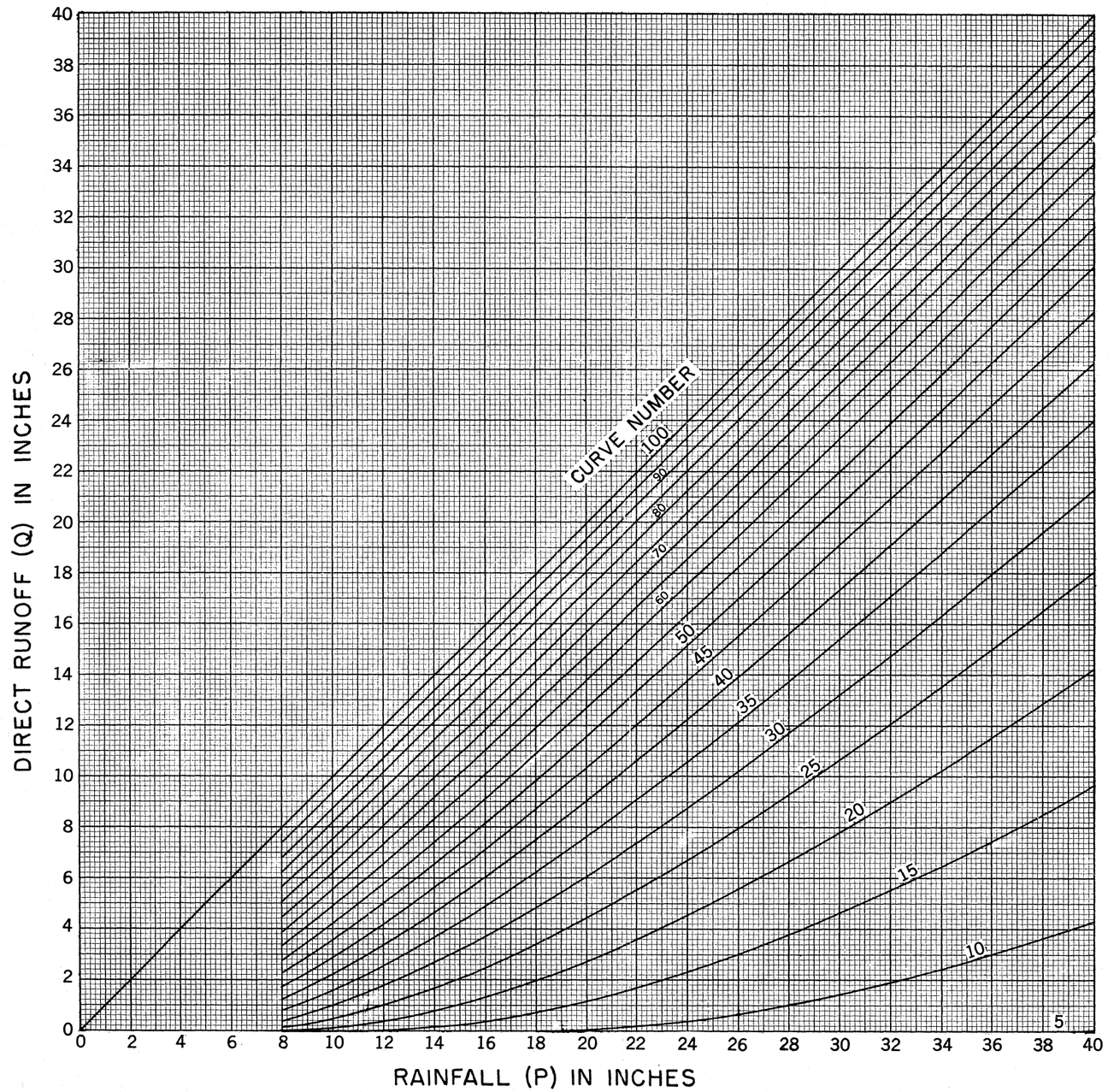


Figure 3.10-1
(2 of 2)

REFERENCE

Mockus, Victor; Estimating direct runoff amounts from storm rainfall: Central Technical Unit, October 1955.

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

ENGINEERING DIVISION - CENTRAL TECHNICAL UNIT

STANDARD DWG. NO.
ES-1001
SHEET 2 OF 2
DATE 6-27-56

3.11 Estimation of runoff from snowmelt

Purpose. This section gives methods for estimating snowmelt runoff volumes for flood damage evaluations. Methods of snowmelt forecasting, for irrigation and similar purposes, are described in the Snow Survey Handbook of the Service.

Details of the thermodynamics of snowmelt are omitted from this section because of their limited value in the methods presented here. Some standard references are:

> Clyde, George D. - Snow-melting characteristics. Technical Bulletin 231, August 1931. Utah Agricultural Experiment Station, Logan, Utah.
>
> Light, Phillip - Analysis of high rates of snow-melting. Pages 195-205, Transactions of the American Geophysical Union, 1941.
>
> Wilson, W. T. An outline of the thermodynamics of snowmelt. Pages 182-195, Transactions of the American Geophysical Union, 1941.

Significance of snowmelt floods. Bankfull capacities in csm are normally greater for small watersheds than for large ones. Since snowmelt rates are relatively low in csm there may be flooding on large watersheds when streams on small watersheds are flowing less than bankfull.

The hydrologist acquainted with an area will know the relative importance of snowmelt as a source of flooding in that area. In doubtful cases the data normally gathered by interview for a historical flood series will usually define the character of flood flows. In other instances, the runoff records will show how important snowmelt flooding is. It is seldom necessary to make detailed hydrologic investigations into the matter.

Regional analysis. This method is one of the most useful for snowmelt floods. See Section 2.8 for details of the method.

Degree-day method, ungaged watersheds. This method is widely used because of its adaptability to usual data conditions. Similar methods going into more detail are available but seldom applicable because of lack of required data.

The degree-day method uses the equation:

$$M = K\,D \qquad (1)$$

where M = the watershed snowmelt in inches per day

K = a constant that varies with watershed and climatic conditions

D = the number of degree-days for a given day.

A degree-day is a day with an average temperature one degree above 32° F. Maximum and minimum temperatures, as found in "Climatological Data", are averaged to get the daily average temperature. A day with an average of 40° F. gives eight degree-days; with an average of 51° F., nineteen degree-days. The general form of the method is given below. A working arrangement of the data is shown on table 3.11-1. In most cases the table can be condensed. The steps in the method are:

1. Using precipitation stations or snow survey data, show either (a) the total available water equivalent at the beginning of the melt season (table 3.11-1) or (b) the precipitation and the water equivalent by days (table 3.11-2). The first procedure is used where there is generally only one melt period per year; the second, where melt periods occur intermittently through the winter and spring. Water equivalent is the depth of water, in inches, that results from melting a given depth of snow, and it is dependent on both depth and density of snow. Snow surveys give field determinations of water equivalents. Where such surveys are not made, it is customary to use one-tenth of the snow depth as the depth of water equivalent.

2. For temperature stations in the watershed, tabulate average temperatures for the melt periods. (Note: maximum and minimum values as given in "Climatological Data" can be averaged mentally to avoid tabulation of averages below 33° F.)

3. Adjust the average temperatures to the average watershed elevation, using the method given below in Adjustment of temperatures for altitude. This step is omitted when elevation data are crude or otherwise unreliable.

4. Compute the watershed average daily temperatures by averaging the station averages (adjusted for altitude, if desirable).

5. Subtract 32° F. from each watershed average daily temperature to get the degree-days per day.

6. Use equation 1 to get an estimate of the potential snowmelt for each day. See K factors below for selection of K.

7. Where the daily potential is not greater than the water equivalent remaining on the watershed, it is shown as an estimate of snowmelt.

Once the estimates of snowmelt are obtained, they are used to obtain hydrographs as described in Section 3.16.

Some hydrologists suggest that the effects of infiltration be subtracted from the estimated snowmelt. However, the K factors as generally developed already include the effects of infiltration. The effects of measures such as contour furrows are obtained as described in Section 3.12. The effects of reservoirs, levees, etc. are obtained as usual.

Refinements in the degree-day method are best made by first improving the accuracy of determinations of snow depth and areal distribution on the watershed. When these are known within small limits of error, then water equivalents should be refined, since the 1/10 ratio is a rough approximation. Refinements in K factors should come last.

Table 3.11-1. Estimation of snowmelt by degree-day method. One melt period.

Dates	Watershed average temperature, °F. 1/	Degree-days	Estimated snowmelt 2/	Total available water equivalent
			Inches	Inches
April 5	32	0	0	4.50
6	35	3	.18	4.32
7	34	2	.12	4.20
8	36	4	.24	3.96
9	48	16	.96	3.00
10	43	11	.66	2.34
etc.	etc.	etc.	etc.	etc.

1/ Average of two stations; adjusted for altitude.

2/ Using K = 0.06 in equation 1.

Table 3.11-2. Estimation of snowmelt by degree-day method. Intermittent melt period.

Dates	Precipitation	Water equivalent 1/	Degree-days	Snowmelt: Potential 2/	Snowmelt: Estimated	Remaining water equivalent
	Inches	Inches		Inches	Inches	Inches
Nov. 3	0.85	0.08				0.08
Nov. 4-18						.08
Nov. 19			5	0.30	0.08	0
Nov. 20-29						0
Nov. 30	3.80	.38				.38
Dec. 1-24						.38
Dec. 25	4.15	.42				.80
Dec. 26-Jan. 18						.80
Jan. 19	.52	.05				.85
Jan. 20-Feb. 2						.85
Feb. 3-20	6.92	.69				1.54
Feb. 21-Mar. 14						1.54
Mar. 15	14.24	1.42				2.96
Mar. 16-28						2.96
Mar. 29			3	.18	.18	2.78
Mar. 30			11	.66	.66	2.12
Mar. 31			22	1.32	1.32	.80
Apr. 1-9						.80
Apr. 10			7	.42	.42	.38
Apr. 11			32	1.92	.38	0

1/ One-tenth of snow depth.
2/ Using K = 0.06 in equation 1.

Degree-day method, gaged watersheds. The degree-day method has a very limited use, if any at all, for flood evaluations on gaged watersheds. When gaging station data are available, those data should be used to estimate flood peaks and volumes on other portions of the watershed.

Adjustment of temperatures for altitude. In general, air temperatures decrease about 3° to 5° for every 1,000 feet of rise in altitude. Other factors influence this "lapse rate", so that refinements are not justified, and an average decrease of 4° F. per 1,000 feet rise should be used.

Example. A watershed with an average elevation of 4,600 feet had temperature station readings of 38° F. at a 5600-foot elevation, and 48° F. at a 3000-foot elevation. The average temperature for the watershed is then:

$$(38) - \frac{4}{1000}(4600 - 5600) = 42.0$$

$$(48) - \frac{4}{1000}(4600 - 3000) = 41.6$$

Sum:	83.6
Average:	41.8
Round off to:	42

While further refinements, such as weighting, can be made, they are seldom justified.

K factors. The constant K in equation 1 is known to vary not only from watershed to watershed, but also from day to day on a given watershed. It is seldom possible to do more than make a broad estimate of K. An average value of 0.06 can be used. The following table may be of assistance in special cases:

Table 3.11-3. K factors.

Condition	K
Extremely low runoff potential	0.02
Average heavily-forested areas; north-facing slopes of open country	.04 - .06 1/
Average	.06
South-facing slopes of forested areas; average open country	.06 - .08 1/
Extremely high runoff potential	.30

1/ Recommended by A. L. Sharp

Concordant flow method. The method of Section 2.7 can be simplified to estimate both peaks and volumes of snowmelt runoff, when at least one streamflow record is available. The method is very similar to the Regional analysis method mentioned above and in Section 2.8.

The volume of snowmelt for an ungaged subwatershed is the same as that for the gaged watershed, assuming equal coverage of snow over both areas. Where it is possible to estimate the amounts or degrees of snow coverage, the snowmelt volumes in inches may be taken as directly proportional to snow depth or degree of coverage. For example, if there is 3.2" snowmelt runoff from a gaged watershed of 82 square miles with 76 percent of the watershed having snow cover, then a subwatershed of 12 square miles and 100 percent snow cover will have an estimated runoff of:

$$3.2 \frac{(100)}{(76)} = 4.2 \text{ inches.}$$

Note that area in square miles is not used in the computation unless acre-feet are needed. If instead of the percents the gaged watershed is known to have an average of 16.2 inches of snow-depth and the ungaged subwatershed 20.4 inches, then the runoff for the subwatershed is:

$$3.2 \frac{(20.4)}{(16.2)} = 4.0 \text{ inches.}$$

Other factors can be brought in, but here again refinement is not justified.

Peaks of snowmelt runoff can be obtained as described in Section 3.16.

Other methods. Where intensive study has been or can be made of a watershed, more detailed and more accurate methods of estimating snowmelt runoff can be used. See figure 2.9-2 for a flow chart of a detailed method used by the Forest Service, and Section 2.9 for the reference to the technical journal where working details are given.

426963 O - 57 - 7

3.12 Hydrologic effects of land use and treatment

The effects that are discussed here are (a) changes in volumes of direct runoff and (b) changes in lag, which affect peak rates of direct runoff. Other effects, such as increases in base flow, are to be covered in NEH-4.

Volume effects. Land use and treatment measures reduce the volume of direct runoff during individual storms by either (1) increasing infiltration rates, or (2) increasing surface storage, or both. Other factors influencing runoff volume are usually of minor importance. Interception increases, for instance, are appreciable only under certain climatic and vegetative conditions and generally need not be considered in Service watershed studies.

The unit hydrograph principle states that with other things constant, the peak rate of flow varies directly with the volume of flow. This principle is the basis for proportionate reductions in peaks when volumes are reduced (see Section 3.16). Figure 3.12-1 shows a typical peak vs. volume relation. The straight line is drawn so that some points are on the line, if possible, with half of the remaining points on one side of the line and the other half on the other side. Drawing a curve is not justified, since other important relations must be accounted for (see Section 3.16) if greater accuracy is required. The figure shows that a 30 percent reduction in volume gives a 30 percent reduction in the peak rate, and so on.

Table 3.12-1 shows the principal effects of land use and treatment measures on direct runoff. The degree of effect of any single measure generally depends on the quantity that can be installed. Contour furrows, however, can be made to have a small or large effect by changing the dimensions of the furrows. The effect of a land use change depends on the change in cover. A change from spring oats to spring wheat would ordinarily be hardly noticeable, while a change from oats to a permanent meadow could have a large effect. Graded terraces with grass outlets to some extent will increase both over-all infiltration and over-all storage. These effects are also confused with a lag effect (see below). It should be noted that lime and fertilizers, by increasing plant or root density, can indirectly reduce direct runoff volumes.

Lag effects. Lag, as used here, means the delay between the production of direct runoff on upland areas and its appearance at a given cross-section in a stream channel. Another discussion of lag is given in Section 3.15.

Table 3.12-1. Principal effects of land use and treatment measures on direct runoff.

Measure	Reduction in direct runoff volume is due to:	
	Increasing infiltration rates 1/	Increasing surface storage
1. Land use change that increases plant or root density. 2/	X	
2. Increasing mulch or litter	X	
3. Contouring		X
4. Contour furrowing		X
5. Level terracing		X
6. Graded terracing		X

1/ Assuming soils not frozen

2/ Example: Row crop to grass for hay. Poor pasture to good pasture.

Land use and treatment measures can produce lag effects by (1) increasing infiltration (reducing surface runoff) and causing the increased infiltration to appear some time later as subsurface flow, or (2) by causing a delay in the arrival of surface runoff by increasing the distance or reducing the velocity of flow.

Either effect is best studied by the methods of Sections 3.15 and 3.16. Table 3.12-2 shows the relative effects of land use and treatment measures on the two types of lag. The subdivisions of small and large watersheds do not depend solely on size in square miles. The methods of Sections 3.15 and 3.16 are necessary in quantitative studies of lag.

Determination of volume effects. The same procedure used in determining the present hydrologic conditions of a watershed is used to estimate future hydrologic conditions. The future effects of land use and treatment changes can be estimated with relatively little additional work. Assuming that present conditions have been studied, the steps are:

1. Determine the hydrologic soil-cover complex number, condition II, for future land use and treatment conditions. (Sections 3.7, 3.8, and 3.9).

2. Obtain complex numbers for conditions I and III (Table 3.10-1).

3. Prepare a working table similar to table 3.12-3.

4. Plot the corresponding present and future values as shown on figure 3.12-2. For example, plot 0.23 vs. 0.02; 0.60 vs. 0.18; and 1.10 vs. 0.43, and draw in the curve for condition I. Similarly for the other conditions.

5. Enter figure 3.12-2 with the present volume and condition for a storm or flood in the evaluation series and find future volume on the appropriate curve.

Determination of lag effects. Increased infiltration appearing some time later as subsurface flow is seldom easy to evaluate quantitatively. Fortunately, however, in most flood prevention surveys the changes in the hydrograph due to this lag effect can generally be neglected. Where it cannot, special studies are needed to determine the source areas (which may vary with infiltrated volumes) and watershed retention. The techniques for these special studies have not been fully developed, however, and the results are likely to be controversial.

Table 3.12-2. Relative effects of land use and treatment measures on types of lag.

Measure	Effect on subsurface flow 1/		Effect of increasing surface flow distance or decreasing velocity	
	Small watersheds	Large watersheds	Small watersheds	Large watersheds
1. Land use changes that increase plant or root density. 2/	Can be large.	Can be large.	Not usually considered.	
2. Increasing mulch or litter.	Can be large.	Can be large.	Not usually considered.	
3. Contouring	Can be large.	Usually negligible.	Can be large.	Negligible.
4. Contour furrowing	Can be large.	Can be large.	Not usually considered.	
5. Level terracing	Can be large	Can be large	Not usually considered.	
6. Graded terracing	Usually negligible.	Usually negligible.	Can be large.	Negligible.

1/ Assuming soils not frozen.

2/ Examples: Row crop to grass; poor pasture to good pasture.

Table 3.12-3. Sample working table. Estimation of effects of future land use and treatment on direct runoff volumes.

Selected values of "P"	Direct runoff, in inches, for selected values of "P", from figure 3.10-1					
	Condition I		Condition II		Condition III	
	Present	Future	Present	Future	Present	Future
0.5	0	0	0	0	0.08	0
1	0	0	.02	0	.35	.12
2	0	0	.38	.11	1.15	.70
3	.23	.02	.97	.50	2.05	1.45
4	.60	.18	1.68	1.03	3.00	2.30
5	1.10	.43	2.46	1.65	3.95	3.20
Curve numbers:	57	45	75	65	91	83

Quite often this first type of lag can be assumed to take place in the manner of the second type of lag, and the technique given below can be used to estimate expected changes in hydrograph quantities.

The effect of causing a delay in the arrival of surface runoff by increasing the distance of flow is easily computed when it must be considered. Figure 3.12-3 shows hydrographs for adjacent treated and untreated watersheds. (For additional data see "Runoff from conservation and non-conservation watersheds" by J. A. Allis, Agricultural Engineering, Vol. 34, No. 11, Nov. 1953.) Two effects are evident. Some of the reduction in peak rate is due to the lesser amount of runoff from the treated watershed. Given the data as shown, the expected peak for the treated watershed would be:

$$1.74 \frac{(1.35)}{(1.68)} = 1.40 \text{ in./hr., since } \frac{q_1}{Q_{2-1}} = \frac{q_2}{Q_2} \text{ when runoff}$$

is uniformly (or nearly so) distributed on each watershed, but the actual value for W-5 is 0.87 in./hr. The difference is due primarily to a lag caused by graded terraces and open-end level terraces (which tend to grade).

Following the methods of Sections 3.15 and 3.16, the additional lag can be computed from data on figure 3.12-3. The time to peak (T_p) for W-3 is about 0.72 hour, and for W-5, about 1.05 hour. The increase in lag (since storm D is essentially identical for both hydrographs) is:

$$1.05 - 0.72 = 0.33 \text{ hour}$$

Since T_p consists of storm duration and time of concentration (see Section 3.16), the changes in either (or both) factors can be studied in a graph like that of figure 3.12-4. This graph shows that, for this case, the second type of lag effect becomes relatively insignificant at about $T_p = 5$ hours.

Ordinarily, in practice, the second type of lag effect is neglected. The technique given above can be used when the second type must be evaluated and, quite often, for evaluations of the first type of lag effect. The altered hydrographs can be reproduced by the methods of Section 3.16.

<u>Determinations for snowmelt runoff</u>. The effects of land treatment on snowmelt runoff may vary considerably from the effects on runoff from rainfall. The principal changes in effects are due partly to changes in the measures themselves, and partly to frost action.

By the time of arrival of the snow season, cultivation and weathering have usually eliminated the mechanical distinction between <u>straight row</u> and <u>contour farming</u> on cultivated lands. Other effects of contouring are usually small enough to be overshadowed by variations in areal

distribution of precipitation and are usually neglected. Graded terracing effects would be confined to the second type of lag and determined by the method shown. Closed-end level terraces and contour furrows are usually dependent on storage, not infiltration, for their effect, which is therefore calculable. The effect of land use or cover on cultivated land and pastures is small enough to be obscured by the effects of topography, fences, roads, and nearby trees and shrubs on the distribution of snow on the ground. The effect of crop rotation is similarly obscured.

In order for land treatment measures to be effective through the snow season, they must either (1) maintain high infiltration rates on soils that have a large water storage potential; or (2) maintain surface storage; but seldom both at once. High infiltration rates are maintained by vegetation that provides heavy litter or large depths of humus. Ordinary practices on cultivated lands and pastures seldom provide sufficient residues and such areas need not be considered. Permanent meadows usually provide enough litter and humus to prevent mild frost action, but not enough to be effective against heavy freezes. Commercial forest and woodland, with the exception of areas like swamps and spruce flats, are effective maintainers of infiltration, and when located on a soil with sufficient internal storage capacity, are very effective in reducing flood runoff from snowmelt. The Forest Service procedure given in Section 3.9 (see figure 3.9-1) covers the evaluation of commercial forest and woodland.

Surface storage in closed-end level terraces, and in contour furrows, may be effective in reducing snowmelt runoff as described below. Generally, on field-size watersheds, the storage has to be quite large in order to control the additional volumes of snowmelt from snow drifting from adjacent smooth fields and caught by the earthwork.

Determination of surface storage effects. Storage in closed-end level terraces and contour furrows can be evaluated on a watershed or subwatershed basis using the equation:

$$Q_s = \frac{A_s (Q_o - S_s) + A_o Q_o}{A_s + A_o} \qquad (1)$$

where Q_s = runoff in inches with storage in effect
A_s = square miles of area draining into storage and including storage pond area
S_s = storage in inches
Q_o = runoff in inches with no storage
A_o = square miles of area not draining into storage

When S_s exceeds Q_o, then only the storage equal to Q_o is effective. For example, if $S_s = 3.0$ inches and $Q_o = 1.2$ inches, then 1.8 inches of storage have not been used, and the effective storage is 1.2 inches, i. e., when $S > Q_o$, use $A_s (Q_o - S_s) = 0$.

(Note: Equation 1 and subsequent equations 2, 4 5a, and 5b, are for use when runoff and storage volumes are distributed uniformly (or nearly so) on a watershed. When the distribution is not uniform, the watershed is divided into subwatersheds on which the distribution may be considered uniform. See remarks accompanying equations 5a and 5b, below.)

Infiltration in the storage area, including that due to increased head, is generally assumed to offset storm rainfall on the storage pond area. When this infiltration is significantly large or small, it can be accounted for on a volumetric basis by changing equation 1 to read:

$$Q_s = \frac{A_p(P - F) + (A_s - A_p)(Q_o - S_s) + A_o Q_o}{A_s + A_o} \qquad (2)$$

where A_p is the average pond surface area in square miles; P is the storm rainfall, in inches; and F is the total infiltration, in inches, on the area occupied by the pond. If P is less than F, use (P - F) equal to zero. When other data are lacking, and the average depth of the pond is less than about 3 feet, F may be approximated using the following equation:

$$F = D\, f_c\, (1.5\, h + 1) \qquad (3)$$

where F = total infiltration in inches on the pond area

D = storm duration in hours for equation 2, or snowmelt duration in hours for equation 4

f_c = minimum infiltration rate in inches per hour

h = average depth of pond in feet during time D

Acres or square feet may be used instead of square miles in equations 1 and 2, but whichever unit is chosen must be used for all the areas in a particular computation.

The effect of storage on snowmelt runoff is generally computed by equation 1, since the increase in infiltration due to head, in the pond area is usually negligible because of the temperature. When this infiltration is important, equation 2 becomes:

$$Q_s = \frac{(A_s - A_p)(Q_o - S_s) + A_o Q_o - A_p(Q_o - F)}{A_s + A_o} \qquad (4)$$

unless there is rainfall on the pond surface during the melt period, in which case equation 2 is used. The effect of the earthwork in increasing the average depth of snow on an area (by catching drifting snow) is important only on very small areas, and is usually ignored.

According to unit hydrograph theory, the effect of surface storage on peak rate of flow is proportional to the effect on volume of flow when both the storage and runoff are about equally distributed over the watershed:

$$\frac{q_s}{q_o} = \frac{Q_s}{Q_o} \qquad (5a)$$

or

$$q_s = q_o \frac{Q_s}{Q_o} \qquad (5b)$$

where q_s is the reduced peak, q_o is the original peak, and the other symbols are as before. Equation 5b is adequate for many watersheds. However, when the distribution of Q_o and S_s is not sufficiently uniform, or when a watershed has a complex drainage pattern, or is unusually shaped, or has channel improvements, it is necessary to determine the storage effects on a subwatershed basis, prepare hydrographs on a subwatershed basis, and route floods, in order to determine q_s. It is usually necessary to follow this routing procedure for large watersheds, since the distribution of Q_o and S_s is nearly always nonuniform on large watersheds.

Section 3.17, Watershed examples, with accompanying tables, discusses applications of the above equations to small watersheds where nearly uniform conditions exist and hydraulic routing is not required.

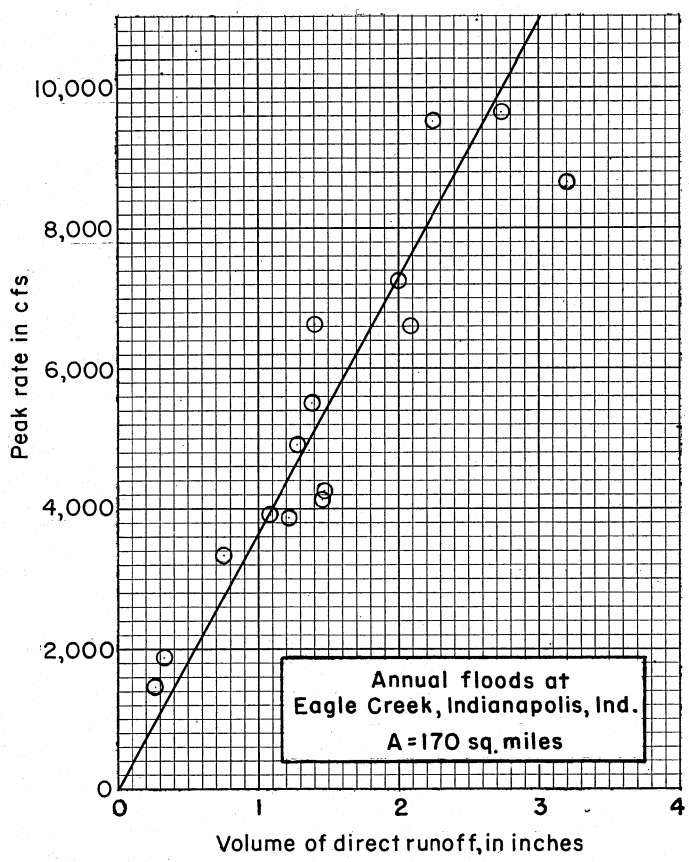


Figure 3.12-1

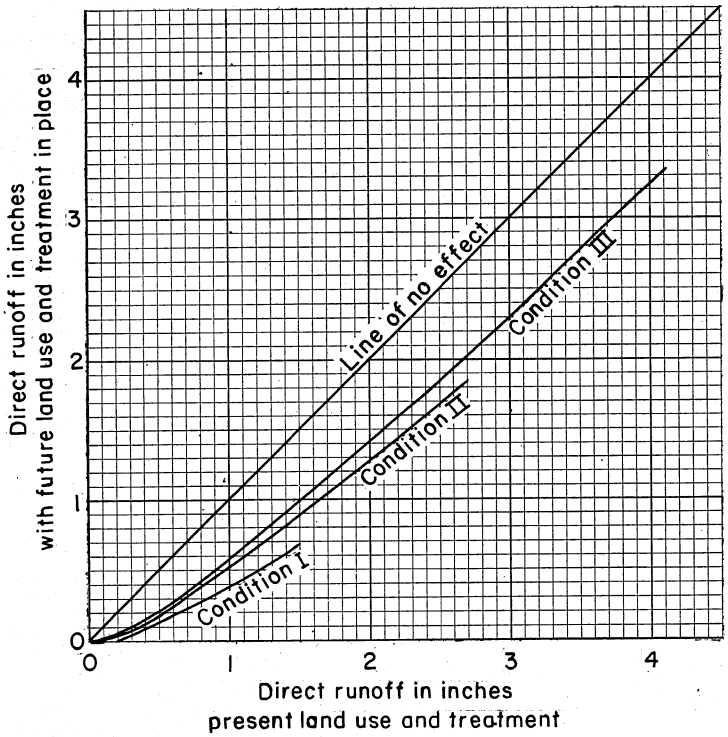


Figure 3.12-2

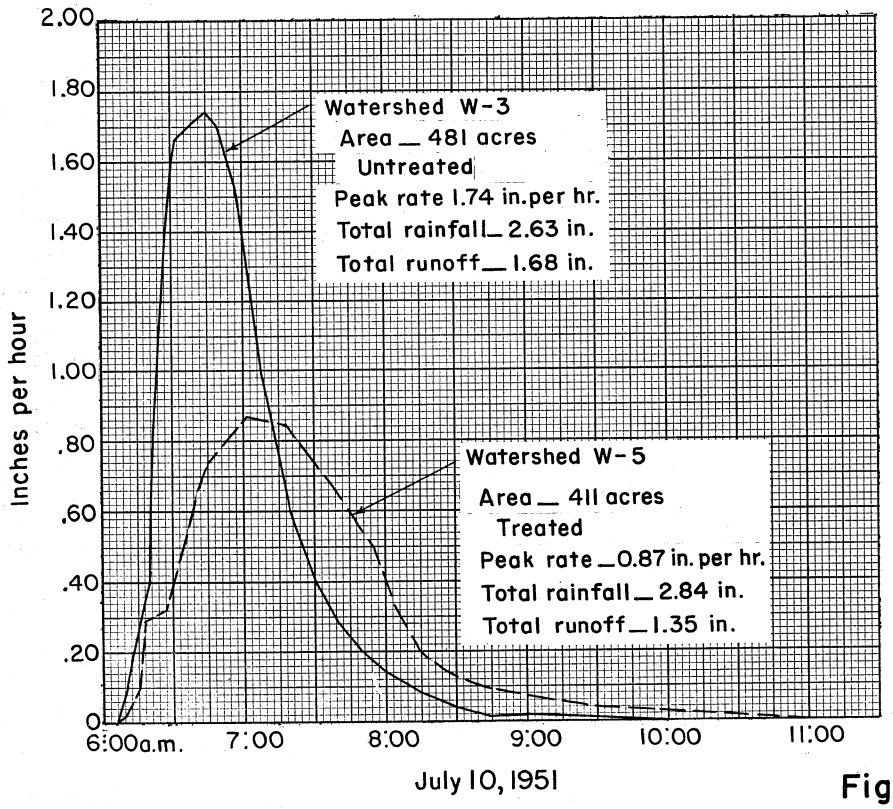


Figure 3.12-3

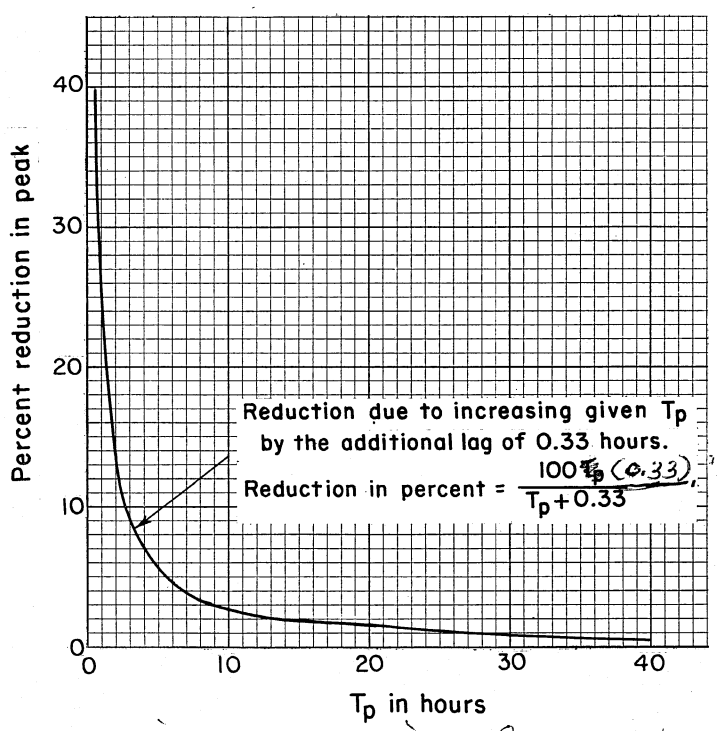


Figure 3.12-4

This curve is drawn slightly too low.

3.13 Area-inundated relations

Purpose. The economist requires data or curves showing the relation between the area inundated and (1) stage, (2) discharge, (3) flood volume, or (4) frequency. The hydrologist generally provides information on these relations, using data obtained in field surveys by both survey engineers and economists. The party leader chooses one of the above relations according to the problem at hand. The hydrologist, therefore, should learn the specific needs of the economist before determining area-inundated relations.

Stage vs. area inundated, simple cases. This method relates the flooded acres in a stream reach to the stage at either end (or middle) of the reach, usually the downstream end, except when the concordant flow method is used (see Section 4.2). As given to the economist, the stage-inundation relation shows the number of acres flooded at depths selected by the economist.

The simplest case occurs when one cross section is used to represent conditions in a reach. Table 3.13-1 shows a typical computation of a stage versus total-area-inundated relation for this case.

The acres inundated at selected depths of flooding are computed as shown in table 3.13-2. Figure 3.13-1(a) shows the results as generally given to the economist. Note that the curves of acres flooded at given depth increments can also be obtained directly from the "total acres" curve by use of an engineer's scale.

Stage vs. area inundated, complex cases. The computation of this relation becomes more laborious when more than one cross section per reach is used, the labor increasing about in proportion to the number of cross sections to be averaged. The computation also becomes complex if a variable length of reach is used, but this procedure is seldom followed for determining acres flooded. The number of acres flooded at various depths is sometimes obtained by planimetering the areas between flow lines plotted on a map of the floodplain.

When two cross sections per reach are used, and the drainage areas at the sections are not significantly different in size, the sections may be averaged as shown in table 3.13-3. Determination of acres flooded for given depth increments follows the procedure of table 3.13-2. When the two cross sections have significantly different sizes of drainage areas, the sections may be averaged as shown in table 3.13-4, with the procedure of table 3.13-2 used to get flooding by depth increments. In this case, the inundated acreage has been related to the foot of the reach. The footnote on table 3.13-3 tells how the acreage may be related to the middle of the reach for that method. The method given in table 3.13-4 is probably at its best when acreage is related to the foot of the reach, as shown.

Table 3.13-1. Sample computation of stage versus area inundated, for a simple case using one representative cross section in the reach.

Stage	Cross section top width	Width minus channel width	Inundated area in reach	Remarks
Feet	Feet	Feet	Acres	
4	24	0	0	Bankfull stage
6	92	68	13.5	
8	367	343	68.2	
10	608	584	116.0	
12	786	762	151.2	
14	872	848	168.2	

Column 4 is computed using Column 3 and the valley length of the reach. In this case the reach is 8640 feet long. To get acres, the formula is:

$$\frac{8640}{43560} \text{(Col. 3)} = 0.1984 \text{ (Col. 3)} = \text{(Col. 4)}$$

Slide rule computations.

Table 3.13-2. Sample computation of stage versus area inundated at selected depths of flooding.

Stage	Total area inundated	Acres inundated at given depths			
		0-2	2-4	4-6	Over 6 1/
(Feet)	(Acres)	(Feet)	(Feet)	(Feet)	(Feet)
4	0	0	0	0	0
6	13.5	13.5	0	0	0
8	68.2	54.7	13.5	0	0
10	116.0	47.8	54.7	13.5	0
12	151.2	35.2	47.8	54.7	13.5
14	168.2	17.0	35.2	47.8	68.2

Values in columns 3, 4, 5, and 6 can also be obtained graphically. See figure 3.13-1(a), and text.

1/ Values in last column are those of Column 2 shifted downward three lines.

426963 O - 57 - 8

Table 3.13-3. Sample computation of stage versus area inundated with 2 cross sections in the reach (head and foot) and drainage areas not significantly different.

Foot of reach Cross section 1		Head of reach Cross section 2		Areas related to foot of reach 1/			
Stage	Top width	Stage	Top width	Stage	Average top width	Average top width minus channel width	Inundated area in reach 2/
(Feet)	(Feet)	(Feet)	(Feet)	(Feet)	(Feet)	(Feet)	(Acres)
10 3/	41	7 3/	30	10 3/	35.5	0	0
12	168	9	125	12	146.5	111.0	10.7
14	646	11	478	14	562.0	526.5	51.0
16	1070	13	786	16	928.0	892.5	86.5

1/ If related to middle of reach, the stages (col. 5) are 8.5, 10.5, 12.5, and 14.5.

2/ Length of valley in reach is 4230 feet, and

$$\frac{4230}{43560} \text{ (col. 7)} = \text{(col. 8)}$$

3/ Bankfull stage.

Table 3.13-4. Sample computation of stage versus area inundated with 2 cross sections in the reach (head and foot), and drainage areas at the sections vary significantly.

Cross section A Foot of reach (D.A.=36.0 sq. mi.)			Cross section B Head of reach (D.A.= 24.0 sq.mi.)		Areas related to stages at foot of reach (Cross section A)		
Stage (Feet)	Discharge (cfs)	Top width (Feet)	Discharge (cfs)	Top width (Feet)	Average top width (Feet)	Average top width minus channel width (Feet)	Inundated area in reach (Acres)
10	720 1/	41	680 1/	32	36.5	0	0
12	1510	168	1430 2/	141	154.5	118.0	11.1
14	3060	646	2890 2/	362	504.0	467.5	43.7
16	5030	1070	4750 2/	858	964.0	927.5	87.0

1/ Bankfull discharge

2/ Proportioned by the bankfull discharge ratio 680/720. For example,

$$\frac{680}{720}(1510) = 1430 \text{ cfs}$$

Length of reach 4080 feet.

In table 3.13-4, column 3, the corresponding discharges at the upstream cross section have been proportioned using the ratio of the bankfull discharges. This method is applicable when the channels are not excessively eroded or silted. The method of taking the same discharge in csm is sometimes used, but this method ignores the fact that the upstream bankfull discharge in csm is normally greater (for natural channels in noncohesive materials and in an equilibrium condition or nearly so) than the downstream bankfull discharge in csm. In these cases the exact discharges that should be used are those of the same frequency. For example, the top width for the 2-year frequency discharge at the upper section is averaged with the top width for the 2-year frequency discharge at the lower section, and so on. When this frequency method is not used and the channel sections vary widely, much accuracy in the averaging should not be expected.

With more than two cross sections, a system of weighting must be used. Figure 3.13-1(b) shows a typical reach with seven cross sections on it. The weight for section A is a/L, for section B it is b/L, and so on. Table 3.13-5 shows a computation using three cross sections. The method of table 3.13-2 is used to complete the work.

Stage vs. area-inundated. Planimetering method. A procedure recommended by R. G. Andrews can be used either to develop a stage vs. area-inundated relation or to check such a relation developed by other methods.

1. Locate the limits of a selected large recent flood at each cross section on aerial photographs (4-inch to the mile preferred).

2. Using a stereoscope, outline the flood plain for this flood.

3. Lay out and match the photographs, and make a tracing of the floodplain outline. Show the cross section locations and details of land use.

4. Planimeter the area flooded in each reach.

5. Compute the area flooded by using the water surface width at each cross section, for each reach, and multiplying by:

$$\frac{\text{reach length in feet}}{43560}$$

Table 3.13-5. Sample computation of stage versus area inundated with 3 cross sections in the reach and drainage areas at the sections not significantly different.

Cross section 1 Weight = 0.22		Cross section 2 Weight = 0.47		Cross section 3 Weight = 0.31		Related to cross section 1		
Stage (feet)	Top width (feet)	Stage (feet)	Top width (feet)	Stage (feet)	width (feet)	Weighted top width (feet)	Weighted top width minus channel width (feet)	Inundated area in reach (acres)
8 1/	42	10 1/	44	7 1/	32	39.8 2/	0	0
10	154	12	250	9	140	194.8	155.0	30.7
12	702	14	540	11	603	595.2	555.4	109.9
14	1100	16	832	13	948	926.9	887.1	175.5

1/ Bankfull stage. Widths at this stage are channel widths.

2/ 39.8 = 0.22 (42) + 0.47 (44) + 0.31 (32). The weights are in proportion to total reach length as shown on figure 3.13-1(b).

Length of reach = 8620 ft.

6. Compare the planimetered area with the computed area. If they differ widely, make a correction factor

$$C_f = \frac{\text{planimetered area}}{\text{computed area}}$$

7. Compute the area for various other floods, using widths as in Step 5, and assuming the flood plain outline increases and decreases parallel to the outline of the selected recent large flood. Use the correction factor of Step 6, if required.

8. Plot area flooded versus stage at the selected cross section.

9. Determine areas flooded at required depth increments (Table 3.13-2).

Other methods involving planimetering are sometimes useful. For example, flood lines for each of several floods may be used to define inundated areas on aerial photos, which are planimetered and related to stage or runoff or frequency. Generally, lack of data on the location of the flood lines of historic floods limits the application of this and similar methods.

Flood peak or volume vs. area inundated. This method is generally used with alluvial fan floods, although it can also be used instead of the stage methods described above.

1. Make field interviews (the economist usually does this) to determine the areas flooded, for as many floods as possible.

2. Determine actual or estimated flood peak or volume for each flood, using a cross section or gage upstream from the fan as a reference point.

3. Plot the flood area, in acres, versus the flood peak or volume for each flood, using arithmetic paper. Draw the relation between area and peak or volume.

Once the relation is determined, the effects of upstream projects can be computed in terms of runoff. A reduced runoff means a reduced area flooded. When a channel system within the fan is proposed for reducing flooding, hydrographs are prepared at the upstream section or gage and routed downstream.

Frequency vs. area-inundated. This method is sometimes used instead of the methods described above. It is applicable to both stream reaches and alluvial fans.

1. Determine the area flooded for all known floods by field interview. The earliest known flood determines the length of record, y.

2. Array the "area flooded" values in order of size, the largest first.

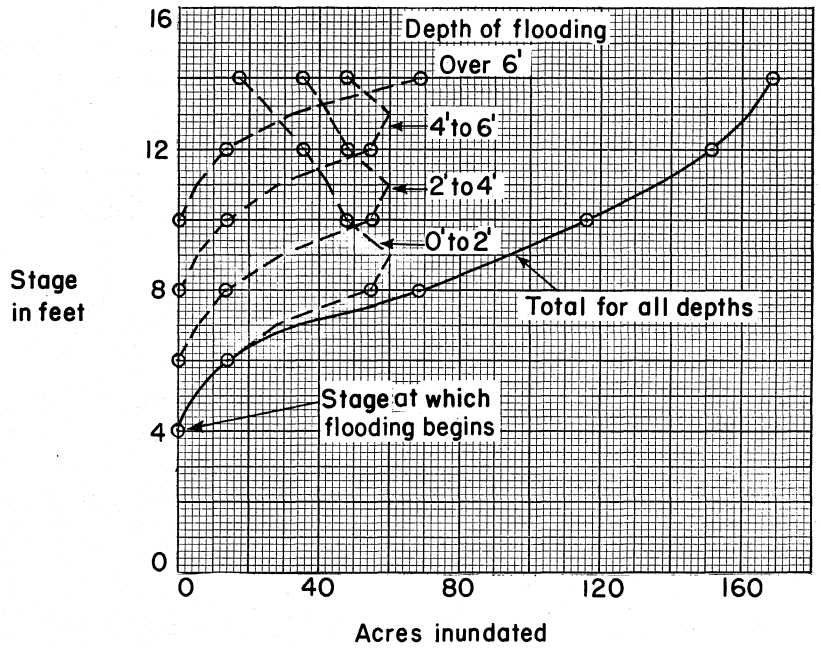
3. Use table 3.18-2 to get frequency plotting positions and tabulate these next to the array for convenience in plotting.

4. Arrange arithmetic graph paper with convenient scales for "area flooded" on the vertical axis and plotting positions on the horizontal axis.

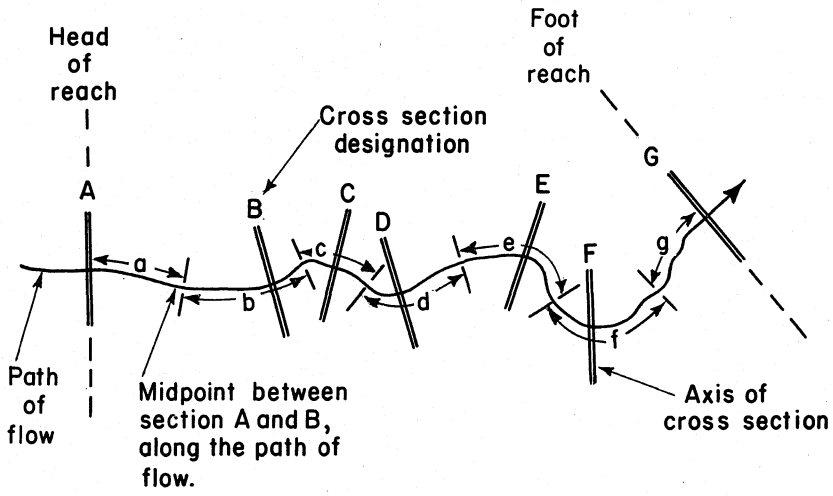
5. Plot the "area flooded" values versus their plotting positions. The point for zero area is determined by field studies.

6. Draw the frequency versus area curve. The area under the curve divided by y gives the average area flooded.

A major objection to this method is that the dollar damage per acre may vary greatly from flood to flood. In such cases, it is more accurate to use a damage-frequency curve as described in Section 3.18.



(a)



(b)

Lengths a, b, c, etc. are measured along the path of flow. Length of reach = L = a + b + c + d + e + f + g. Cross section A has the weight $\frac{a}{L}$; while B has the weight $\frac{b}{L}$; and so on.

Figure 3.13-1

3.14 Stage-discharge relations 1/, 2/

Purpose. Stage-discharge or rating curves show the depth of flow (or the stage) at a cross section versus the discharge. In flood prevention investigations stage-discharge curves are used as a basis for determining damages and developing working curves for use in flood routing. They are also used in studying the operation of flood prevention measures.

Direct measurement. The stage-discharge relation at a specific location on a stream can be determined by current meter measurements or by the installation of a weir or other type of hydraulic structure for which the head-discharge relation is known or can be estimated from the dimensions and characteristics of the structure. Service hydrologists will rarely do stream gaging or flow measurement work during flood prevention investigations, but will need to understand the principles of the work. Descriptions of gaging operations can be studied in "Stream Gaging Procedure", by Don M. Corbett et al, U. S. Geological Survey, Water-Supply Paper 888. Hydrologists from the Engineering and Watershed Planning Units and the State Conservation Engineers will be in a position to advise on arrangements with the U. S. Geological Survey or other agencies in cases where it has been administratively determined that gaging operations are to be carried out.

Manning's formula. Refer to NEH-5 for discussions of Manning's formula. The formula is repeated here because it is a necessary part of all of the methods of this section.

$$v = \frac{1.486}{n} r^{2/3} s^{1/2} \qquad (1)$$

$$Q = \frac{1.486}{n} a r^{2/3} s^{1/2} \qquad (2)$$

v = mean velocity in feet per second
r = hydraulic radius in feet
s = slope of the energy gradient or rate of loss of energy
a = cross-sectional area of flow in square feet
Q = discharge in cfs
n = Manning's roughness coefficient

A discussion of the roughness coefficient and a method of estimating its value are given in Supplement B, NEH-5.

1/ This section was prepared by Woody L. Cowan, Head, Central Technical Unit.

2/ In this section, Q is used as a symbol for discharge in cfs since it has been so used in references cited in this section. During actual watershed investigations it is preferable and more convenient to use the lower case (q) to mean discharge in cfs and to reserve the capital (Q) for runoff volume in inches.

Isolated cross sections. A reliable estimate of either stage or discharge at a single cross section can be made only when the channel is uniform in cross section, slope, and roughness condition, and the discharge occurs at normal depth. In Manning's formula the slope term is the slope of the energy gradient and is equal to the channel slope only in the case of normal flow.

When the stage-discharge relation in a reach of non-uniform channel is to be estimated, Manning's formula is combined with the energy equation (Bernoulli's equation). In this combination, Manning's formula is used in making empirical estimates of the head lost by friction, while the energy equation is used in determinations of the depth of flow or stage at individual cross sections.

In practice, stage-discharge relations at a single section are sometimes estimated by substituting channel slope for slope in Manning's formula. It is important to recognize that s in Manning's formula is not channel slope, but the slope of the energy gradient, which is the rate of loss of energy by friction. In natural streams the variations in depth, cross section, channel slope, and roughness coefficient cause the rate of energy loss at different sections to range from much less than, to several times greater than, the average channel slope.

In view of the facts briefly discussed above, a stage-discharge curve at a section when computed using average channel slope or average flood-plain slope in Manning's formula must be regarded as an approximation. The discharge for a given stage taken from such curves may be seriously in error and there is no definite way of judging its degree of reliability. Figure 3.14-1 shows stage-discharge curves at a single section based on assumed uniform flow compared with a curve based on stages determined by water-surface profile computations. The curve based on water-surface profile computations must generally be regarded as the most reliable.

Concordant flow. This method uses Manning's formula. High-water marks (HWM) from a large flood in the watershed are required. Stage-discharge curves are computed using a trial n value; the HWM discharges are adjusted to unit runoff from the contributing watershed; these adjusted discharges are plotted versus drainage area on log paper, a straight line is drawn through the plotted points, and readjusted discharges are picked off the line for the respective drainage areas; the trial n values are changed to give discharges agreeing with the readjusted discharges. See Section 2.7 for a step procedure and flow chart, and Section 4.2 for a watershed application.

Slope-area estimates. These provide a means of determining one or more points on the stage-discharge curve at a selected location or of determining the peak discharge of a flood. Where it is possible to do so, three or four cross sections are surveyed so that two or more independent estimates of discharge, based on pairs of cross sections, can be made.

The field work required for slope-area measurements consists of selecting the stream reach and surveying to determine channel profile, high-water profile, and cross sections. The work is guided by the following:

The reach selected is as nearly uniform as to channel slope, size and shape of cross section, and factors affecting the roughness coefficient as it is practicable to obtain. Sudden breaks in channel bottom grade, such as shallow drops at rock ledges, are avoided.

Elevations of high water are determined on both sides of the stream. High-water marks are taken on the ground or at points where the velocity of the flow is low. The high-water profile is surveyed upstream from, through, and downstream from the reach.

The three or more cross sections are located so as to represent as closely as possible the hydraulic characteristics of the reach. Distances between sections are taken long enough to prevent errors involved in determining stage from having too great an effect on the final estimate.

Field notes are made with sufficient description of factors affecting roughness conditions to permit realistic estimates of n.

Slope-area estimates of discharge require the solution of the energy equation. Equation A.21, NEH-5, Supplement A, is readily applicable to slope-area estimates of discharge:

$$\frac{Q^2}{2g} = \frac{E_1 - E_2}{U_2^+ - U_1^-} \qquad (3a)$$

from which

$$Q = \left[\frac{2g\ (E_1 - E_2)}{U_2^+ - U_1^-}\right]^{1/2} \qquad (3b)$$

Also, from Supplement A

$$U_2^+ = \frac{1}{a_2^2} + \frac{\ell g s_o}{Q_{n,d_2}^2} \qquad (4)$$

$$U_1^- = \frac{1}{a_1^2} - \frac{\ell g s_o}{Q_{n,d_1}^2} \qquad (5)$$

where ℓ is the length along the channel between pairs of sections and all other symbols are the same as defined in Supplement A. U_2^+ is evaluated on the basis of data at the downstream cross section, U_1^- is evaluated from data at the upstream section, $(E_1 - E_2)$ is the difference between water-surface elevations at the upstream and downstream sections, and the discharge is computed by equation (3b). The alignment charts in Supplement A will be an aid to the evaluation of U_2^+ and U_1^-.

The following examples illustrate the computation of slope-area discharge. Example 1 shows the computations for a within-channel discharge where subdivision of the cross sections is not required. Example 2 shows the computations for a case where floodplain flow is involved and subdivision of the cross sections is required.

<u>Example 1</u>. The data used in this example were taken by the U.S.G.S. on Touby Run at Mansfield, Ohio for the flood of January 30, 1947. Figure 3.14-2 shows the profile and cross sections used in the estimate of discharge. The data and computations for U_2^+, based on Section E, and U_1^-, based on Section F, are listed in the following table. Length of reach ℓ = 49 feet, n for both sections = 0.030, $E_1 - E_2$ = 0.22 feet. Values in the table are obtained as follows: F from ES-76; $\frac{s_o}{Q_{n,d}^2}$ from ES-77; $\frac{1}{a^2}$ by finding a on the left-hand scales of ES-77 and reading $\frac{1}{a^2}$ on the right-hand scales, the reading is direct.

Sec.	Sta.	W.S. elev.	a	p	F	$\frac{s_o}{Q_{n,d}^2}$	$\frac{1}{a^2}$	$\ell g \frac{s_o}{Q_{n,d}^2}$	U_1^-	U_2^+
F	4+84	5.69	81.9	35.2	214	1.97×10^{-8}	1.49×10^{-4}	3.11×10^{-5}	1.180×10^{-4}	
E	4+35	5.47	80.4	35.0	208	2.08×10^{-8}	1.548×10^{-4}	3.28×10^{-5}	1.876×10^{-4}	

$$Q = \sqrt{\frac{2g\,(E_1 - E_2)}{U_2^+ - U_1^-}} = \sqrt{\frac{64.4\times0.22}{6.96\times10^{-5}}} = \sqrt{203.563} = 451 \text{ cfs}$$

Example 2. The data used in this example were taken by the U.S.G.S. on the Concho River near San Angelo, Texas for the flood of September 17, 1936 and may be found in Water-Supply Paper 816, 1937. Figure 3.14-3 shows the high-water profile and cross sections used in the estimate of discharge. The data and computations for U_2^+, based on Section B, and U_1^-, based on Section A, are listed in the following table. Length of reach ℓ = 680 feet, n varies as shown in the table for each section, $E_1 - E_2$ = 0.517 feet. See Example 1 for explanation of column headings.

Sub-section	a	p	F	n	$\frac{Q_{n,d}}{s_o^{1/2}}$
	Cross Section A at Station 4+20				
1	2354	252	1.55×10^4	0.080	1.94×10^5
2	12691	735	12.60×10^4	.030	42.00×10^5
3	5862	231	7.50×10^4	.050	15.00×10^5
4	5385	167	8.1×10^4	.035	23.14×10^5
5	2523	135	2.64×10^4	.100	2.64×10^5
6	2498	350	1.38×10^4	.050	2.76×10^5
7	3416	645	1.54×10^4	.035	4.40×10^5
Total	34729				91.88×10^5

$$\frac{1}{(\sum a)^2} = \frac{1}{(34.729)^2} = 8.29 \times 10^{-10}$$

$$\frac{s_o}{Q_{n,d}^2} = \frac{1}{(91.88 \times 10^5)^2} = 1.18 \times 10^{-14}$$

$$\ell g \left(\frac{s_o}{Q_{n,d}} \right) = (2.19 \times 10^4)\ (1.18 \times 10^{-14}) = 2.58 \times 10^{-10}$$

$$U_1^- = (8.29 \times 10^{-10}) - (2.58 \times 10^{-10}) = 5.71 \times 10^{-10}$$

Sub-section	a	p	F	n	$\frac{Q_{n,d}}{s_o^{1/2}}$
			Cross Section B at Station 11+00		
1	1598	236	8.50×10^4	0.080	1.06×10^5
2	11750	725	11.18×10^4	.030	37.27×10^5
3	4750	227	5.37×10^4	.045	11.93×10^5
4	2086	78	3.71×10^4	.055	6.75×10^5
5	4944	153	7.43×10^4	.035	21.23×10^5
6	3455	134	4.47×10^4	.100	4.47×10^5
7	2270	273	1.38×10^4	.045	3.07×10^5
8	1518	513	0.465×10^4	.035	1.33×10^5
Total	32,771				87.11×10^5

$$\frac{1}{(\Sigma a)^2} = \frac{1}{(32,7710)^2} = 9.31 \times 10^{-10}$$

$$\frac{s_o}{Q_{n,d}^2} = \frac{1}{(87.11 \times 10^5)} = 1.32 \times 10^{-14}$$

$$\lg \left(\frac{s_o}{Q_{n,d}^2} \right) = (2.19 \times 10^4)\ (1.32 \times 10^{-14}) = 2.89 \times 10^{-10}$$

$$U_2^+ = (9.31 - 2.89) \times 10^{-10} = 12.20 \times 10^{-10}$$

$$Q = \sqrt{\frac{2g(E_2 - E_1)}{U_2^+ - U_1^-}} = \sqrt{\frac{64.4 \times 0.517}{(12.20 - 5.71)10^{-10}}} = 10^5 \sqrt{\frac{33.3}{6.49}}$$

$$Q = 10^5 \sqrt{5.13} = 2.265 \times 10^5$$

$$Q = 226,500 \text{ cfs}$$

Water Surface Profiles. The computation of water surface profiles is a reliable means of establishing stage-discharge and other hydraulic relations used in estimating damages and benefits and in studying the operation of flood prevention measures.

Methods for computing water surface profiles make use of the energy equation, and of either Manning's or some other empirical open channel formula for evaluating the friction loss. Step methods require trial-and-error computations for each reach, based on data for the reach and the results of the computations for the preceding reach. There are graphical methods that give direct solutions for stage at any cross section.

In water surface profile work it is important that: (1) reaches be short to keep to a minimum errors resulting from the averaging of hydraulic elements or rates of friction loss. Sections and reach lengths should be such as to represent average hydraulic conditions of the streams. (2) Flow conditions be determined in order to show whether water surface profiles are to be computed in the upstream or downstream direction. Where the depths of flow are greater than critical, the profile is computed upstream; where depths are less than critical, the profile is computed downstream. Supplement A of NEH-5 gives the methods and criteria for determining flow conditions. (3) Hydraulic determinations for natural channels be made by subdividing cross sections. This recognizes the fact that there are different n values associated with the channel and flood plain and makes it possible to more effectively deal with inconsistencies in hydraulic elements through certain ranges of depth in cross sections of natural streams. Figure 3.14-4 gives general rules and recommendations on acceptable practice in determining the hydraulic elements of subdivided cross sections.

Working procedures for four methods of computing water surface profiles are described and illustrated below. These methods are Doubt's, Escoffier's, Leach's, and a step method.

Doubt's Method. Working details of this method are presented in NEH-5, Supplement A. The method, as presented, is based on the assumption that the discharges at all cross sections are equal and constant during any time interval, and recognizes velocity head changes. In determining steady flow water surface profiles in a system of watershed channels, it is preferable to modify the method so that the discharge at any cross section is proportional to the drainage area upstream from the section. Equation A.21, Supplement A, may be expressed:

$$\frac{E_1 - E_2}{U_2^+ - \beta U_1^-} = \frac{Q_2^2}{2g} \qquad (6a)$$

$$\text{with } \beta = \left[\frac{Q_1}{Q_2}\right]^2 = \left[\frac{A_1}{A_2}\right]^2 \qquad (6b)$$

A_1 = drainage area upstream from section 1

A_2 = drainage area upstream from section 2

Q_1, Q_2 = discharges at sections 1 and 2 obtained by multiplying A_1 and A_2 by any selected unit area rate of runoff

The following steps apply in computing water surface profiles by Doubt's method: 1/

1. Plot cross sections on cross-section paper. Show the channel and floodplain subdivisions 2/ and the estimated n for each subdivision. Determine the cross-sectional area and wetted perimeter for each subdivision, also the total area and the top width of flow of the complete section at 4 to 8 different depths. See suggestions on figure 3.14-4, sheet 1. These data may be listed in tables on the sheets on which the cross sections are plotted or they may be listed on separate tabular sheets that are clearly referenced to the appropriate cross sections.

2. Plot the stream profile, using channel distances, for all parts of the stream system in which it has been determined that water surface profiles will be computed. Show the locations of all cross sections and bridges.

3. Prepare a table like table 3.14-1. In this table list and compute the indicated values for all cross sections for a project. For sections at bridges, use columns 1 to 16 inclusive and column 22.

Column 1. List section number or other section designation, bottom elevation, station, β, ℓ d (distance to next downstream section), ℓ u (distance to next upstream section). List ℓ d and ℓ u for both the lower and higher stages when there is a significant difference between channel and valley length. See figure 3.14-4, sheet 1.

Column 2. List depths in the section at about 1-foot intervals including the depths selected in Step 1 for which the area, wetted perimeter and other hydraulic values are determined.

Column 3. List elevations for the depths in column 2.

Columns 4, 5, 6, 7. List the values of a, p, F (computed using ES-76), $\frac{Q_{n,d}}{s_o^{1/2}}$ (computed using ES-77).

The values are listed only for the depths selected in Step 1.

1/When there are a large number of water surface profiles to be computed by this method, the advantages of using an electronic computer should be examined. See Engineering Memorandum No. ~~11,~~ 16 dated ~~Nov. 7, 1956,~~ Aug 23, 1957 for ~~Remington Rand Univac contract and data specifications.~~ instructions on submitting data.

2/Subdivisions are called segments in Engineering Memorandum No. 11.

This set of columns applies to one section subdivision. The table must include a set of columns identical to columns 4, 5, 6, and 7 for each subdivision in a section. List the n for each section subdivision.

The numbers of the following columns, which are used only to facilitate description, are for the case when only two subdivisions for each section are used. If three subdivisions are used, the first column below would be 16.

Column 12. List $\frac{Q_{n,d}}{s_o^{1/2}}$ for the complete section; that is, the sum of these values for the section subdivisions. Compute only for the depths selected in Step 1. Plot depth vs. $\frac{Q_{n,d}}{s_o^{1/2}}$ on log paper. See figure 3.14-5.

Column 13. List $\frac{s_o}{Q_{n,d}^2}$ the square of the reciprocal of column 12. The values can be obtained from the right-hand scales on ES-77. The numbers on the right of the scales are the squares of the reciprocals of the numbers on the left.

Column 14. List $\sum a$, area of the complete section; that is, the sum of the areas of the section subdivisions.

Column 15. List $\frac{1}{(\sum a)^2}$ taken from the right-hand scales on ES-77.

Column 16. List top width, T, for the complete section, determined in Step 1.

Column 17. Compute and list $\frac{\ell_d g s_o}{Q_{n,d_1}^2}$, that is, $\ell_d g$ times column 13.

Column 18. Compute and list $\frac{\ell_u g s_o}{Q_{n,d_2}^2}$, that is, $\ell_u g$ times column 13.

Column 19. Compute and list $U_1^- = \frac{1}{(\sum a)^2} - \frac{\ell_d g s_o}{Q_{n,d_1}^2}$, that is, column 15 minus 17.

Column 20. Compute and list $U_2^+ = \frac{1}{(\sum a)^2} + \frac{\ell_u g s_o}{Q_{n,d_2}^2}$, that is column 15 plus column 18.

When the U_2^+ values for the depths selected in Step 1 have been computed, plot depth versus U_2^+ on log paper. See figure 3.14-6. From this plotting read and list U_2^+ for each depth listed in column 2.

426963 O - 57 - 9

Column 21. Compute and list βU_1^-, that is, Beta times column 19. When the βU_1^- values for the depths selected in Step 1 have been computed, plot depth versus βU_1^- on log paper. See figure 3.14-7. From this plotting read and list βU_1^- for each depth listed in column 2.

Column 22. Compute and list $Q_{c,d}$. Use ES-75 with $a = \sum a$ from column 14, and T from column 16. When $Q_{c,d}$ for the depths selected in Step 1 have been computed, plot these depths as d_c versus $Q_{c,d}$ on log paper. See figure 3.14-8.

4. Determine whether any of the sections are control sections for the range of discharges to be considered. Prepare table 3.14-2.

Column 1. List the cross-section number or other designation.

Column 2. List the unit area rates of runoff to be considered; that is, those for which water surface profiles are to be computed.

Column 3. List the drainage area upstream from each cross section in square miles.

Column 4. Compute and list $\beta = \left[\frac{A_1}{A_2}\right]^2$ where A_1 is the drainage area above the section under consideration and A_2 is the drainage area above the next downstream section.

Column 5. Compute and list Q_2; that is, column 2 times column 3.

Column 6. Compute and list $\frac{Q_2^2}{2g}$ for use in Step 6.

Column 7. List the critical depths for two of the discharges in column 5. One discharge should be somewhat less than the approximate channel capacity and the other the maximum value in column 5. These depths are taken from the plotting of d_c versus $Q_{c,d}$ referred to under column 22 of table 3.14-1. See figure 3.14-8. It will usually be found that if a section is or is not a control for both a low and high discharge this will also be true for all discharges between the two. As a first test, examine only these two discharges by the computations in columns 8 through 13. Where a section is found to be a control or not a control for both the low and high discharge, this may be accepted as applying to all discharges to be considered. If a section is or is not a control for one of the two selected discharges, but not for the other, then intermediate depths are used in column 7 and the computations in columns 8 through 13 made for these additional depths.

Column 8. List values of $\frac{Q_n \; d_c}{s_c^{1/2}}$ associated with the critical depths in column 7.

These values are taken from the plot of depth versus $\frac{Q_{n,d}}{s_o^{1/2}}$ referred to under column 12 of table 3.14-1, Step 3. See figure 3.14-5.

Column 9. List $s_c^{1/2}$ computed by dividing column 5 by column 8.

Column 10. List s_c, the square of column 9.

Column 11. List the slope upstream from the section listed in column 1.

Column 12. List the slope downstream from the section listed in column 1.

In computing the bottom slopes for columns 11 and 12, where the channel length is materially greater than floodplain length and for depths in column 7 up to a minimum of 1 or 2 feet above bankfull stage, channel length may be used in computing slope. For greater depths flood plain lengths may be used in computing slope. See figure 3.14-4, sheet 2.

Column 13. This column shows whether a section is or is not a control. Flow conditions are determined as follows. When:

a. Col. 11 < col. 10 < col. 12.
$(s_u < s_c < s_d)$

The section listed in col. 1 is a control and the water surface profiles must be computed upstream and downstream from this section. The starting stage is that associated with the critical depth for the discharge considered.

b. Col. 11 < col. 10 > col. 12.
$(s_u < s_c > s_d)$

The section is not a control, subcritical flow occurs and the water surface profiles must be computed upstream from some section that is downstream from this section.

c. Col. 11 > col. 10 < col. 12.
$(s_u > s_c < s_d)$

The section is not a control, supercritical flow occurs and the water surface profile must be computed downstream from some section that is upstream from this section.

Column 14. The water surface elevations to be listed here are for the various unit area discharges and are to be determined in Step 6.

5. Plot the curves of U_2^+ and βU_1^- versus elevation. These are the working curves by which the graphical solution for the water surface elevation at each section is made. The setup of the work sheets is illustrated by figure 3.14-9. The curves of U_2^+ are plotted from data in columns 3 and 20, table 3.14-1, Step 3, and curves of U_1^- are plotted from columns 3 and 21 of this table. Values of βU_1^- may be either positive or negative and must be plotted accordingly.

In working with natural streams where a wide range of discharges are considered, working curves of U_2^+ and βU_1^- through a wide range of values will be required. It is recommended that 20 or 24-inch wide cross-section paper ruled 10 x 10 to the inch be used. The vertical elevation scale for each stream reach between two cross sections should be continuous from bottom to top of sheet. The origin of the horizontal scales of U_2^+ and βU_1^- should be at the center of the sheet with a minimum of 10 inches on each side of the origin. As many pairs of U_2^+ and βU_1^- curves as needed to span the working range of values should be plotted. For example, one pair of curves may be for horizontal scale values times 10^{-4}; the next higher pair for scale values times 10^{-6}, and so forth. There should be a minimum overlap of 2 feet in elevation for each successive pair of curves.

There must be a reference point and an associated slope scale of $\frac{Q_2^2}{2g}$ values associated with each pair of U_2^+ and βU_1^- curves. The slope scale is fixed by the elevation scale, the scale values of U_2^+ and βU_1^-, and the distance from the reference point to the slope scale. For example, assume that: (a) 1 inch on the vertical elevation scale represents 2 feet, (b) 1 inch on the horizontal scale of U_2^+ and βU_1^- represents 10^{-5}, (c) the slope scale is 10 inches horizontally from the reference point. The value of $\frac{Q_2^2}{2g}$, per inch on the slope scale for these assumptions is:

$$\frac{Q_2^2}{2g} \text{ (per inch)} = \frac{2}{10\text{x}10^{-5}} = 2\text{x}10^4$$

6. Make the graphical solution for the water surface elevations of the various cross sections for each of the uniform unit area rates of discharge. List elevations in column 14, table 3.14-2, described under Step 4. Figure 3.14-9 illustrates the graphical solution. Additional details are given in Supplement A, NEH-5. In some cases the line representing $\frac{Q_2^2}{2g}$, when drawn from a point E_2 on the U_2^+

curve for a section, will intersect the U_2^+ curve at a second point that is higher in elevation. This condition can be encountered during the graphical solution in two cases. First, it may be caused by too great a distance between cross sections. Second, it is likely to be encountered in computing the profile upstream from a control section. Methods for dealing with each of these cases are outlined on Figure 3.14-10, sheets 1 and 2.

In the case of long reaches, if the difference in elevation between the two points on the U_2^+ curve is only a few tenths of a foot, the intersection of the $\frac{Q_2^2}{2g}$ line with the βU_1^- curve for the next upstream section may be accepted as a reasonable approximation of E_1 at that section. However, if this difference in elevation is more than 1.5 feet, the solution for E_1 should not be accepted as reliable and the procedure outlined on Sheet 1, Figure 3.14-10 followed. The procedure recommended is one of expediency. The water-surface profile computed by this procedure will be accurate within practicable limits if: (a) the cross sections of the ends of the reach reflect the average hydraulic characteristics of the reach; (b) the channel slope is approximately uniform through the reach. If the hydraulic conditions or the channel slope vary widely in the reach, additional cross sections should be taken in the field.

In the case of control sections certain procedures are necessary in computing upstream through the drop-down part of the surface profile. Both depth and rate of energy loss change rapidly from section to section along the drop-down curve. Therefore, a $\frac{Q_2^2}{2g}$ line extended from E_2 for critical depth on the U_2^+ curve for a control section to the βU_1^- curve for a section several hundred feet upstream does not give a reliable solution for E_1. A satisfactory procedure for computing the profile upstream from control sections is outlined on sheet 2, figure 3.14-10.

<u>Escoffier's Method</u>. Example 2, NEH-5, pages 5.4-52 through 5.4-60 explains and illustrates Escoffier's method as applied to channels in which the steady discharges are equal from section to section. The method neglects velocity head changes. In watershed investigations, it is desirable to modify the method so that the discharge at a cross section is proportional to the drainage area upstream from the section. The equation solved graphically in this method is:

$$\frac{h_f}{P_2 + P_1} = \frac{Q^2 \ell}{2} \qquad (7)$$

The friction loss, h_f, which in this case is the difference between water-surface elevations at the head and foot of the reach, may be replaced by $E_1 - E_2$ so that the above equation may be written

$$\frac{E_1 - E_2}{P_2 + P_1} = \frac{Q^2 \ell}{2} \qquad (8)$$

For watershed work the equation is further modified to read:

$$\frac{E_1 - E_2}{P_2 + P_1} = \frac{Q_2^2 \ell}{2} \qquad (9)$$

where

E_1 = water-surface elevation at the upstream section of a reach

E_2 = water-surface elevation at the downstream section of a reach

ℓ = length of reach in feet

Q_2 = A_2 times the unit area rate of runoff considered

$\beta = \left[\frac{A_1}{A_2}\right]^2$

A_1 = drainage area above upstream section in square miles

A_2 = drainage area above downstream section in square miles

$P_2 = \left[\frac{1}{\frac{Q_{n,d}}{s_o^{1/2}}}\right]^2 = \frac{s_o}{Q_{n,d}^2}$ at section 2 (downstream)

$P_1 = \left[\frac{1}{\frac{Q_{n,d}}{s_o^{1/2}}}\right]^2 = \frac{s_o}{Q_{n,d}^2}$ at section 1 (upstream)

The following steps apply in computing water surface profiles by Escoffier's method:

1. Same as for Doubt's method.
2. Same as for Doubt's method.
3. Compute values of P for each section.

Prepare a table similar to table 3.14-3. Compute the indicated values for all cross sections for a project. For sections at bridge openings, use columns 1 to 12 inclusive.

Column 1. List the section number or other section designation, bottom elevation, station (Beta).

Column 2. List the depths in the section at about 1-foot intervals including the depths selected in step 1.

Column 3. List elevations for the depths in column 2.

Columns 4, 5, 6 and 7. List the values of a, p, F (computed using ES-76), $\frac{Q_{n,d}}{s_o^{1/2}}$ (computed using ES-77).

These values need be listed only for the depths selected in Step 1. The table must include a set of columns identical to columns 4, 5, 6 and 7 for each subdivision in a section. List the n for each section subdivision.

Column 12. (Column 12 if 2 subdivisions are used, and column 16 for 3 subdivisions). List $\frac{Q_{n,d}}{s_o^{1/2}}$ for the complete section; that is, the sum of these values for the section subdivisions. Compute only for the depths selected in Step 1.

Column 13. List $P = \frac{s_o}{Q_{n,d}^2}$, the square of the reciprocal of column 12.

These values can be obtained from the right-hand scales on ES-77. The numbers on the right of any scale are the squares of the reciprocals of the numbers on the left. When P values for the depths selected in Step 1 have been computed, plot depth vs. P on log paper. See figure 3.14-11. From this plotting, read and list P for each depth listed in column 2.

Column 14. Compute P; that is times the values in column 13.

4. Plot the working curves of P_2 and βP_1 versus elevation. The layout of the work sheet is illustrated by figure 3.14-12. The P_2 curves are plotted on the left of the work sheet from columns 3 and 13 of table 3.14-3. The βP_1 curves are plotted on the right of the

sheet from columns 3 and 14 of table 3.14-3. A P_2 curve represents a section at the downstream end of a reach and a βP_1 curve represents a section at the upstream end of a reach. In order to span the required range of values, it will often be necessary to plot two or more pairs of P_2 and P_1 curves for the same reach. These are plotted on a single sheet having the elevation scale continuous from bottom to top.

There must be a reference point and an associated slope scale of $\frac{Q_2^2 \ell}{2}$ related to each pair of P_2 and βP_1 curves. The slope scale in each case is fixed by the elevation scale, the P_2 and βP_1 scales and the distance from the reference point to the slope scale. Assume: (a) 1 inch on the elevation scale equals 4 feet; (b) 1 inch on the P_2 and βP_1 scales equals 10^{-6}; (c) the slope scale is 5 inches horizontally from the reference point. The value of $\frac{Q_2^2 \ell}{2}$ per inch on the slope scale is:

$$\frac{Q_2^2 \ell}{2} = \frac{4}{5 \times 10^{-6}} = 0.8 \times 10^6 \text{ or } 8.0 \times 10^5$$

5. Solve graphically for the water-surface elevations at each section for the unit area rates of discharge. The solution in any reach is made as follows: (a) locate the point E_2 on the P_2 curve representing the water-surface elevation at section 2; (b) construct a line having the slope $\frac{Q_2^2 \ell}{2}$ and extending from the point E_2 to intersect the βP_1 curve for section 1. This point of intersection establishes E_1.

Compute values of Q_2, $\frac{Q_2^2 \ell}{2}$ and record the water-surface elevations at the successive sections in a table similar to table 3.14-4.

Leach's Method. This method neglects velocity head changes, and makes use of the following arrangement of Manning's formula:

$$s = \left[\frac{Q}{K_d}\right]^2 \qquad (10)$$

where

s = slope of water surface through a reach, since velocity head is neglected.

Q = discharge in cfs

$K_d = \frac{Q_{n,d}}{s_o^{1/2}}$ for the complete section

The method considers the cross sections surveyed in the field to be average cross sections for the various reaches. The upstream and downstream ends of reaches are taken as being halfway between the surveyed cross sections. Figure 3.14-13, sheet 1, shows a sketch illustrating this.

The following steps are taken in the use of this method:

1. Same as described in Doubt's method.

2. Same as described in Doubt's method. Show location of head and foot of each reach.

3. Compute the K_d values for each section at the depths selected in Step 1. These computations are made in a table like table 3.14-5. Instructions for this table are the same as those given for table 3.14-1, step 3 of Doubt's method. Column 1 may be omitted if desired, but may be used to list selected general data. Columns 2 to 12, inclusive, should be used and the instructions given under each apply. For the purposes of this method, the $\frac{Q_{n,d}}{s_o^{1/2}}$ values for the complete section in column 12 are equal to K_d values for the complete section. When the K_d values for the depths selected in Step 1 have been computed, plot depth versus K_d on log paper. From this plotting, read and list K_d in column 12 for each depth in column 2. See figure 3.14-14.

4. For the discharge at each section, and unit area rate of runoff, compute and plot a family of curves like those shown on figure 3.14-13, sheet 2.

(a). Plot elevation versus K_d for each cross section from columns 3 and 12 of the table developed in Step 3. See figure 3.14-15.

(b). Compute the data for each family of curves as shown on table 3.14-6. These computations are made for each unit area of runoff. The reaches are numbered consecutively from 1 at the downstream end of the project. It is desirable to show the numbered reaches on the profile.

(c). Plot the curves of water-surface elevations at the foot and head of reaches as indicated by sheet 2, figure 3.14-13. These curves, one for each reach, are plotted from columns 12 and 13, table 3.14-6. The horizontal and vertical scales must be identical. Since a family of these curves is valid for only one discharge, each sheet must be clearly marked with the csm discharge.

5. Compute the water-surface elevations at the foot and head of successive reaches for any given csm discharge in this way:

(a). Enter the family of curves for the given csm discharge with the known or assumed water-surface elevation at the foot of reach 1. This defines a starting point on the curve for reach 1.

(b). From this point draw a vertical line to intersect the curve for reach 2. This point defines the water-surface elevation at the foot of reach 2 and the head of reach 1.

(c). From this point, draw a horizontal line to intersect the curve for reach 3. This point defines the water-surface elevation at the foot of reach 3 and the head of reach 2. This process of drawing successive vertical and horizontal lines to intersect the successive reach curves is continued to the farthest upstream reach.

Where bridges are encountered, the water-surface elevation at the section immediately upstream from the bridge is determined by methods described below under "Road and bridge crossings". The section above the bridge is the foot of a reach and the solution is continued upstream as before.

Step Method. The method described here neglects velocity head changes. From the similarity of certain parts of the method to those of Leach's method, this step method is sometimes called a variation of Leach's method. See Section 4.1. A step method that does not neglect velocity head changes is given in NEH-5, subsection 4.7.5, Example 1, pages 5.4-49 through 5.4-51.

The equation solved by trial and error is:

$$E_1 - E_2 = \frac{\ell}{2}\left[\left(\frac{Q_1}{K_{d_1}}\right)^2 + \left(\frac{Q_2}{K_{d_2}}\right)^2\right] \qquad (11)$$

where

E_2 = water-surface elevation at the foot of a reach

E_1 = water-surface elevation at the head of a reach

ℓ = length of reach

Q_1 = discharge at the head of the reach = drainage area above the section times unit area discharge

Q_2 = discharge at the foot of the reach - area times the unit area discharge rate

$K_{d_1} = \frac{Q_{n,d_1}}{s_o^{1/2}}$ for the complete section at the head of the reach

$K_{d_2} = \frac{Q_{n,d_2}}{s_o^{1/2}}$ for the complete section at the foot of the reach

The following steps are taken:

1. Same as described in Doubt's method.

2. Same as described in Doubt's method.

3. Same as described in step 3 under Leach's method. Plot elevation versus K_d for each cross section from columns 3 and 12 of the table 3.14-5.

4. Make the trial and error solution for E_1 for each reach in a table similar to table 3.14-7.

<u>Column 1</u>. List the sections at foot and head of reach and the drainage areas upstream from each section, also the unit area discharges.

<u>Column 2</u>. List length of reach.

<u>Column 3</u>. List half of reach length; that is, half of column 2.

Column 4. List water-surface elevation, E_2, at the foot of the reach; E_2 will be known or assumed at the foot of the first reach. In the second and succeeding reaches, E_2 for the reach is E_1 for the preceding reach.

Column 5. List assumed water-surface elevation, E_1, at the head of the reach.

Column 6. List $Q_2 = A_2$ times the csm considered.

Column 7. List $Q_1 = A_1$ times the csm considered.

Column 8. List the K_{d_2} associated with E_2 in column 4; K_{d_2} is read from the plot of K_d versus elevation for the section at the foot of the reach.

Column 9. List the K_{d_1} associated with the assumed E_1 in column 5 for the section at the head of the reach.

Column 10. List $\left[\frac{Q_2}{K_{d_2}}\right]^2$; that is, $\left[\frac{\text{Col. 6}}{\text{Col. 8}}\right]^2$

Column 11. List $\left[\frac{Q_1}{K_{d_1}}\right]^2$; that is, $\left[\frac{\text{Col. 7}}{\text{Col. 9}}\right]^2$

Values of $\left[\frac{Q}{K_d}\right]^2$ are given in King's Handbook, 3rd edition, Table 144, page 535, 4th edition, page 8-65.

Column 12. List s_f, the sum of columns 10 and 11.

Column 13. List h_f, column 3 times column 12.

Column 14. List computed water-surface elevation, E_1, at the head of the reach; column 4 plus column 13. If the computed and assumed E_1 (columns 5 and 14) are in sufficiently close agreement, accept the trial solution; if not, make a new assumption in column 5 and repeat computations through column 14.

Column 15. May be used or omitted. In either case, the values in columns 5, 9, 11 and 14 for the accepted solution should be heavily underscored or bracketed to show that they are the values to be used in computations for the next reach.

Summary of Water-Surface Elevations for Six Mile Creek. Data for a part of the stream system of Six Mile Creek, Arkansas, are used to illustrate the computations associated with the methods described above. Figure 3.14-16 shows the channel profile, locations of cross sections, and the water surface profiles computed by Doubt's method.

Six Mile Creek is a tributary of the Arkansas River near Paris, Arkansas. Cross section 1 on Six Mile Creek is about 2 miles upstream from the confluence with the Arkansas River. No data were available on which to base an estimate of the stage-discharge curve at section 1. However, it is likely that the stage at this section is affected by the stage of the Arkansas River, particularly during flood periods on the Arkansas River. In view of these conditions, the computations by each method were based on the assumption that normal discharge occurred at section 1. Thus the starting elevation at section 1 for each discharge was the stage associated with normal depth at that section.

Table 3.14-8 shows the water-surface elevations at the various sections computed by Doubt's and the Step Method. The water surface profiles were not computed by Leach's or Escoffier's method. However, if computed, the water-surface elevations should, except for minor variations, agree with those computed by the Step Method since Leach's, Escoffier's, and the Step Method neglect velocity head changes.

Doubt's method is the only method presented that recognizes velocity head changes. The elevations in Table 3.14-8 show that in most cases the values computed by Doubt's method and the Step Method are either equal or in close agreement. There are, however, a few cases in which the differences between elevations computed by the two methods range from about 0.5 to 1.0 foot. Where these larger differences occur, they result primarily from the fact that Doubt's method considers velocity head changes whereas the Step Method neglects those changes.

Starting Elevations. Any method of computing water surface profiles requires that the stages for different discharges at the foot of the farthest downstream reach be known or be determined within practicable limits of accuracy. There are three general types of cases that may be encountered.

First, there may be a stream-gaging station at or near the foot of the downstream reach. This fortunate condition is not found in many watersheds. However, where it applies and where a rating curve has been established, a cross section should be surveyed at the gaging station so that the rating curve may be used to establish the starting elevations for the various discharges.

Second, the section at the foot of the downstream reach may be a control section. This is also an unusual and fortunate condition. Where it applies, the curve of critical depth versus critical discharge is used to establish the starting elevations for various discharges.

Third, when the section at the foot of the downstream reach is not a control section, a direct means of establishing a stage-discharge curve is not available. In these cases, and where flow is in the subcritical range, the following approach should be used:

(a) A minimum of three or four auxiliary cross sections should be surveyed at intervals of 500 to 1500 feet downstream from the foot of the farthest downstream reach through which water surface profiles are desired. For convenience in the following description, the section at the foot of this reach is referred to as section 1, and the farthest downstream of the auxiliary sections as section A.

(b) Make the hydraulic computations for each of the auxiliary cross sections that are required by the method of water surface profile computation that is to be used.

(c) Select four to six discharges in the range to be considered. For each discharge assume several stages at section A. Compute water surface profiles up to section 1, starting with each of the assumed stages at section A. For a limited range of assumed stages at section A the profile for any selected discharge will converge to about the same stage at section 1. These common stages for the selected discharges define a stage-discharge curve at section 1. The starting elevation at section 1 for any discharge for which the water surface profile is to be computed may be taken from this curve.

Selection of Method. A decision as to the method of water surface profile computation to be used should be based on the accuracy required, the reliability of different methods, and the time required for making the computations by the various methods.

Accuracy depends on the caliber and amount of field data as well as on the method of computation. A sound method of computation cannot assure accurate surface profiles unless the cross sections, channel profile, and other field data reliably define the conditions to be dealt with in the computations.

All four methods presented in this Section are reliable within the limitations of the fact that Doubt's method recognizes velocity head changes, and the other three methods neglect velocity head changes. Therefore, insofar as as accuracy is concerned, the selection of method rests upon recognition or neglect of velocity head changes.

It is not feasible to give a full discussion of the effects of neglecting velocity head changes under the wide range of conditions that may be encountered. However, the following will serve as a guide to judgment. In accelerated, subcritical flow, the water surface profile will be lower if computed by neglecting velocity head than it will be if computed by recognizing velocity head. In retarded, subcritical flow, the water surface profile will be higher if computed by neglecting velocity head than it will be if computed by recognizing velocity head. It is common for portions of water surface profiles computed by neglecting velocity head to be from 0.5 to 1.5 feet above or below the profiles computed by recognizing velocity head. In some cases the differences between the two profiles may be as high as 2 to 3 feet.

There is a great deal of similarity between the four methods described, not only as to the general steps involved in the work, but also as to the values used in the computations. When any of the methods are used, important savings in time can be obtained if all phases of the job are organized and carried out in a systematic manner. The tables and graphs recommended in the descriptions of the different methods have been set up with the objective of providing a systematized routine for making the necessary computations.

The following summary statements may be useful in the selection of method:

Doubt's Method. Accuracy: Most accurate of the four methods described, and as accurate as any method available.

Relative time required: As favorable as for any method available where the profiles for two or more discharges are to be computed. Requires about the same time as Escoffier's method. Requires less time than either Leach's or the Step Method where the profiles for more than one discharge are to be computed.

Escoffier's method. Accuracy: Neglects velocity head changes and is, therefore, less accurate than Doubt's method.

Relative time required: Requires about the same time as Doubt's method. It requires less time than Leach's or the Step Method where the profiles for more than one discharge are to be computed.

Leach's method. Accuracy: Neglects velocity head changes and is less accurate than Doubt's method. This method considers a single cross section to represent the average hydraulic values for a reach. Most methods consider the hydraulic values for a reach to be the averages of the values associated with the cross sections at the ends of the reach. Thus, this method is less realistic and probably should be regarded as somewhat less accurate than either Escoffier's or the Step method.

Relative time required: Requires more time than either Doubt's or Escoffier's method if the profiles for more than one discharge are to be computed. This requirement of additional time results from the fact that an independent set of computations and working curves must be developed for each discharge considered.

Step method. Accuracy: Neglects velocity head changes and is less accurate than Doubt's. Equal in accuracy to Escoffier's method.

Relative time required: Requires more time than either Doubt's or Escoffier's method if more than one discharge is to be considered. Requires about the same amount of time as Leach's method. The relatively high amount of time required by this method results from the fact that independent trial and error solutions must be made for each reach for each discharge considered.

Road and bridge crossings. The determination of continuous water surface profiles along streams requires a method by which the drop in water surface through bridges may be estimated within practical limits of accuracy. Flood stages upstream from bridges may be several feet higher than stages downstream. Where all discharges in a reach downstream from a bridge occur in the subcritical range, the bridge has no effect on stages downstream from the bridge.

The general approach to the determination of stage upstream from a bridge for a given discharge is as follows: (1) Compute the water surface profile up to a cross section just downstream from the bridge; (2) estimate the drop in water surface through the bridge and establish the stage at a cross section just upstream from the bridge. One or two conditions may exist:

(a) The depth of flow in the bridge section may be greater than the critical depth for the discharge under consideration. Then the depth of flow at the section upstream from the bridge depends on the depth at the section downstream from the bridge. The estimate of stage at the upstream section proceeds from the known depth at the downstream section.

(b) The depth of flow at the bridge section may be equal to or less than the critical depth for the discharge under consideration. Then the depth of flow upstream from the bridge does not depend on the depth downstream from the bridge. The estimate of upstream stage is based on the occurrence of critical depth in the bridge opening. This requires a determination of flow conditions at the bridge opening.

An estimate of stage upstream from a bridge requires three cross sections as follows: Section 1 is taken just upstream from the bridge; section 2 is taken approximately on the center line of the bridge; section 3 is taken just downstream from the bridge. Sections 1 and 3 should not be affected by the embankment or borrow pits, thus they may be from 50 to 300 feet upstream and downstream from the bridge.

Two methods of estimating stage upstream from a bridge are described:

First method. The energy equations for the two reaches established by sections 1, 2, and 3 are:

$$\frac{v_1^2}{2g} + d_1 + z_1 = \frac{v_2^2}{2g} + d_2 + z_2 + h_f' + h_c \qquad (12)$$

$$\frac{v_2^2}{2g} + d_2 + z_2 = \frac{v_3^2}{2g} + d_3 + h_f'' + h_e \qquad (13)$$

Datum is the low point of section 3; subscripts 1, 2, and 3 denote sections 1, 2, and 3.

v = velocity
d = depth
g = acceleration due to gravity
z = elevation head
Q = discharge
a = cross-sectional area of flow
h_f' = head lost by friction between sections 1 and 2
h_c = head lost by contraction between sections 1 and 2
h_f'' = head lost by friction between sections 2 and 3
h_e = head lost by expansion between sections 2 and 3

Substituting $v = \frac{Q}{a}$ in equations (12) and (13), and rearranging:

$$\frac{Q^2}{2ga_2^2} + d_2 = \frac{Q^2}{2ga_1^2} + d_1 + (z_1 - z_2) - (h_f' + h_c) \qquad (14)$$

$$\frac{Q^2}{2ga_2^2} + d_2 = \frac{Q^2}{2ga_3^2} + d_3 - z_2 + (h_f'' + h_e) \qquad (15)$$

426963 O - 57 - 10

h_f', h_c, h_f'' and h_e are evaluated within sufficiently close limits by the following equations:

$$h_f' = \frac{\ell_1}{2}\left[\left(\frac{Q}{K_{d_1}}\right)^2 + \left(\frac{Q}{K_{d_2}}\right)^2\right] \tag{16}$$

where ℓ_1 = distance between sections 1 and 2 in feet

$$K_{d_1} = \frac{1.486}{n}\, a_1\, r_1^{2/3} \quad \text{(at section 1)}$$

$$K_{d_2} = \frac{1.486}{n}\, a_2\, r_2^{2/3} \quad \text{(at section 2)}$$

$$h_c = K_3 \frac{Q^2}{2ga_2^2} \tag{17}$$

K_3 depends on $\sqrt{\frac{a_1}{a_2}}$ and may be taken from figure 5.5-3, page 5.5-8, NEH-5

$$h_f'' = \frac{\ell_2}{2}\left[\left(\frac{Q}{K_{d_2}}\right)^2 + \left(\frac{Q}{K_{d_3}}\right)^2\right] \tag{18}$$

where ℓ_2 = distance between sections 2 and 3 in feet

K_{d_2} and $K_{d_3} = \frac{1.486}{n}\, a\, r^{2/3}$ at sections 2 and 3 respectively.

$$h_e = K_2 \frac{Q^2}{2ga_3^2} \tag{19}$$

K_2 depends on $\frac{a_3}{a_2}$ and may be taken from figure 3.14-17.

Substituting the expressions for h_f', h_f'', h_c and h_e from equations 16, 17, 18, and 19 in 14 and 15.

$$\frac{Q^2}{2ga_2^2} + d_2 = \frac{Q^2}{2ga_1^2} + d_1 + (z_1 - z_2) - \frac{\ell_1}{2}\left[\left(\frac{Q}{K_{d_1}}\right)^2 + \left(\frac{Q}{K_{d_2}}\right)^2\right] - K_3 \frac{Q^2}{2ga_2^2} \tag{20}$$

$$\frac{Q^2}{2ga_2^2} + d_2 = \frac{Q^2}{2ga_3^2} + d_3 - z_2 + \frac{\ell_2}{2}\left[\left(\frac{Q}{Kd_2}\right)^2 + \left(\frac{Q}{Kd_3}\right)^2\right] + K_2 \frac{Q^2}{2ga_3^2} \qquad (21)$$

Steps in the first method are:

1. Develop the data and plot curves of depth versus area, and depth versus K_d, for sections 1 and 3. When Doubt's method is used for determining water surface profiles, depth, area, and K_d values can be taken from table 3.14-1; $K_d = \frac{Q}{s_o^{1/2}}$ in that table. When Escoffier's method is used, the curves can be plotted from table 3.14-3. When Leach's method or the Step method is used, the curves can be plotted from table 3.14-5.

2. Develop data and plot curves of depth versus area, depth versus K_d, and depth versus critical discharge for section 2.

(a). The area computed for different depths at a bridge opening should be the net area; that is, total area minus the area of pile bents or piers. See figure 3.14-4, sheet 2.

(b). The wetted perimeter for different depths should include the boundaries or piers or pile bents; that is, the total wetted perimeter at a given depth equals the cross section boundary plus 2 times the wetted length of piers or pile bents at that depth. See figure 3.14-4, sheet 2.

(c). Determine critical discharge using the alignment chart, ES-75. Net area and net top width are used in this determination.

3. Plot depth versus discharge at section 3. Data for this curve are available from the water-surface elevations determined at section 3.

The remaining steps apply to each of a number of discharges selected to cover the necessary range.

4. In equation (21) evaluate $\frac{Q^2}{2ga_3^2} + d_3 - z_2$ for two values of d_3, one about 0.5 foot less than, and the other about 0.5 foot greater than the known depth for Q at section 3 as shown by the curve plotted in step 3. Plot $\frac{Q^2}{2ga_3^2} + d_3 - z_2$ on the "energy" scale versus d_3 as shown schematically on figure 3.14-18.

5. Evaluate $\frac{Q^2}{2ga_2^2} + d_2$ on the left side of equations 20 and 21 for d_2 ranging from d_c at section 2 (d_c is taken from the curve of depth versus critical discharge plotted in step 2) to a depth that extends the energy curve at section 2 up through the range covered by the energy values computed in step 4. Plot these values of energy versus depth at section 2 as shown schematically on figure 3.14-18.

6. From the plot made in steps 4 and 5 read the two values of d_2 associated with the two depths at section 3 used in step 4. See figure 3.14-18.

7. Compute $\frac{\ell_2}{2}\left[\left(\frac{Q}{K_{d_2}}\right)^2 + \left(\frac{Q}{K_{d_3}}\right)^2\right]$ and $K_2 \frac{Q^2}{2ga_3^2}$ in equation (21).

Evaluate these two terms for each of the two pairs of d_3 and d_2 determined in steps 4 and 6. Add the values of these two terms, which represent friction and expansion losses between sections 2 and 3, to the values of

$$\frac{Q^2}{2ga_3^2} + d_3 - z_2$$

computed in step 4, which evaluates the right-hand side of equation (21). Plot these values versus d_3 on figure 3.14-18 and connect the two points by a straight line. Where the vertical line representing d_3 for Q intersects this curve, draw the horizontal line A.

8. In equation (20) evaluate $\frac{Q^2}{2ga_1^2} + d_1 + (z_1 - z_2)$ through a range that spans the energy value of line A. Two values of d_1 within 1 to 2 feet of each other that will cause $\frac{Q^2}{2ga_1^2} + d_1 + (z_1 - z_2)$ to be greater and less than the energy value of line A can be selected by inspection of figure 3.14-18. Plot $\frac{Q^2}{2ga_1^2} + d_1 + (z_1 - z_2)$ on the energy scale versus d_1.

9. On figure 3.14-18 read two values of d_2 associated with the two values of d_1 used in step 8.

10. Compute $\frac{\ell_1}{2}\left[\left(\frac{Q}{K_{d_1}}\right)^2 + \left(\frac{Q}{K_{d_2}}\right)^2\right]$ and $K_3 \frac{Q^2}{2ga_2^2}$ in equation (20).

Evaluate these two terms for each of the two pairs of d_1 and d_2 obtained in step 9. The values of these two terms, which represent friction and contraction losses between sections 1 and 2, are subtracted from the values of

$$\frac{Q^2}{2ga_1^2} + d_1 + (z_1 - z_2)$$

computed in step 8, which evaluates the right-hand side of equation (20). Plot these values versus d_1 on figure 3.14-18 and connect plotted points with a straight line. The point at which the horizontal line A intersects this line determines d_1, the depth at the upstream section, for the Q under consideration.

Cases in which critical flow occurs in the bridge opening may be detected in steps 6 and 7. The conditions encountered at the completion of step 5 are shown schematically on figure 3.14-19, with the energy curve for section 2 not extending low enough to span the range of energy at section 3 for depths adjacent to the known d_3 for Q. In step 6, use d_c at section 2 in association with the d_3 used to define the lower of the two points obtained by step 4, then compute h_f' and h_e as described in step 7. If the point representing d_3 for Q on the line obtained in step 7 shows an energy value less than the point representing d_c at section 2, draw line A through the latter point and proceed with steps 8, 9, and 10 to determine d_1. This solution causes d_1 to depend on d_c at section 2 since the stage at section 1 is independent of the stage at section 3 when critical flow occurs at the bridge opening.

<u>Second Method</u>. This method is based on the contracted opening formula:

$$Q = C\, Ma_2 h^{1/2} \qquad (22)$$

where Q = discharge in cfs

a_2 = cross-sectional area of flow in the contracted section; that is, the cross-sectional area of flow at section 2, in square feet.

h = drop in water surface or the difference between the stages at sections 1 and 2, in feet

$$M = \sqrt{\frac{2g}{1 - \left(\frac{a_2}{a_1}\right)^2}}$$ may be taken from figure 3.14-20

a_1 = cross-sectional area of flow at section 1, in square feet

g = acceleration of gravity

C = coefficient of discharge.

In estimating C, if conditions are such that flow approaches the bridge opening with relatively low turbulence, the appropriate value of C is about 0.90. In the majority of cases C probably is in the 0.70 to 0.90 range. For very poor conditions (much turbulence), it may be as low as 0.40 to 0.50. In judging a given case, consider the following.

(1) Whether the abutments are square-cornered or shaped so as to reduce turbulence

(2) the number and shape of piers

(3) the degree of skew

(4) the number and spacing of pile bents since closely-spaced bents increase turbulence

(5) the existence of trees, drift, or other types of obstruction at the bridge or in the approach reach.

(6) C may decrease as discharge increases.

In the procedure for the second method, the stages at section 3, that is, the section just downstream from the bridge, for the discharges for which stages upstream are to be estimated, will be known from water surface profile determinations. Make the following computations for each discharge in a table with columns as described:

Column 1. List the known water surface elevation of section 3 for the discharge under consideration.

Column 2. List an assumed water-surface elevation of section 1.

Column 3. List a_1, the cross-sectional area at section 1 for the assumed water-surface elevation listed in column 2.

Column 4. Assume the water-surface elevation at section 2 as equal to the water-surface elevation at section 3, which is listed in column 1, and list a_2, the cross-sectional area at section 2 for this water surface.

Column 5. Compute $\frac{a_2}{a_1}$

Column 6. List M taken from figure 3.14-20.

Column 7. List the coefficient C.

Column 8. List the discharge Q for which h is being estimated.

Column 9. Compute $h = \left(\frac{Q}{CMa_2}\right)^2$.

Column 10. List the computed water-surface elevation at section 1, which is obtained by adding h in column 9 to the water-surface elevation in column 1. If this computed water-surface elevation is not within reasonably close agreement--that is, within ± 0.1 to 0.2 feet, with the assumed value in column 2, make a new assumption in column 2 and repeat the trial-and-error solution through column 10.

Illustrations of the computations to be made by the first and second methods are shown in Examples 1 and 2 below. These examples are based on the bridge crossing for State Highway 23, Hurricane Creek, Arkansas. The location of this bridge is shown on the profiles on Figure 3.14-16.

Example 1. Estimation of stage upstream from State Highway 23, Hurricane,Creek, Arkansas, by the first method.

Discharge, Q for 50 csm = 2505 cfs.

Bottom elev.	section	1	= 355.40
"	"	2	= 355.50
"	"	3	354.30

$z_1 = 355.40 - 354.30 = 1.10$

$z_2 = 355.50 - 354.30 = 1.20$

Stage for 2505 cfs at section 3 = 366.50, $d_3 = 12.20$

Figure 3.14-21 shows the plot of elevation versus area for the sections at the bridge.

Figure 3.14-22 shows the plot of elevation versus K_d for the sections at the bridge.

Figure 3.14-23 gives a curve of velocity versus velocity head from which $\frac{Q^2}{2ga^2}$ may be obtained.

The following computations show the work involved in steps 4 through 10 of the first method.

4. Evaluation of $\frac{Q^2}{2ga_3^2} + d_3 - z_2$

Elev.	d_3	a_3	$\frac{Q}{a_3}$	$\frac{Q^2}{2ga_3^2}$	$\frac{Q^2}{2ga_3^2} + d_3 - z_2$
367.00	12.70	1950	1.283	0.025	11.525
366.50	12.20				
366.00	11.70	1250	2.005	0.062	10.562

d_3 versus $\frac{Q^2}{2ga_3^2} + d_3 - z_2$ is plotted on figure 3.14-25.

5. Evaluation of $\frac{Q^2}{2ga_2^2} + d_2$

Elev.	d_2	a_2	$\frac{Q}{a_2}$	$\frac{Q^2}{2ga_2^2}$	$\frac{Q^2}{2ga_2^2} + d_2$
360.20	4.70	260	9.62	1.450	6.150
361.50	6.00	420	5.96	0.525	6.525
363.50	8.00	780	3.25	0.165	8.165
365.50	10.00	1280	1.95	0.059	10.059
367.50	12.00	1900	1.32	0.027	12.027

$d_2 = d_c = 4.70$ is taken from figure 3.14-24 which shows depth versus critical discharge at the bridge section.

d_2 versus $\frac{Q^2}{2ga_2^2} + d_2$ is plotted on figure 3.14-25.

6. Obtain depths d_2 associated with d_3 values in step 4. These values of d_2 are read from figure 3.14-25.

d_3	d_2
12.70	11.50
11.70	10.51

7. Compute $\frac{\ell_2}{2}\left[\left(\frac{Q}{K_{d_2}}\right)^2 + \left(\frac{Q}{K_{d_3}}\right)^2\right]$ and $K_2 \frac{Q^2}{2ga_3^2}$

$\ell_2 = 200.$

d_3	d_2	K_{d_2}	K_{d_3}	$\frac{\ell_2}{2}\left[\left(\frac{Q}{K_{d_2}}\right)^2 + \left(\frac{Q}{K_{d_3}}\right)^2\right]$
12.70	11.50	117,000	121,000	100(0.00045 + 0.00044) = 0.089
11.70	10.51	94,000	76,000	100(0.00072 + 0.0011) = 0.182

a_3	a_2	$\frac{a_3}{a_2}$	K_2	$\frac{Q}{a_3}$	$\frac{Q^2}{2ga_3^2}$	$K_2 \frac{Q^2}{2ga_3^2}$
1950	1710	1.14	0	1.283	0.025	0
1250	1410	*0.9	0	2.005	0.062	0

*Ratio $\frac{a_3}{a_2}$ indicates contraction. However, in either contraction or expansion the loss is negligible where the areas of flow are approximately equal.

Evaluation of right side of equation (21).

For d_3 = 12.70: 11.525 + 0.089 + 0 = 11.614

For d_3 = 11.70: 10.562 + 0.182 + 0 = 10.744

d_3 versus the values of the right side of equation are plotted on figure 3.14-25.

8. Evaluate $\frac{Q^2}{2ga_1^2} + d_1 + (z_1 - z_2)$

d_1	a_1	$\frac{Q}{a_1}$	$\frac{Q^2}{2ga_1^2}$	$z_1 - z_2$	$\frac{Q^2}{2ga_1^2} + d_1 + (z_1 - z_2)$
11.6	2300	1.09	0.018	-0.10	11.518
10.6	1120	2.23	0.078	-0.10	10.578

9. Obtain depths d_2 associated with d_1 values in step 8. These values are read from figure 3.14-25.

d_1	d_2
11.60	11.48
10.60	10.52

10. Compute $\frac{\ell_1}{2}\left[\left(\frac{Q}{K_{d_1}}\right)^2 + \left(\frac{Q}{K_{d_2}}\right)^2\right]$ and $K_3 \frac{Q^2}{2ga^2}$

$\ell_1 = 200$

d_1	d_2	K_{d_1}	K_{d_2}	$\frac{\ell_1}{2}\left[\left(\frac{Q}{K_{d_1}}\right)^2 + \left(\frac{Q}{K_{d_2}}\right)^2\right]$
11.60	11.48	118,000	118,000	100(0.00049 + 0.00049) = 0.098
10.60	10.52	63,000	94,000	100(0.0016 + 0.00074) = 0.234

a_1	a_2	$\left(\frac{a_1}{a_2}\right)^{1/2}$	K_3	$\frac{Q}{a_2}$	$\frac{Q^2}{2ga_2^2}$	$K_3 \frac{Q^2}{2ga_2^2}$
2300	1710	1.16	0.10	1.46	0.033	0.003
1120	1410	*0.89	-	1.77	.048	-

*Expansion, but loss is negligible.

Evaluation of right side of equation (20)

For d_1 = 11.60: 11.518 - 0.098 - 0.003 = 11.417

For d_1 = 10.60: 10.578 - 0.234 - 0.00 = 10.344

d_1 versus the values of the right side of equation (20) are plotted on figure 3.14-25. From this figure, d_1 for 2505 cfs is 11.36. The stage at section 1 is 355.40 + 11.36 = 366.76.

Example 2. Estimation of stage upstream from State Highway 23, Hurricane Creek, Arkansas, by the second method.

Discharge Q for 50 csm = 2505 cfs

Stage for 2505 cfs at section 3 = 366.50

The value of C = 0.60

1	2	3	4	5	6	7	8	9	10
W.S. elev. Sec.3	Assmd. elev. Sec.1	a_1	a_2	$\frac{a_2}{a_1}$	M	C	Q	h	Computed elev. sec. 1
366.50	366.60	1650	1550	0.94	23	0.60	2505	0.014	366.514
	366.55	1610	1550	0.96	28	0.60	2505	0.009	366.509

The limited degree of contraction in this case does not permit a reliable estimate of h. The friction head lost through the bridge is undoubtedly greater than the drop in water surface, h, immediately upstream from the contracted section.

Discussion. The first method recognizes the energy relationships associated with flow at bridge contractions. Estimates made by this method will be reliable from a practical standpoint. It probably will require slightly more time than the second method. However, the procedures are systematic and after working through two or three cases, a man will be able to make the necessary estimates for a given bridge in 2 to 4 hours.

The second method, which is based on the contracted opening formula, has definite limitations for this type of determination. First, the method assumes the stage at the contracted section to be the same as the stage at the downstream section. This assumption may result in significant error since it establishes the value of a_2 and affects the value of M. Note that M increases rapidly for values of $\frac{a_2}{a_1}$ greater than 0.6, thus, a minor error in estimating a_2 may result in a very significant error in h. Second, the method neglects friction loss. Third, firm criteria to guide the selection of C are not available since very few field determinations of C have been reported.

Overtopping of embankment. Where the embankment of a bridge is overtopped, the total discharge is equal to the discharge through the bridge opening plus the discharge over the embankment. A reliable estimate of upstream stage versus discharge under these conditions is difficult to obtain. However, an approach that should give reasonably good results is outlined below.

The work is divided into 3 major steps that are discussed in connection with the schematic sketch on figure 3.14-26. The stage versus discharge curve for section 3 (the section just downstream from the bridge) will have been established by previous water surface profile determinations.

Step 1 of this work is the determination of the stage-discharge curve at section 1, assuming that no discharge over the embankment occurs. Point a, figure 3.14-26, represents the stage and discharge at which the bridge opening flows full. Discharges through the opening greater than this value are usually considered to be under orifice control. Conditions at the bridge will determine the location of point a. The stage at which the bridge opening flows full may be higher or lower than the stage of the low point of the embankment. Part A of the stage-discharge curve at section 1 is defined by Method 1 or 2 of this section. Regardless of the position of point a, part C of the stage-discharge curve at section 1 is defined by orifice discharge relationships. Furthermore, part B, that is, the part of the stage-discharge curve at section 1 above the low point of the embankment, is determined assuming that no discharge occurs over the embankment. Part B of the curve is dashed to indicate that it is hypothetical. Orifice discharge is expressed by:

$$Q = Ca\sqrt{2g\Delta h} \qquad (23)$$

where Q = discharge in cfs
a = cross-sectional area of bridge opening in square feet
C = coefficient of discharge
Δh = water-surface elevation at section 1 minus water surface elevation at section 3

Table 3.14-1 Computation of U_2^+ and βU_1^-; Six Mile Creek

(Sheet 1 of 2)

(1) General Data	(2) d	(3) Elev.	(4) a	(5) p	(6) F	(7) $\frac{Q_{n,d}}{s_o^{1/2}}$	(8) a	(9) p	(10) F	(11) $\frac{Q_{n,d}}{s_o^{1/2}}$	(12) $\frac{Q_{n,d}}{s_o^{1/2}}$	(13) $\frac{s_o}{Q_{n,d}^2}$	(14) Σa	(15) $\frac{1}{(\Sigma a)^2}$	(16) T	(17) $\frac{\ell_d \delta s_o}{Q_{n,d_1}^2}$	(18) $\frac{\ell_u \delta s_o}{Q_{n,d_2}^2}$	(19) U_1^-	(20) U_2^+	(21) βU_1^-	(22) $Q_{c,d}$
Section No. 1				n = 0.06				n = 0.075													
bottom elev. 334.2	1.8	336.0	30	24	52	8.67×10^{2}					8.67×10^{2}	1.33×10^{-6}	30	1.11×10^{-3}	22		1.70×10^{-1}		1.71×10^{-1}		199
Sta. 165+00	3.8	338.0																	7.90×10^{-3}		
β	4.8	339.0																	3.00×10^{-3}		
ℓ_d	5.8	340.0	168	47	583	9.71×10^{3}					9.71×10^{3}	1.06×10^{-8}	168	3.54×10^{-5}	43		1.35×10^{-3}		1.39×10^{-3}		1885
ℓ_u 3960	6.8	341.0																	7.00×10^{-4}		
	7.8	342.0																	3.90×10^{-4}		
	9.8	344.0																	1.50×10^{-4}		
	11.3	345.5	460	69	2430	4.06×10^{4}					4.06×10^{4}	6.09×10^{-10}	460	4.73×10^{-6}	63		7.77×10^{-5}		8.24×10^{-5}		7050
	12.3	346.5																	6.00×10^{-5}		
	13.3	347.5																	4.55×10^{-5}		
	14.3	348.5																	3.55×10^{-5}		
	15.1	349.3	783	118	4120	6.87×10^{4}	0				6.87×10^{4}	2.12×10^{-10}	783	1.63×10^{-6}	107		2.70×10^{-5}		2.86×10^{-5}		12020
	16.1	350.3																	1.90×10^{-5}		
	17.1	351.3	1003	124	5990	9.97×10^{4}	308	368	407	5.42×10^{3}	1.05×10^{5}	9.07×10^{-11}	1311	5.81×10^{-7}	481		1.16×10^{-5}		1.22×10^{-5}		12280
	18.1	352.3																	6.5×10^{-5}		
	19.8	354.0	1308	124	9370	1.56×10^{5}	1762	685	4925	6.56×10^{4}	2.22×10^{5}	2.02×10^{-11}	3070	1.06×10^{-7}	798		2.58×10^{-6}		2.69×10^{-6}		34200
	21.8	356.0																	9.50×10^{-7}		
	23.8	358.0																	3.50×10^{-7}		
	25.8	360.0	1908	124	18750	3.12×10^{5}	9502	1450	49500	6.60×10^{5}	9.72×10^{5}	1.06×10^{-12}	11488	7.59×10^{-9}	1563		1.35×10^{-7}		1.43×10^{-7}		130875
	27.8	362.0																	7.4×10^{-8}		
	29.8	364.0	2438	124	26350	4.39×10^{5}	15432	1550	106221	1.42×10^{6}	1.85×10^{6}	2.91×10^{-13}	17870	3.14×10^{-9}	1663		3.71×10^{-8}		4.02×10^{-8}		254950
Section No. 2				n = 0.06				n = 0.08													
bottom elev. 335.9	2.1	338.0	25	19	45	7.5×10^{2}					7.5×10^{2}	1.78×10^{-6}	25	1.60×10^{-3}	17	2.27×10^{-1}	4.54×10^{-1}	-2.25×10^{-1}	4.56×10^{-1}	-1.22×10^{-1}	172
Sta. 204+60	3.1	339.0																	7.70×10^{-2}	-2.20×10^{-2}	
β 0.54	4.1	340.0																	2.15×10^{-2}	-5.80×10^{-2}	
ℓ_d 3960	5.1	341.0																	7.80×10^{-3}	-2.15×10^{-3}	
ℓ_u 7920	6.1	342.0	152	44	518	8.63×10^{3}					8.63×10^{3}	1.34×10^{-8}	152	4.33×10^{-5}	40	1.71×10^{-3}	3.42×10^{-3}	-1.67×10^{-3}	3.46×10^{-3}	-9.03×10^{-4}	1680
	7.1	343.0																	1.75×10^{-3}	-4.50×10^{-4}	
	8.1	344.0																	9.70×10^{-4}	-2.50×10^{-4}	
	9.1	345.0																	5.70×10^{-4}	-1.45×10^{-4}	
	10.1	346.0																	3.40×10^{-4}	-8.80×10^{-5}	
	11.1	347.0																	2.30×10^{-4}	-5.80×10^{-5}	
	12.1	348.0	497	82	2455	4.09×10^{4}					4.09×10^{4}	5.98×10^{-10}	497	4.05×10^{-6}	74	7.63×10^{-5}	1.53×10^{-4}	-7.23×10^{-5}	1.57×10^{-4}	-3.90×10^{-5}	7300
	13.1	349.0																	1.05×10^{-4}	-2.70×10^{-5}	
	14.1	350.0																	7.40×10^{-5}	-1.85×10^{-5}	
	15.3	351.2	796	112	4370	7.29×10^{4}	0				7.29×10^{4}	1.88×10^{-10}	796	1.58×10^{-6}	102	2.40×10^{-5}	4.79×10^{-5}	-2.24×10^{-5}	4.95×10^{-5}	-1.21×10^{-5}	12600
	16.3	352.2																	3.00×10^{-5}	-7.5×10^{-6}	
	17.1	353.0	980	112	6210	1.04×10^{5}	470	400	779	9.73×10^{3}	1.14×10^{5}	7.75×10^{-11}	1450	4.76×10^{-11}	502	9.88×10^{-6}	1.98×10^{-5}	-9.40×10^{-6}	2.03×10^{-5}	-5.08×10^{-6}	13900
	19.6	355.5																	3.00×10^{-6}	-6.60×10^{-7}	
	22.1	358.0	1490	112	12470	2.08×10^{5}	11940	2505	50355	6.29×10^{5}	8.37×10^{5}	1.43×10^{-12}	13430	5.54×10^{-9}	2607	1.82×10^{-7}	3.65×10^{-7}	-1.76×10^{-7}	3.71×10^{-7}	-9.52×10^{-8}	165420
	24.6	360.5																	1.38×10^{-7}	-3.4×10^{-8}	
	27.1	363.0	2000	112	20400	3.40×10^{5}	25065	2745	162693	2.03×10^{6}	2.37×10^{4}	1.78×10^{-13}	27065	1.37×10^{-9}	2847	2.27×10^{-8}	4.54×10^{-8}	-2.13×10^{-8}	4.68×10^{-8}	-1.15×10^{-8}	473200

424963 O-57 (Face p. 156) No. 1

Table 3.14-1 Computation of U_2^+ and U_1^-; Six Mile Creek

Table 3.14-1 Computation of U_2^+ and βU_1^-; Six Mile Creek

(1) General Data	(2) d ft.	(3) Elev. MSL	(4) a ft.^{2}	(5) p ft.	(6) F	(7) $\frac{Q_{n,d}}{S_o^{1/2}}$	(8) a ft.^{2}	(9) p ft.	(10) F	(11) $\frac{Q_{n,d}}{S_o^{1/2}}$	(12) $\frac{Q_{n,d}}{S_o^{1/2}}$	(13) $\frac{S_o}{Q_{n,d}^2}$	(14) Σa	(15) $\frac{1}{(\Sigma a)^2}$	(16) T	(17) $\frac{\ell_d g S_o}{Q_{n,d_1}^2}$	(18) $\frac{\ell_u g S_o}{Q_{n,d_2}^2}$	(19) U_1^-	(20) U_2^+	(21) βU_1^-	(22) $Q_{c,d}$
Section No. 3				n = 0.06				n = 0.07													
bottom elev. 337.0	4	341.0	90	41	225	3.75×10^3					3.75×10^3	7.11×10^{-8}	90	1.23×10^{-4}	37	1.81×10^{-2}	1.09×10^{-2}	-1.80×10^{-2}	1.10×10^{-2}	-1.77×10^{-2}	796
Sta. 283+80	5	342.0																		-6.60×10^{-3}	
β 0.986	6	343.0																		-2.90×10^{-3}	
ℓ_d 7920	7	344.0																		-1.45×10^{-3}	
ℓ_u 4750	8	345.0																		-7.90×10^{-4}	
	9	346.0																		-4.65×10^{-4}	
	10	347.0	360	60	1760	2.93×10^4					2.93×10^4	1.19×10^{-9}	360	7.71×10^{-6}	55	2.98×10^{-4}	1.79×10^{-4}	-2.90×10^{-4}	1.87×10^{-4}	-2.86×10^{-4}	5220
	11	348.0																		-1.90×10^{-4}	
	12	349.0																		-1.33×10^{-4}	
	13	350.0	572	86	3010	5.02×10^4	0	0	0	0	5.02×10^4	3.98×10^{-10}	572	3.06×10^{-6}	78	1.01×10^{-4}	6.09×10^{-5}	-9.80×10^{-5}	6.40×10^{-5}	-9.66×10^{-5}	8800
	14	351.0																		-6.10×10^{-5}	
	15	352.0																		-3.90×10^{-5}	
	16	353.0																		-2.60×10^{-5}	
	17.7	354.7	1070	126	6620	1.10×10^5	610	303	1445	2.07×10^4	1.31×10^5	5.82×10^{-11}	1680	3.54×10^{-7}	418	1.48×10^{-5}	8.90×10^{-6}	-1.44×10^{-5}	9.25×10^{-6}	-1.42×10^{-5}	19100
	18.7	355.7	1202	165	6730	1.12×10^5	1320	800	2740	3.92×10^4	1.51×10^5	4.38×10^{-11}	2552	1.58×10^{-7}	950	1.12×10^{-5}	6.70×10^{-6}	-1.10×10^{-5}	6.86×10^{-6}	-1.08×10^{-5}	23375
	20	357.0																		-2.80×10^{-6}	
	21	358.0																		-1.05×10^{-6}	
	22	359.0																		-4.40×10^{-7}	
	23	360.0	1847	165	13770	2.28×10^5	13687	2655	60731	8.68×10^5	1.10×10^6	8.30×10^{-13}	15534	4.14×10^{-9}	2805	2.12×10^{-7}	1.27×10^{-7}	-2.08×10^{-7}	1.31×10^{-7}	-2.05×10^{-7}	207330
	24	361.0																		-1.30×10^{-7}	
	26	363.0																		-5.45×10^{-8}	
	28	365.0	2597	165	24200	4.03×10^5	27187	2745	185637	2.65×10^6	3.05×10^6	1.07×10^{-13}	29784	1.13×10^{-9}	2895	2.73×10^{-8}	1.64×10^{-8}	-2.62×10^{-8}	1.75×10^{-8}	-2.60×10^{-8}	541800
	30	367.0																		-1.43×10^{-8}	
	32	369.0																		-8.60×10^{-9}	
	33	370.0	3347	165	37100	6.19×10^5	41125	2830	363554	5.19×10^6	5.81×10^6	2.96×10^{-14}	44472	5.06×10^{-10}	2980	7.55×10^{-9}	4.53×10^{-9}	-7.04×10^{-9}	5.04×10^{-9}	-6.94×10^{-9}	974260

Table 3.14-2. Determination of Control Sections and Record of Water Surface Elevations, Six Mile Creek

(1) Section	(2) Unit Area runoff (csm)	(3) Drainage Area (sq.mi.)	(4) β	(5) Q_2	(6) $\frac{Q_2^2}{2g}$	(7) d_c	(8) $\frac{Q_m d_c}{s_c^{1/2}}$	(9) $s_{\phi_c}^{1/2}$	(10) s_c	(11) s (upstream)	(12) s (downstream)	(13) Remarks	(14) Water Surface Elevation
No. 1	2	251.6	-	503	3.93×10^3					0.00043			343.1
	5			1258	2.46×10^4	4.7	6.3×10^3	0.2000	0.0400			$s_c>s_u$	348.6
	10			2516	9.83×10^4							Not	351.7
	20			5032	3.93×10^5							a	354.3
	50			12580	2.46×10^6							Control	357.4
	100			25160	9.83×10^6								360.7
	150			37740	2.20×10^7								363.7
	200			50320	3.93×10^7	21.2	3.5×10^5	0.1437	0.0206			$s_c>s_u$	365.8
No. 2	2	184.7	0.54	369	2.12×10^3					0.00014	0.00043		344.6
	5			924	1.32×10^4	4.6	4.5×10^3	0.2054	0.0421			$s_u<s_c>s_d$	349.9
	10			1847	5.30×10^4								353.0
	20			3694	2.12×10^5							Not	355.4
	50			9235	1.32×10^6							a	358.6
	100			18470	5.30×10^6							Control	362.0
	150			27700	1.19×10^7								364.8
	200			36940	2.12×10^7	19.0	2.4×10^5	0.1540	0.0237			$s_u<s_c>s_d$	367.0
No. 3	2	183.5	0.986	367	2.09×10^3						0.00014		346.7
	5			918	1.31×10^4	4.3	4.4×10^3	0.2084	0.0435			$s_u<s_c>s_d$	351.5
	10			1835	5.23×10^4								354.8
	20			3670	2.09×10^5							Not	356.8
	50			9175	1.31×10^6							a	359.4
	100			18350	5.23×10^6							Control	362.7
	150			27525	1.17×10^7								365.3
	200			36700	2.09×10^7	19.8	2.7×10^5	0.1360	0.0185			$s_u<s_c>s_d$	367.5

CODEX BOOK COMPANY, INC. NORWOOD, MASSACHUSETTS. PRINTED IN U.S.A.

426963 O - 57 (Face p. 158) No. 2

Table 3.14-3. Computation of P values; Six Mile Creek

(1) General Data	(2) d	(3) Elev.	(4) a	(5) p	(6) F	(7) $\frac{Q_{n,d}}{s_o^{1/2}}$	(8) a	(9) p	(10) F	(11) $\frac{Q_{n,d}}{s_o^{1/2}}$	(12) $\frac{Q_{n,d}}{s_o^{1/2}}$	(13) P	(14) βP
			n = 0.06				n = 0.075						
Section 1	1.8	336.0	30	24	52	8.67×10^{2}					8.67×10^{2}	1.33×10^{-6}	
bottom elev. 334.2	3.8	338.0										6.10×10^{-8}	
Sta. 165+00	4.8	339.0										2.32×10^{-8}	
β	5.8	340.0	168	47	582	9.71×10^{3}					9.71×10^{3}	1.06×10^{-8}	
ℓ_d	6.8	341.0										5.25×10^{-9}	
ℓ_u 3960	7.8	342.0										2.95×10^{-9}	
	9.8	344.0										1.09×10^{-9}	
	11.3	345.5	460	69	2430	4.06×10^{4}					4.06×10^{4}	6.09×10^{-10}	
	12.3	346.5										4.50×10^{-10}	
	13.3	347.5										3.38×10^{-10}	
	14.3	348.5										2.60×10^{-10}	
	15.1	349.3	783	118	4120	6.87×10^{4}	0	0	0	0	6.87×10^{4}	2.12×10^{-10}	
	16.1	350.3										1.40×10^{-10}	
	17.1	351.3	1003	124	5990	9.97×10^{4}	308	368	407	5.42×10^{3}	1.05×10^{5}	9.08×10^{-11}	
	18.1	352.3										5.00×10^{-11}	
	19.8	354.0	1308	124	9370	1.57×10^{5}	1762	685	4925	6.56×10^{4}	2.22×10^{5}	2.02×10^{-11}	
	21.8	356.0										7.40×10^{-12}	
	23.8	358.0										2.75×10^{-12}	
	25.8	360.0	1908	124	18750	3.12×10^{5}	9502	1450	49500	6.60×10^{5}	9.72×10^{5}	1.06×10^{-12}	
	27.8	362.0										5.60×10^{-13}	
	29.8	364.0	2438	124	26350	4.39×10^{5}	15432	1550	106221	1.42×10^{6}	1.85×10^{6}	2.91×10^{-13}	
Section 2				n = 0.06			n = 0.08						
bottom elev. 335.9	2.1	338.0	25	19	45	7.50×10^{2}					7.50×10^{2}	1.78×10^{-6}	9.60×10^{-7}
Sta. 204+60	3.1	339.0										3.00×10^{-7}	1.62×10^{-7}
β 0.54	4.1	340.0										8.20×10^{-8}	4.43×10^{-8}
ℓ_d 3960	5.1	341.0										3.00×10^{-8}	1.62×10^{-8}
ℓ_u 7920	6.1	342.0	152	44	518	8.63×10^{3}					8.63×10^{3}	1.34×10^{-8}	7.22×10^{-9}
	7.1	343.0										6.80×10^{-9}	3.67×10^{-9}
	8.1	344.0										3.70×10^{-9}	2.00×10^{-9}
	9.1	345.0										2.21×10^{-9}	1.19×10^{-9}
	10.1	346.0										1.33×10^{-9}	7.19×10^{-10}
	11.1	347.0										8.80×10^{-10}	4.75×10^{-10}
	12.1	348.0	497	82	2455	4.09×10^{4}					4.09×10^{4}	5.98×10^{-10}	3.23×10^{-10}
	13.1	349.0										4.10×10^{-10}	2.22×10^{-10}
	14.1	350.0										2.80×10^{-10}	1.51×10^{-10}
	15.3	351.2	796	112	4370	7.29×10^{4}	0	0	0	0	7.29×10^{4}	1.88×10^{-10}	1.02×10^{-10}
	16.3	352.2										1.17×10^{-10}	6.32×10^{-11}
	17.1	353.0	980	112	6210	1.04×10^{5}	470	400	779	9.73×10^{3}	1.14×10^{5}	7.75×10^{-11}	4.18×10^{-11}
	19.6	355.5										1.05×10^{-11}	5.67×10^{-12}
	22.1	358.0	1490	112	12470	2.08×10^{5}	11940	2505	50355	6.29×10^{5}	8.37×10^{5}	1.43×10^{-12}	7.73×10^{-13}
	24.6	360.5										5.20×10^{-13}	2.81×10^{-13}
	27.1	363.0	2000	112	20400	3.40×10^{5}	25065	2745	162693	2.03×10^{6}	2.37×10^{6}	1.78×10^{-13}	9.60×10^{-14}

Table 3.14-4. Computed Water Surface Elevations for Escoffier's Method; Six Mile Creek

From	To	length	Q	$\frac{Q^2\ell}{2}$	Elev.$_2$	Q	$\frac{Q^2\ell}{2}$	Elev.$_2$	Q	$\frac{Q^2\ell}{2}$	Elev.$_2$	Q	$\frac{Q^2\ell}{2}$	Elev.$_2$
			2 csm			5 csm			10 csm			20 csm		
Sec. 1	Sec. 2	3960	503	5.01×10^{8}	343.1	1260	3.14×10^{9}	348.6	2516	1.25×10^{10}	351.7	5032	5.01×10^{10}	354.3
2	3	7920	369	5.40×10^{8}	344.6	924	3.38×10^{9}	349.8	1847	1.35×10^{10}	353.1	3694	5.40×10^{10}	355.4
3	4	4750	367	3.19×10^{8}	346.8	917	2.00×10^{9}	351.6	1835	8.00×10^{9}	354.9	3670	3.19×10^{10}	356.7
			50 csm			100 csm			150 csm			200 csm		
Sec. 1	Sec. 2		12580	3.13×10^{11}	357.4	25160	1.25×10^{12}	360.7	37700	2.81×10^{12}	363.7	50320	5.01×10^{12}	365.8
2	3		9235	3.38×10^{11}	358.6	18470	1.35×10^{12}	362.0	27700	3.04×10^{12}	364.7	36940	5.40×10^{12}	366.8
3	4		9175	2.00×10^{11}	359.4	18350	8.00×10^{11}	362.7	27500	1.80×10^{12}	365.3	36700	3.19×10^{12}	367.3

CODEX BOOK COMPANY, INC. NORWOOD, MASSACHUSETTS. PRINTED IN U.S.A.

426963 O - 57 (Face p. 158) No. 3

Table 314-5. Computation of K_d Values for Leach's or Step Method; Six Mile Creek

(1) General Data	(2) d	(3) Elev.	(4) a	(5) p	(6) F	(7) K_d	(8) a	(9) p	(10) F	(11) K_d	(12) ΣK_d
				n = 0.06				n = 0.075			
Section 1	1.8	336.0	30	24	52	8.67×10^2					8.67×10^2
bottom elev.334.2	3.8	338.0									4.05×10^3
Sta. 165+00	4.8	339.0									6.55×10^3
	5.8	340.0	168	47	582	9.71×10^3					9.71×10^3
	6.8	341.0									1.37×10^4
	7.8	342.0									1.81×10^4
	9.8	344.0									2.98×10^4
	11.3	345.5	460	69	2430	4.06×10^4					4.06×10^4
	12.3	346.5									4.70×10^4
	13.3	347.5									5.45×10^4
	14.3	348.5									6.20×10^4
	15.1	349.3	783	118	4120	6.87×10^4	0	0	0	0	6.87×10^4
	16.1	350.3									8.60×10^4
	17.1	351.3	1003	124	5990	9.97×10^4	308	368	407	5.42×10^3	1.05×10^5
	18.1	352.3									1.45×10^5
	19.8	354.0	1308	124	9370	1.57×10^5	1762	685	4925	6.56×10^4	2.22×10^5
	21.8	356.0									3.80×10^5
	23.8	358.0									6.10×10^5
	25.8	360.0	1908	124	18750	3.12×10^5	9502	1450	49500	6.60×10^5	9.72×10^5
	27.8	362.0									1.31×10^6
	29.8	364.0	2438	124	26350	4.39×10^5	15432	1550	106221	1.42×10^6	1.85×10^6
Section 2				n = 0.06				n = 0.08			
bottom elev.335.9	2.1	338.0	25	19	45	7.50×10^2					7.50×10^2
Sta. 204+60	3.1	339.0									1.93×10^3
	4.1	340.0									3.48×10^3
	5.1	341.0									5.80×10^3
	6.1	342.0	152	44	518	8.63×10^3					8.63×10^3
	7.1	343.0									1.22×10^4
	8.1	344.0									1.64×10^4
	9.1	345.0									2.11×10^4
	10.1	346.0									2.70×10^4
	11.1	347.0									3.33×10^4
	12.1	348.0	497	82	2455	4.09×10^4					4.09×10^4
	13.1	349.0									4.95×10^4
	14.1	350.0									5.95×10^4
	15.3	351.2	796	112	4370	7.29×10^4	0	0	0	0	7.29×10^4
	16.3	352.2									9.30×10^4
	17.1	353.0	980	112	6210	1.04×10^5	470	400	779	9.73×10^3	1.14×10^5
	19.6	355.5									3.00×10^5
	22.1	358.0	1490	112	12470	2.08×10^5	11940	2505	50355	6.29×10^5	8.37×10^5
	24.6	360.5									1.38×10^6
	27.1	363.0	2000	112	20400	3.40×10^5	25065	2745	162693	2.03×10^6	2.37×10^6

Table 3.14-6. Computation of W.S. Elevations at Foot and Head of Reaches for Leach's Method.

csm =

Foot Station	Head Station	Mid. Sec. Station	Assumed Elev. at Mid. Sec.	K_d at Mid. Sec.	$\frac{Q}{K_d}$	$S = \frac{Q^2}{K_d}$	Distance to Foot of Reach	Distance to Head of Reach	Drop to Foot of Reach	Rise to Head of Reach	W.S. Elev. Foot	W.S. Elev. Head
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
			Reach 1. Mid-section is _____. Q = Area above mid-section times the selected csm rate.									
			Reach 2. Mid-section is _____. Q = _____.									
Take Station from profile												
From profile												
From profile			Assume elevations through a range of 3 to 5 feet and at intervals of 0.5 to 1.0 feet. For small Q take this range relatively low in the section and move the range up for larger discharges.									
				List K_d taken from plot made in Step 4, a, for each elevation in column 4.								
					Compute by slide rule or calculator							
						S may be computed or taken from table on page 865, King's Handbook, 4th edition; Table 144, page 535, King's Handbook, 3rd edition.						
							Difference between columns 1 and 3.					
								Difference between columns 2 and 3.				
									Column 7 times column 8.			
										Column 7 times column 9.		
											Column 4 minus column 10.	
												Column 4 plus column 11.

CODEX BOOK COMPANY, INC. NORWOOD, MASSACHUSETTS. PRINTED IN U.S.A.

426963 O-57 (Face p. 158) No. 4

Table 3.14-7 Computation of W.S. Profiles by Step Method; Six Mile Creek

(1) From (foot)	(1) To (head)	(2) ℓ	(3) $\frac{\ell}{2}$	(4) E_2 (foot)	(5) Assm'd. E_1 (head)	(6) Discharge Q_2	(7) Discharge Q_1	(8) K_{d_2}	(9) K_{d_1}	(10) $\frac{Q_2^2}{K_{d2}^2}$	(11) $\frac{Q_1^2}{K_{d1}}$	(12) s_f	(13) h_f	(14) Comp'd E_1	(15) Remarks
Sec. 1	Sec. 2														
A_1=251.6	A_2=184.7	3960	1980	343.1	344.5	502	369	24700	18700	0.000413	0.000390	0.000803	1.59	344.69	
2 csm		"	"	"	(344.6)	"	"	"	(19200)	"	(0.000370)	0.000783	1.55	(344.65)	
		"	"	"	344.7	"	"	"	19800	"	0.000347	0.000760	1.50	344.60	
5 csm		"	"	348.6	350.0	1260	924	62500	58000	0.000406	0.000254	0.000660	1.31	349.91	
		"	"	"	(349.9)	"	"	"	(57500)	"	(0.000258)	0.000664	1.31	(349.91)	
10 csm		"	"	351.7	353.0	2516	1847	118000	114000	0.000453	0.000262	0.000715	1.42	353.12	
		"	"	"	(353.1)	"	"	"	(117000)	"	(0.000249)	0.000702	1.39	(353.09)	
20 csm		"	"	354.3	355.2	5032	3694	243000	260000	0.000428	0.000202	0.000630	1.25	355.55	
		"	"	"	355.4	"	"	"	290000	"	0.000162	0.000590	1.17	355.47	
		"	"	"	(355.5)	"	"	"	(300000)	"	(0.000152)	0.000580	1.15	(355.45)	
50 csm		"	"	357.4	(358.6)	12580	9235	550000	(960000)	0.000522	(0.000092)	0.000614	1.22	(358.62)	
100 csm		"	"	360.7	362.0	25160	18470	1110000	1880000	0.000512	0.000097	0.000609	1.20	361.90	
		"	"	"	(361.9)	"	"	"	(1840000)	"	(0.000101)	0.000613	1.21	(361.91)	
150 csm		"	"	363.7	(364.7)	37740	27705	1790000	(3500000)	0.000445	(0.000063)	0.000508	1.00	(364.70)	
200 csm		"	"	365.8	(366.8)	50320	36940	2400000	(5600000)	0.000439	(0.000044)	0.000483	0.96	(365.76)	

Table 3.14-8. Water Surface Elevations Determined by Different Methods; Six Mile Creek Watershed

Q csm	Method	Sec.1	Sec.2	Sec.3	Sec.4	Sec.5 2/	Sec.6	Sec.7	Sec.8	Below RR	Above RR	Below Hy.22	Sec.5 2/	Sec.1	Sec.2	Below Hy.23	Above Hy.23	Sec.3
2	Doubt	343.1	344.6	346.7	347.4	347.6	351.5	353.9	358.5	359.4	360.1	365.0	347.6	351.0	354.7	357.9	359.1	359.4
	Step		344.6	346.8	347.5	347.8	351.6	353.9	358.7	359.7	360.3	366.0	347.8	351.4	355.5	359.1	359.9	360.1
	Escoffier 1/		344.6	346.8														
5	Doubt	348.6	349.9	351.5	352.3	352.6	355.2	357.2	362.6	363.2	363.5	366.9	352.6	354.2	356.7	361.0	361.2	361.5
	Step		349.9	351.6	352.6	352.9	355.4	357.3	362.7	363.3	363.6	367.1	352.9	354.4	356.9	361.1	361.3	361.7
	Escoffier 1/		349.8	351.6														
10	Doubt	351.7	353.0	354.8	355.8	356.0	358.3	360.2	365.8	366.2	366.4	368.6	356.0	357.4	359.0	362.8	363.0	363.4
	Step		353.1	354.9	356.0	356.3	358.3	360.3	365.9	366.4	366.4	369.0	356.3	357.4	358.8	362.9	363.1	363.5
	Escoffier 1/		353.1	354.9														
20	Doubt	354.3	355.4	356.8	358.0	358.4	360.2	362.8	368.9	369.1	369.3	371.0	358.4	359.3	360.4	364.3	364.6	365.3
	Step		355.5	356.9	358.3	358.6	360.1	362.8	369.4	369.6	368.8	371.3	358.6	359.4	360.6	364.6	364.9	365.5
	Escoffier 1/		355.4	356.7														
50	Doubt	357.4	358.6	359.4	360.9	361.4	362.7	365.4	372.5	372.6	372.8	374.2	361.4	362.4	363.2	366.5	366.8	367.7
	Step		358.6	359.6	361.2	361.4	362.7	365.4	372.4	372.6	372.8	374.2	361.4	362.5	363.3	366.6	366.8	367.2
	Escoffier 1/		358.6	359.4														
100	Doubt	360.7	362.0	362.7	364.3	364.9	365.7	368.0	374.3	374.5	374.9	376.4	364.9	365.7	366.4	368.4	368.7	369.4
	Step		361.9	362.6	364.6	365.0	365.7	368.0	374.2	374.5	374.9	376.4	365.0	365.8	366.4	368.9	369.2	369.4
	Escoffier 1/		362.0	362.7														
150	Doubt	363.7	364.8	365.3	366.7	367.3	368.0	369.7	375.2	375.4	375.9	377.8	3/					
	Step		364.7	365.3	367.3	367.7	368.3	369.9	375.0	375.2	375.7	377.6						
	Escoffier 1/		364.7	365.3														
200	Doubt	365.8	367.0	367.5	368.7	3/												
	Step		366.8	367.3	369.3													
	Escoffier 1/		366.8	367.3														

1/ Profiles computed by Escoffier's Method only to section 3 on Six Mile Creek. If computed for all sections the W.S. elevations would be approximately the same as those shown for the Step Method.

2/ Section 5 on Six Mile Creek is just downstream from the confluence of Hurricane Creek, which is a tributary to Six Mile Creek.

3/ Cross sections were not surveyed high enough in elevation to permit determination of W.S. elevations for 150 and 200 csm for the sections upstream from Section 5.

426963 O-57 (Face p. 158) No. 5

Assuming no discharge over the embankment:

$$\Delta h = \frac{Q^2}{2gC^2a^2} \tag{24}$$

Since the water-surface elevation at section 3 is known, the water-surface elevations at section 1 for selected discharges may be quickly computed after values of Δh are determined.

Step 2 requires the determination of stage versus free discharge over the embankment. Curve D represents the free discharge determined by

$$Q_f = 3.1\ C\ T\ H_e^{3/2} \tag{25}$$

where Q_f = discharge in cfs

T = top width of flow over embankment in feet

H_e = energy head in feet

C = coefficient - the following approximate values are recommended for use:

For road and highway fills, C = 0.80 to 0.90

For single-track railroad fills, C = 0.70 to 0.80

For double-track railroad fills, C = 0.60 to 0.70

To compute stage at section 1 versus free discharge:

(1) Measure the cross-sectional area, a, and T for various stages in the cross section at the center line of the embankment.

(2) Compute $H_e = \cancel{\frac{3a}{2T}}\ d_c + \frac{a}{2T}$ for each stage.

(3) Compute Q_f by equation (25).

(4) Compute stage at section 1 by adding H_e to the elevation of the low point on the embankment.

Step 3 requires the determination of that part of the stage-versus-discharge curve at section 1 that represents discharge over the embankment plus discharge through the bridge opening. This part of the curve is shown as E on figure 3.14-26 and is computed by the set of tabulated computations described below. The following symbols are used:

Q_f = free discharge over embankment

Q_o = discharge through bridge opening

Q_t = total discharge = $Q_o + Q_f$

H_2 = submergence = water-surface elevation at section 3 for Q_t minus elevation of low point of embankment

$\frac{H_2}{H_e}$ = submergence ratio

Q_s = submerged discharge over embankment

R = Q_s/Q_f, so that $Q_s = R\ Q_f$

The following tabulations and computations are made:

<u>Column 1.</u> Water-surface elevation at section 1 at about 1-foot intervals.

<u>Column 2.</u> Q_f for the elevations in Col. 1. Obtain from Curve D.

<u>Column 3.</u> Q_o for the elevations in Col. 1. Obtain from Curve C.

<u>Column 4.</u> $Q_t = Q_f + Q_o$

<u>Column 5.</u> H_e for the discharges in Col. 2.

<u>Column 6.</u> Water-surface elevation at section 3 for the total discharges in Col. 4.

<u>Column 7.</u> H_2 = Col. 6 minus elevation of low point of embankment.

<u>Column 8.</u> Submergence ratio = $\frac{H_2}{H_e} = \frac{\text{Col. 7}}{\text{Col. 5}}$

<u>Column 9.</u> R for the various submergence ratios. Take from figure 3.4 of Drop Spillway Section, NEH-11.

<u>Column 10.</u> $Q_s = R\ Q_f$ = Col. 9 X Col. 2.

<u>Column 11.</u> Corrected $Q_t = Q_o + Q_s$ = Col. 3 + Col. 10. Plot curve E from Cols. 1 and 11 to complete the stage-discharge curve at section 1.

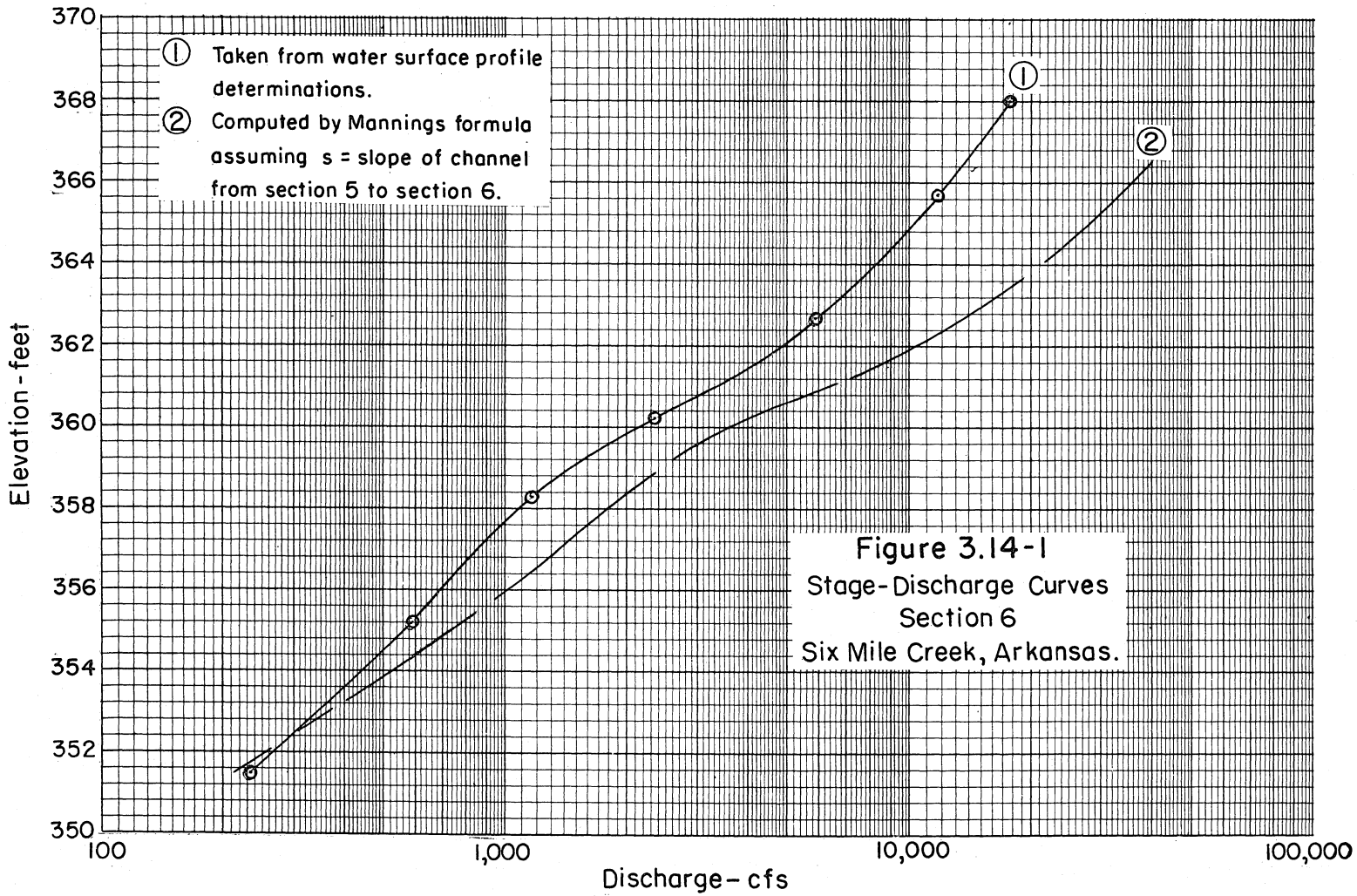


Figure 3.14-1
Stage-Discharge Curves
Section 6
Six Mile Creek, Arkansas.

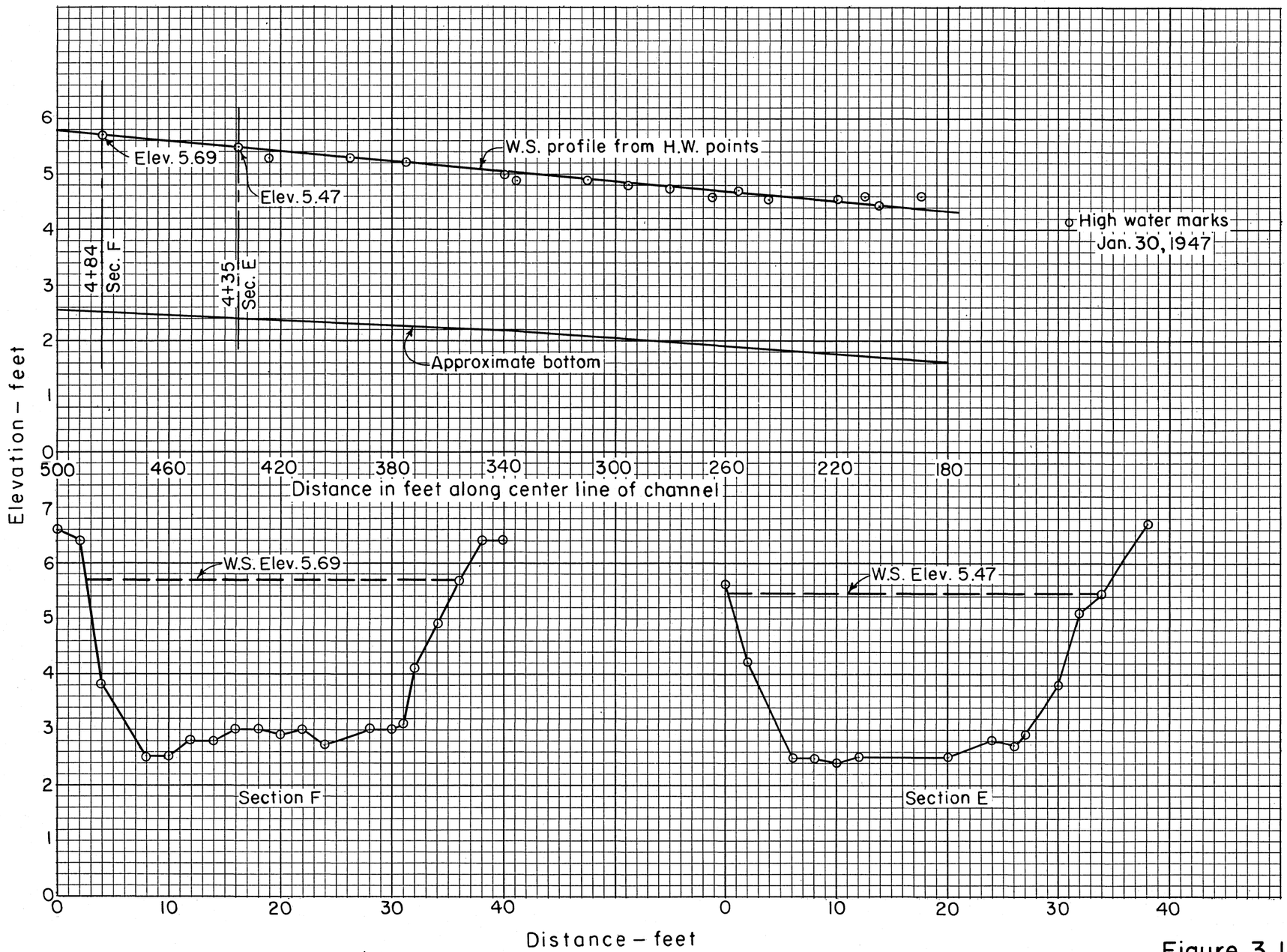


Figure 3.14-2
Profile and Cross Sections
Touby Run, Mansfield, Ohio.

426963 O-57 (Face p. 160 blank) No. 1

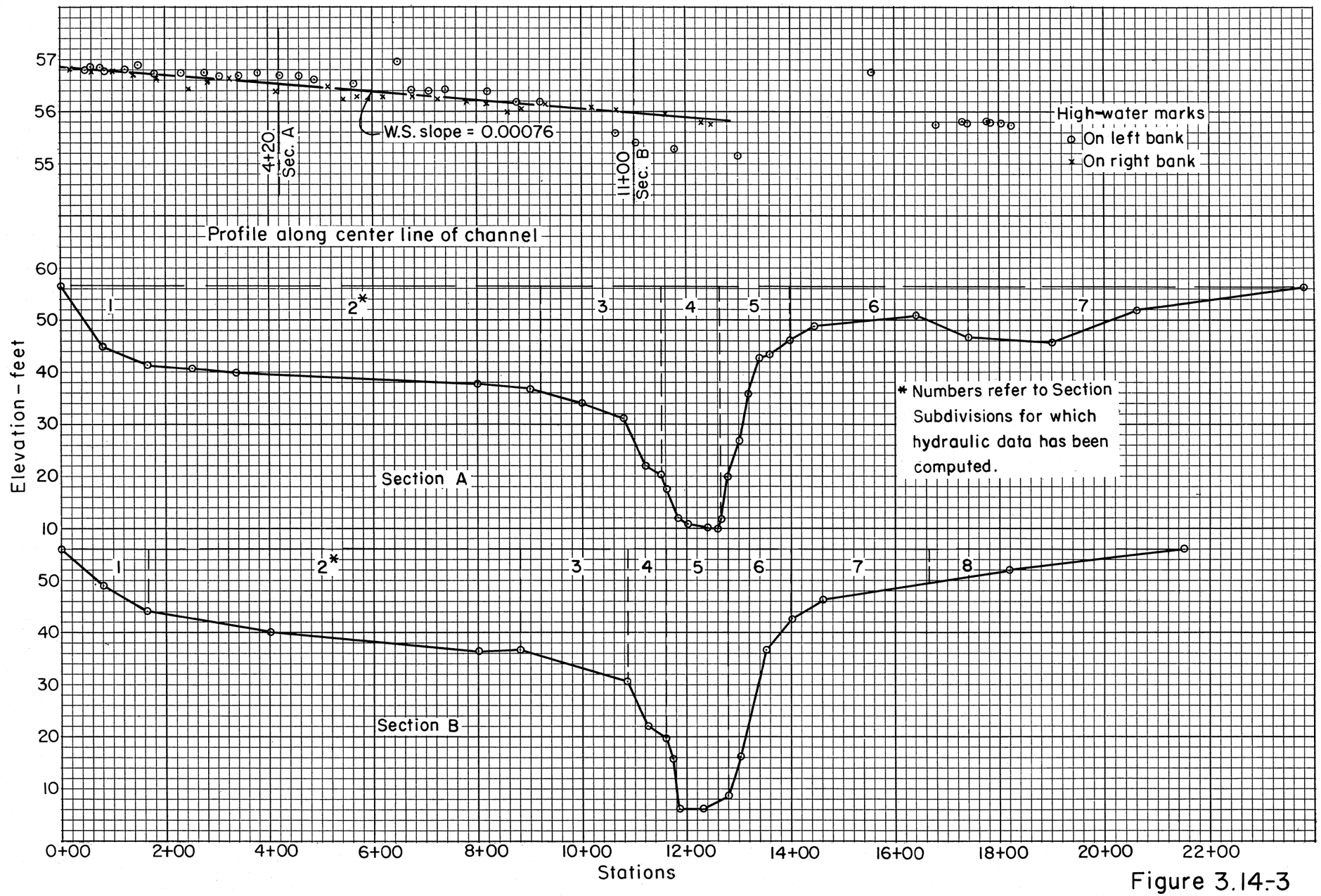


Figure 3.14-3
Profile and Cross Sections
Concho River, near
San Angelo, Texas.

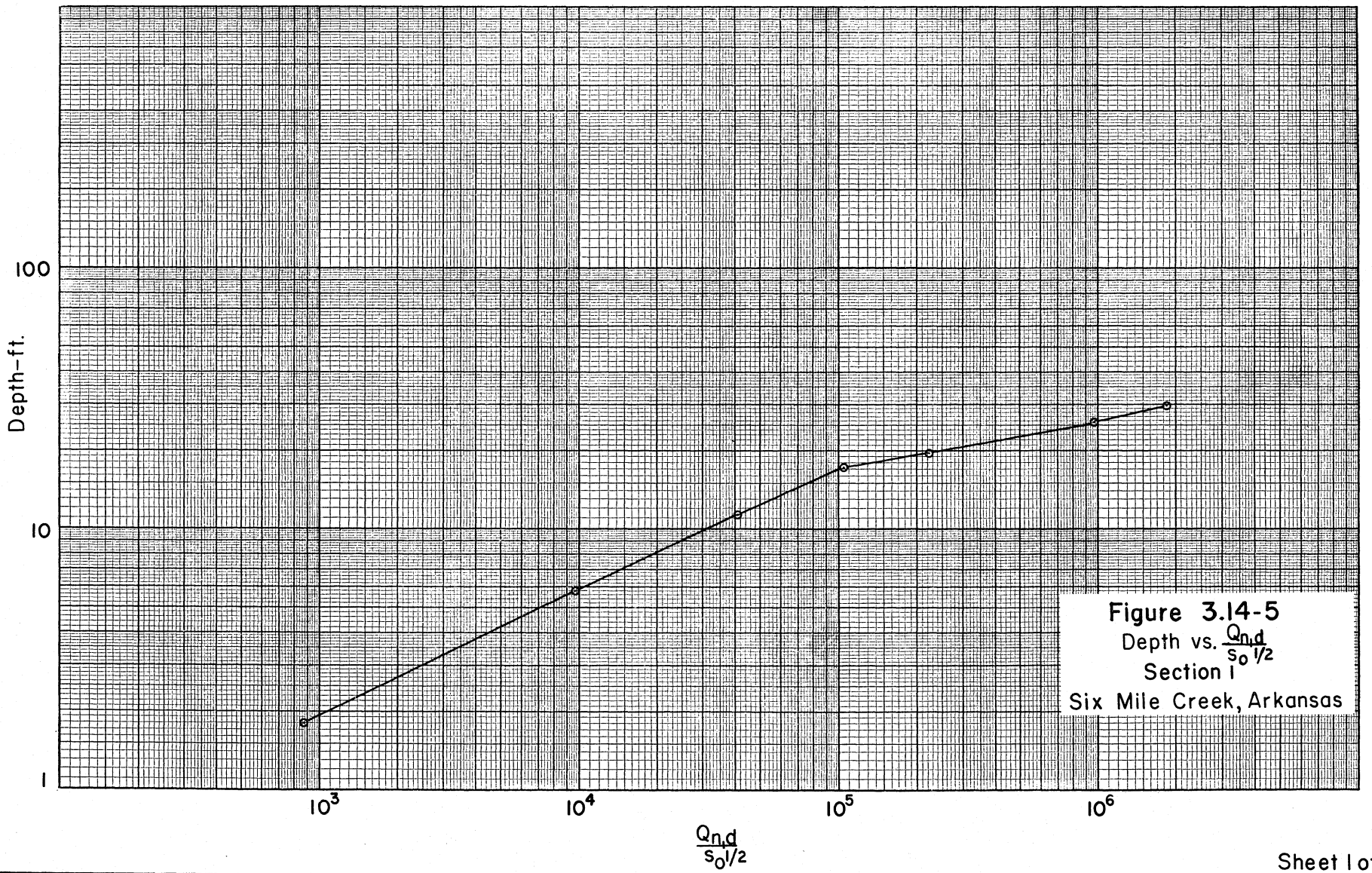


Figure 3.14-5
Depth vs. $\frac{Q_n d}{S_o^{1/2}}$
Section I
Six Mile Creek, Arkansas

Sheet I of 3

426963 O - 57 - 11

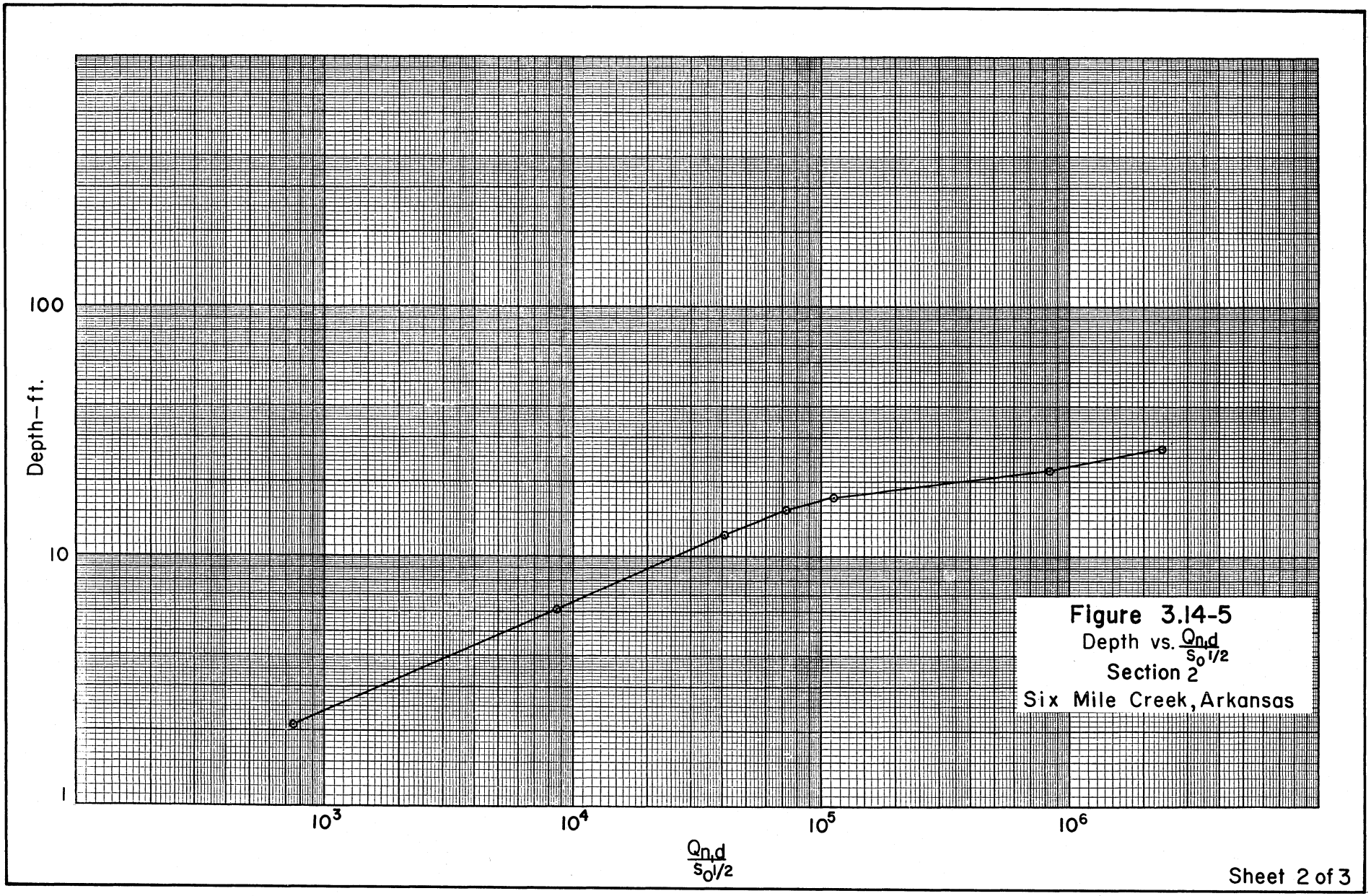


Figure 3.14-5
Depth vs. $\frac{Q_{n,d}}{S_o^{1/2}}$
Section 2
Six Mile Creek, Arkansas

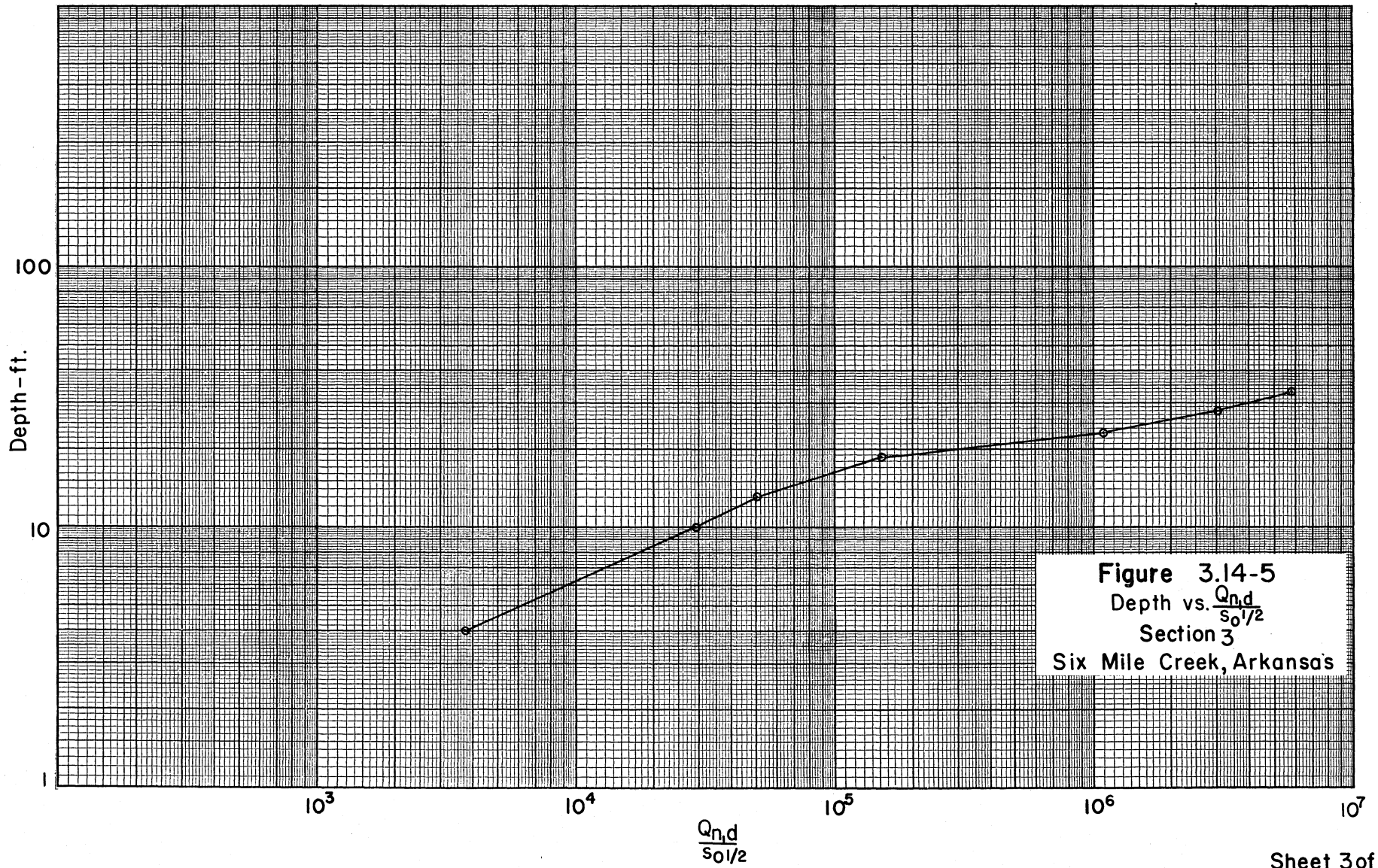


Figure 3.14-5
Depth vs. $\frac{Q_{n_1}d}{S_o^{1/2}}$
Section 3
Six Mile Creek, Arkansas

Sheet 3 of 3

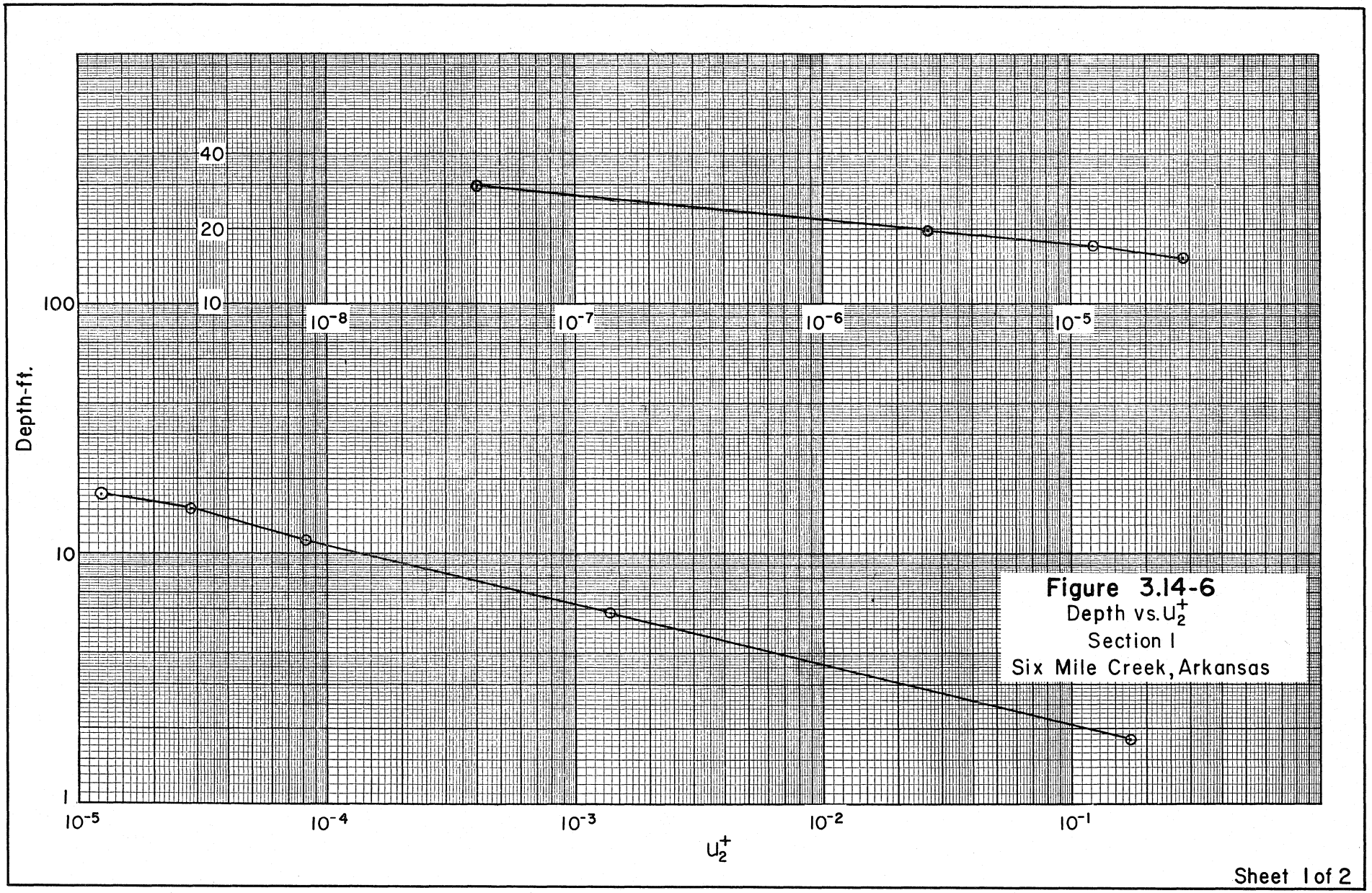


Figure 3.14-6
Depth vs. u_2^+
Section I
Six Mile Creek, Arkansas

Sheet 1 of 2

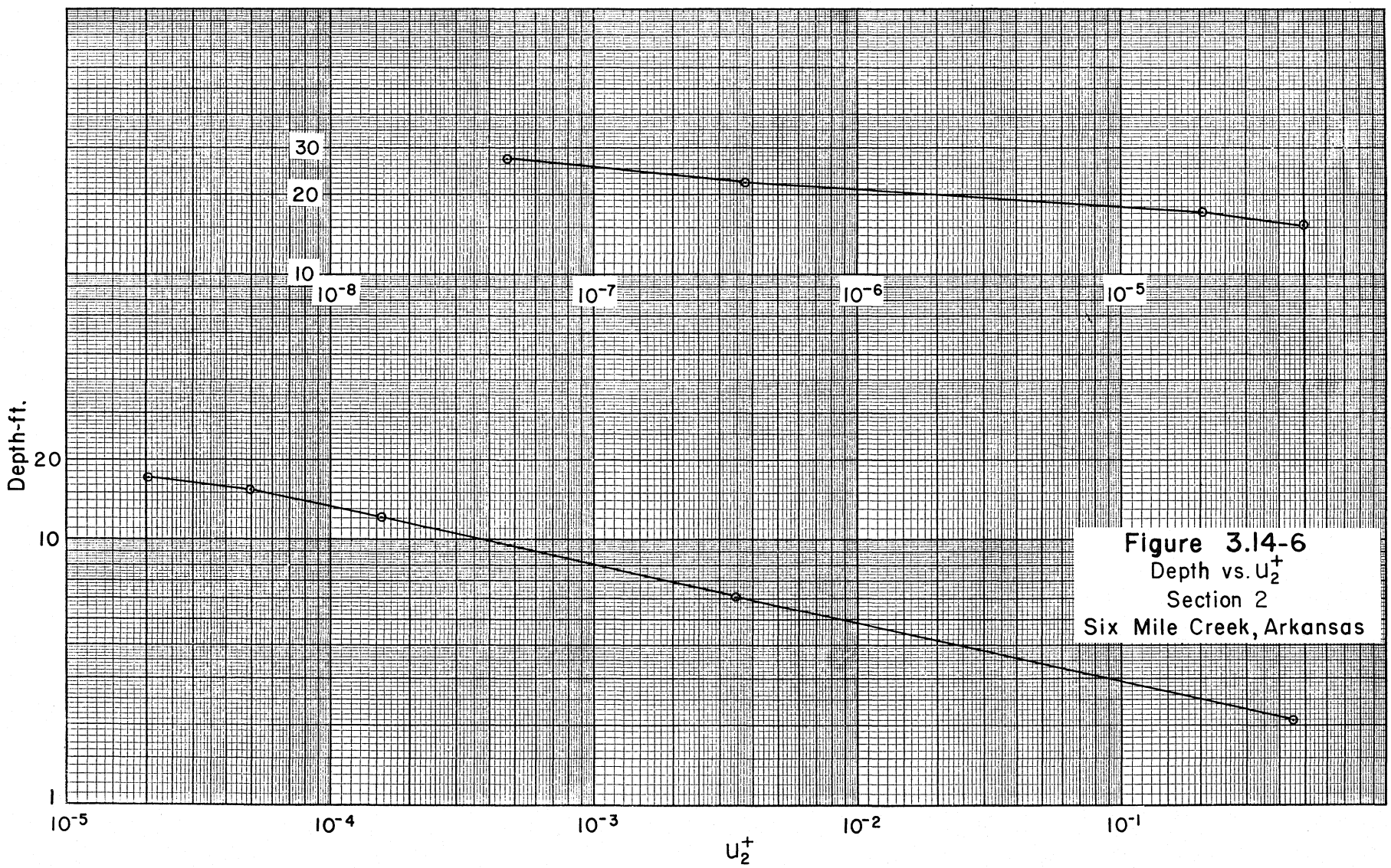


Figure 3.14-6
Depth vs. u_2^+
Section 2
Six Mile Creek, Arkansas

Sheet 2 of 2

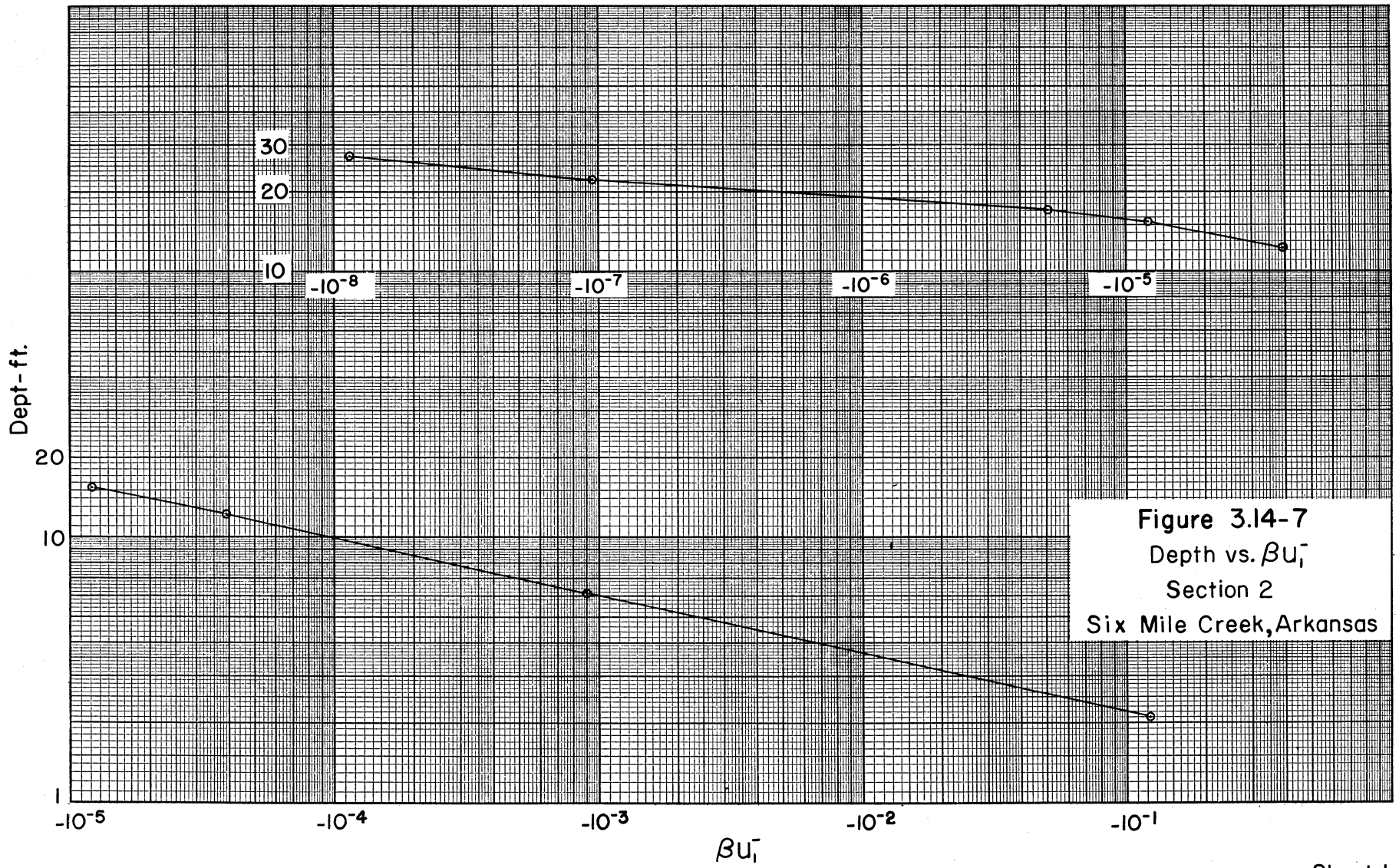


Figure 3.14-7
Depth vs. βu_1^-
Section 2
Six Mile Creek, Arkansas

Sheet 1 of 2

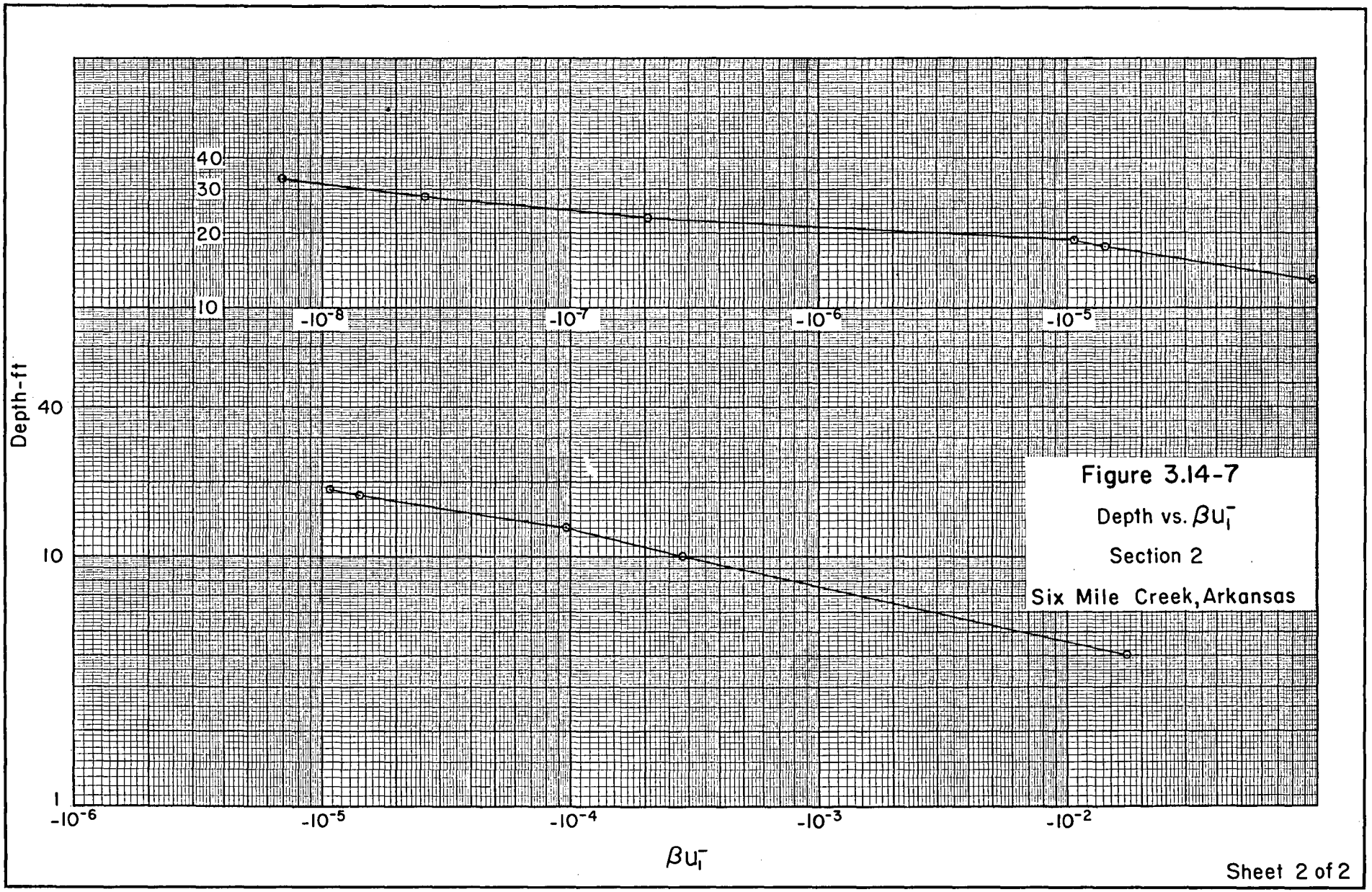


Figure 3.14-7

Depth vs. βu_1^-

Section 2

Six Mile Creek, Arkansas

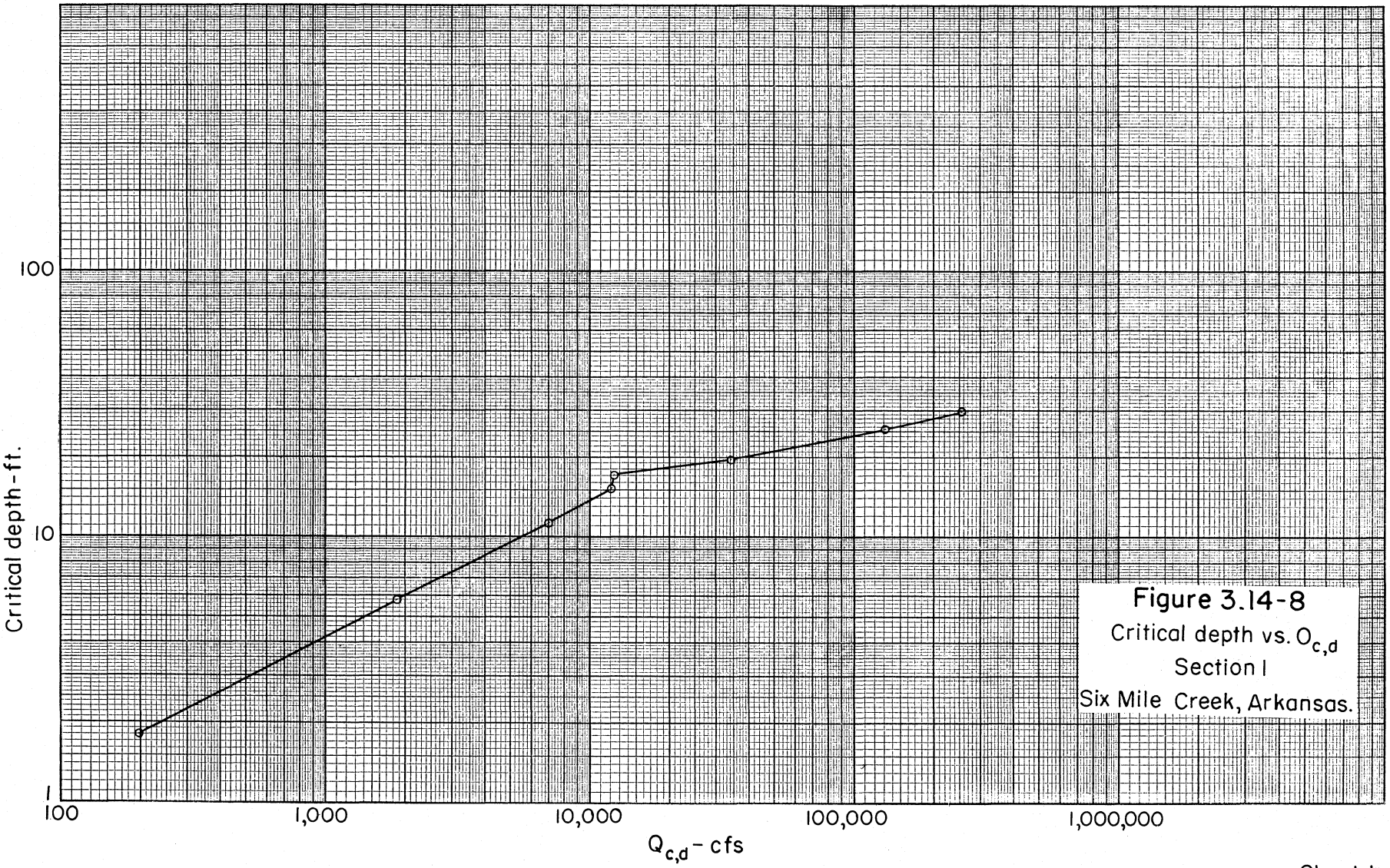


Figure 3.14-8
Critical depth vs. $Q_{c,d}$
Section I
Six Mile Creek, Arkansas.

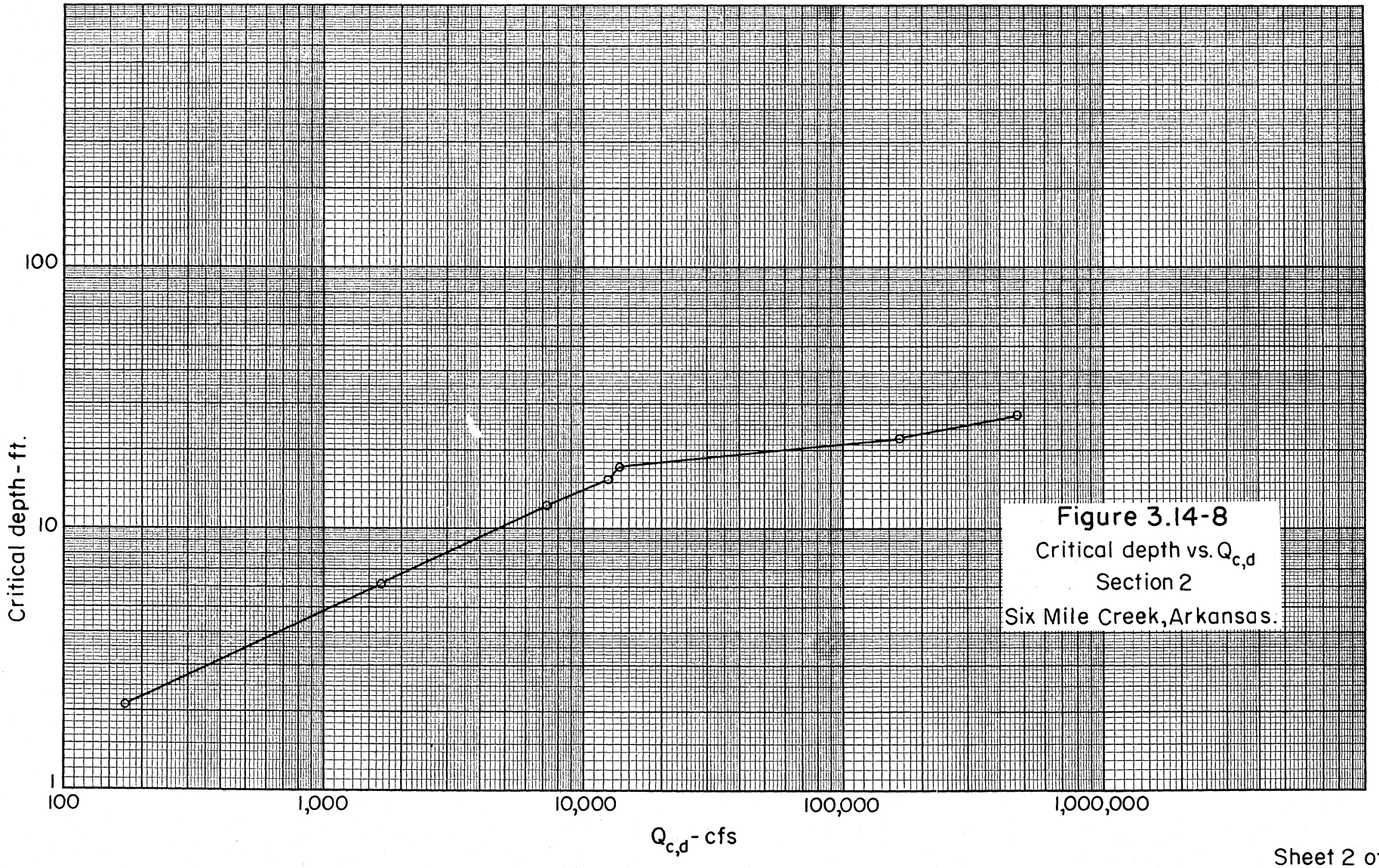


Figure 3.14-8
Critical depth vs. $Q_{c,d}$
Section 2
Six Mile Creek, Arkansas.

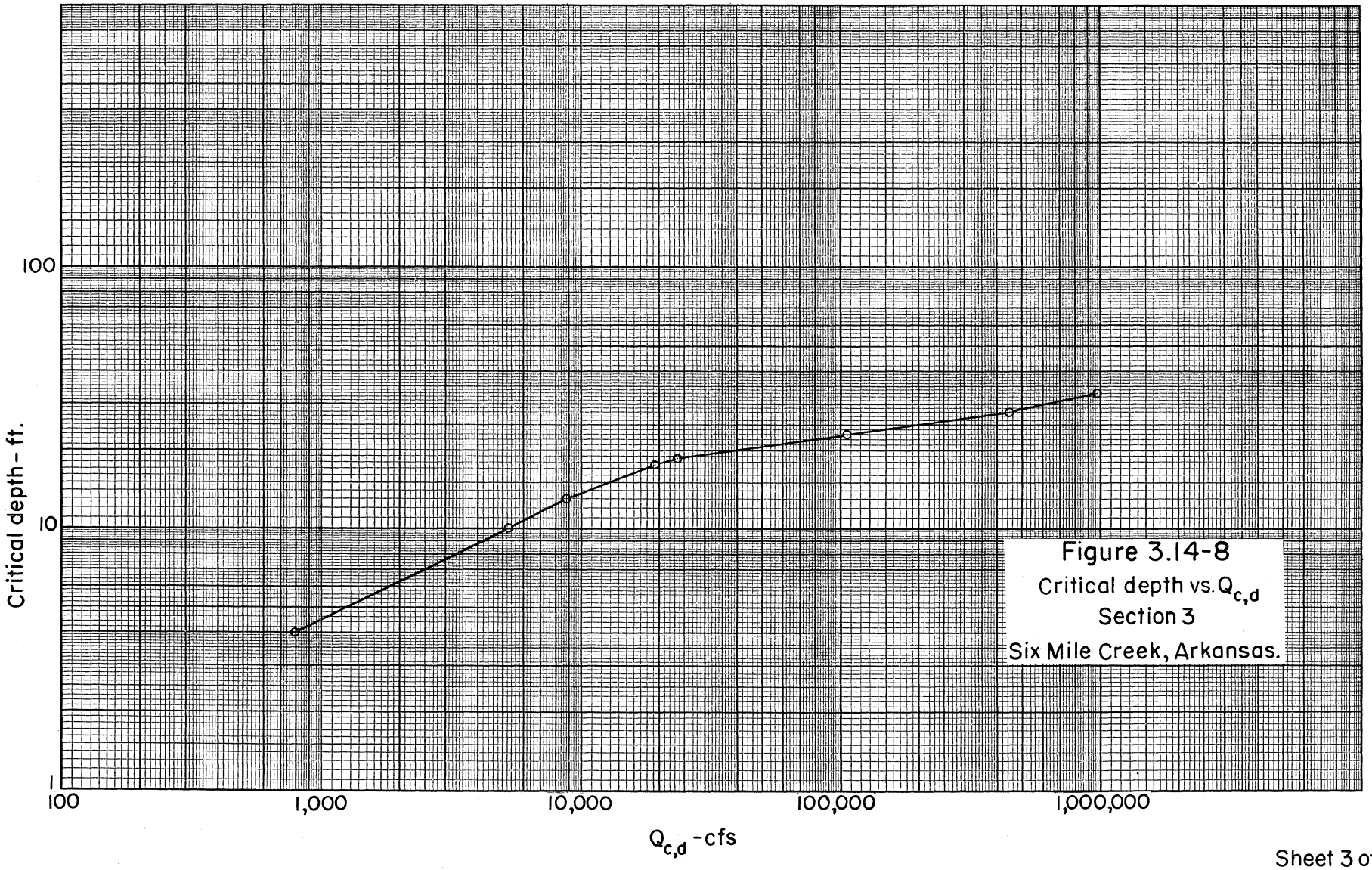
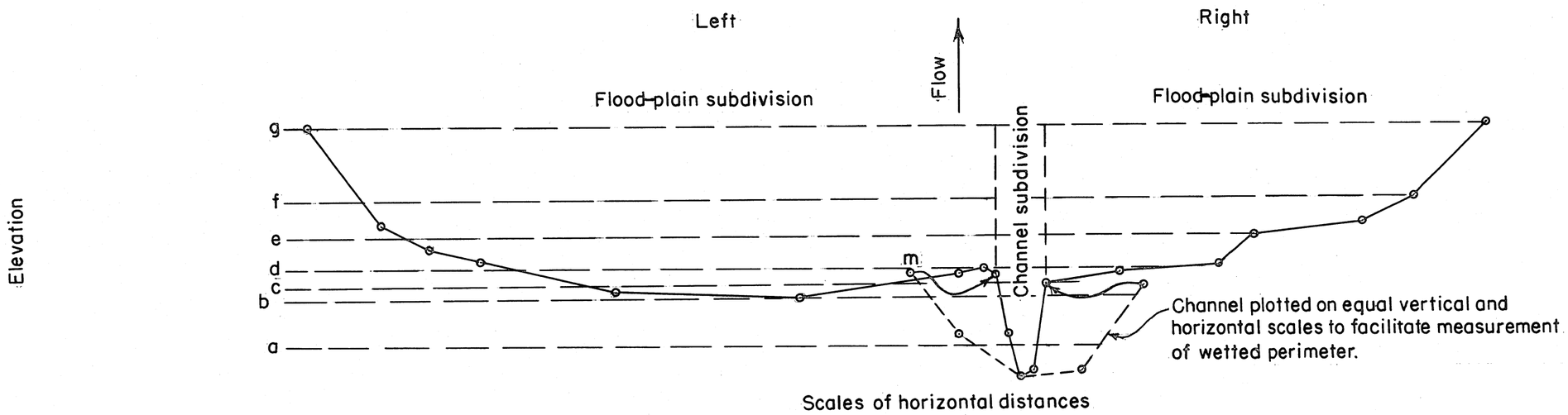


Figure 3.14-8
Critical depth vs. $Q_{c,d}$
Section 3
Six Mile Creek, Arkansas.



1. It is good and generally accepted practice to:
   a. Plot all sections looking downstream so that right and left on the sheets consistently represents the right and left of the stream.

   b. Where reasonably good accuracy is important, plot the channel portion on equal vertical and horizontal scales.

2. In subdividing sections recognize the channel and flood plain and materially different roughness conditions in the flood plain. Normally 2 or 3 subdivisions will assure adequate results. Dealing with 5 or 6 subdivisions increases the amount of work but in the majority of cases does not materially improve the result.

3. Select a limited number of stages at which the area and wetted perimeter are to be determined. These stages are represented as a, b, c, d, e, f and g above. The objective is to select stages between which the area and wetted perimeter for each subdivision increase at relatively uniform rates. In fairly regular sections 4 or 5 stages may serve this objective. In irregular sections 7 or 8 stages may be required.

4. When the cross sections are surveyed they should be taken up to 5 or 10 ft. above the apparent high-water stages.

5. The following rules apply in the measurement of wetted perimeter:

   a. Wetted perimeter includes only that part of the cross-section boundary in contact with water; that is, distances along the verticals drawn to define subdivisions are not included in wetted perimeter.

   Thus, in the channel subdivision above, wetted perimeter remains constant at stages above point m.

   b. In a flood-plain subdivision the top width is acceptable as a sufficiently accurate measurement of wetted perimeter.

6. Some hydraulicians, when dealing with discharges involving flood-plain flow, prefer to use distance along the flood plain instead of channel distance as reach length. The various ideas that are followed in making this change in reach length are arbitrary and should be recognized as such. Where there is a meandering channel of relatively low capacity a change in reach length may be desirable but the following points should be recognized.

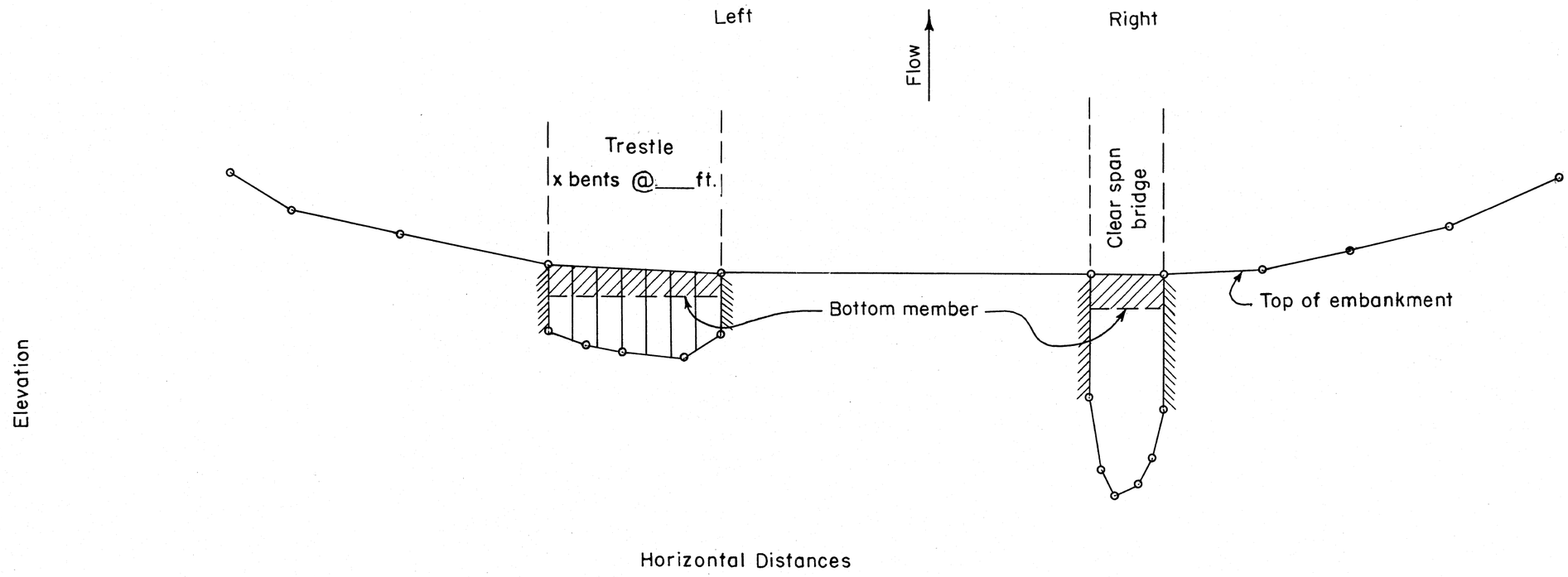
   a. The reduced reach length should be measured with reasonable accuracy along a line connecting the low points of the valley disregarding the channel.

   b. In making the hydraulic computations the reduced length should be applied at stages no less than 1 to 2 feet above the bankfull stage.

Figure 3.14-4
Recommendations on Subdivision of Cross Sections and Determination of the Hydraulic Elements.

Sheet 1 of 2

426963 O-57 (Face p. 160 blank) No. 2



1. Cross sections at bridge openings should be plotted in accordance with the general practice stated under item 1, Sheet 1. The openings should be plotted on scales that will permit determination of areas and wetted perimeters to an acceptable degree of accuracy.

2. Where the First Method of Determination of Stage Upstream From Bridges is to be used, the value of roughness coefficient, n, for each opening is to be estimated.

3. In making field surveys the following data should be obtained:

   a. For trestles, the size of piles and the spacing of pile bents.

   b. For bridges having piers, the size and spacing of piers.

   c. The top of embankment section should be taken up to elevations high enough to permit determination of stage-discharge relations for discharges that over-top the embankment.

   d. Elevation of the bottom members of each bridge opening.

4. In the case of trestles and bridges having piers:

   a. Wetted perimeter should be taken as the length of the cross section boundary plus twice the height of piers or pile bents up to the stage under consideration.

   b. Cross sectional area should be the net area, i.e., the area of the cross section minus the area of bents or piers at the stage under consideration.

   c. Top width of flow, T, should be the net width, i.e., the width of the section minus the width of bents or piers.

Figure 3.14-4

Sheet 2 of 2

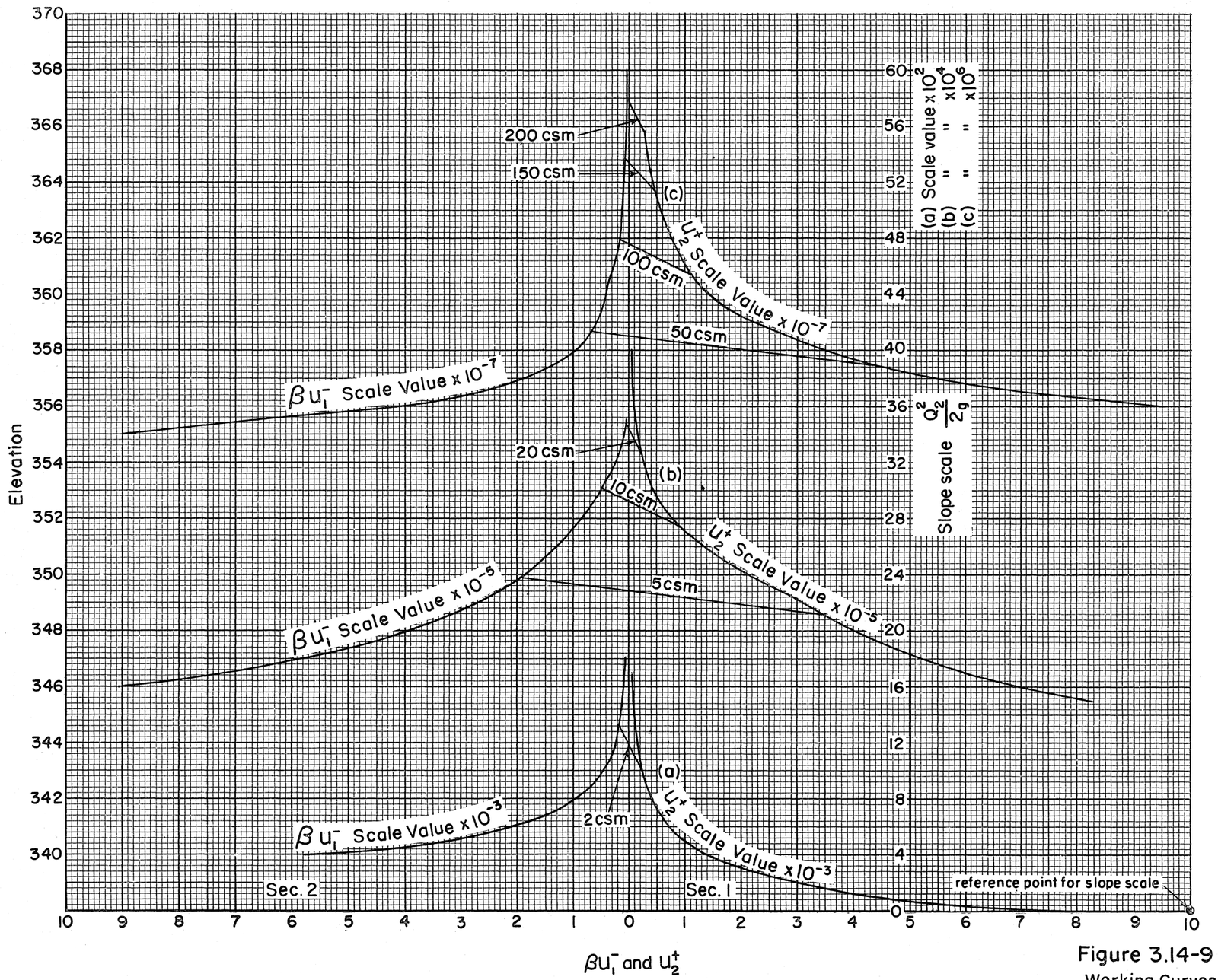


Figure 3.14-9
Working Curves
Doubt's Method
Six Mile Creek, Arkansas.

Sheet 1 of 2

426963 O-57 (Face p. 170) No. 1

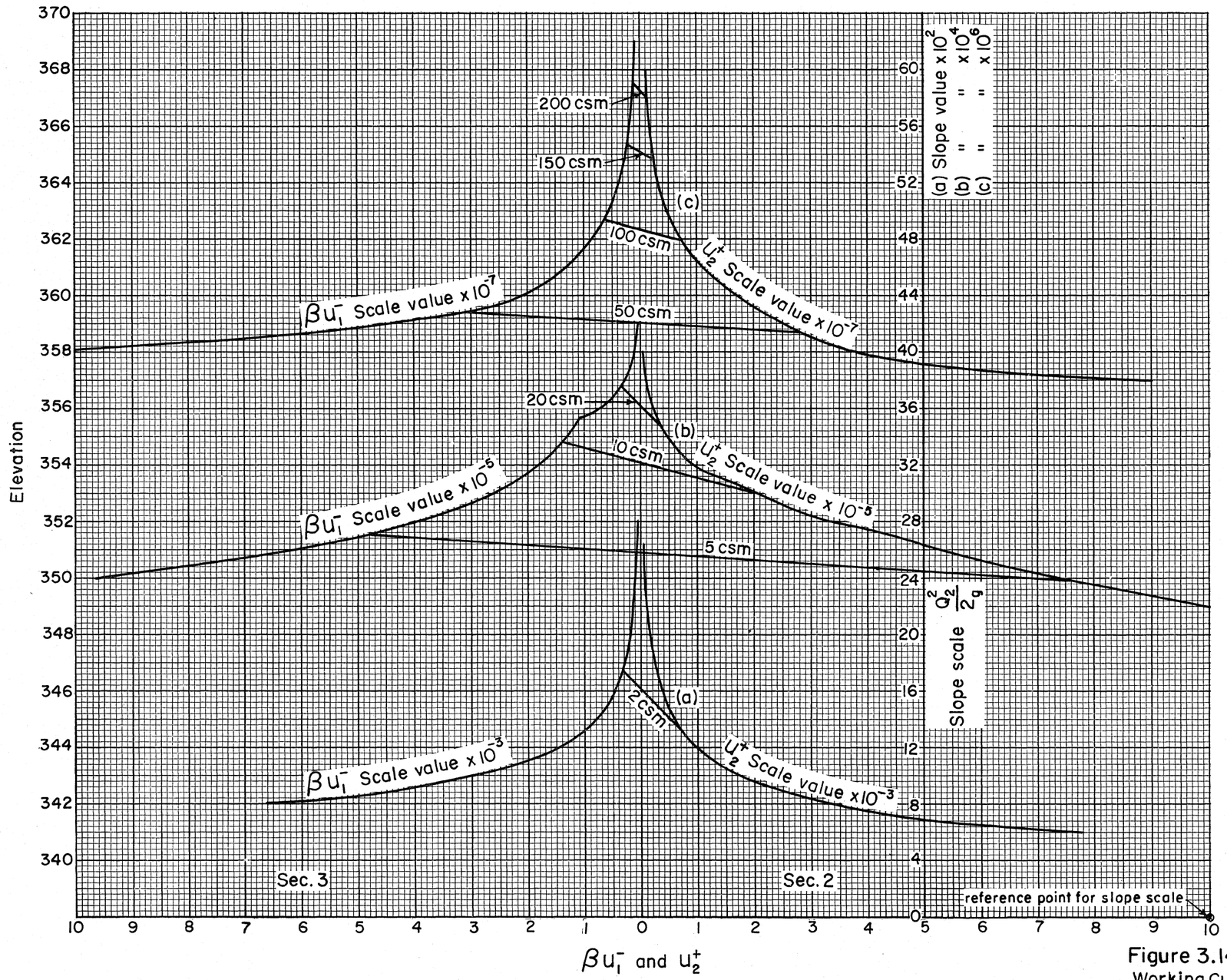
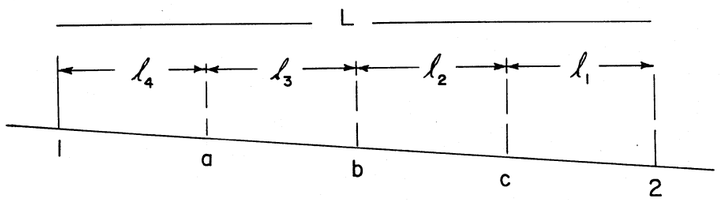


Figure 3.14-9
Working Curves
Doubt's Method
Six Mile Creek, Arkansas.

Sheet 2 of 2



Sections 1 and 2 are sections that have been surveyed in the field. Sections a, b and c are synthetic sections for which the U_2^+ and βU_1^- curves are develope from data at sections 1 and 2.

1. Break reach L into sub-reaches. The schematic sketch at the left indicates 4 sub-reaches. In a given situation, 2 or more may be used as required.

2. Determine β for the synthetic sections on the assumption that drainage area increases uniformily through the reach L.

3. Compute a new U_2^+ curve based on the sub-lengths of L; also a new βU_1^- curve based on the appropriate β and the sub-lengths of L.

4. Determine values for U curves of the synthetic sections as follows:
   a. At equal depths in sections 1 and 2.
      (1) Average the values of Σ_a for the complete sections and compute $\frac{1}{(\Sigma a)^2}$
      (2) Average the values of $\frac{Q_{n,d}}{So^{\frac{1}{2}}}$ for the complete sections and compute $\frac{So}{Q_{n,d}^2}$
      (3) Plot the average values obtained in (1) and (2) versus the equal depths in sections 1 and 2.
   b. Compute U_a^+, U_b^+, U_c^+, βU_a^-, βU_b^-, and βU_c^- from the above average values recognizing the appropriate values of β and sub-reach lengths.

5. Plot the U curves for the various synthetic sections from the values computed in 4, (b). The elevations at which these values plot depend on the bottom of channel elevations at the positions on the profile selected for the synthetic sections a, b, and c.

6. Proceed with the solution starting with E_2 as determined from the next downstream reach.

   (Note that the sketch at the left is schematic. The βU_a^-, βU_b^-, and βU_c^- curves may fall in either the positive or negative range.)

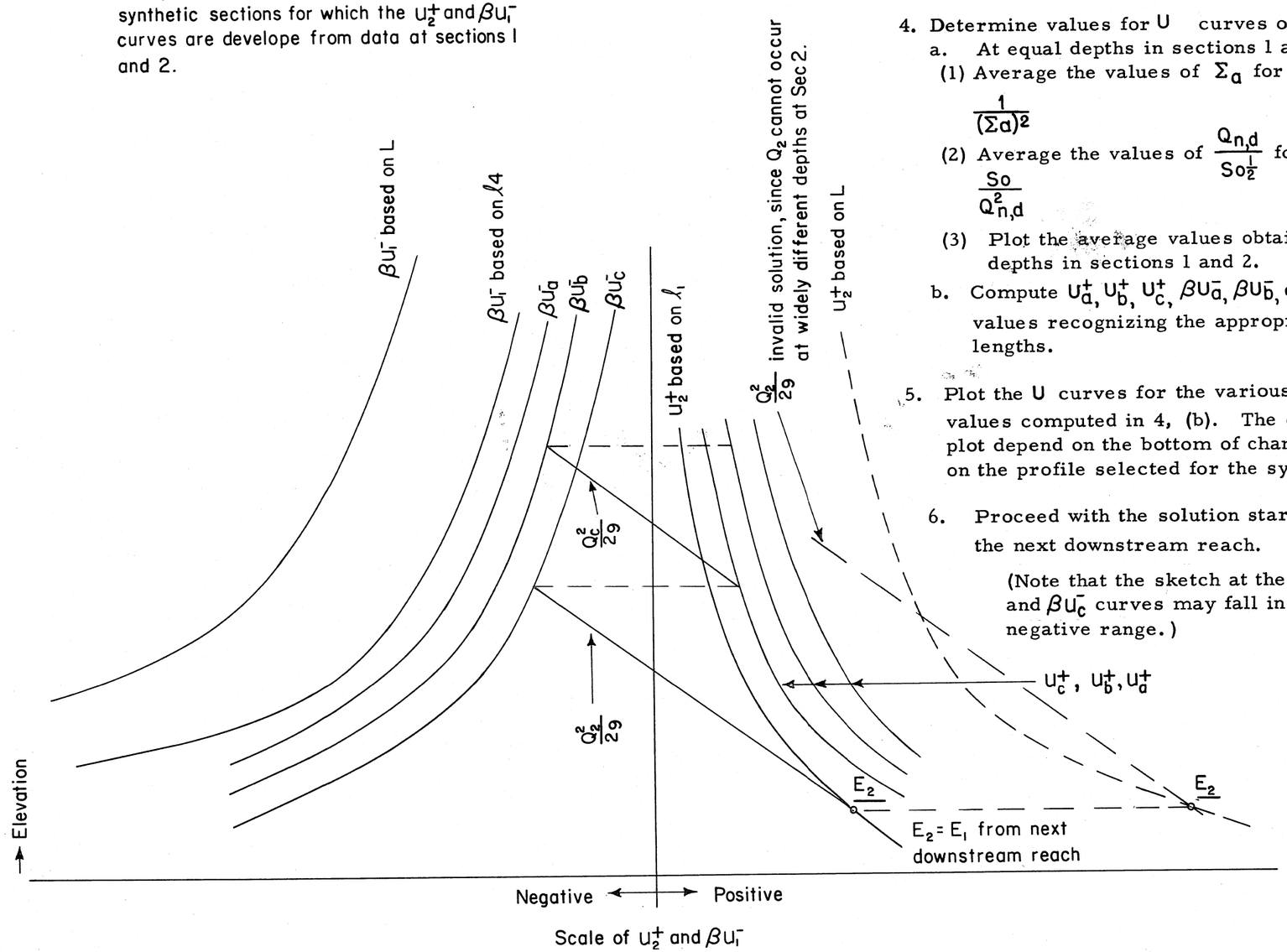
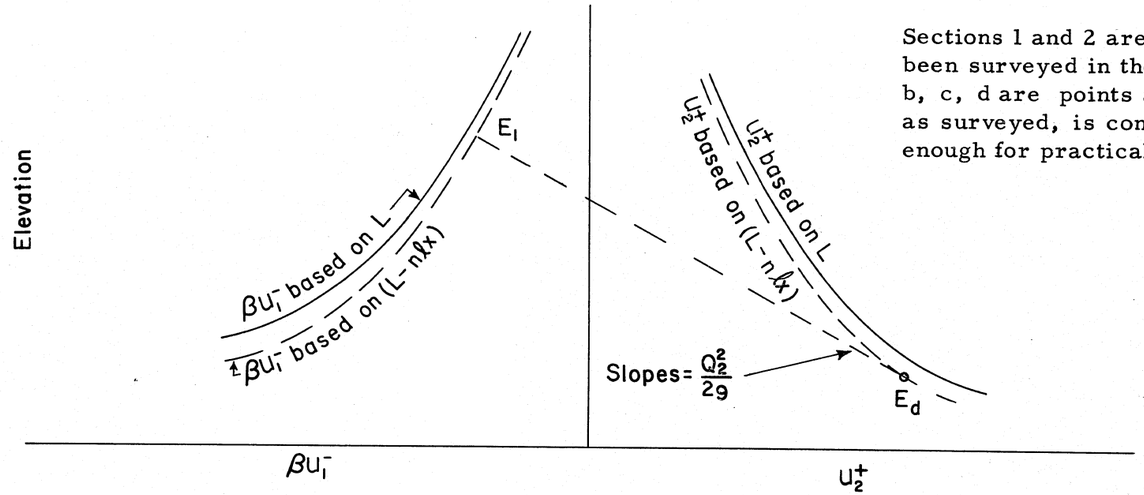


Figure 3.14-10
Procedure for computing E_1 when slope line for $\frac{Q_2^2}{2g}$ intersects U_2^+ curve at two widely separate points.

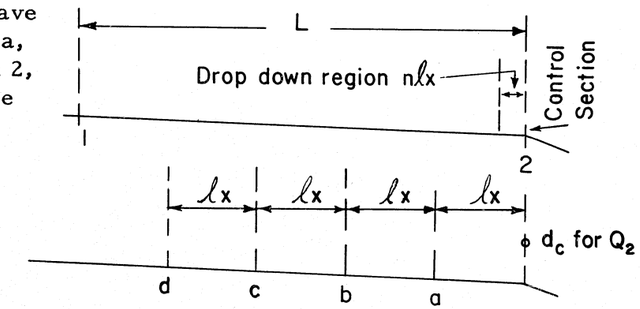
Sheet 1 of 2

426963 O-57 (Face p. 170) No. 2

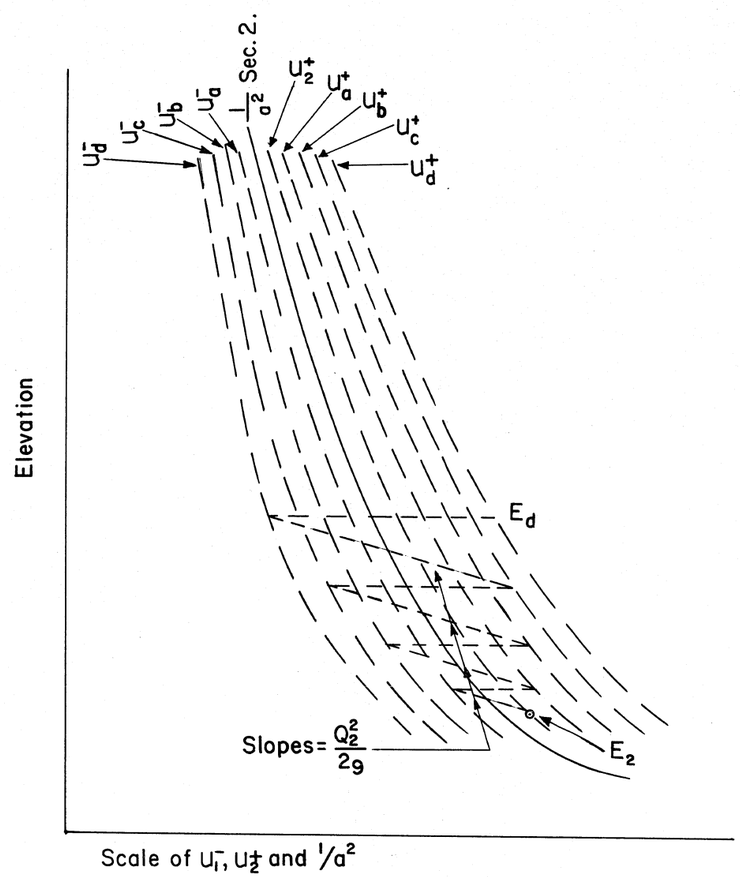


Working curves for reach L

Sections 1 and 2 are sections that have been surveyed in the field. Points a, b, c, d are points at which section 2, as surveyed, is considered accurate enough for practical purposes.



Drop down part of profile to enlarged scale.



To compute upstream through the drop-off region above a control section:

1. Assume that section 2 represents the cross section at points a, b, c, d spaced at intervals ℓ_x along the profile. Where d_c at section 2 is less than 10 ft. use $\ell_x = 10$ ft. where d_c is 10 ft. or more use $\ell_x = 20$ to 40 ft.

2. Set up a separate work sheet like that at the lower left; use a relatively large scale for U_1^-, U_2^+ and $\frac{1}{a^2}$
   a. Plot the curve of $\frac{1}{a^2}$ vs elevation for section 2.
   b. Compute $\ell_x g\left(\frac{so}{Q^2_{n_1 d_2}}\right)$ at 3 or 4 elevations at intervals of about 1.0 ft. starting at about the elevation at section 2 associated with d_c for Q_2
   c. Plot the family of U curves as shown. To make this plot, lay off, at each elevation, distances equal to $\ell_x g\left(\frac{so}{Q^2_{n_1 d_2}}\right)$ to the right and left of the curve of $\frac{1}{a^2}$. At any elevation:

$$U_2^+ = \frac{1}{a^2} + \ell_x g\left(\frac{so}{Q^2_{n_1 d_2}}\right);\ U_a^+ = \frac{1}{a^2} + 2\,\ell_x g\left(\frac{so}{Q^2_{n_1 d_2}}\right);\ U_b^+ = \frac{1}{a^2} + 3\,\ell_x g\left(\frac{so}{Q^2_{n_1 d_2}}\right) \text{ etc.,}$$

and

$$U_a^- = \frac{1}{a^2} - \ell_x g\left(\frac{so}{Q^2_{n_1 d_2}}\right);\ U_b^- = \frac{1}{a^2} - 2\,\ell_x g\left(\frac{so}{Q^2_{n_1 d_2}}\right);\ U_c^- = \frac{1}{a^2} - \ell_x g\left(\frac{so}{Q^2_{n_1 d_2}}\right) \text{ etc.}$$

   d. Starting with E_2 associated with d_c for Q_2 solve graphically for E_d.

3. Plot U_2^+ and βU_1^- curves based on $(L - n\ell x)$ on the sheet of working curves for reach L. Solve for E_1 starting with E_d as determined above.

Values of $\frac{1}{a^2}$ and $\frac{so}{Q^2_{n_1 d}}$ may be taken from Table 3.14-1 and d_c for any Q_2 from Figure 3.14-8; therefore, the steps outlined above can be carried out in a few minutes.

Figure 3.14-10
Procedure for computing W.S.
Profile upstream from a control section

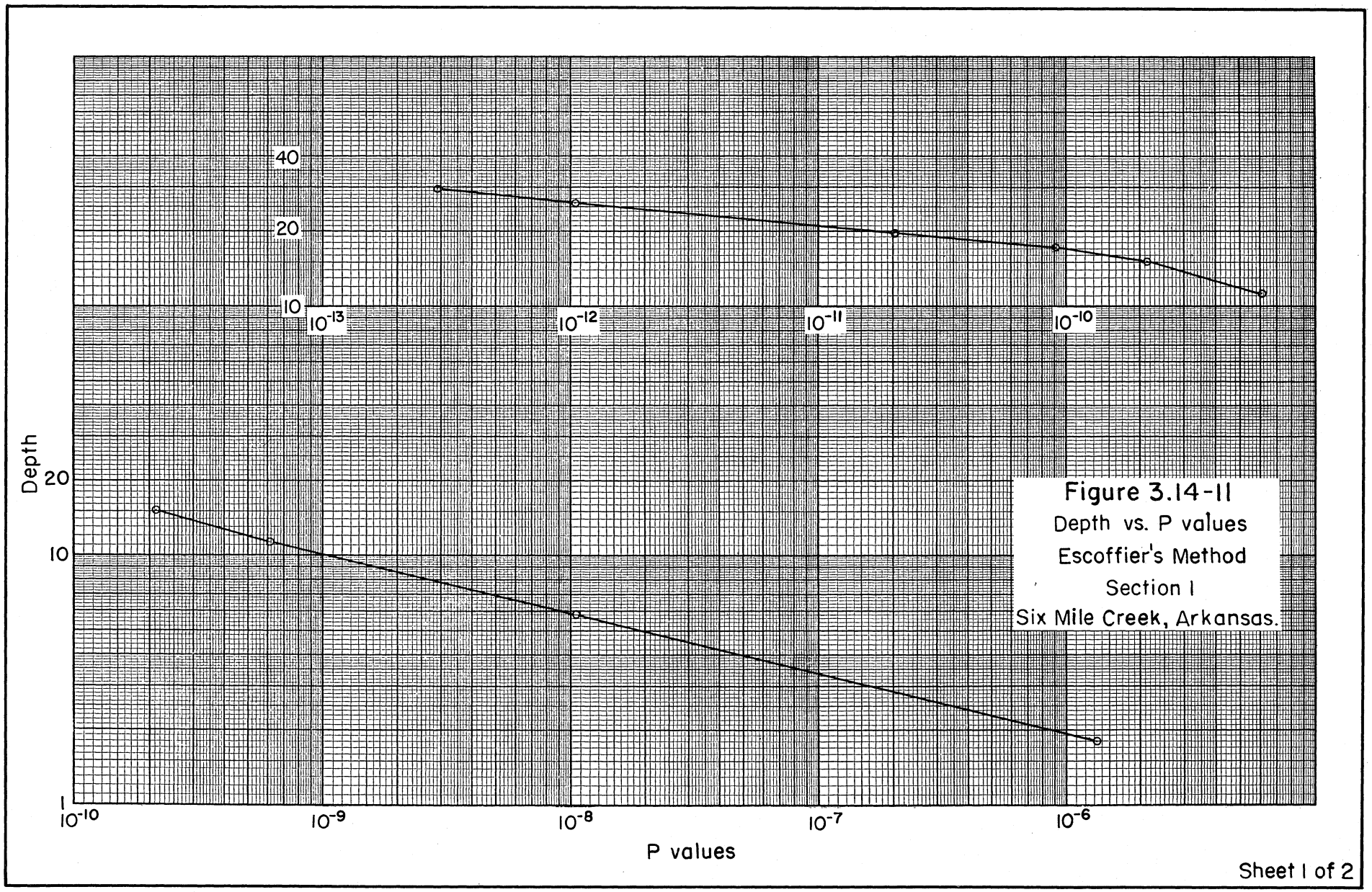


Figure 3.14-11
Depth vs. P values
Escoffier's Method
Section I
Six Mile Creek, Arkansas.

Sheet 1 of 2

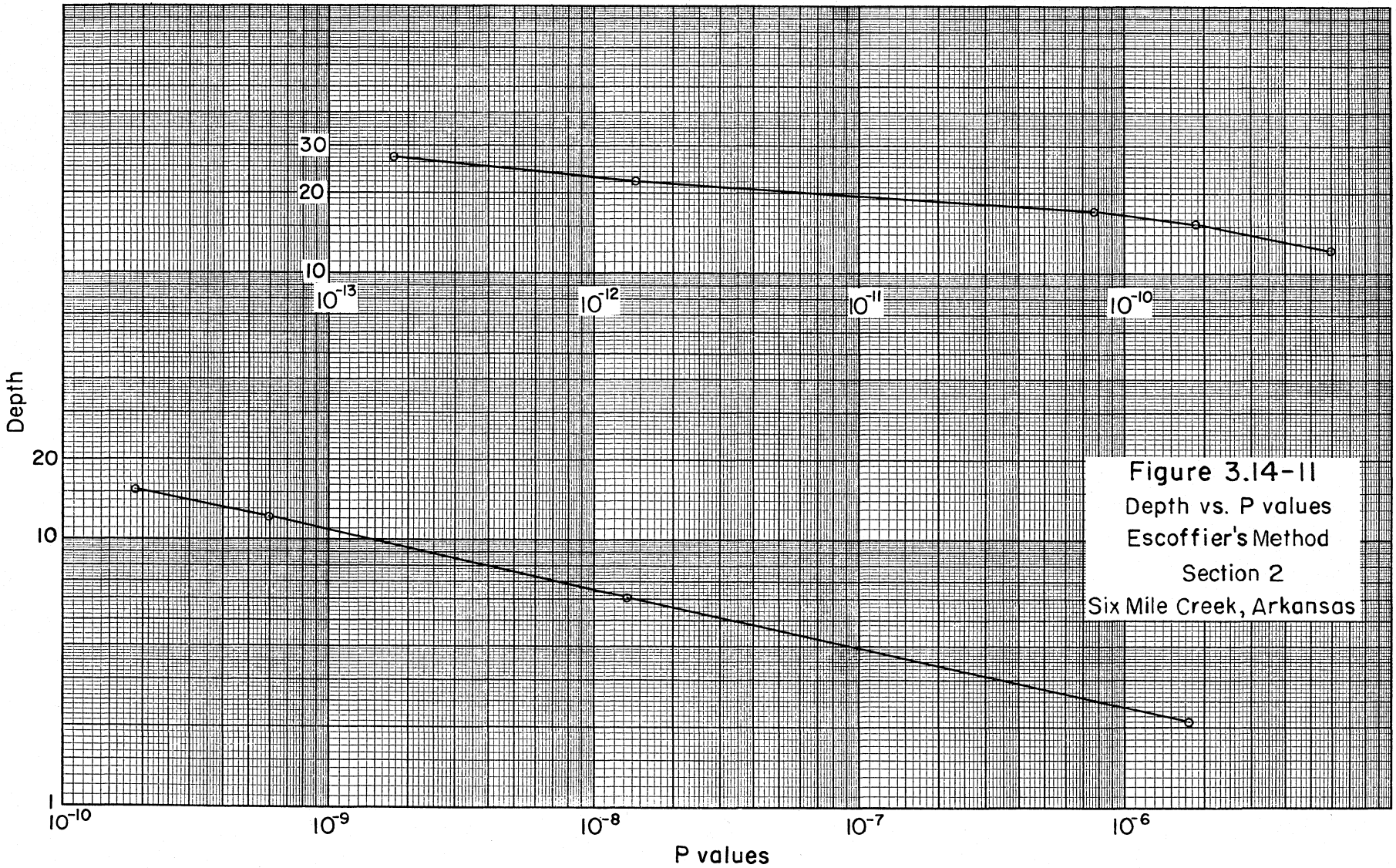


Figure 3.14-11
Depth vs. P values
Escoffier's Method
Section 2
Six Mile Creek, Arkansas

Sheet 2 of 2

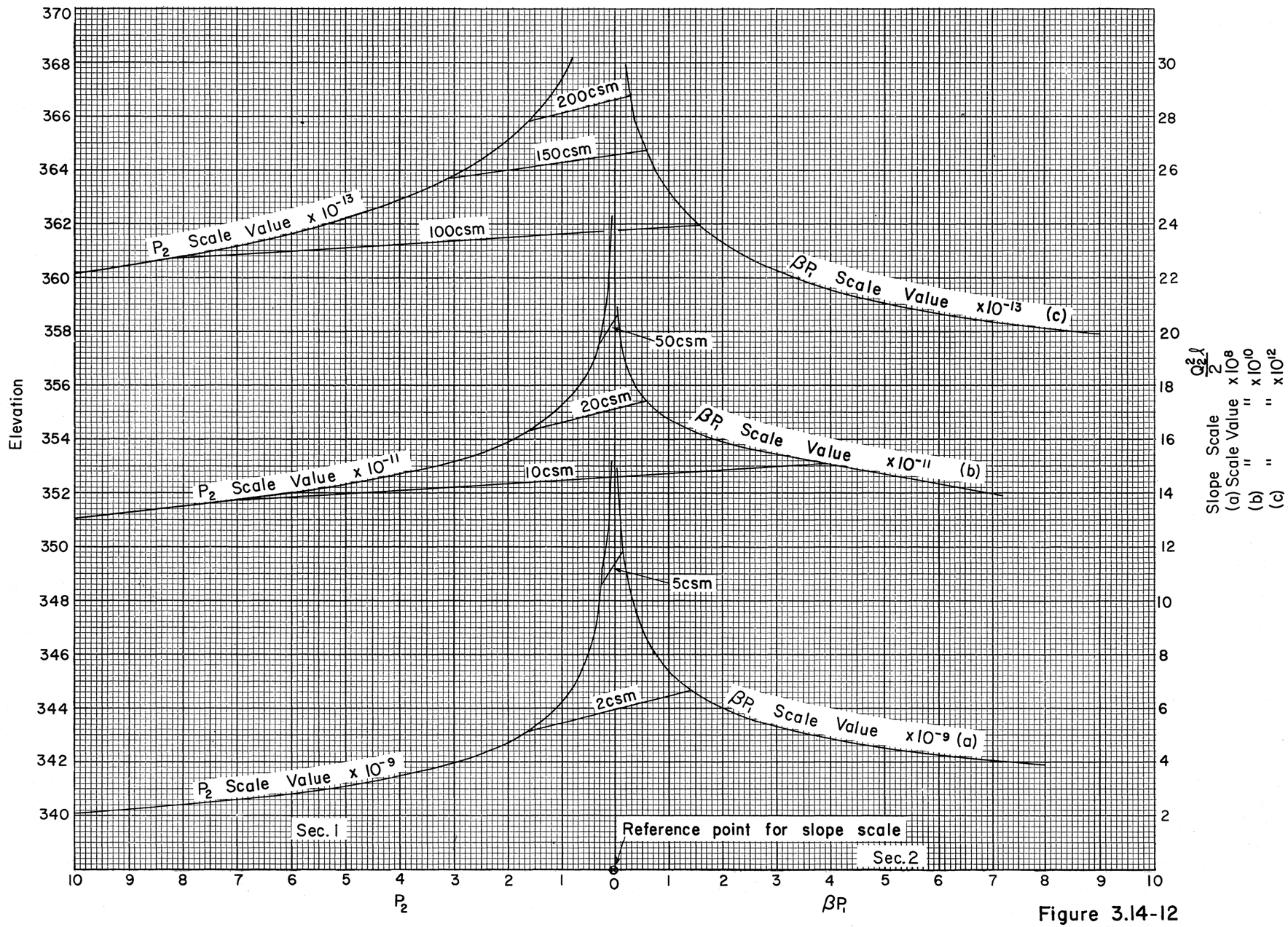


Figure 3.14-12
Working Curves
Escoffier's Method
Six Mile Creek, Arkansas
Sheet 1 of 2

426963 O-57 (Face p. 172) No. 1

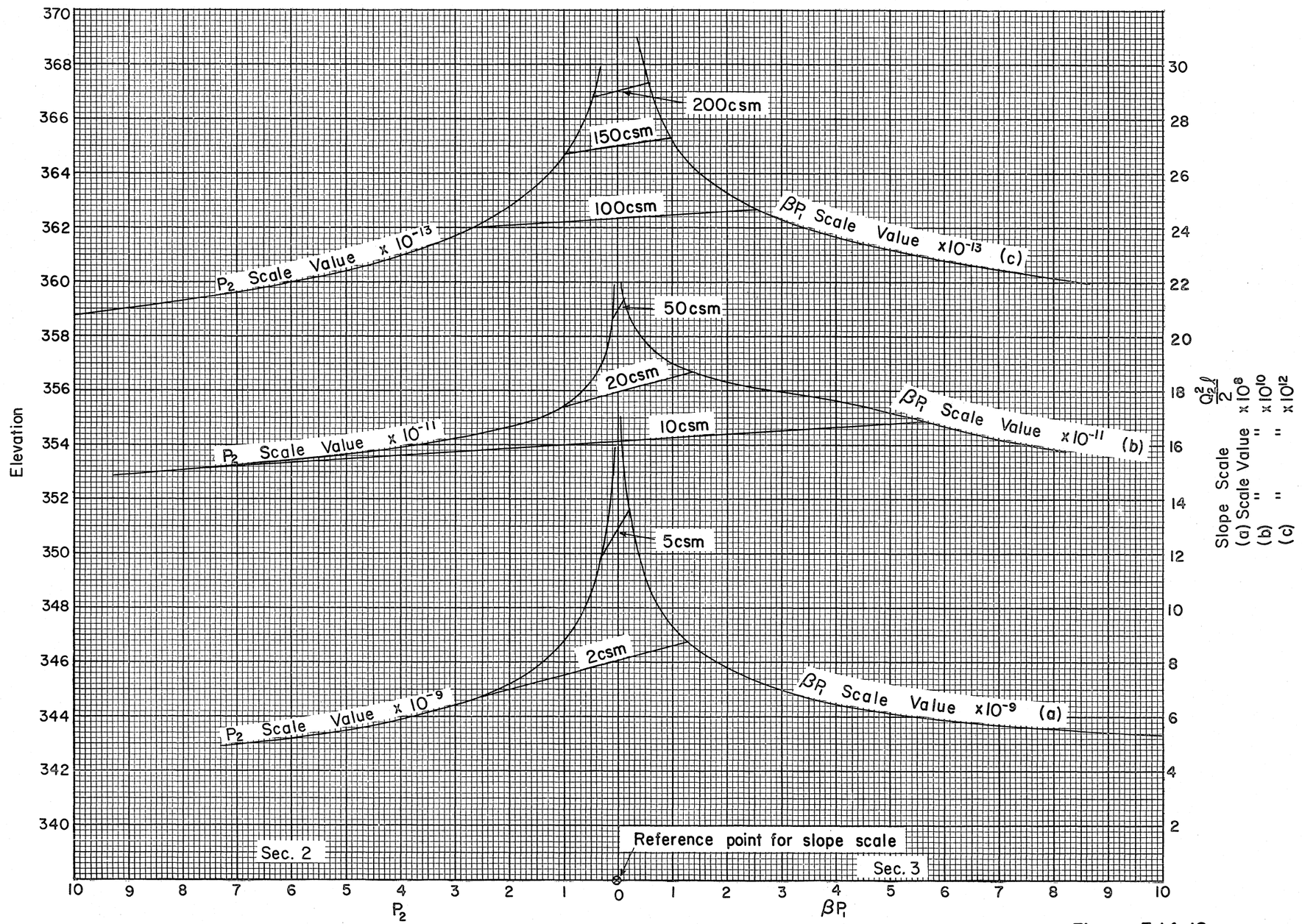
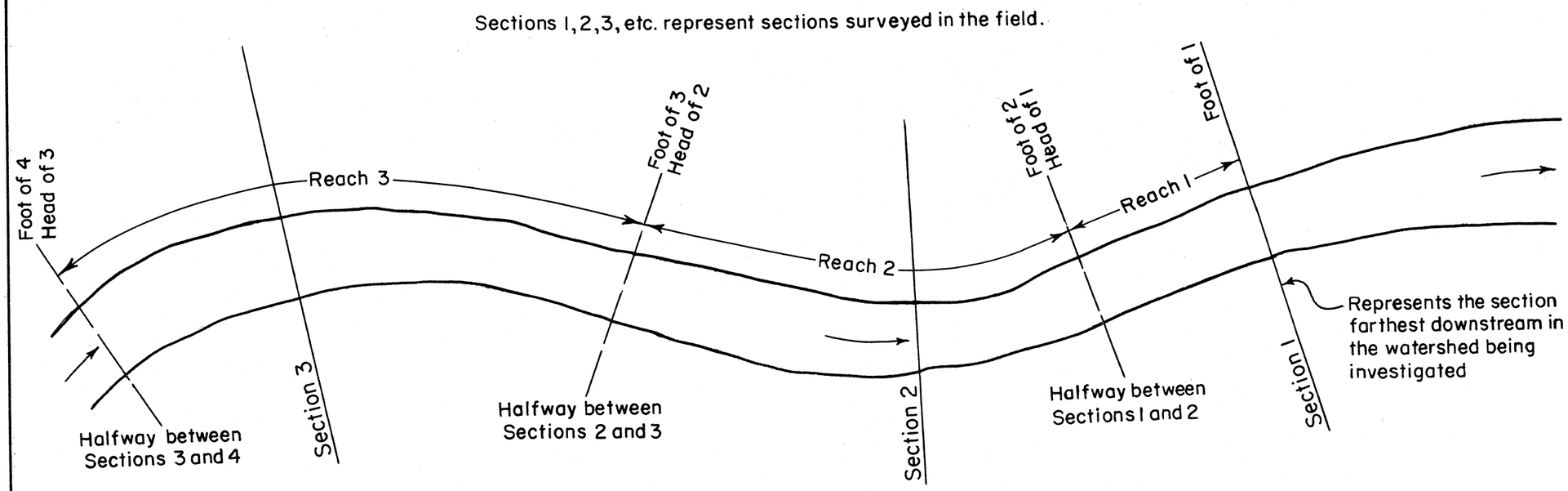


Figure 3.14-12
Working Curves
Escoffier's Method
Six Mile Creek, Arkansas

Sheet 2 of 2



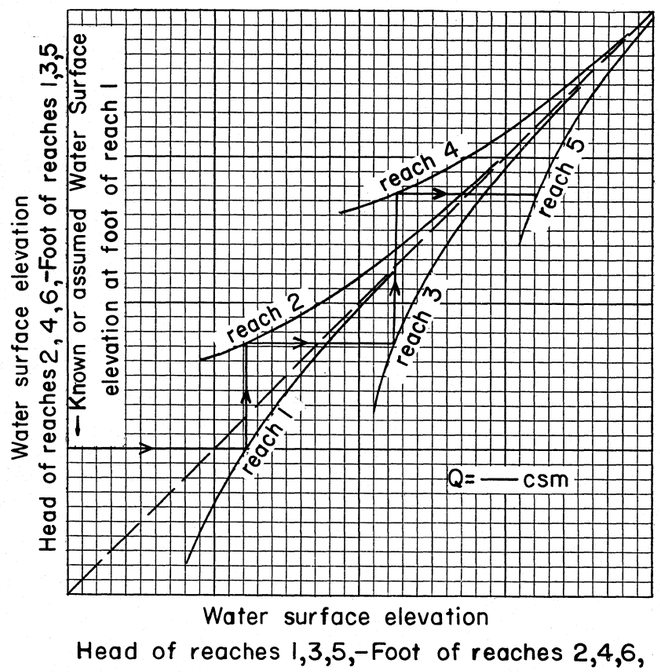
Leach's method considers the cross sections surveyed in the field to be the average cross sections for the various reaches. The reaches may be selected by either one of the following:

1. Cross section representing the average conditions may be selected and surveyed in the field, then the foot and head of each reach placed at points halfway between the surveyed cross sections, as illustrated above.

2. The foot and head of each reach, in which conditions are reasonably uniform, may be located in the field. Then cross sections are surveyed at approximately the midpoints of these reaches. These cross sections are then used as average cross sections for the selected reaches.

Figure 3.14-13
Stream reaches for use
in Leach's method

426963 O -57 (Face p. 172) No. 2



The drawing above illustrates the working curves used in Leach's method. Data for curves of this type are computed in step 4, table 3.14-6. Note that a family of these curves is valid for only one discharge.

To compute the water surface elevations enter the curves with the known or assumed water surface elevation at the foot of reach 1 for the given csm discharge. Draw horizontal and vertical lines to intersect the successive reach curves. Read the water surface elevations at the head and foot of reaches on the appropriate scale.

Figure 3.14-13

Working curves
Leach's method

Sheet 2 of 2

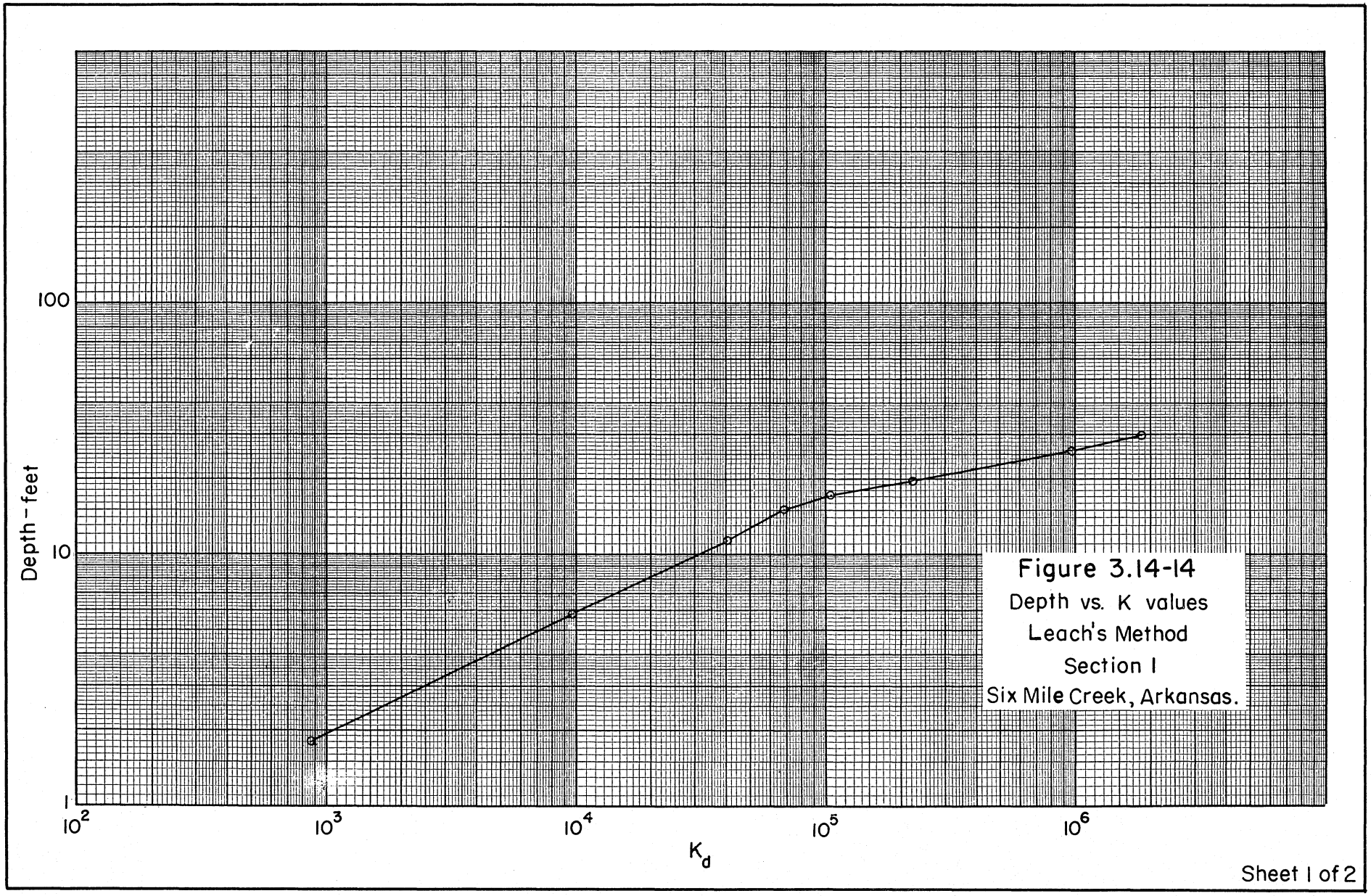


Figure 3.14-14
Depth vs. K values
Leach's Method
Section I
Six Mile Creek, Arkansas.

Sheet 1 of 2

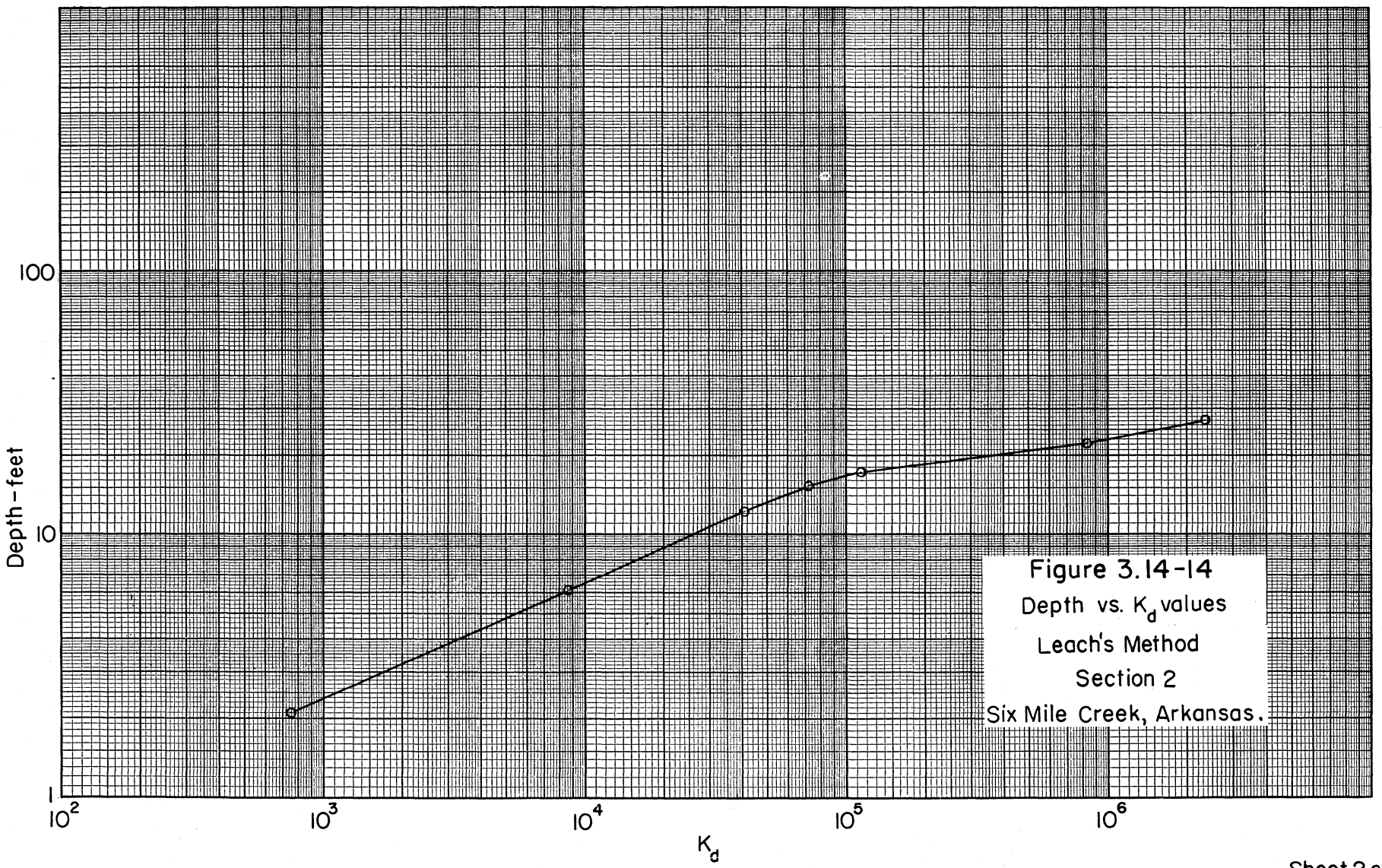


Figure 3.14-14
Depth vs. K_d values
Leach's Method
Section 2
Six Mile Creek, Arkansas.

Sheet 2 of 2

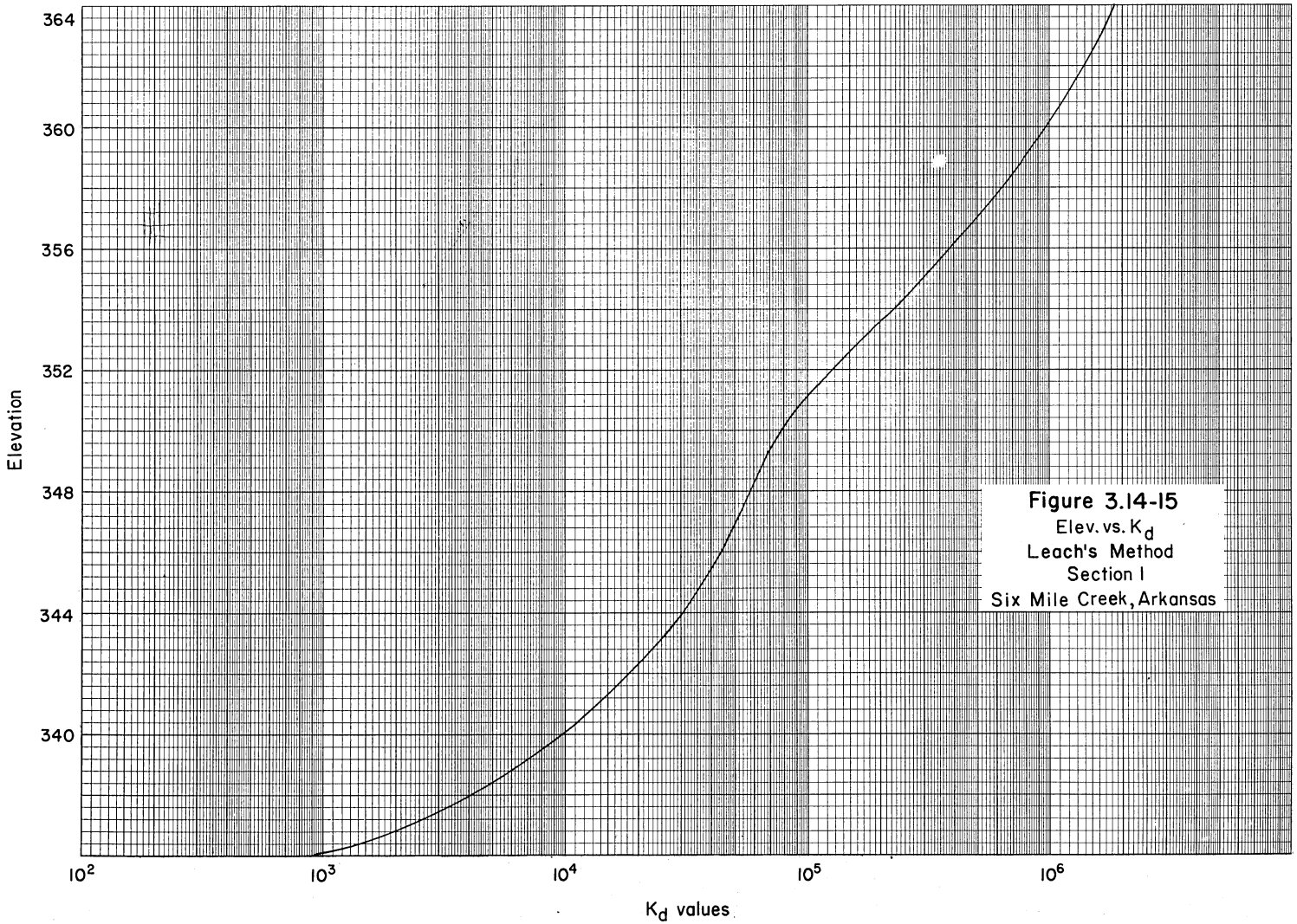


Figure 3.14-15
Elev. vs. K_d
Leach's Method
Section I
Six Mile Creek, Arkansas

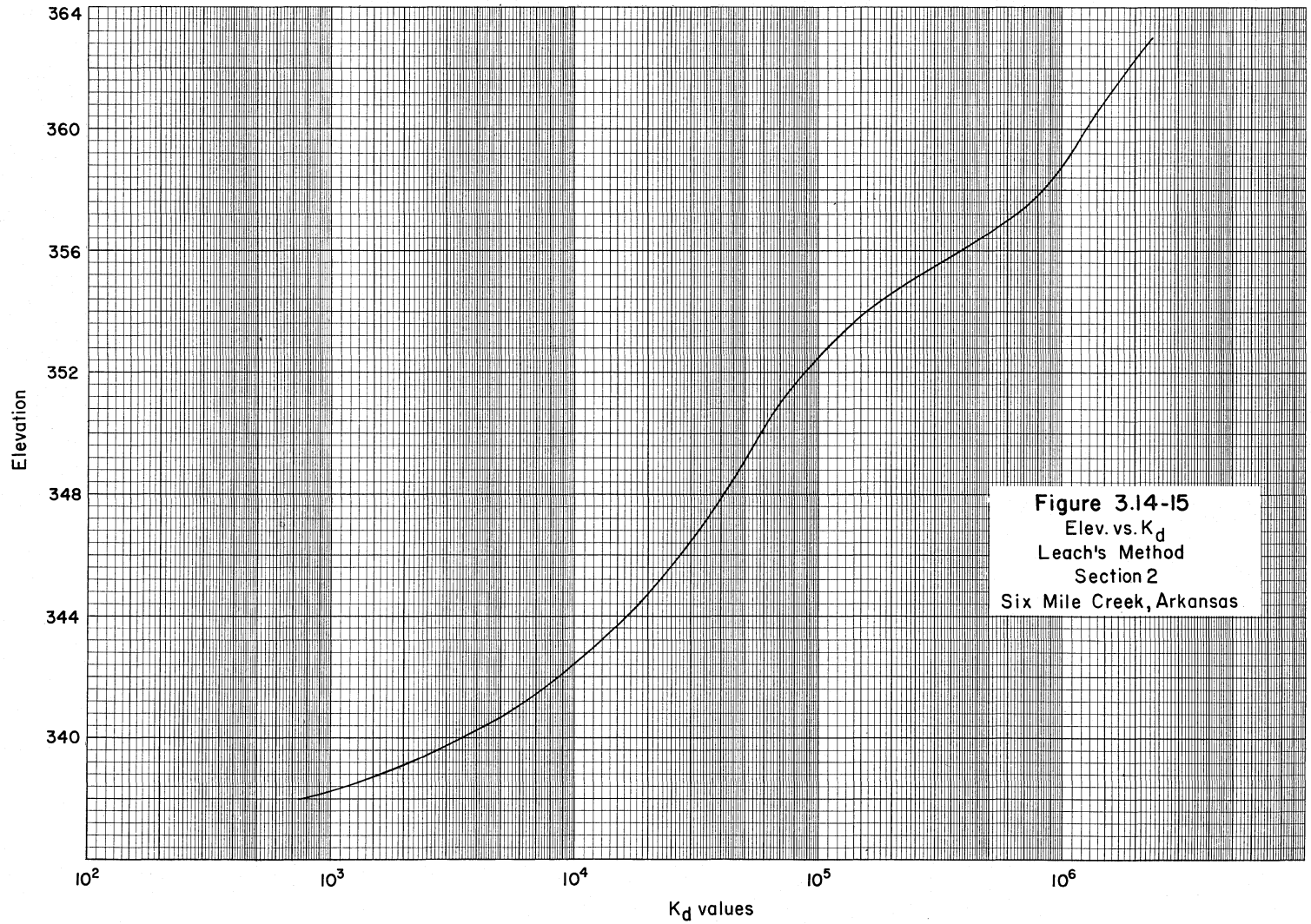


Figure 3.14-15
Elev. vs. K_d
Leach's Method
Section 2
Six Mile Creek, Arkansas

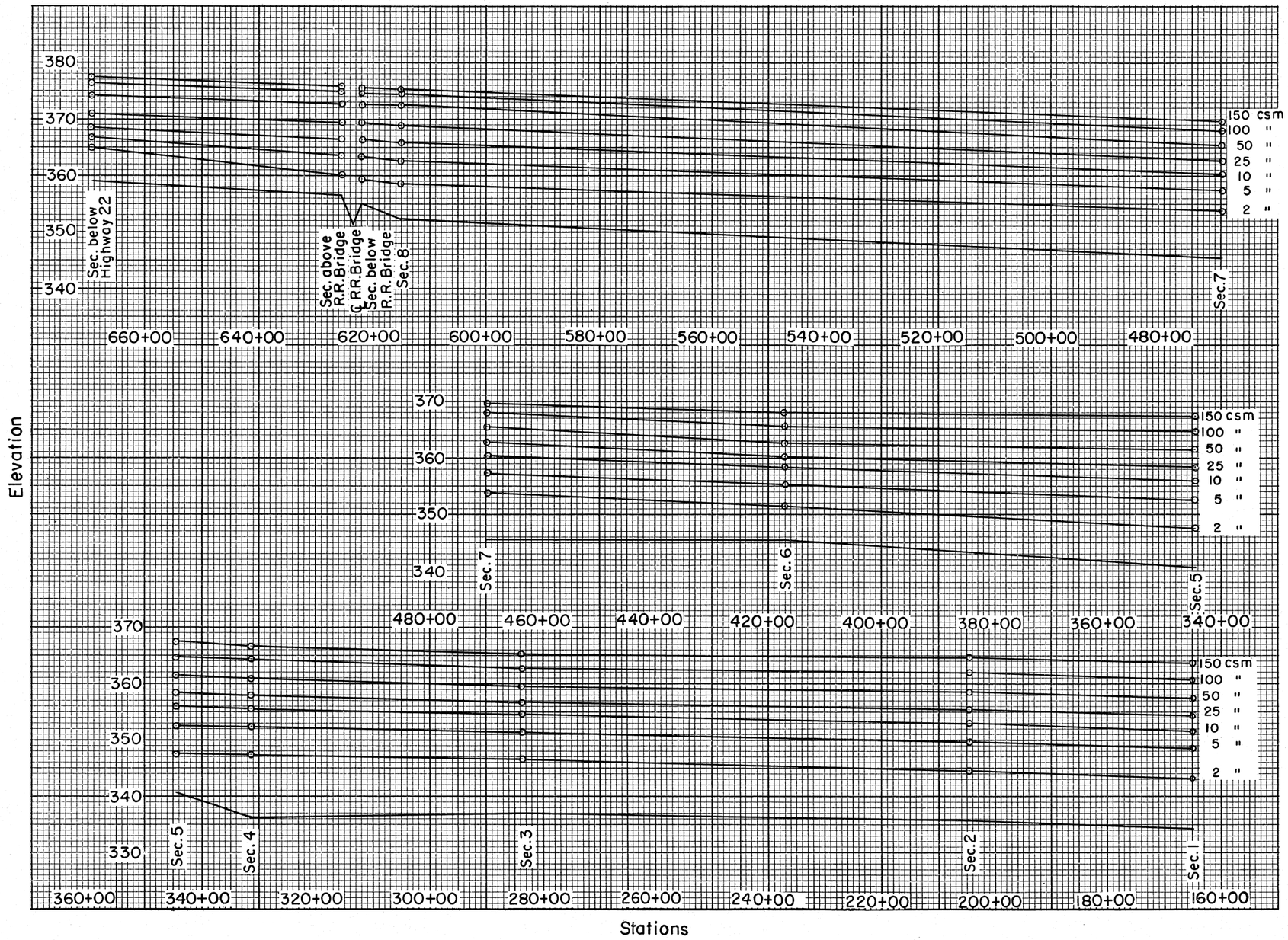


Figure 3.14-16
Profiles of Six Mile Creek, Arkansas.

Sheet 1 of 2

426963 O -57 (Face p. 176)

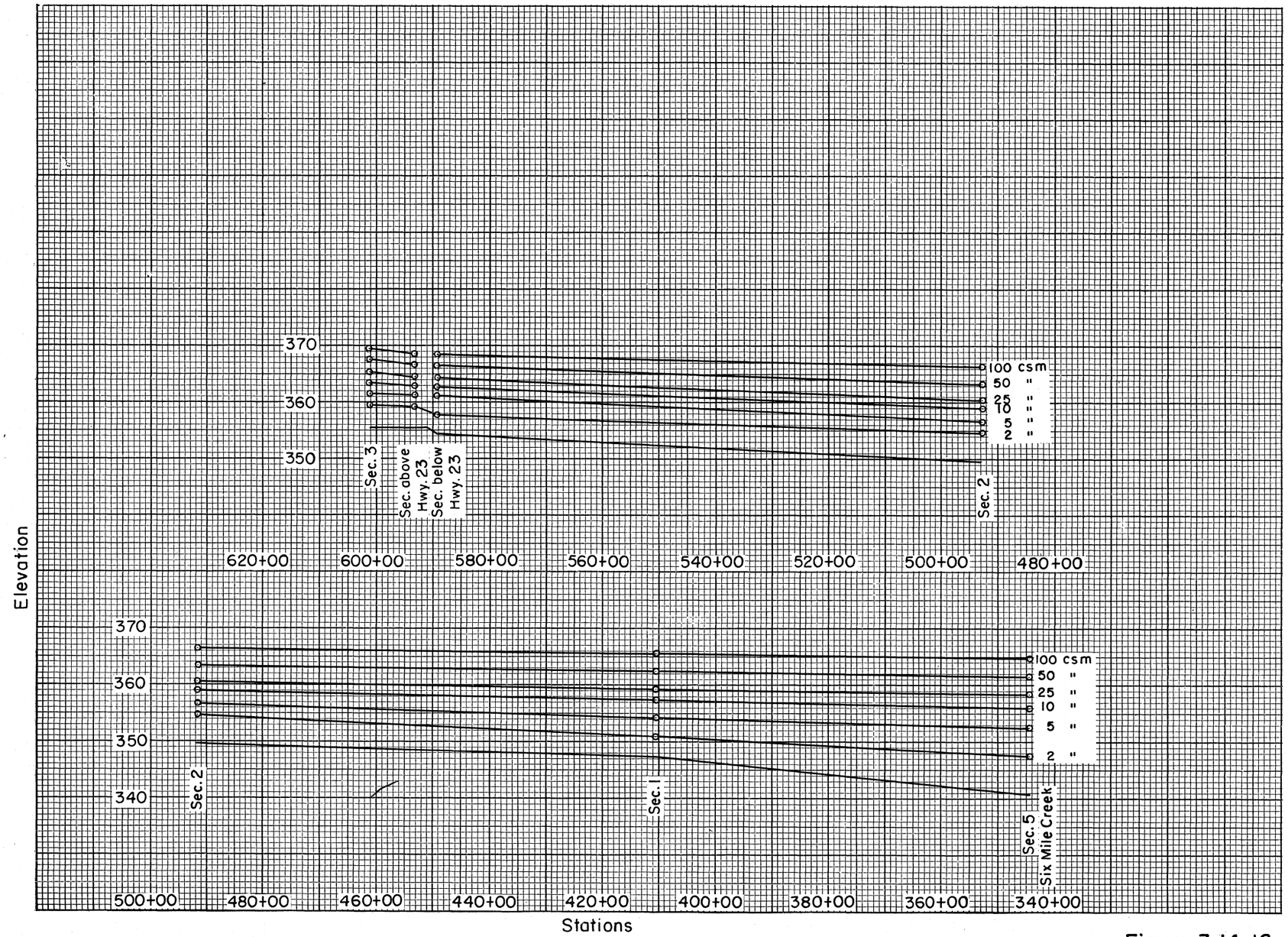


Figure 3.14-16

Profiles of Hurricane Creek, Arkansas.

Sheet 2 of 2

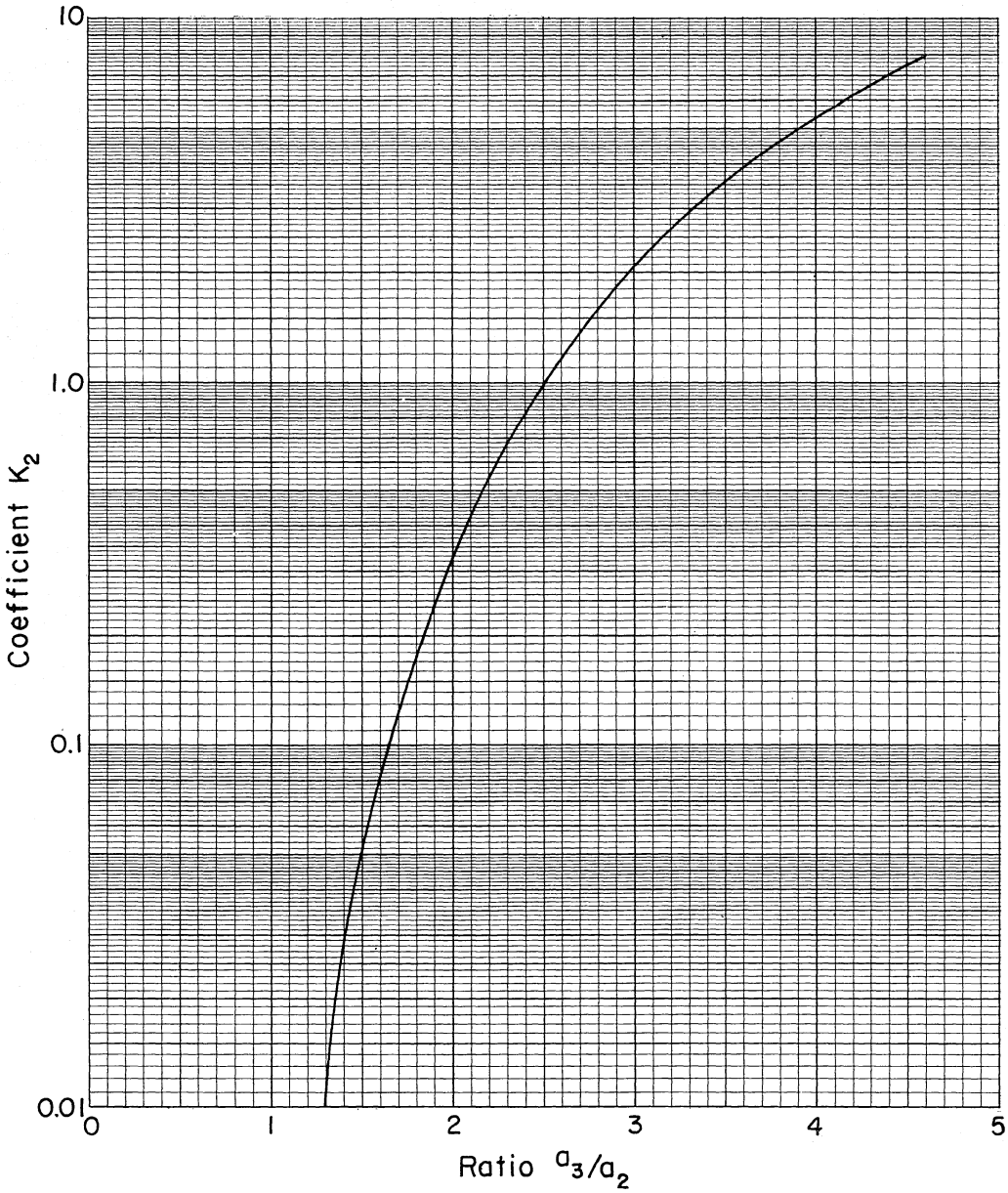


Figure 3.14-17
Coefficient of expansion loss, K_2, for use in equation 19

a_3 = Area of flow after expansion is completed.

a_2 = Area of flow at the contracted section.

426963 O - 57 - 12

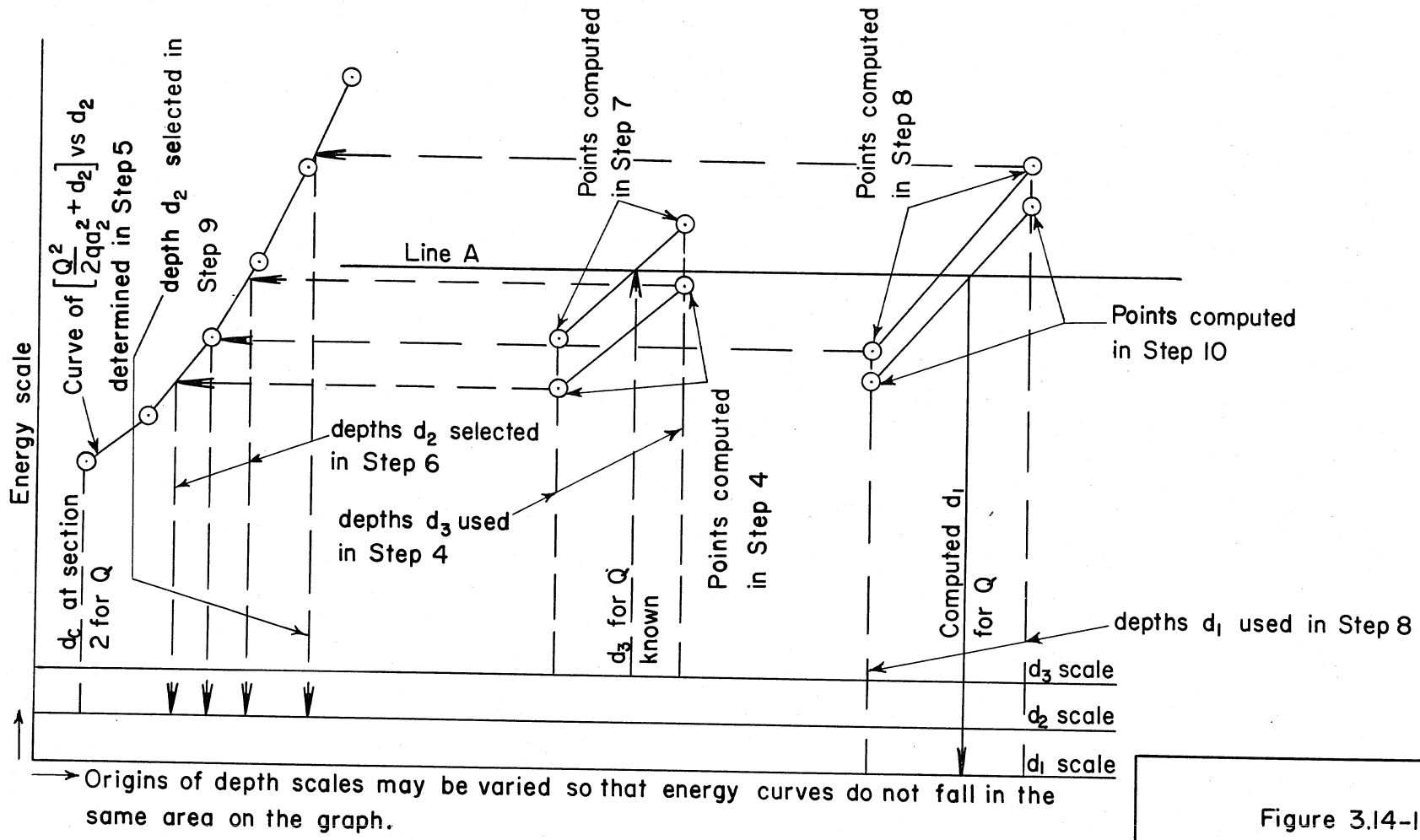


Figure 3.14-18
Schematic illustration of determination of stage upstream from a bridge

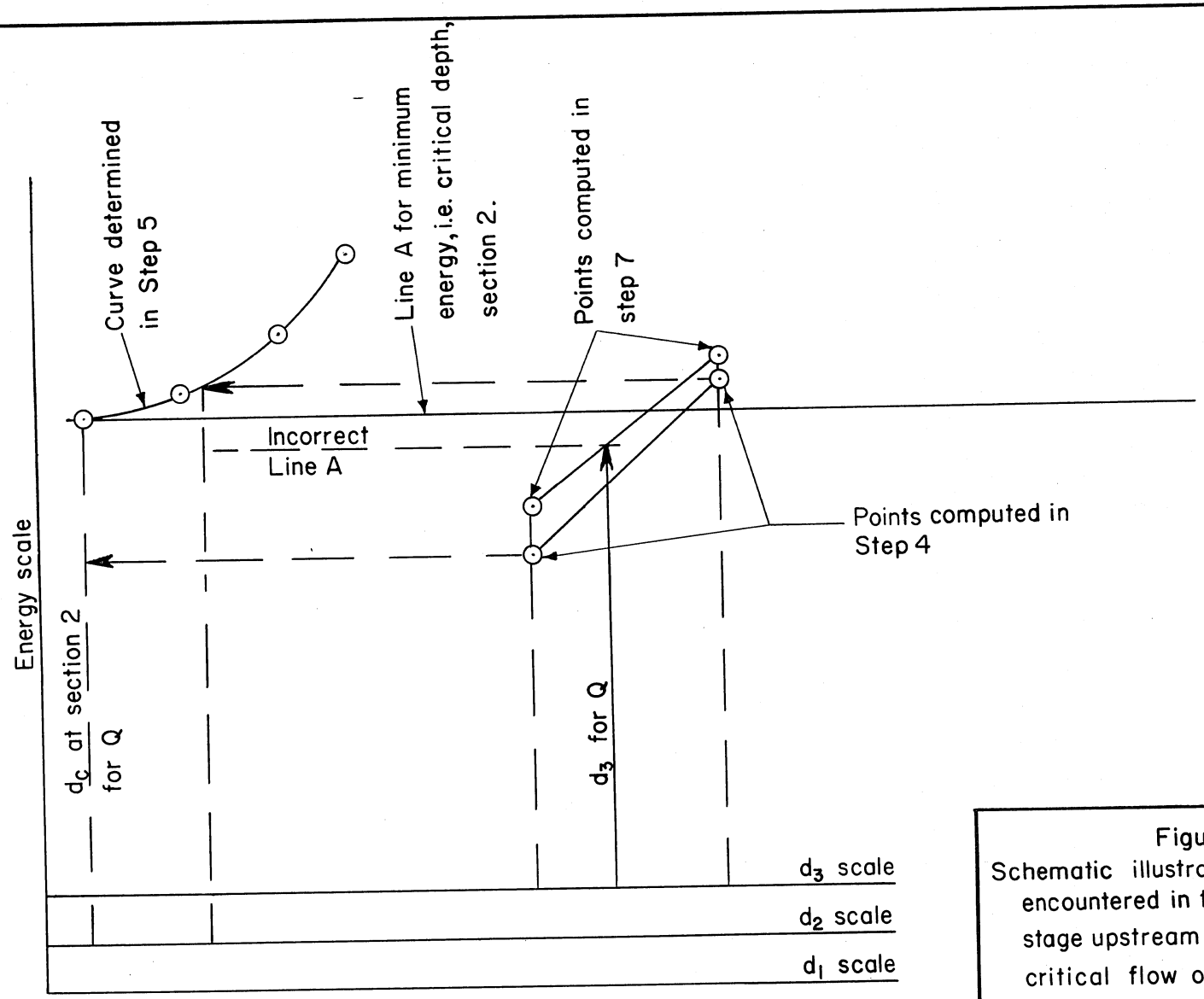
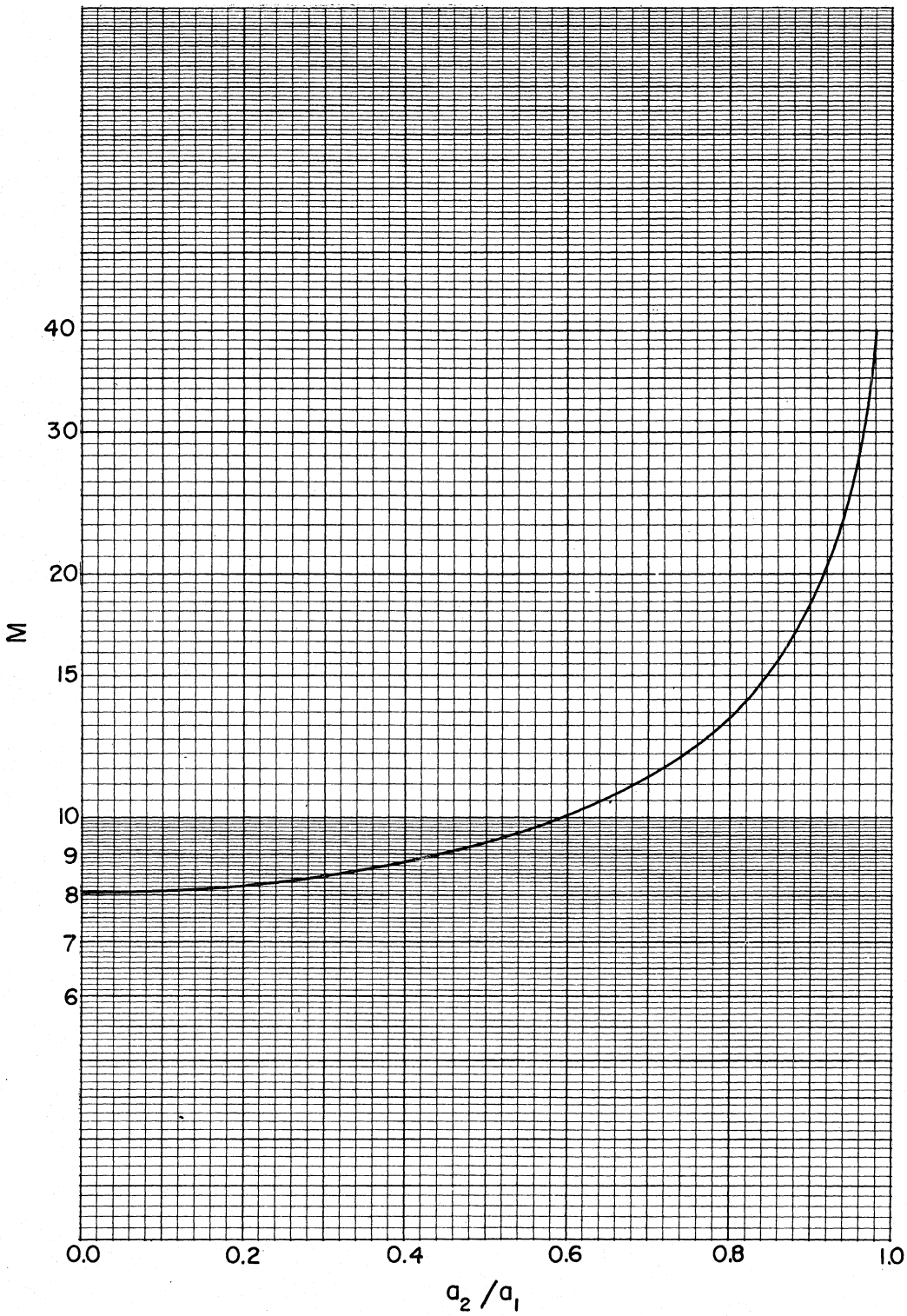


Figure 3.14-19
Schematic illustration of conditions encountered in the determination of stage upstream from a bridge where critical flow occurs at section 2.



a_1 = area of flow at section upstream from bridge.

a_2 = area of flow at the contracted section.

Figure 3.14-20
Values of M for use in Equation 22

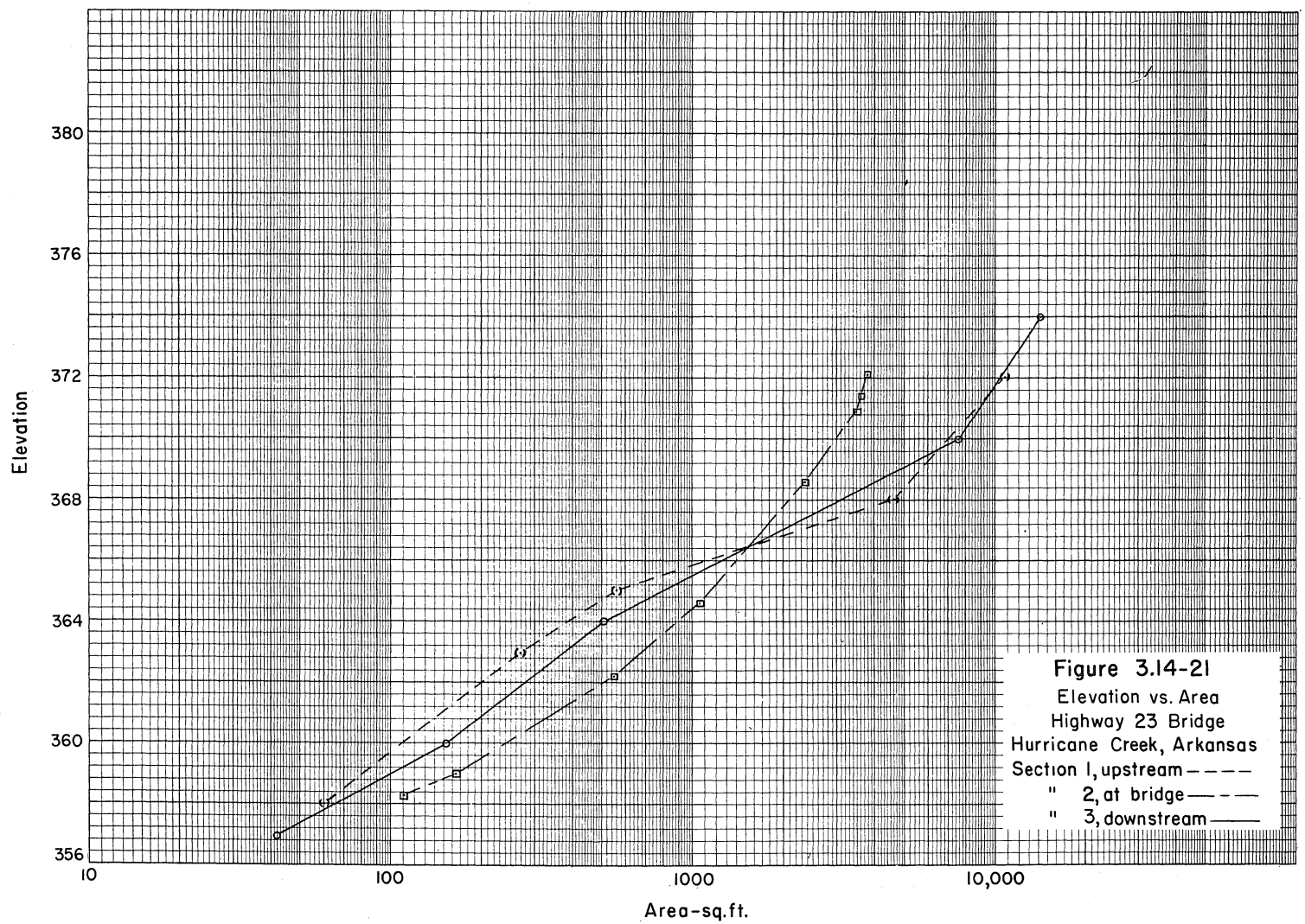


Figure 3.14-21
Elevation vs. Area
Highway 23 Bridge
Hurricane Creek, Arkansas
Section 1, upstream - - - -
" 2, at bridge — - —
" 3, downstream ——

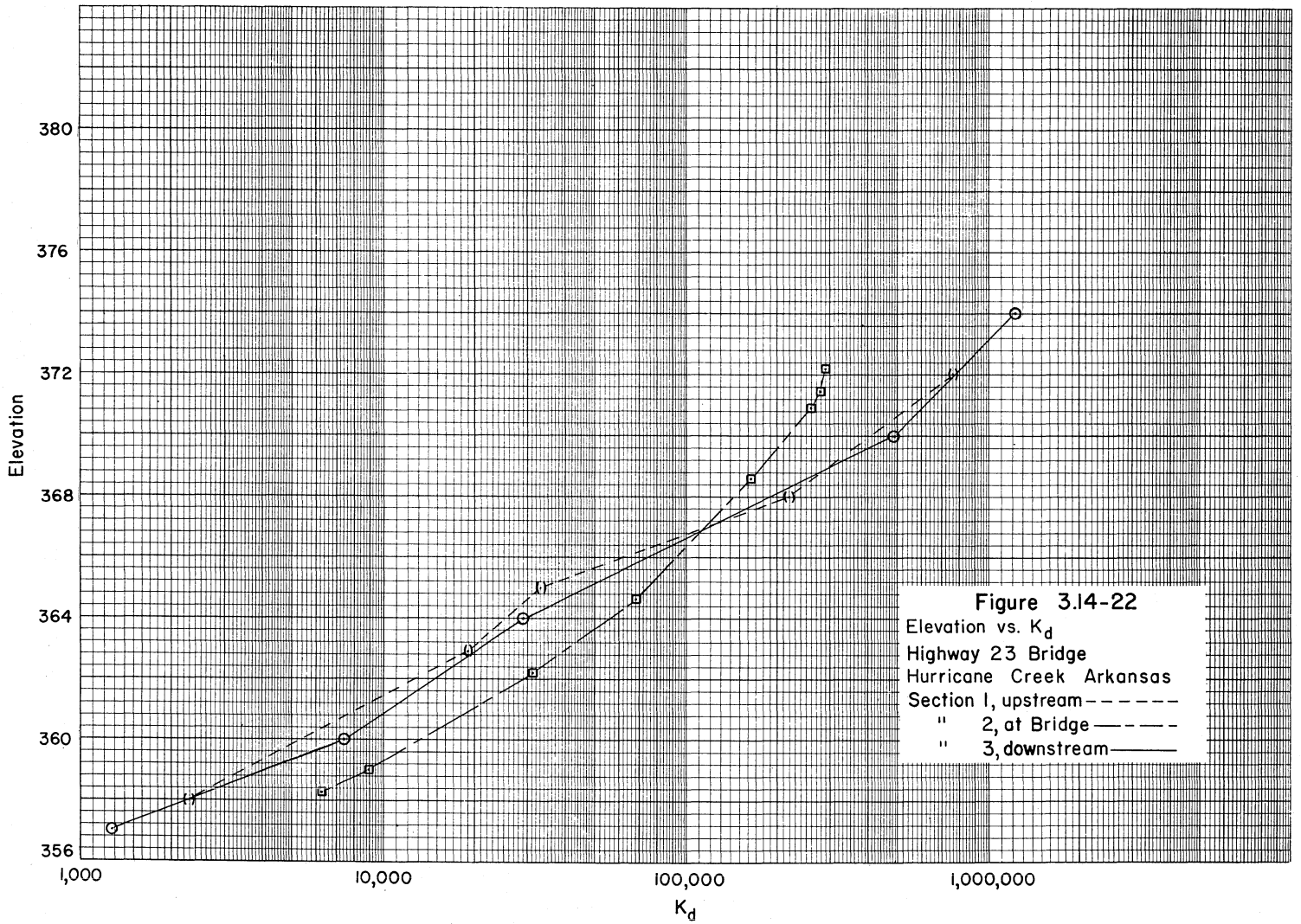


Figure 3.14-22
Elevation vs. K_d
Highway 23 Bridge
Hurricane Creek Arkansas
Section 1, upstream - - - - - -
" 2, at Bridge — - — - -
" 3, downstream ———

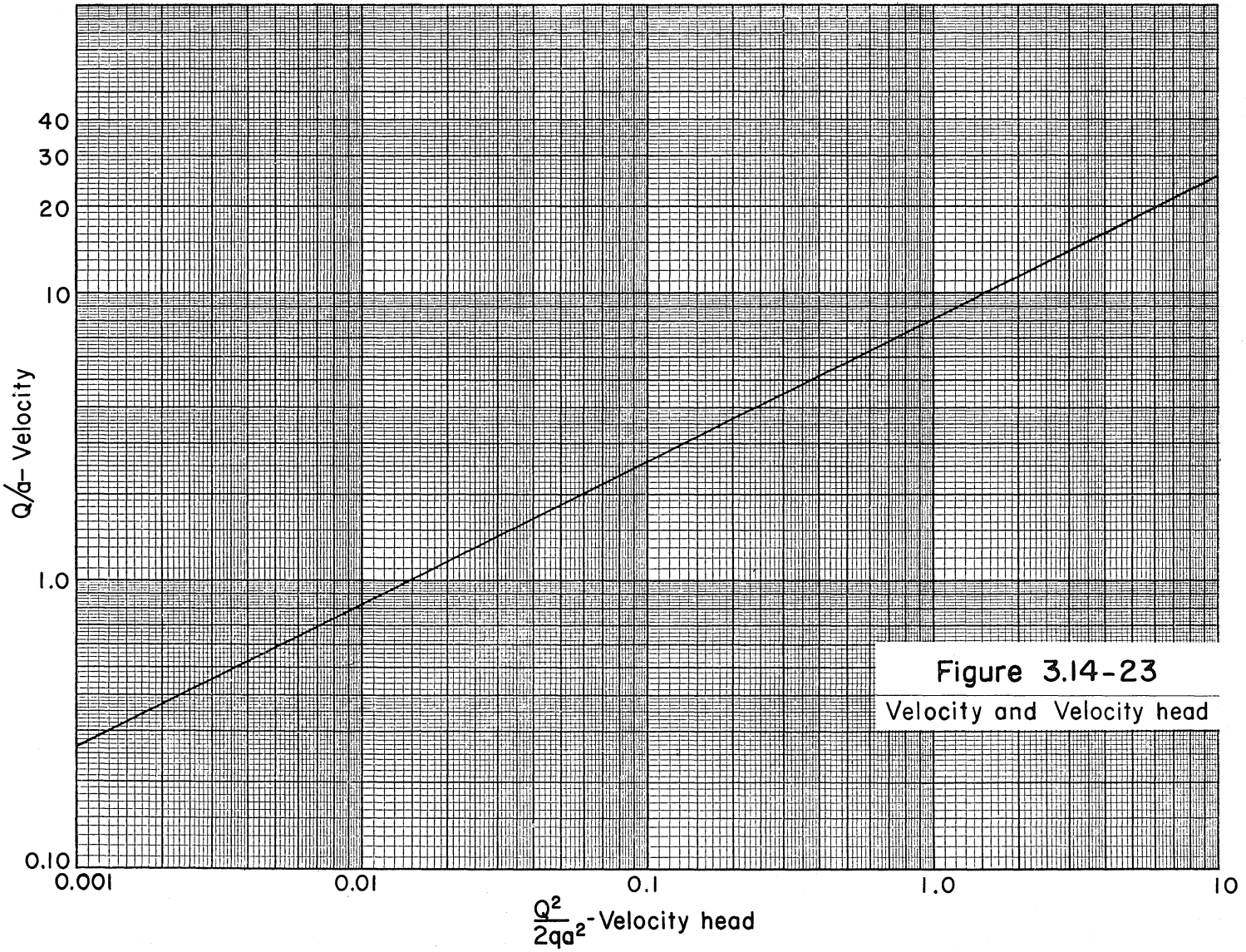


Figure 3.14-23

Velocity and Velocity head

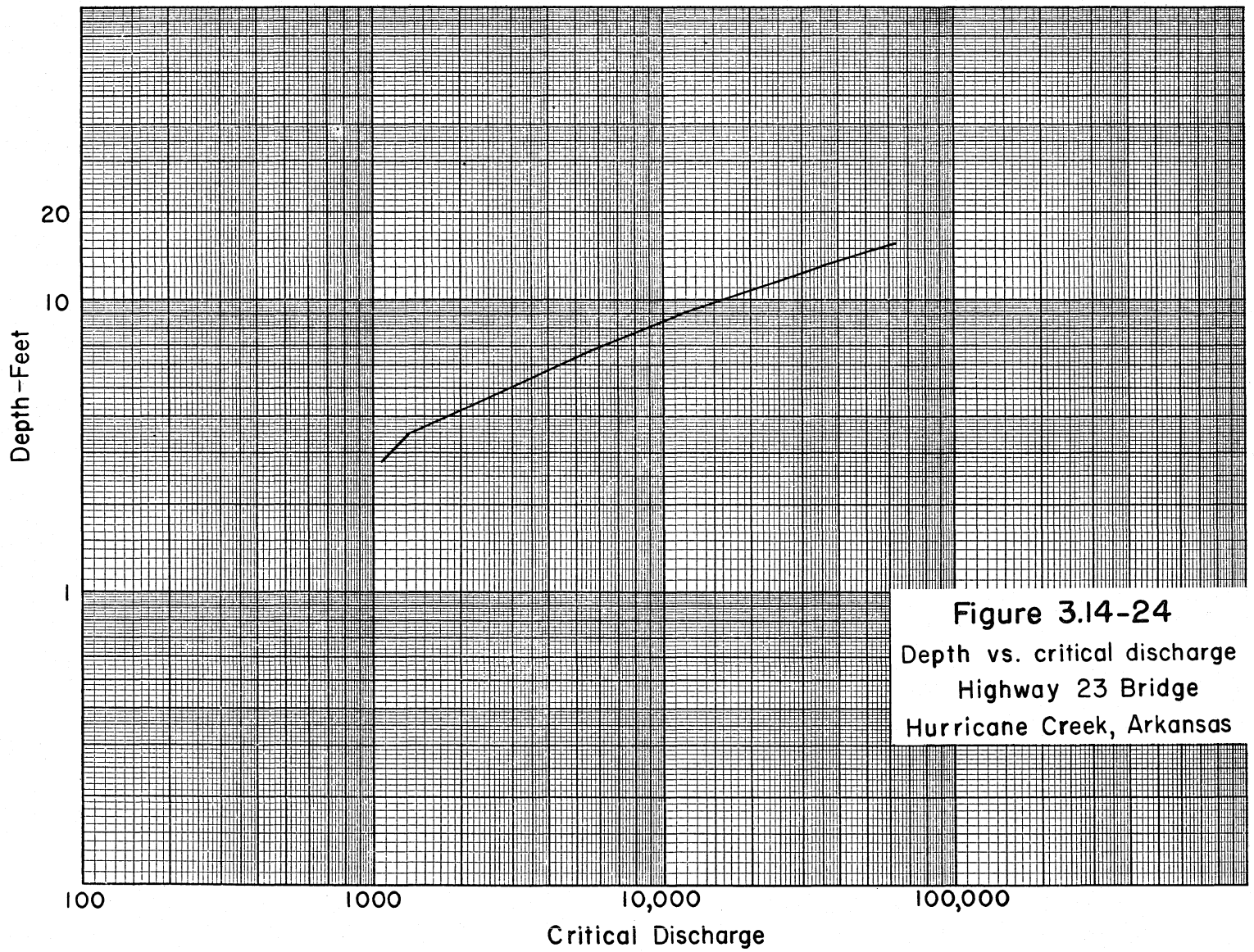


Figure 3.14-24
Depth vs. critical discharge
Highway 23 Bridge
Hurricane Creek, Arkansas

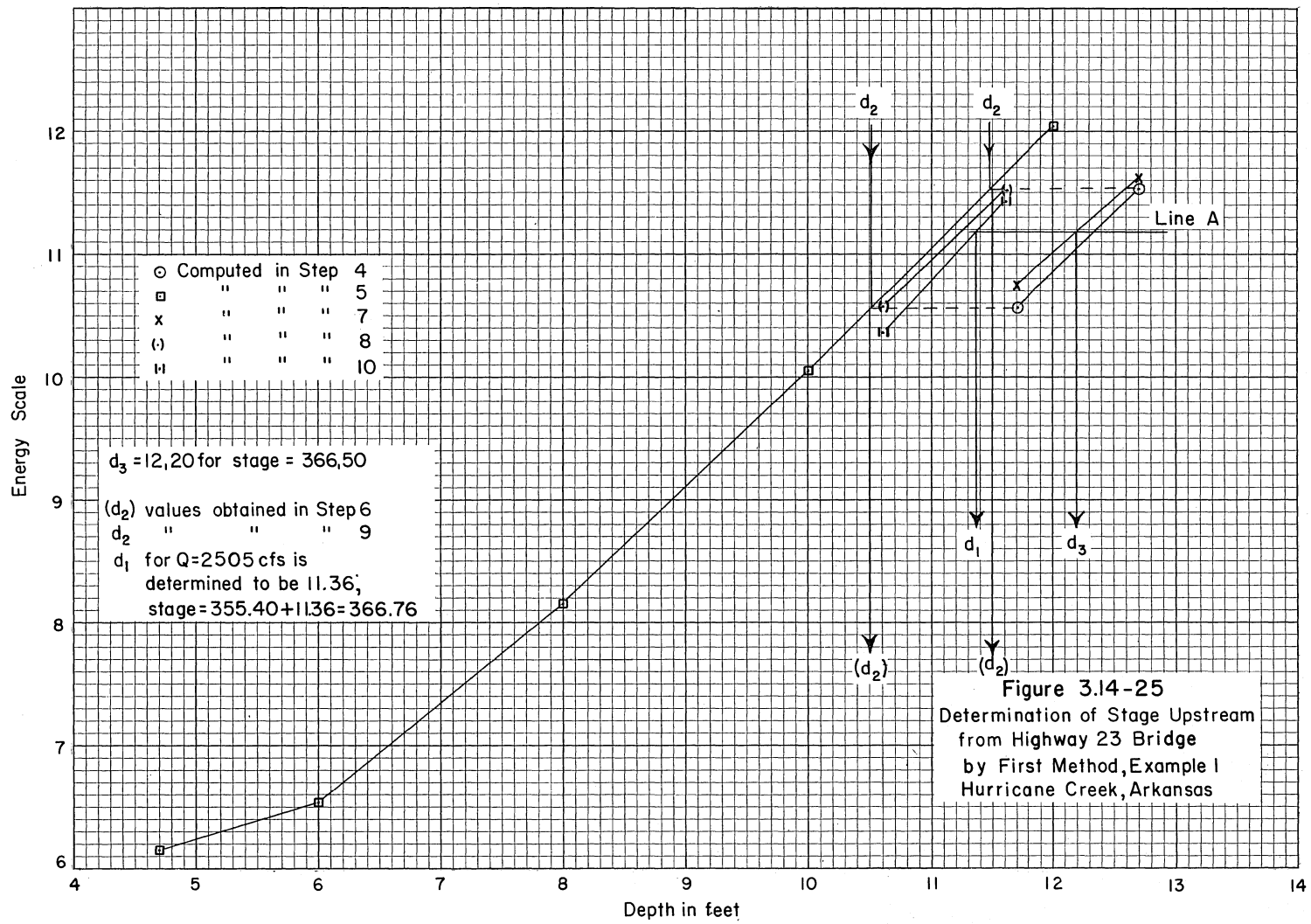
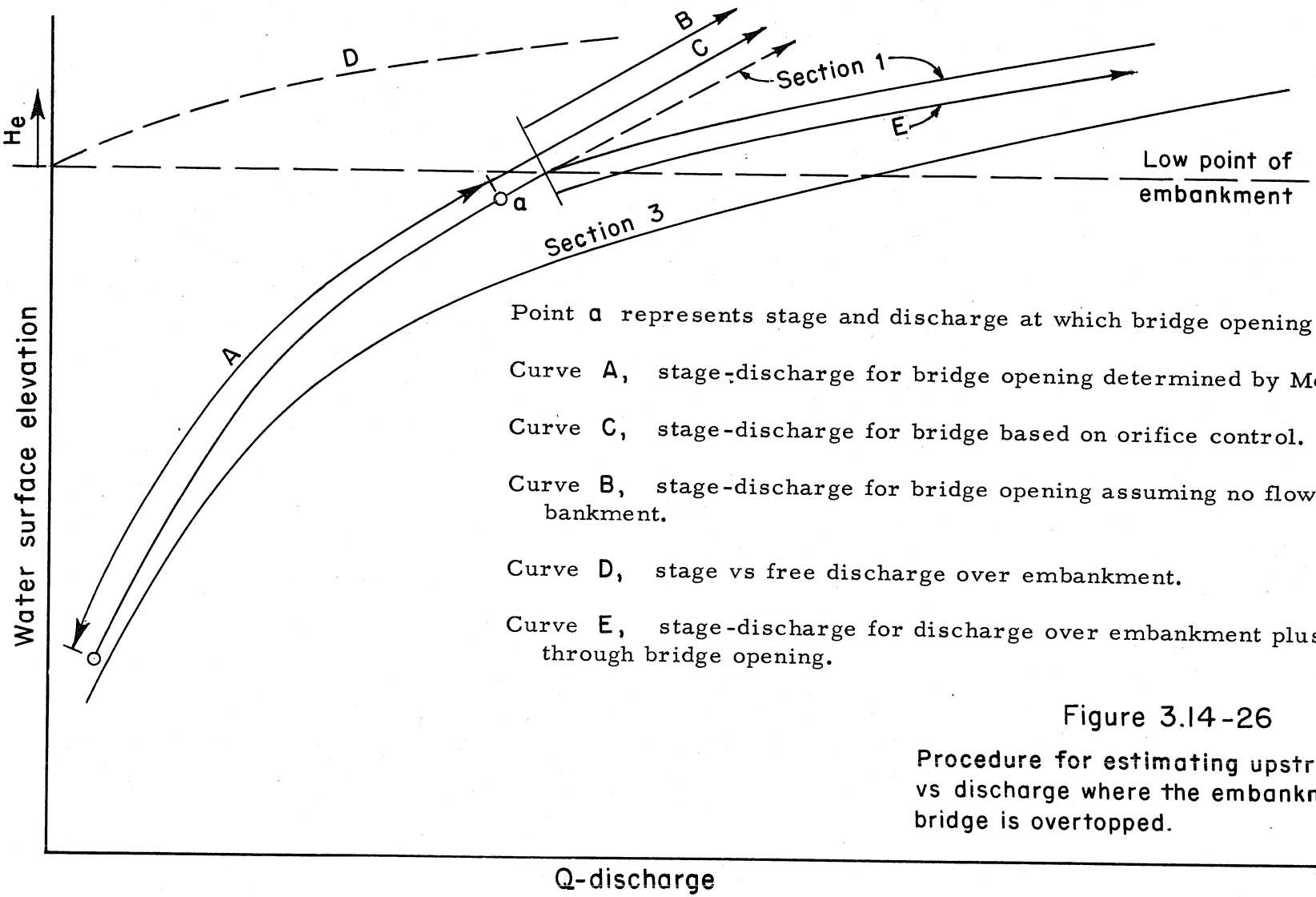


Figure 3.14-25
Determination of Stage Upstream from Highway 23 Bridge by First Method, Example I
Hurricane Creek, Arkansas



Point a represents stage and discharge at which bridge opening flows full.

Curve A, stage-discharge for bridge opening determined by Method 1 or 2.

Curve C, stage-discharge for bridge based on orifice control.

Curve B, stage-discharge for bridge opening assuming no flow over embankment.

Curve D, stage vs free discharge over embankment.

Curve E, stage-discharge for discharge over embankment plus discharge through bridge opening.

Figure 3.14-26

Procedure for estimating upstream stage vs discharge where the embankment at a bridge is overtopped.

3.15 Time of concentration and lag

Purpose. Lag is a watershed characteristic on which information is required for determining peak flow times and rates. Lags can be most easily estimated by using times of concentration, and this section presents ways by which times of concentration can be determined.

Physical significance. Time of concentration (T_c) is the time it takes for water to travel from the hydraulically most distant part of a watershed to the watershed outlet. In hydrograph analysis, it is the time from the end of excessive rainfall to the point on the falling limb of the hydrograph where the recession curve begins.

Lag (L) may be thought of as a weighted time of concentration. If for a given storm the watershed is divided into increments, and the travel times from the centers of the increments to the main watershed outlet are determined, then the lag is:

$$L = \frac{\sum(a_x Q_x T_x)}{A Q_a} \qquad (1)$$

where L = the lag in hours
a_x = the x-th increment of area in square miles
Q_x = the runoff in inches from area a_x
T_x = the travel time in hours from the center of a_x to the main watershed outlet
A = the total area of the watershed in square miles
Q_a = the average runoff in inches from the total area or $\sum(a_x Q_x)/A$

When runoff is uniform (or nearly so) on a watershed or subwatershed, it is usually sufficient to estimate the lag from the empirical equation:

$$L = 0.6\ T_c \qquad (2)$$

When watershed drainage patterns are complex, they may be treated as discussed in Section 3.16. Methods for determining the time of concentration are described below.

Variability of T_c. Since T_c depends on the velocity of streamflow, it will vary because of changes in vegetation, slope, and other hydraulic elements of the stream channels. In most cases, this variation in T_c is less important than the variation in storm characteristics, and T_c can be assumed to be a constant. It should be remembered, however, that when a storm covers only a part of a watershed, the time of concentration is from the hydraulically most distant part of the excess rainfall area in that storm to the watershed outlet.

For example, when a storm occurs on the lower half of a watershed, the applicable T_c is obtained by computing the water travel time from the watershed outlet to the hydraulically farthest point in the storm area where runoff occurred.

With these considerations in mind, a time of concentration can be determined for any watershed and storm combination.

T_c from hydrographs. In hydrograph analysis T_c is the time from the end of the excessive rainfall to the point on the falling side of the hydrograph where the recession curve begins. That point is nearly always difficult to determine and a less precise but more practical method may be used. The steps of the method are:

1. Obtain several hydrographs for the watershed.

2. If there is base flow, separate this flow from the hydrographs (Section 3.5).

3. Plot the recession curves of the hydrographs on semi-log paper, as shown on figure 3.15-1, using the portion of a recession that occurs after rainfall has stopped.

4. Assuming a straight line through each recession plotting, find the slope of each recession line by the equation:

$$b = \frac{\log q_1 - \log q_2}{t_2 - t_1} \qquad (3)$$

where b = the slope (Note: The slope of a recession line is always negative, but equation 3 has been arranged to give a positive value to simplify the application.)

$\log q_1$ = the common logarithm (base 10) of the discharge in cfs at time t_1 in hours

$\log q_2$ = the common logarithm of the discharge in cfs at time t_2 in hours.

5. Average the slopes.

6. Using the average b, find T_c by the equation:

$$T_c = \frac{0.71}{b} \qquad (4)$$

(Note: The coefficient 0.71 is an average that applies in many but not all cases. The coefficient will differ for watersheds with channel losses, and it may vary widely for those with channel accretions. A check on the coefficient is made by determining T_c as the time from the end of excessive rainfall to the recession point of inflection, computing b, and taking the product $b(T_c)$ as the coefficient.)

Table 3.15-1 shows details of the computations for an average T_c. Once the procedure becomes familiar, several shortcuts will become apparent. For example, the three recession lines can be averaged graphically and some arithmetic eliminated.

T_c from flood observations. An average time of concentration can be obtained using the time from beginning of rise to peak (T_p) in the equation:

$$T_p = \sqrt{T_c} + 0.6\ T_c \qquad (5)$$

Figure 3.15-2 permits an easy solution for T_c once the T_p is known. Several values of T_p should be averaged in order to improve the estimate of T_c. The T_p values should be for simple hydrographs or from field observations of simple hydrographs.

T_c from stream hydraulics. This method is recommended for the usual case where no hydrographs are available. Previous computations of the field survey data give the necessary stream lengths and flow velocities.

1. Find the approximate channel bankfull discharge for each reach. (Note: The 2-year frequency discharge is a better value to use, if it is available.)

2. Compute the average velocity for the bankfull discharge of each reach.

3. Use the average velocity and the valley length of the reach to compute the travel time through the reach.

4. Add the travel times of Step 3 to get the T_c for the watershed.

Note that channel bankfull discharges and valley lengths are used. This is a practical compromise to get an average T_c. For special cases, use whatever average velocities and lengths are appropriate.

In some cases, the stream data do not extend to the watershed ridge. The remaining T_c can be estimated by the method given below.

Table 3.15-1. Details of computation of T_c from hydrograph recession lines, Buckhannon River at Buckhannon, West Virginia. See Figure 3.15-1.

1.	Storm date:	March 30-31,1940	May 16-17,1941	Sept.24-25, 1940
2.	q_1 1/ (cfs)	2650	2100	760 2/
3.	q_2 1/ (cfs)	350	310	137
4.	log q_1	3.424	3.323	2.881
5.	log q_2	2.544	2.492	2.136
6.	(4) - (5)	0.880	0.831	0.745
7.	t_1 hours	0	0	0
8.	t_2	50	50	50
9.	$(t_2 - t_1)$	50	50	50
10.	(6)/(9) = b	0.0176	0.0166	0.0149
11.	$0.71/b = T_c$ in hours	40.3	42.7	47.7
12.	Average T_c in hours	$\frac{40.3 + 42.7 + 47.7}{3} = 43.6$		

1/ Base flow separation has been made.

2/ From extension of lower portion of recession line, figure 3.15-1.

T_c from watershed approximate slope. The nomograph, figure 3.15-3, was developed from Kirpich's paper, which is based on small watershed data. The original material was intended to give the time from beginning of rise to peak of hydrograph. As used here, the equation is taken to give the T_c as defined above.

1. Find the length (L) of the longest waterway from the watershed outlet to the ridge, in feet.

2. Get the difference in elevation (H) between the watershed outlet and the farthermost point, in feet, but omit drops due to gully overfalls, waterfalls, etc.

3. Use figure 3.15-3 to estimate T_c, in hours.

Since the equation is based only on a rough consideration of stream channel slope, and also neglects Manning's n and hydraulic radius, it gives only a rough approximation of T_c.

Reference:

Kirpich, P. Z. Time of concentration of small agricultural watersheds. Civil Engineering, June 1940.

T_c as a function of area. When T_c values are available for watersheds with similar topography, a plotting of T_c versus drainage area, on log paper, may show a close relation from which T_c values for other watersheds with such topography may be estimated. Figure 3.15-4 shows typical relations obtained using hydrograph data from watersheds in Texas and Ohio. Note that while the fit is good for the Ohio data, the Texas data plot erratically because of a less consistent drainage pattern. It is not desirable to extend the lines very much beyond the data. The lines have been drawn with a slope of 0.6, but this slope may not apply to other data nor to areas beyond the range shown.

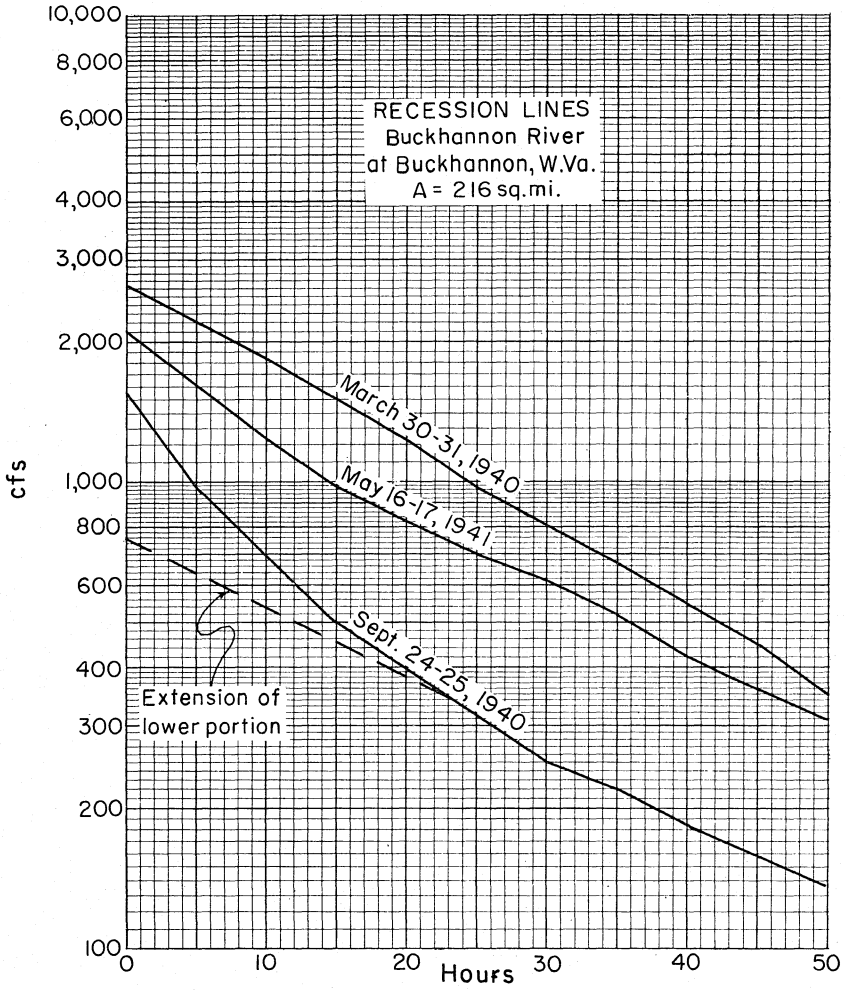


Figure 3.15-1

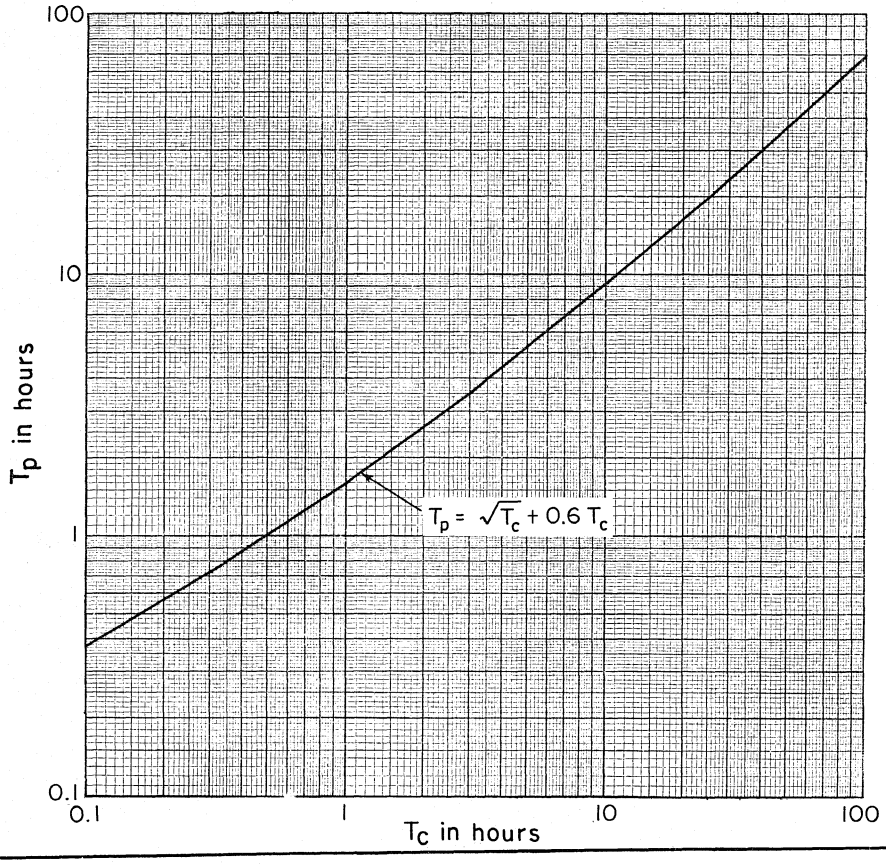


Figure 3.15-2

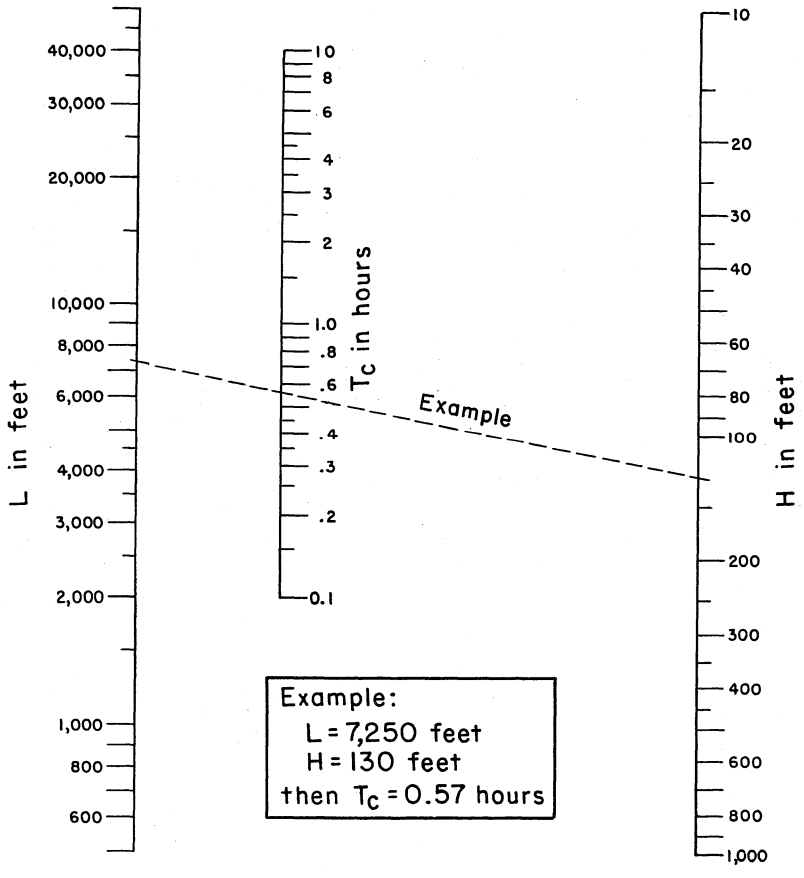


Figure 3.15-3

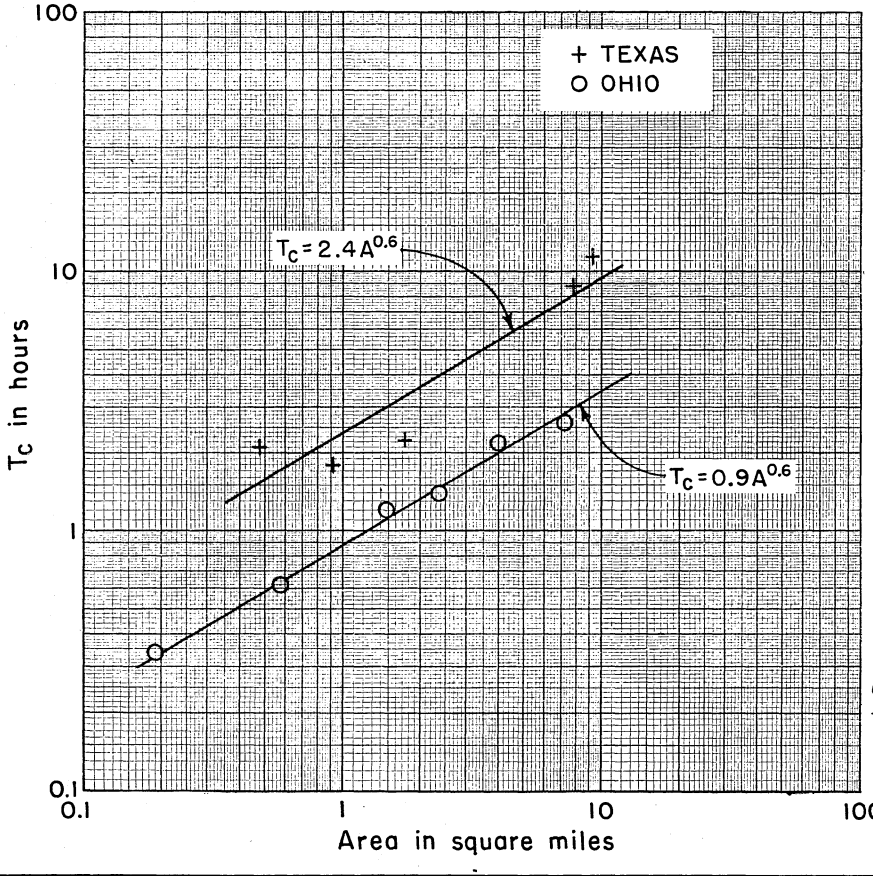


Figure 3.15-4

EACH GROUP OF POINTS FOR TEXAS + OHIO REPRESENT A SMALL LOCALITY IN THOSE STATES, AND NOT THE ENTIRE STATES.

426963 O - 57 - 13

3.16 Hydrographs

Purpose. Hydrographs, or some elements of them such as peak rates, are used in the design of watershed measures, or to show the hydrologic effects of existing and proposed watershed projects. Knowledge of the hydrologic effects is used by the economist in the determination of economic effects.

Types of hydrographs. It is customary to speak of natural hydrographs, which are obtained directly from the stage records of gaged watersheds; and synthetic hydrographs, which are obtained by using watershed and storm characteristics to modify a given average hydrograph that is based on flow records.

A simple hydrograph has only one rise, one peak, and one recession. A complex hydrograph is any combination of simple hydrographs, and has more than one rise, or peak, or recession.

The unit hydrograph is a simple natural or synthetic hydrograph for 1 inch of direct runoff from the watershed occurring in a specified unit of time. Unit hydrograph relations are discussed only briefly in this section. A more complete discussion will be included in the Hydrology Section of the National Engineering Handbook. A "Preliminary Draft, Subsection on Unit Hydrograph; National Engineering Handbook, Section 4, Hydrology" was distributed in 1954 to State Conservation Engineers and Engineering and Watershed Planning Unit hydrologists.

A dimensionless hydrograph is made from simple natural hydrographs by taking the time to peak (T_p) and peak rate (q_p) as units, and plotting t/T_p versus q/q_p, where t is a given time and q is the rate of flow at the given time.

Elements of a hydrograph. Figure 3.16-1(a) shows a typical simple hydrograph of runoff from rainfall. The graph is self-explanatory.

For further analysis, the excess rainfall and the runoff are given simple geometrical shapes, as shown in figure 3.16-1(b). The excess rainfall is best represented by an average rate, since any storm, however complex, can be closely represented by subdivision into short time increments with average rates. One triangular or curvilinear hydrograph is associated with each such increment of excess rainfall. The representation of a curvilinear hydrograph by a triangular hydrograph is a useful tool, since much time is saved and little accuracy is lost. Such representation means that an increase or decrease in the dimensions of the triangular hydrograph is accompanied by a similar change in the curvilinear hydrograph.

The elements shown on figure 3.16-1 can be used to develop a useful equation for peak rates of flow. Using the symbols shown on the figure, the runoff volume of the hydrograph of figure 3.16-1(b) is

$$Q = \frac{q_i \, T_p}{2} + \frac{q_i \, T_r}{2} \qquad (1)$$

$$= \frac{q_i}{2} \, (T_p + T_r) \qquad (2)$$

with units of inches and hours. Solving for the peak rate (q_i) in inches per hour gives:

$$q_i = \frac{2\,Q}{T_p + T_r} \qquad (3)$$

The empirical relation of T_p to T_r as found by the analysis of a great number of natural hydrographs is shown below in the discussion of triangular hydrographs. This relation when applied to triangular hydrographs is:

$$T_r = 1.67\, T_p \qquad (4)$$

so that equation 3 can be written:

$$q_i = \frac{2\,Q}{T_p + 1.67\, T_p} = \frac{2\,Q}{2.67\, T_p} \qquad (5)$$

so that

$$q_i = \frac{0.75\,Q}{T_p} \qquad (6)$$

which can be used to get peak rates in inches per hour (q_i). To get the peak rate in cfs (q), multiply by 645 A, where A is the drainage area in square miles, and 645 is the conversion constant. This gives:

$$q = \frac{484\,A\,Q}{T_p} \qquad (7)$$

but figure 3.16 1(b) shows that:

$$T_p = \frac{D}{2} + L \qquad (8)$$

where D is the duration of excess rainfall, in hours, and L is the watershed lag, in hours, so that:

$$q = \frac{484\,A\,Q}{\frac{D}{2} + L} \qquad (9)$$

The average relation of lag (L) to time of concentration (T_c) is given in Section 3.15 as

$$L = 0.6\ T_c \tag{10}$$

so that equation 9 can be written:

$$q = \frac{484\ A\ Q}{\frac{D}{2} + 0.6\ T_c} \tag{11}$$

This equation may be used to compute peak discharges. The nature of the relation on figure 3.16-1(b) implies a limit to D, in terms of its proportion to T_c, so that generally D should not be greater than T_c. The effects of using and not using such a limit are illustrated for selected cases in the subsection on composite hydrographs.

A nomograph for the solution of equation 11 is given in figure 3.16-2.

General relations between variables. Actual values of q and Q for a watershed are plotted on figure 3.12-1. Equation 7 shows that with A and Q constant, q varies inversely with T_p. In other words, if A = 170 and Q = 1", the q's for the various assumed values of T_p can be computed to give the straight lines shown on figure 3.16-4. The characteristic T_p for this watershed is represented by the line on figure 3.12-1, and is

$$T_p = \frac{484\ A\ Q}{q} = \frac{(484)\ (170)\ (1)}{3650} = 22.5 \text{ hours}$$

When the T_c is known (Section 3.15) the duration defined by this line (D_a) can be computed using equations 8 and 10 to get:

$$D_a = 2\ T_p - 1.2\ T_c \tag{12}$$

An investigation of D_a values for similar plottings of data from many watersheds and their relation to T_c values gave:

$$D_a = 2\sqrt{T_c} \tag{13}$$

When this is used in equation 8, the T_p equation becomes:

$$T_p = \sqrt{T_c} + 0.6\ T_c \tag{14}$$

and equation 11 becomes

$$q = \frac{484\ A\ Q}{\sqrt{T_c} + 0.6\ T_c} \qquad (15)$$

for the duration represented in the straight line drawn through the points of a q versus Q plotting, as shown on figure 3.12-1. Equations 12 through 15 are primarily useful in preparing flood frequency lines, where a long series of data is averaged out, and should not be used for individual floods.

Dimensionless hydrograph. Table 3.16-1 gives ratios and figure 3.16-3 shows a plotting of an average dimensionless hydrograph that was obtained by first plotting without dimensions (using q/q_p and T/T_p values) a large number of natural unit hydrographs for watersheds varying widely in size and geographical location. From these an average dimensionless hydrograph was determined and then smoothed by a flood-routing procedure.

The peak rate (q_p), time to peak (T_p), and total runoff (Q), of this hydrograph are related to each other by equations 7, 8, and 10. Only two of these factors need be known to use the hydrograph since the third can be computed for any given case. The peak for the hydrograph can be obtained using equation 11.

The procedure of making a hydrograph is shown on table 3.16-2. Note that all the time ratios of table 3.16-1 have not been used so that the plotted hydrograph will have some angularity. This can be removed if desired, by taking the time ratios at closer intervals and using a French curve to connect the points.

Comparison of hydrographs. Figure 3.16-5(a) is a dimensionless plotting comparing the dimensionless hydrograph with the Commons hydrograph and a hydrograph developed in the Milwaukee area.

Triangular hydrographs. Figure 3.16-5(b) shows a triangular hydrograph compared with the dimensionless hydrograph. The legs of the triangle are computed so that areas of both hydrographs are equal.

It is evident that the triangular hydrograph can usually be substituted for all three hydrographs shown on figure 3.16-5(a). Use of the triangular hydrograph usually speeds up reservoir routing, stream routing, and the preparation of complex hydrographs without a significant loss of accuracy. It should not be used, however, when the full effect of the curvilinear hydrograph must be presented.

Table 3.16-1. Ratios for basic dimensionless hydrograph and mass curve

Time ratios (T/T_p)	Hydrograph discharge ratios (q/q_p)	Mass curve ratios (Q_a/Q)
0	0	0
0.1	0.015	0.001
.2	.075	.006
.3	.16	.018
.4	.28	.037
.5	.43	.068
.6	.60	.110
.7	.77	.163
.8	.89	.223
.9	.97	.300
1.0	1.00	.375
1.1	.98	.450
1.2	.92	.517
1.3	.84	.577
1.4	.75	.634
1.5	.66	.683
1.6	.56	.727
1.8	.42	.796
2.0	.32	.848
2.2	.24	.888
2.4	.18	.916
2.6	.13	.938
2.8	.098	.954
3.0	.075	.967
3.5	.036	.984
4.0	.018	.993
4.5	.009	.997
5.0	.004	.999
infinity	0	1.000

Table 3.16-2. Computation of discharges for synthetic curvilinear hydrograph.

Given: $A = 8.0$ sq. mi.
$T_c = 3.0$ hrs.
$D = 2.0$ hrs.
$Q = 1$ in.

$$T_p = \frac{D}{2} + 0.6\ T_c = \frac{2}{2} + 0.6\ (3.0) = 2.8 \text{ hrs.}$$

$$q_p = \frac{484\ A\ Q}{T_p} = \frac{(484)(8.0)(1)}{2.8} = 1380 \text{ cfs}$$

Time ratios (From table 3.16-1) (T/T_p)	Time (Col. 1 x 2.8) (hours)	Discharge ratios (From table 3.16-1) (q/q_p)	Discharge (Col. 3 x 1380) (cfs)
0	0	0	0
0.2	0.56	0.075	103
.4	1.12	.28	387
.6	1.68	.60	828
.8	2.24	.89	1230
1.0	2.80 (T_p)	1.00	1380 (q_p)
1.2	3.36	.92	1280
1.4	3.92	.75	1040
1.6	4.48	.56	773
1.8	5.04	.42	580
2.0	5.60	.32	442
2.2	6.16	.24	331
2.4	6.72	.18	249
2.6	7.28	.13	180
2.8	7.84	.098	135
3.0	8.40	.075	103
3.5	9.80	.036	50
4.0	11.20	.018	25
4.5	12.60	.009	12
5.0	14.00	1/	1/

1/ Taken as zero to terminate hydrograph.

The time to peak (T_p) is

$$T_p = \frac{D}{2} + 0.6\ T_c \tag{16}$$

or as given in equation 8. The time from the peak to the end of the triangle (or the residual time T_r) is computed as:

$$T_r = 1.67\ T_p \tag{17}$$

so that the base time (T_b) of the triangle is:

$$T_b = T_p + T_r \tag{18}$$

$$= T_p + 1.67\ T_p$$

or

$$T_b = 2.67\ T_p \tag{19}$$

A convenient form in which to remember the relations is:

$$T_p = \frac{3}{8}\ T_b \tag{20}$$

$$T_r = \frac{5}{8}\ T_b \tag{21}$$

The triangular hydrograph data can be quickly obtained for plotting, as shown below:

<u>1. Given A, Q, and q_p</u>

Let A = 12 sq. mi.
Q = 2.5 inches
q_p = 8800 cfs

then $$t_p = \frac{484\ A\ Q}{q_p} \tag{22}$$

or $$T_p = \frac{(484)(12)(2.5)}{8800} = 1.65 \text{ hrs.}$$

and $$T_b = 2.67\ T_p$$

$$= 2.67\ (1.65) = 4.40 \text{ hrs.}$$

Plot the points at the start of rise, peak and end of base, and connect them with straight lines to complete the hydrograph.

2. Given A, T_c, D, and q_p

Let A = 17 sq. mi.

T_c = 6.0 hrs.

D = 3.0 hrs.

q_p = 12,500 cfs

$$\text{then } T_p = \frac{D}{2} + 0.6\ T_c \qquad (16)$$

$$= \frac{3.0}{2} + 0.6\ (6.0)$$

$$= 5.1 \text{ hours}$$

and using equation 19,

$$T_b = 2.67\ (5.1) = 13.6 \text{ hours.}$$

The hydrograph can be plotted as above. To find Q from the above data, use equation 7 to get:

$$Q = \frac{q_p\ T_p}{484\ A} \qquad (23)$$

$$\text{and } Q = \frac{12{,}500\ (5.1)}{484\ (17.0)} = 7.77 \text{ inches}$$

Other given values can be used with the basic equations 11, 16 and 19.

Composite hydrographs.

Effect of watershed drainage pattern. Figure 3.16-6(a) shows a watershed that can be divided into subwatersheds 1 and 2. The following data and computations are given for a storm that covers the area as given below:

Item	Subwatershed No. 1	Subwatershed No. 2
A (sq. mi.)	12.0	3.0
T_c (hours)	9.0	1.2
Q (inches)	2.4	2.4
D (hours)	2.1	2.1
T_p (eq. 16)	6.45	1.77
T_b (eq. 19)	17.2	4.72
q_p (eq. 7 or 11)	2,160	1,960

The individual and composite hydrographs are shown on figure 3.16-6(b). The simple hydrograph for A = 15.0 and T_c = 9.0 is also shown.

Effect of uneven runoff distribution. Figure 3.16-7(a) shows a watershed on which a storm has occurred that produced a runoff distribution so uneven that it must be accounted for in the hydrograph. The watershed is first divided into subwatersheds 1 and 2, as shown, and the average runoff in each is obtained. The following data and computations are given to obtain the composite hydrograph:

Item	Subwatershed No. 1	Subwatershed No. 2
A (sq. mi.)	6.0	6.0
T_c (hours)	2.3 (B to A)	3.9 (D to A)
Q (inches)	2.4	5.1
D (hours)	2.0	2.0
T_p (eq. 16)	2.38	3.34
T_b (eq. 19)	6.36	8.91
q_p (eq. 7 or 11)	2,920	4,430
T_t (passage time, steady flow, in hrs. C to A)	-	1.5
Setback time (0.5 T_t)	-	0.75

Figure 3.16-7(b) shows the composite hydrograph compared to a simple hydrograph obtained assuming a uniform distribution of runoff and with A = 12; T_c = 3.9; Q = 3.75; and D = 2.0. In other cases, the differences between the two large hydrographs may be more pronounced. Note that the hydrograph from the upper half of the watershed is set back only 0.5 T_t, not the full T_t. Empirical relations from observed hydrographs show that this setback time may vary widely from 0.5 T_t. However, 0.5 T_t should be used unless there are data to support a specific adjustment.

Effect of storm duration. When D exceeds T_c the resulting hydrograph varies according to the ratio D/T_c, if runoff remains uniform. This effect can be computed as shown below. The whole watershed of figure 3.16-7(a) is used, and Q is assumed to occur uniformly over the watershed and during the storm. ΔD is taken as a whole number, for convenience, but the smaller the ΔD, the better the resulting hydrograph.

Watershed and Storm data

A (sq. mi.)	12.0	
T_c (hours)	3.9	
D (hours)	5.0	Total duration
ΔD (hours)	1.0	
T_p (eq. 16)	2.84	Using ΔD = 1.0
T_b (eq. 19)	7.58	For ΔD = 1.0
Q (inches)	4.2	Total runoff
ΔQ (inches)	0.84	For ΔD = 1.00
q_p (eq. 7 or 11)	1,720	For ΔQ = 0.84; ΔD = 1.0

Figure 3.16-8(a) shows the composite hydrograph and compares it with one obtained using a simple hydrograph--that is, using equation 11 once, with D = 5.0 and Q = 4.2.

Figure 3.16-8(b) shows the composite hydrograph for a storm continuing at the same rate for a total of ten hours. The simple hydrograph for D = 10.0 and Q = 8.4 is also shown.

Effect of a variable storm. The runoff variations due to variations in storm intensity can produce large changes in hydrographs. Figure 3.16-8(c) shows the composite hydrographs for the watershed of figure 3.16-7(a) with the hourly runoff amounts as follows:

Hour	Direct runoff in inches	
	Storm A	Storm B
1	1.60	0.08
2	1.43	.25
3	1.26	.42
4	1.09	.59
5	.92	.76
6	.76	.92
7	.59	1.09
8	.42	1.26
9	.25	1.43
10	.08	1.60
	8.40	8.40

The simple hydrograph, for D = 10 and Q = 8.4, is shown. The effect of breaking the storms into two parts is also shown, with $Q_1 = 6.3$, $Q_2 = 2.1$ for Storm A; and $Q_1 = 2.1$, $Q_2 = 6.3$ for Storm B.

Summary. Good estimates of peak rates for simple hydrographs can be made by equation 11 except when watershed and storm characteristics vary considerably from the average, as the following table shows.

Figure	Peak rate in cfs		% difference
	Composite	Simple	
3.16-6(b)	2560	2700	5
3.16-7(b)	6100	6500	6
3.16-8(a)	5800	5030	- 13
3.16-8(b)	6500	6600	2
3.16-8(c)	9300	6600	- 29
3.16-8(c)	9000	6600	- 27

Figure 3.16-8(c) shows that a two-increment breakdown for an unusual storm improves the relative accuracy without adding much work. In general, a composite hydrograph, even when relatively crude, shows the watershed and storm influences, and the possibility of an unusual case is not overlooked, as with the simple hydrograph. A complete hydrograph should be made by subdividing either the watershed or the storm, or both, as the case may be, into enough parts to show the invluence of an unusual feature. When a watershed is subdivided, a simple hydrograph should be made for each subwatershed for each storm subdivision, disregarding further areal complexities within the subwatershed.

Snowmelt hydrographs. Snowmelt, on watersheds of less than 400 square miles, provides runoff of duration D usually of great length in comparison with T_c. The most important variable in snowmelt hydrographs is the amount of snowmelt each day; this is also the most difficult variable to determine (see Section 3.11).

Once a daily snowmelt is estimated, the distribution of that day's melt may be estimated from figure 3.16-8(d). For example, a watershed with a T_c of 18 hours would have the following distribution of flow for a one-day melt period:

Day	Distribution (Percent)	Remarks
1	51	Day on which melt occurs.
2	46	
3	3	

The procedure for computing the hydrograph for this watershed is shown in table 3.16-3, using the melt data of table 3.11-1. The data of column 3 are plotted as of noon since they represent daily means, and the points are connected by straight lines. The peak rate can be assumed to occur on the day with the highest daily mean, and be very little higher than the mean.

Use of one typical hydrograph for the entire snowmelt season should not be attempted unless the seasonal pattern of snowmelt does not vary much from year to year.

Snowmelt and rainfall hydrographs. Combinations of rain with snowmelt, such as those occurring in New England and in the North Central states, may produce unusual-shaped hydrographs on watersheds less than 400 square miles in size. When the runoff amounts can be estimated for small time intervals, a hydrograph can be made by the composite hydrograph method shown in table 3.16-4.

Design storm hydrographs. A simple triangular hydrograph will be sufficient for many design purposes. When the storm and watershed characteristics need to be well represented, the methods used above for composite hydrographs will give adequate hydrographs.

Table 3.16-3. Sample computation of hydrograph for a snowmelt period.

Watershed T_c = 18 hours
" A = 83 sq. mi.

	April 5	6	7	8	9	10	etc.
Melt in inches: 1/	0	.18	.12	.24	.96	.66	etc.
Melt in cfs days: 2/	0	401	268	535	2140	1470	etc.

Distribution 3/ (percent)	Day	Total rate 4/ (cfs)	Mean daily rate (cfs)						
51	April 5	0	0						
46	6	205	0	205					
3	7	320	0	184	136				
	8	408		12	123	273			
	9	1344			8	246	1090		
	10	1751				16	985	750	
	etc.	etc.					64	676	etc.
								44	etc.

1/ From Table 3.11-1

2/ cfs day = 26.9 AQ = 26.9 (83) Q = 2230 Q

3/ From figure 3.16-8(d) for T_c = 18 hours

4/ This is a mean daily rate of flow

Example. Figure 3.16-9 shows the map and profile of a watershed for which a design hydrograph is required. Two major items must be well represented: (1) the design storm runoff sequence, and (2) the abrupt change in the watershed's topography. Table 3.16-4 shows the required computations, and figure 3.16-9 the composite hydrograph. The simple hydrograph, with A = 6.3, Q = 10.62, and T_p = 6.5, would have a peak of 4980 cfs at 6.5 hours.

On figure 3.16-9 the hydrographs for subwatershed I have been set back 2.0 hours, which is half the travel time from point B to point C.

Table 3.16-4. Sample computation of design storm hydrograph.

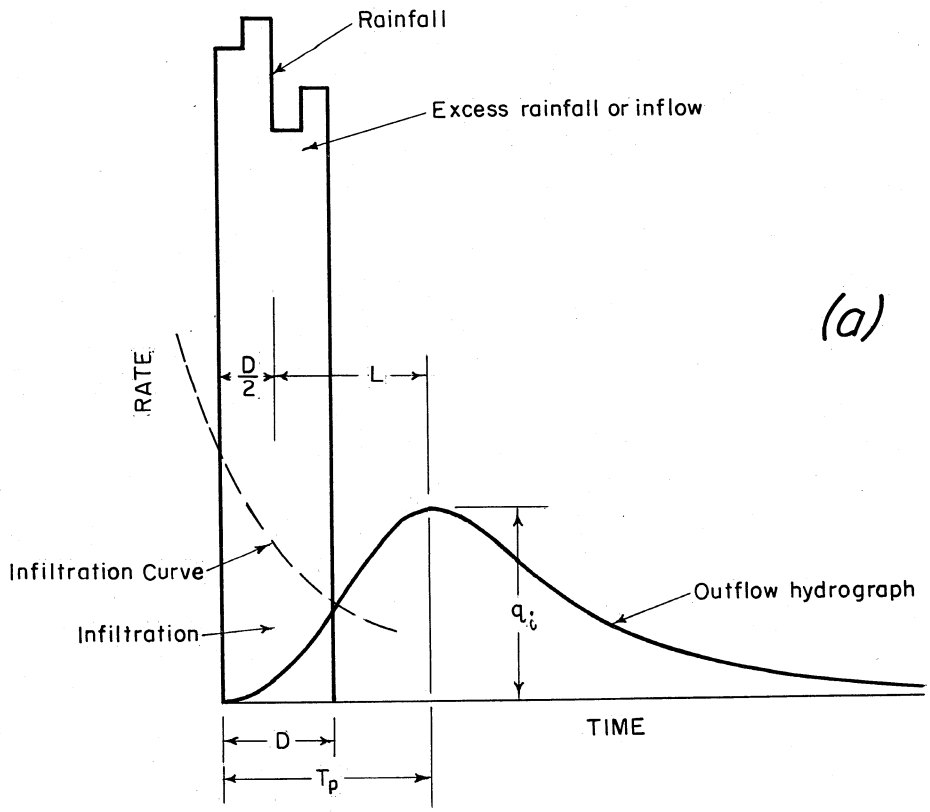
Watershed and storm data

Item	Unit	Entire watershed	Subwatershed I	Subwatershed II
1	A square miles	6.3	3.15	3.15
2	T_c hours	5.8	5.8(A to C)	4.2(D to C)
3	III-curve number	-	92	88
4	P inches	13.3	-	-
5	Adj. P* inches	11.9	11.9	11.9
6	ΔD hours	-	1.0	1.0
7	T_p hours (Eq. 17)	-	4.0	3.0
8	T_b hours (Eq. 20)	-	10.7	8.0
9	q_x cfs	-	381 Q_x	507 Q_x
10	T_t, travel time, hours	-	4.0(B to C)	-
11	Setback time, hours		2.0	

Peak computations

Time	Acc. P	Subwatershed I			Subwatershed II		
		Acc. Q	ΔQ	Peak	Acc. Q	ΔQ	Peak
(hours)	(inches)	(inches)	(inches)	(cfs)	(inches)	(inches)	(cfs)
0	0	0			0		
			0.35	133		0.20	102
1	.95	.35			.20		
			1.60	610		1.40	710
2	2.75	1.95			1.60		
			5.50	2095		5.35	2710
3	8.40	7.45			6.95		
			1.55	590		1.60	812
4	9.95	9.00			8.55		
			1.00	381		1.00	507
5	11.00	10.00			9.55		
			.90	343		.90	457
6	11.90	10.90			10.45		

*Adjusted for area using the "Fletcher" curve of figure 3.21-4.



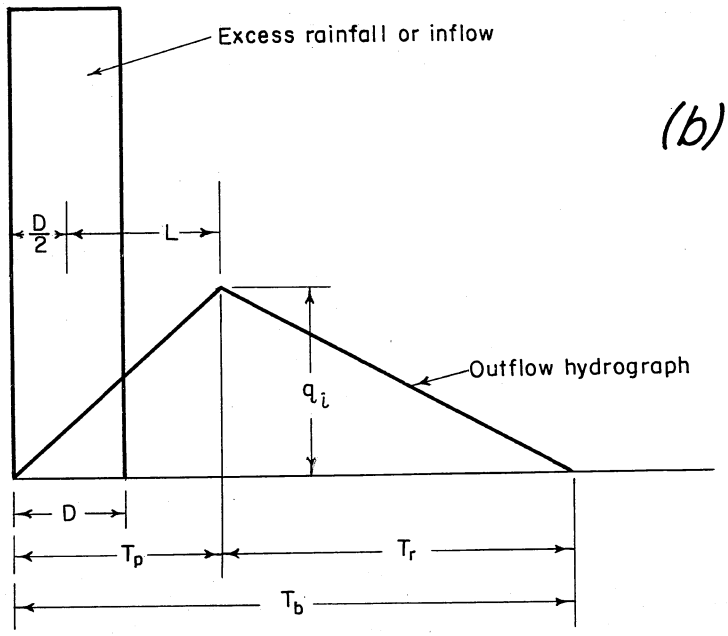


Figure 3.16-1

426963 O - 57 - 14

HYDROLOGY — Peak rates when Q = 1 inch

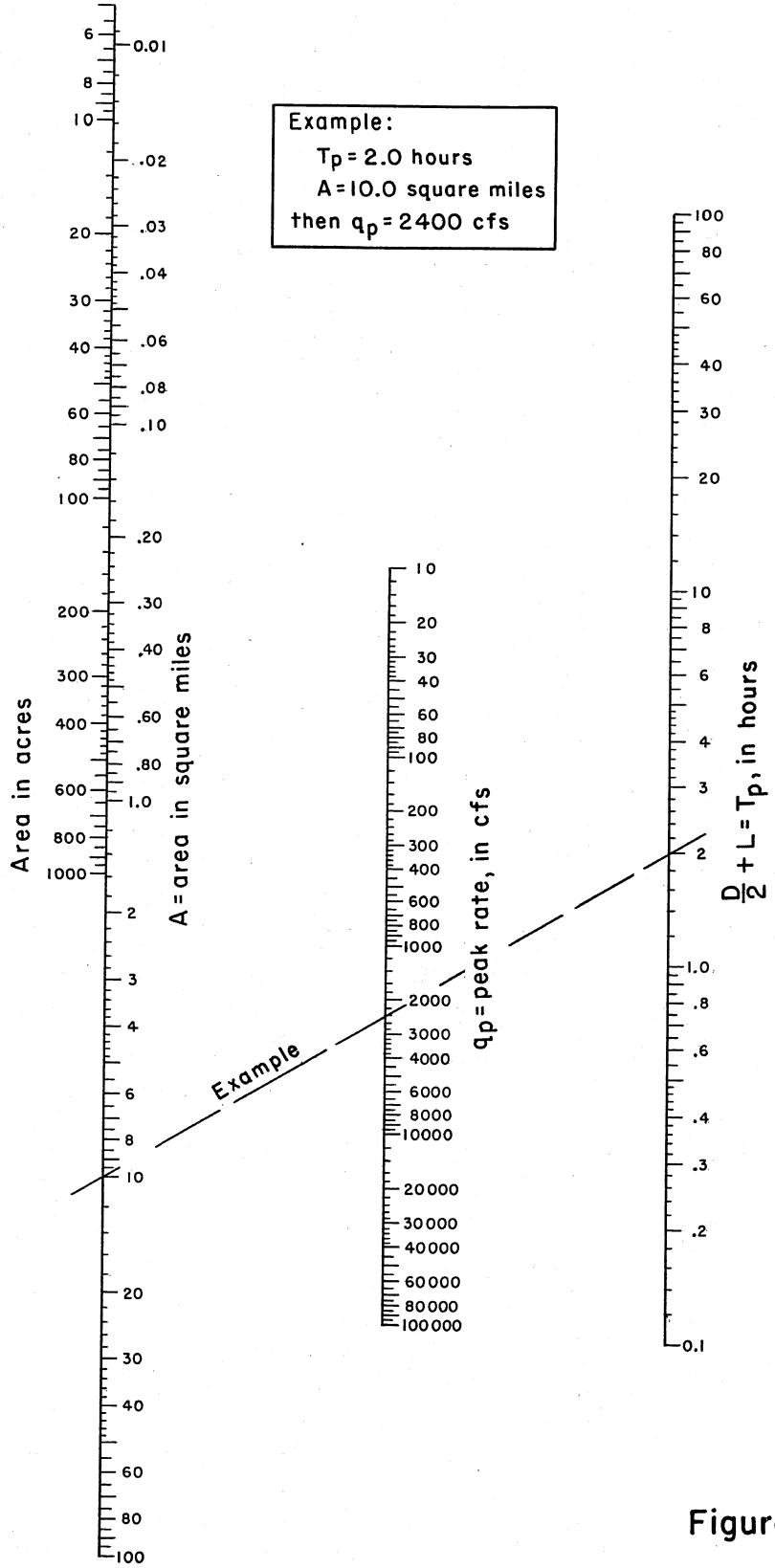


Figure 3.16-2

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
Nomogram developed by Victor Mockus	SOIL CONSERVATION SERVICE	ES-1000
	ENGINEERING DIVISION - CENTRAL TECHNICAL UNIT	SHEET 1 OF 1
		DATE 2-19-57

HYDROLOGY— Dimensionless hydrograph and mass curve.

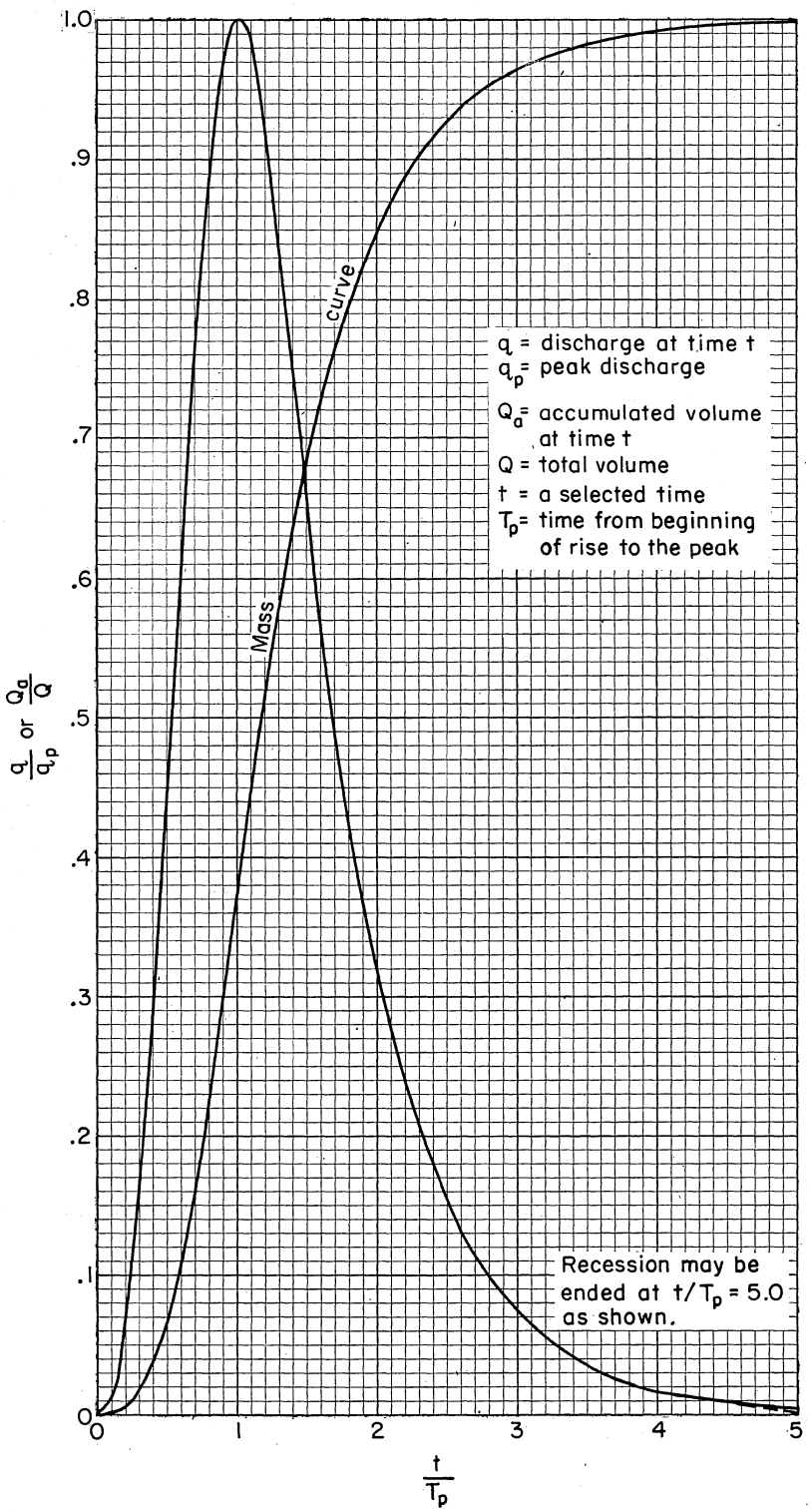


Figure 3.16–3

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
Developed by Victor Mockus.	SOIL CONSERVATION SERVICE	ES-1004
	ENGINEERING DIVISION - CENTRAL TECHNICAL UNIT	SHEET 1 OF 1
		DATE 2-19-57

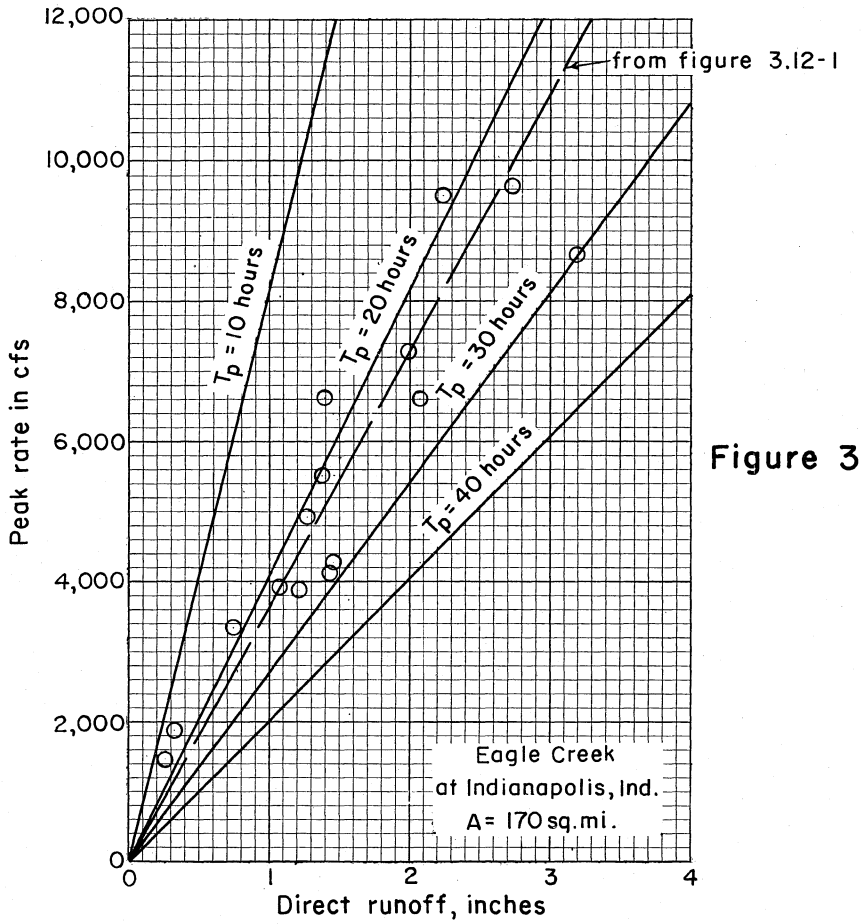


Figure 3.16-4

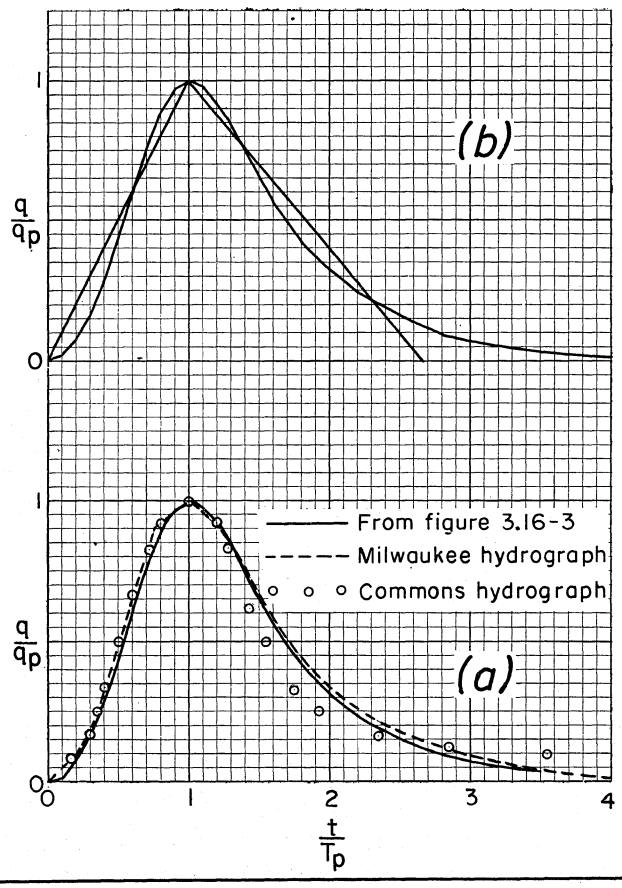


Figure 3.16-5

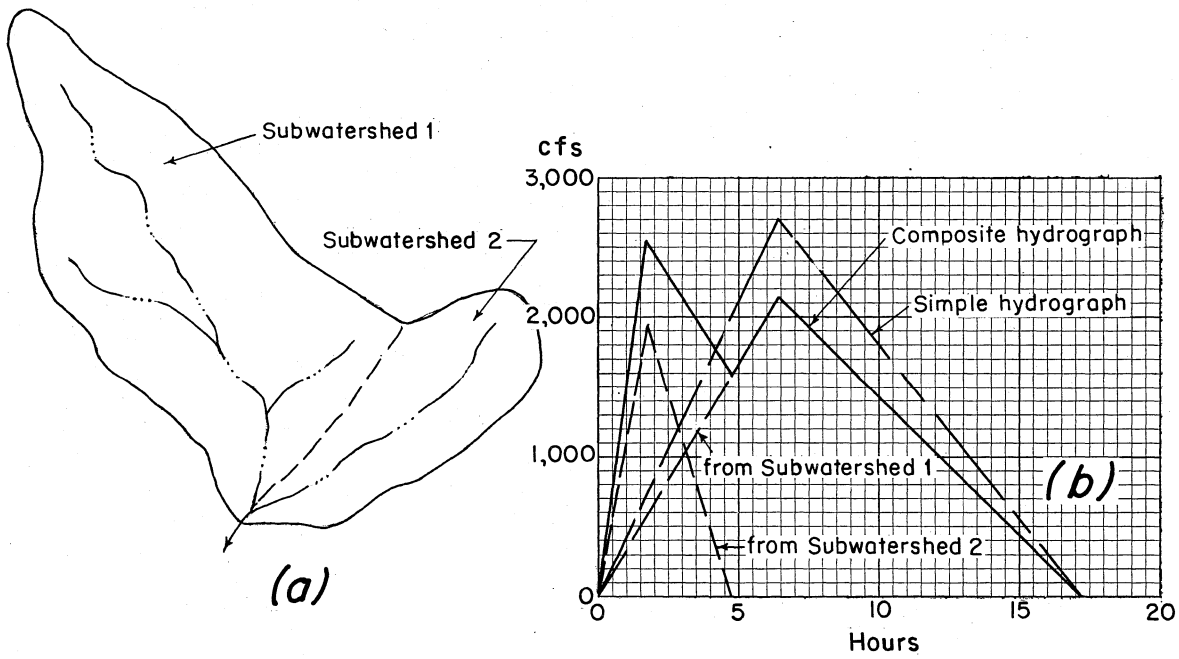


Figure 3.16-6

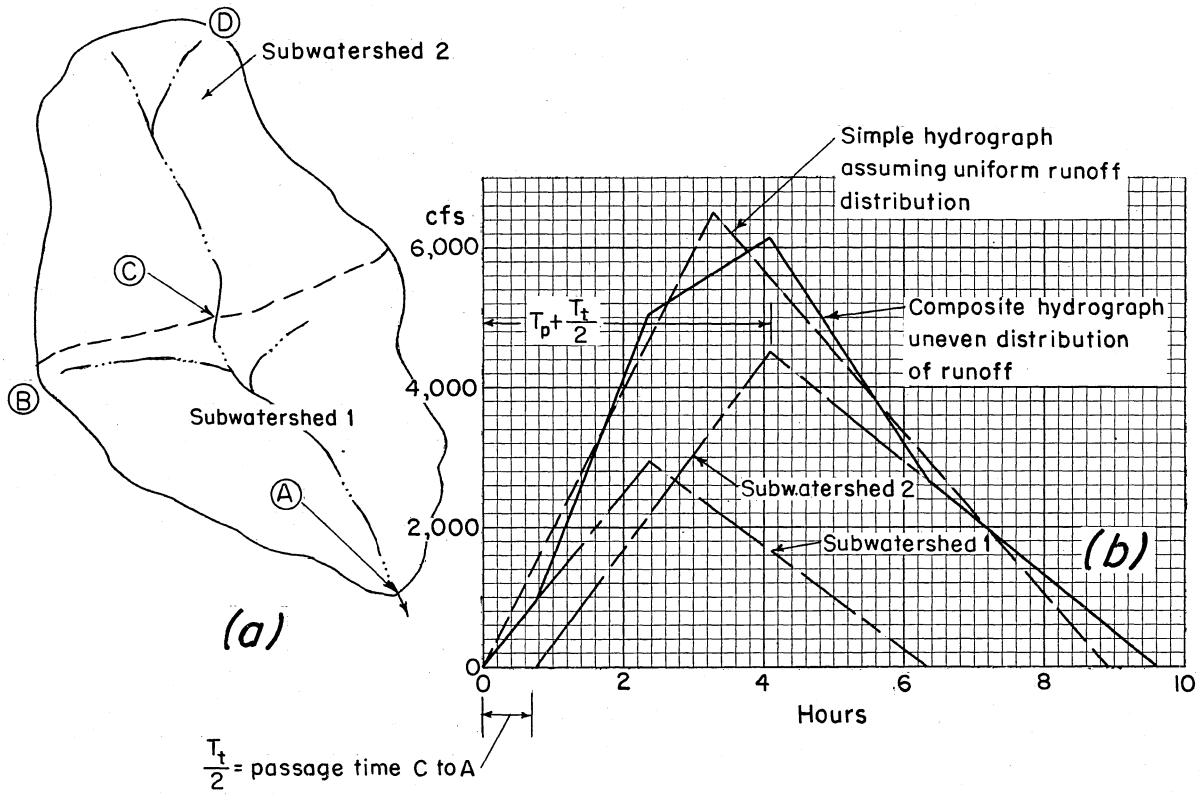


Figure 3.16-7

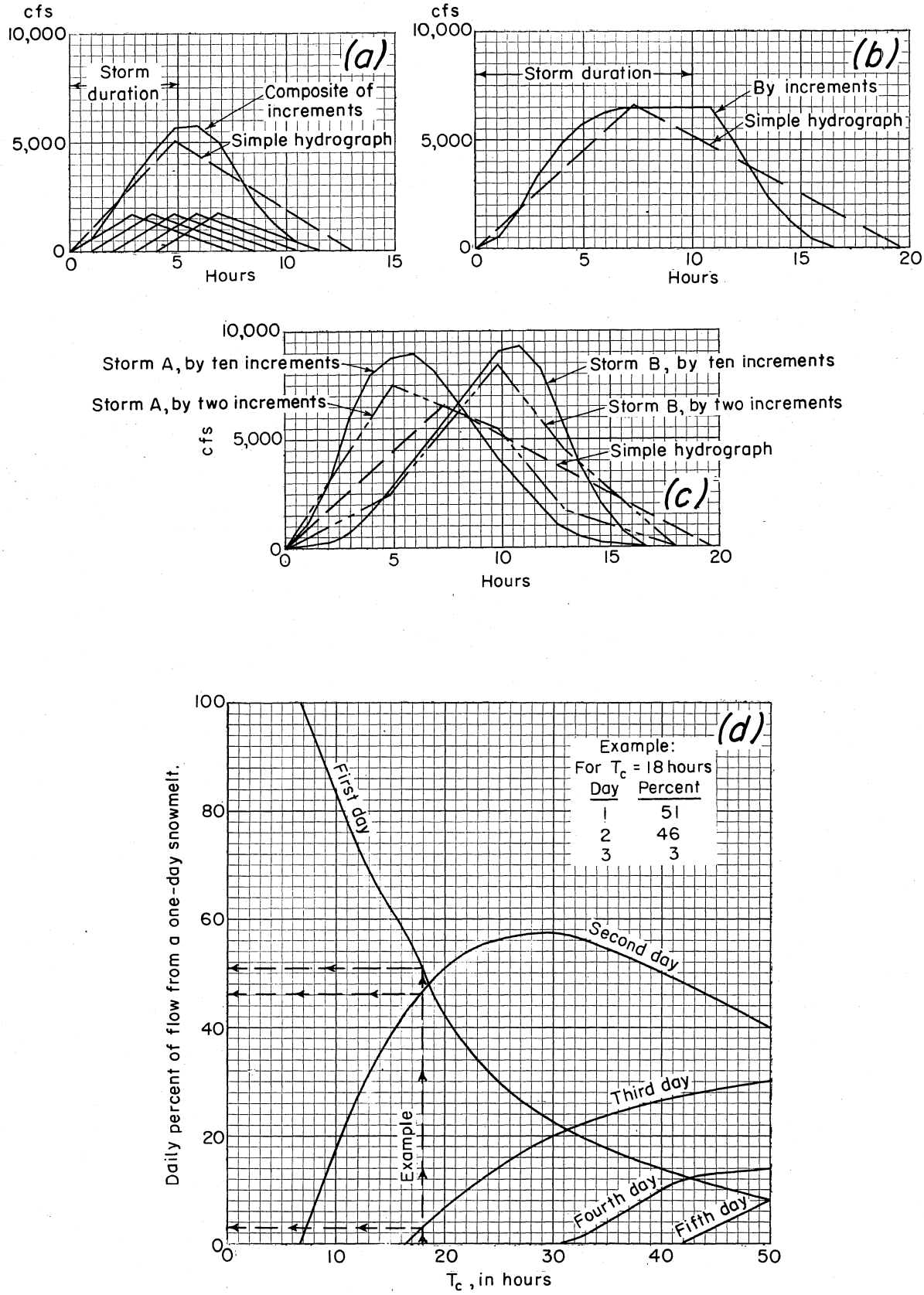

cfs
10,000
5,000
0
(a)
Storm duration
Composite of increments
Simple hydrograph
0
5
10
15
Hours
cfs
10,000
5,000
0
(b)
Storm duration
By increments
Simple hydrograph
0
5
10
15
20
Hours
10,000
5,000
cfs
0
Storm A, by ten increments
Storm A, by two increments
Storm B, by ten increments
Storm B, by two increments
Simple hydrograph
(c)
0
5
10
15
20
Hours
100
80
60
40
20
0
Daily percent of flow from a one-day snowmelt.
(d)
Example:
For T_c = 18 hours
Day Percent
1 51
2 46
3 3
First day
Second day
Third day
Fourth day
Fifth day
Example
0
10
20
30
40
50
T_c, in hours

Figure 3.16-8

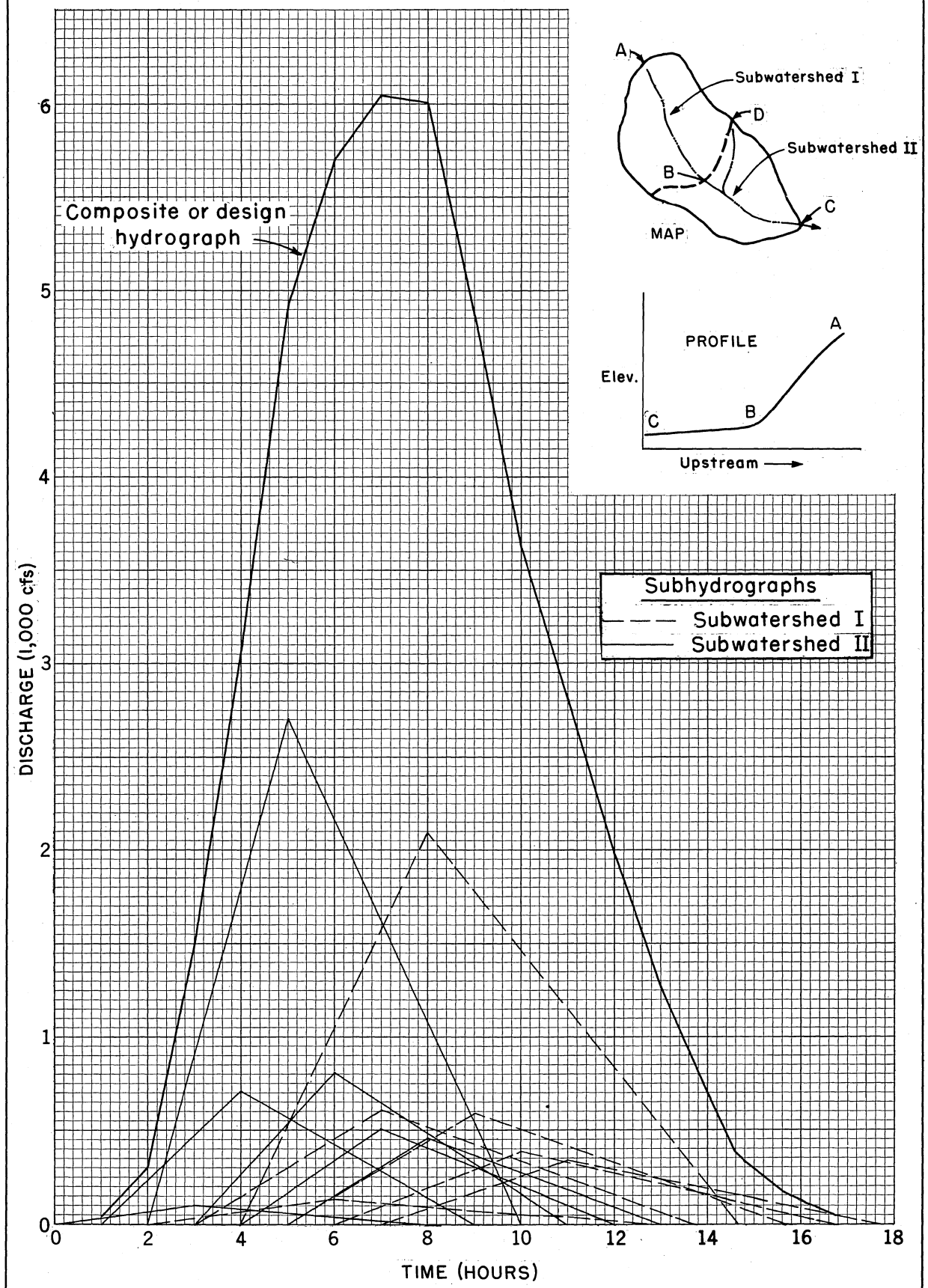

Composite or design hydrograph
DISCHARGE (1,000 cfs)
TIME (HOURS)
0
1
2
3
4
5
6
0
2
4
6
8
10
12
14
16
18
Subhydrographs
Subwatershed I
Subwatershed II
A
B
C
D
Subwatershed I
Subwatershed II
MAP
PROFILE
Elev.
Upstream

Figure 3.16-9

3.17 Flood routing

Purpose. This is an extension of the methods given in Section 3.16. The common use of flood routing is to determine the changes made in a flood hydrograph by the flood's passage through a stream reach or a reservoir.

Types. The storage type of routing is based on some expanded form of equation 1 (given below). Three methods of this type are included in this section: Storage-indication method, Goodrich-Wisler method, and the Wilson method. The first, or storage-indication method is recommended for general usage.

The methods of hydrograph development described in Section 3.16, and similar ones such as the concordant flow method, are sometimes considered types of routings. These are not ordinarily used for flood routing through reservoirs or through stream channels with storage characteristics that vary widely.

Accuracy of each type depends on factors other than the mathematics and physics of routing. For example, errors in estimates of upstream and local runoff cannot be reduced by improving the routing technique. The choice of a type of routing method, therefore, depends on the relative accuracy of available data, as well as the purpose for which the routing is made.

Basic equation for storage type of routing. Almost all storage methods are based directly on the continuity equation

$$I - O = \Delta S \qquad (1)$$

where I = volume of inflow for a given time interval
O = volume of outflow for that time interval
ΔS = change in volume of storage for that time interval

The working equation is generally identical with or a modification of:

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

where Δt = time increment ($\Delta t = t_2 - t_1$)
I_1 = inflow at time 1
I_2 = inflow at time 2
O_1 = outflow at time 1
O_2 = outflow at time 2
S_1 = storage at time 1
S_2 = storage at time 2

The time is usually in hours, inflow and outflow in cfs, and storage in cfs hours.

Equation 2 can be used for either streamflow or reservoir routing. For streamflow routing, it is important that: (1) Short reaches be used; and (2) the routing interval Δt be no larger than the travel time of flow through the reach, and that the same Δt value be used in constructing the working curves for routing.

For reservoir routing the first item does not apply, and the routing interval Δt should not be larger than the travel time determined by the slope of the storage vs. discharge curve. The same Δt value should be used in constructing the working curves for routing.

Storage-indication method

Summary. This is also known as the modified Puls method. Equation 2 is rewritten:

$$I_{av} + \left[\frac{S_1}{\Delta t} - \frac{O_1}{2}\right] = \left[\frac{S_2}{\Delta t} + \frac{O_2}{2}\right] \qquad (3)$$

where $I_{av} = (I_1 + I_2)/2$.

For each reach of a stream, or for the reservoir:

1. The hydraulic and storage data are used to prepare the storage-indication working curve, which is used in the solution of equation 2;

2. Inflow hydrographs are obtained (Section 3.16);

3. A routing table is prepared; and

4. The hydrographs of item 2 are routed.

Stream reach example. Table 3.17-1 shows the data and computations for a storage-indication curve for a 10,000-foot stream reach in which four cross sections have been taken. The following are needed before table 3.17-1 can be prepared:

1. Stage-discharge curves for each cross section.

2. Stage versus cross-sectional area curves for each cross section.

3. Distances between cross sections.

The steps in preparing a working curve are:

1. Make a work sheet with headings like those on table 3.17-1.

2. Select a series of discharges, from nearly zero to a discharge slightly greater than is expected to be routed, and tabulate these discharges. (Column 1 of Table 3.17-1)

3. For each cross section, enter the stage-discharge curve with a given discharge and read stage. Enter stage versus end-area curve with the stage and read area in square feet. Tabulate as shown in Columns 2, 3, 4, and 5.

4. Get total length of reach and compute weights for sub-reaches between cross sections. For example:

From cross section	To cross section	Length (ft.)	Weighted value
1	2	1000	0.1
2	3	6000	.6
3	4	3000	.3
	Sum:	10,000	

Tabulate weights in boxes of columns 6, 7, and 8 as shown.

5. Compute weighted end-areas of columns 6, 7, and 8. For example, at 50 cfs, cross section 1 has 40 square feet and cross section 2 has 27 square feet. $\frac{40 + 27}{2} = 34$, which when multiplied by the weight of subreach 1-2 (weight = 0.1) gives 3.4. This is rounded off to 3. See column 6.

6. For a given discharge, sum up the weighted end-areas and write sum in column 9. (Note: A calculating machine can be used to combine steps 5 and 6 into a simple operation of accumulative multiplication. Instead of dividing by 2 in Step 5, use one-half of each of the weights of columns 6, 7, and 8.)

7. Compute storages (S) in <u>cfs hours</u>, column 10:

$$S_x = \frac{L\ A_x}{3600} \qquad (4)$$

where L = total length of reach in feet; A_x = average end-area in square feet at q_x; and S_x = storage at the discharge q_x.

At q_x = 50 cfs, for example, we get:

$$S_{50} = \frac{(10{,}000)\ (25)}{3600} = 70 \text{ cfs hours.}$$

Table 3.17-1. Computation of storage-indication working curve.

Column: 1	2	3	4	5	6 1/	7 1/	8 1/	9	10	11 2/	12	13 2/
Outflow (O)	End-Areas				Weighted end-areas			Average end-area	S	$\frac{S}{\Delta t}$	$\frac{O}{2}$	$\frac{S}{\Delta t} + \frac{O}{2}$
	Cross Sec. 1	Cross Sec. 2	Cross Sec. 3	Cross Sec. 4	Weight 0.1	Weight 0.6	Weight 0.3					
(cfs)	(sq.ft.)	(sq.ft.)	(sq.ft.)	(sq.ft.)	(sq.ft.)	(sq.ft.)	(sq.ft.)	(sq.ft.)	(cfs.hrs.)	(cfs)	(cfs)	(cfs)
0	0	0	0	0	0	0	0	0	0	0	0	0
50	40	27	21	33	3	14	8	25	70	93	25	118
150	90	44	44	64	7	26	16	49	164	219	75	294
300	150	83	83	100	12	50	27	89	248	331	150	481
800 3/	470	180	220	325	32	120	82	234	651	869	400	1269
1500	950	310	460	700	63	231	174	468	1302	1740	750	2490
3500	2500	640	1200	2000	157	552	480	1189	3300	4400	1750	6150
5000	3250	860	1700	2700	205	768	660	1633	4540	6060	2500	8560
7000	4400	1050	2050	3400	272	930	819	2021	5620	7500	3500	11000
10000	5800	1300	2550	4500	355	1155	1055	2565	7130	9520	5000	14520

(Note: These are slide-rule computations).

1/ Weights used in this column computed in step 4.

2/ Δt used in this column computed in steps 8 and 9.

3/ Approximate bankfull discharge.

8. Determine travel time (T_t, in hours) through the entire reach for an approximate average bankfull discharge.

$$T_t = \frac{LA}{3600\ q_b} \qquad (5)$$

where L = total length of reach in feet

A = average end-area in square feet for q_b (from column 9)

q_b = the bankfull discharge in cfs

For this example, q_b is 800 cfs, and

$$T_t = \frac{(10000)}{(3600)} \frac{(234)}{(800)} = 0.813 \text{ hr.}$$

9. Round off T_t to a lower convenient clock interval (5, 10, 15, etc. minutes; 1, 1.5, 2, etc. hours). In this case, 0.813 is rounded off to 0.75 hour (45 minutes). The rounded figure is Δt, the routing interval, in hours.

10. Compute $S/\Delta t$ as shown in column 11.

11. Compute O/2 as shown in column 12.

12. Compute $(S/\Delta t) + (O/2)$ from columns 11 and 12, and tabulate in column 13.

13. Prepare graph sheet for working curve. The graph may have either arithmetic or log scales. The log paper is preferred, since it gives comparable accuracy throughout the range of work, and is easier to use (See figure 3.17-1.)

14. Plot the working curve, using discharges in column 1 as ordinates and corresponding values in column 13 as abscissas (See figure 3.17-1.)

15. Connect the plotted points by a smooth curve. In cases where the discharge and storage data are more roughly estimated, a straight line with a slope of 1 may be drawn through the plotted points. Such a straight line can also be used when making a shortcut of this method. (See below "Storage-Indication method - Shortcuts")

The working curve is then ready for use. A routing is performed as follows:

1. Prepare a working table with headings like those of table 3.17-2.

2. In column 1, tabulate times in units of Δt.

3. In column 2, tabulate average inflow (I_{av}) for the Δt intervals, for the hydrograph to be routed. (See the triangle hydrograph on figure 3.17-2 (a) for the source of the values shown.)

4. Route the hydrograph by following the series of operations shown below (also see table 3.17-2).

Time	I_{av}	$\frac{S}{\Delta t} + \frac{O}{2}$	O	Remarks
0	0	0	0	Given
.75	750			Given
		750		0 - 0 + 750 = 750
			490	From fig. 3.17-1
1.50	2250			Given
		2510		750 - 490 + 2250 = 2510
			1530	From fig. 3.17-1
2.25	3750			Given
		4730		2510 - 1530 + 3750 = 4730
			2650	From fig. 3.17-1
etc.	etc.	etc.	etc.	etc.

The series of operations is a series of solutions of equation 3. After each time write down a value of $(S/\Delta t) + (O/2)$, enter the working curve with that value, and read the corresponding outflow. For example, with $(S/\Delta t) + (O/2)$ of 2510, the curve of figure 3.17-1 gives an outflow of about 1530 cfs.

5. After the routing is carried as far as required (usually it is stopped at some proportionately small discharge) the outflow hydrograph is plotted.

Comments. Computations in table 3.17-2 indicate that outflow continues indefinitely, and it is necessary to stop the routing at a convenient low discharge.

Table 3.17-2. Working table for stream reach routing by storage-indication method.

Time (hrs)	I_{av} (cfs)	$\frac{S}{\Delta t} + \frac{O}{2}$ (cfs)	O (cfs)
0	0	0	0
.75	750	750	490
1.50	2250	2510	1510
2.25	3750	4750	2670
3.00	5250	7330	4150
3.75	5550	8730	5050
4.50	4650	8330	4750
5.25	3750	7330	4150
6.00	2850	6030	3500
6.75	1950	4480	2580
7.50	1050	2950	1770
8.25	300	1480	900
9.00	0	580	370
9.75	0	210	90
10.50	0	120	50
etc.	etc.	etc.	etc.
Check:	32100		32030

After inflow stops, I_{av} values are zero, and the routing operation becomes simpler. For example, at time 9.00, outflow is 300 cfs; at 9.75 the $(S/\Delta t) + (O/2)$ is 480 - 300 + 0 = 180.

The outflow discharges are at the <u>ends of the time intervals</u>.

The routing alters a hydrograph that enters at the head of the reach into the hydrograph that leaves at the foot of the reach, except for local inflow (see below).

The outflow of a reach is the inflow of a downstream reach, except for local inflow (see below).

If the routing has been carried to a low-outflow discharge, a check on the operation can be made by summing the inflows (column 2) and the outflows (column 4) and comparing. The outflows will seldom equal, and generally be less than the inflows, so that an exact check will not be obtained.

Another check is shown on figure 3.17-2. The peak outflow occurs when inflow and outflow are equal.

<u>Routing upstream</u>. A hydrograph can be routed <u>upstream</u> by starting with the hydrograph at the foot of the reach and reversing the routing operation in its entirety. Using the data of table 3.17-2, for example, would give:

Time	O	$\frac{S}{\Delta t} + \frac{O}{2}$	I_{av}	Remarks
9.00	370	580 1/	0	Given
8.25	900			Given
		1480		From fig. 3.17-1
			0	580+900-1480 = 0
7.50	1770			Given
		2950		From fig. 3.17-1
			300	1480+1770-2950 = 300
6.75	2580			Given
		4480		From fig. 3.17-1
			1050	2950+2580-4480 = 1050
etc.	etc.	etc.	etc.	etc.

1/ When starting well up on the outflow hydrograph, as in this example, this value may be difficult to estimate. The error of the estimate is reduced by starting as low on the outflow hydrograph as possible, preferably where the outflow is zero.

Local inflow. The flow that enters the reach at places between the head and foot of the reach is local inflow, as shown in figure 3.17-2 (b).

When local inflow is large enough to be important, a hydrograph is made for it (Section 3.16). The hydrograph is added either to the inflow of the reach before routing, or to the outflow of the reach after routing. On figure 3.17-2 (b), if tributary A is large, its hydrograph is added to the inflow at the head of the reach before routing. If tributary B is large, its hydrograph is added to the routed outflow leaving the reach. The hydrograph of tributary C, if large, would be added to the routed outflow hydrograph; the hydrograph of tributary D, if large, would be added to the inflow at the head of the reach before routing. The hydrograph of local inflow for the area outside of important tributaries is added to the routed outflow. When none of the individual tributaries is important, one hydrograph for local inflow is made, and added to the routed outflow leaving the reach.

Watershed example. Figure 3.17-3 shows the routing of hydrographs through a watershed stream system. The flood occurs from uniform runoff conditions. Table 3.17-3 shows the details of the routing.

The hydrographs of local inflow were added after routing. In every case, the total outflow from a reach was plotted to obtain I_{av} values for inflow into the next reach.

Reservoir example. The process is similar for reservoir routing. Table 3.17-4 gives data and computations for a working curve. Here the Δt in hours is usually best obtained by examination of the inflow hydrograph; Δt should always be small enough to define the inflow hydrograph with reasonable accuracy. A Δt of 0.2 hour was selected for the inflow hydrograph given in column 2 of table 3.17-5.

The storage-indication curve may be plotted in either of two ways as in figure 3.17-4. The method of figure 3.17-4 (a) is generally preferable for reservoirs.

The routing given in table 3.17-5 starts with the pool full to the emergency spillway crest. It can be started at any other level by altering the given values at zero time.

426963 O - 57 - 15

Table 3.17-3. Sample computation. Flood routing by the storage-indication method.

Damage Reach C, Δt = 1.5 hr.

1	2	3	4	5	6
Acc. hrs.	I_{av} I* cfs	$\frac{S}{\Delta t} + \frac{O}{2}$ cfs	O cfs	I_ℓ II* cfs	O_t cfs
0	0	0	0	0	0
1.5	100	100	10	190	200
3.0	295	385	80	380	460
4.5	490	795	190	570	760
6.0	685	1290	380	760	1140
7.5	880	1790	550	950	1500
9.0	1050	2290	750	1130	1880
10.5	1245	2785	950	1120	2070
12.0	1400	3235	1150	1010	2160
13.5	1300	3385	1210	895	2105
15.0	1185	3360	1200	785	1985
16.5	1070	3230	1150	670	1820
18.0	955	3035	1020	560	1580
19.5	840	2855	995	450	1445
21.0	720	2580	870	335	1205
22.5	605	2315	760	225	985
24.0	495	2050	650	110	760
25.5	380	1780	540	0	540
27.0	265	1505	440	0	440
28.5	150	1215	320	0	320
30.0	35	930	230	0	230
31.5	0	700	160	0	160
33.0	0	540	110	0	110
34.5	0	430	80	0	80
36.0	0	350	60	0	60
37.5	0	290	50	0	50
39.0	0	240	40	0	40
40.5	0	200	30	0	30
42.0	0	170	20	0	20
43.5	0	150	10	0	10
45.0	etc.	140	etc.	etc.	etc.
etc.		etc.			

I_{av} = Average inflow at head of reach.
I = Local inflow at end of Δt.
O_t = Total outflow at end of Δt.

*Subarea numbers.

Damage Reach B, Δt = 1 hr.

7	8	9	10	11	12
Acc. hrs.	I_{av} cfs	$\frac{S}{\Delta t} + \frac{O}{2}$ cfs	O cfs	I_ℓ III* cfs	O_t cfs
0	0	0	0	0	0
1	60	60	30	125	155
2	200	230	90	285	375
3	365	505	200	370	570
4	550	855	320	495	815
5	760	1295	490	620	1110
6	1000	1805	670	745	1415
7	1250	2385	850	870	1720
8	1500	3035	1080	995	2075
9	1750	3705	1320	1120	2440
10	1940	4225	1520	1095	2615
11	2070	4775	1740	1020	2760
12	2145	5180	1930	940	2870
13	2145	5395	2020	865	2885
14	2100	5475	2050	790	2840
15	2030	5455	2040	715	2755
16	1940	5355	2000	635	2635
17	1820	5175	1925	560	2485
18	1690	4940	1820	485	2305
19	1560	4680	1700	410	2110
20	1430	4310	1600	335	1935
21	1285	4095	1475	255	1730
22	1140	3760	1340	180	1520
23	985	3405	1210	105	1315
24	830	3025	1080	30	1110
25	690	2635	940	0	940
26	580	2275	830	0	830
27	480	1925	710	0	710
28	400	1615	605	0	605
29	325	1335	500	0	500
30	270	1105	425	0	425
31	210	890	340	0	340
32	160	710	280	0	280
33	115	545	220	0	220
34	95	420	165	0	165
35	70	325	130	0	130
36	65	260	105	0	105
37	55	210	85	0	85
38	50	175	75	0	75
39	45	145	70	0	70
40	40	115	60	0	60
41	30	85	40	0	40
42	25	70	30	0	30
etc.	etc.	etc.	etc.	etc.	etc.

Damage Reach E, Δt = 3 hr.

13	14	15	16	17	18
Acc. hrs.	I_{av} IV* cfs	$\frac{S}{\Delta t} + \frac{O}{2}$ cfs	O cfs	I_ℓ V* cfs	O_t cfs
0	0	0	0	0	0
3	200	200	150	385	535
6	620	670	400	785	1185
9	1035	1305	760	1185	1945
12	1425	1970	1160	1555	2715
15	1420	2230	1360	1115	2475
18	1180	2050	1250	875	2125
21	930	1730	1020	630	1650
24	690	1400	810	390	1200
27	445	1035	610	150	760
30	200	625	390	0	390
33	0	235	160	0	160
36	0	75	50	0	40
39	0	35	25	0	25
42	etc.	10	etc.	etc.	etc.
etc.		etc.			

Damage Reach D, Δt = 2 hr.

19	20	21	22	23	24
Acc. hrs.	I_{av} cfs	$\frac{S}{\Delta t} + \frac{O}{2}$ cfs	O cfs	I_ℓ VI* cfs	O_t cfs
0	0	0	0	0	0
2	170	170	140	200	340
4	535	565	400	410	810
6	955	1120	750	620	1510
8	1430	1800	1250	455	1705
10	1945	2495	1690	360	2050
12	2480	3285	2100	225	2325
14	2650	3835	2360	190	2550
16	2485	3960	2410	0	2410
18	2270	3820	2350	0	2350
20	1985	3455	2160	0	2160
22	1670	2965	1950	0	1950
24	1360	2375	1610	0	1610
26	1040	1805	1250	0	1250
28	760	1315	910	0	910
30	490	895	600	0	600
32	300	595	420	0	420
34	160	335	255	0	255
36	70	150	120	0	120
38	35	65	50	0	50
40	25	40	20	0	20
42	5	25	10	0	10
etc.	etc.	etc.	etc.	etc.	etc.

Damage Reach D, Δt = 2 hr.

25	26	27	28	29	30
Acc. hrs.	I_{av} cfs	$\frac{S}{\Delta t} + \frac{O}{2}$ cfs	O cfs	I_ℓ VII* cfs	O_t cfs
0	0	0	0	0	0
1	150	150	140	120	260
2	500	510	450	255	705
3	900	960	780	385	1165
4	1350	1530	1130	520	1650
5	1850	2250	1540	650	2190
6	2370	3080	1920	785	2705
7	2980	4140	2340	925	3265
8	3520	5320	2780	1055	3835
9	3990	6530	2900	1185	4085
10	4430	8060	3100	1320	4420
11	4830	9790	3500	1430	4930
12	5105	11395	4000	1355	5335
13	5275	12670	4500	1275	5775
14	5365	13535	4850	1195	6045
15	5345	14030	5080	1115	6195
16	5205	14155	5140	1035	6175
17	5020	14035	5080	995	6075
18	4785	13740	4950	875	5825
19	4510	13300	4750	795	5545
20	4225	12775	4550	715	5265
21	3910	12135	4280	620	4900
22	3585	11440	4010	550	4560
23	3265	10695	3760	475	4235
24	2930	9865	3510	390	3900
25	2600	8955	3300	310	3610
26	2250	7905	3090	230	3320
27	1910	6725	2950	150	3100
28	1600	5375	2690	70	2760
29	1330	4015	2300	0	2300
30	1100	2815	1810	0	1810
31	905	1910	1370	0	1370
32	735	1275	980	0	980
33	590	885	700	0	700
34	465	650	550	0	550
35	365	465	400	0	400
36	275	340	310	0	310
37	210	240	220	0	220
38	150	170	140	0	140
39	130	160	120	0	120
40	80	120	100	0	100
41	40	60	40	0	40
42	20	40	20	0	20
etc.	etc.	etc.	etc.	etc.	etc.

3.17-10

Table 3.17-4 Computation of storage - indication working curve reservoir example.

Note: $\Delta t = 0.2$ hour

Elevation (ft.)	Storage (AF)	Storage (cfs hours)	Spillway discharge Principal (cfs)	Spillway discharge Emergency (cfs)	Spillway discharge Total (cfs)	$\frac{S}{\Delta t} + \frac{O}{2}$ (cfs)
61.6	0	0	0		0	0
62	3.0	36	3		3	182
64	24.5	296	16		16	1488
66	50.1	607				
68	80.6	975	20		20	4885
70	111.6	1350				
72	162.0	1960	22		22	9811
74	220.8	2670				
76	267.0	3230	24		24	16162
77.4	308.0	3730	25	0	25	18662
78	327.0	3960	26	154	180	19890
79	358.0	4330	26	570	596	21948
80	394.0	4760	26	1220	1246	24423
81	430.0	5200	26	1960	1986	26993
82	469.0	5670	27	2915	2942	29821

Table 3.17-5a. Routing table, reservoir example.

(Reservoir full to emergency spillway crest.)

Time (hrs)	I_{av} (cfs)	$\frac{S}{\Delta t} + \frac{O}{2}$ (cfs)	O (cfs)
0	0	18660 1/	25 2/
.2	60	18695	30
.4	370	19035	60
.6	980	19955	190
.8	1740	21505	490
1.0	2310	23325	940
1.2	2470	24855	1370
1.4	2200	25685	1600
1.6	1780	75865	1660
1.8	1370	25575	1570
2.0	1050	25055	1420
2.2	800	24435	1250
2.4	600	23785	1060
2.6	460	23185	940
2.8	340	22585	740
3.0	260	22105	630
etc.	etc.	etc.	etc.

1/ From table 3.17-4, at elevation of emergency spillway crest. Rounded to nearest 10.

2/ From table 3.17-4 at elevation 77.4. When this discharge is small it may be neglected.

Table 3.17-5(b). Routing table, reservoir example.

(Reservoir full to elevation 72)

Time	Original hydrograph I_{av}	Original hydrograph Acc. I_{av}	Revised hydrograph Acc. I_{av}	$\frac{S}{\Delta t} + \frac{O}{2}$	O
etc.	etc.	etc.	etc.	-	(25) 2/
1.0	2310	5460	0 1/	-	(25) 2/
1.2	2470	7930	0 1/	18660 3/	(25) 2/
1.4	2200	10130	1281 4/	19916	180
1.6	1780	-	1780	21516	530
1.8	1370	-	1370	22356	700
2.0	1050	-	1050	22706	780
2.2	800	-	800	22726	790
2.4	600	-	600	22536	730
etc.	etc.	-	etc.	etc.	etc.

1/ Inflow still filling the storage between elevation 72 and the emergency spillway crest.

2/ Approximate principal spillway discharge.

3/ From figure 3.17-4, at elevation of emergency spillway crest.

4/ Accumulation of inflow hydrograph to this time is 10130, which is more than the difference in $(S/\Delta t) + (O/2)$ between elevation 72 and the elevation of the emergency spillway crest--in this case 8849. Then:

$$10130 - 8849 = 1281$$

The succeeding inflow values remain as before.

In this case, it is very tedious to route using $\Delta t = 0.2$ hour when only the principal spillway is operating, and no significant accuracy is lost if the first part of the routing is roughly approximated. Suppose, for example, that the pool is full to elevation 72 when the routing starts. The $(S/\Delta t + (O/2)$ at that elevation is 9811 cfs, while at the elevation where significant outflow begins it is 18660 cfs. Then 18660 - 9811 = 8849, which is how far the inflow hydrograph must be accumulated before flow through the emergency spillway begins. The revised inflow hydrograph is prepared and routed as shown on table 3.17-5(b).

The use of the $(S/\Delta t) + (O/2)$ values in this manner is satisfactory only when (O/2) is negligible. The more general case is handled as follows, using the same example. Storage at elevation 72 (S_1) is 1960 cfs hours (Col. 3, table 3.17-4); at elevation 77.4 (S_2) it is 3730 cfs hours; and Δt is 0.2 hour.

Then:

$$\frac{S_2 - S_1}{\Delta t} = \frac{3730 - 1960}{0.2} = 8850 \text{ cfs}$$

which is used as before.

<u>Shortcuts</u>. When a shortcut procedure is needed, it is better to shortcut individual operations of this method rather than to invent another method that calls for memorizing a whole new set of symbols and operations. Table 3.17-6 suggests some ways by which the storage-indication method can be shortcut. Under many conditions the hydrologic methods of routing given below can be used.

A shortcut should be used only when there is some justification for it, as discussed in Section 1.3.

Table 3.17-6 Storage-indication method, shortcuts

Item	For a shortcut:
1. Selection of reaches	1. Use longer reaches.
2. Computation of cross-sectional areas.	2. Use one cross section that most represents all in the reach.
3. Preparation of storage-indication curve.	3a. Use fewer steps in discharge (Col. 1 of table 3.17-1).
	3b. Use as large a Δt as possible (but no greater than the travel time through the reach).
	3c. Compute storage and discharge only at break-points in cross sections. Plot storage-indication values on log paper and connect with straight lines.
4. Hydrographs	4. Use triangle hydrographs for inflows.

Goodrich-Wisler method.

A routing method popular with some engineers is a modified form of a method described by R. D. Goodrich in Trans. ASCE, Vol. 104, 1939; Discussion of "Flood Routing" by Rutter, Graves and Snyder. The method differs from the storage-indication method only in the plotting and use of the working curves.

The computations up through column 10 of table 3.17-1 are required, and they are completed as shown in table 3.17-7 to get the working curves. Figure 3.17-5 shows the working curves, which must be plotted on arithmetic graph paper.

The hydrograph used in ~~table 3.17-2~~ FIGURE 3.17-5 is used to show the routing operations in table 3.17-8. After the $(I_1 + I_2)$ column is computed, this column is used, together with the working curves (figure 3.17-7a) and an engineer's scale, to route the flood. The routing sequence is as follows:

Time	$I_1 + I_2$	O	Remarks
0	0	0	Start of routing
.75	1500		Place zero of engr. scale on $(2S/\Delta t) - O$ curve where O is zero.
.75		480	At 1500 on engr. scale read 480 on $(2S/\Delta t)+O$ curve.
1.50	4500		Place zero of engr. scale on $(2S/\Delta t)-O$ curve where O = 480.
1.50		1550	At 4500 on engr. scale read 1550 on $(2S/\Delta t) + O$ curve.
2.25	7500		Place zero of engr. scale on $(2S/\Delta t) - O$ curve where O = 1550.
etc.	etc.	etc.	etc.

The routing operation for the falling side of the hydrograph is very rapid, since there are no $(I_1 + I_2)$ values to use and the engineer's scale is not needed.

Table 3.17-7. Computation of working curves for modified Goodrich-Wisler method.

Outflow (o) (cfs)	Storage (s) (cfs hrs)	$\frac{2S}{\Delta t}$ (cfs)	$\frac{2S}{\Delta t} + O$ (cfs)	$\frac{2S}{\Delta t} - O$ (cfs)
0	0	0	0	0
50	70	188	238	138
150	165	438	588	288
300	248	664	964	364
800	651	1740	2540	940
1500	1302	3480	4980	1980
3500	3300	8830	12330	5330
5000	4540	12140	17140	7140
7000	5620	15020	22020	8020
10000	7130	19080	29080	9080

Column 1 taken from Column 1 of table 3.17-1.
Column 2 taken from column 10 of table 3.17-1.
Column 3 uses $\Delta t = 0.75$ hour, as previously computed in steps 8 and 9 of the storage indication stream reach example.

Table 3.17-8. Working table, stream reach routing example, Goodrich-Wisler method.

Time (hours)	I (cfs)	$I_1 + I_2$ (cfs)	O (cfs)
0	0	0	0
.75	1500	1500	480
1.50	3000	4500	1550
2.25	4500	7500	2750
3.00	6000	10500	4180
3.75	5050	11050	5070
4.50	4150	9200	4750
5.25	3300	7450	4080
6.00	2400	5700	3400
6.75	1500	3900	2650
7.50	600	2100	1820
8.25	0	600	1020
9.00	0	0	400
9.75	0	0	150
10.50	0	0	60
etc.	etc.	etc.	etc.
Check:	32000		32360

Wilson method.

This is a very rapid and simple graphical method that gives results equal to the storage-indication method when the storage-indication working curve plots as an almost straight line with a slope of 1 on log paper. By further elaboration not given here it can be made to duplicate results from other methods such as the Muskingum (or McCarthy) and the Tatum methods.

The original method is described in W. T. Wilson's paper "Graphical Flood Routing Method" in Trans. American Geophysical Union, 1941; pages 893-7, with a discussion by W. B. Langbein on pages 897-8.

The simplest form of the Wilson method is described here. Referring to figure 3.17-6, the method is as follows:

1. Compute a travel time (T_t) for a channel reach as described in Step 8 of the storage-indication method, stream reach example; and for a reservoir by the method illustrated on figure 3.17-8.

2. Plot the inflow hydrograph, setting it back T_t hours from zero time.

3. Choose a routing interval Δt, in hours, small enough to define the inflow hydrograph adequately, and not larger than T_t.

4. To start the routing, place a straightedge from a point of zero flow (point A, figure 3.17-6), at T_t hours ahead of the inflow hydrograph to the midpoint (point B) of the first Δt on the inflow hydrograph, at the average inflow rate for that Δt. Draw line AC.

5. Continue the routing with the straightedge for the next Δt at points C and D. Draw line CE.

6. Continue as in step 5 for the remainder of the routing.

The inflow hydrograph may be plotted at its proper time position, if desired, to compare the inflow and outflow hydrograph shapes.

The equation for this routing process is:

$$O_2 = O_1 + K\,(I_1 + I_2 - 2\,O_1) \qquad (6)$$

where:

$$K = \frac{\Delta t}{2T_t + \Delta t} \qquad (7)$$

The equations can be used to route arithmetically with a variable T_t. For most purposes, however, a constant T_t is adequate.

Figure 3.17-7 shows a channel routing, using the inflow hydrograph of figure 3.17-2(a) and a T_t of 0.80 hour which is rounded off from the T_t of 0.813 hour obtained in step 8 of the stream reach example.

Figure 3.~~18~~17-8 shows a discharge-storage curve drawn from the data in columns 3 and 5 of table 3.17-4, and its use in computing a T_t value for a reservoir, for use in routing a flood through an emergency spillway.

Figure 3.17-9 shows an inflow hydrograph (data from column 2 of table 3.17-5) and a routed hydrograph, for the reservoir of figure 3.17-8. Note that the routing was started with a full pool. The method of table 3.17-5(b) can be used to modify an inflow hydrograph for other pool conditions.

Approximate routing methods.

Peak equations. Equation 11 of Section 3.16 is

$$q = \frac{484\,A\,Q}{\frac{D}{2} + 0.6\,T_c}$$

This equation can be successfully used as a substitute for streamflow routings when the runoff (Q) is about uniformly distributed, and when 0.6 T_c closely represents watershed lag (see Section 3.15). When runoff or watershed conditions are complex, subwatersheds can be used. However, the simplicity of using the equation is gradually lost as runoff and watershed complexity increases, and in extreme cases the method is more laborious than a procedure that involves the construction of subwatershed hydrographs and routing by the storage-indication method.

The inflow hydrograph of figure 3.17-2 will be used to demonstrate use of the equation in a reach routing. Accompanying data for the inflow hydrograph are:

$A = 18.6$ sq. mi.
$Q = 2.0$ inches
$D = 1.5$ hours
$T_c = 3.75$ hours

so that for the inflow hydrograph:

$$q = \frac{484\ (18.6)(2.0)}{\frac{1.5}{2} + 0.6\ (3.75)} = \frac{18000}{3.00} = 6000 \text{ cfs}$$

The change in the hydrograph peak due to passage through the reach can be computed when T_t is known. For this case, it is known that $T_t = 0.813$ hour (step 8 of the stream reach example). No local inflow occurred, so A and Q remain the same and only the T_p of 3.00 hours is increased:

$$3.00 + 0.6\ (0.813) = 3.49 \text{ hours}$$

This new T_p gives a peak of:

$$\frac{18000}{3.49} = 5170 \text{ cfs}$$

Figure 3.17-2 shows about 5100 cfs obtained by the storage-indication method. If there is local inflow with increases in A and Q, then equation 11 of Section 3.16 is used in the usual way.

Equation 2 of Section 3.15 applies to watersheds. A similar relation for reservoirs is $L = 0.8\ T_c$, which is used when equation 11 of Section 3.16 is applied in reservoir routings of simple hydrographs. For example, the inflow hydrograph of figure 3.17-9 has

$$q = \frac{484\ A\ Q}{T_p} = \frac{2720}{1.1} = 2470 \text{ cfs.}$$

The emergency spillway T_t, as determined on figure 3.17-8 is 0.70 hour. Then:

$$\frac{2720}{1.1 + 0.8(0.7)} = \frac{2720}{1.66} = 1640 \text{ cfs}$$

which is in close agreement with the peak of the routed hydrograph of figure 3.17-9. Whenever the entire hydrograph is needed, the dimensionless hydrograph of Section 3.16 may be used, but only if the inflow hydrograph was also based on the dimensionless hydrograph or its equivalent triangle. The equation method will not apply for complex discharge-storage relations or complex hydrographs, and in such cases the storage-indication method is required.

General equations. Equation 11 of Section 3.16 can be used to derive equations for watershed project evaluation. Letting subscript wp mean with project, and subscript o mean without project, then equation 11 of Section 3.16 reads

$$q_{wp} = \frac{484\ A_{wp}\ Q_{wp}}{(T_p)_{wp}}$$

and

$$q_o = \frac{484\ A_o\ Q_o}{(T_p)_o}$$

The ratio of the peaks cancels out 484, and gives the general equation:

$$q_{wp} = q_o \frac{A_{wp}\ Q_{wp}\ (T_p)_o}{A_o\ Q_o\ (T_p)_{wp}} \qquad (9)$$

Corresponding factors cancel out if unchanged by the project.

When the floodwater retarding dam or other structural releases must also be taken into account, the general equation is:

$$q_{wp} = q_o \frac{(A_{wp} - A_r)\ Q_{wp}\ (T_p)_o}{(AQ)_o\ (T_p)_{wp}} + q_{csm}\ (A_r) \qquad (10)$$

where q_{csm} is the average release rate in csm, and A_r is the total reservoired area in square miles.

Watershed examples. Equation 9 is used as follows to evaluate a land use and treatment project. When neither the area (A) nor the T_p is going to change, equation 9 becomes:

$$q_{wp} = q_o \frac{Q_{wp}}{Q_o} \qquad (9a)$$

Table 3.17-9 shows a sample computation for a small watershed where the present watershed condition is typified by an 80 curve on figure 3.10-1 and the future condition by a 70 curve.

On small watersheds, when evaluating the effects of large contour furrows or closed-end level terraces on flood peaks, equation 9 can be used in the following form if the storm runoff and furrows or terraces are distributed about uniformly over the watershed:

$$q_{wp} = q_o \frac{(AQ)_{wp}}{(AQ)_o} \qquad (9b)$$

Table 3.17-10 shows a sample computation for a case where in addition to a land use and treatment project, 30 percent of the watershed has contour furrows that can store 1.5" of runoff but which, when overtopped, do not fail. As used, equation 9b is:

$$q_{wp} = q_o \frac{A_s(Q_{wp} - S_s) + A_o Q_{wp}}{(A_s + A_o) Q_o} \qquad (9c)$$

where:

A_s = area draining into furrows

Q_{wp} = runoff with land treatment in place

S_s = storage of furrows

A_o = area not draining into furrows

Q_o = runoff without land treatment in place

See Section 3.12, Determination of surface storage effects, for a more detailed discussion of equation 9c and similar equations.

The last column of table 3.17-9 is used in table 3.17-10 to show the application of equation 10 when the effects of a floodwater retarding reservoir are to be evaluated. In this case only one structure is evaluated and at one point only. Two or more structures can be similarly evaluated as a group if the structures are about uniformly distributed over the watershed. Equation 10 for this case is simplified:

$$q_{wp} = q_o \left(\frac{A - A_r}{A}\right) + q_{csm} \ (A_r) \qquad (10a)$$

It is in this form that the equation is used in the <u>concordant flow method</u>, (See Section 4.2), and applied to each cross section in a reach or to each cross section in the watershed.

Equation 10a is useful when the runoff is about uniformly distributed, and the total watershed is small. When flood volumes exceed storage capacity, reservoir flood routing is necessary to determine flows through emergency spillways. If these flows are large, equation 10a does not apply to them, and their effects must be determined by other routing methods.

Table 3.17-9. Computation of effect of land use and treatment program on flood peaks by use of equation 9a.

q_o (cfs)	Q_o (inches)	Q_{wp} (inches)	q_{wp} (cfs)
1880	3.24	2.38	1380
1540	3.62	2.67	1140
1500	2.92	2.06	1060
1200	2.10	1.38	790
1060	2.66	1.86	740
1020	1.53	.92	610
610	1.14	.63	340
570	1.56	.95	350
400	1.12	.61	220
240	.56	.23	100

Last column rounded off to nearest ten.

Table 3.17-10. Computation using equation 9c to show effect of contour furrows on flood peaks.

$A_s + A_o$ = Total watershed area = 3.40 sq. mi.
A_s = Area furrowed = 1.02 " "
A_o = Area not furrowed = 2.38 " "
S_s = Furrow storage = 1.50 inches

q_o (cfs)	Q_o (inches)	$(A_s + A_o) \times Q_o$ (in.mi.)	Q_{wp} (inches)	$(Q_{wp} - S_s)$ (inches)	$A_s(Q_{wp} - S_s)$ (in.miles)	$A_s(Q_{wp})$ (in.mi.)	Col. 6 plus Col. 7 (in.miles)	Col. 8 divided by Col.3= q_{wp} (cfs)
1880	3.24	11.0	2.38	0.88	0.90	5.66	6.56	1120
1540	3.62	12.3	2.67	1.17	1.19	6.35	7.54	945
1500	2.92	9.93	2.06	.56	.57	4.90	5.47	827
1200	2.10	7.13	1.38	0	0	3.28	3.28	551
1060	2.66	9.04	1.86	.36	.37	4.42	4.79	561
1020	1.53	5.20	.92	0	0	2.19	2.19	429
610	1.14	3.88	.63	0	0	1.50	1.50	236
570	1.56	5.31	.95	0	0	2.26	2.26	242
400	1.12	3.80	.61	0	0	1.45	1.45	153
240	.56	1.90	.23	0	0	.55	.55	70

Columns 1, 2, and 4 from table 3.17-7. Slide rule computations.

426963 O - 57 - 16

Figure 3.17-10(a) shows a typical situation where equation 10a must be expanded. The effects of the structures on the damages in the reach must be determined, but one structure has an effect on only a part of the reach. In this case, and others where the reservoired areas are about uniformly distributed on the watershed, and nearly uniform runoff occurs for the storms, equation 10a is changed to:

$$q_{wp} = q_o \left[\frac{A - A_r}{2A} + \frac{B - B_r}{2B} \right] + q_{csm} \left[\frac{A_r + B_r}{2} \right] \qquad (10b)$$

where

A = total watershed area draining into the head of the reach

B = total watershed area (including area A) draining into the foot of the reach

A_r = reservoired area above the head of the reach

B_r = <u>all</u> the reservoired area above the foot of the reach

Other symbols are as given previously.

Equation 10(b) is generally computed for several selected floods, as shown in table 3.17-12, and the results plotted as shown in figure 3.17-~~5~~10(b) to give an easy solution for all floods in the evaluation series. Note the following on the graph:

1. The effect of the reservoirs decreases as q_o decreases until at about 850 cfs and below the reservoirs have no effect.

2. The last line of the computation table of table 3.17-10(b) gives q_{wp} of 616 cfs for a q_o of 500. This does <u>not</u> mean that the small flood has been increased in size. Whenever q_{wp} exceeds q_o, then q_{wp} is taken as equal to q_o. As shown on the graph, the lowest computed point is used only to help define the "use line".

3. The topmost point, where q_o equals 16000 cfs, is the maximum flood to be evaluated, and its volume does not exceed the flood storage volumes of the structures. For greater floods (with greater volumes) the effect of the structure diminishes. When floods that produce emergency spillway flow are being evaluated, flood routing of some kind through the reservoir and down the reach will be required to define additional points for extending the "use line".

Table 3.17-11. Computation using equation 10a to show effect of a retarding reservoir on flood peaks.

A = Total watershed area = 3.40 sq. mi.

A_r = Area reservoired = 1.02 sq. mi.

Reservoir flood storage* = 3.00 inches

q_{csm} = Average release rate = 15.0 csm

q_o** (cfs)	$\frac{A - A_r}{A}$	Col. 1 times Col. 2 (cfs)	Average release rate (cfs)	Col. 3 plus Col. 4 (cfs)
1380	0.70	967	15.3***	982
1140	"	798	"	813
1060	"	742	"	757
790	"	553	"	568
740	"	518	"	533
610	"	427	"	442
340	"	238	"	253
350	"	245	"	260
220	"	154	"	169
100	"	70	"	85

*At emergency spillway crest
**From last column of table 3.17-9
***Usually rounded off--in this case, to 15 cfs.

Table 3.17-12. Computation using equation 10b to show effects of a system of floodwater retarding structures on flood peaks.

$A = 43.0$ sq. mi. $\qquad A_r = 17.0$ sq. mi.

$B = 75.0$ sq. mi. $\qquad B_r = 21.0$ sq. mi.

$q_{csm} = 15$ csm

$$\left[\frac{A - A_r}{2A} + \frac{B - B_r}{2B}\right] = \left[\frac{43.0 - 17.0}{86.0} + \frac{75.0 - 21.0}{150.0}\right] = 0.663$$

$$q_{csm}\left[\frac{A_r + B_r}{2}\right] = 15.0\left[\frac{17.0 + 21.0}{2}\right] = 285 \text{ cfs.}$$

$$q_{wp} = 0.663\, q_o + 285$$

q_o (cfs)	$0.663\, q_o$ (cfs)	q_{wp} (cfs)
16000	10600 ~~11000~~	10885 ~~11285~~
8000	5300	5585
4000	2650	2935
2000	1320	1605
1000	663	948
500	331	616

See graph, figure 3.17-10(b)

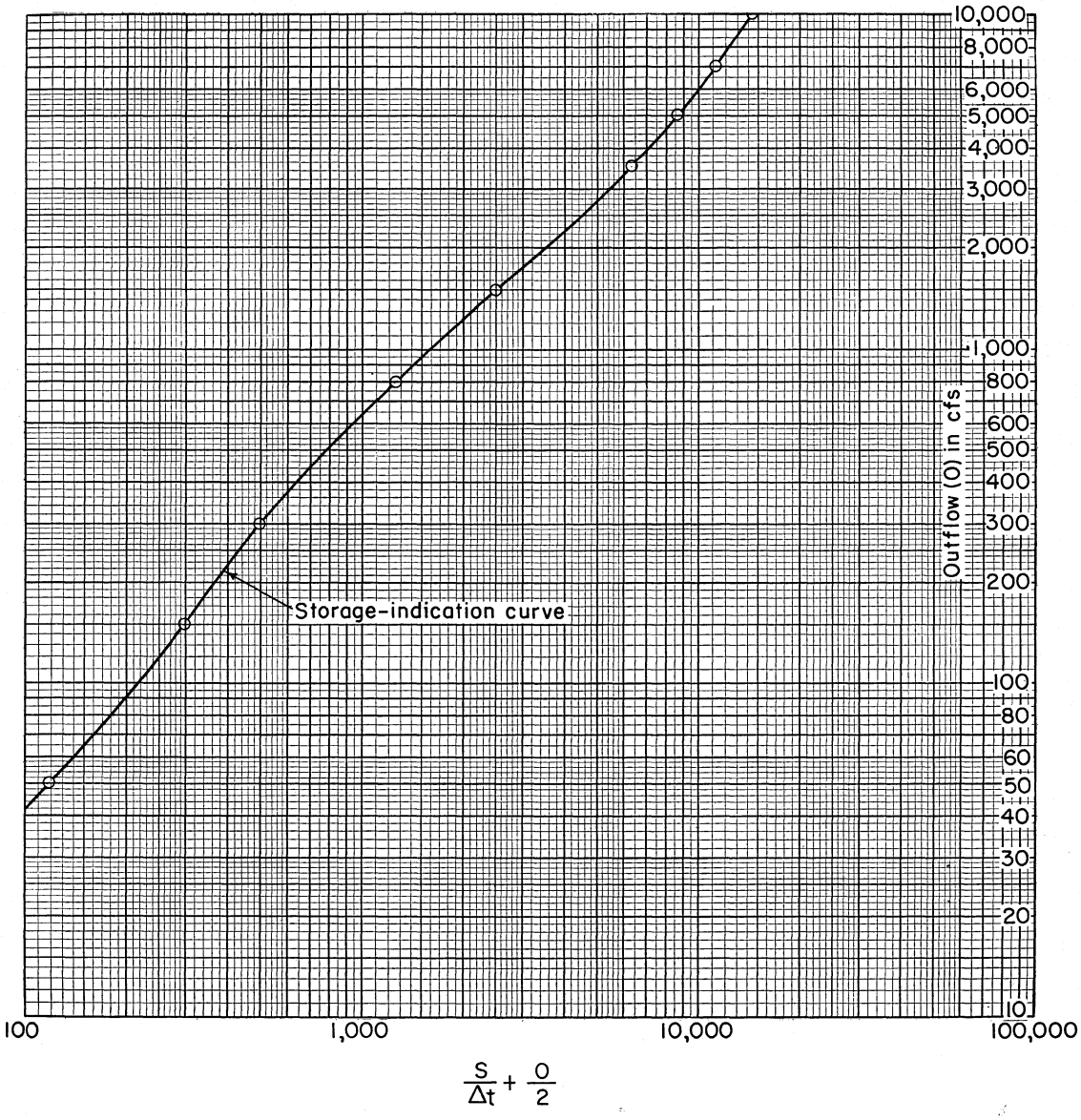
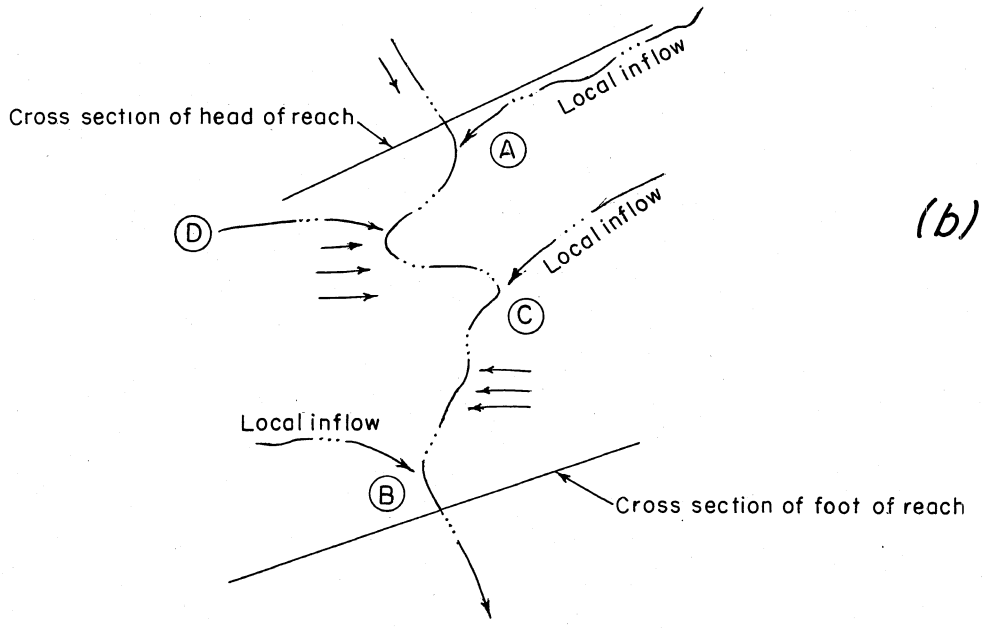
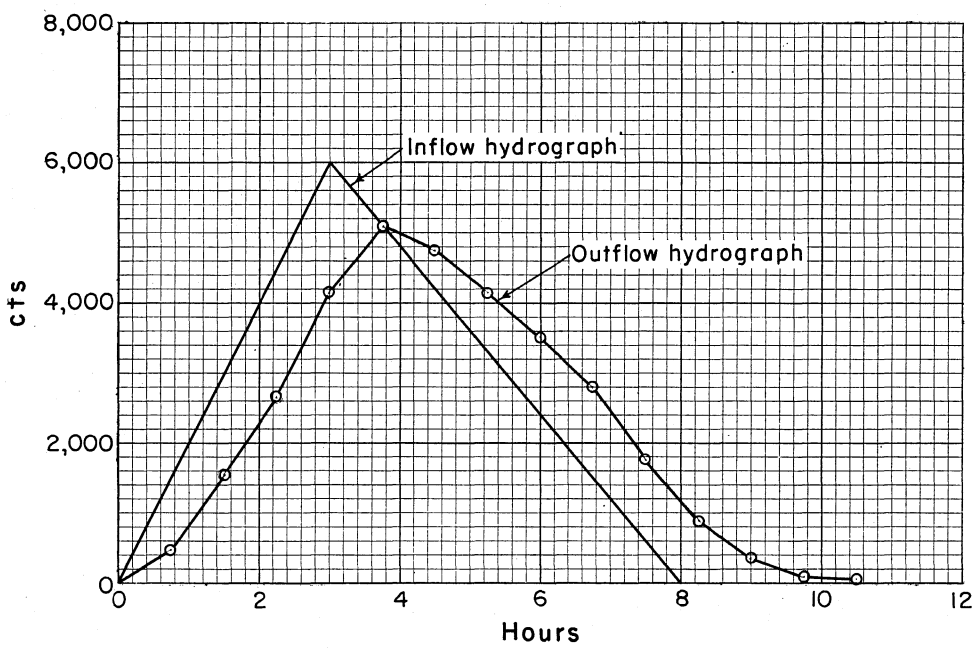

10,000
8,000
6,000
5,000
4,000
3,000
2,000
1,000
800
600
500
400
300
200
100
80
60
50
40
30
20
10
Outflow (O) in cfs
Storage-indication curve
100
1,000
10,000
100,000
$\frac{S}{\Delta t} + \frac{O}{2}$

Figure 3.17-1

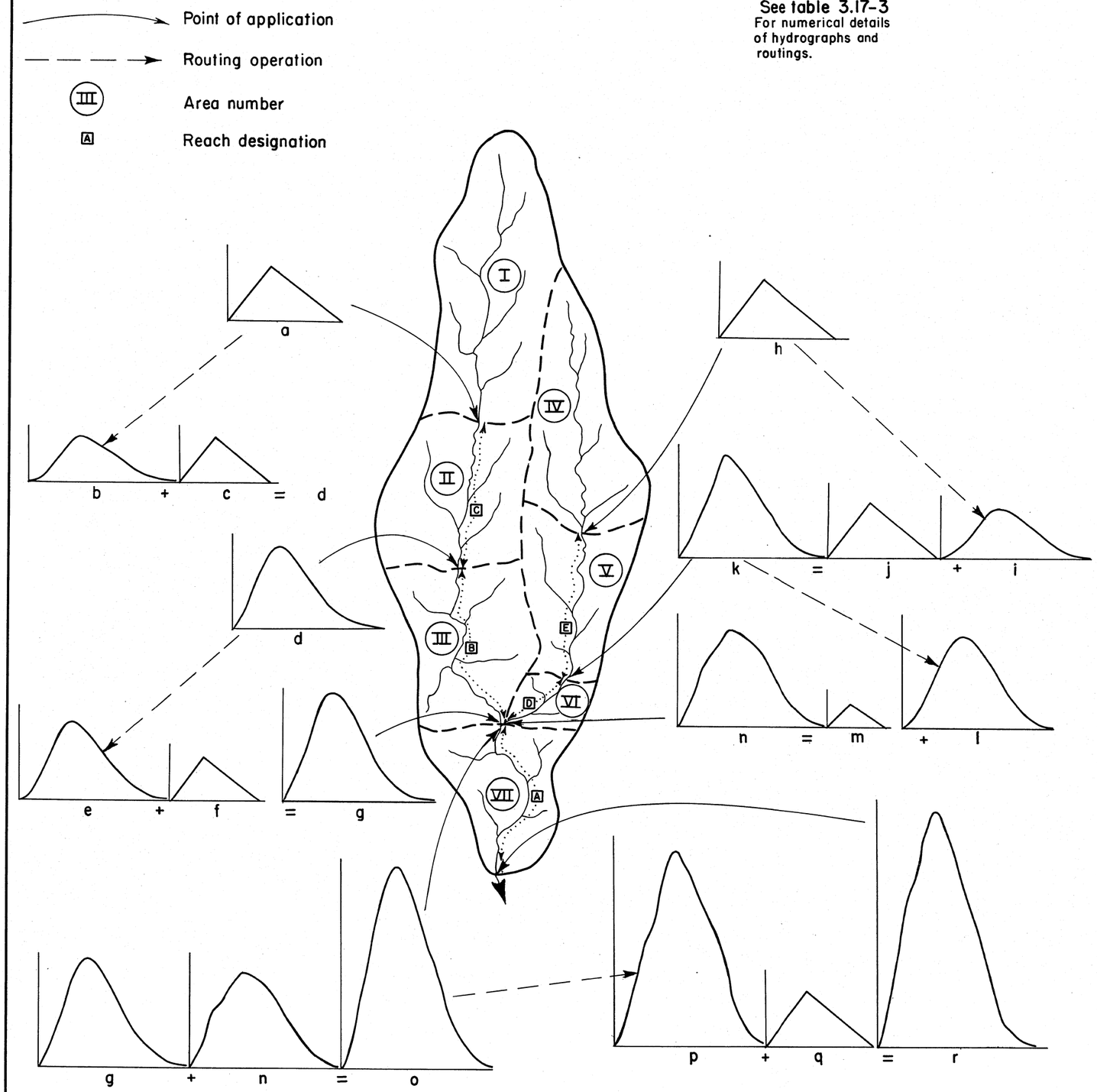


(b)



(a)

Figure 3.17-2



See table 3.17-3
For numerical details of hydrographs and routings.

Typical sequence of flood routing operations in a watershed

Hydrograph a is the hydrograph of runoff from Area I, which enters at the head of Reach C. This hydrograph is routed to give:

Hydrograph b, and to this is added:

Hydrograph c, the hydrograph of local inflow from Area II, to give:

Hydrograph d, the hydrograph of runoff leaving the foot of Reach C and entering the head of Reach B. This is routed through the reach to give:

Hydrograph e. To this is added:

Hydrograph f, the triangular hydrograph of local inflow from Area III, to give:

Hydrograph g, the hydrograph of inflow from the West Branch, entering the head of Reach A. The hydrographs for the East Branch are then routed:

Hydrograph h is the hydrograph of runoff from Area IV, which enters at the head of Reach E. This hydrograph is routed to give:

Hydrograph i, and to this is added:

Hydrograph j, the hydrograph of local inflow from Area V, to give:

Hydrograph k, the hydrograph of runoff entering the head of Reach D. This is routed through Reach D to give:

Hydrograph l. To this is added:

Hydrograph m, the hydrograph of local inflow from Area VI, which gives:

Hydrograph n, the hydrograph of runoff from the East Branch, entering the head of Reach A. This hydrograph is added to hydrograph g to give:

Hydrograph o, the hydrograph of combined inflow entering the head of Reach A. This is routed through the reach to give:

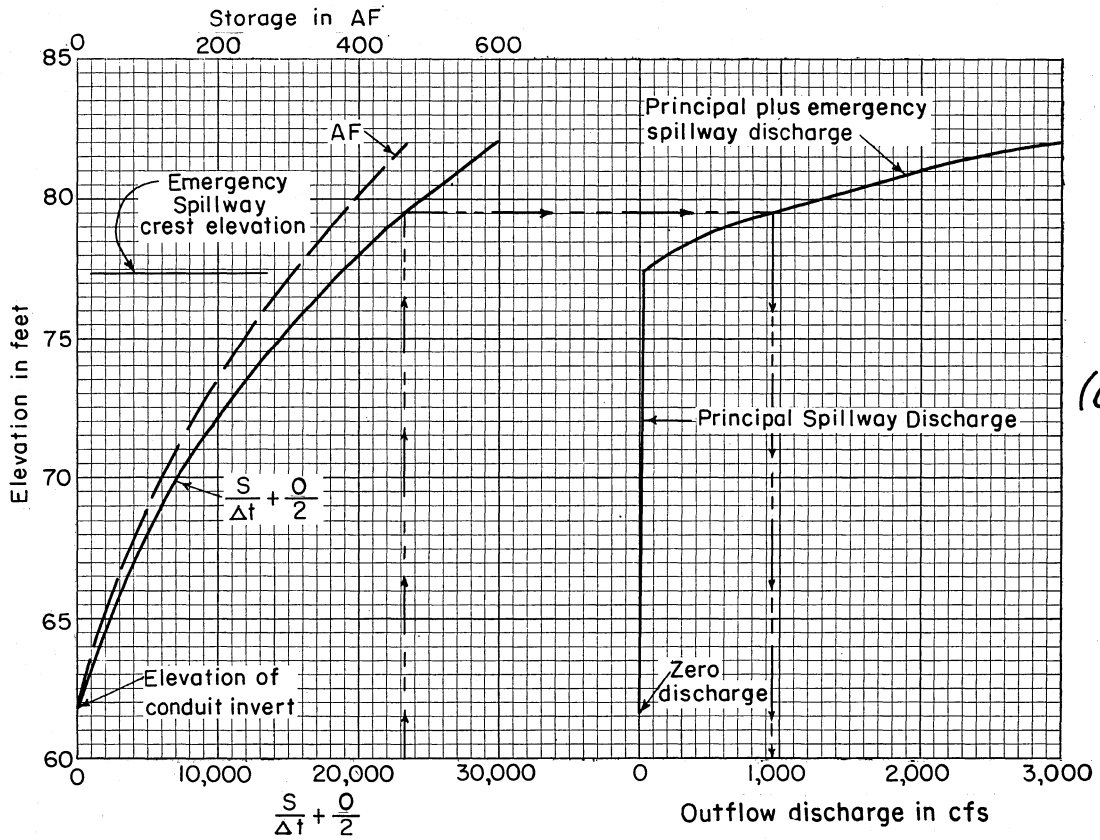
Hydrograph p, to which is added:

Hydrograph q, the hydrograph of local inflow from Area VII, which gives:

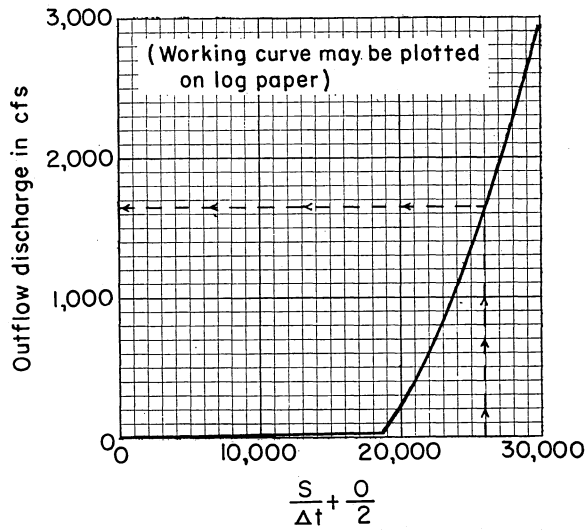
Hydrograph r, the outflow hydrograph for the watershed.

Figure 3.17-3

426963 O -57 (Face p. 246)



(a)



(b)

Figure 3.17-4

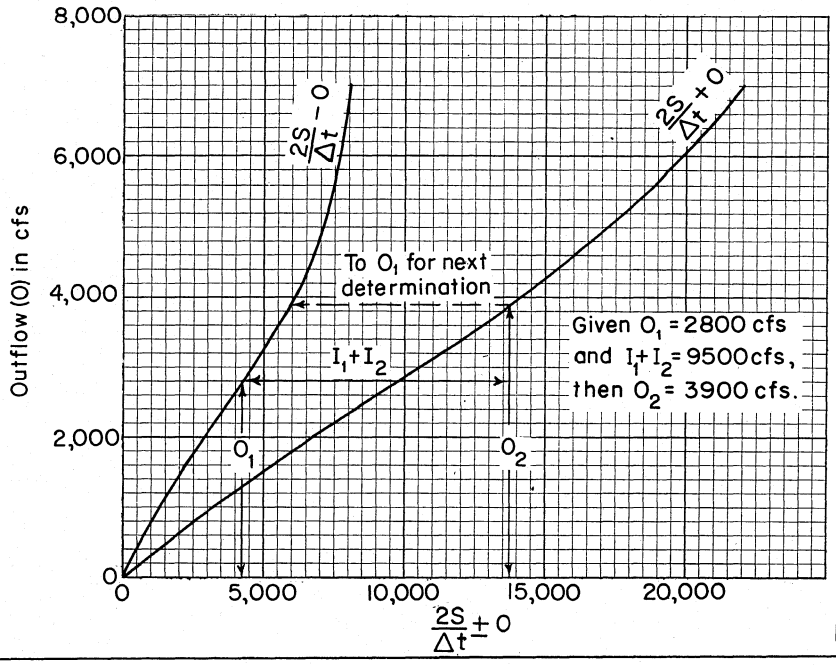


Figure 3.17-5

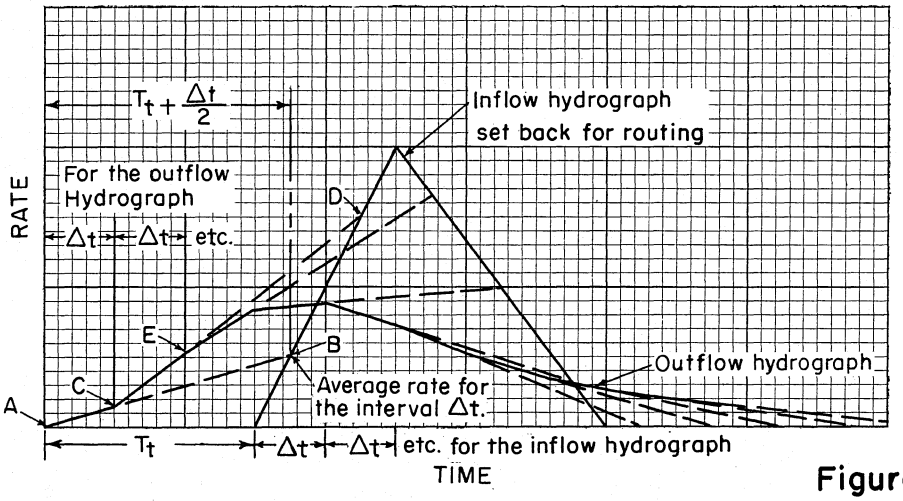


Figure 3.17-6

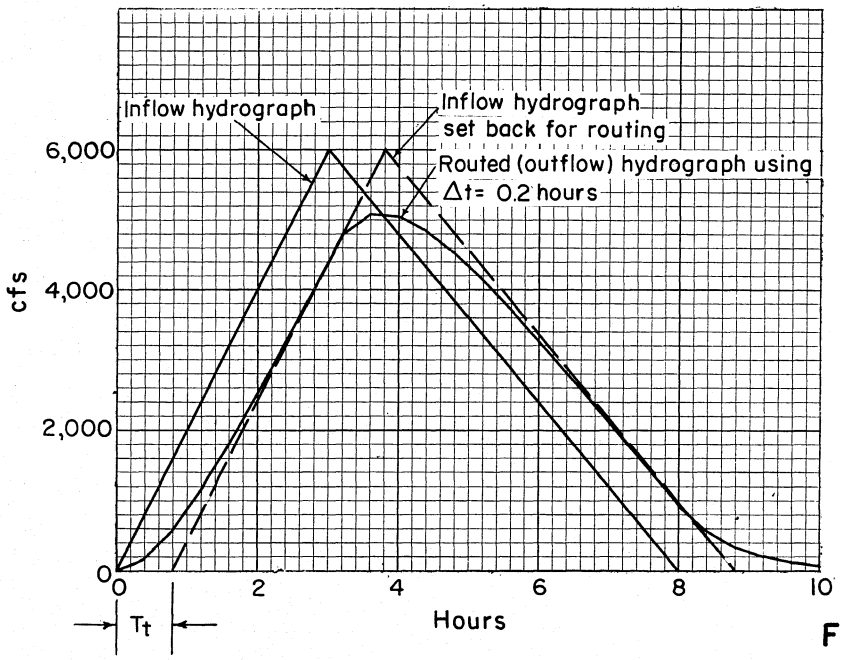


Figure 3.17-7

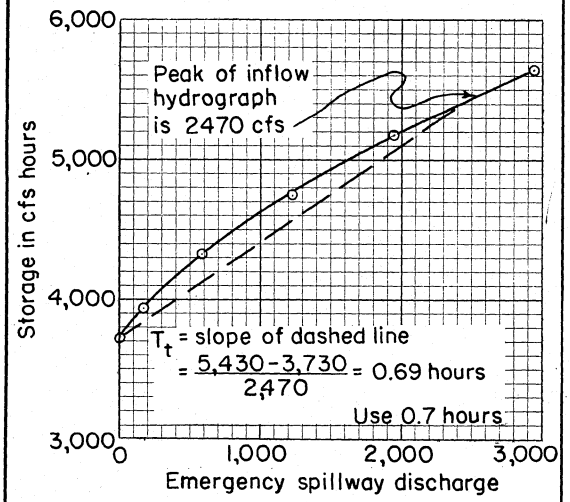


Figure 3.17-8

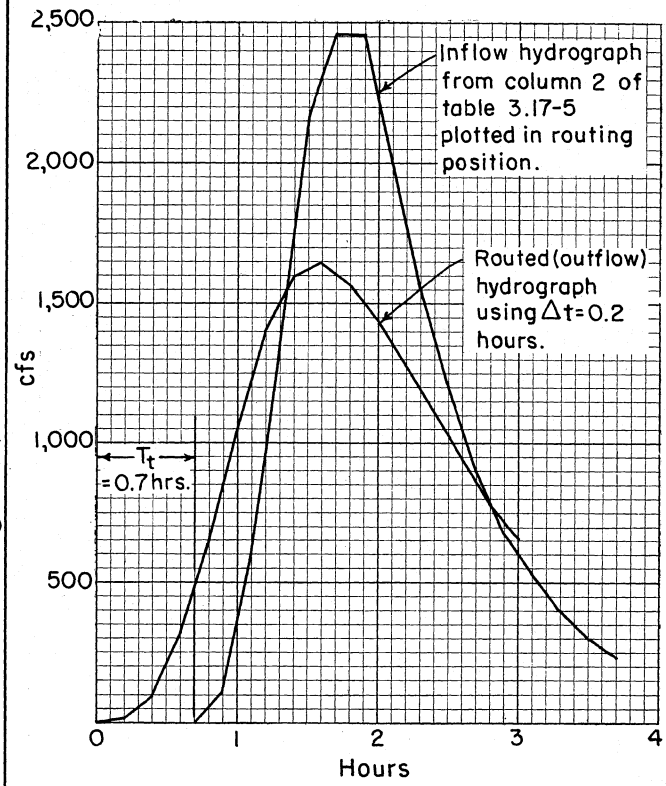


Figure 3.17-9

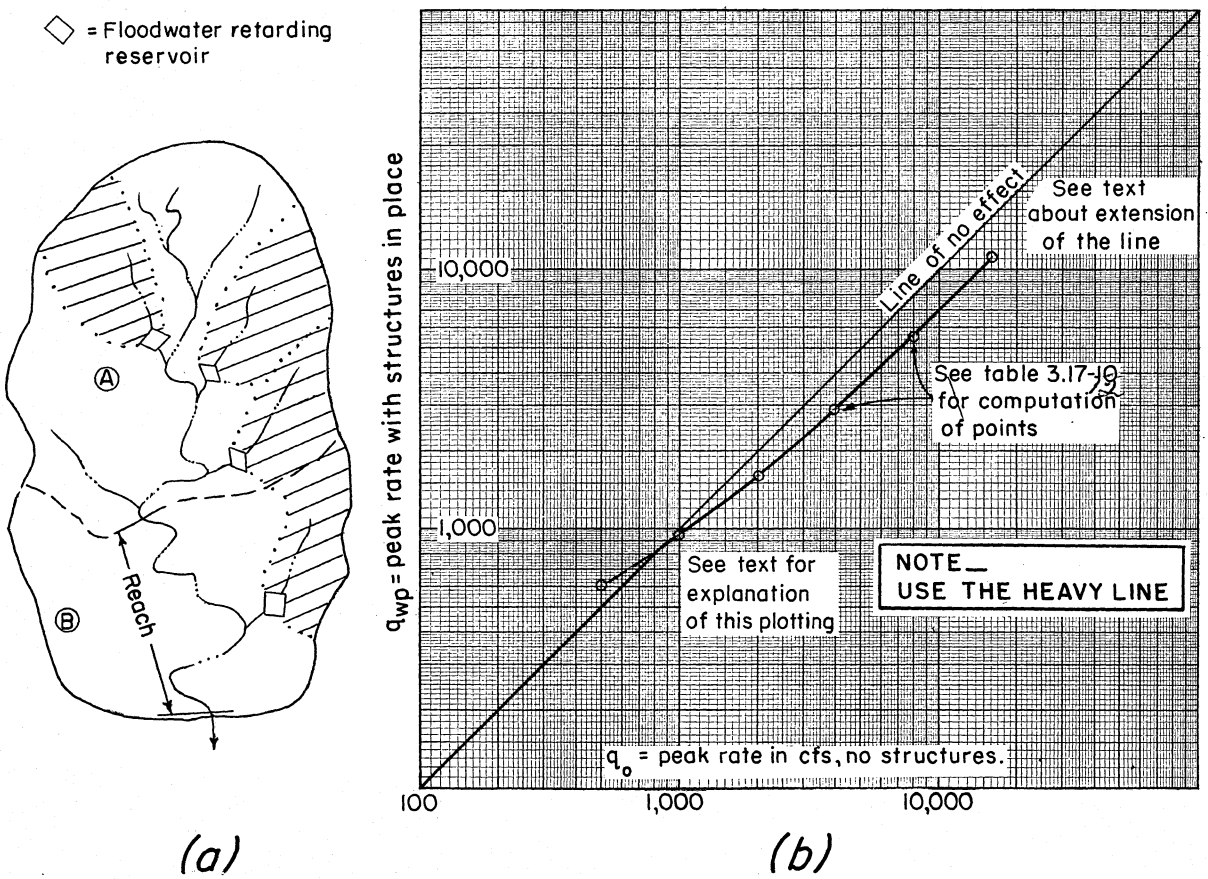


Figure 3.17-10

3.18 Frequency methods

Purpose. These methods are used to estimate the frequency of events such as flood peaks, to test such data for normality, and to extend the range of usefulness of the data. In practice, a 20-year record of flood peaks, for example, can be used to determine a flood frequency line; if this line is found adequate by a test, it is extended beyond the actual range of data to obtain the 100-year frequency flood or other required frequencies. The methods are equally useful with similar types of data, such as flood damages, sediment loads, and rainfall amounts or intensities.

Types of frequency series. Two types of frequency series are used, the annual series, which uses only the largest value in each year, and the partial duration series, which uses all the values in each year equal to or above an arbitrary base, usually the lowest annual series value. For example, if floods of 3000, 2000, 1500, and 1000 cfs occur on a watershed in a given year, the 3000 cfs flood is the annual flood and is used in the annual series. If the base flood is 1200 cfs, then the first three floods are used in the partial duration series. The distinction between the two series can be made for many other kinds of data, but with yearly data-- such as total annual runoff-- the annual and partial duration series are identical.

In general, the annual series should be used, since the partial duration series can be easily computed from it, and there is less total labor involved.

Time units. The calendar or water year is the usual time unit. Other time units may be growing seasons, months, weeks, or other arbitrary periods shorter than a year, and they are used in either of two ways: (1), the largest flood in each June, for example, may be used to make a frequency line, and the results apply only to the months of June; or (2), the largest flood in each month is selected so that twelve in each year are used, and the results apply to one month in the year without specifying which month. When the time unit is less than a year, skewness is either introduced or increased, and must be corrected (see "skewed frequency lines") or it will often be found that a 100-year frequency value, for instance, will be larger than if based on the whole year as a unit.

Probability paper. The methods described below use logarithmic normal probability paper, also called log-normal or Hazen paper. The specific type of paper recommended for Service use has a three-cycle logarithmic scale for the data, and both percent chance and standard deviation scales for the normal distribution, which extends at least three standard deviations, plus and minus, from the mean. A sample is shown on figure 3.18-1.

Frequency lines. Annual series

Plotting method. The following example uses a record of annual flood peaks to illustrate the plotting method. The steps would be the same with other types of data.

1. Tabulate the annual floods, as shown in column 2 of table 3.18-1.

2. Arrange the floods in order of size, with the largest first (column 4 of table 3.18-1). This arrangement is an array.

3. Tabulate the plotting positions (F_a). These positions are computed using the Hazen equation:

$$F_a = \frac{100\ (2n-1)}{2y} \qquad (1)$$

where F_a = the plotting position in percent
n = the rank number
y = the number of years of record.

Table 3.18-2 gives values of F_a from y = 6 to y = 50, computed using equation 1.

4. Prepare the vertical scale of the log-normal paper as shown on figure 3.18-1, and plot the floods versus their F_a positions. The logarithmic scale is used for the floods and the percent scale for the F_a values.

5. Draw a straight line through the plotted points as follows: Place one draftsman's triangle on the paper and get the general slope or trend of the plotted points, giving less weight to a point that plots far out of line. Hold this position and place a second triangle against the first. Using them together as in drawing parallel lines, slide the first triangle to a position where some points are on the line, if possible, and half of the remaining points are above the line and half are below. When this position is found, draw a straight line, which will be the frequency line for that set of data. See "Skewed frequency lines" for the procedure when the plotted points do not line up reasonably well.

The percent scale of figure 3.18-1 is also called the percent chance scale. For example, at the 50 percent chance line, the discharge as given by the frequency line is about 1150 cfs. The probability is 50 percent (or fifty chances in a hundred) that a flood of this size will be equalled or exceeded in any given year. At the 1 percent chance line, the frequency line shows a discharge of about 3150 cfs, with a probability of 1 percent (or one chance in a hundred) that this size flood will be equalled or exceeded in any given year.

Table 3.18-1. Data for constructing annual flood frequency line, Boulder Creek near Leonia, Idaho. 1/

Water year	Annual flood (cfs)	n	q_a	F_a
1929	880	1	2700	2.5
1930	683	2	2170	7.5
31	840	3	2050	12.5
32	1330	4	1710	17.5
33	1330	5	1540	22.5
34	1540	6	1330	27.5
35	1280	7	1330	32.5
36	1100	8	1280	37.5
37	920	9	1220	42.5
38	2050	10	1170	47.5
39	854	11	1100	52.5
1940	760	12	1080	57.5
41	2170	13	920	62.5
42	871	14	880	67.5
43	1170	15	871	72.5
44	488	16	854	77.5
45	1080	17	840	82.5
46	1220	18	760	87.5
47	2700	19	683	92.5
48	1710	20	488	97.5

1/ Data in column 2 from USGS Water-Supply Paper 1080.

Table 3.18-2. Values of F_a for y = 6 to y = 50

$$F_a = 100 \frac{2n - 1}{2y}$$

n \ y	6	7	8	9	10	11	12	13	14	15	16
1	8.3	7.1	6.2	5.6	5.0	4.5	4.2	3.8	3.6	3.3	3.1
2	25.0	21.4	18.8	16.7	15.0	13.6	12.5	11.5	10.7	10.0	9.4
3	41.7	35.7	31.2	27.8	25.0	22.7	20.8	19.2	17.9	16.7	15.6
4	58.3	50.0	43.8	38.9	35.0	31.8	29.2	26.9	25.0	23.3	21.9
5	75.0	64.3	56.2	50.0	45.0	40.9	37.5	34.6	32.1	30.0	28.1
6	91.7	78.6	68.8	61.1	55.0	50.0	45.8	42.3	39.3	36.7	34.4
7		92.9	81.2	72.2	65.0	59.1	54.2	50.0	46.4	43.3	40.6
8			93.8	83.3	75.0	68.2	62.5	57.7	53.6	50.0	46.9
9				94.4	85.0	77.3	70.8	65.4	60.7	56.7	53.[illegible]
10					95.0	86.4	79.2	73.1	67.9	63.3	59.4
11						95.5	87.5	80.8	75.0	70.0	65.6
12							95.8	88.5	82.1	76.7	71.9
13								96.2	89.3	83.3	78.1
14									96.4	90.0	84.4
15										96.7	90.6
16											96.9

n \ y	17	18	19	20	21	22	23	24	25	26	27	28
1	2.9	2.8	2.6	2.5	2.4	2.3	2.2	2.1	2.0	1.9	1.9	1.8
2	8.8	8.3	7.9	7.5	7.1	6.8	6.5	6.2	6.0	5.8	5.6	5.4
3	14.7	13.9	13.2	12.5	11.9	11.4	10.9	10.4	10.0	9.6	9.3	8.9
4	20.6	19.4	18.4	17.5	16.7	15.9	15.2	14.6	14.0	13.5	13.0	12.5
5	26.5	25.0	23.7	22.5	21.4	20.5	19.6	18.8	18.0	17.3	16.7	16.1
6	32.4	30.6	28.9	27.5	26.2	25.0	23.9	22.9	22.0	21.2	20.4	19.6
7	38.2	36.1	34.2	32.5	31.0	29.5	28.3	27.1	26.0	25.0	24.1	23.2
8	44.1	41.7	39.5	37.5	35.7	34.1	32.6	31.2	30.0	28.8	27.8	26.8
9	50.0	47.2	44.7	42.5	40.5	38.6	37.0	35.4	34.0	32.7	31.5	30.4
10	55.9	52.8	50.0	47.5	45.2	43.2	41.3	39.6	38.0	36.5	35.2	33.9
11	61.8	58.3	55.3	52.5	50.0	47.7	45.7	43.7	42.0	40.4	38.9	37.5
12	67.6	63.9	60.5	57.5	54.8	52.3	50.0	47.9	46.0	44.2	42.6	41.1
13	73.5	69.4	65.8	62.5	59.5	56.8	54.3	52.1	50.0	48.1	46.3	44.6
14	79.4	75.0	71.1	67.5	64.3	61.4	58.7	56.3	54.0	51.9	50.0	48.2
15	85.3	80.6	76.3	72.5	69.0	65.9	63.0	60.4	58.0	55.8	53.7	51.8
16	91.2	86.1	81.6	77.5	73.08	70.5	67.4	64.6	62.0	59.6	57.4	55.4
17	97.1	91.7	86.8	82.5	78.6	75.0	71.7	68.8	66.0	63.5	61.1	58.9
18		97.2	92.1	87.5	83.3	79.5	76.1	72.9	70.0	67.3	64.8	62.5
19			97.4	92.5	88.1	84.1	80.4	77.1	74.0	71.2	68.5	66.1
20				97.5	92.9	88.6	84.8	81.2	78.0	75.0	72.2	69.6
21					97.6	93.2	89.1	85.4	82.0	78.8	75.9	73.2
22						97.7	93.5	89.6	86.0	82.7	79.6	76.8
23							97.8	93.8	90.0	86.5	83.3	80.4
24								97.9	94.0	90.4	87.0	83.9
25									98.0	94.2	90.7	87.5
26										98.1	94.4	91.1
27											98.1	94.6
28												98.2

Table 3.18-2 (Cont'd)

n \ y	29	30	31	32	33	34	35	36	37	38	39	40
1	1.7	1.7	1.6	1.6	1.5	1.5	1.4	1.4	1.4	1.3	1.3	1.3
2	5.2	5.0	4.8	4.7	4.5	4.4	4.3	4.2	4.1	3.9	3.8	3.8
3	8.6	8.3	8.1	7.8	7.6	7.4	7.1	6.9	6.8	6.6	6.4	6.3
4	12.1	11.7	11.3	10.9	10.6	10.3	10.0	9.7	9.5	9.2	9.0	8.8
5	15.5	15.0	14.5	14.1	13.6	13.2	12.9	12.5	12.2	11.8	11.5	11.3
6	19.0	18.3	17.7	17.2	16.7	16.2	15.7	15.3	14.9	14.5	14.1	13.8
7	22.4	21.7	21.0	20.3	19.7	19.1	18.6	18.1	17.6	17.1	16.7	16.3
8	25.9	25.0	24.2	23.4	22.7	22.1	21.4	20.8	20.3	19.7	19.2	18.8
9	29.3	28.3	27.4	26.6	25.8	25.0	24.3	23.6	23.0	22.4	21.8	21.3
10	32.8	31.7	30.6	29.7	28.8	27.9	27.1	26.4	25.7	25.0	24.4	23.8
11	36.2	35.0	33.9	32.8	31.8	30.9	30.0	29.2	28.4	27.6	26.9	26.3
12	39.7	38.3	37.1	35.9	34.8	33.8	32.9	31.9	31.1	30.3	29.5	28.8
13	43.1	41.7	40.3	39.1	37.9	36.8	35.7	34.7	33.8	32.9	32.0	31.3
14	46.6	45.0	43.5	42.2	40.9	39.7	38.6	37.5	36.5	35.5	34.6	33.8
15	50.0	48.3	46.8	45.3	43.9	42.6	41.4	40.3	39.2	38.2	37.2	36.3
16	53.4	51.7	50.0	48.4	47.0	45.6	44.3	43.1	41.9	40.8	39.7	38.8
17	56.9	55.0	53.2	51.6	50.0	48.5	47.1	45.8	44.6	43.4	42.3	41.3
18	60.3	58.3	56.5	54.7	53.0	51.5	50.0	48.6	47.3	46.1	44.9	43.8
19	63.8	61.7	59.7	57.8	56.1	54.4	52.9	51.4	50.0	48.7	47.4	46.2
20	67.2	65.0	62.9	60.9	59.1	57.4	55.7	54.2	52.7	51.3	50.0	48.8
21	70.7	68.3	66.1	64.1	62.1	60.3	58.6	56.9	55.4	53.9	52.6	51.2
22	74.1	71.7	69.4	67.2	65.2	63.2	61.4	59.7	58.1	56.6	55.1	53.8
23	77.6	75.0	72.6	70.3	68.2	66.2	64.3	62.5	60.8	59.2	57.7	56.2
24	81.0	78.3	75.8	73.4	71.2	69.1	67.1	65.3	63.5	61.8	60.3	58.7
25	84.5	81.7	79.0	76.6	74.2	72.1	70.0	68.1	66.2	64.5	62.8	61.2
26	87.9	85.0	82.3	79.7	77.3	75.0	72.9	70.8	68.9	67.1	65.4	63.7
27	91.4	88.3	85.5	82.8	80.3	77.9	75.7	73.6	71.6	69.7	68.0	66.2
28	94.8	91.7	88.7	85.9	83.3	80.9	78.6	76.4	74.3	72.4	70.5	68.7
29	98.3	95.0	91.9	89.1	86.4	83.8	81.4	79.2	77.0	75.0	73.1	71.2
30		98.3	95.2	92.2	89.4	86.8	84.3	81.9	79.7	77.6	75.6	73.7
31			98.4	95.3	92.4	89.7	87.1	84.7	82.4	80.3	78.2	76.2
32				98.4	95.5	92.6	90.0	87.5	85.1	82.9	80.8	78.7
33					98.5	95.6	92.9	90.3	87.8	85.5	83.3	81.2
34						98.5	95.7	93.1	90.5	88.2	85.9	83.7
35							98.6	95.8	93.2	90.8	88.5	86.2
36								98.6	95.9	93.4	91.0	88.7
37									98.6	96.1	93.6	91.2
38										98.7	96.2	93.7
39											98.7	96.2
40												98.7

Table 3.18-2 (Cont'd)

n \ y	41	42	43	44	45	46	47	48	49	50
1	1.2	1.2	1.2	1.1	1.1	1.1	1.1	1.0	1.0	1.0
2	3.7	3.6	3.5	3.4	3.3	3.3	3.2	3.1	3.1	3.0
3	6.1	6.0	5.8	5.7	5.6	5.4	5.3	5.2	5.1	5.0
4	8.5	8.3	8.1	8.0	7.8	7.6	7.4	7.3	7.1	7.0
5	11.0	10.7	10.5	10.2	10.0	9.8	9.6	9.4	9.2	9.0
6	13.4	13.1	12.8	12.5	12.2	12.0	11.7	11.5	11.2	11.0
7	15.9	15.5	15.1	14.8	14.4	14.1	13.8	13.5	13.3	13.0
8	18.3	17.9	17.4	17.0	16.7	16.3	16.0	15.6	15.3	15.0
9	20.7	20.2	19.8	19.3	18.9	18.5	18.1	17.7	17.3	17.0
10	23.2	22.6	22.1	21.6	21.1	20.7	20.2	19.8	19.4	19.0
11	25.6	25.0	24.4	23.9	23.3	22.8	22.3	21.9	21.4	21.0
12	28.0	27.4	26.7	26.1	25.6	25.0	24.5	24.0	23.5	23.0
13	30.5	29.8	29.1	28.4	27.8	27.2	26.6	26.0	25.5	25.0
14	32.9	32.1	31.4	30.7	30.0	29.3	28.7	28.1	27.6	27.0
15	35.4	34.5	33.7	33.0	32.2	31.5	30.9	30.2	39.6	29.0
16	37.8	36.9	36.0	35.2	34.4	33.7	33.0	32.3	31.6	31.0
17	40.2	39.3	38.4	37.5	36.7	35.9	35.1	34.4	33.7	33.0
18	42.7	41.7	40.7	39.7	38.9	38.0	37.2	36.5	35.7	35.0
19	45.1	44.0	43.0	42.0	41.1	40.2	39.4	38.5	37.8	37.0
20	47.6	46.4	45.3	44.3	43.3	42.4	41.5	40.6	39.8	39.0
21	50.0	48.8	47.7	46.6	45.6	44.6	43.6	42.7	41.8	41.0
22	52.4	51.2	50.0	48.9	47.8	46.7	45.7	44.8	43.9	43.0
23	54.9	53.6	52.3	51.1	50.0	48.9	47.9	46.9	45.9	45.0
24	57.3	56.0	54.7	53.4	52.2	51.1	50.0	49.0	48.0	47.0
25	59.8	58.3	57.0	55.7	54.4	53.3	52.1	51.0	50.0	49.0
26	62.2	60.7	59.3	58.0	56.7	55.4	54.3	53.1	52.0	51.0
27	64.6	63.1	61.6	60.3	58.9	57.6	56.4	55.2	54.1	53.0
28	67.1	65.5	64.0	62.5	61.1	59.8	58.5	57.3	56.1	55.0
29	69.5	67.9	66.3	64.8	63.3	62.0	60.6	59.4	58.2	57.0
30	72.0	70.2	68.6	67.0	65.6	64.1	62.8	61.5	60.2	59.0
31	74.4	62.6	70.9	69.3	67.8	66.3	64.9	63.5	62.2	61.0
32	76.8	75.0	73.3	71.6	70.0	68.5	67.0	56.6	64.3	63.0
33	79.3	77.4	65.6	73.9	72.2	70.7	69.1	67.7	66.3	65.0
34	81.7	79.8	77.9	76.1	74.4	72.8	71.3	69.8	68.4	67.0
35	84.1	82.1	80.2	78.4	76.7	75.0	73.4	71.9	70.4	69.0
36	86.6	84.5	82.6	80.7	78.9	77.2	75.5	74.0	72.4	71.0
37	89.0	86.9	84.9	83.0	81.1	79.3	77.7	76.0	74.5	73.0
38	91.5	89.3	87.2	85.2	83.3	81.5	79.8	78.1	76.5	75.0
39	93.9	91.7	89.5	87.5	85.6	83.7	81.9	80.2	78.6	77.0
40	96.3	94.0	91.9	89.8	87.8	85.9	84.0	82.3	80.6	79.0
41	98.8	96.4	94.2	92.0	90.0	88.0	86.2	84.4	82.7	81.0
42		98.8	96.5	94.3	92.2	90.2	88.3	86.5	84.7	83.0
43			98.8	96.6	94.4	92.4	90.4	88.5	86.7	85.0
44				98.9	96.7	94.6	92.6	90.6	88.8	87.0
45					98.9	96.7	94.7	92.7	90.8	89.0
46						98.9	96.8	94.8	92.9	91.0
47							98.9	97.9	94.9	93.0
48								99.0	96.9	95.0
49									99.0	97.0
50										99.0

A frequency, in years, is obtained using the relation:

$$\frac{100}{\text{percent chance}} = \text{frequency} \qquad (2)$$

For example, the 1 percent chance flood is the 100-year frequency flood, and the 50 percent chance flood is the 2-year flood.

Skewed frequency lines. The purpose of the special probability paper is to make the plotted points fall into reasonable alinement so that a straight line can be drawn and easily extended beyond the range of data. Some data will plot so that a curved frequency line would better fit the points. Such a curved line is a skewed frequency line in terms of the log-normal paper.

When data plot on a curved line, one or more of the following conclusions can generally be made:

(a) The data are a poor sample.

(b) The data are affected by some physical occurrence that causes a consistent bias in the data.

(c) The data are not well represented by the log-normal distribution.

Poor samples are always possible, though they are less likely to occur as the size of the sample increases. Double-mass analysis (NEH-4, subsection 4.3.3) or the regression method (see a statistical text for details) can be used to improve the application of poor samples. The subsection on "Length of record", given below, will assist in estimating the value of a sample.

Physical occurrences that cause a bias or consistent error may be of many kinds, with the bias unaffected by length of record. A leaky rain gage or an observer who neglects to record small rains will bias rainfall data. A stream channel with high transmission losses, like many in the West, will bias runoff data by losing all of the smaller flows and part of the others. Industrial waste water flowing continually into a stream will bias runoff data in the opposite way.

The conclusion (c) above is usually justified only when a very long period of record is available, since many data are needed to show by statistical tests that conclusions (a) or (b) are not more appropriate.

426963 O - 57 - 17

The following rapid graphical procedure may be used to rectify skewed data. The procedure should be used only when field evidence shows the data are greatly biased. Referring to figure 3.18-2, take the following steps:

1. Plot the original data on log-normal paper, following the procedure given above, but draw no line through the points.

2. Add an assumed constant value to each plotted point and replot. See "Rectified data" of figure 3.18-2.

3. If the rectified values do not plot in reasonable alinement, assume a different constant and repeat step 2 until such alinement is obtained.

4. Draw a straight line through the rectified points, following the process described above. This is a preliminary line.

5. Subtract the constant value from the preliminary line. Plot the resulting points and draw a curve through them. This is the completed frequency line, from which various flood frequencies are selected for use.

It is possible for the data to plot originally with the curvature opposite to that shown on figure 3.18-2. In such cases the constant is found by trial-and-error subtraction, the preliminary line is below the completed line, and the completed line flattens out horizontally.

Note that the procedure is not precise. Much time should not be spent on the trial-and-error process, nor should the constant be given any meaning or physical importance.

Other more complex mathematical skew correction procedures are available, but are not recommended for general use. If an understanding of them is needed in order to weigh the significance of work done by those procedures, refer to:

Jarvis, Clarence S., and others. -- Floods in the United States. USGS Water-Supply Paper 771, 1936. (See paper by Thorndike Saville)

Foster, H. A. -- Theoretical Frequency Curves. Trans. Amer. Soc. Civil Engin., vol. 87. 1924.

Hazen, Allen. -- Flood Flows. John Wiley & Sons, Inc., New York. 1930.

Length of record. Both the historical and synthetic series should be checked for sufficient length of record before being used. Generally the check is either (1) for hydrologic design purposes, where assurance is needed only that the record is long enough to give reasonably close estimates; or (2) for watershed project evaluations, where assurance is needed that the record is neither too short nor too long, since a too-short record will usually give a poor estimate of project benefits, and a too-long record will increase the evaluation labor without significantly increasing the accuracy of the benefit estimates.

Figure 3.18-3 shows a relation between the slope of a frequency line on log-normal paper and the minimum years of record needed to make that line reasonably representative of a normal period. This ~~line~~ RELATION is used to obtain the checks mentioned above.

The method for estimating required sample size is based on the equation 1/:

$$t_h = \frac{(\bar{x} - m)\sqrt{n - 1}}{s} \qquad (3)$$

where t_h = "Student's" t at the h level of significance

$\bar{x}$ = mean of the sample

m = given value with which $\bar{x}$ is compared

n = number of items in the sample

s = standard deviation of the sample 2/

For the derivation of equation 3 see Cramér, "Mathematical Methods of Statistics", Sections 18.2 and 29.4. Another form 2/ of equation 3 is used in Snedecor's "Statistical Methods", Section 2.14, (4th ed.)

The percent error of the standardized mean is in terms of logarithms. A 10 percent plus or minus error at a 10 percent level of significance (Cramér, Snedecor) provides sufficient accuracy in frequency lines for Service purposes. This limit results in setting

$$m = \bar{x} - 0.1 \qquad (4)$$

Equation 3 then becomes:

$$(n - k) = (10\ s\ t_{10})^2 \qquad (5)$$

where (n-k) represents degrees of freedom (Cramér, Snedecor). The relation on log-normal paper

$$s = 0.430 \log R \qquad (6)$$

1/A comparable method using variances showed that the test based on equation 3 is sufficient.

2/Computed using the so-called "biased" method, or $\Sigma(x-\bar{x})^2/n$. The method given on tables 3.18-3, 3.18-5, and 3.18-6 is the "unbiased" method or $\Sigma(x-\bar{x})^2/(n-1)$, and the t equation is $t = (\bar{x}-m)(\sqrt{n})/s$. This is the form used by Snedecor. The t test is the same in either case. Equation 3 ~~was~~ was more convenient for the purpose here.

with logarithms to base 10, and where R is the ratio

$$R = \frac{\text{magnitude of the 100-year event}}{\text{magnitude of the 2-year event}} \qquad (7)$$

is used to obtain

$$n = (4.30\ t_{10} \log R)^2 + k \qquad (8)$$

Loss of degrees of freedom makes k = 6. Substituting y_m for n, where y_m is the minimum acceptable years of record, equation 8 becomes:

$$y_m = (4.30\ t_{10} \log R)^2 + 6 \qquad (9)$$

This is the equation of the curve plotted on figure 3.18-3. The following very important facts about the curve should be remembered:

(a) The curve is used as though it were a precise divider of acceptable and non-acceptable lengths of record. Theoretically, the further a point is below the curve, the less acceptable it is, because of the probability nature of the curve. However, the criteria used in preparing the curve are generous enough that lowering of the criteria is undesirable; that is, at R = 6, for example, where y_m = 38, if a record length of 37 years is accepted today (for being only a small percent lower!), there is no reason why 36 years should not be accepted tomorrow, 35 years the next day, then 34, and so on. It is relatively easy to lengthen a period of record by double-mass analysis, by regression methods, or by adding to the storm series and computing runoff. Therefore, lengths of record falling below the curve will be regarded as unacceptable.

(b) Minimum acceptable length of record depends on the slope of the frequency line. This is true regardless of the statements in item (a) above, or of the level of significance used, or of the mechanics of frequency line construction.

(c) The test for acceptability is made using the annual series.

(d) Equation 9 and figure 3.18-3 do not operate in reverse. They give only the minimum acceptable years of record for a given R. Once this minimum is exceeded the R will remain about the same as new data are added unless (1) the new data are biased, (2) the population from which the sample is being taken changes radically, or (3) an error exists in the original data.

(e) When data in group A have a certain R, then data in Group B, which is derived from Group A, may have an R larger or smaller than the one for Group A. Precipitation data, for example, will usually plot with flatter slopes than runoff data, so that an acceptable length of record obtained using storm rainfall may be unacceptable for the runoff from that rainfall, so that a longer period of record is needed for the runoff.

Figure 3.18-3 is used as follows for both the historical and synthetic series:

1. Plot the data on log-normal paper.

2. If the data are skewed, rectify them and use the slope of the preliminary line (see figure 3.18-2) to make the length of record test.

3. If the scatter of the plotting indicates a poor sample, or if the length of record is too short, increase the size of the sample by the use of the double-mass or regression analysis method or by adding to the storm series. Then rearrange and replot the data.

4. When the data are in reasonable alinement, draw a straight line using the method described previously.

5. Determine the ratio R from the 2-year and 100-year frequency values.

6. Use figure 3.18-3 with the computed R to find y_m. If the actual length of record is not less than y_m, the record is acceptable. If the actual length of record is less than y_m, go back to step 3 and repeat the process.

Examples. On figure 3.18-1(a), R = 3150/1150 = 2.74, with an actual record of 20 years' length. Figure 3.18-3 gives y_m = 17.4 years. The actual record is greater; therefore the frequency line has an adequate length of record.

On figure 3.18-1(b) and table 3.18-3, R = 56/30 = 1.87, and actual record length is 15 years. Figure 3.18-3 gives y_m = 10.5, and the actual length is adequate.

On figure 3.18-2 the preliminary line is used. R = 22200/9550 = 2.33, for which y_m is 14.5 years, and is exceeded in length by the actual record of 35 years. Therefore the actual length of record is acceptable.

Frequency lines. Annual series.

Computing method. This method has the advantage that any two persons using it will obtain identical frequency lines. The plotting method given above may sometimes lead to differences that are important. This method is independent of equation 1.

Table 3.18-3 shows data and the procedure. The computations were made on a calculator so that many of them did not actually need to be written. For example, the sum of squares in step 4 was obtained directly from column 3 by accumulative multiplication. The squares in column 4 need to be written down when doing the computations with a slide rule.

Table 3.18-3. Computation of frequency line of annual precipitation at El Dorado, Kansas.

1 Year	2 P (inches)	3 log P = X	4 $(\log P)^2 = X^2$
1905	38.1	1.581	2.500
06	39.0	1.591	2.531
07	31.8	1.502	2.256
08	39.7	1.599	2.557
09	30.2	1.480	2.190
1910	24.0	1.380	1.904
11	35.5	1.550	2.402
12	30.9	1.490	2.220
13	22.7	1.356	1.839
14	23.0	1.362	1.855
1915	45.4	1.657	2.746
16	30.4	1.483	2.199
17	17.9	1.253	1.570
18	33.8	1.529	2.338
1919	22.1	1.344	1.806

Step

1. Number of items = N = 15.

The Standard Deviation is abbreviated "s" in the table although the definition on page 5.1-10 give it as "s.d.".

2. Sum of X's = S(X) = 22.157

3. Mean = $\frac{S(X)}{N} = \bar{X} = \frac{22.157}{15} = 1.477$

4. Sum of squares = $S(X^2) = 32.913$

5. Squared sum = $[S(X)]^2 = (22.157)^2 = 490.993$

6. Correction for sum of squares = $\frac{[S(X)]^2}{N} = \frac{490.933}{15} = 32.729$

7. Sum of squared deviations = $S(X^2) - \frac{[S(X)]^2}{N} = S(d^2) = 32.913 - 32.729 = 0.184$

8. Variance = $\frac{S(d^2)}{N - 1} = (s^2) = \frac{0.184}{14} = 0.0131$

9. Standard deviation = $\sqrt{s^2} = (s) = \sqrt{0.0131} = 0.114$

10. Mean plus (s) = 1.477 + 0.114 = 1.591

11. Mean minus (s) = 1.477 - 0.114 = 1.363

12. Antilog mean = antilog 1.477 = 30.0". Plot on 50% chance line as shown on figure 3.18-1(b).

13. Antilog mean plus (s) = antilog 1.591 = 39.0". Plot on 15.9% chance line.

14. Antilog mean minus (s) = antilog 1.363 = 23.1". Plot on 84.1% chance line.

15. Draw straight line through plotted points as shown on figure 3.18-1(b).

After step 15, the computed line is tested for length of record before it is used. Quite often the data are also plotted in order to see how well the computed line fits the data.

Use of logarithms. The common method of writing logarithms of numbers between 0 and 1 (e.g., 9.834-10 as the logarithm of 0.683) leads to clumsy operations on a calculator. Instead, either the negative logarithms should be used, or all of the data to be computed should be multiplied by 10 (or 100 or 1000, etc.) to make them greater than 1. The multiple is removed after the computations. Only one multiple is used for a set of computations.

Table 3.18-4 gives the positive or negative logarithms of a sample series of numbers.

Table 3.18-4. Sample series of logarithms

Number	Positive logarithm	Negative logarithm
100.0	2.000	
10.0	1.000	
1.01	0.004	
1	0	0
.990		- 0.004
.101		- 0.996
.100		- 1.000
.099		- 1.004
.010		- 2.000
.0099		- 2.004

The negative logarithm of a number may be obtained from a table of logarithms by taking the reciprocal of the number, finding the logarithm of the reciprocal, and placing a negative sign before the figure. For example, the negative logarithm of 0.101 is obtained as follows:

Number:	0.101
Reciprocal:	9.901 = 1 ÷ 0.101
Characteristic of reciprocal:	0
Mantissa of reciprocal:	.996 (from log table)
Negative log of 0.101:	-0.996

Negative logarithms can be obtained very rapidly on slide rules with L and CI scales. Set the slide with the indexes in line; find the number on the CI scale; read the mantissa of negative logarithm on the L scale. The characteristics are obtained mentally, using table 3.18-4 as a guide.

Sample computations showing the use of negative logarithms are given in table 3.18-5.

A <u>multiple</u> is usually a multiple of 10. For example, to make each item of the original data of table 3.18-5 all greater than 1, a multiple of 1000 could be used. Then the data and their logs would appear as shown in table 3.18-6. <u>The multiple can be easily removed only at the end of certain computations</u>. Referring to the steps of table 3.18-3, the multiple is easy to remove in steps 2, 3, 10, 11, 12, 13, and 14; the results in steps 7, 8, and 9 have the multiple already removed; and the multiple is difficult to remove from the results of steps 4, 5, and 6. In practice, the multiple would be removed by division of the antilogs of steps 12, 13, and 14. For example, if a multiple of 100 had been used throughout, step 12 would read:

Antilog mean = antilog 3.477 = 3000";

and division by the multiple gives

3000/100 = 30.0". Plot on 50% chance line.

<u>Frequency lines. Partial duration series.</u>

Using flood peaks as an example, common practice is to pick all flood peaks above a base from the runoff records, place them in order of size, find a plotting position (F_{pd}) by the equation (also see equation 2):

$$F_{pd} = \frac{2\,y}{2n - 1} \qquad (10)$$

plot the floods on log paper, and fit a curved line to the points by eye. An equally good or better partial duration line can be computed from the annual flood line when the flood line is prepared by either of the two preceding methods. The data for Boulder Creek (table 3.18-1) will be used to illustrate the method. The steps are:

1. Prepare the annual flood line by either of the two methods given above.

2. From the plotted annual flood line get the peak values for selected F_a values. See columns 1 and 2 of table 3.18-7.

3. Convert F_a values to F_{pd} values, using table 3.18-8, interpolating if necessary. Estimate the average number of floods per year equal to or greater than the smallest annual flood in the watershed.

Table 3.18-5. Use of positive and negative logarithms to get standard deviation.

Original data	Log 1/	Square 1/ $= (\log)^2$
23.84	1.378	1.90
.683	- 0.167	.028
.0042	- 2.377	5.64
9.75	0.989	.978
6.02	0.780	.609
	0.603	9.155

1/ By slide rule.

1. Number of items = 5
2. Sum = 0.603
3. Mean = 0.603/5 = 0.121
4. Sum of squares = 9.155
5. Squared sum = $(0.603)^2 = 0.364$
6. Correction for sum of squares = 0.364/5 = 0.073
7. Squared deviation = 9.155 - 0.073 = 9.082
8. Variance = 9.082/4 = 2.270
9. Standard deviation = $\sqrt{2.270} = 1.507$

Table 3.18-6. Use of a multiple to get standard deviation.

Original data	Original data times 1000	Logs 1/	2/Square = $(\log)^2$
23.84	23840	4.378	19.167
.683	683	2.834	8.032
.0042	4.2	0.624	0.389
9.75	9750	3.989	15.912
6.02	6020	3.780	14.288
			57.788

1/ By slide rule.
2/ By calculating machine.

1. Number of items = 5
2. Sum = 15.605
3. Mean = 15.605/5 = 3.121
4. Sum of squares = 57.788
5. Squared sum = $(15.605)^2$ = 243.516
6. Correction for sum of squares = 243.516/5 = 48.703
7. Squared deviation = 57.788 - 48.703 = 9.085
8. Variance = 9.085/4 = 2.271
9. Standard deviation = $\sqrt{2.271}$ = 1.507

(Note: Multiple removed ~~on~~ in step 7 and remains out in steps 8 and 9, as mentioned in text.)

This information can be obtained from streamflow records of a watershed, or from those of a nearby gaged watershed if available. The average number for mixed-cover watersheds is closely related to the average number of "excessive storms" per year. Yarnell's publication, U.S.D.A. Misc. Pub. 204, pages 62-67, gives excessive storm data that are suitable for Northern States, but which are underestimates for the Southern States. See Yarnell's text for the difference in "excessive storm" criteria.

4. Plot the discharges versus their F_{pd} values on the sheet of log paper and draw in the partial duration series frequency line.

A comparison with the annual flood frequency line will show that important differences are for frequencies between zero and 2 years, with lesser differences for frequencies between 2 and 10 years, and no important differences (except plotting and similar errors) for those between 10 and 100 years.

Another conversion method is the Langbein method (Langbein, W. B., "Annual floods and the partial duration flood series", Trans. Amer. Geophys. Union, Dec. 1949). This conversion is only for about 2.5 floods per year, which should be remembered when comparisons between methods are made.

Synthetic series. This series is made using a historical series, or is developed from frequency line characteristics determined by some form of regional analysis. Since the hydrologist seldom determines which series will be used in an evaluation, this subsection presents only the data, conditions, and methods usually used for preparation of a synthetic series, without discussing relative merits of the various series.

The synthetic series is seldom used where data are available for a historic series at each evaluation point in a watershed. Where such data are available at one or more (but not all) evaluation points, the data may be used in a regional analysis to develop a synthetic series at the remaining points. When no historical data are available for the watershed, they are either transposed or regional analysis is used. Transposition is generally used with precipitation data, since they are more easily transposed than runoff data. Regional analysis is generally used with runoff data when such data are available, and the work involving precipitation is eliminated.

In the following, it will be assumed that runoff data are available either from gages, by computation from precipitation, or by transposition, and that a synthetic series is to be prepared.

Table 3.18-7. Conversion of annual flood frequencies to partial duration frequencies, Boulder Creek near Leonia, Idaho.

(1)	(2)	(3)
F_a	cfs	F_{pd}
1	3150	99.2
2	2800	49.2
4	2470	24.2
7	2200	13.5
10	2010	9.26
20	1670	4.26
33	1400	2.26
50	1150	1.26
60	1030	.93
70	920	.69
80	800	.51
90	660	.37
95	565	.31
98	475	.28
99	420	.27

F_{pd} for $\frac{N}{y}$ = 3.7 obtained by interpolation on table 3.18-8.

Table 3.18-8. Conversion of annual flood plotting positions to partial duration plotting positions for selected values of T/y.

F_{pd} for given T/y

F_a \ T/y	1.0	1.2	1.4	1.6	2.0	2.5	3.0	3.5	4.0	6.0	10.0	20.0
1	100.	99.8	99.7	99.6	99.5	99.4	99.3	99.3	99.2	99.2	99.1	99.0
2	50.0	49.8	49.7	49.6	49.5	49.4	49.3	49.3	49.2	49.2	49.1	49.0
4	25.0	24.8	24.7	24.6	24.5	24.4	24.3	24.3	24.2	24.2	24.1	24.0
7	14.3	14.1	14.0	13.9	13.8	13.7	13.7	13.6	13.5	13.5	13.4	13.3
10	10.0	9.83	9.72	9.62	9.50	9.40	9.33	9.29	9.25	9.17	9.10	9.05
20	5.00	4.83	4.72	4.62	4.50	4.40	4.33	4.29	4.25	4.17	4.10	4.05
33	3.00	2.83	2.72	2.62	2.50	2.40	2.33	2.29	2.25	2.17	2.10	2.05
50	2.00	1.83	1.72	1.62	1.50	1.40	1.33	1.29	1.25	1.17	1.10	1.05
60	1.67	1.50	1.38	1.30	1.17	1.07	1.00	.96	.92	.84	.77	.72
70	1.43	1.26	1.14	1.06	.93	.83	.76	.72	.68	.60	.53	.48
80	1.25	1.08	.96	.88	.75	.65	.58	.54	.50	.42	.35	.30
90	1.11	.94	.82	.74	.61	.51	.44	.40	.36	.28	.21	.16
95	1.05	.88	.76	.68	.55	.45	.38	.34	.30	.22	.15	.10
98	1.02	.85	.74	.64	.52	.42	.35	.31	.27	.19	.12	.07
99	1.01	.84	.72	.63	.51	.41	.34	.30	.26	.18	.11	.06

The equation for the conversion is $F_{pd} = \frac{100}{F_a} - (1 - \frac{y}{T})$, where T is the number of floods in the partial duration series at and above the lowest.
Then T/y is the average number of floods per year.

Table 3.18-9, columns 1 and 2, shows a historical series of damaging flood peaks at a given cross section and the years in which they occurred. There were no damaging floods at this section for years not shown. It would be helpful to know the annual floods for the missing years in order to increase the accuracy of the frequency line, but the line can be constructed with the given data. A synthetic series for a 50-year period is required. The procedure is as follows:

1. Select the annual floods and tabulate in order of size (column 4).

2. Compute the period of record. In this case, the evaluation is being made in 1950; therefore the period of record is

$$1950 - 1923 + 1 = 28 \text{ years.}$$

3. Tabulate F_a values for $y = 28$, using equation 1 or table 3.18-2, going only as far as the data column requires.

4. Plot the floods on log-normal paper and determine the frequency line as previously described.

5. Test the line for adequacy of length of record, using figure 3.18-3. In this case,

$$R = \frac{6150}{1600} = 3.84$$

and figure 3.18-3 shows $y_m = 25$, so that the 28-year record is adequate.

6. The annual-series frequency line is converted to a partial-duration line by the method described above, if the economist wants to use more than one flood per time unit. The remaining steps apply to both the annual- and partial-duration series.

7. Either of two methods of presenting the synthetic series may be used. Table 3.18-10 gives the first, with the development of the synthetic series shown. This type of table can be shortened in the lower portion by letting one peak represent 5 or 10, and multiplying the damage for this peak by 5 or 10, as the case may be. The second method is given on table 3.18-11; its development is shown, and the resulting damage curve is shown on figure 3.18-5. A discharge-damage curve, which is not shown, was used to convert the peaks of column 3 into the dollars damage of column 4. The damage computations are also shown in figure 3.18-5.

Table 3.18-9. Preparation of synthetic series for watershed project evaluation.

Year	Peak	n	Annual flood	28-year period F_a
	(cfs)		(cfs)	
1923	4450	1	5150	1.8
	3200	2	4450	5.4
1925	2700	3	3450	8.9
1926	2170	4	2950	12.5
1928	1640	5	2900	16.1
1929	3450	6	2700	19.6
	1680	7	2530	23.2
1932	2950	8	2170	26.8
1933	1800	9	1980	30.4
1934	1870	10	1930	33.9
1937	1870	11	1870	37.5
1941	2530	12	1870	41.1
1943	1980	13	1800	44.6
1944	5150	14	1640	48.2
	2020			
	1630			
1945	2900			
1946	1930			

Damage begins at 1600 cfs. Evaluations made in 1950, therefore period of record is 1950 - 1923 + 1 = 28 years.

For both methods, a set of peaks is worked up for (1) present, (2) with future land treatment measures assumed in place, and (3) with future land treatment and structural measures assumed in place, and similar tables are made. Benefits for (2) and (3) are differences between average annual damages.

The curve method (table 3.18-11 and figure 3.18-5) is probably the most rapid method of calculating synthetic series damages.

The synthetic series on figure 3.18-5 gives $9120 as the average annual damages. The historical series (taking 1923-50, inclusive, as the evaluation period) gives $9004, by calculations not shown here. The discrepancy may be due to any of several things. Sampling variation may cause the historical series to give results very much more--or less--than the synthetic series. When there is an unusually high flood in the historical series, it often is omitted so the series will better represent a normal period.

When a partial-duration series is used to get average annual damages, it is necessary to have some method of avoiding double-counting of damages. The historical series also requires such a method when more than one flood per damage season occurs. See the Economics Guide for details on this subject.

Table 3.18-10. Synthetic series -- First method of presentation and use, annual series.

Length of series = 50 years = y_s

n	F_{as} 1/	Flood 2/ (cfs)	Economic calculations Inundation 3/ (acres)	Damage 3/ ($)
1	2	5300		
2	4	4450		
3	6	3950		
4	8	3600		
5	10	3350		
6	12	3250		
7	14	3000		
8	16	2850		
9	18	2720		
10	20	2600		
11	22	2500		
12	24	2410		
13	26	2330		
14	28	2250		
15	30	2170		
16	32	2080		
etc.	etc.	etc.		

(To discharge where damage begins)

$$\text{Average annual damage} = \frac{\text{Sum of damages}}{y_s}$$

1/ For this and similar cases $F_{as} = \frac{100\ n}{y_s}$.

2/ Taken from the frequency line, for the computed F_{as}.

3/ Taken from appropriate curves by the economist.

426963 O - 57 - 18

Table 3.18-11. Synthetic series -- Second method of presentation and use, annual series.

1/ n	2/ F_{as}	3/ Flood (cfs)	4/ Damage ($)
1	2	5300	
2	4	4450	
4	8	3600	
8	16	2850	
12	24	2410	
15	30	2170	
20	40	1830	
23	46	1700	
25	50	1600 5/	

(Note: Columns 2 and 4 are used to plot the curve, figure 3.18-5, and the area under the curve gives the average annual damage, as shown there. Use of this table without plotting will result in an underestimate of damages.)

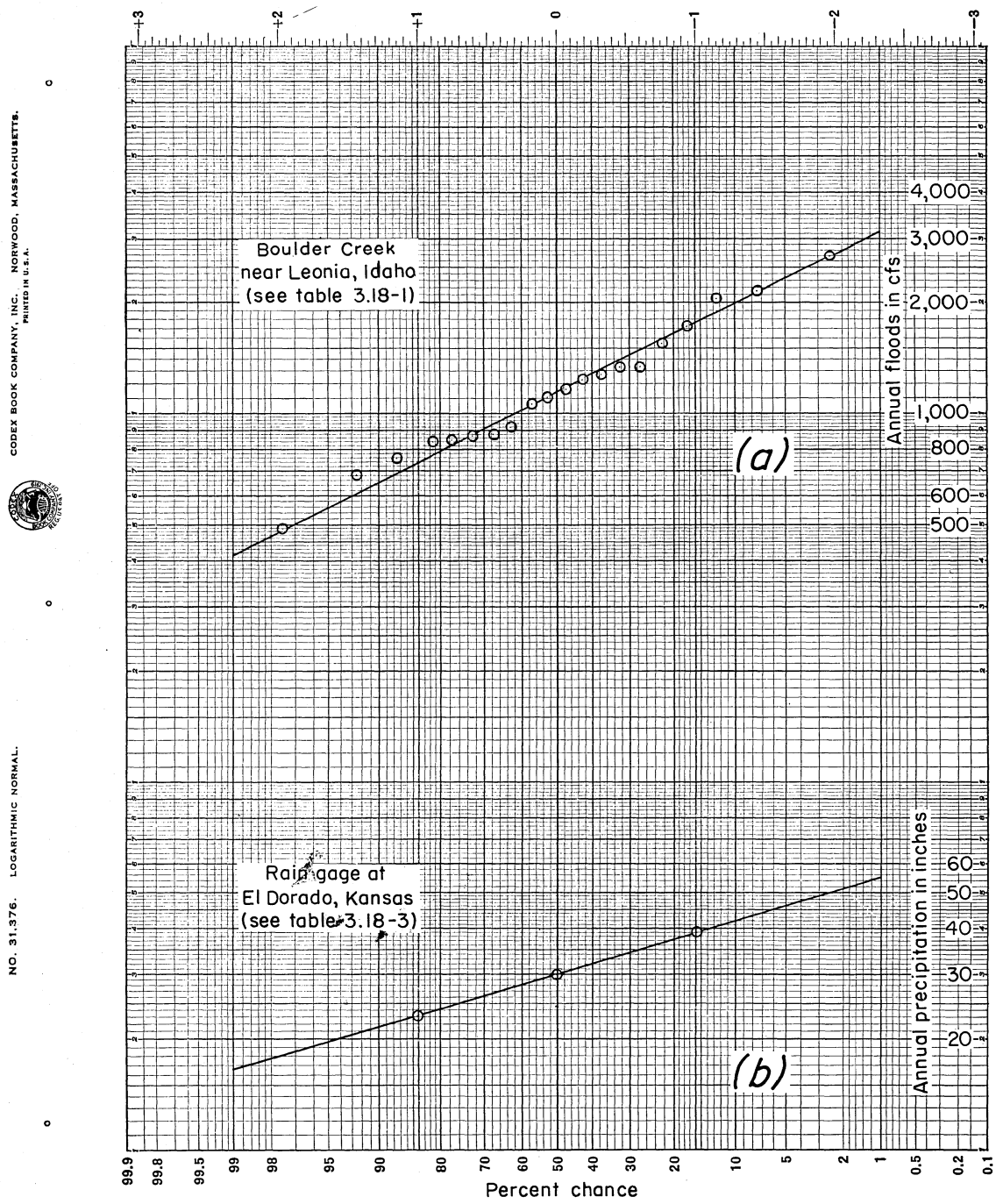
1/ Number of times the peak is equaled or exceeded during the evaluation period y_s.

2/ $F_{as} = \frac{100\ n}{y_s}$

3/ Taken from the frequency line for the computed F_{as}.

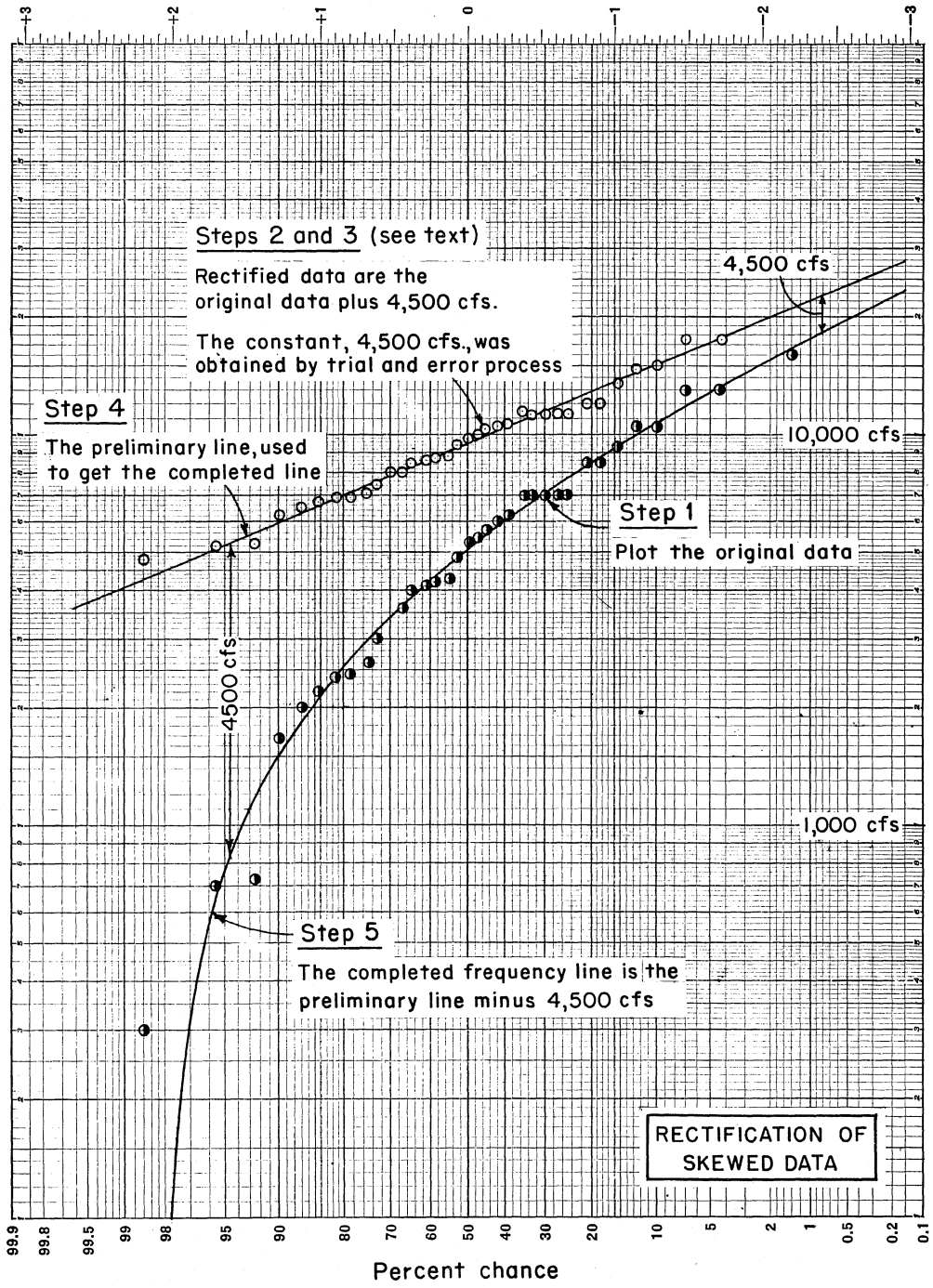
4/ Column filled in by economist.

5/ Damage begins at this discharge.



NO. 31.376. LOGARITHMIC NORMAL. CODEX BOOK COMPANY, INC. NORWOOD, MASSACHUSETTS. PRINTED IN U.S.A.

Figure 3.18-1



NO. 31.376. LOGARITHMIC NORMAL. CODEX BOOK COMPANY, INC. NORWOOD, MASSACHUSETTS. PRINTED IN U.S.A.

Figure 3.18-2

HYDROLOGY: FREQUENCY ANALYSIS - Criterion for minimum acceptable years of record

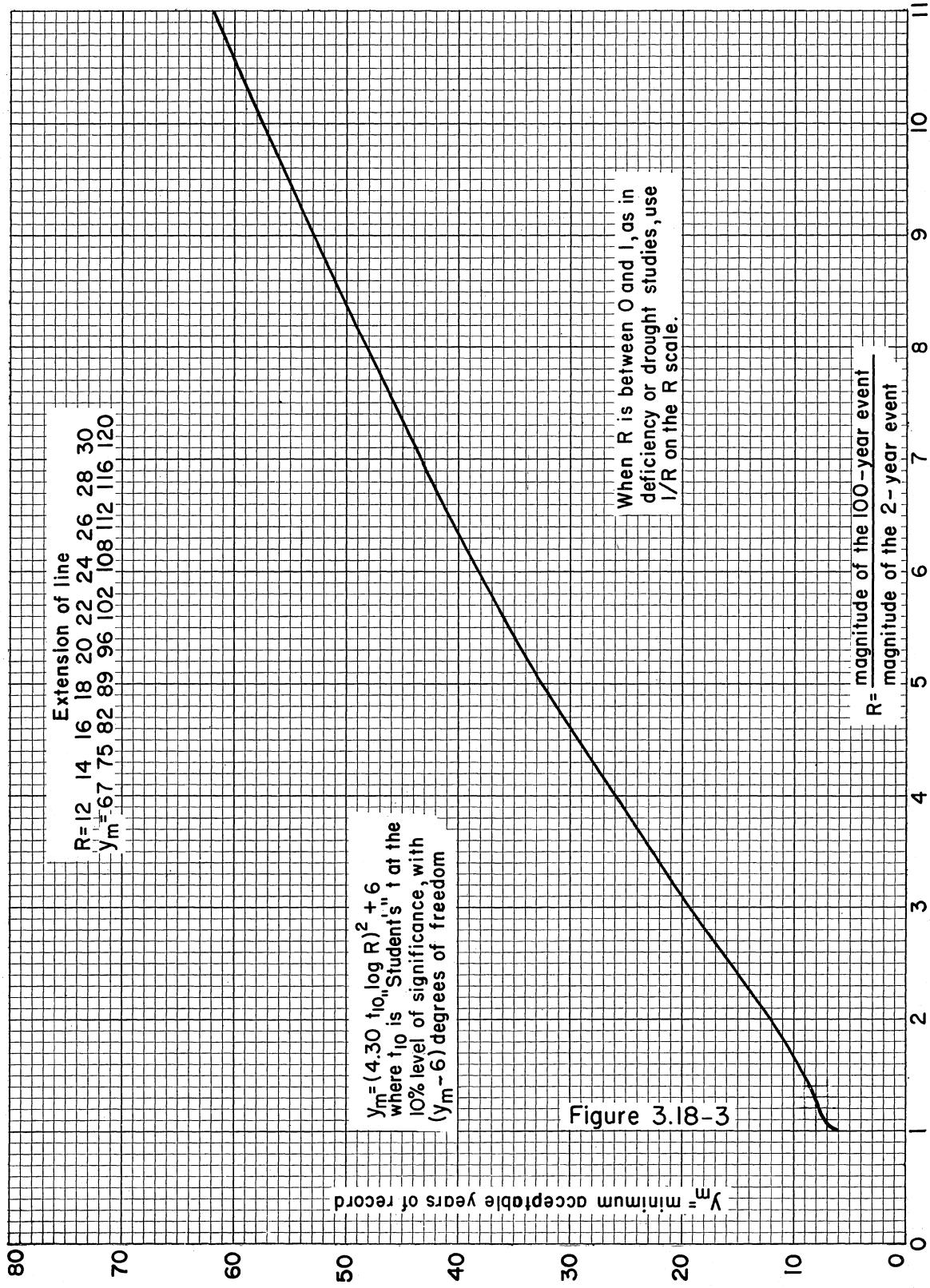


Figure 3.18-3

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - CENTRAL TECHNICAL UNIT	STANDARD DWG. NO. ES- 1005 SHEET 1 OF 1 DATE 3-14-57
Section 3.18 of the Hydrology Guide.		

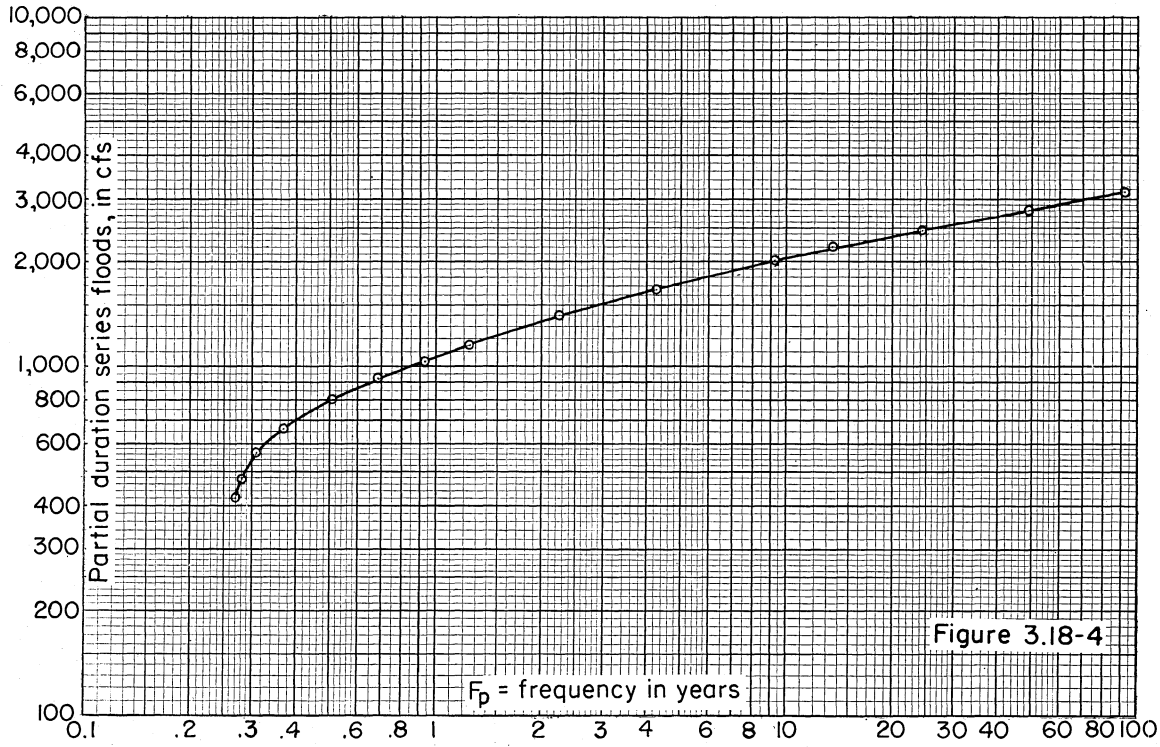
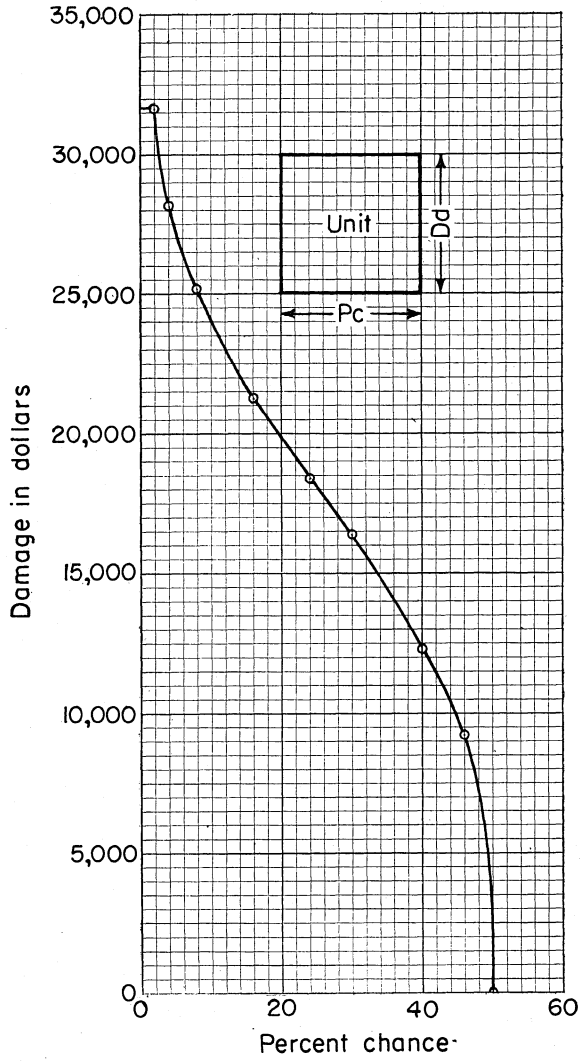


Figure 3.18-4



See table 3.18-11 for computation of curve

$$\text{Unit} = \frac{Dd \times Pc}{100^*}$$

$$= \frac{5000 \times 20}{100}$$

$$= \$1000^{\underline{00}}$$

Area under curve is 9.12 units, or $\$9120^{\underline{00}}$ average annual damages.

*This is always 100 regardless of the period of the series.

Figure 3.18-5

3.19 Transmission losses

Purpose. Whenever water flows through a normally dry stream channel, the volume of flow is reduced due to infiltration or seepage into the bed and banks of the stream. The transmission loss is recognized in irrigation canal design, and both data and procedures are available. Data for natural streams are scarce, and procedures that are available must be adapted to local conditions. This section describes the problem further and suggests procedures for practical use.

Effect on project evaluation. Transmission losses may affect watershed project evaluations in either of two ways:

(a) Indirectly, by affecting the rainfall-runoff relation. Since some of the runoff is lost on its passage to the stream-gaging station, there will be a strong tendency for runoff from a moisture condition III to appear like that of a II; that from a II to appear like that of a I; and when the condition is a I on the uplands, to make the loss so large that a moisture condition 0 may seem appropriate. Adjustment of the watershed antecedent moisture conditions does not solve the problem and tends further to complicate the problem of dealing with rainfall-runoff relations; it is necessary to recognize the transmission loss and to correct for it. Without such a correction, the rainfall-runoff relation is in error, resulting in errors in the evaluation and design of a watershed project.

(b) Directly by exaggerating the effects of an upstream measure on large watersheds. A typical instance is a farm or stock pond, which, because it stores, for example, 10 AF in its reservoir pool, is usually assumed to reduce the flow volume at a point 100 miles downstream by 10 AF. Actually, if there had been no storage, possibly only 3 or 4 of that 10 AF would have reached the downstream point. In some watersheds the transmission loss may be greater than all other losses combined, including evaporation from the pond and seepage in the pond area. In evaluating land treatment measures, the total effect of the transmission loss is similar, but since it is not so readily observed, it is usually considered negligible when actually it may be large.

Still another direct effect of transmission losses is to change the shapes of hydrographs. Watersheds with large losses usually have hydrographs that depart from the expected pattern typified by figure 3.16-23 by being steeper during the rise. Rather than prepare a typical hydrograph for these variable cases, it is better to estimate the transmission loss for each case (it may vary from storm to storm on a watershed) and to adjust the typical hydrograph based on total upland runoff by reducing the ordinates of flow during the rise. Such adjustments cannot be accurate without detailed knowledge of the amounts and distribution of the transmission loss.

Types of procedures. It is possible to set up equations for estimating transmission losses, but the constants in the equations will vary from stream to stream. The difficult part of the task is to determine those constants for a particular watershed.

Major influences on transmission losses are (1) elevation of the ground-water table, (2) character of the materials in the channel bed and banks, and (3) wetted perimeter.

The ground-water table may be taken into account when it is of importance, by dividing runoff periods into two parts, one in which the water table is high and transmission losses may be safely assumed to be negligible, and the other when the water table is low and bed and bank character governs. This division may sometimes be made using flow records since recession curves may indicate the presence or lack of a water table. Without flow records, precipitation and temperature records will be of assistance in estimating the division of runoff periods.

Channels in coarse sands and gravels will usually have large losses, continuous during the flow period. Channels in finer materials, such as silts or loams, will have a similar but lesser loss. The same equation applies to both groups, with only the constants differing:

$$Q_L = K\left(Q_T\right)^S - G \qquad (1)$$

where: Q_L = the reduced runoff volume in inches

Q_T = the original upstream runoff volume in inches

K,S,G = constants that are different for each watershed and which are estimated from flow records

The constant G in equation 1 represents heavy initial losses that take place before a more steady seepage rate prevails. Equation 3 of Section 3.12 can also be used for estimating this type of loss by using D in that equation to mean duration of streamflow at a given h.

Channels may have geologic formations within them that cause large transmission losses. Here the loss is a discontinuous one in the sense that it has no relation to the loss upstream or downstream from its location. These losses are preferably determined by the use of streamflow stations bracketing the loss area. Qualitative or visual estimates of the losses may greatly mislead because of further unknown discontinuities in the formations. A study of watersheds with similar formations will usually indicate maximum and minimum limits within which the problem watershed may be presumed to fall. Although the actual loss cannot then be known, it may be possible to use both limits (in two sets of calculations) and at least learn the possible extremes of their effects on a watershed project. However, the streamflow measurements mentioned above are necessary for more specific information.

Channels in fat, heavy clays will have very small losses unless the clays have had the opportunity to dry and crack. In that case, there may be a larger initial loss so that an equation would read:

$$Q_L = Q_T - C \qquad (2)$$

where C is the initial loss. However, losses in this kind of channel are usually neglected.

Estimation of transmission losses. Methods based on equation 1 will be discussed here. The area of application of a particular method is not defined by latitude and longitude, geologic regions, or great soil groups, since the character of bed and banks of a stream is the sole criterion and may vary widely among neighboring streams.

Figure 3.19-1 shows one form in which equation 1 may be used. Losses are expressed in relative terms to permit wider application of the loss curves. Since estimates using these curves are for average conditions, results on individual watersheds may depart widely from indicated values. Curves of the type in figure 3.19-1 are used as follows:

Given 1.22 inches of direct runoff on a 1.85-square-mile watershed, and assuming the storm has about uniform coverage over both watersheds, which have similar cover and soil,

(a) What is the direct runoff for a 12.0-square-mile watershed?

Using curve A, we read a ratio at 1.85 square miles of 0.88 and at 12 square miles of 0.61. Then the expected direct runoff for the larger watershed is

$$\frac{0.61}{0.88}(1.22) = 0.85 \text{ inches.}$$

(b) What is the transmission loss?

This is:

$$(1.22 - 0.85)\ (12.0 \text{ sq. mi.})\ (53.3) = 237 \text{ AF}$$

where 53.3 is a conversion constant (see Section 5.2).

Curve B is used in the same way. The curves may also be used to adjust for drainage area size. Curves for a specific watershed may be made when streamflow data are available, if differences in precipitation distribution and hydrologic soil-cover complexes are not large, by plotting either actual or relative runoff amounts versus drainage area. Graphs of runoff versus length of main channel may be more specific, in some cases, and are used similarly.

The graph, figure 3.19-2, is modified from methods used by the Bureau of Reclamation to estimate irrigation canal seepage. This type of plotting uses the equation:

$$q_R = k\,(A)^{0.5} \qquad (3)$$

where q_R = seepage loss in cfs per channel mile

k = a constant that is larger for more pervious materials

A = cross section of the flow in square feet.

This is a modified form of equation (1) and applies only after initial losses are satisfied and the seepage has become relatively constant for a constant flow. The initial loss must be estimated before using figure 3.19-2. Table 3.19-1 shows a sample computation in which the initial loss and transmission losses by figure 3.19-2 are computed separately, and their effects shown on a hydrograph routed through a reach.

Discussion. The methods described above should be modified by local data wherever possible. It is preferable to neglect the transmission loss where it is probably small or within the margin of error of the other estimates.

In computations of transmission losses, such as those in table 3.19-1 and those given in the text, rough approximations of various kinds must usually be made because data are lacking. These approximations can sometimes be made in two sets: (1) probable maximum, and (2) probable minimum. Then working limits can be established, one of which will be a conservative value according to the given case.

Table 3.19-1. Sample computation using figure 3.19-2 to estimate change in hydrograph due to transmission losses.

(Given: Length of channel, 3.2 miles; type of channel, coarse sand and gravel to about 6 feet then finer sands and gravels; local inflow, none.)

Time (Hrs.)	Hydrograph: At head of reach (cfs)	Hydrograph: Routed to foot of reach (cfs)	After initial loss 1/ (cfs)	Cross-section area 2/ (sq.ft.)	3/ Loss (cfs)	Modified hydrograph 4/ (cfs)
0	0	0	0	-	0	0
1	75	30	0	-	0	0
2	280	100	0	-	0	0
3	600	280	0	-	0	0
4	890	520	0	-	0	0
5	1000	740	544	180	18	526
6	920	850	850	253	22	828
7	750	830	830	248	22	808
8	560	740	740	230	21	719
etc.	etc.	etc.	etc.	etc.	etc.	etc.

1/ THE Computations of initial loss (~~L_i, in cfs hours~~):

~~C = Channel length in feet = 16,900 feet~~

~~W = Approximate average width of flow = 40 feet~~

~~I = Approximate average depth to high-seepage-rate inhibiting material = 6 feet.~~

Should also include the porosity (V) OF THE CHANNEL BED MATERIAL, WHICH IS THE RATIO OF THE VOLUME OF VOIDS TO THE TOTAL VOLUME OF THE CHANNEL BED MATERIAL DOWN TO THE HIGH-SEEPAGE-RATE INHIBITING MATERIAL.

THE GENERAL EQUATION SHOULD READ:

~~Then~~

$$L_i = \frac{CWIV}{3600} = \frac{(16,900)(40)(6)}{3600} = 1126 \text{ cfs hrs.}$$

This amount is subtracted from the hydrograph at the foot of the reach as follows: Col. 3 is accumulated to the hour that gives a total of more than 1126 cfs hours. Then subtract 1126 and show remainder at that hour:

1670 (at hour 5) - 1126 = 544

2/ Using stage-discharge and stage-cross-section area curves, and Col. 4 values as average rates in the reach.

3/ Using Col. 5 and "sandy and gravelly" curve, figure 3.19-2; multiplied by 3.2 miles.

4/ Col. 4 minus Col. 6 equals Col. 7.

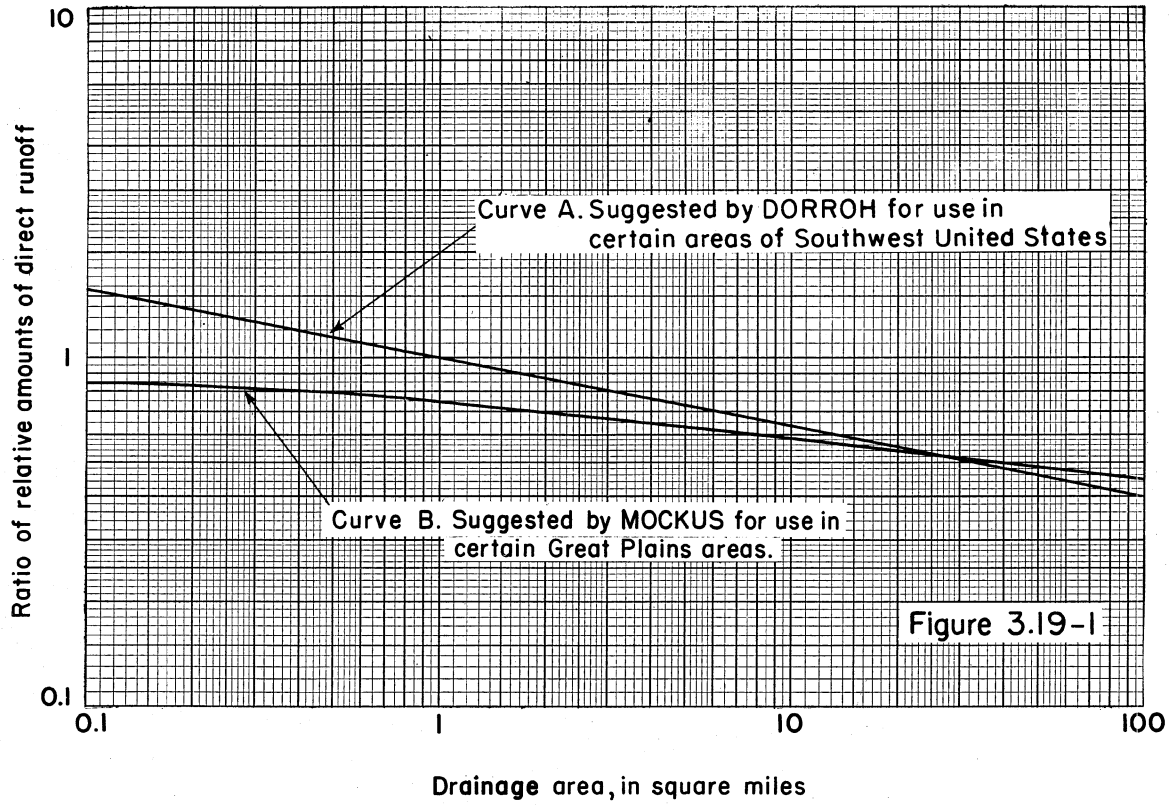


Figure 3.19-1

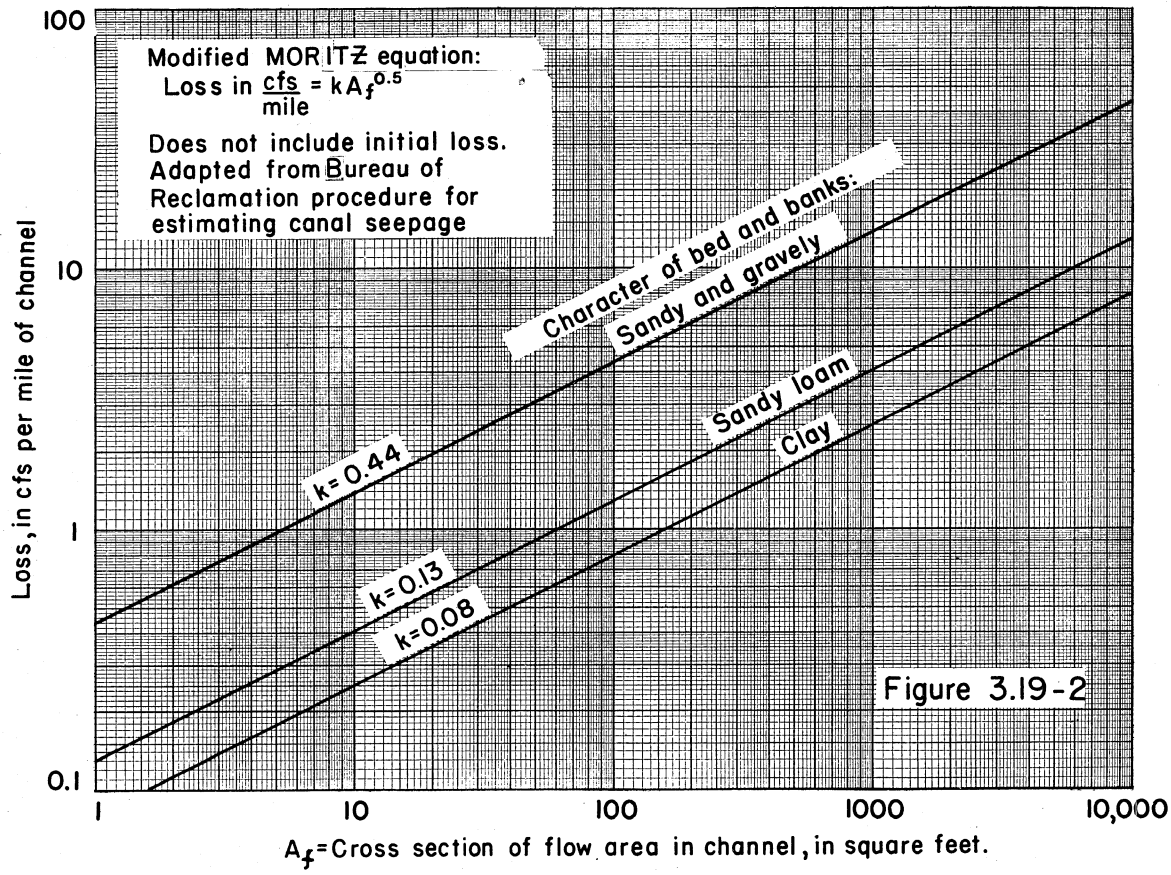


Figure 3.19-2

3.20 Watershed yield

Purpose. The water yield of a watershed, by years or seasons or months, is used in the planning and design of some watershed projects, especially those involving irrigation. The hydrologist supplies estimates of these yields, as required, or supplies methods adapted to specific local conditions by which others may make the estimates. This section contains general methods for estimating water yields on ungaged watersheds, with suggestions for such modifications as local conditions may justify.

Summary of problems. Watershed yield is dependent on many physical factors, most of which usually cannot be quantitatively determined during ordinary field operations. Methods of estimating yield from ungaged watersheds may be classified as follows:

(a) Using only climatic factors. Examples are graphs or equations using precipitation and temperature, or only precipitation.

(b) Using only geographic location. Examples are maps having lines of equal runoff, or the practice of estimating yield by interpolation between gaged watersheds.

(c) Using watershed and climatic factors. Examples are (1) water accounting method, (2) regional analysis, and (3) use of figure 3.10-1 and daily rainfall.

The choice of method often rests on the type of runoff to be estimated, which may be classified as:

(a) Yield as a residual of precipitation after evapotranspiration. Examples are watersheds where base flow predominates. Water accounting methods are useful with this type.

(b) Yield as an excess of surface supply over watershed surface intake. Examples are watersheds where surface runoff predominates. Methods using rainfall and infiltration are needed, such as a method utilizing figure 3.10-1.

(c) Yield as a diverted flow. Examples are watersheds having irrigation projects that get their supply outside of the watershed and their return flows occur inside; or watersheds with surface runoff predominating, whose streams carry return or waste flows from irrigation projects or municipal and industrial plants that pump their supplies from deep wells or receive them from outside the watershed.

Instrumentation and watershed conditions may suggest or govern the choice of method. These conditions may vary with watershed size--that is, instrumentation or methods suitable for a small watershed having surface runoff may be unsuitable for a large watershed (into which the small one drains) that has a high percent of base flow. The conditions may similarly vary with geographic location, the presence of water tables, elevation, aspect, and latitude. Other factors that have influence can also be listed. However, evaluation of the listed and unlisted factors is still more properly a research activity. In practice, the primary factors that can ordinarily be considered for ungaged streams are: (1) streamflow on nearby watersheds, (2) precipitation, (3) hydrologic soil-cover complexes, (4) evapotranspiration, (5) temperature, (6) transmission losses, and (7) base flow accretions.

Determinations of water yield will usually have two types of error, (1) that due to insufficient recognition of the natural fluctuations of yield from year to year, and (2) that due to insufficient recognition of the most important influences on yield in a given watershed. The first type of error can be reduced by working with long records, the second by further studies of all possible major influences. However, increasing the time spent on yield estimates does not always assure greater accuracy in the estimates. Therefore, the methods given below should be considered as giving estimates so broad that the influence of specific factors have large margins of error.

Methods for estimating yields. A fuller account of such methods will be given in the National Engineering Handbook, Section 4, Hydrology.

Regional analysis. The general procedure is described in Section 2.8 of the Guide. For water yield, the method is used with annual, seasonal, or monthly flows of gaged watersheds. The slopes of the frequency lines will vary, being flattest for annual yields and becoming steeper (larger R on figure 3.18-3) as smaller divisions of a year are used.

This method is suitable for estimating the first two types of runoff mentioned above. It is readily adapted to watershed conditions, when data are available, since the watersheds can be selected for whatever factors can be used. However, the factors (and not the regional analysis method) may very strongly govern the accuracy of the results for watershed yield. For example, if one of the important factors on the problem watershed is aspect, and it is too vaguely represented by the gaged watersheds used in the analysis, then the accuracy of the results of the regional analysis will suffer. Transmission losses, for example, may be insufficiently detected by this method, and additional field studies may be required to determine those losses.

Water accounting. This method is suitable for estimating the first type of runoff mentioned above. As presented here, the method is A. L. Sharp's modification and enlargement of a method proposed by C. W. Thornthwaite in Trans. Amer. Geophys. Union, pp. 686-693, April 1944. The transmission loss is not estimated by this method and must be determined by other methods (Section 3.19).

The flow chart in Section 2.10 will assist in understanding the following steps.

1. Obtain soils and land treatment data for the watershed.

2. Obtain estimates of the water-holding capacity of each soil or soil group, expressed as inches depth of water between the amounts at field capacity and wilting point. The soil depth for which this capacity is needed is the depth of the intensive root zone, or 3 feet, whichever is lesser.

3. Compute the water-holding capacity of the watershed, weighting by areal extent of the soils or soil groups.

4. Obtain watershed cover data for the season or seasons for which yields are to be estimated. Data needed are (1) types of cover, and (2) areal extent.

5. Compute potential evapotranspiration (potential ET), or consumptive use by months for each major crop or land use. The Blaney-Criddle method of computing potential ET is generally used as given in "Determining Water Requirements in Irrigated Areas from Climatological and Irrigation Data", by Harry F. Blaney and Wayne D. Criddle, Soil Conservation Service, U.S.D.A., SCS-TP-96, Washington, D. C., revised 1952.

6. Compute monthly weighted potential ET for the watershed.

7. Obtain monthly rainfall data for the watershed, for a period of years estimated to be long enough to give adequate yield values (see Section 3.18 on length of record). The estimate of length should be made after previous use of figure 3.18-3 with other yield data in the vicinity.

8. Compute average rainfall over the watershed, by months, for each year of record.

9. Tabulate rainfall and ET data as shown on table 3.20-1, and compute runoff, by months, for each year of record.

Table 3.20-1. Sample computation by water accounting method

Line	Item	October	November	December	January	February	March	April	May	Seasonal runoff
	All units in inches				1947 - 1948					
1	1/Average rainfall	5.65	1.04	1.88	2.41	2.34	5.48	10.04	1.34	
2	2/Initial soil moisture	0.00 3/	2.87	1.74	2.62	3.20	3.20	3.20	3.20	
3	Total available moisture	5.65	3.91	3.62	5.03	5.54	8.68	13.24	4.54	
4	4/Potential evapotranspiration	2.78	2.17	1.00	0.90	1.00	2.69	3.18	3.89	
5	5/Actual evapotranspiration	2.78	2.17	1.00	0.90	1.00	2.69	3.18	3.89	
6	Remaining available moisture	2.87	1.74	2.62	4.13	4.54	5.99	10.06	0.65	
7	6/Final soil moisture	2.87	1.74	2.62	3.20	3.20	3.20	3.20	0.65	
8	Runoff	0.00	0.00	0.00	0.93	1.34	2.79	6.86	0.00	11.92
					1948 - 1949					
1	1/Average rainfall	0.75	0.84	3.53	1.24	2.22	7.34	0.03	0.46	
2	2/Initial soil moisture	0.00 3/	0.00	0.00	2.53	2.87	3.20	3.20	0.05	
3	Total available moisture	0.75	0.84	3.53	3.77	5.09	10.54	3.23	0.51	
4	4/Potential evapotranspiration	2.78	2.17	1.00	0.90	1.00	2.69	3.18	3.89	
5	5/Actual evapotranspiration	0.75	0.84	1.00	0.90	1.00	2.69	3.18	0.51	
6	Remaining available moisture	0.00	0.00	2.53	2.87	4.09	7.85	0.05	0.00	
7	6/Final soil moisture	0.00	0.00	2.53	2.87	3.20	3.20	0.05	0.00	
8	Runoff	0.00	0.00	0.00	0.00	0.89	4.65	0.00	0.00	5.54

1/ Average over the watershed for each month of record.
2/ At start of month. Same as "Final soil moisture" for previous month.
3/ See text, Step 9, notes (a) and (b).
4/ Average annual values for the month.
5/ Total available moisture, or potential ET, whichever is smaller.
6/ At end of month. Same as "Initial soil moisture" for next month. This is never larger than the water-holding capacity determined in Step 3 of the text--in this case, 3.20 inches.

Note: Data are for a West Coast area of the United States, where the June-September precipitation is negligible.

(a) In table 3.20-1, the computation starts with a month when available soil moisture is fully depleted. It could start equally well with a month when the soils are fully saturated.

(b) If there is a break in the year, as in table 3.20-1, the first month after the break should have either of the moisture conditions given in (a) above.

(c) When the precipitation is snowfall, convert to water equivalent (watershed average) before using in line 1 (see Section 3.11 for methods). Watersheds consistently having snowfall on one portion and rainfall on the other should be subdivided and the yields of the subdivisions computed separately, then combined for total watershed yield.

(d) Work with subdivisions if the watershed soils differ in water-holding capacities by more than about 100% of the smallest capacity or by more than about 1 inch, whichever is greater.

(e) Work with subdivisions if the watershed precipitation consistently varies widely in amount at different localities. This may be determined using average annual precipitation. The variation over a watershed (or subdivision) should not be greater than about 30% of the smallest value, or about 3 inches, whichever is greater.

10. After completion of the computations for the selected length of record, test the runoff estimates for adequacy of length of record, using the method of Section 3.18. The test should be made with values that will be used in planning or design. For example, if annual values are to be used, then they are tested; if monthly values are to be used, then all October values are tested separately, next all November, and so on. If the length of record is not adequate, additional years of precipitation are added and the yield computations extended.

Transmission losses are subtracted after Step 10. If these losses are proportionately large, it may be necessary to test the modified yields for adequacy of length of record.

Direct runoff method. Daily rainfall values and figure 3.10-1 can be used to estimate yields when these are of the second type described above. Generally it may be assumed that direct runoff is being estimated. The procedure consists of using the method of Section 3.10 with all rainfalls. Snowmelt runoff is estimated separately using the methods of Section 3.11.

Table 3.9-1, which is used to determine curve numbers on figure 3.10-1, gives average values for the year. In using this table for yield estimates it is usually necessary to go into more detail about the cover, so that the weighted hydrologic soil-cover complex number varies not only for antecedent moisture conditions but also for the variation in cover throughout a given year and from year to year.

426963 O - 57 - 19

The direct runoff method is usually very tedious, since all daily precipitation in a long period of record must be accounted for, day by day, using soil-cover complex numbers that vary from month to month or even oftener. The laboriousness of the procedure, however, does not guarantee close accuracy in the yield estimate.

Major errors with this method will generally be in the determinations of soil-cover complexes (which will vary through the year) and in antecedent moisture conditions (which will vary not only with precipitation and temperature, but also with soil-cover complexes.) This method is more suitable for small watersheds than for large ones, since the large watersheds will have some base flow, which may be a significant proportion of total yield. Estimates by this method generally will have such a margin of error that the effects of individual factors should not be given much significance.

Climatic and geographic factors. In areas where there is no abrupt change in precipitation, hydrologic soil-cover complexes, or geology, yield may be readily estimated using maps with lines of equal runoff. Generalized national maps, such as Plate 1 of U.S.G.S. Circular 52, should be used with great caution. The text of the Circular, page 9, states that "Figure 2 and plate 1 should not be used to estimate runoff from ungaged areas". More localized maps, however, such as those prepared by John H. Dorroh, Jr. for the Southwestern States, will be very useful, especially where the advice of the map's originator may be sought.

K. M. Kent has used a form of the "direct runoff method" described above to prepare typical yield frequency lines for selected soil-cover complex numbers, which are used with a state map giving precipitation indices. Given the soil-cover complex number, the yield for a given frequency is quickly estimated for any locality in that state.

Graphs and equations of precipitation and temperature, or precipitation alone, have been used in the past much more than they are today. Figure 2 of U.S.G.S. Circular 52 is an example (but see remark above about Plate 1). Such graphs and equations should be used with great caution since so many factors are ignored.

Discussion. Since so many factors enter into the estimating of yields, and since both the relative importance and quantitative influences of some factors are nearly always unknown, estimates of yield should be conservative, according to the use they will have. The planners and designers who will use the yield estimates will be best able to state the direction and degree of conservativeness required. The hydrologist can obtain the conservativeness by the use of the methods given above, and those in Section 3.18, Frequency methods.

3.21 Design hydrographs

Purpose. This section gives data and procedures for preparing the design hydrographs needed in determining emergency spillway capacities that meet the criteria given in Engineering Memorandum No. 3.

Summary of method. The following data for the watershed above the structure are always needed:

(a) Geographic location
(b) Drainage area
(c) Hydrologic soil-cover complex
(d) Time of concentration

The usual method by which the design hydrograph is obtained is then as follows:

1. Take a 6-hour point rainfall amount from figure 3.21-1, 3.21-2, or 3.21-3 for the appropriate geographical location of the structure.

2. Modify the 6-hour point rainfall amount for size of drainage area above the structure, using one of the curves of figure 3.21-4.

3. Tabulate accumulated storm rainfall, using one of the curves of figure 3.21-5.

4. Determine the hydrologic soil-cover complex number of the watershed above the structure.

5. Select a runoff curve (figure 3.10-1) for an antecedent moisture condition, using the criteria of Engineering Memorandum No. 3, and table 3.10-1.

6. Estimate accumulated direct runoff, using the accumulated rainfall of step 3 and the selected runoff curve of step 5 in figure 3.10-1.

7. Obtain increments of direct runoff for uniform time intervals from the accumulated runoff of step 6.

8. Compute the time to peak (T_p) and base time (T_b) for the watershed above the structure, using the selected uniform time interval of step 7, the time of concentration, and equations 16 and 19 of Section 3.16.

9. Compute the peak rate for a triangular hydrograph for each runoff increment of step 7, using equation 11 of Section 3.16.

10. Plot the incremental triangular hydrographs on plain coordinate paper.

11. Add the plotted hydrographs of step 10 and obtain a composite hydrograph. This is the design inflow hydrograph for use in flood routing to determine the required capacity of the emergency spillway less freeboard.

Storm criteria. Figures 3.21-1, 3.21-2, and 3.21-3 were prepared using maximum rainfall amounts measured at Weather Bureau-type rain gages in the United States and Puerto Rico. Published records of maxima of about 8,000 rainfall stations were examined. "Bucket survey" data were not used.

The maximum rainfall amounts for various durations were brought to a 6-hour equivalent using the empirical equation:

$$P_6 = P_D \, (6/D)^{0.4} \qquad (1)$$

where P_6 = equivalent amount for 6 hours

P_D = amount caught in D hours

D = time in hours.

The maximum 6-hour equivalents were plotted on maps and used in one of three ways:

(1) For figure 3.21-1, distance from storm source influenced the drawing of isohyets enveloping the maximum plotted values.

(2) For figure 3.21-2, the 6-hour equivalents were plotted on the Service map, "Problem Areas in Soil Conservation"; zones of equal expected rainfall amounts were delineated; and the maximum amount experienced in each zone was used for the entire zone.

(3) For figure 3.21-3, elevation was the main factor in locating isohyetals.

Figure 3.21-4 was prepared for use with design storms only. The "Sharp" curve was recommended by A. L. Sharp for certain areas on the West Coast of the United States. The "Fletcher" curve for the average case was prepared from an equation in the paper by Robert D. Fletcher, "Rainfall depth-area-duration", Trans. Amer. Geophys. Union, June 1950, pages 344-348. The "Dorroh" curve was recommended by John H. Dorroh, Jr., for certain areas of southwest United States. The Engineering and Watershed Planning Unit hydrologists will recommend one of these curves for watersheds in their areas.

Figure 3.21-5 was prepared from field recommendations. The curves are intended to give the maximum experienced (or greater) amount for durations of up to 6 hours, when used with a 6-hour amount from figures 3.21-1, 3.21-2, and 3.21-3. Curve A was recommended by John H. Dorroh, Jr. for certain localities in the Southwest. Curve B is an average pattern for use in general cases. Curve C was recommended by A. L. Sharp for certain localities in the West, and Curve A_2 was recommended by R. G. Andrews for certain areas in the western Gulf States. The Engineering and Watershed Planning Unit hydrologists will recommend one of these curves for watersheds in their areas.

When comparing the storm criteria for any locality, using the above maps and graphs with storm values actually experienced in that locality, it is important to compare on a maximum basis, not on an average or frequency basis. For example, if figure 3.21-2 shows an amount of 4 inches for a certain locality, then this amount should be compared only with maximum 6-hour amounts (or equivalents computed using equation 1) in that area, and not compared with values derived by extensions of frequency lines. It is possible for gages with widely different amounts for a given frequency to have about the same 6-hour maximum.

Example. Ordinary case. A design hydrograph will be prepared using the following information:

Location. NE part of _____ County, State of _____.

Drainage area. 1.86 square miles.

Hydrologic soil-cover-complex number. For Condition III = 82.

Time of concentration. 1.25 hours.

Following the method given above, the steps are:

1. On figure 3.21-1, the 6-hour point rainfall for the NE part of _____, State of _____, is about 13.0 inches by interpolation.

2. The "Fletcher" curve of figure 3.21-4 is appropriate for the location, and with a drainage area of 1.86 square miles, that curve gives a ratio of 0.94, and

$$0.94\ (13.0) = 12.2 \text{ inches}$$

which is the design-storm rainfall for the watershed.

3. Curve B of figure 3.21-5 is appropriate for the location. This curve and the design storm rainfall of step 2 give column 2 of table 3.21-1. The cumulative rainfall in this case is tabulated at one-half hour intervals, and shown at the end of each interval. Other convenient time intervals are used with other T_c values, as shown in the following table, which can be used as a guide.

T_c (hours)	ΔD (hours)
Less than 1	0.5
1	0.5
3	1.0
6	1.0

When T_c is greater than 6 hours, the length of the design storm is increased and the ΔD should remain not longer than one-fifth of the T_c. Note that the shorter the ΔD, the better the composite hydrograph is defined.

4. The hydrologic soil-cover complex number for Condition II is 64, using table 3.9-1 and land treatment data for the watershed.

5. Condition III is required for this case. By interpolation on table 3.10-1 the III-curve is 82.

6. Using curve 82 on figure 3.10-1, and the data of column 2, table 3.21-1, accumulated direct runoff is estimated as given in column 3, table 3.21-1.

7. Compute column 4 using data in column 3.

8. Compute T_p using equation 16 of Section 3.16.

$$T_p = \frac{D}{2} + 0.6\ T_c = \frac{0.5}{2} + 0.6\ (1.25) = 1.00 \text{ hours}$$

The ΔD is used in place of D in this case. The base time (T_b) is computed using equation 19 of Section 3.16;

$$T_b = 2.67\ T_p = 2.67\ (1.00) = 2.67 \text{ hours}$$

which is rounded to 2.70 hours.

Table 3.21-1. Sample computation of design hydrograph

Watershed and storm data

A = 1.86 square miles	ΔD = 0.50 hour
T_c = 1.25 hours	T_p = 1.00 hour
III-curve number = 82	T_b = 2.70 hours
P = 13.0 inches	q_x = 898 cfs
Modified P = 12.2 inches	

Computation of subhydrograph peaks

Time (hours)	Acc. P (inches)	Acc. Q (inches)	ΔQ (inches)	898 (ΔQ) (cfs)
0	0	0		
			0	0
0.5	0.43	0		
			0.09	81
1.0	0.95	0.09		
			0.31	278
1.5	1.65	0.40		
			0.81	727
2.0	2.80	1.21		
			3.97	3560
2.5	7.32	5.18		
			1.26	1130
3.0	8.60	6.44		
			0.84	755
3.5	9.50	7.28		
			0.68	611
4.0	10.20	7.96		
			0.59	530
4.5	10.80	8.55		
			0.50	449
5.0	11.30	9.05		
			0.48	431
5.5	11.80	9.53		
			0.40	359
6.0	12.20	9.93		

9. The peak rate for 1 inch of runoff is

$$q_x = \frac{484\ A\ Q}{T_p} = \frac{(484)\ (1.86)(1)}{1.00} = 898 \text{ cfs}$$

This value is used with the Q's of column 4 of table 3.21-1 to get column 5.

10. The hydrographs for the increments of runoff in this case have the following time positions:

Hydrograph for runoff in interval (hours)	Starting time 1/ (hours)	Peak time 2/ (hours)	Ending time 3/ (hours)
0 - 0.5	0	1.0	2.7
0.5 - 1.0	.5	1.5	3.2
1.0 - 1.5	1.0	2.0	3.7
1.5 - 2.0	1.5	2.5	4.2
2.0 - 2.5	2.0	3.0	4.7
2.5 - 3.0	2.5	3.5	5.2
3.0 - 3.5	3.0	4.0	5.7
3.5 - 4.0	3.5	4.5	6.2
4.0 - 4.5	4.0	5.0	6.7
4.5 - 5.0	4.5	5.5	7.2
5.0 - 5.5	5.0	6.0	7.7
5.5 - 6.0	5.5	6.5	8.2

1/ Starting time is the time each increment of runoff begins.

2/ Peak time = starting time plus T_p.

3/ Ending time = starting time plus T_b.

Plot the triangular hydrographs as shown in figure 3.21-6.

11. Combine the plotted incremental hydrographs by adding the rates at each break point (starting, peak, and ending times of each hydrograph), and plotting the sum points as shown on figure 3.21-6. Straight lines are drawn connecting the sum points to get the completed design hydrograph, which is ready for use in routing.

The curves of figure 3.10-1 do not start at zero (except curve 100). The smaller the curve number, the more time between the start of rainfall and the start of runoff. Generally, runoff starts within a time increment, not at its beginning or end. However, if ΔD is chosen small enough, there will be only an insignificant error in the first triangular hydrograph, and a special T_p and T_b for the first runoff increment need not be used.

Complex watersheds. When the watershed above a structure has some differences between soils, or cover, or topography, the differences can be ignored (1) if they are about uniformly distributed over the watershed, or (2) if they are at the hydraulically remotest parts of the watershed.

When a difference is on a large proportion of the watershed and is concentrated in one part of the watershed, or if it is near enough to the watershed outlet to affect the hydrograph strongly, then its effect should be taken into account in the design hydrograph. This can be done by subdividing the watershed. An example is given in Section 3.16. Table 3.16-4 is the working table, and figure 3.16-9 shows the watershed subdivisions and the design hydrograph. This method can be used to prepare design hydrographs for watersheds with unusual shapes.

Large watersheds. Watersheds having a T_c greater than 6 hours require the following changes in procedure:

1. Duration of the design storm is increased.
2. The 6-hour accumulative rainfall curve is adjusted to the new duration.
3. The total storm rainfall is increased.
4. The time increments, ΔD, are made larger.

Referring to table 3.21-3 for the data of an example, the changes are:

Storm duration is made equal to T_c; for the case in table 3.21-3, 20 hours. The duration should be rounded to a convenient figure.

Six-hour ratios are proportioned as shown in columns 2 and 3 of table 3.21-3.

Storm rainfall is increased using table 3.21-2. In this case, 1.619 (5.7) = 9.25 inches.

Time increments, ΔD, are taken at 4-hour intervals, as shown on table 3.21-3. Generally the intervals should not be larger than one-fifth of the T_c except for very small watersheds, as previously discussed.

Once these changes have been made, the method is the same as previously given. Table 3.21-3 shows the computations. The hydrograph plottings are not shown.

Table 3.21-2. Constants for extension of design storm.

Duration (hours)	Constant
1	0.489
2	0.644
3	0.758
4	0.850
5	0.930
6	1.000
7	1.064
8	1.121
9	1.176
10	1.227
11	1.274
12	1.320
14	1.402
16	1.481
18	1.552
20	1.619
22	1.682
24	1.741
28	1.852
32	1.953
36	2.048

Example: Given a 6-hour storm rainfall of 11.0 inches, what is the equivalent amount for a duration of 12 hours?

From the table, at 12 hours the constant is 1.320. Then

$$1.320\ (11.0) = 14.52 \text{ inches,}$$

which is rounded to 14.5 inches.

Table 3.21-3. Sample computation of a design hydrograph for a watershed with T_c greater than 6 hours.

Watershed and storm data:

A = 100 square miles

T_c = 20.0 hours

III-curve number = 80

P = 8.5 inches (6-hour amount)

1st Adj. P = 5.7 inches (using "Fletcher" curve of figure 3.21-4)

2nd Adj. P = 9.25 inches (using table 3.21-2 to increase P for increased duration)

ΔD = 4.0 hours

T_p = 14.~~2~~ 0 hours

T_b = ~~38.0~~ 37.4 hours

q_x = 3400 60 cfs (slide rule comp.)

Computations of subhydrograph peaks

Time (hrs.)	Equivalent time on fig.3.21-5 (hours)	Ratios from fig. 3.21-5 1/ for times in Col. 2	Acc. P 2/ (ins.)	Acc. Q (ins.)	ΔQ (ins.)	3400 60 (ΔQ) (cfs)
0	0	0	0	0		
					0.06	~~200~~ 204
4	1.2	.10	.92	.06		
					2.65	~~9010~~ 9169
8	2.4	.52	4.81	2.71		
					2.29	~~7780~~ 7917
12	3.6	.79	7.31	5.00		
					1.03	~~3500~~ 3562
16	4.8	.91	8.42	6.03		
					.79	~~2690~~ 2737
20	6.0	1.00	9.25	6.82		

1/ Using "Fletcher" curve

2/ Column 3 times 9.25

A general simplified method. W. J. Owen of the Engineering and Watershed Planning Unit, Lincoln, Nebraska, developed a family of semi-dimensionless hydrographs for use in computing design hydrographs. The method described here covers a more complete range of variables and uses a different approach in developing the design hydrographs.

In most cases the hydrographs required may be computed from the data for a 6-hour storm with a time distribution according to one of the four curves on figure 3.21-5. The 6-hour storm period may be used so long as the ratio, duration of excess rainfall to time to peak of the unit hydrograph, is in the range from 1.0 to 40.0. For ratios outside this range a storm duration will have to be selected to make the ratio fall within the 1.0 to 40.0 range. The distribution of this selected storm will be assumed to be in accord with one of the four curves on figure 3.21-9. The curves on this figure are identical to those on figure 3.21-5 except that both the time scale and the depth scale are dimensionless.

Although Engineering Memorandum No. 3 (March 14, 1958) does not permit the use of durations less than 6 hours for design purposes, such durations may be useful for project evaluation hydrographs or hydrographs used for other purposes.

Computation of hydrograph for 6-hour storm distributed according to any curve on figure 3.21-5. The watershed data for this illustration are the same as the data for the example on page 3.21-3 and are listed in the heading of table 3.21-4. The Hydrograph Computation Form, of which table 3.21-4 is an example, is to be printed by the Government Printing Office and can be requisitioned through normal supply channels. The computational steps as illustrated in table 3.21-4 are as follows:

1. Enter figure 3.21-7, sheet 3, with P = 12.2 and curve 82; note that the point located by the values falls in the area for hydrograph family 1. Figure 3.21-7, sheets 1, 2 and 4, apply to storm distribution curves A, A_2 and C.

2. Enter figure 3.10-1 with P = 12.2 and curve 82; read Q = 9.93 in.

3. Compute $T_p = 0.7\ T_c = (0.7)(1.25) = 0.88$ hr.

4. Enter figure 3.21-8, sheet 3, with P = 12.2 and curve 82; read $T_o = 5.5$ hr. Sheets 1, 2 and 4, figure 3.21-8, apply to storm distribution curves A, A_2 and C.

5. Compute $T_o/T_p = 5.5/0.88 = 6.25$; enter on the form as computed T_o/T_p: Note that dimensionless hydrographs are available for T_o/T_p = 1, 1.5, 2, 3, 4, 6, 10, 16, 25 and 36. Select the value nearest to 6.25, which is 6, and enter this on the form as the T_o/T_p used. Compute the revised $T_p = T_o/6 = 0.92$ hr.

6. Compute $q_p = \dfrac{484\ A}{\text{Rev. } T_p} = \dfrac{(484)(1.86)}{0.92} = 978$ cfs.

7. Compute $Qq_p = (9.93)(978) = 9712$ cfs.

Table 3.21-4

Hydrograph Computation Form

Watershed or project _____ State _____

Structure site or subarea _____

Dr. Area 1.86 **sq. mi.** T_c 1.25 **hr.** **Runoff condition No.** III

Runoff curve No. 82 . **Storm distrib.curve** B . **Hydrograph Family No.** 1

Storm duration 6 **hr.** **Rainfall: Point** 13.0 **in.** **Area** 12.2 **in.**

Q 9.93 in. **Computed** T_p 0.88 **hr.** T_o 5.5 **hr.**

$(T_o \div T_p)$: **Computed** 6.25 ; **Used** 6 . **Revised** T_p 0.92 .

$q_p = \frac{484\ A}{Rev.T_p}$ = 978 cfs. Qq_p = 9712 cfs.

t(column) = (t/T_p) Rev. T_p. q(column) = $(q_c/q_p)\ Qq_p$

Line No.	t hours	q cfs	Line No.	t hours	q cfs	Line No.	t hours	q cfs
1	0.40	0	21	8.50	49	41		
2	0.81	29	22	8.91	29	42		
3	1.21	126	23	9.31	19	43		
4	1.62	398	24	9.72	10	44		
5	2.02	816	25	10.12	0	45		
6	2.43	1710	26			46		
7	2.83	3750	27	$Q = \frac{(\Delta t)(\Sigma q)}{645\ A}$		47		
8	3.24	4825	28	Δt = .4048		48		
9	3.64	4175	29	Σq = 29942		49		
10	4.05	3255	30	$Q = \frac{(.4048)(29942)}{(645)(1.86)}$		50		
11	4.45	2505	31	= 10.10 inches		51		
12	4.86	1960	32			52		
13	5.26	1595	33			53		
14	5.67	1350	34			54		
15	6.07	1205	35			55		
16	6.48	971	36			56		
17	6.88	583	37			57		
18	7.29	320	38			58		
19	7.69	175	39			59		
20	8.10	87	40			60		

8. From table 3.21-5 determine the page on which the dimensionless coordinates for storm curve B, hydrograph family 1 and $T_o/T_p = 6$ are tabulated. Using these tabulated coordinates, compute the t and q columns by slide rule or calculator. Note that the lines are numbered on the tables of dimensionless hydrograph coordinates and the Hydrograph Computation Form for easy reference.

9. Check the computed hydrograph as follows:

(a) Add the q column and list the total, Σq.

(b) Compute the depth of runoff reflected by the hydrograph:

$$Q(\text{hydrograph}) = \frac{(\Delta t)(\Sigma q)}{645\ A} \text{ in inches.}$$

Δt = the uniform interval at which t is recorded in the t column, in hr.

Σq = sum of discharges in cfs.

A = drainage area, sq. mi.

This check is shown on table 3.21-4.

If no significant error has been made, this computed Q will check the Q in the heading within plus or minus 2 or 3 percent. Where the hydrograph Q varies from the Q in the heading by more than 5 percent check q_p, q_pQ, discharges in the q column and the sum of the q column.

<u>Computation of hydrograph for storm of any duration distributed according to any curve on figure 3.21-9</u>. The watershed data listed on page 3.21-1 are required. Data for this example are listed in the heading of table 3.21-6.

1. List the watershed data as shown on table 3.21-6, but omit the hydrograph family, storm duration and rainfall amounts until after the completion of step 2.

2. Assume a 6-hour storm. For this case, the 6-hour point rainfall = 14.0 inches and the areal rainfall = 10.9 inches.

(a) Enter figure 321.8, sheet 3, with P = 10.9 and curve 83, read T_o = 5.5 hr.

(b) Compute $T_p = (0.7)(10.5) = 7.35$ hr.

(c) Compute $T_o/T_p = 5.5/7.35 = 0.75$. This ratio is too small to permit computation of a hydrograph based on a 6-hour storm.

3. The storm duration selected should be such that the duration of excess rainfall will be equal to or greater than 0.7 of the time of concentration. Where the time of concentration is small and T_o/T_p based on a six-hour storm is greater than 40, do not shorten the storm duration more than is required to have T_o/T_p range from 25 to 40.

Table 3.21-5. Page Numbers on which dimensionless coordinates of hydrographs are tabulated. In each case page numbers are preceded by 3.21.

Hydrograph Family No.	T_o/T_p Values									
	1	1.5	2	3	4	6	10	16	25	36
Storm Type A										
1	20	20	20	20	21	21	21	22	22	23
2	24	24	24	24	25	25	25	26	26	26 & 27
3	28	28	28	28	29	29	29	29	30	30
4	31	31	31	31	32	32	32	32	33	33
5	34	34	34	34	35	35	35	35	36	36
Storm Type A_2										
1	37	37	37	37	38	38	38	38	39	39
2	40	40	40	40	41	41	41	41	42	42
Storm Type B										
1	43	43	43	43	44	44	44	44	45	45
2	46	46	46	46	47	47	47	47	48	48
3	49	49	49	49	50	50	40	40	51	51
4	52	52	52	52	53	53	53	53 & 54	54	
5	55	55	55	55	56	56	56	56	57	
Storm Type C										
1	58	58	58	58	59	59	59	59	60	60
2	61	61	61	61	62	62	62	62	63	63
3	64	64	64	64	65	65	65	65	66	
4	67	67	67	67	68	68	68	68	69	
5	70	70	70	70	71	71	71	71		

Hydrograph Computation Form

Watershed or project _____ State _____

Structure site or subarea _____

Dr. Area 35.0 **sq. mi.** T_c 10.5 **hr.** **Runoff condition No.** III

Runoff curve No. 83 . **Storm distrib.curve** B . **Hydrograph Family No.** 1

Storm duration 12 **hr.** **Rainfall: Point** 18.5 **in.** **Areal** 14.5 **in.**

Q 12.3 in. **Computed** T_p 7.35 **hr.** T_o 11.3 **hr.**

$(T_o \div T_p)$: **Computed** 1.54 ; **Used** 1.5 . **Revised** T_p 7.53 .

$q_p = \frac{484\ A}{Rev.T_p}$ = 2250 cfs. Qq_p = 27,650 cfs.

t(column) = (t/T_p) Rev. T_p. q(column) = $(q_c/q_p)\ Qq_p$

Line No.	t hours	q cfs	Line No.	t hours	q cfs	Line No.	t hours	q cfs
1	0	0	21	48.19	0	41		
2	2.41	332	22			42		
3	4.82	3265	23			43		
4	7.23	10400	24			44		
5	9.64	19700	25			45		
6	12.05	22550	26			46		
7	14.46	19900	27	$Q = \frac{(\Delta t)(\Sigma q)}{645\ A}$		47		
8	16.87	14550	28			48		
9	19.28	9750	29	Δt =2.41 Σq =117201		49		
10	21.69	6220	30	$Q = \frac{(2.41)(117,201)}{(645)(35.0)}$		50		
11	24.10	3950	31	= 12.5 inches		51		
12	26.50	2490	32			52		
13	28.91	1575	33			53		
14	31.32	1025	34			54		
15	33.73	664	35			55		
16	36.14	415	36			56		
17	38.55	221	37			57		
18	40.96	111	38			58		
19	43.37	55	39			59		
20	45.78	28	40			60		

4. Select a 12-hour storm and record it in the heading.

   (a) 12-hour point rainfall (see table 3.21-2) = 1.32 P_6 = (1.32)(14) = 18.5 in.

   (b) Areal rainfall (Fletcher curve) = (18.5)(0.78) = 14.5 in.

5. Enter figure 3.21-7, sheet 3, with P = 14.5 and curve 83; note that this case requires hydrograph family 1.

6. Enter figure 3.10-1 with P = 14.5 and curve 83, read Q = 12.3 in.

7. Compute T_o, T_o/T_p and revised T_p:

   (a) Enter figure 3.21-10, which is plotted from table 3.10-1, with curve 83; read rainfall prior to excess rainfall = 0.4 in.

   (b) Compute the ratio $\frac{\text{rainfall prior to excess}}{\text{areal rainfall}} = \frac{0.4}{14.5} =$ 0.028.

   (c) Enter figure 3.21-9 on the vertical scale with 0.028; read 0.06 on the horizontal scale for curve B. Then, T_o = storm duration (1 - 0.06) = (12)(0.94) = 11.3 hr.

   (d) T_o/T_p = 11.3/7.35 = 1.54, use 1.5.

   (e) Rev. T_p = 11.3/1.5 = 7.53 hr.

8. Compute q_p and Qq_p:

   (a) $q_p = \frac{484\ A}{7.53} = \frac{(484)(35)}{7.53} = 2250$ cfs.

   (b) Qq_p = (12.3)(2250) = 27,650 cfs.

9. Turn to page on which t/T_p and q_c/q_p values for storm curve B, hydrograph family 1, and T_o/T_p = 1.5 are listed and compute the t and q columns.

10. Check the hydrograph for Q as shown on table 3.21-6.

<u>Computation of peak discharge for a storm of any duration distributed according to any curve on figure 3.21-9.</u> Estimation of the peak discharge requires the data shown for the following example:

Drainage area, 0.6 sq. mi.
Time of concentration, 0.4 hr.
Runoff curve, 78.
Storm duration, 4 hr.
Storm rainfall, 3.5 in.
Storm distribution curve, B.

1. Enter figure 3.21-7, sheet 3, with P = 3.5 and curve 78; read that this case requires hydrograph family 3 under rainfall curve B.

2. Enter figure 3.10-1 with P = 3.5 and curve 78; read Q = 1.52. No reduction of point rainfall to areal rainfall is taken for this small drainage area.

3. Compute T_p and q_p:

$$q_p = \frac{484\ A}{T_p} = \frac{(484)(0.6)}{0.28} = 1037 \text{ cfs.}$$

4. Compute T_o and T_o/T_p:

(a) Rainfall prior to excess for curve 78, taken from figure 3.21-10 = 0.56.

(b) Ratio $\frac{\text{rainfall prior to excess}}{\text{storm rainfall}} = \frac{0.56}{3.5} = 0.16$

(c) Enter figure 3.21-9 on the vertical scale with 0.16; read 0.27 on the horizontal scale for curve B.

(d) $T_o = (1.00 - 0.27)\ 4 \text{ hr.} = (0.73)(4) = 2.92 \text{ hr.}$

(e) $T_o/T_p = 2.92/0.28 = 10.42$.

5. Enter plot on the lower part of sheet 3, figure 3.21-13, with $T_o/T_p = 10.42$; read $q_{cp}/q_p = 0.295$.

6. Peak discharge, $q_{cp} = Qq_p \left[\frac{q_{cp}}{q_p}\right] = (1.52)(1037)(0.295)$

$= 465$ cfs.

<u>Development of basic hydrographs and other working material</u>. This method was developed so that the procedures would make use of 4 systems of dimensionless composite hydrographs and other related graphs. Each system of hydrographs is associated with one of the 4 curves of rainfall distribution, A, A_2, B and C on figure 3.21-5. In addition to the time distribution curves of rainfall, the composite hydrographs are based on the rainfall-runoff curves on figure 3.10-1, and the dimensionless unit hydrograph on figure 3.16-3.

The dimensionless, composite hydrographs and related graphs were developed as follows:

1. Curves of 6-hour rainfall versus duration of excess rainfall were plotted for rainfall curves A, A_2, B and C and runoff curves 40 to 100. These curves are shown on figure 3.21-8, sheets 1, 2, 3, and 4.

2. Dimensionless curves of accumulated excess rainfall were computed for rainfall curves A, A_2, B and C. These curves were based on storm rainfall ranging from 3 to 40 inches. The ordinate of these curves

is the ratio Q/Q_t and the abscissa the ratio t/T_o, where:

Q = accumulated excess rainfall at any selected time, inches.

Q_t = total excess rainfall, inches.

t = any selected time, hours.

T_o = duration excess rainfall, hours.

These dimensionless curves of accumulated excess rainfall were developed so that for each rainfall curve, a limited number of medial curves, representing selected ranges within the dimensionless plot could be drawn. Each medial curve is the basis for a family of dimensionless, composite hydrographs. Five such curves were selected to represent the range of conditions for each of the rainfall curves A, B and C, and two for rainfall curve A_2.

The dimensionless curves of accumulated excess rainfall are not required in the working application of the method and are therefore not included here.

3. Plots were made to show the ranges of rainfall and runoff curve numbers represented by each family of composite hydrographs. These plots are shown on figure 3.21-7. The user should note that these are area charts. Any value of rainfall between 3 and 40, associated with any curve number between 40 and 100 will define a point on the chart. The area in which the point falls on the chart determines the hydrograph family required for the case under consideration.

4. The composite hydrographs shown on figures 3.21-11, 3.21-12, 3.21-13 and 3.21-14 were computed by establishing a consistent relationship between the dimensionless time scales of the unit hydrograph and the medial curves of accumulated runoff described under the second step.

By definition, under the unit hydrograph method:

$$N\Delta D = T_o \qquad (2)$$

T_o = duration of excess rainfall

ΔD = unit period

N = number of unit periods

The primary elements of the unit hydrograph are defined by the following empirical equations from Section 3.16.

$$T_p = \frac{\Delta D}{2} + 0.6\,T_c \qquad (3)$$

$$q_p = \frac{484\,A}{\frac{\Delta D}{2} + 0.6\,T_c} \qquad (4)$$

T_p = time to peak of unit hydrograph, hr.

T_c = time of concentration, hr.

A = drainage area, sq. mi.

q_p = peak discharge of unit hydrograph, cfs.

To systematize the computations and reduce the total amount of work, the unit period was held constant at:

$$\Delta D = T_p/4 \tag{5}$$

Then, from equations (2) and (5),

$$T_o/T_p = N/4 \tag{6}$$

$$T_p = \frac{4T_o}{N} \tag{7}$$

Also, from equations (3) and (5),

$$T_p = 0.7\ T_c \quad \text{(rounded from 0.686)} \tag{8}$$

Dividing both sides of equation (7) into t, any selected time, results in:

$$\frac{t}{T_p} = \frac{t}{T_o}\left[\frac{N}{4}\right] \tag{9}$$

This equation defines the relationship between the dimensionless time scales of the unit hydrograph and the medial curves of accumulated excess rainfall. The composite hydrographs were computed by accumulating the ordinates of successive unit period hydrographs. The procedure for accumulating ordinates of unit period hydrographs is standard in the unit hydrograph method.

Examination of equation (6) shows that varying N, the number of unit periods during which excess rainfall occurs, varies the ratio T_o/T_p. When N = 4, T_o/T_p = 1; that is, The duration of excess rainfall is equal to time to the peak of the unit hydrograph. As N is increased from 4, the duration of excess rainfall is caused to increase in relation to time to the peak of the unit hydrograph. In order to span a wide range of variables, N was selected so that composite hydrographs were computed for T_o/T_p values: 1, 1.5, 2, 3, 4, 6, 10, 16, 25 and 36.

Table 3.21-7 (Sheet 1 of 52)

Hydrograph Coordinates

Storm Distribution Curve: A Hydrograph Family: 1

T_o/T_p = 1			T_o/T_p = 1.5			T_o/T_p = 2			T_o/T_p = 3		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.30	.082	2	0.32	.080	2	0.33	.073	2	0.36	.070
3	0.60	.328	3	0.64	.320	3	0.66	.286	3	0.72	.271
4	0.90	.698	4	0.96	.611	4	0.99	.544	4	1.08	.471
5	1.20	.843	5	1.28	.700	5	1.32	.617	5	1.44	.512
6	1.50	.786	6	1.60	.678	6	1.65	.585	6	1.80	.462
7	1.80	.604	7	1.92	.578	7	1.98	.518	7	2.16	.410
8	2.10	.417	8	2.24	.445	8	2.31	.450	8	2.52	.358
9	2.40	.271	9	2.56	.305	9	2.64	.359	9	2.88	.315
10	2.70	.176	10	2.88	.190	10	2.97	.247	10	3.24	.279
11	3.00	.114	11	3.20	.120	11	3.30	.158	11	3.60	.230
12	3.30	.076	12	3.52	.078	12	3.63	.100	12	3.96	.155
13	3.60	.050	13	3.84	.047	13	3.96	.060	13	4.32	.093
14	3.90	.032	14	4.16	.032	14	4.29	.038	14	4.68	.057
15	4.20	.021	15	4.48	.019	15	4.62	.023	15	5.04	.033
16	4.50	.013	16	4.80	.011	16	4.95	.014	16	5.40	.018
17	4.80	.007	17	5.12	.005	17	5.28	.009	17	5.76	.011
18	5.10	.004	18	5.44	.003	18	5.61	.005	18	6.12	.007
19	5.40	.002	19	5.76	.002	19	5.94	.003	19	6.48	.004
20	5.70	.001	20	6.08	.001	20	6.27	.002	20	6.84	.002
21	6.00	.000	21	6.40	.000	21	6.60	.001	21	7.20	.001
						22	6.93	.000	22	7.56	.000

Hydrograph Coordinates

Storm Distribution Curve: A Hydrograph Family: 1

$T_o/T_p = 4$			$T_o/T_p = 6$			$T_o/T_p = 10$			$T_o/T_p = 10$		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	34	13.86	.001
2	0.37	.061	2	0.40	.047	2	0.42	.022	35	14.28	
3	0.74	.250	3	0.80	.210	3	0.84	.136	36	14.70	.000
4	1.11	.413	4	1.20	.352	4	1.26	.267			
5	1.48	.452	5	1.60	.378	5	1.68	.300			
6	1.85	.400	6	2.00	.324	6	2.10	.262			
7	2.22	.355	7	2.40	.277	7	2.52	.222			
8	2.59	.311	8	2.80	.240	8	2.94	.189			
9	2.96	.275	9	3.20	.213	9	3.36	.164			
10	3.33	.242	10	3.60	.191	10	3.78	.147			
11	3.70	.216	11	4.00	.175	11	4.20	.132			
12	4.07	.198	12	4.40	.159	12	4.62	.121			
13	4.44	.177	13	4.80	.143	13	5.04	.113			
14	4.81	.133	14	5.20	.130	14	5.46	.106			
15	5.18	.081	15	5.60	.124	15	5.88	.101			
16	5.55	.047	16	6.00	.122	16	6.30	.096			
17	5.92	.028	17	6.40	.113	17	6.72	.091			
18	6.29	.017	18	6.80	.083	18	7.14	.086			
19	6.66	.010	19	7.20	.050	19	7.56	.081			
20	7.03	.005	20	7.60	.028	20	7.98	.076			
21	7.40	.003	21	8.00	.016	21	8.40	.072			
22	7.77	.002	22	8.40	.009	22	8.82	.071			
23	8.14	.001	23	8.80	.005	23	9.24	.070			
24	8.51	.000	24	9.20	.003	24	9.66	.070			
			25	9.60	.002	25	10.08	.070			
			26	10.00	.001	26	10.50	.064			
			27	10.40		27	10.92	.046			
			28	10.80	.000	28	11.34	.026			
						29	11.76	.014			
						30	12.18	.008			
						31	12.60	.005			
						32	13.02	.003			
						33	13.44	.002			

Table 3.21-7 (Sheet 3 of 52)

Hydrograph Coordinates

Storm Distribution Curve: A Hydrograph Family: 1

T_o/T_p = 16			T_o/T_p = 16			T_o/T_p = 25			T_o/T_p = 25		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	34	19.80	.001	1	0.00	.000	34	23.43	.028
2	0.60	.028	35	20.40		2	0.71	.025	35	24.14	.028
3	1.20	.165	36	21.00	.000	3	1.42	.123	36	24.85	.028
4	1.80	.238				4	2.13	.184	37	25.56	.024
5	2.40	.203				5	2.84	.155	38	26.27	.013
6	3.00	.163				6	3.55	.122	39	26.98	.005
7	3.60	.132				7	4.26	.097	40	27.69	.003
8	4.20	.111				8	4.97	.081	41	28.40	.002
9	4.80	.096				9	5.68	.070	42	29.11	.001
10	5.40	.086				10	6.39	.063	43	29.82	.000
11	6.00	.078				11	7.10	.057			
12	6.60	.073				12	7.81	.053			
13	7.20	.069				13	8.52	.050			
14	7.80	.066				14	9.23	.058			
15	8.40	.063				15	9.94	.046			
16	9.00	.060				16	10.65	.044			
17	9.60	.057				17	11.36	.042			
18	10.20	.055				18	12.07	.041			
19	10.80	.053				19	12.78	.040			
20	11.40	.051				20	13.49	.039			
21	12.00	.049				21	14.20	.038			
22	12.60	.047				22	14.91	.037			
23	13.20	.045				23	15.62	.036			
24	13.80	.043				24	16.33	.035			
25	14.40	.043				25	17.04	.033			
26	15.00	.043				26	17.75	.031			
27	15.60	.043				27	18.46	.030			
28	16.20	.043				28	19.17	.029			
29	16.80	.034				29	19.88	.028			
30	17.40	.014				30	20.59	.028			
31	18.00	.008				31	21.30	.028			
32	18.60	.004				32	22.01	.028			
33	19.20	.002				33	22.72	.028			

Hydrograph Coordinates

Storm Distribution Curve: A Hydrograph Family: 1

T_o/T_p = 36			T_o/T_p = 36			T_o/T_p =			T_o/T_p =		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	34	29.70	.019						
2	0.90	.025	35	30.60	.019						
3	1.80	.106	36	31.50	.019						
4	2.70	.144	37	32.40	.019						
5	3.60	.115	38	33.30	.019						
6	4.50	.087	39	34.20	.019						
7	5.40	.069	40	35.10	.019						
8	6.30	.057	41	36.00	.019						
9	7.20	.051	42	36.90	.011						
10	8.10	.047	43	37.80	.005						
11	9.00	.043	44	38.70	.002						
12	9.90	.039	45	39.60	.001						
13	10.80	.036	46	40.50	.000						
14	11.70	.033									
15	12.60	.031									
16	13.50	.030									
17	14.40	.029									
18	15.30	.028									
19	16.20	.028									
20	17.10	.027									
21	18.00	.027									
22	18.90	.026									
23	19.80	.026									
24	20.70	.025									
25	21.60	.025									
26	22.50	.024									
27	23.40	.023									
28	24.30	.022									
29	25.20	.021									
30	26.10	.020									
31	27.00	.020									
32	27.90	.019									
33	28.80	.019									

Table 3.21-7 (Sheet 5 of 52)

Hydrograph Coordinates

Storm Distribution Curve: A Hydrograph Family: 2

T_o/T_p = 1			T_o/T_p = 1.5			T_o/T_p = 2			T_o/T_p = 3		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.34	.100	2	0.35	.075	2	0.37	.069	2	0.38	.049
3	0.68	.375	3	0.70	.313	3	0.74	.290	3	0.76	.225
4	1.02	.713	4	1.05	.560	4	1.11	.503	4	1.14	.394
5	1.36	.833	5	1.40	.691	5	1.48	.578	5	1.52	.450
6	1.70	.705	6	1.75	.668	6	1.85	.565	6	1.90	.438
7	2.04	.482	7	2.10	.563	7	2.22	.521	7	2.28	.412
8	2.38	.300	8	2.45	.390	8	2.59	.435	8	2.66	.379
9	2.72	.186	9	2.80	.245	9	2.96	.294	9	3.04	.348
10	3.06	.114	10	3.15	.144	10	3.33	.178	10	3.42	.311
11	3.40	.071	11	3.50	.089	11	3.70	.106	11	3.80	.224
12	3.74	.044	12	3.85	.053	12	4.07	.062	12	4.18	.146
13	4.08	.027	13	4.20	.032	13	4.44	.035	13	4.56	.081
14	4.42	.016	14	4.55	.020	14	4.81	.020	14	4.94	.046
15	4.76	.008	15	4.90	.011	15	5.18	.012	15	5.32	.027
16	5.10	.003	16	5.25	.005	16	5.55	.006	16	5.70	.015
17	5.44	.001	17	5.60	.002	17	5.92	.003	17	6.08	.009
18	5.78	.000	18	5.95	.001	18	6.29	.001	18	6.46	.005
				6.30	.000	19	6.66	.000	19	6.84	.003
									20	7.22	.001
									21	7.60	.000

Hydrograph Coordinates

Storm Distribution Curve: A Hydrograph Family: 2

T_o/T_p = 4			T_o/T_p = 6			T_o/T_p = 10			T_o/T_p = 10		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	34	14.52	.000
2	0.39	.042	2	0.41	.027	2	0.44	.015			
3	0.78	.188	3	0.82	.140	3	0.88	.095			
4	1.17	.333	4	1.23	.261	4	1.32	.189			
5	1.56	.378	5	1.64	.297	5	1.76	.219			
6	1.95	.366	6	2.05	.283	6	2.20	.209			
7	2.34	.343	7	2.46	.263	7	2.64	.189			
8	2.73	.318	8	2.87	.242	8	3.08	.174			
9	3.12	.293	9	3.28	.226	9	3.52	.160			
10	3.51	.269	10	3.69	.209	10	3.96	.148			
11	3.90	.253	11	4.10	.196	11	4.40	.138			
12	4.29	.233	12	4.51	.184	12	4.84	.130			
13	4.68	.187	13	4.92	.174	13	5.28	.123			
14	5.07	.119	14	5.33	.165	14	5.72	.117			
15	5.46	.068	15	5.74	.157	15	6.16	.112			
16	5.85	.039	16	6.15	.149	16	6.60	.108			
17	6.24	.022	17	6.56	.134	17	7.04	.104			
18	6.63	.012	18	6.97	.087	18	7.48	.100			
19	7.02	.007	19	7.38	.051	19	7.92	.097			
20	7.41	.004	20	7.79	.028	20	8.36	.095			
21	7.80	.002	21	8.20	.015	21	8.80	.093			
22	8.19	.001	22	8.61	.008	22	9.24	.091			
23	8.58	.000	23	9.02	.005	23	9.68	.089			
			24	9.43	.003	24	10.12	.087			
			25	9.84	.002	25	10.56	.077			
			26	10.25	.001	26	11.00	.050			
			27	10.66	.000	27	11.44	.027			
						28	11.88	.015			
						29	12.32	.009			
						30	12.76	.005			
						31	13.20	.003			
						32	13.64	.002			
						33	14.08	.001			

Table 3.21-7 (Sheet 7 of 52)

Hydrograph Coordinates

Storm Distribution Curve: A Hydrograph Family: 2

T_o/T_p = 16			T_o/T_p = 25			T_o/T_p = 25			T_o/T_p = 36		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	34	25.41	.034	1	0.00	.000
2	0.67	.026	2	0.77	.020	35	26.18	.017	2	0.92	.018
3	1.34	.124	3	1.54	.086	36	26.95	.006	3	1.84	.066
4	2.01	.163	4	2.31	.120	37	27.72	.003	4	2.76	.092
5	2.68	.148	5	3.08	.112	38	28.49	.002	5	3.68	.085
6	3.35	.128	6	3.85	.094	39	29.26	.001	6	4.60	.071
7	4.02	.113	7	4.62	.082	40	30.03	.000	7	5.52	.060
8	4.69	.101	8	5.39	.073				8	6.44	.053
9	5.36	.093	9	6.16	.067				9	7.36	.049
10	6.03	.088	10	6.93	.062				10	8.28	.045
11	6.70	.084	11	7.70	.057				11	9.20	.042
12	7.37	.080	12	8.47	.054				12	10.12	.040
13	8.04	.077	13	9.24	.053				13	11.04	.039
14	8.71	.074	14	10.01	.052				14	11.96	.038
15	9.38	.071	15	10.78	.051				15	12.88	.037
16	10.05	.068	16	11.55	.050				16	13.80	.036
17	10.72	.065	17	12.32	.049				17	14.72	.035
18	11.39	.062	18	13.09	.048				18	15.64	.034
19	12.06	.060	19	13.86	.046				19	16.56	.034
20	12.73	.058	20	14.63	.045				20	17.48	.033
21	13.40	.056	21	15.40	.044				21	18.40	.033
22	14.07	.055	22	16.17	.042				22	19.32	.032
23	14.74	.055	23	16.94	.041				23	20.24	.031
24	15.41	.055	24	17.71	.039				24	21.16	.030
25	16.08	.055	25	18.48	.038				25	22.08	.029
26	16.75	.043	26	19.25	.037				26	23.00	.028
27	17.42	.014	27	20.02	.036				27	23.92	.027
28	18.09	.005	28	20.79	.035				28	24.84	.027
29	18.76	.002	29	21.56	.035				29	25.76	.026
30	19.43	.001	30	22.33	.035				30	26.68	.026
31	20.10		31	23.10	.035				31	27.60	.025
32	20.77	.000	32	23.87	.035				32	28.52	.025
			33	24.64	.035				33	29.44	.024

Hydrograph Coordinates

Storm Distribution Curve: A Hydrograph Family: 2

T_o/T_p = 36			T_o/T_p =			T_o/T_p =			T_o/T_p =		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
34	30.36	.024									
35	31.28	.024									
36	32.20	.024									
37	33.12	.024									
38	34.04	.024									
39	34.96	.024									
40	35.88	.024									
41	36.80	.016									
42	37.72	.005									
43	38.64	.002									
44	39.56	.001									
45	40.48	.000									

Table 3.21-7 (Sheet 9 of 52)

Hydrograph Coordinates

Storm Distribution Curve: A Hydrograph Family: 3

T_o/T_p = 1			T_o/T_p = 1.5			T_o/T_p = 2			T_o/T_p = 3		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.28	.049	2	0.35	.051	2	0.40	.050	2	0.44	.043
3	0.56	.201	3	0.70	.238	3	0.80	.239	3	0.88	.199
4	0.84	.480	4	1.05	.485	4	1.20	.442	4	1.32	.344
5	1.12	.728	5	1.40	.643	5	1.60	.546	5	1.76	.406
6	1.40	.840	6	1.75	.700	6	2.00	.576	6	2.20	.427
7	1.68	.762	7	2.10	.618	7	2.40	.550	7	2.64	.412
8	1.96	.580	8	2.45	.450	8	2.80	.416	8	3.08	.402
9	2.24	.406	9	2.80	.277	9	3.20	.252	9	3.52	.357
10	2.52	.269	10	3.15	.166	10	3.60	.140	10	3.96	.234
11	2.80	.180	11	3.50	.101	11	4.00	.079	11	4.40	.128
12	3.08	.120	12	3.85	.060	12	4.40	.046	12	4.84	.067
13	3.36	.081	13	4.20	.036	13	4.80	.025	13	5.28	.036
14	3.64	.054	14	4.55	.023	14	5.20	.014	14	5.72	.017
15	3.92	.035	15	4.90	.013	15	5.60	.007	15	6.16	.009
16	4.20	.023	16	5.25	.006	16	6.00	.003	16	6.60	.004
17	4.48	.015	17	5.60	.003	17	6.40	.001	17	7.04	.002
18	4.76	.009	18	5.95	.001	18	6.80	.000	18	7.48	.001
19	5.04	.005	19	6.30	.000				19	7.92	.000
20	5.32	.002	20								
21	5.60	.001	21								
22	5.88	.000	22								

Hydrograph Coordinates

Storm Distribution Curve: A Hydrograph Family: 3

T_o/T_p = 4			T_o/T_p = 6			T_o/T_p = 10			T_o/T_p = 16		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.46	.035	2	0.49	.025	2	0.70	.034	2	0.77	.022
3	0.92	.157	3	0.98	.120	3	1.40	.119	3	1.54	.081
4	1.38	.277	4	1.47	.204	4	2.10	.152	4	2.31	.103
5	1.84	.326	5	1.96	.236	5	2.80	.158	5	3.08	.104
6	2.30	.333	6	2.45	.242	6	3.50	.154	6	3.85	.102
7	2.76	.325	7	2.94	.235	7	4.20	.146	7	4.62	.099
8	3.22	.316	8	3.43	.225	8	4.90	.137	8	5.39	.095
9	3.68	.307	9	3.92	.216	9	5.60	.130	9	6.16	.091
10	4.14	.298	10	4.41	.209	10	6.30	.126	10	6.93	.086
11	4.60	.250	11	4.90	.204	11	7.00	.123	11	7.70	.082
12	5.06	.154	12	5.39	.201	12	7.70	.120	12	8.47	.080
13	5.52	.080	13	5.88	.198	13	8.40	.118	13	9.24	.079
14	5.98	.040	14	6.37	.185	14	9.10	.117	14	10.01	.078
15	6.44	.018	15	6.86	.129	15	9.80	.116	15	10.78	.077
16	6.90	.011	16	7.35	.068	16	10.50	.104	16	11.55	.076
17	7.36	.006	17	7.84	.034	17	11.20	.050	17	12.32	.075
18	7.82	.003	18	8.33	.017	18	11.90	.018	18	13.09	.074
19	8.28	.001	19	8.82	.008	19	12.60	.006	19	13.86	.073
20	8.74	.000	20	9.31	.004	20	13.30	.003	20	14.63	.072
			21	9.80	.002	21	14.00	.001	21	15.40	.071
			22	10.29	.001	22	14.70	.000	22	16.17	.070
			23	10.78	.000				23	16.94	.046
									24	17.71	.013
									25	18.48	.004
									26	19.25	.002
									27	20.02	.001
									28	20.79	.000

Table 3.21-7 (Sheet 11 of 52)

Hydrograph Coordinates

Storm Distribution Curve: A Hydrograph Family: 3

T_o/T_p = 25			T_o/T_p = 36			T_o/T_p =			T_o/T_p =		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000						
2	1.16	.030	2	1.33	.024						
3	2.32	.067	3	2.67	.049						
4	3.48	.071	4	4.00	.050						
5	4.64	.068	5	5.33	.049						
6	5.80	.065	6	6.67	.047						
7	6.96	.062	7	8.00	.045						
8	8.12	.059	8	9.33	.043						
9	9.28	.057	9	10.67	.041						
10	10.44	.055	10	12.00	.040						
11	11.60	.053	11	13.33	.039						
12	12.76	.051	12	14.67	.038						
13	13.92	.050	13	16.00	.037						
14	15.08	.049	14	17.33	.036						
15	16.24	.049	15	18.67	.035						
16	17.40	.048	16	20.00	.034						
17	18.56	.048	17	21.33	.034						
18	19.72	.047	18	22.67	.034						
19	20.88	.047	19	24.00	.034						
20	22.04	.046	20	25.33	.034						
21	23.20	.046	21	26.67	.034						
22	24.36	.046	22	28.00	.034						
23	25.52	.043	23	29.33	.033						
24	26.68	.015	24	30.67	.033						
25	27.84	.003	25	32.00	.033						
26	29.00	.001	26	33.33	.033						
27	30.16	.000	27	34.67	.032						
			28	36.00	.032						
			29	37.33	.011						
			30	38.67	.001						
			31	40.00	.000						

Hydrograph Coordinates

Storm Distribution Curve: A Hydrograph Family: 4

$T_o/T_p = 1$			$T_o/T_p = 1.5$			$T_o/T_p = 2$			$T_o/T_p = 3$		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.30	.027	2	0.32	.016	2	0.32	.012	2	0.35	.008
3	0.60	.167	3	0.64	.110	3	0.64	.073	3	0.70	.053
4	0.90	.433	4	0.96	.298	4	0.96	.209	4	1.05	.136
5	1.20	.735	5	1.28	.519	5	1.28	.370	5	1.40	.251
6	1.50	.867	6	1.60	.682	6	1.60	.493	6	1.75	.338
7	1.80	.757	7	1.92	.744	7	1.92	.580	7	2.10	.397
8	2.10	.542	8	2.24	.638	8	2.24	.633	8	2.45	.433
9	2.40	.354	9	2.56	.450	9	2.56	.596	9	2.80	.458
10	2.70	.230	10	2.88	.287	10	2.88	.460	10	3.15	.470
11	3.00	.149	11	3.20	.180	11	3.20	.305	11	3.50	.444
12	3.30	.099	12	3.52	.117	12	3.52	.193	12	3.85	.342
13	3.60	.064	13	3.84	.075	13	3.84	.121	13	4.20	.218
14	3.90	.043	14	4.16	.046	14	4.16	.077	14	4.55	.130
15	4.20	.027	15	4.48	.030	15	4.48	.048	15	4.90	.079
16	4.50	.018	16	4.80	.018	16	4.80	.031	16	5.25	.048
17	4.80	.011	17	5.12	.011	17	5.12	.020	17	5.60	.028
18	5.10	.006	18	5.44	.005	18	5.44	.012	18	5.95	.017
19	5.40	.003	19	5.76	.003	19	5.76	.007	19	6.30	.011
20	5.70	.000	20	6.08	.001	20	6.08	.003	20	6.65	.006
			21	6.40	.000	21	6.40	.001	21	7.00	.003
						22	6.72	.000	22	7.35	.001
									23	7.70	.000

Table 3.21-7 (Sheet 13 of 52)

Hydrograph Coordinates

Storm Distribution Curve: A Hydrograph Family: 4

T_o/T_p = 4			T_o/T_p = 6			T_o/T_p = 10			T_o/T_p = 16		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.42	.009	2	0.52	.009	2	0.68	.008	2	0.95	.012
3	0.84	.054	3	1.04	.045	3	1.36	.028	3	1.90	.028
4	1.26	.128	4	1.56	.106	4	2.04	.067	4	2.85	.046
5	1.68	.224	5	2.08	.166	5	2.72	.104	5	3.80	.069
6	2.10	.284	6	2.60	.206	6	3.40	.132	6	4.75	.086
7	2.52	.321	7	3.12	.224	7	4.08	.140	7	5.70	.089
8	2.94	.344	8	3.64	.233	8	4.76	.142	8	6.65	.090
9	3.36	.355	9	4.16	.240	9	5.44	.144	9	7.60	.090
10	3.78	.361	10	4.68	.244	10	6.12	.146	10	8.55	.090
11	4.20	.362	11	5.20	.246	11	6.80	.147	11	9.50	.091
12	4.62	.321	12	5.72	.248	12	7.48	.148	12	10.45	.093
13	5.04	.214	13	6.24	.249	13	8.16	.148	13	11.40	.094
14	5.46	.117	14	6.76	.199	14	8.84	.141	14	12.35	.094
15	5.88	.063	15	7.28	.102	15	9.52	.146	15	13.30	.089
16	6.30	.034	16	7.80	.049	16	10.20	.157	16	14.25	.088
17	6.72	.018	17	8.32	.023	17	10.88	.110	17	15.20	.093
18	7.14	.011	18	8.84	.010	18	11.56	.044	18	16.15	.105
19	7.56	.005	19	9.36	.004	19	12.24	.016	19	17.10	.055
20	7.98	.002	20	9.88	.002	20	12.92	.007	20	18.05	.015
21	8.40	.001	21	10.40	.001	21	13.60	.002	21	19.00	.003
22	8.82	.000	22	10.92	.000	22	14.28	.001	22	19.95	.001
						23	14.96	.000	23	20.90	.000

Hydrograph Coordinates

Storm Distribution Curve: A Hydrograph Family: 4

T_o/T_p = 25			T_o/T_p = 36			T_o/T_p =			T_o/T_p =		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000						
2	1.20	.011	2	1.90	.012						
3	2.40	.020	3	3.80	.018						
4	3.60	.028	4	5.70	.025						
5	4.80	.038	5	7.60	.033						
6	6.00	.050	6	9.50	.040						
7	7.20	.059	7	11.40	.040						
8	8.40	.059	8	13.30	.040						
9	9.60	.058	9	15.20	.040						
10	10.80	.057	10	17.10	.040						
11	12.00	.057	11	19.00	.040						
12	13.20	.057	12	20.90	.041						
13	14.40	.057	13	22.80	.041						
14	15.60	.057	14	24.70	.041						
15	16.80	.058	15	26.60	.041						
16	18.00	.060	16	28.50	.039						
17	19.20	.060	17	30.40	.039						
18	20.40	.057	18	32.30	.041						
19	21.60	.055	19	34.20	.044						
20	22.80	.057	20	36.10	.050						
21	24.00	.063	21	38.00	.007						
22	25.20	.070	22	39.90	.001						
23	26.40	.025	23	41.80	.000						
24	27.60	.007									
25	28.80	.002									
26	30.00	.000									

Table 3.21-7 (Sheet 15 of 52)

Hydrograph Coordinates

Storm Distribution Curve: A Hydrograph Family: 5

T_o/T_p = 1			T_o/T_p = 1.5			T_o/T_p = 2			T_o/T_p = 3		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.26	.011	2	0.25	.005	2	0.25	.003	2	0.34	.005
3	0.52	.066	3	0.50	.031	3	0.50	.020	3	0.68	.026
4	0.78	.210	4	0.75	.100	4	0.75	.062	4	1.02	.070
5	1.04	.480	5	1.00	.218	5	1.00	.134	5	1.36	.137
6	1.30	.750	6	1.25	.371	6	1.25	.231	6	1.70	.219
7	1.56	.880	7	1.50	.547	7	1.50	.340	7	2.04	.298
8	1.82	.804	8	1.75	.708	8	1.75	.458	8	2.38	.369
9	2.08	.620	9	2.00	.788	9	2.00	.558	9	2.72	.428
10	2.34	.432	10	2.25	.726	10	2.25	.655	10	3.06	.497
11	2.60	.297	11	2.50	.574	11	2.50	.694	11	3.40	.557
12	2.86	.204	12	2.75	.414	12	2.75	.626	12	3.74	.490
13	3.12	.141	13	3.00	.284	13	3.00	.491	13	4.08	.348
14	3.38	.097	14	3.25	.200	14	3.25	.354	14	4.42	.211
15	3.64	.067	15	3.50	.140	15	3.50	.243	15	4.76	.132
16	3.90	.046	16	3.75	.099	16	3.75	.169	16	5.10	.082
17	4.16	.033	17	4.00	.068	17	4.00	.120	17	5.44	.049
18	4.42	.022	18	4.25	.047	18	4.25	.084	18	5.78	.029
19	4.68	.014	19	4.50	.033	19	4.50	.058	19	6.12	.017
20	4.94	.009	20	4.75	.023	20	4.75	.041	20	6.46	.011
21	5.20	.005	21	5.00	.016	21	5.00	.027	21	6.80	.006
22	5.46	.002	22	5.25	.010	22	5.25	.019	22	7.14	.003
23	5.72	.000	23	5.50	.006	23	5.50	.014	23	7.48	.001
			24	5.75	.003	24	5.75	.009	24	7.82	.000
			25	6.00	.001	25	6.00	.005			
			26	6.25	.000	26	6.25	.003			
						27	6.50	.001			
						28	6.75	.000			

Hydrograph Coordinates

Storm Distribution Curve: A Hydrograph Family: 5

T_o/T_p = 4			T_o/T_p = 6			T_o/T_p = 10			T_o/T_p = 16		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.40	.004	2	0.40	.003	2	0.65	.005	2	0.82	.006
3	0.80	.026	3	0.80	.014	3	1.30	.021	3	1.64	.014
4	1.20	.065	4	1.20	.035	4	1.95	.037	4	2.46	.022
5	1.60	.119	5	1.60	.060	5	2.60	.053	5	3.28	.030
6	2.00	.177	6	2.00	.087	6	3.25	.070	6	4.10	.039
7	2.40	.234	7	2.40	.116	7	3.90	.088	7	4.92	.050
8	2.80	.284	8	2.80	.145	8	4.55	.106	8	5.74	.060
9	3.20	.326	9	3.20	.171	9	5.20	.120	9	6.56	.067
10	3.60	.363	10	3.60	.194	10	5.85	.131	10	7.38	.074
11	4.00	.410	11	4.00	.215	11	6.50	.140	11	8.20	.080
12	4.40	.462	12	4.40	.232	12	7.15	.148	12	9.02	.086
13	4.80	.386	13	4.80	.246	13	7.80	.156	13	9.84	.090
14	5.20	.239	14	5.20	.259	14	8.45	.164	14	10.66	.093
15	5.60	.134	15	5.60	.277	15	9.10	.174	15	11.48	.096
16	6.00	.078	16	6.00	.309	16	9.75	.198	16	12.30	.099
17	6.40	.043	17	6.40	.340	17	10.40	.233	17	13.12	.103
18	6.80	.024	18	6.80	.286	18	11.05	.146	18	13.94	.110
19	7.20	.014	19	7.20	.177	19	11.70	.058	19	14.76	.118
20	7.60	.007	20	7.60	.100	20	12.35	.023	20	15.58	.137
21	8.00	.003	21	8.00	.056	21	13.00	.009	21	16.40	.165
22	8.40	.001	22	8.40	.033	22	13.65	.003	22	17.22	.081
23	8.80	.000	23	8.80	.017	23	14.30	.001	23	18.04	.025
			24	9.20	.009	24	14.95	.000	24	18.86	.008
			25	9.60	.005				25	19.68	.002
			26	10.00	.002				26	20.50	.000
			27	10.40	.001						
			28	10.80	.000						

Table 3.21-7 (Sheet 17 of 52)

Hydrograph Coordinates

Storm Distribution Curve: A Hydrograph Family: 5

T_o/T_p = 25			T_o/T_p = 36			T_o/T_p =			T_o/T_p =		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000						
2	1.02	.004	2	1.40	.004						
3	2.04	.009	3	2.80	.008						
4	3.06	.014	4	4.20	.011						
5	4.08	.018	5	5.60	.014						
6	5.10	.022	6	7.00	.017						
7	6.12	.027	7	8.40	.019						
8	7.14	.032	8	9.80	.021						
9	8.16	.037	9	11.20	.024						
10	9.18	.042	10	12.60	.027						
11	10.20	.046	11	14.00	.030						
12	11.22	.049	12	15.40	.033						
13	12.24	.052	13	16.80	.035						
14	13.26	.055	14	18.20	.037						
15	14.28	.057	15	19.60	.039						
16	15.30	.059	16	21.00	.040						
17	16.32	.061	17	22.40	.041						
18	17.34	.062	18	23.80	.042						
19	18.36	.064	19	25.20	.043						
20	19.38	.066	20	26.60	.045						
21	20.40	.068	21	28.00	.047						
22	21.42	.071	22	29.40	.049						
23	22.44	.075	23	30.80	.052						
24	23.46	.083	24	32.20	.056						
25	24.48	.097	25	33.60	.061						
26	25.50	.118	26	35.00	.070						
27	26.52	.037	27	36.40	.090						
28	27.54	.007	28	37.80	.017						
29	28.56	.001	29	39.20	.003						
30	29.58	.000	30	40.60	.000						

Hydrograph Coordinates

Storm Distribution Curve: A_2 Hydrograph Family: 1

T_o/T_p = 1			T_o/T_p = 1.5			T_o/T_p = 2			T_o/T_p = 3		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.24	.055	2	0.25	.040	2	0.28	.037	2	0.31	.027
3	0.48	.250	3	0.50	.190	3	0.56	.190	3	0.62	.156
4	0.72	.550	4	0.75	.460	4	0.84	.460	4	0.93	.375
5	0.96	.821	5	1.00	.713	5	1.12	.678	5	1.24	.553
6	1.20	.900	6	1.25	.824	6	1.40	.748	6	1.55	.635
7	1.44	.820	7	1.50	.785	7	1.68	.698	7	1.86	.602
8	1.68	.630	8	1.75	.652	8	1.96	.573	8	2.17	.532
9	1.92	.467	9	2.00	.505	9	2.24	.433	9	2.48	.411
10	2.16	.333	10	2.25	.374	10	2.52	.325	10	2.79	.314
11	2.40	.238	11	2.50	.266	11	2.80	.225	11	3.10	.236
12	2.64	.170	12	2.75	.186	12	3.08	.159	12	3.41	.173
13	2.88	.120	13	3.00	.130	13	3.36	.109	13	3.72	.122
14	3.12	.088	14	3.25	.091	14	3.64	.074	14	4.03	.084
15	3.36	.061	15	3.50	.062	15	3.92	.047	15	4.34	.055
16	3.60	.043	16	3.75	.044	16	4.20	.030	16	4.65	.034
17	3.84	.031	17	4.00	.031	17	4.48	.021	17	4.96	.020
18	4.08	.021	18	4.25	.021	18	4.76	.014	18	5.27	.012
19	4.32	.014	19	4.50	.014	19	5.04	.009	19	5.58	.008
20	4.56	.009	20	4.75	.009	20	5.32	.005	20	5.89	.005
21	4.80	.006	21	5.00	.005	21	5.60	.003	21	6.20	.004
22	5.04	.004	22	5.25	.003	22	5.88	.002	22	6.51	.003
23	5.28	.002	23	5.50	.002	23	6.16	.001	23	6.82	.002
24	5.52	.001	24	5.75	.001	24	6.44		24	7.13	.001
25	5.76	.000	25	6.00		25	6.72	.000	25	7.44	
			26	6.25	.000				26	7.75	.000

Table 3.21-7 (Sheet 19 of 52)

Hydrograph Coordinates

Storm Distribution Curve: A_2 Hydrograph Family: 1

T_o/T_p = 4			T_o/T_p = 6			T_o/T_p = 10			T_o/T_p = 16		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.35	.025	2	0.40	.026	2	0.50	.018	2	0.75	.029
3	0.70	.154	3	0.80	.126	3	1.00	.109	3	1.50	.122
4	1.05	.355	4	1.20	.280	4	1.50	.220	4	2.25	.185
5	1.40	.499	5	1.60	.397	5	2.00	.280	5	3.00	.202
6	1.75	.546	6	2.00	.427	6	2.50	.293	6	3.75	.191
7	2.10	.513	7	2.40	.407	7	3.00	.282	7	4.50	.174
8	2.45	.448	8	2.80	.363	8	3.50	.257	8	5.25	.154
9	2.80	.363	9	3.20	.311	9	4.00	.227	9	6.00	.135
10	3.15	.278	10	3.60	.256	10	4.50	.196	10	6.75	.116
11	3.50	.208	11	4.00	.204	11	5.00	.166	11	7.50	.097
12	3.85	.156	12	4.40	.155	12	5.50	.138	12	8.25	.081
13	4.20	.116	13	4.80	.116	13	6.00	.113	13	9.00	.066
14	4.55	.083	14	5.20	.087	14	6.50	.089	14	9.75	.052
15	4.90	.055	15	5.60	.067	15	7.00	.069	15	10.50	.041
16	5.25	.034	16	6.00	.052	16	7.50	.053	16	11.25	.032
17	5.60	.019	17	6.40	.040	17	8.00	.042	17	12.00	.025
18	5.95	.012	18	6.80	.028	18	8.50	.034	18	12.75	.021
19	6.30	.007	19	7.20	.017	19	9.00	.030	19	13.50	.019
20	6.65	.004	20	7.60	.009	20	9.50	.027	20	14.25	.017
21	7.00	.003	21	8.00	.005	21	10.00	.024	21	15.00	.015
22	7.35	.002	22	8.40	.004	22	10.50	.019	22	15.75	.014
23	7.70	.001	23	8.80	.003	23	11.00	.011	23	16.50	.011
24	8.05	.001	24	9.20	.002	24	11.50	.005	24	17.25	.006
25	8.40		25	9.60	.001	25	12.00	.003	25	18.00	.003
26	8.75	.000	26	10.00	.001	26	12.50	.002	26	18.75	.002
			27	10.40		27	13.00	.001	27	19.50	.001
			28	10.80	.000	28	13.50	.001	28	20.25	.000
						29	14.00	.000			
						30	14.50	.000			

Hydrograph Coordinates

Storm Distribution Curve: A_2 Hydrograph Family: 1

T_o/T_p = 25			T_o/T_p = 36			T_o/T_p =			T_o/T_p =		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000						
2	1.25	.047	2	1.50	.037						
3	2.50	.118	3	3.00	.086						
4	3.75	.136	4	4.50	.097						
5	5.00	.127	5	6.00	.094						
6	6.25	.114	6	7.50	.087						
7	7.50	.099	7	9.00	.077						
8	8.75	.084	8	10.50	.068						
9	10.00	.071	9	12.00	.059						
10	11.25	.060	10	13.50	.051						
11	12.50	.050	11	15.00	.044						
12	13.75	.040	12	16.50	.038						
13	15.00	.030	13	18.00	.032						
14	16.25	.023	14	19.50	.026						
15	17.50	.017	15	21.00	.021						
16	18.75	.014	16	22.50	.017						
17	20.00	.012	17	24.00	.014						
18	21.25	.011	18	25.50	.011						
19	22.50	.010	19	27.00	.009						
20	23.75	.009	20	28.50	.008						
21	25.00	.007	21	30.00	.007						
22	26.25	.004	22	31.50	.006						
23	27.50	.002	23	33.00	.006						
24	28.75	.001	24	34.50	.005						
25	30.00	.000	25	36.00	.005						
			26	37.50	.004						
			27	39.00	.002						
			28	40.50	.000						

Table 3.21-7 (Sheet 21 of 52)

Hydrograph Coordinates

Storm Distribution Curve: A_2 Hydrograph Family: 2

T_o/T_p = 1			T_o/T_p = 1.5			T_o/T_p = 2			T_o/T_p = 3		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.25	.046	2	0.28	.035	2	0.30	.026	2	0.30	.013
3	0.50	.218	3	0.56	.180	3	0.60	.161	3	0.60	.092
4	0.75	.520	4	0.84	.490	4	0.90	.409	4	0.90	.251
5	1.00	.799	5	1.12	.708	5	1.20	.640	5	1.20	.438
6	1.25	.891	6	1.40	.807	6	1.50	.733	6	1.50	.572
7	1.50	.807	7	1.68	.722	7	1.80	.673	7	1.80	.611
8	1.75	.629	8	1.96	.580	8	2.10	.552	8	2.10	.571
9	2.00	.453	9	2.24	.423	9	2.40	.426	9	2.40	.480
10	2.25	.325	10	2.52	.299	10	2.70	.304	10	2.70	.390
11	2.50	.224	11	2.80	.200	11	3.00	.208	11	3.00	.311
12	2.75	.155	12	3.08	.133	12	3.30	.137	12	3.30	.241
13	3.00	.108	13	3.36	.088	13	3.60	.089	13	3.60	.182
14	3.25	.077	14	3.64	.059	14	3.90	.058	14	3.90	.127
15	3.50	.054	15	3.92	.040	15	4.20	.037	15	4.20	.086
16	3.75	.037	16	4.20	.027	16	4.50	.024	16	4.50	.056
17	4.00	.026	17	4.48	.018	17	4.80	.014	17	4.80	.034
18	4.25	.017	18	4.76	.011	18	5.10	.008	18	5.10	.020
19	4.50	.012	19	5.04	.006	19	5.40	.005	19	5.40	.013
20	4.75	.007	20	5.32	.003	20	5.70	.003	20	5.70	.008
21	5.00	.004	21	5.60	.002	21	6.00	.002	21	6.00	.005
22	5.25	.002	22	5.88	.001	22	6.30	.001	22	6.30	.003
23	5.50	.001	23	6.16	.000	23	6.60	.000	23	6.60	.002
24	5.75	.000							24	6.90	.001
									25	7.20	.001
									26	7.50	
									27	7.80	.000

Hydrograph Coordinates

Storm Distribution Curve: A_2 Hydrograph Family: 2

T_o/T_p = 4			T_o/T_p = 6			T_o/T_p = 10			T_o/T_p = 16		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.40	.016	2	0.50	.018	2	0.65	.015	2	0.82	.012
3	0.80	.127	3	1.00	.120	3	1.30	.098	3	1.64	.073
4	1.20	.312	4	1.50	.268	4	1.95	.191	4	2.46	.127
5	1.60	.465	5	2.00	.365	5	2.60	.249	5	3.28	.164
6	2.00	.520	6	2.50	.393	6	3.25	.262	6	4.10	.173
7	2.40	.489	7	3.00	.371	7	3.90	.250	7	4.92	.168
8	2.80	.405	8	3.50	.316	8	4.55	.227	8	5.74	.158
9	3.20	.307	9	4.00	.241	9	5.20	.192	9	6.56	.144
10	3.60	.232	10	4.50	.175	10	5.85	.148	10	7.38	.125
11	4.00	.175	11	5.00	.129	11	6.50	.107	11	8.20	.103
12	4.40	.133	12	5.50	.098	12	7.15	.080	12	9.02	.081
13	4.80	.088	13	6.00	.080	13	7.80	.063	13	9.84	.062
14	5.20	.052	14	6.50	.061	14	8.45	.052	14	10.66	.048
15	5.60	.028	15	7.00	.036	15	9.10	.044	15	11.48	.038
16	6.00	.015	16	7.50	.017	16	9.75	.038	16	12.30	.033
17	6.40	.008	17	8.00	.008	17	10.40	.033	17	13.12	.028
18	6.80	.004	18	8.50	.004	18	11.05	.017	18	13.94	.026
19	7.20	.003	19	9.00	.003	19	11.70	.007	19	14.76	.024
20	7.60	.002	20	9.50	.002	20	12.35	.003	20	15.58	.022
21	8.00	.001	21	10.00	.001	21	13.00	.002	21	16.40	.018
22	8.40		22	10.50	.000	22	13.65	.001	22	17.22	.008
23	8.80	.000				23	14.30		23	18.04	.003
						24	14.95	.000	24	18.86	.002
									25	19.68	.001
									26	20.50	.000

Table 3.21-7 (Sheet 23 of 52)

Hydrograph Coordinates

Storm Distribution Curve: A2 Hydrograph Family: 2

T_o/T_p = 25			T_o/T_p = 36			T_o/T_p =			T_o/T_p =		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000						
2	1.10	.013	2	1.45	.012						
3	2.20	.057	3	2.90	.048						
4	3.30	.089	4	4.35	.065						
5	4.40	.106	5	5.80	.075						
6	5.50	.114	6	7.25	.082						
7	6.60	.111	7	8.70	.078						
8	7.70	.105	8	10.15	.074						
9	8.80	.098	9	11.60	.069						
10	9.90	.090	10	13.05	.064						
11	11.00	.079	11	14.50	.059						
12	12.10	.068	12	15.95	.052						
13	13.20	.055	13	17.40	.044						
14	14.30	.041	14	18.85	.036						
15	15.40	.032	15	20.30	.029						
16	16.50	.028	16	21.75	.023						
17	17.60	.024	17	23.20	.018						
18	18.70	.020	18	24.65	.016						
19	19.80	.018	19	26.10	.015						
20	20.90	.017	20	27.55	.014						
21	22.00	.016	21	29.00	.013						
22	23.10	.015	22	30.45	.012						
23	24.20	.014	23	31.90	.011						
24	25.30	.012	24	33.35	.010						
25	26.40	.004	25	34.80	.010						
26	27.50	.002	26	36.25	.008						
27	28.60	.001	27	37.70	.002						
28	29.70	.000	28	39.15	.001						
			29	40.60	.000						

Hydrograph Coordinates

Storm Distribution Curve: B Hydrograph Family: 1

$T_o/T_p = 1$			$T_o/T_p = 1.5$			$T_o/T_p = 2$			$T_o/T_p = 3$		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.28	.029	2	0.32	.012	2	0.29	.007	2	0.35	.005
3	0.56	.150	3	0.64	.118	3	0.58	.035	3	0.70	.027
4	0.84	.472	4	0.96	.377	4	0.87	.164	4	1.05	.101
5	1.12	.798	5	1.28	.711	5	1.16	.432	5	1.40	.302
6	1.40	.901	6	1.60	.815	6	1.45	.669	6	1.75	.563
7	1.68	.776	7	1.92	.719	7	1.74	.740	7	2.10	.650
8	1.96	.568	8	2.24	.526	8	2.03	.680	8	2.45	.576
9	2.24	.389	9	2.56	.352	9	2.32	.561	9	2.80	.460
10	2.52	.258	10	2.88	.225	10	2.61	.441	10	3.15	.374
11	2.80	.173	11	3.20	.143	11	2.90	.319	11	3.50	.290
12	3.08	.115	12	3.52	.090	12	3.19	.212	12	3.85	.201
13	3.36	.078	13	3.84	.057	13	3.48	.140	13	4.20	.127
14	3.64	.052	14	4.16	.037	14	3.77	.094	14	4.55	.078
15	3.92	.036	15	4.48	.024	15	4.06	.063	15	4.90	.047
16	4.20	.024	16	4.80	.015	16	4.35	.042	16	5.25	.028
17	4.48	.016	17	5.12	.008	17	4.64	.028	17	5.60	.016
18	4.76	.009	18	5.44	.004	18	4.93	.017	18	5.95	.009
19	5.04	.005	19	5.76	.002	19	5.22	.011	19	6.30	.005
20	5.32	.002	20	6.08	.001	20	5.51	.007	20	6.65	.003
21	5.60	.001	21	6.40	.000	21	5.80	.004	21	7.00	.002
22	5.88	.000				22	6.09	.002	22	7.35	.001
						23	6.38	.001	23	7.70	.000
						24	6.67	.000			

Table 3.21-7 (Sheet 25 of 52)

Hydrograph Coordinates

Storm Distribution Curve: B Hydrograph Family: 1

T_o/T_p = 4			T_o/T_p = 6			T_o/T_p = 10			T_o/T_p = 16		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.35	.003	2	0.44	.003	2	0.56	.002	2	0.66	.001
3	0.70	.015	3	0.88	.013	3	1.12	.013	3	1.32	.006
4	1.05	.049	4	1.32	.041	4	1.68	.027	4	1.98	.015
5	1.40	.122	5	1.76	.084	5	2.24	.047	5	2.64	.027
6	1.75	.298	6	2.20	.176	6	2.80	.071	6	3.30	.037
7	2.10	.528	7	2.64	.386	7	3.36	.115	7	3.96	.047
8	2.45	.585	8	3.08	.497	8	3.92	.278	8	4.62	.062
9	2.80	.518	9	3.52	.430	9	4.48	.394	9	5.28	.092
10	3.15	.413	10	3.96	.335	10	5.04	.322	10	5.94	.223
11	3.50	.334	11	4.40	.258	11	5.60	.235	11	6.60	.309
12	3.85	.273	12	4.84	.202	12	6.16	.174	12	7.26	.243
13	4.20	.231	13	5.28	.164	13	6.72	.136	13	7.92	.171
14	4.55	.185	14	5.72	.139	14	7.28	.110	14	8.58	.124
15	4.90	.128	15	6.16	.124	15	7.84	.092	15	9.24	.097
16	5.25	.080	16	6.60	.100	16	8.40	.079	16	9.90	.081
17	5.60	.047	17	7.04	.060	17	8.96	.073	17	10.56	.070
18	5.95	.028	18	7.48	.033	18	9.52	.068	18	11.22	.061
19	6.30	.017	19	7.92	.018	19	10.08	.065	19	11.88	.055
20	6.65	.010	20	8.36	.009	20	10.64	.053	20	12.54	.050
21	7.00	.006	21	8.80	.005	21	11.20	.027	21	13.20	.047
22	7.35	.004	22	9.24	.003	22	11.76	.012	22	13.86	.045
23	7.70	.003	23	9.68	.002	23	12.32	.006	23	14.52	.044
24	8.05	.002	24	10.12	.001	24	12.88	.003	24	15.18	.043
25	8.40	.001	25	10.56	.000	25	13.44	.002	25	15.84	.040
26	8.75	.000				26	14.00	.001	26	16.50	.034
						27	14.56	.000	27	17.16	.020
									28	17.82	.008
									29	18.48	.004
									30	19.14	.002
									31	19.80	.001
									32	20.46	.000

Hydrograph Coordinates

Storm Distribution Curve: B Hydrograph Family: 1

T_o/T_p = 25			T_o/T_p = 36			T_o/T_p =			T_o/T_p =		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000						
2	1.22	.002	2	1.70	.002						
3	2.44	.009	3	3.40	.008						
4	3.66	.018	4	5.10	.014						
5	4.88	.027	5	6.80	.020						
6	6.10	.036	6	8.50	.026						
7	7.32	.046	7	10.20	.033						
8	8.54	.116	8	11.90	.077						
9	9.76	.232	9	13.60	.177						
10	10.98	.146	10	15.30	.101						
11	12.20	.088	11	17.00	.058						
12	13.42	.062	12	18.70	.044						
13	14.64	.051	13	20.40	.036						
14	15.86	.045	14	22.10	.030						
15	17.08	.039	15	23.80	.027						
16	18.30	.035	16	25.50	.024						
17	19.52	.031	17	27.20	.022						
18	20.74	.027	18	28.90	.020						
19	21.96	.025	19	30.60	.018						
20	23.18	.025	20	32.30	.017						
21	24.40	.025	21	34.00	.017						
22	25.62	.020	22	35.70	.017						
23	26.84	.005	23	37.40	.004						
24	28.06	.002	24	39.10	.002						
25	29.28	.000	25	40.80	.000						

Table 3.21-7 (Sheet 27 of 52)

Hydrograph Coordinates

Storm Distribution Curve: B Hydrograph Family: 2

T_o/T_p = 1			T_o/T_p = 1.5			T_o/T_p = 2			T_o/T_p = 3		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.28	.026	2	0.22	.003	2	0.28	.004	2	0.32	.003
3	0.56	.170	3	0.44	.041	3	0.56	.040	3	0.64	.017
4	0.84	.480	4	0.66	.161	4	0.84	.170	4	0.96	.093
5	1.12	.802	5	0.88	.362	5	1.12	.428	5	1.28	.311
6	1.40	.885	6	1.10	.604	6	1.40	.645	6	1.60	.530
7	1.68	.770	7	1.32	.740	7	1.68	.715	7	1.92	.615
8	1.96	.550	8	1.54	.790	8	1.96	.677	8	2.24	.575
9	2.24	.380	9	1.76	.746	9	2.24	.574	9	2.56	.487
10	2.52	.257	10	1.98	.640	10	2.52	.472	10	2.88	.409
11	2.80	.166	11	2.20	.536	11	2.80	.369	11	3.20	.344
12	3.08	.113	12	2.42	.414	12	3.08	.247	12	3.52	.279
13	3.36	.078	13	2.64	.303	13	3.36	.168	13	3.84	.206
14	3.64	.052	14	2.86	.219	14	3.64	.113	14	4.16	.135
15	3.92	.034	15	3.08	.160	15	3.92	.075	15	4.48	.087
16	4.20	.023	16	3.30	.117	16	4.20	.050	16	4.80	.054
17	4.48	.015	17	3.52	.088	17	4.48	.034	17	5.12	.032
18	4.76	.009	18	3.74	.064	18	4.76	.021	18	5.44	.019
19	5.04	.004	19	3.96	.047	19	5.04	.014	19	5.76	.012
20	5.32	.002	20	4.18	.035	20	5.32	.008	20	6.08	.008
21	5.60	.001	21	4.40	.025	21	5.60	.004	21	6.40	.005
22	5.88	.000	22	4.62	.018	22	5.88	.003	22	6.72	.003
			23	4.84	.012	23	6.16	.002	23	7.04	.002
			24	5.06	.007	24	6.44	.001	24	7.36	.001
			25	5.28	.004	25	6.72	.000	25	7.68	.000
			26	5.50	.003						
			27	5.72	.002						
			28	5.94	.001						
			29	6.16	.000						

Hydrograph Coordinates

Storm Distribution Curve: B Hydrograph Family: 2

T_o/T_p = 4			T_o/T_p = 6			T_o/T_p = 10			T_o/T_p = 16		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.32	.002	2	0.34	.001	2	0.63	.002	2	0.90	.002
3	0.64	.009	3	0.68	.005	3	1.26	.009	3	1.80	.007
4	0.96	.036	4	1.02	.015	4	1.89	.027	4	2.70	.020
5	1.28	.129	5	1.36	.037	5	2.52	.063	5	3.60	.037
6	1.60	.332	6	1.70	.098	6	3.15	.236	6	4.50	.148
7	1.92	.501	7	2.04	.244	7	3.78	.364	7	5.40	.277
8	2.24	.550	8	2.38	.407	8	4.41	.307	8	6.30	.214
9	2.56	.500	9	2.72	.464	9	5.04	.226	9	7.20	.149
10	2.88	.422	10	3.06	.429	10	5.67	.172	10	8.10	.112
11	3.20	.358	11	3.40	.367	11	6.30	.136	11	9.00	.088
12	3.52	.302	12	3.74	.309	12	6.93	.113	12	9.90	.073
13	3.84	.274	13	4.08	.261	13	7.56	.097	13	10.80	.063
14	4.16	.230	14	4.42	.224	14	8.19	.085	14	11.70	.056
15	4.48	.195	15	4.76	.193	15	8.82	.078	15	12.60	.052
16	4.80	.147	16	5.10	.169	16	9.45	.074	16	13.50	.048
17	5.12	.099	17	5.44	.152	17	10.08	.069	17	14.40	.045
18	5.44	.061	18	5.78	.139	18	10.71	.053	18	15.30	.044
19	5.76	.037	19	6.12	.129	19	11.34	.025	19	16.20	.042
20	6.08	.023	20	6.46	.113	20	11.97	.009	20	17.10	.023
21	6.40	.013	21	6.80	.085	21	12.60	.004	21	18.00	.006
22	6.72	.008	22	7.14	.055	22	13.23	.002	22	18.90	.003
23	7.04	.005	23	7.48	.035	23	13.86	.001	23	19.80	.001
24	7.36	.004	24	7.82	.020	24	14.49	.000	24	20.70	.000
25	7.68	.003	25	8.16	.012						
26	8.00	.002	26	8.50	.008						
27	8.32	.001	27	8.84	.005						
28	8.64	.000	28	9.18	.004						
			29	9.52	.003						
			30	9.86	.002						
			31	10.20	.001						
			32	10.54	.000						

Table 3.21-7 (Sheet 29 of 52)

Hydrograph Coordinates

Storm Distribution Curve: B Hydrograph Family: 2

T_o/T_p = 25			T_o/T_p = 36			T_o/T_p =			T_o/T_p =		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000						
2	1.30	.002	2	1.79	.002						
3	2.60	.006	3	3.58	.006						
4	3.90	.014	4	5.37	.012						
5	5.20	.024	5	7.16	.019						
6	6.50	.088	6	8.95	.057						
7	7.80	.210	7	10.74	.157						
8	9.10	.146	8	12.53	.104						
9	10.40	.097	9	14.32	.068						
10	11.70	.072	10	16.11	.047						
11	13.00	.057	11	17.90	.040						
12	14.30	.049	12	19.69	.034						
13	15.60	.044	13	21.48	.030						
14	16.90	.039	14	23.27	.026						
15	18.20	.035	15	25.06	.025						
16	19.50	.033	16	26.85	.023						
17	20.80	.031	17	28.64	.021						
18	22.10	.029	18	30.43	.020						
19	23.40	.028	19	32.22	.019						
20	24.70	.027	20	34.01	.018						
21	26.00	.014	21	35.80	.017						
22	27.30	.004	22	37.59	.007						
23	28.60	.001	23	39.38	.001						
24	29.90	.000	24	41.17	.000						

Hydrograph Coordinates

Storm Distribution Curve: B Hydrograph Family: 3

T_o/T_p = 1			T_o/T_p = 1.5			T_o/T_p = 2			T_o/T_p = 3		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.26	.048	2	0.29	.028	2	0.30	.012	2	0.34	.004
3	0.52	.219	3	0.58	.190	3	0.60	.123	3	0.68	.088
4	0.78	.521	4	0.87	.450	4	0.90	.343	4	1.02	.289
5	1.04	.762	5	1.16	.656	5	1.20	.570	5	1.36	.489
6	1.30	.844	6	1.45	.734	6	1.50	.657	6	1.70	.543
7	1.56	.778	7	1.74	.685	7	1.80	.630	7	2.04	.507
8	1.82	.621	8	2.03	.585	8	2.10	.562	8	2.38	.445
9	2.08	.441	9	2.32	.445	9	2.40	.484	9	2.72	.385
10	2.34	.305	10	2.61	.350	10	2.70	.379	10	3.06	.340
11	2.60	.214	11	2.90	.199	11	3.00	.267	11	3.40	.294
12	2.86	.149	12	3.19	.132	12	3.30	.177	12	3.74	.223
13	3.12	.103	13	3.48	.089	13	3.60	.116	13	4.08	.149
14	3.38	.070	14	3.77	.057	14	3.90	.076	14	4.42	.096
15	3.64	.048	15	4.06	.038	15	4.20	.050	15	4.76	.056
16	3.90	.034	16	4.35	.025	16	4.50	.033	16	5.10	.033
17	4.16	.024	17	4.64	.015	17	4.80	.020	17	5.44	.019
18	4.42	.016	18	4.93	.008	18	5.10	.011	18	5.78	.013
19	4.68	.010	19	5.22	.005	19	5.40	.006	19	6.12	.008
20	4.94	.006	20	5.51	.003	20	5.70	.004	20	6.46	.004
21	5.20	.003	21	5.80	.002	21	6.00	.002	21	6.80	.003
22	5.46	.001	22	6.09	.001	22	6.30	.001	22	7.14	.002
23	5.72	.000	23	6.38	.000	23	6.60	.000	23	7.48	.001
									24	7.82	.000

Table 3.21-7 (Sheet 31 of 52)

Hydrograph Coordinates

Storm Distribution Curve: B Hydrograph Family: 3

T_o/T_p = 4			T_o/T_p = 6			T_o/T_p = 10			T_o/T_p = 16		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.36	.003	2	0.42	.002	2	0.54	.001	2	0.90	.002
3	0.72	.044	3	0.84	.021	3	1.08	.008	3	1.80	.016
4	1.08	.203	4	1.26	.138	4	1.62	.069	4	2.70	.122
5	1.44	.400	5	1.68	.320	5	2.16	.231	5	3.60	.230
6	1.80	.478	6	2.10	.390	6	2.70	.303	6	4.50	.185
7	2.16	.450	7	2.52	.363	7	3.24	.269	7	5.40	.139
8	2.52	.397	8	2.94	.314	8	3.78	.223	8	6.30	.113
9	2.88	.342	9	3.36	.270	9	4.32	.188	9	7.20	.094
10	3.24	.296	10	3.78	.232	10	4.86	.159	10	8.10	.081
11	3.60	.257	11	4.20	.199	11	5.40	.139	11	9.00	.072
12	3.96	.234	12	4.62	.174	12	5.94	.122	12	9.90	.064
13	4.32	.210	13	5.04	.155	13	6.48	.108	13	10.80	.057
14	4.68	.169	14	5.46	.144	14	7.02	.097	14	11.70	.053
15	5.04	.111	15	5.88	.137	15	7.56	.089	15	12.60	.050
16	5.40	.067	16	6.30	.127	16	8.10	.081	16	13.50	.049
17	5.76	.037	17	6.72	.101	17	8.64	.078	17	14.40	.048
18	6.12	.022	18	7.14	.063	18	9.18	.077	18	15.30	.047
19	6.48	.014	19	7.56	.033	19	9.72	.077	19	16.20	.046
20	6.84	.008	20	7.98	.018	20	10.26	.075	20	17.10	.024
21	7.20	.006	21	8.40	.010	21	10.80	.055	21	18.00	.006
22	7.56	.004	22	8.82	.005	22	11.34	.030	22	18.90	.004
23	7.92	.002	23	9.24	.003	23	11.88	.012	23	19.80	.002
24	8.28	.001	24	9.66	.002	24	12.42	.006	24	20.70	.000
25	8.64	.000	25	10.08	.001	25	12.96	.004			
			26	10.50		26	13.50	.002			
			27	10.92	.000	27	14.04	.001			
						28	14.58	.000			

Hydrograph Coordinates

Storm Distribution Curve: B Hydrograph Family: 3

T_o/T_p = 25			T_o/T_p = 36			T_o/T_p =			T_o/T_p =		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000						
2	1.23	.002	2	1.62	.002						
3	2.46	.009	3	3.24	.006						
4	3.69	.073	4	4.86	.047						
5	4.92	.173	5	6.48	.130						
6	6.15	.132	6	8.10	.097						
7	7.38	.096	7	9.72	.069						
8	8.61	.076	8	11.34	.052						
9	9.84	.064	9	12.96	.045						
10	11.07	.055	10	14.58	.041						
11	12.30	.050	11	16.20	.037						
12	13.53	.046	12	17.82	.034						
13	14.76	.042	13	19.44	.031						
14	15.99	.038	14	21.06	.028						
15	17.22	.035	15	22.68	.025						
16	18.45	.033	16	24.30	.024						
17	19.68	.032	17	25.92	.024						
18	20.91	.031	18	27.54	.024						
19	22.14	.031	19	29.16	.024						
20	23.37	.031	20	30.78	.023						
21	24.60	.031	21	32.40	.023						
22	25.83	.025	22	34.02	.023						
23	27.06	.004	23	35.64	.023						
24	28.29	.001	24	37.26	.007						
25	29.52	.000	25	38.88	.003						
			26	40.50	.000						

Table 3.21-7 (Sheet 33 of 52)

Hydrograph Coordinates

Storm Distribution Curve: B Hydrograph Family: 4

T_o/T_p = 1			T_o/T_p = 1.5			T_o/T_p = 2			T_o/T_p = 3		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.28	.051	2	0.28	.038	2	0.32	.031	2	0.28	.018
3	0.56	.220	3	0.56	.166	3	0.64	.173	3	0.56	.086
4	0.84	.490	4	0.84	.360	4	0.96	.360	4	0.84	.200
5	1.12	.738	5	1.12	.551	5	1.28	.494	5	1.12	.311
6	1.40	.830	6	1.40	.651	6	1.60	.555	6	1.40	.386
7	1.68	.751	7	1.68	.686	7	1.92	.567	7	1.68	.415
8	1.96	.573	8	1.96	.650	8	2.24	.555	8	1.96	.422
9	2.24	.392	9	2.24	.543	9	2.56	.490	9	2.24	.417
10	2.52	.259	10	2.52	.392	10	2.88	.370	10	2.52	.402
11	2.80	.174	11	2.80	.267	11	3.20	.242	11	2.80	.394
12	3.08	.118	12	3.08	.180	12	3.52	.150	12	3.08	.387
13	3.36	.079	13	3.36	.120	13	3.84	.098	13	3.36	.363
14	3.64	.053	14	3.64	.081	14	4.16	.063	14	3.64	.316
15	3.92	.036	15	3.92	.055	15	4.48	.038	15	3.92	.236
16	4.20	.025	16	4.20	.036	16	4.80	.024	16	4.20	.164
17	4.48	.017	17	4.48	.024	17	5.12	.013	17	4.48	.108
18	4.76	.011	18	4.76	.015	18	5.44	.008	18	4.76	.073
19	5.04	.006	19	5.04	.009	19	5.76	.004	19	5.04	.047
20	5.32	.003	20	5.32	.005	20	6.08	.002	20	5.32	.030
21	5.60	.001	21	5.60	.003	21	6.40	.001	21	5.60	.020
22	5.88	.000	22	5.88	.001	22	6.72	.000	22	5.88	.013
			23	6.16	.000				23	6.16	.008
									24	6.44	.005
									25	6.72	.003
									26	7.00	.002
									27	7.28	.001
									28	7.56	
									29	7.84	.000

Hydrograph Coordinates

Storm Distribution Curve: B Hydrograph Family: 4

T_o/T_p = 4			T_o/T_p = 6			T_o/T_p = 10			T_o/T_p = 16		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.40	.023	2	0.40	.014	2	0.50	.015	2	0.62	.015
3	0.80	.143	3	0.80	.088	3	1.00	.079	3	1.24	.064
4	1.20	.272	4	1.20	.191	4	1.50	.151	4	1.86	.112
5	1.60	.326	5	1.60	.244	5	2.00	.177	5	2.48	.128
6	2.00	.340	6	2.00	.250	6	2.50	.170	6	3.10	.119
7	2.40	.337	7	2.40	.246	7	3.00	.159	7	3.72	.105
8	2.80	.323	8	2.80	.240	8	3.50	.152	8	4.34	.097
9	3.20	.306	9	3.20	.233	9	4.00	.146	9	4.96	.094
10	3.60	.293	10	3.60	.223	10	4.50	.141	10	5.58	.091
11	4.00	.286	11	4.00	.212	11	5.00	.136	11	6.20	.089
12	4.40	.266	12	4.40	.202	12	5.50	.131	12	6.82	.087
13	4.80	.197	13	4.80	.194	13	6.00	.126	13	7.44	.085
14	5.20	.122	14	5.20	.189	14	6.50	.121	14	8.06	.082
15	5.60	.067	15	5.60	.187	15	7.00	.116	15	8.68	.079
16	6.00	.036	16	6.00	.185	16	7.50	.112	16	9.30	.076
17	6.40	.021	17	6.40	.175	17	8.00	.112	17	9.92	.074
18	6.80	.013	18	6.80	.131	18	8.50	.111	18	10.54	.072
19	7.20	.008	19	7.20	.080	19	9.00	.111	19	11.16	.071
20	7.60	.005	20	7.60	.046	20	9.50	.110	20	11.78	.070
21	8.00	.002	21	8.00	.027	21	10.00	.110	21	12.40	.069
22	8.40	.001	22	8.40	.016	22	10.50	.100	22	13.02	.069
23	8.80	.000	23	8.80	.009	23	11.00	.065	23	13.64	.069
			24	9.20	.005	24	11.50	.033	24	14.26	.069
			25	9.60	.003	25	12.00	.025	25	14.88	.069
			26	10.00	.002	26	12.50	.007	26	15.50	.069
			27	10.40	.001	27	13.00	.004	27	16.12	.068
			28	10.80	.000	28	13.50	.002	28	16.74	.053
						29	14.00	.001	29	17.36	.023
						30	14.50	.000	30	17.98	.009
									31	18.60	.004
									32	19.22	.002
									33	19.84	.001

Table 3.21-7 (Sheet 35 of 52)

Hydrograph Coordinates

Storm Distribution Curve: B Hydrograph Family: 4

T_o/T_p = 16			T_o/T_p = 25			T_o/T_p =			T_o/T_p =		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
34	20.46	.000	1	0.00	.000						
			2	1.02	.025						
			3	2.04	.070						
			4	3.06	.090						
			5	4.08	.082						
			6	5.10	.068						
			7	6.12	.062						
			8	7.14	.059						
			9	8.16	.056						
			10	9.18	.055						
			11	10.20	.054						
			12	11.22	.053						
			13	12.24	.052						
			14	13.26	.050						
			15	14.28	.049						
			16	15.30	.047						
			17	16.32	.046						
			18	17.34	.045						
			19	18.36	.044						
			20	19.38	.044						
			21	20.40	.044						
			22	21.42	.044						
			23	22.44	.044						
			24	23.46	.044						
			25	24.48	.044						
			26	25.50	.039						
			27	26.52	.012						
			28	27.54	.004						
			29	28.56	.001						
			30	29.58	.000						

Hydrograph Coordinates

Storm Distribution Curve: B Hydrograph Family: 5

T_o/T_p = 1			T_o/T_p = 1.5			T_o/T_p = 2			T_o/T_p = 3		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	.000	.000
2	0.26	.021	2	0.25	.013	2	0.25	.010	2	.034	.010
3	0.52	.106	3	0.50	.065	3	0.50	.048	3	0.68	.068
4	0.78	.289	4	0.75	.173	4	0.75	.127	4	1.02	.150
5	1.04	.530	5	1.00	.306	5	1.00	.227	5	1.36	.229
6	1.30	.740	6	1.25	.434	6	1.25	.318	6	1.70	.283
7	1.56	.848	7	1.50	.562	7	1.50	.389	7	2.04	.315
8	1.82	.767	8	1.75	.680	8	1.75	.448	8	2.38	.339
9	2.08	.590	9	2.00	.737	9	2.00	.523	9	2.72	.378
10	2.34	.406	10	2.25	.673	10	2.25	.609	10	3.06	.459
11	2.60	.279	11	2.50	.530	11	2.50	.642	11	3.40	.509
12	2.86	.193	12	2.75	.381	12	2.75	.576	12	3.74	.446
13	3.12	.134	13	3.00	.262	13	3.00	.450	13	4.08	.310
14	3.38	.092	14	3.25	.185	14	3.25	.322	14	4.42	.190
15	3.64	.065	15	3.50	.129	15	3.50	.222	15	4.76	.117
16	3.90	.044	16	3.75	.090	16	3.75	.156	16	5.10	.069
17	4.16	.030	17	4.00	.063	17	4.00	.109	17	5.44	.040
18	4.42	.021	18	4.25	.045	18	4.25	.075	18	5.78	.025
19	4.68	.015	19	4.50	.031	19	4.50	.053	19	6.12	.016
20	4.94	.009	20	4.75	.022	20	4.75	.037	20	6.46	.009
21	5.20	.005	21	5.00	.014	21	5.00	.025	21	6.80	.005
22	5.46	.002	22	5.25	.009	22	5.25	.017	22	7.14	.003
23	5.72	.000	23	5.50	.005	23	5.50	.011	23	7.48	.001
			24	5.75	.003	24	5.75	.007	24	7.82	.000
			25	6.00	.001	25	6.00	.004			
			26	6.25	.000	26	6.25	.002			
						27	6.50	.001			
						28	6.75	.000			

Table 3.21-7 (Sheet 37 of 52)

Hydrograph Coordinates

Storm Distribution Curve: B Hydrograph Family: 5

T_o/T_p = 4			T_o/T_p = 6			T_o/T_p = 10			T_o/T_p = 16		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.36	.010	2	0.52	.015	2	0.67	.013	2	0.80	.008
3	0.72	.053	3	1.04	.070	3	1.34	.061	3	1.60	.046
4	1.08	.124	4	1.56	.130	4	2.01	.091	4	2.40	.060
5	1.44	.181	5	2.08	.159	5	2.68	.102	5	3.20	.065
6	1.80	.220	6	2.60	.172	6	3.35	.107	6	4.00	.067
7	2.16	.243	7	3.12	.178	7	4.02	.110	7	4.80	.067
8	2.52	.256	8	3.64	.182	8	4.69	.111	8	5.60	.068
9	2.88	.263	9	4.16	.183	9	5.36	.111	9	6.40	.068
10	3.24	.273	10	4.68	.184	10	6.03	.112	10	7.20	.068
11	3.60	.308	11	5.20	.218	11	6.70	.112	11	8.00	.068
12	3.96	.380	12	5.72	.285	12	7.37	.112	12	8.80	.068
13	4.32	.427	13	6.24	.324	13	8.04	.116	13	9.60	.068
14	4.68	.377	14	6.76	.267	14	8.71	.160	14	10.40	.068
15	5.04	.260	15	7.28	.133	15	9.38	.198	15	11.20	.068
16	5.40	.155	16	7.80	.064	16	10.05	.212	16	12.00	.068
17	5.76	.094	17	8.32	.029	17	10.72	.168	17	12.80	.086
18	6.12	.055	18	8.84	.016	18	11.39	.074	18	13.60	.121
19	6.48	.032	19	9.36	.007	19	12.06	.027	19	14.40	.133
20	6.84	.019	20	9.88	.003	20	12.73	.010	20	15.20	.136
21	7.20	.012	21	10.40	.001	21	13.40	.005	21	16.00	.137
22	7.56	.007	22	10.92	.000	22	14.07	.002	22	16.80	.098
23	7.92	.004				23	14.74	.000	23	17.60	.033
24	8.28	.002							24	18.40	.012
25	8.64	.000							25	19.20	.004
									26	20.00	.001
									27	20.80	.000

Hydrograph Coordinates

Storm Distribution Curve: B Hydrograph Family: 5

T_o/T_p = 25			T_o/T_p =			T_o/T_p =			T_o/T_p =		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000									
2	1.25	.015									
3	2.50	.039									
4	3.75	.043									
5	5.00	.044									
6	6.25	.044									
7	7.50	.044									
8	8.75	.044									
9	10.00	.044									
10	11.25	.044									
11	12.50	.044									
12	13.75	.044									
13	15.00	.044									
14	16.25	.044									
15	17.50	.044									
16	18.75	.045									
17	20.00	.067									
18	21.25	.083									
19	22.50	.087									
20	23.75	.087									
21	25.00	.088									
22	26.25	.035									
23	27.50	.006									
24	28.75	.002									
25	30.00	.000									

Table 3.21-7 (Sheet 39 of 52)

Hydrograph Coordinates

Storm Distribution Curve: C Hydrograph Family: 1

$T_o/T_p = 1$			$T_o/T_p = 1.5$			$T_o/T_p = 2$			$T_o/T_p = 3$		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.25	.007	2	0.30	.007	2	0.30	.005	2	0.30	.002
3	0.50	.047	3	0.60	.037	3	0.60	.021	3	0.60	.008
4	0.75	.196	4	0.90	.150	4	0.90	.077	4	0.90	.056
5	1.00	.485	5	1.20	.397	5	1.20	.190	5	1.20	.077
6	1.25	.804	6	1.50	.730	6	1.50	.409	6	1.50	.133
7	1.50	.933	7	1.80	.866	7	1.80	.671	7	1.80	.227
8	1.75	.837	8	2.10	.758	8	2.10	.800	8	2.10	.371
9	2.00	.641	9	2.40	.556	9	2.40	.722	9	2.40	.590
10	2.25	.445	10	2.70	.352	10	2.70	.533	10	2.70	.704
11	2.50	.311	11	3.00	.234	11	3.00	.378	11	3.00	.652
12	2.75	.219	12	3.30	.150	12	3.30	.249	12	3.30	.525
13	3.00	.150	13	3.60	.097	13	3.60	.160	13	3.60	.395
14	3.25	.107	14	3.90	.063	14	3.90	.103	14	3.90	.273
15	3.50	.075	15	4.20	.041	15	4.20	.068	15	4.20	.180
16	3.75	.054	16	4.50	.027	16	4.50	.044	16	4.50	.118
17	4.00	.036	17	4.80	.017	17	4.80	.029	17	4.80	.078
18	4.25	.026	18	5.10	.010	18	5.10	.018	18	5.10	.052
19	4.50	.017	19	5.40	.005	19	5.40	.012	19	5.40	.034
20	4.75	.011	20	5.70	.002	20	5.70	.007	20	5.70	.022
21	5.00	.007	21	6.00	.001	21	6.00	.004	21	6.00	.013
22	5.25	.003	22	6.30	.000	22	6.30	.002	22	6.30	.008
23	5.50	.001				23	6.60	.001	23	6.60	.005
24	5.75	.000				24	6.90	.000	24	6.90	.003
									25	7.20	.002
									26	7.50	.001
									27	7.80	.000

Hydrograph Coordinates

Storm Distribution Curve: C Hydrograph Family: 1

T_o/T_p = 4			T_o/T_p = 6			T_o/T_p = 10			T_o/T_p = 16		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.42	.002	2	0.51	.002	2	0.70	.003	2	0.98	.003
3	0.84	.014	3	1.02	.016	3	1.40	.016	3	1.96	.010
4	1.26	.054	4	1.53	.041	4	2.10	.030	4	2.94	.019
5	1.68	.100	5	2.04	.069	5	2.80	.045	5	3.92	.028
6	2.10	.157	6	2.55	.099	6	3.50	.061	6	4.90	.037
7	2.52	.252	7	3.06	.134	7	4.20	.078	7	5.88	.046
8	2.94	.498	8	3.57	.195	8	4.90	.096	8	6.86	.056
9	3.36	.636	9	4.08	.400	9	5.60	.117	9	7.84	.067
10	3.78	.530	10	4.59	.538	10	6.30	.259	10	8.82	.086
11	4.20	.386	11	5.10	.413	11	7.00	.430	11	9.80	.202
12	4.62	.261	12	5.61	.287	12	7.70	.296	12	10.78	.328
13	5.04	.156	13	6.12	.201	13	8.40	.184	13	11.76	.186
14	5.46	.085	14	6.63	.136	14	9.10	.123	14	12.74	.105
15	5.88	.047	15	7.14	.073	15	9.80	.089	15	13.72	.072
16	6.30	.026	16	7.65	.034	16	10.50	.058	16	14.70	.055
17	6.72	.014	17	8.16	.016	17	11.20	.029	17	15.68	.046
18	7.14	.007	18	8.67	.007	18	11.90	.011	18	16.66	.030
19	7.56	.004	19	9.18	.004	19	12.60	.005	19	17.64	.010
20	7.98	.002	20	9.69	.002	20	13.30	.002	20	18.62	.003
21	8.40	.001	21	10.20	.001	21	14.00	.001	21	19.60	.001
22	8.82	.000	22	10.71	.000	22	14.70	.000	22	20.58	.000

Table 3.21-7 (Sheet 41 of 52)

Hydrograph Coordinates

Storm Distribution Curve: C Hydrograph Family: 1

T_o/T_p = 25			T_o/T_p = 36			T_o/T_p =			T_o/T_p =		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000						
2	1.09	.002	2	1.55	.002						
3	2.18	.005	3	3.10	.004						
4	3.27	.009	4	4.65	.007						
5	4.36	.014	5	6.20	.010						
6	5.45	.018	6	7.75	.013						
7	6.54	.023	7	9.30	.016						
8	7.63	.027	8	10.85	.019						
9	8.72	.032	9	12.40	.022						
10	9.81	.036	10	13.95	.026						
11	10.90	.041	11	15.50	.029						
12	11.99	.045	12	17.05	.032						
13	13.08	.052	13	18.60	.037						
14	14.17	.075	14	20.15	.051						
15	15.26	.185	15	21.70	.142						
16	16.35	.237	16	23.25	.177						
17	17.44	.142	17	24.80	.083						
18	18.53	.076	18	26.35	.044						
19	19.62	.053	19	27.90	.035						
20	20.71	.043	20	29.45	.029						
21	21.80	.037	21	31.00	.025						
22	22.89	.031	22	32.55	.021						
23	23.98	.027	23	34.10	.019						
24	25.07	.024	24	35.65	.018						
25	26.16	.013	25	37.20	.008						
26	27.25	.003	26	38.75	.002						
27	28.34	.001	27	40.30	.000						
28	29.43	.000									

Hydrograph Coordinates

Storm Distribution Curve: C Hydrograph Family: 2

T_o/T_p = 1			T_o/T_p = 1.5			T_o/T_p = 2			T_o/T_p = 3		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.25	.004	2	0.25	.002	2	0.34	.003	2	0.28	.001
3	0.50	.047	3	0.50	.014	3	0.68	.027	3	0.56	.006
4	0.75	.208	4	0.75	.066	4	1.02	.091	4	0.84	.017
5	1.00	.511	5	1.00	.219	5	1.36	.320	5	1.12	.040
6	1.25	.819	6	1.25	.483	6	1.70	.649	6	1.40	.087
7	1.50	.929	7	1.50	.752	7	2.04	.800	7	1.68	.200
8	1.75	.826	8	1.75	.867	8	2.38	.702	8	1.96	.402
9	2.00	.628	9	2.00	.808	9	2.72	.516	9	2.24	.620
10	2.25	.438	10	2.25	.647	10	3.06	.328	10	2.52	.707
11	2.50	.306	11	2.50	.470	11	3.40	.203	11	2.80	.661
12	2.75	.215	12	2.75	.333	12	3.74	.127	12	3.08	.559
13	3.00	.148	13	3.00	.232	13	4.08	.079	13	3.36	.450
14	3.25	.106	14	3.25	.161	14	4.42	.047	14	3.64	.348
15	3.50	.073	15	3.50	.113	15	4.76	.029	15	3.92	.250
16	3.75	.053	16	3.75	.080	16	5.10	.019	16	4.20	.170
17	4.00	.036	17	4.00	.055	17	5.44	.011	17	4.48	.114
18	4.25	.026	18	4.25	.039	18	5.78	.005	18	4.76	.075
19	4.50	.017	19	4.50	.027	19	6.12	.002	19	5.04	.052
20	4.75	.011	20	4.75	.019	20	6.46	.001	20	5.32	.035
21	5.00	.007	21	5.00	.012	21	6.80	.000	21	5.60	.023
22	5.25	.004	22	5.25	.007				22	5.88	.016
23	5.50	.002	23	5.50	.004				23	6.16	.010
24	5.75	.000	24	5.75	.002				24	6.44	.006
			25	6.00	.001				25	6.72	.004
			26	6.25	.000				26	7.00	.003
									27	7.28	.002
									28	7.56	.001
									29	7.84	.000

Table 3.21-7 (Sheet 43 of 52)

Hydrograph Coordinates

Storm Distribution Curve: C Hydrograph Family: 2

$T_o/T_p = 4$			$T_o/T_p = 6$			$T_o/T_p = 10$			$T_o/T_p = 16$		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.34	.001	2	0.41	.001	2	0.52	.001	2	0.95	.001
3	0.68	.006	3	0.82	.005	3	1.04	.003	3	1.90	.004
4	1.02	.018	4	1.23	.014	4	1.56	.010	4	2.85	.011
5	1.36	.040	5	1.64	.026	5	2.08	.017	5	3.80	.018
6	1.70	.068	6	2.05	.044	6	2.60	.026	6	4.75	.026
7	2.04	.130	7	2.46	.070	7	3.12	.036	7	5.70	.036
8	2.38	.310	8	2.87	.110	8	3.64	.049	8	6.65	.050
9	2.72	.532	9	3.28	.224	9	4.16	.064	9	7.60	.075
10	3.06	.638	10	3.69	.431	10	4.68	.087	10	8.55	.220
11	3.40	.571	11	4.10	.550	11	5.20	.160	11	9.50	.330
12	3,74	.460	12	4.51	.477	12	5.72	.335	12	10.45	.208
13	4.08	.368	13	4.92	.360	13	6.24	.438	13	11.40	.119
14	4.42	.289	14	5.33	.276	14	6.76	.356	14	12.35	.085
15	4.76	.207	15	5.74	.220	15	7.28	.245	15	13.30	.067
16	5.10	.132	16	6.15	.180	16	7.80	.182	16	14.25	.058
17	5.44	.082	17	6.56	.138	17	8.32	.143	17	15.20	.052
18	5.78	.050	18	6.97	.085	18	8.84	.117	18	16.15	.045
19	6.12	.030	19	7.38	.048	19	9.36	.100	19	17.10	.021
20	6.46	.018	20	7.79	.026	20	9.88	.087	20	18.05	.005
21	6.80	.011	21	8.20	.013	21	10.40	.074	21	19.00	.002
22	7.14	.006	22	8.61	.007	22	10.92	.041	22	19.95	.001
23	7.48	.004	23	9.02	.004	23	11.44	.020	23	20.90	.000
24	7.82	.003	24	9.43	.003	24	11.96	.012			
25	8.16	.002	25	9.84	.002	25	12.48	.006			
26	8.50	.001	26	10.25	.001	26	13.00	.003			
27	8.84	.000	27	10.66	.000	27	13.52	.002			
						28	14.04	.001			
						29	14.56	.000			

Table 3.21-7 (Sheet 44 of 52)

Hydrograph Coordinates

Storm Distribution Curve: C Hydrograph Family: 2

T_o/T_p = 25			T_o/T_p = 36			T_o/T_p =			T_o/T_p =		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000						
2	1.03	.001	2	1.70	.001						
3	2.06	.002	3	3.40	.003						
4	3.09	.004	4	5.10	.005						
5	4.12	.008	5	6.80	.008						
6	5.15	.012	6	8.50	.011						
7	6.18	.016	7	10.20	.014						
8	7.21	.020	8	11.90	.018						
9	8.24	.024	9	13.60	.023						
10	9.27	.028	10	15.30	.030						
11	10.30	.034	11	17.00	.043						
12	11.33	.046	12	18.70	.143						
13	12.36	.090	13	20.40	.169						
14	13.39	.197	14	22.10	.091						
15	14.42	.233	15	23.80	.046						
16	15.45	.160	16	25.50	.037						
17	16.48	.088	17	27.20	.032						
18	17.51	.062	18	28.90	.028						
19	18.54	.051	19	30.60	.026						
20	19.57	.046	20	32.30	.024						
21	20.60	.041	21	34.00	.022						
22	21.63	.038	22	35.70	.020						
23	22.66	.035	23	37.40	.006						
24	23.69	.032	24	39.10	.001						
25	24.72	.030	25	40.80	.000						
26	25.75	.019									
27	26.78	.005									
28	27.81	.002									
29	28.84	.001									
30	29.87	.000									

Table 3.21-7 (Sheet 45 of 52)

Hydrograph Coordinates

Storm Distribution Curve: C Hydrograph Family: 3

T_o/T_p = 1			T_o/T_p = 1.5			T_o/T_p = 2			T_o/T_p - 3		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.23	.005	2	0.27	.003	2	0.31	.003	2	0.28	.002
3	0.46	.061	3	0.54	.030	3	0.62	.017	3	0.56	.007
4	0.69	.243	4	0.81	.150	4	0.93	.130	4	0.84	.020
5	0.92	.537	5	1.08	.454	5	1.24	.380	5	1.12	.066
6	1.15	.796	6	1.35	.715	6	1.55	.660	6	1.40	.191
7	1.38	.907	7	1.62	.834	7	1.86	.760	7	1.68	.397
8	1.61	.844	8	1.89	.780	8	2.17	.690	8	1.96	.600
9	1.84	.679	9	2.16	.620	9	2.48	.561	9	2.24	.660
10	2.07	.501	10	2.43	.458	10	2.79	.410	10	2.52	.616
11	2.30	.369	11	2.70	.317	11	3.10	.274	11	2.80	.527
12	2.53	.265	12	2.97	.213	12	3.41	.173	12	3.08	.448
13	2.76	.189	13	3.24	.146	13	3.72	.112	13	3.36	.375
14	2.99	.135	14	3.51	.100	14	4.03	.073	14	3.64	.296
15	3.22	.097	15	3.78	.069	15	4.34	.047	15	3.92	.216
16	3.45	.070	16	4.05	.046	16	4.65	.029	16	4.20	.146
17	3.68	.050	17	4.32	.031	17	4.96	.018	17	4.48	.099
18	3.91	.036	18	4.59	.021	18	5.27	.010	18	4.76	.065
19	4.14	.026	19	4.86	.014	19	5.58	.005	19	5.04	.043
20	4.37	.018	20	5.13	.008	20	5.89	.003	20	5.32	.027
21	4.60	.012	21	5.40	.004	21	6.20	.002	21	5.60	.018
22	4.83	.008	22	5.67	.002	22	6.51	.001	22	5.88	.013
23	5.06	.005	23	5.94	.001	23	6.82	.000	23	6.16	.009
24	5.29	.003	24	6.21	.000				24	6.44	.006
25	5.52	.001							25	6.72	.004
26	5.75	.000							26	7.00	.003
									27	7.28	.002
									28	7.56	.001
									29	7.84	.000

Hydrograph Coordinates

Storm Distribution Curve: C Hydrograph Family: 3

T_o/T_p = 4			T_o/T_p = 6			T_o/T_p = 10			T_o/T_p = 16		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.33	.002	2	0.38	.002	2	0.51	.001	2	0.77	.001
3	0.66	.008	3	0.76	.005	3	1.02	.003	3	1.54	.003
4	0.99	.019	4	1.14	.014	4	1.53	.009	4	2.31	.008
5	1.32	.050	5	1.52	.026	5	2.04	.018	5	3.08	.015
6	1.65	.130	6	1.90	.050	6	2.55	.029	6	3.85	.024
7	1.98	.320	7	2.28	.106	7	3.06	.045	7	4.62	.033
8	2.31	.519	8	2.66	.240	8	3.57	.080	8	5.39	.052
9	2.64	.594	9	3.04	.418	9	4.08	.186	9	6.16	.139
10	2.97	.531	10	3.42	.502	10	4.59	.328	10	6.93	.239
11	3.30	.439	11	3.80	.449	11	5.10	.400	11	7.70	.302
12	3.63	.367	12	4.18	.361	12	5.61	.326	12	8.47	.203
13	3.96	.305	13	4.56	.290	13	6.12	.232	13	9.24	.132
14	4.29	.256	14	4.94	.238	14	6.63	.180	14	10.01	.101
15	4.62	.202	15	5.32	.201	15	7.14	.148	15	10.78	.083
16	4.95	.140	16	5.70	.174	16	7.65	.125	16	11.55	.072
17	5.28	.088	17	6.08	.152	17	8.16	.107	17	12.32	.064
18	5.61	.057	18	6.46	.127	18	8.67	.095	18	13.09	.058
19	5.94	.038	19	6.84	.094	19	9.18	.086	19	13.86	.053
20	6.27	.025	20	7.22	.053	20	9.69	.079	20	14.63	.049
21	6.60	.015	21	7.60	.031	21	10.20	.073	21	15.40	.046
22	6.93	.008	22	7.98	.017	22	10.71	.056	22	16.17	.043
23	7.26	.005	23	8.36	.010	23	11.22	.030	23	16.94	.026
24	7.59	.003	24	8.74	.006	24	11.73	.016	24	17.71	.009
25	7.92	.002	25	9.12	.004	25	12.24	.008	25	18.48	.004
26	8.25	.001	26	9.50	.002	26	12.75	.004	26	19.25	.002
27	8.58		27	9.88	.001	27	13.26	.002	27	20.02	.001
28	8.91	.000	28	10.26	.	28	13.77	.001	28	20.79	.000
			29	10.64	.000	29	14.28				
						30	14.79	.000			

Table 3.21-7 (Sheet 47 of 52)

Hydrograph Coordinates

Storm Distribution Curve: C Hydrograph Family: 3

T_o/T_p = 25			T_o/T_p =			T_o/T_p =			T_o/T_p =		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000									
2	1.16	.001									
3	2.32	.003									
4	3.48	.007									
5	4.64	.012									
6	5.80	.018									
7	6.96	.025									
8	8.12	.033									
9	9.28	.107									
10	10.44	.171									
11	11.60	.214									
12	12.76	.124									
13	13.92	.073									
14	15.08	.057									
15	16.24	.049									
16	17.40	.045									
17	18.56	.041									
18	19.72	.037									
19	20.88	.034									
20	22.04	.032									
21	23.20	.030									
22	24.36	.028									
23	25.52	.024									
24	26.68	.007									
25	27.84	.003									
26	29.00	.001									
27	30.16	.000									

Hydrograph Coordinates

Storm Distribution Curve: C Hydrograph Family: 4

$T_o/T_p = 1$			$T_o/T_p = 1.5$			$T_o/T_p = 2$			$T_o/T_p = 3$		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.23	.018	2	0.25	.004	2	0.27	.004	2	0.27	.003
3	0.46	.112	3	0.50	.064	3	0.54	.080	3	0.54	.013
4	0.69	.294	4	0.75	.241	4	0.81	.240	4	0.81	.059
5	0.92	.640	5	1.00	.511	5	1.08	.422	5	1.08	.175
6	1.15	.816	6	1.25	.718	6	1.35	.563	6	1.35	.396
7	1.38	.887	7	1.50	.787	7	1.62	.712	7	1.62	.548
8	1.61	.799	8	1.75	.745	8	1.89	.677	8	1.89	.613
9	1.84	.650	9	2.00	.632	9	2.16	.563	9	2.16	.579
10	2.07	.478	10	2.25	.499	10	2.43	.502	10	2.43	.509
11	2.30	.347	11	2.50	.369	11	2.70	.391	11	2.70	.444
12	2.53	.250	12	2.75	.259	12	2.97	.280	12	2.90	.401
13	2.76	.178	13	3.00	.180	13	3.24	.196	13	3.24	.337
14	2.99	.128	14	3.25	.127	14	3.51	.134	14	3.51	.280
15	3.22	.092	15	3.50	.089	15	3.78	.090	15	3.78	.218
16	3.45	.067	16	3.75	.060	16	4.05	.062	16	4.05	.155
17	3.68	.049	17	4.00	.043	17	4.32	.041	17	4.32	.107
18	3.91	.035	18	4.25	.030	18	4.59	.029	18	4.59	.071
19	4.14	.025	19	4.50	.021	19	4.86	.018	19	4.86	.047
20	4.37	.018	20	4.75	.014	20	5.13	.011	20	5.13	.032
21	4.60	.012	21	5.00	.009	21	5.40	.007	21	5.40	.020
22	4.83	.008	22	5.25	.005	22	5.67	.004	22	5.67	.014
23	5.06	.005	23	5.50	.003	23	5.94	.003	23	5.94	.009
24	5.29	.003	24	5.75	.002	24	6.21	.002	24	6.21	.006
25	5.52	.001	25	6.00	.001	25	6.48	.001	25	6.48	.004
26	5.75	.000	26	6.25	.000	26	6.75	.000	26	6.75	.003
									27	7.02	.002
									28	7.29	.001
									29	7.56	
									30	7.83	.000

Table 3.21-7 (Sheet 49 of 52)

Hydrograph Coordinates

Storm Distribution Curve: C Hydrograph Family: 4

T_o/T_p = 4			T_o/T_p = 6			T_o/T_p = 10			T_o/T_p = 16		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.36	.003	2	0.46	.004	2	0.66	.006	2	0.72	.005
3	0.72	.018	3	0.92	.015	3	1.32	.016	3	1.44	.011
4	1.08	.080	4	1.38	.050	4	1.98	.030	4	2.16	.017
5	1.44	.229	5	1.84	.170	5	2.64	.113	6	2.88	.023
6	1.80	.447	6	2.30	.370	6	3.30	.263	7	3.60	.058
7	2.16	.546	7	2.76	.466	7	3.96	.364	8	4.32	.137
8	2.52	.500	8	3.22	.390	8	4.62	.272	9	5.04	.214
9	2.88	.415	9	3.68	.304	9	5.28	.190	10	5.76	.276
10	3.24	.350	10	4.14	.244	10	5.94	.148	11	6.48	.214
11	3.60	.290	11	4.60	.205	11	6.60	.123	12	7.20	.136
12	3.96	.249	12	5.06	.175	12	7.26	.105	13	7.92	.105
13	4.32	.212	13	5.52	.150	13	7.92	.092	14	8.64	.087
14	4.68	.165	14	5.98	.134	14	8.58	.083	15	9.36	.077
15	5.04	.108	15	6.44	.110	15	9.24	.076	16	10.08	.071
16	5.40	.064	16	6.90	.074	16	9.90	.070	17	10.80	.065
17	5.76	.037	17	7.36	.040	17	10.56	.058	18	11.52	.059
18	6.12	.020	18	7.82	.020	18	11.22	.027	19	12.24	.054
19	6.48	.012	19	8.28	.012	19	11.88	.012	20	12.96	.051
20	6.84	.007	20	8.74	.006	20	12.54	.005	21	13.68	.048
21	7.20	.004	21	9.20	.003	21	13.20	.002	22	14.40	.045
22	7.56	.002	22	9.66	.001	22	13.86	.001	23	15.12	.042
23	7.92	.001	23	10.12		23	14.52	.000	24	15.84	.039
24	8.28		24	10.58	.000				25	16.56	.033
25	8.64	.000							26	17.28	.016
									27	18.00	.006
									28	18.72	.003
									29	19.44	.002
									30	20.16	.001
									31	20.88	.000

Hydrograph Coordinates

Storm Distribution Curve: C Hydrograph Family: 4

T_o/T_p = 25			T_o/T_p =			T_o/T_p =			T_o/T_p =		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000									
2	1.10	.004									
3	2.20	.009									
4	3.30	.014									
5	4.40	.019									
6	5.50	.055									
7	6.60	.104									
8	7.70	.155									
9	8.80	.202									
10	9.90	.104									
11	11.00	.070									
12	12.10	.059									
13	13.20	.052									
14	14.30	.048									
15	15.40	.044									
16	16.50	.040									
17	17.60	.037									
18	18.70	.034									
19	19.80	.031									
20	20.90	.030									
21	22.00	.029									
22	23.10	.028									
23	24.20	.027									
24	25.30	.025									
25	26.40	.008									
26	27.50	.002									
27	28.60	.001									
28	29.70	.000									

Table 3.21-7 (Sheet 51 of 52)

Hydrograph Coordinates

Storm Distribution Curve: C Hydrograph Family: 5

$T_o/T_p = 1$			$T_o/T_p = 1.5$			$T_o/T_p = 2$			$T_o/T_p = 3$		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.26	.050	2	0.28	.045	2	0.25	.018	2	0.32	.016
3	0.52	.228	3	0.56	.198	3	0.50	.101	3	0.64	.120
4	0.78	.532	4	0.84	.453	4	0.75	.273	4	0.96	.313
5	1.04	.761	5	1.12	.641	5	1.00	.472	5	1.28	.476
6	1.30	.843	6	1.40	.716	6	1.25	.599	6	1.60	.523
7	1.56	.765	7	1.68	.686	7	1.50	.635	7	1.92	.489
8	1.82	.614	8	1.96	.599	8	1.75	.609	8	2.24	.438
9	2.08	.440	9	2.24	.474	9	2.00	.559	9	2.56	.392
10	2.34	.300	10	2.52	.340	10	2.25	.506	10	2.88	.350
11	2.60	.210	11	2.80	.228	11	2.50	.437	11	3.20	.310
12	2.86	.144	12	3.08	.153	12	2.75	.350	12	3.52	.267
13	3.12	.099	13	3.36	.103	13	3.00	.260	13	3.84	.203
14	3.38	.069	14	3.64	.068	14	3.25	.185	14	4.16	.129
15	3.64	.047	15	3.92	.046	15	3.50	.128	15	4.48	.082
16	3.90	.033	16	4.20	.031	16	3.75	.090	16	4.80	.051
17	4.16	.023	17	4.48	.021	17	4.00	.063	17	5.12	.032
18	4.42	.015	18	4.76	.013	18	4.25	.043	18	5.44	.020
19	4.68	.009	19	5.04	.008	19	4.50	.029	19	5.76	.013
20	4.94	.005	20	5.32	.004	20	4.75	.020	20	6.08	.009
21	5.20	.003	21	5.60	.002	21	5.00	.013	21	6.40	.006
22	5.46	.001	22	5.88	.001	22	5.25	.008	22	6.72	.003
23	5.72	.000	23	6.16	.000	23	5.50	.005	23	7.04	.001
						24	5.75	.003	24	7.36	
						25	6.00	.002	25	7.68	.000
						26	6.25	.001			
						27	6.50				
						28	6.75	.000			

Hydrograph Coordinates

Storm Distribution Curve: C Hydrograph Family: 5

T_o/T_p = 4			T_o/T_p = 6			T_o/T_p = 10			T_o/T_p = 16		
Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p	Line No.	t/T_p	q_c/q_p
1	0.00	.000	1	0.00	.000	1	0.00	.000	1	0.00	.000
2	0.33	.010	2	0.47	.017	2	0.50	.011	2	0.87	.026
3	0.66	.087	3	0.94	.110	3	1.00	.054	3	1.74	.070
4	0.99	.245	4	1.41	.279	4	1.50	.127	4	2.61	.154
5	1.32	.400	5	1.88	.383	5	2.00	.240	5	3.48	.231
6	1.65	.457	6	2.35	.336	6	2.50	.299	6	4.35	.153
7	1.98	.428	7	2.82	.285	7	3.00	.249	7	5.22	.108
8	2.31	.387	8	3.29	.247	8	3.50	.199	8	6.09	.095
9	2.64	.346	9	3.76	.215	9	4.00	.169	9	6.96	.086
10	2.97	.307	10	4.23	.187	10	4.50	.153	10	7.83	.079
11	3.30	.273	11	4.70	.166	11	5.00	.138	11	8.70	.073
12	3.63	.248	12	5.17	.151	12	5.50	.124	12	9.57	.067
13	3.96	.227	13	5.64	.140	13	6.00	.113	13	10.44	.061
14	4.29	.205	14	6.11	.130	14	6.50	.104	14	11.31	.055
15	4.62	.170	15	6.58	.110	15	7.00	.096	15	12.18	.051
16	4.95	.123	16	7.05	.067	16	7.50	.090	16	13.05	.050
17	5.28	.077	17	7.52	.035	17	8.00	.086	17	13.92	.049
18	5.61	.047	18	7.99	.017	18	8.50	.082	18	14.79	.048
19	5.94	.029	19	8.46	.009	19	9.00	.079	19	15.66	.047
20	6.27	.017	20	8.93	.006	20	9.50	.077	20	16.53	.039
21	6.60	.010	21	9.40	.004	21	10.00	.076	21	17.40	.016
22	6.93	.006	22	9.87	.002	22	10.50	.067	22	18.27	.005
23	7.26	.004	23	10.34	.001	23	11.00	.042	23	19.14	.002
24	7.59	.003	24	10.81	.000	24	11.50	.022	24	20.01	.001
25	7.92	.002				25	12.00	.010	25	20.88	.000
26	8.25	.001				26	12.50	.005			
27	8.58					27	13.00	.002			
28	8.91	.000				28	13.50	.001			
						29	14.00				
						30	14.50	.000			

HYDROLOGY: SIX-HOUR POINT RAINFALL, United States, except west.

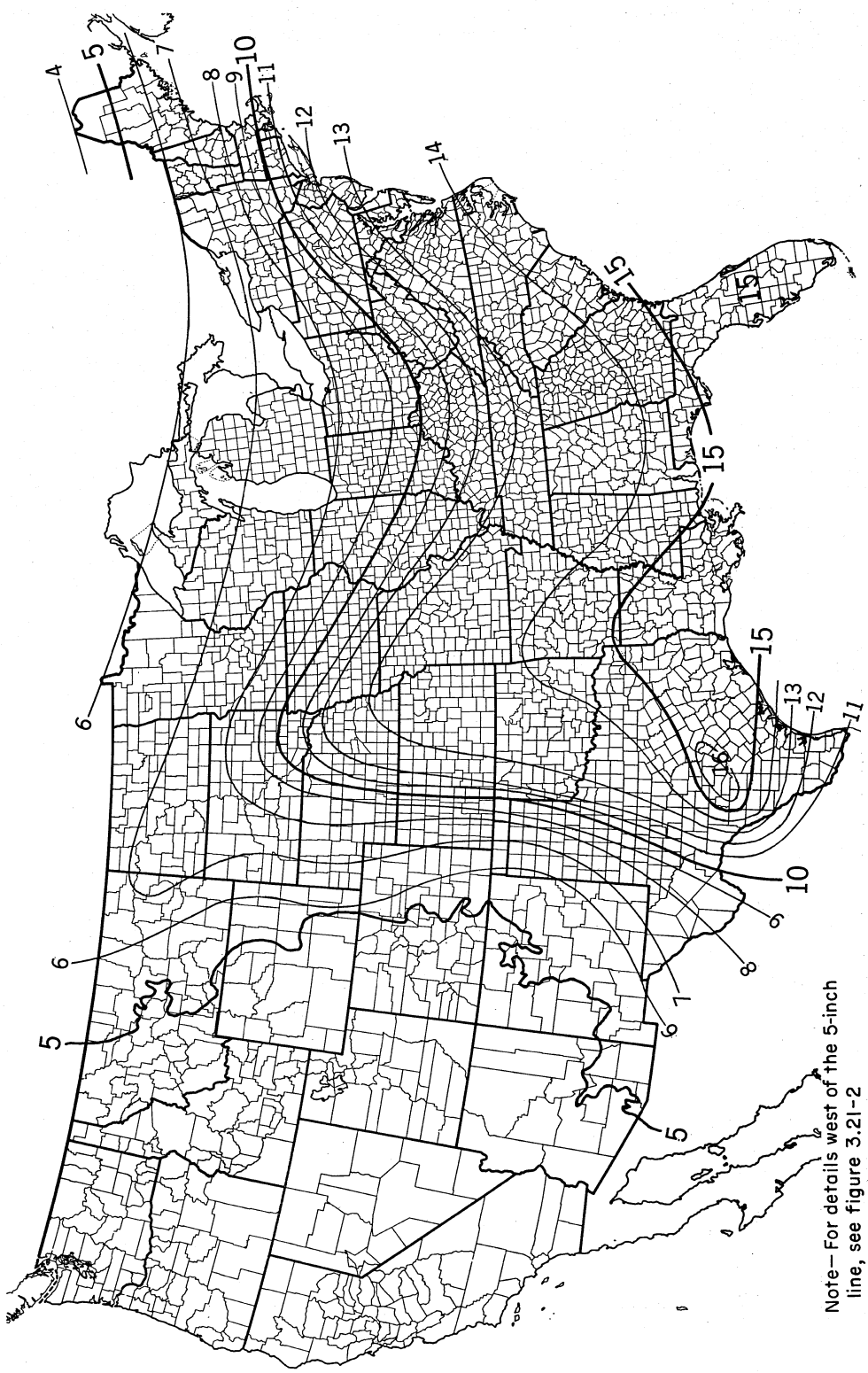


Figure 3.21-1

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - CENTRAL TECHNICAL UNIT	STANDARD DWG. NO. ES-1002 SHEET 1 OF 2 DATE 7-5-55

HYDROLOGY: SIX-HOUR POINT RAINFALL, Western U.S. and Puerto Rico.

Figure 3.21-2

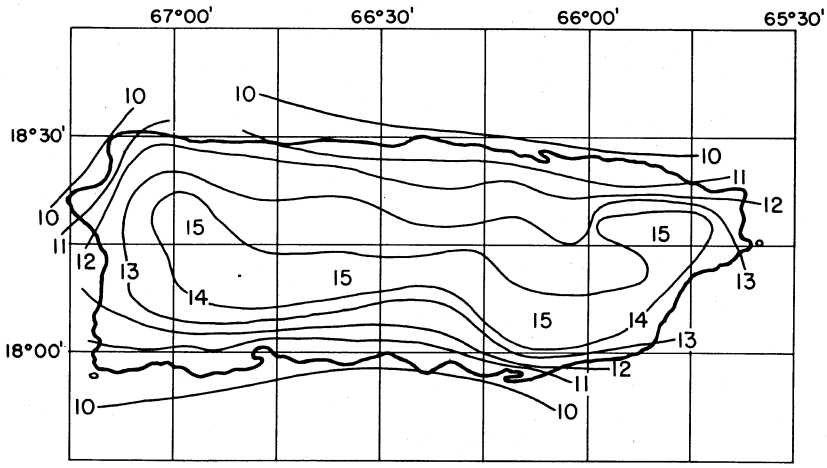


Figure 3.21-3

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - CENTRAL TECHNICAL UNIT	STANDARD DWG. NO. ES-1002 SHEET 2 OF 2 DATE 7-3-56

HYDROLOGY: DESIGN STORM CHARACTERISTICS, Area and Time distribution

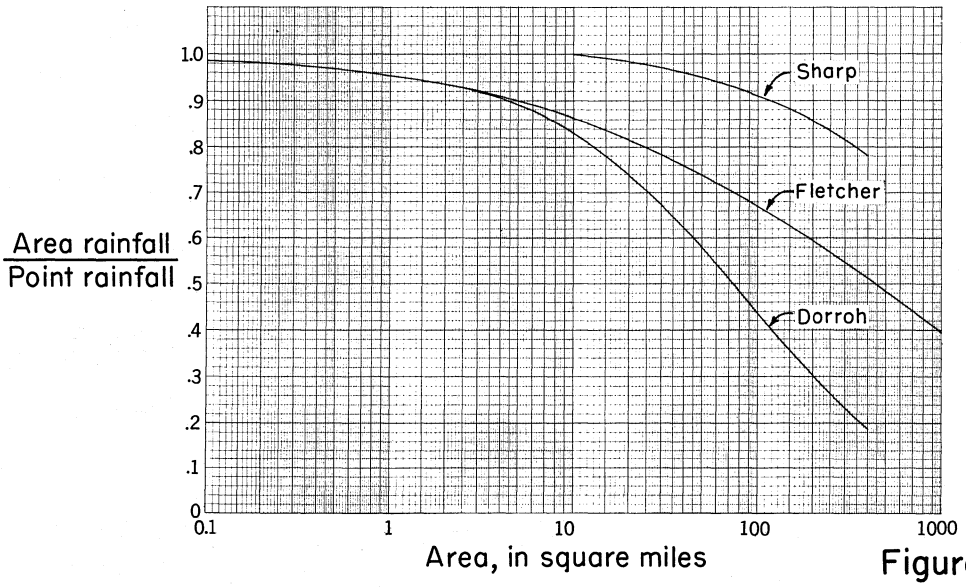


Figure 3.21-4

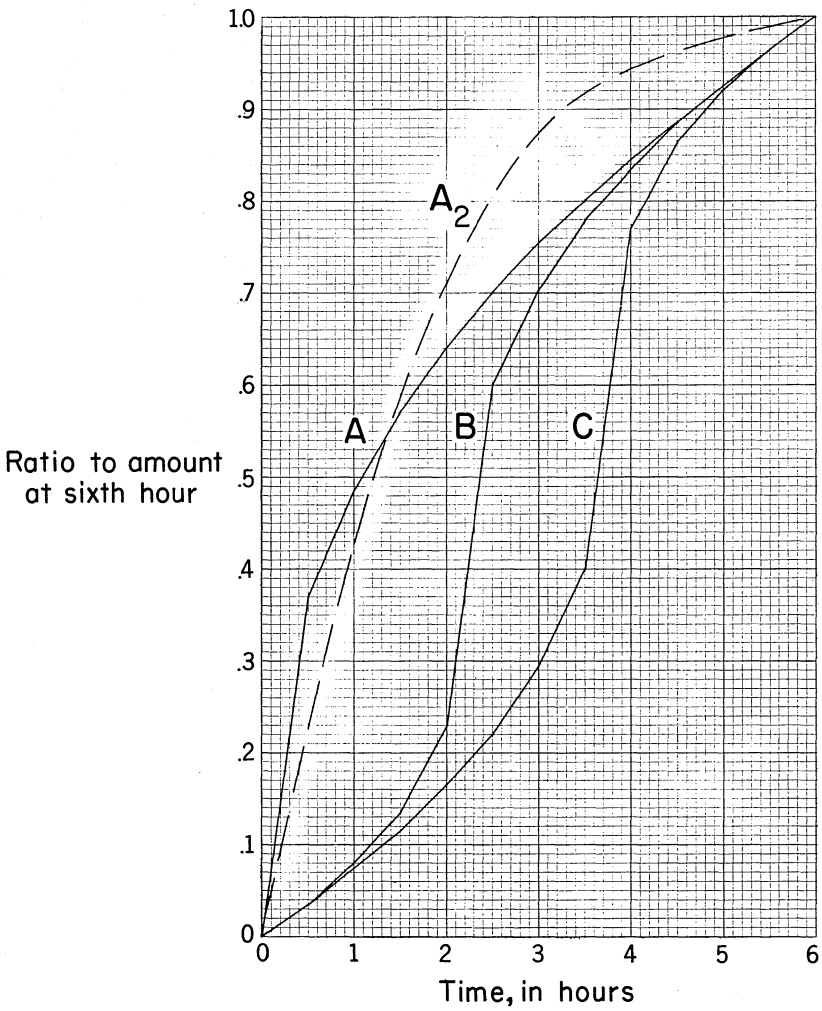


Figure 3.21-5

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
See text, Section 3.21 of Hydrology Guide, for references to figures 3.21-4 and 3.21-5	SOIL CONSERVATION SERVICE ENGINEERING DIVISION - CENTRAL TECHNICAL UNIT	ES-1003 SHEET 1 OF 1 DATE 7-2-56

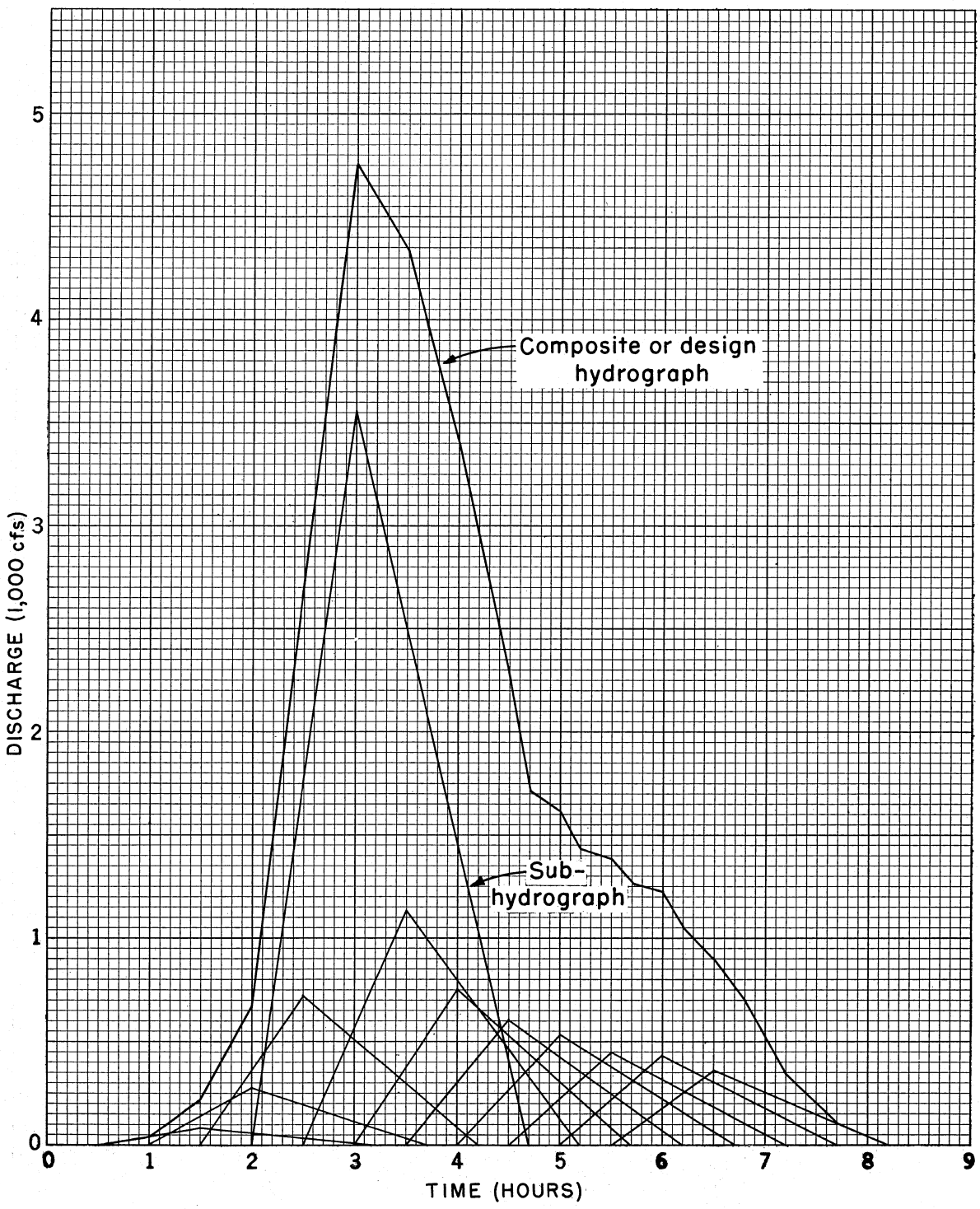

Composite or design hydrograph
Sub-hydrograph
DISCHARGE (1,000 cfs)
TIME (HOURS)
0
1
2
3
4
5
6
7
8
9

Figure 3.21-6

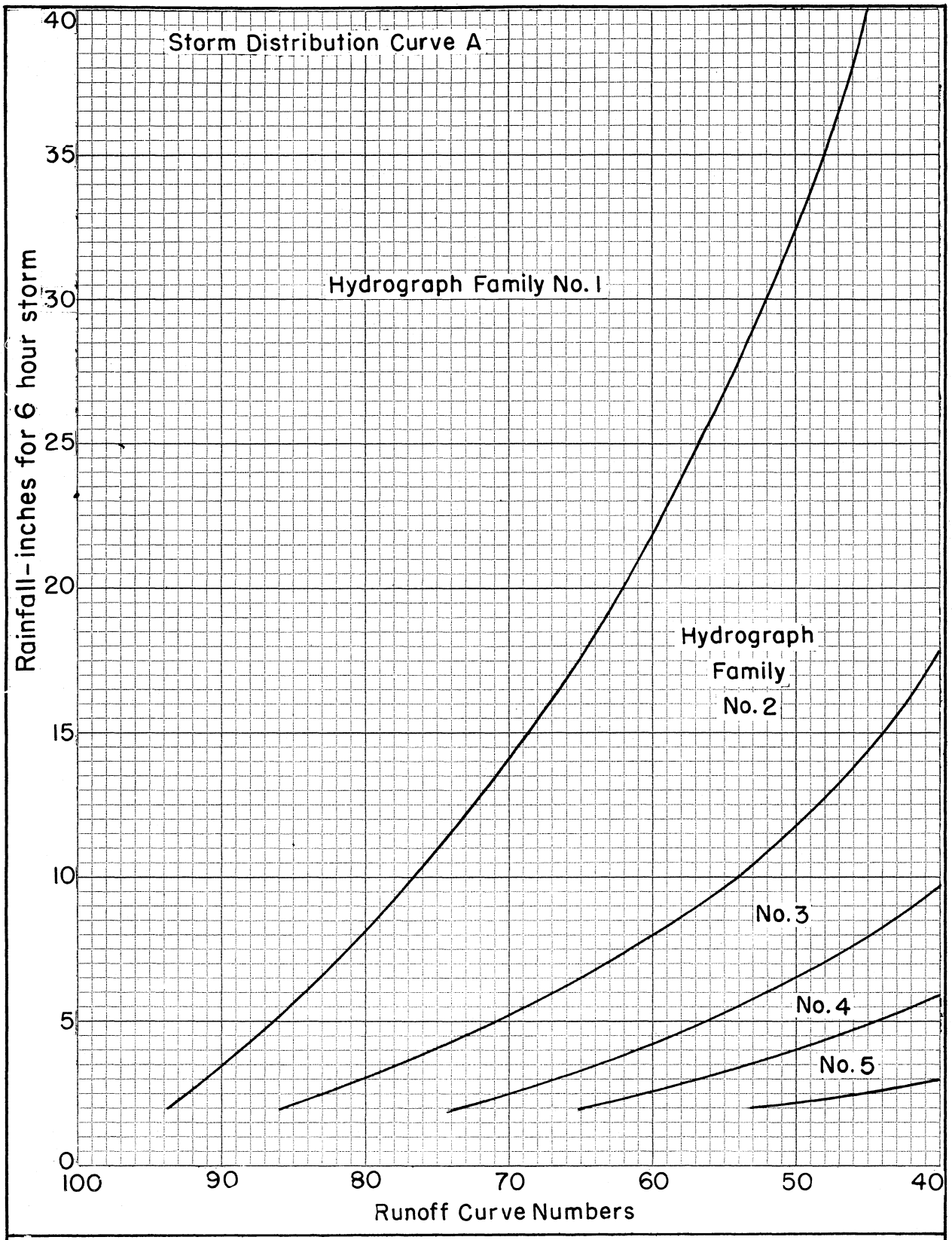


Chart for selecting a hydrograph family for a given 6 hour rainfall and runoff curve number.

Figure 3.21-7
(1 of 4)

COMPUTED M.H.K.	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
CHECKED	SOIL CONSERVATION SERVICE	ES-1011
DRAFTED	CENTRAL TECHNICAL UNIT	SHEET 1 OF 4
		DATE 2-14-58

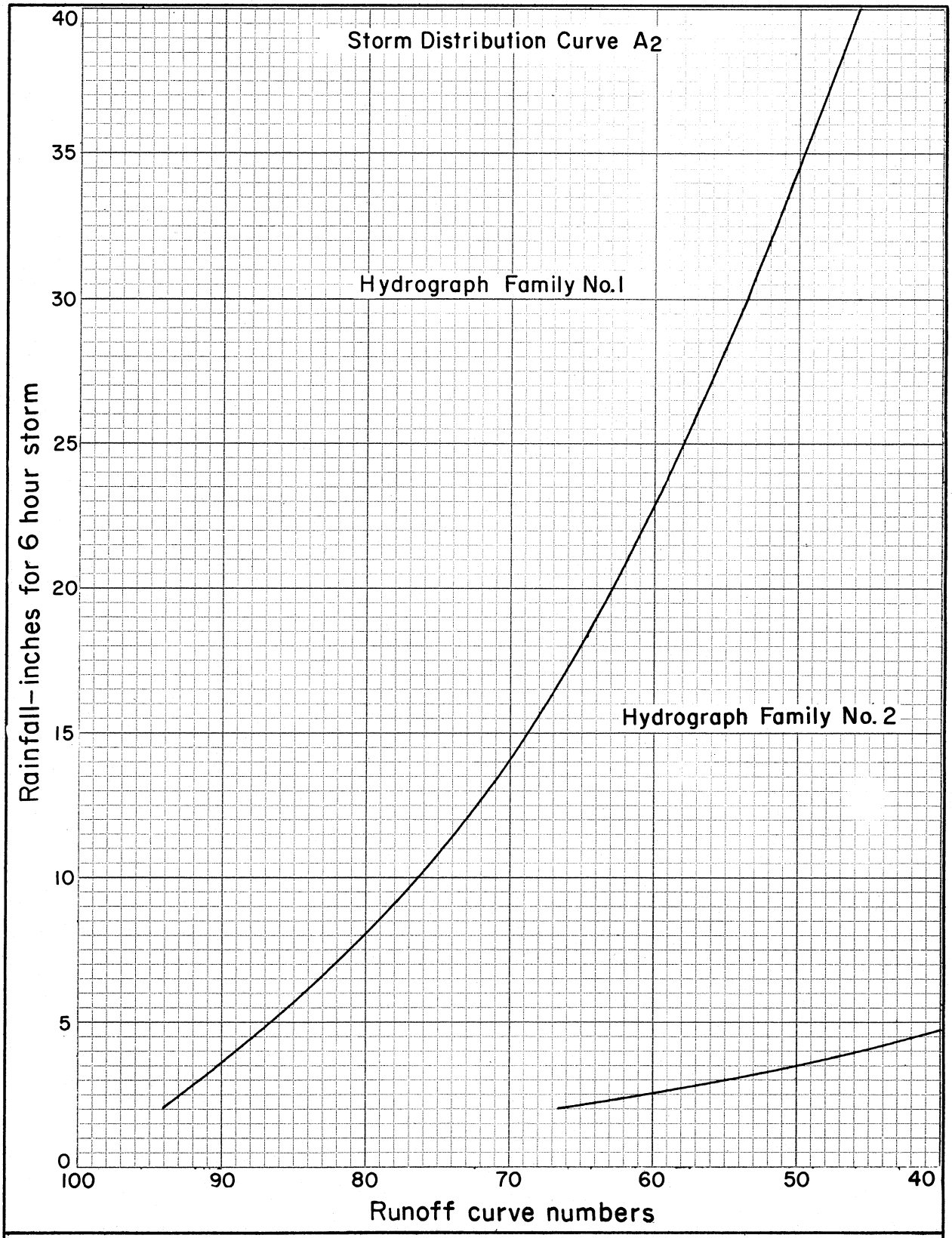


Chart for selecting a hydrograph family for a given 6 hour rainfall and runoff curve number.

Figure 3.21-7
(2 of 4)

COMPUTED M.H.K.	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
CHECKED	SOIL CONSERVATION SERVICE	ES-1011
DRAFTED	CENTRAL TECHNICAL UNIT	SHEET 2 OF 4
		DATE 2-14-58

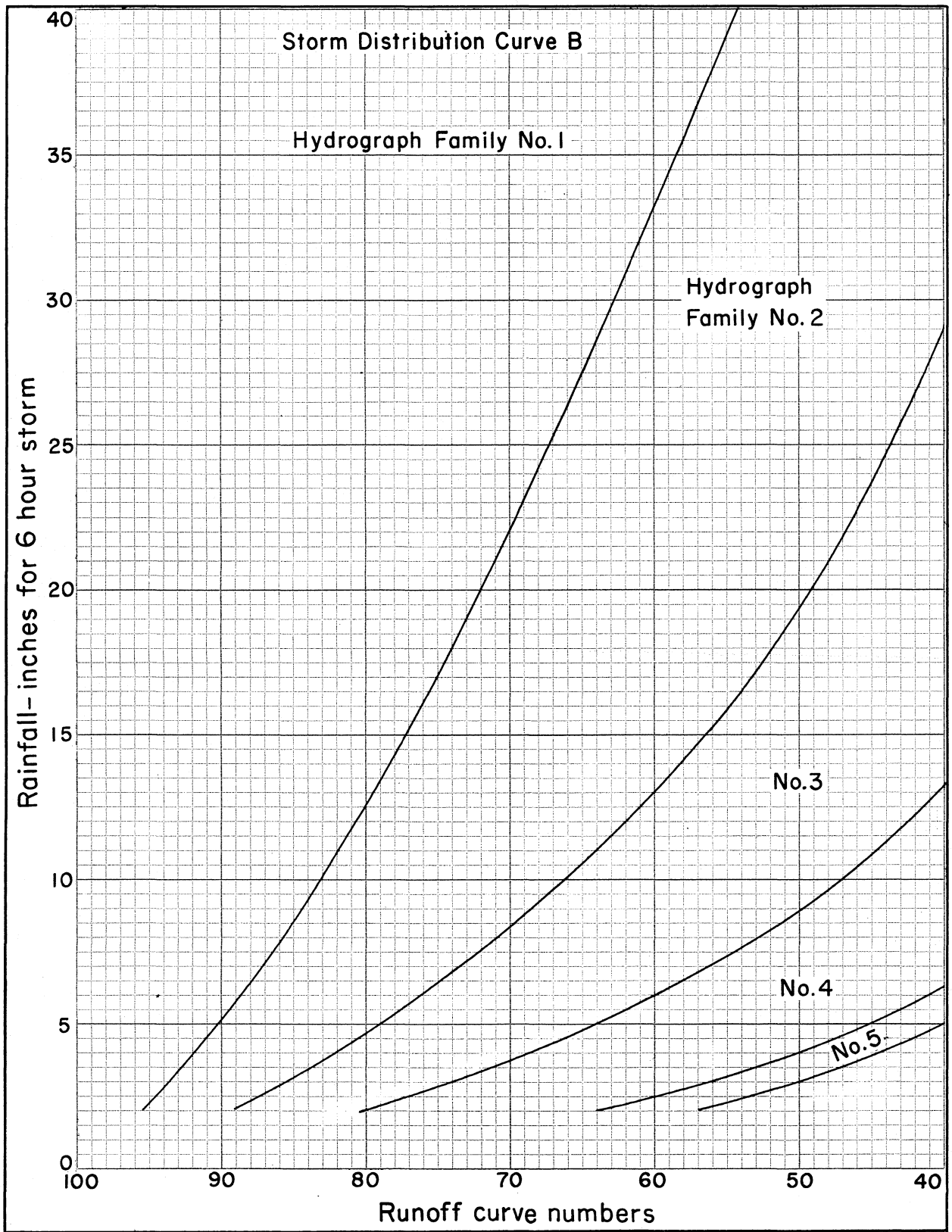


Chart for selecting a hydrograph family for a given 6 hour rainfall and runoff curve number.

Figure 3.21-7 (3 of 4)

COMPUTED M.H.K.	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
CHECKED	SOIL CONSERVATION SERVICE	ES-1011
DRAFTED	CENTRAL TECHNICAL UNIT	SHEET 3 OF 4
		DATE 2-14-58

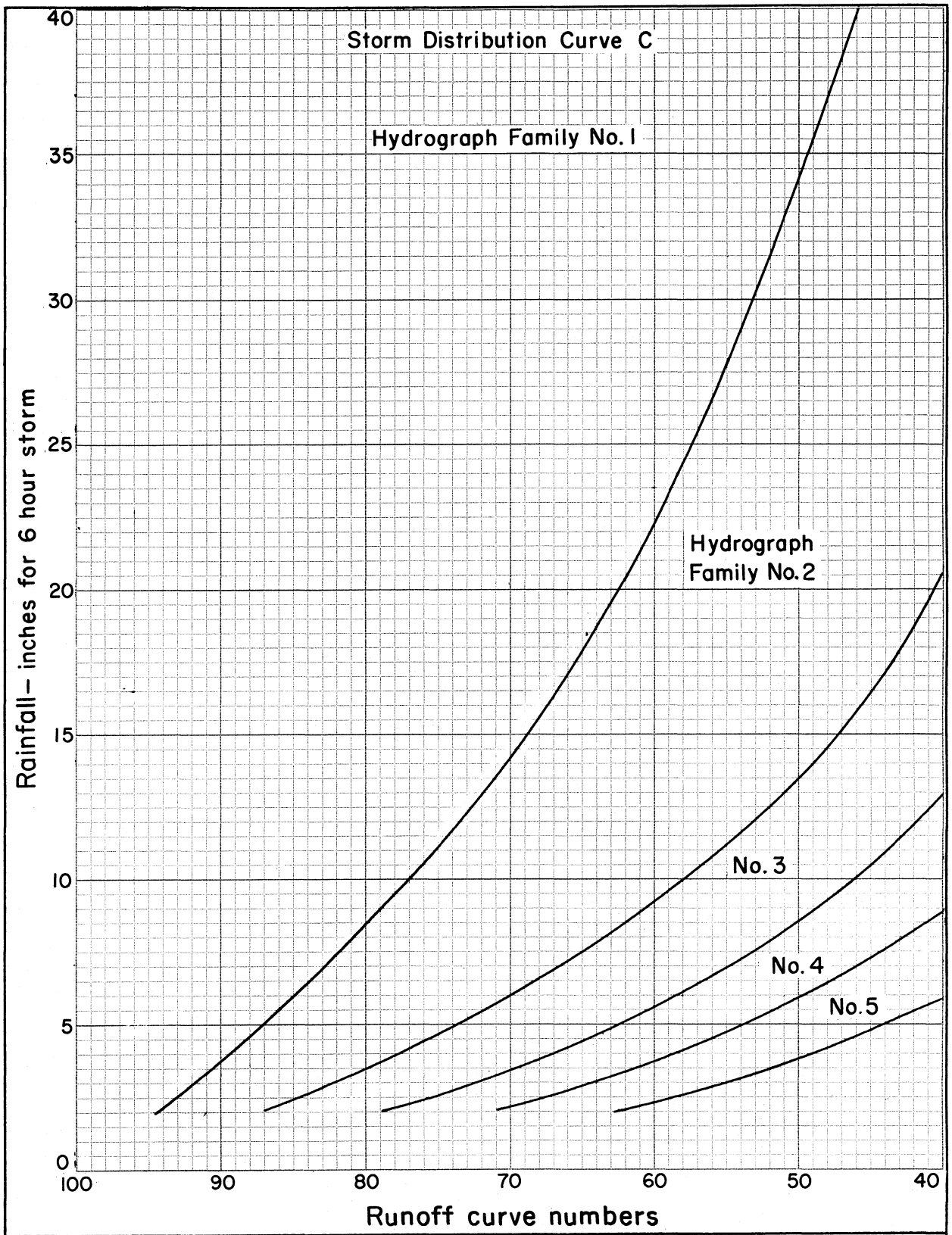
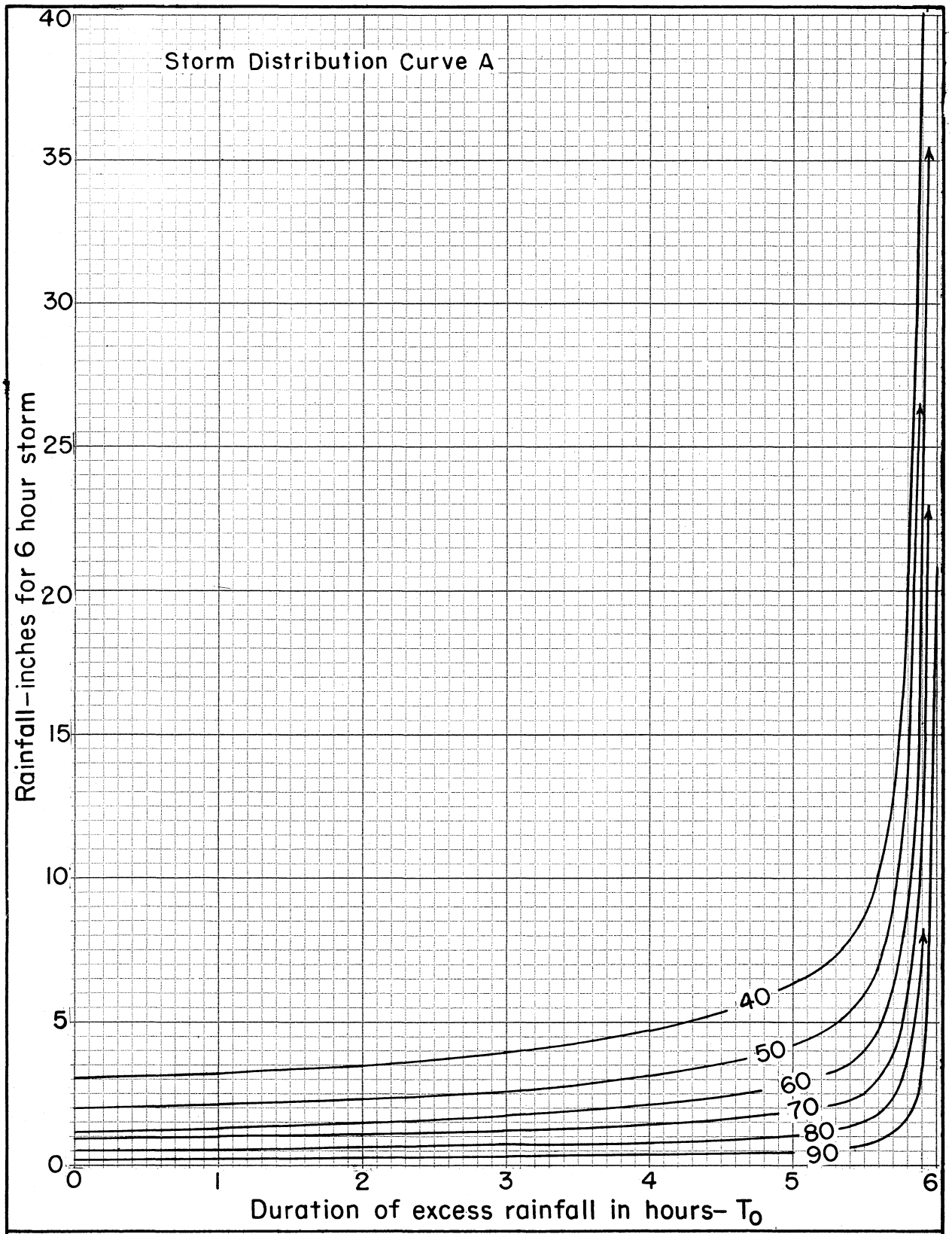


Chart for selecting a hydrograph family for a given 6 hour rainfall and runoff curve number.

Figure 3.21-7 (4 of 4)

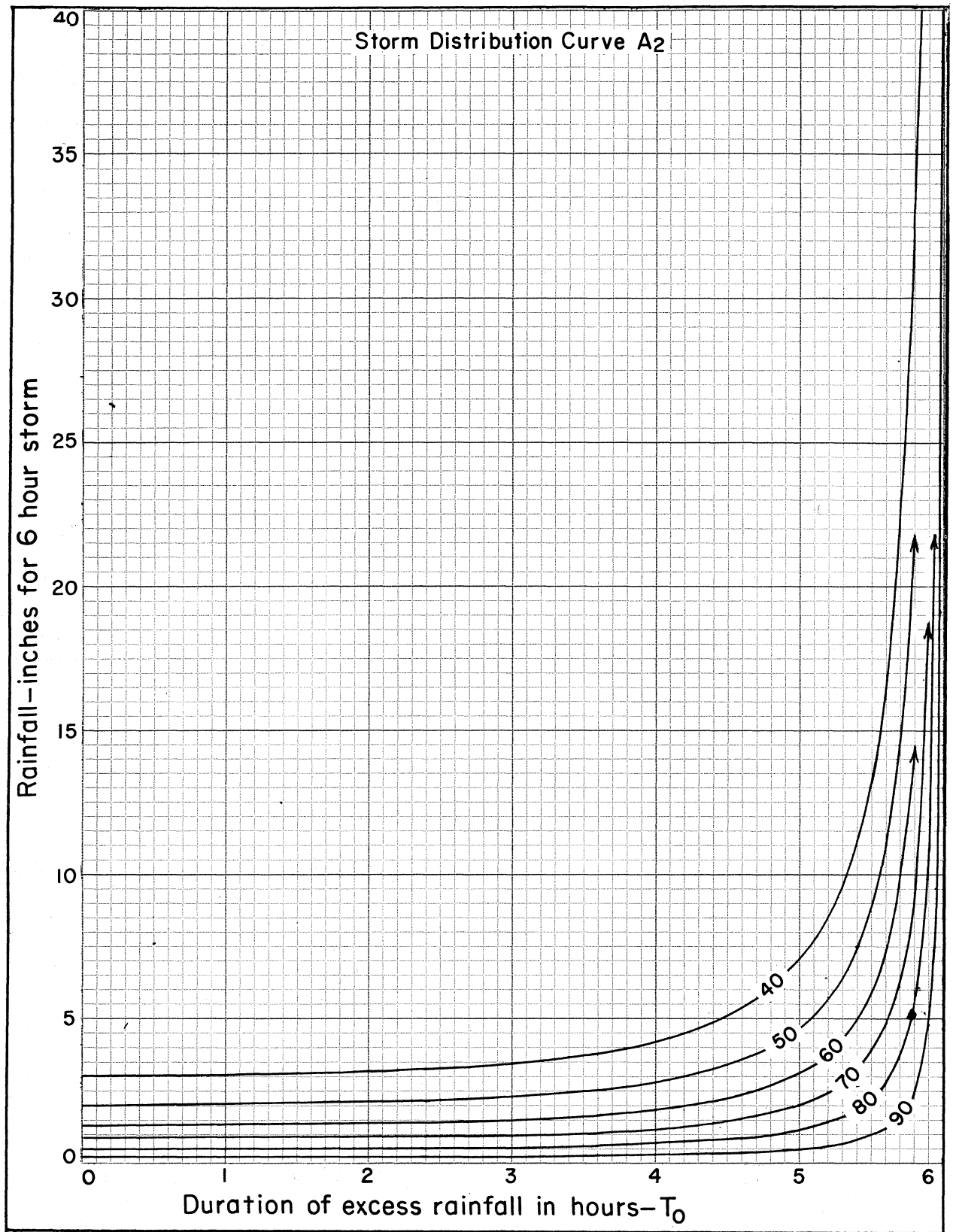
COMPUTED M.H.K.	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO
CHECKED	SOIL CONSERVATION SERVICE	ES-1011
DRAFTED	CENTRAL TECHNICAL UNIT	SHEET 4 OF 4
		DATE 2-14-58



Duration of excess rainfall for runoff curve numbers 40 to 100

Figure 3.21-8
(1 of 4)

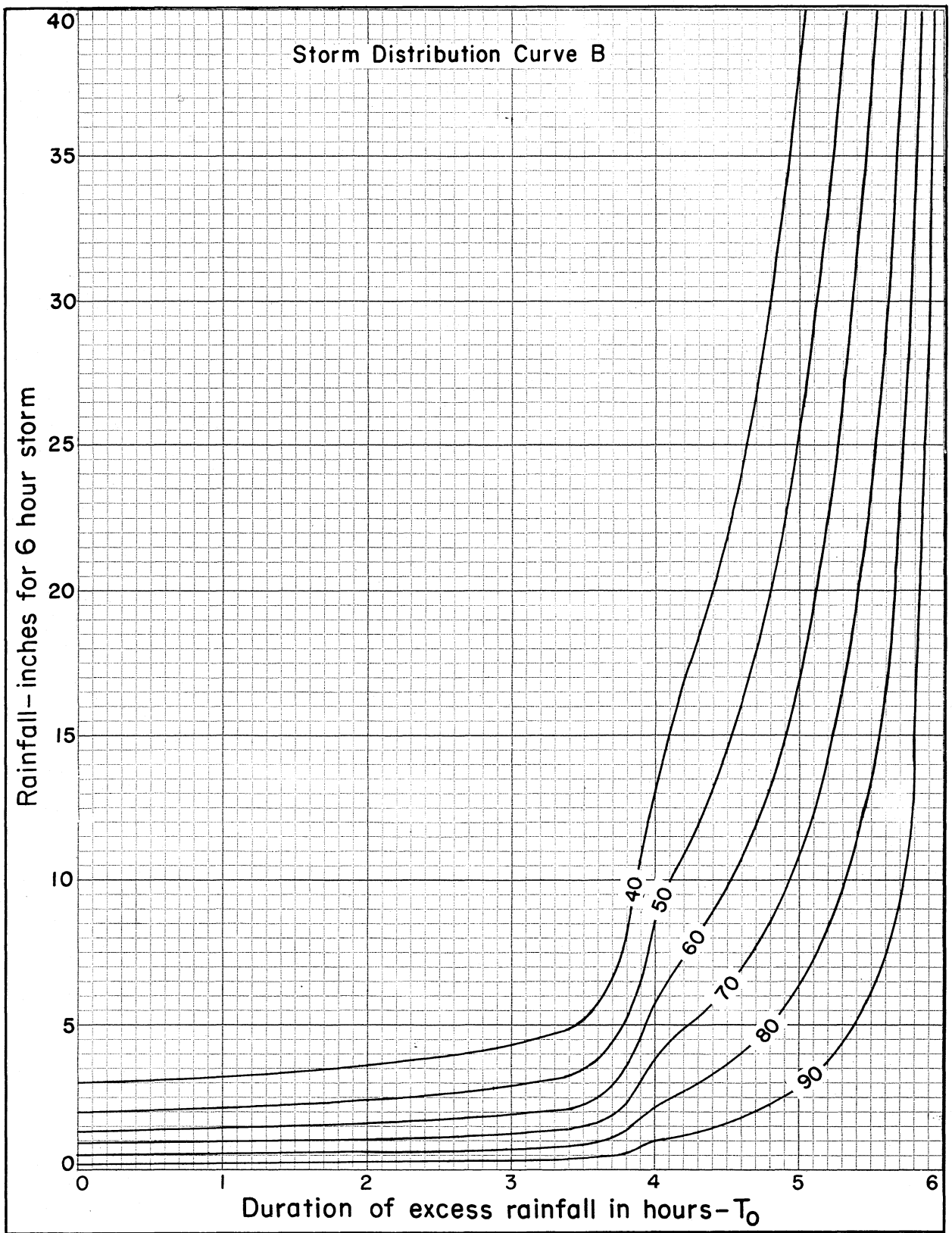
COMPUTED M.H.K.	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
CHECKED	SOIL CONSERVATION SERVICE	ES-1012
DRAFTED	CENTRAL TECHNICAL UNIT	SHEET 1 OF 4
		DATE 2-14-58



Duration of excess rainfall for runoff curve numbers 40 to 100

Figure 3.21-8
(2 of 4)

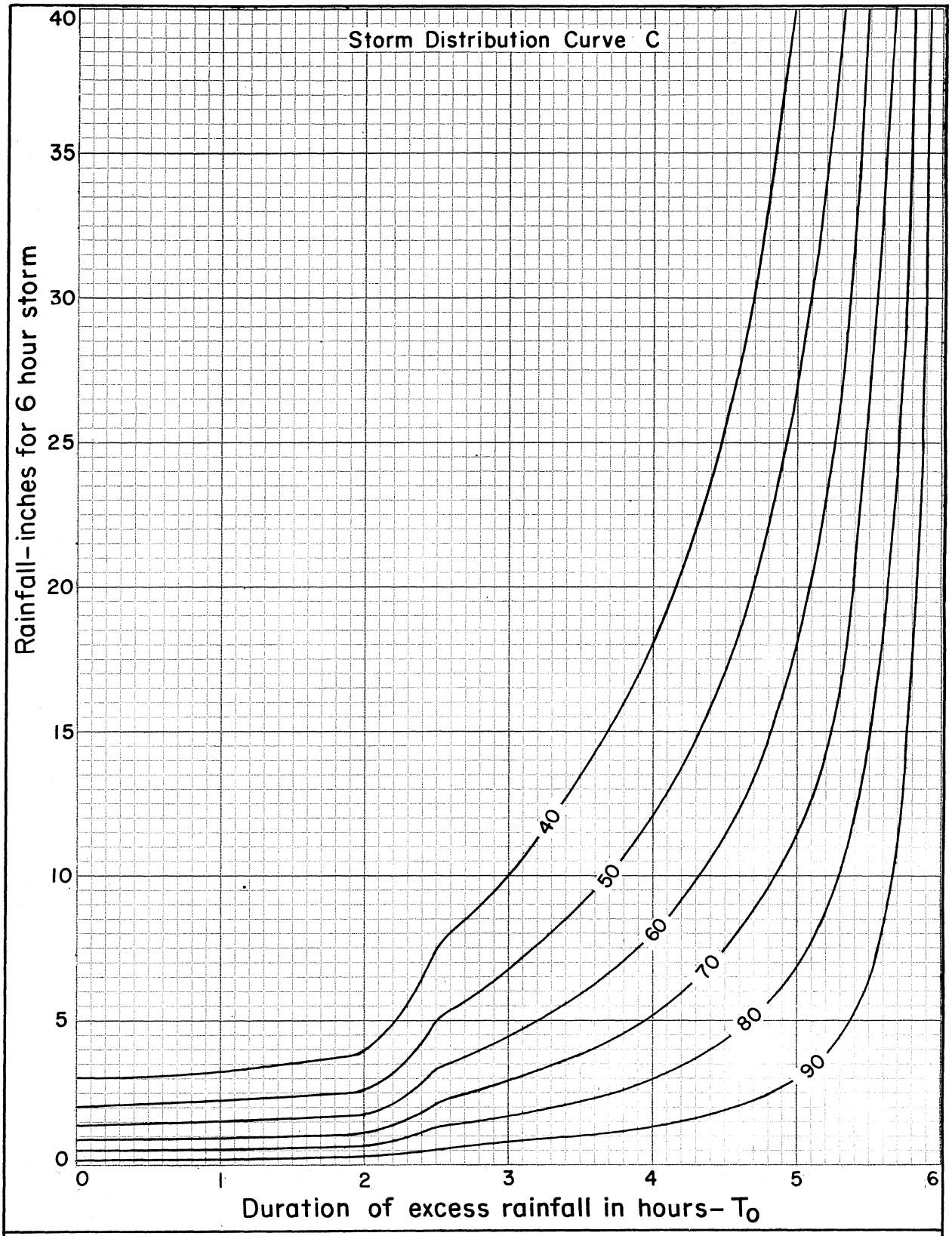
COMPUTED M.H.K. CHECKED DRAFTED	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE CENTRAL TECHNICAL UNIT	STANDARD DWG. NO. ES-1012 SHEET 2 OF 4 DATE 2-14-58



Duration of excess rainfall for runoff curve numbers 40 to 100

Figure 3.21-8
(3 of 4)

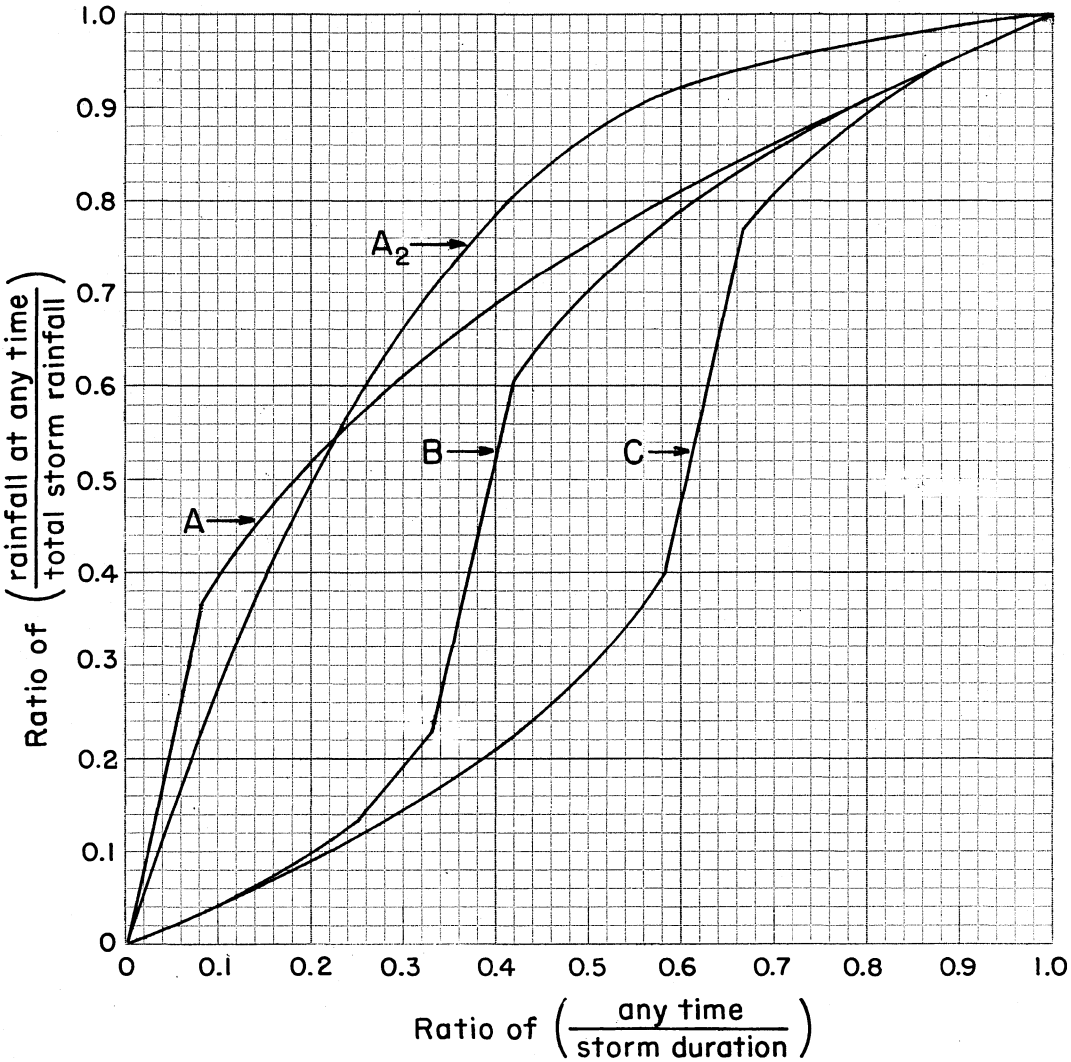
COMPUTED M.H.K. CHECKED DRAFTED	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE CENTRAL TECHNICAL UNIT	STANDARD DWG. NO. ES-1012 SHEET 3 OF 4 DATE 2-14-58



Duration of excess rainfall for runoff curve numbers 40 to 400

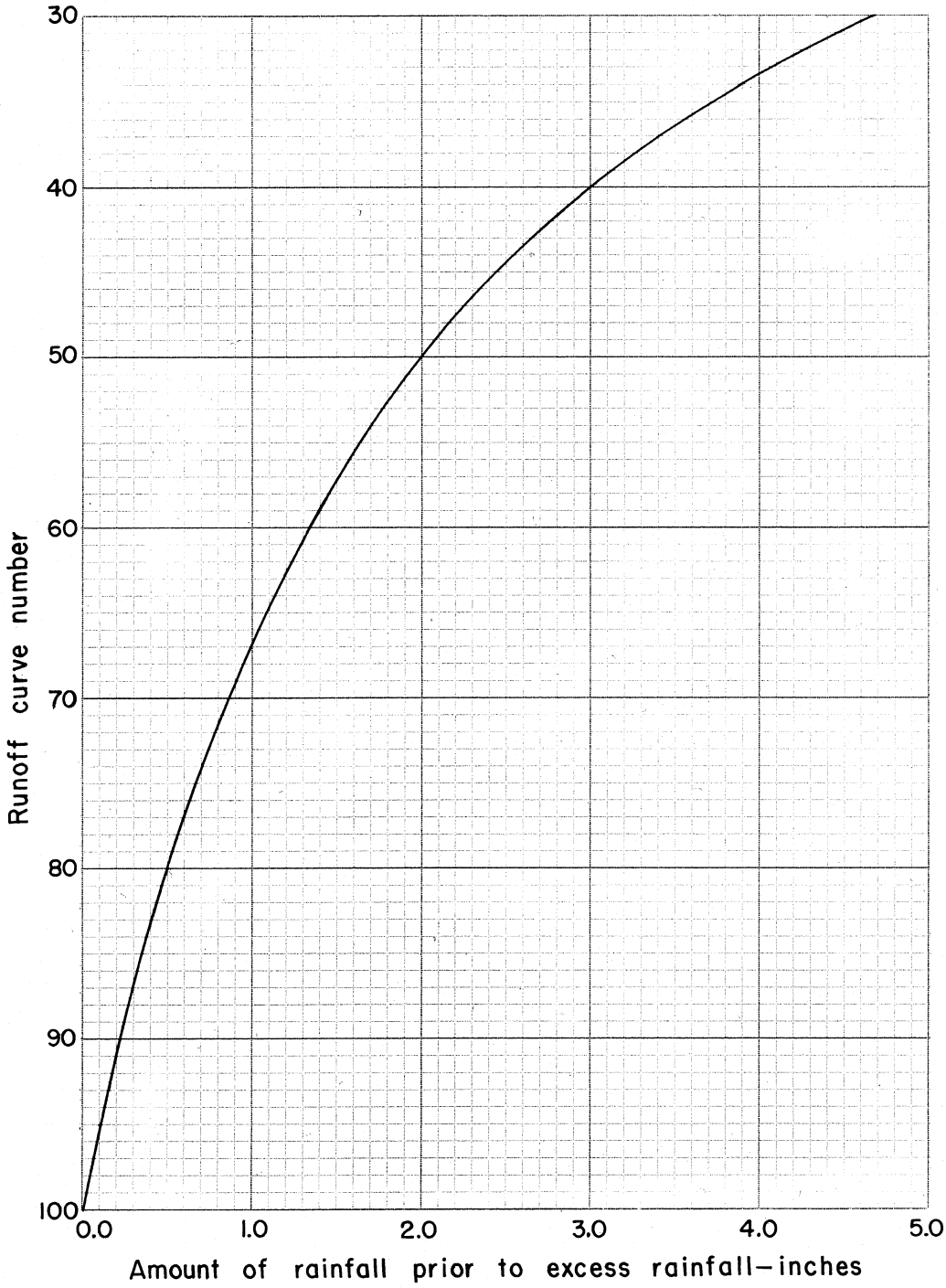
Figure 3 21-8
(4 of 4)

COMPUTED M.H.K. CHECKED DRAFTED	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE CENTRAL TECHNICAL UNIT	STANDARD DWG. NO. ES-1012 SHEET 4 OF 4 DATE 2-14-58



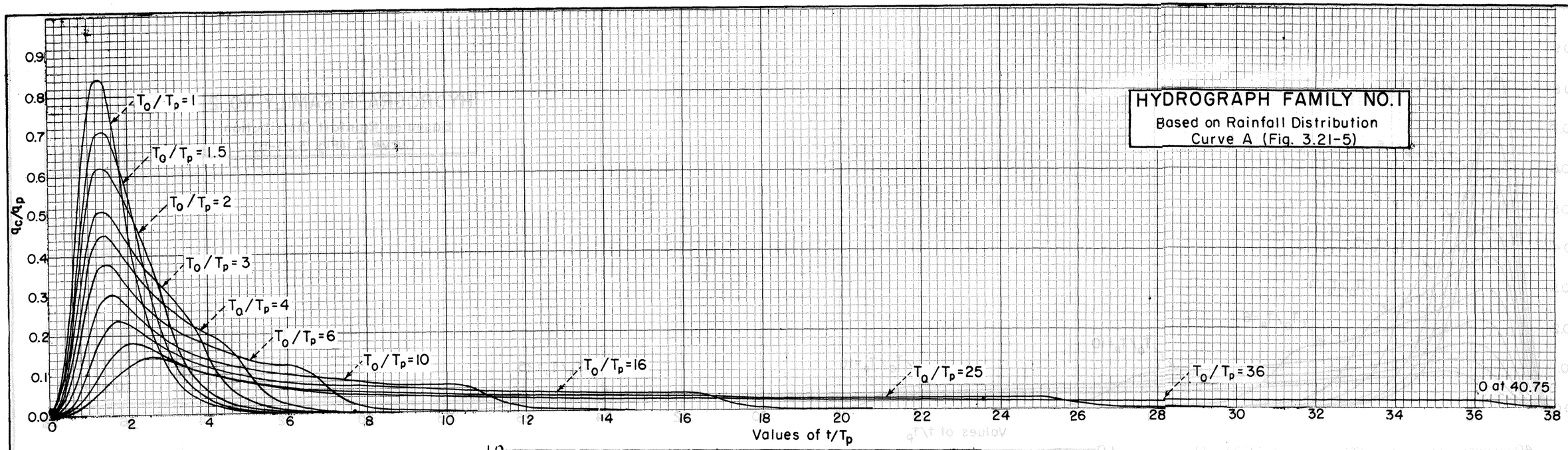
DIMENSIONLESS STORM DISTRIBUTION CURVES

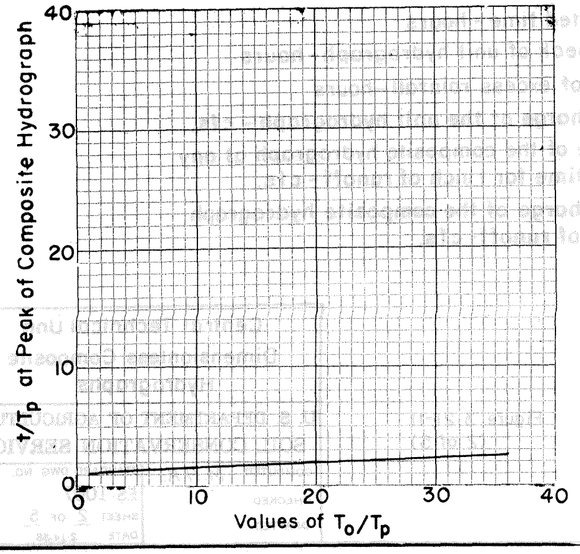
REFERENCE	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - CENTRAL TECHNICAL UNIT	Figure 3.21-9 DATE 5-12-58

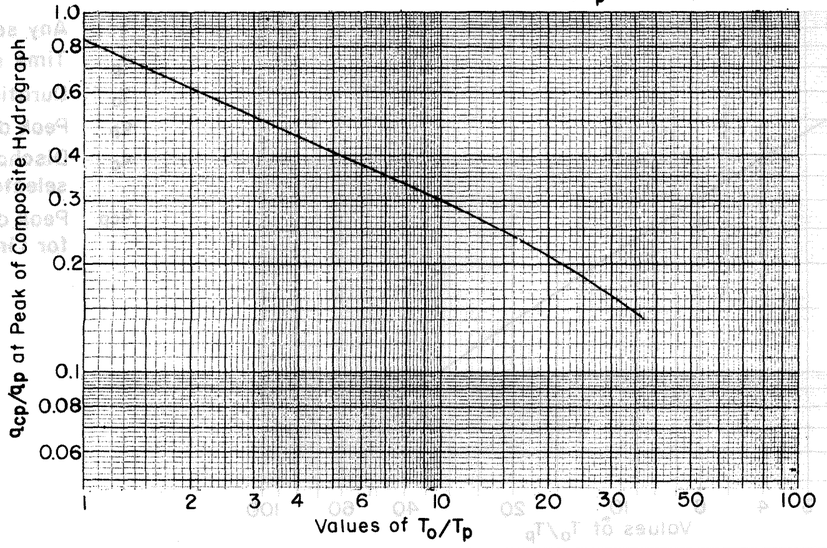


REFERENCE	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - CENTRAL TECHNICAL UNIT	Figure 3.21-10 DATE 5-12-58

☆U. S. GOVERNMENT PRINTING OFFICE : 1958 O - 441940 (SCS-254)







t	Any selected time - hours
T_p	Time of peak of unit hydrograph - hours
T_0	Duration of excess rainfall - hours
q_p	Peak discharge of the unit hydrograph - cfs
q_c	Discharge of the composite hydrograph at any selected time for 1 inch of runoff - cfs.
q_{cp}	Peak discharge of the composite hydrograph for 1 inch of runoff - cfs.

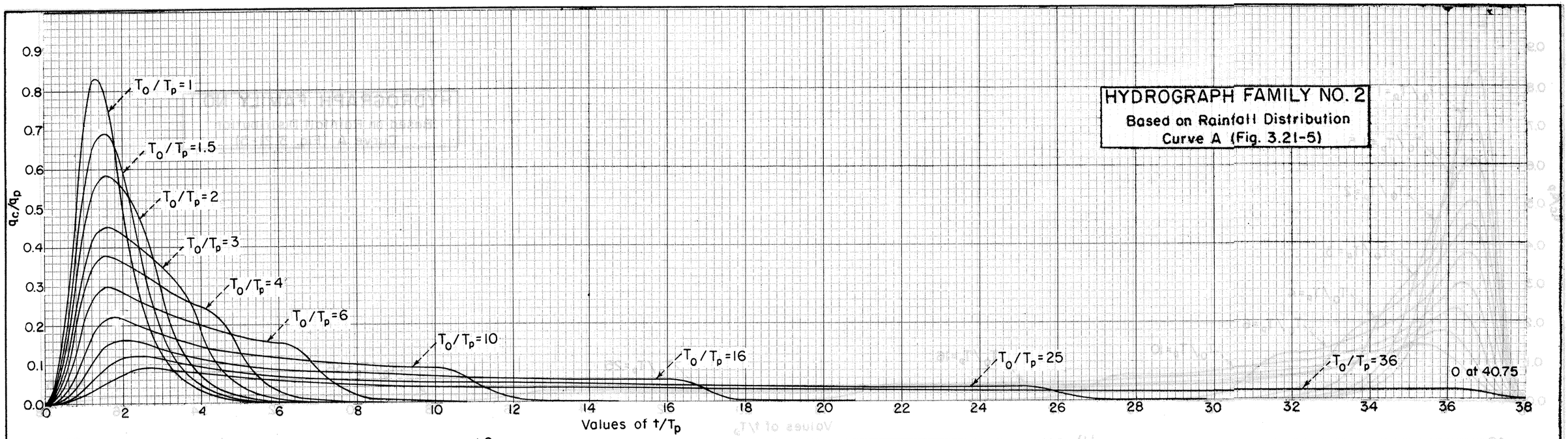
Figure 3.21-11
(1 of 5)

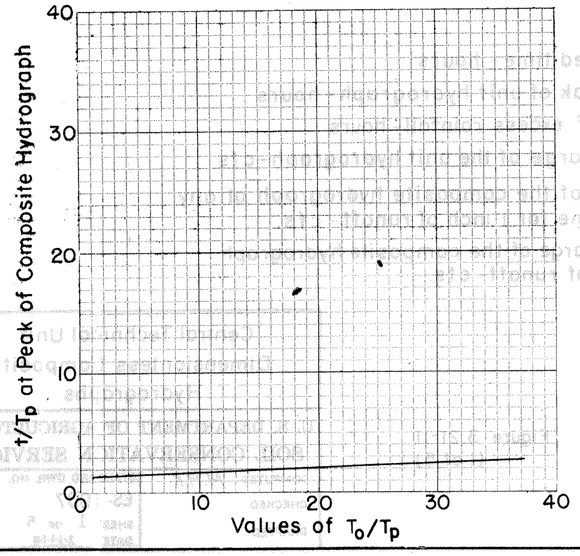
Central Technical Unit
Dimensionless Composite Hydrographs

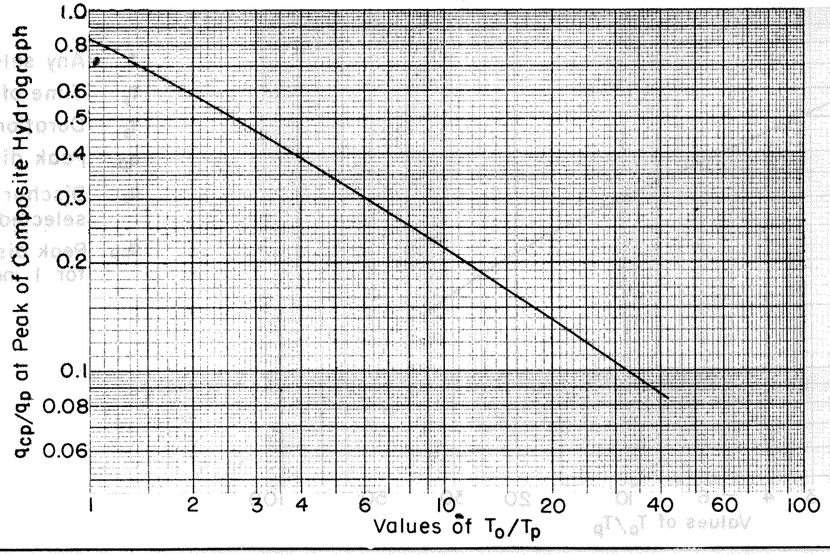
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

COMPUTED M.H.K.
CHECKED
DRAFTED

STANDARD DWG. NO. ES-1007
SHEET 1 OF 5
DATE 3-14-58







t	Any selected time - hours
T_p	Time of peak of unit hydrograph - hours
T_0	Duration of excess rainfall—hours
q_p	Peak discharge of the unit hydrograph - cfs.
q_c	Discharge of the composite hydrograph at any selected time for 1 inch of runoff - cfs.
q_{cp}	Peak discharge of the composite hydrograph for 1 inch of runoff - cfs.

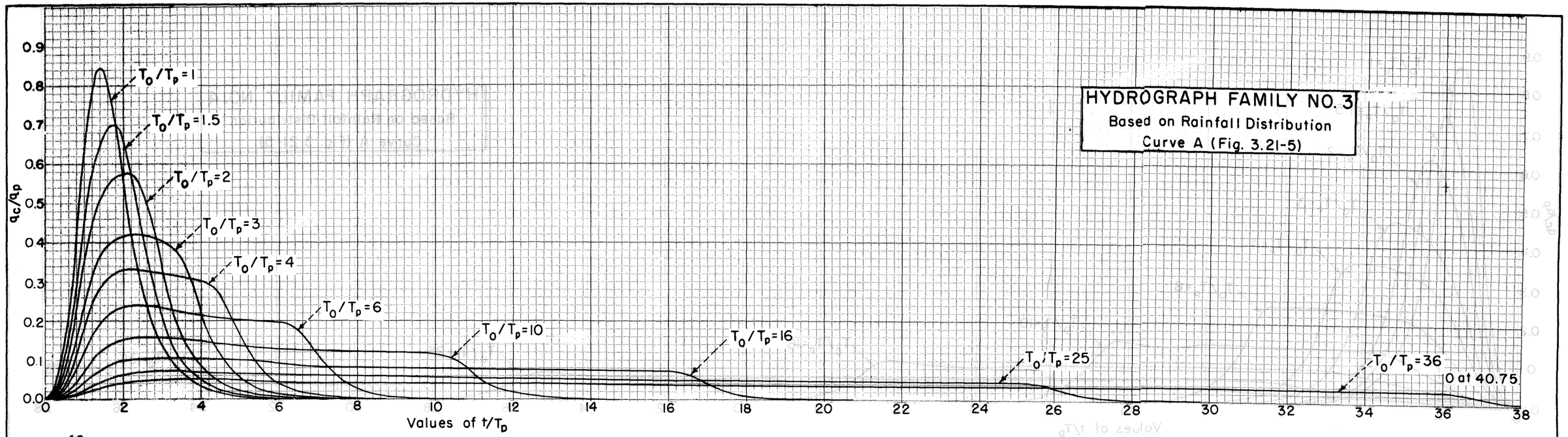
Figure 3.21-11
(2 of 5)

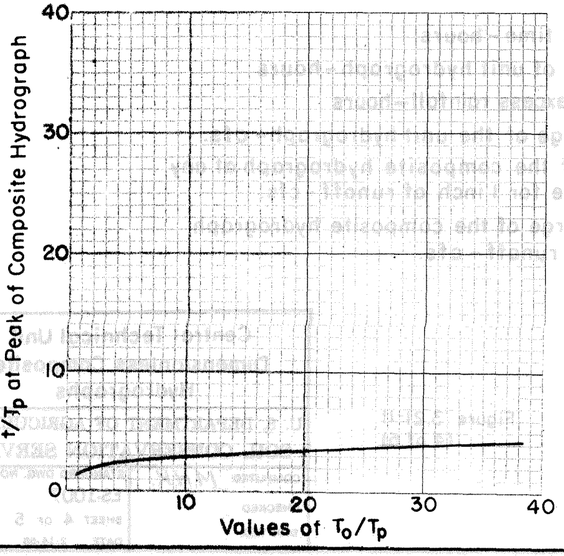
Central Technical Unit
Dimensionless Composite Hydrographs

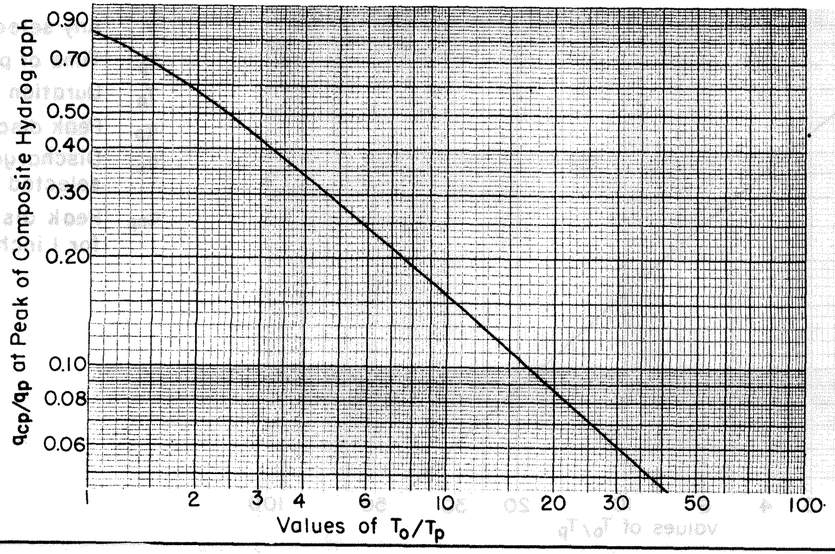
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

COMPUTED M.H.K.
CHECKED
DRAFTED

STANDARD DWG. NO. ES-1007
SHEET 2 OF 5
DATE 2-14-58







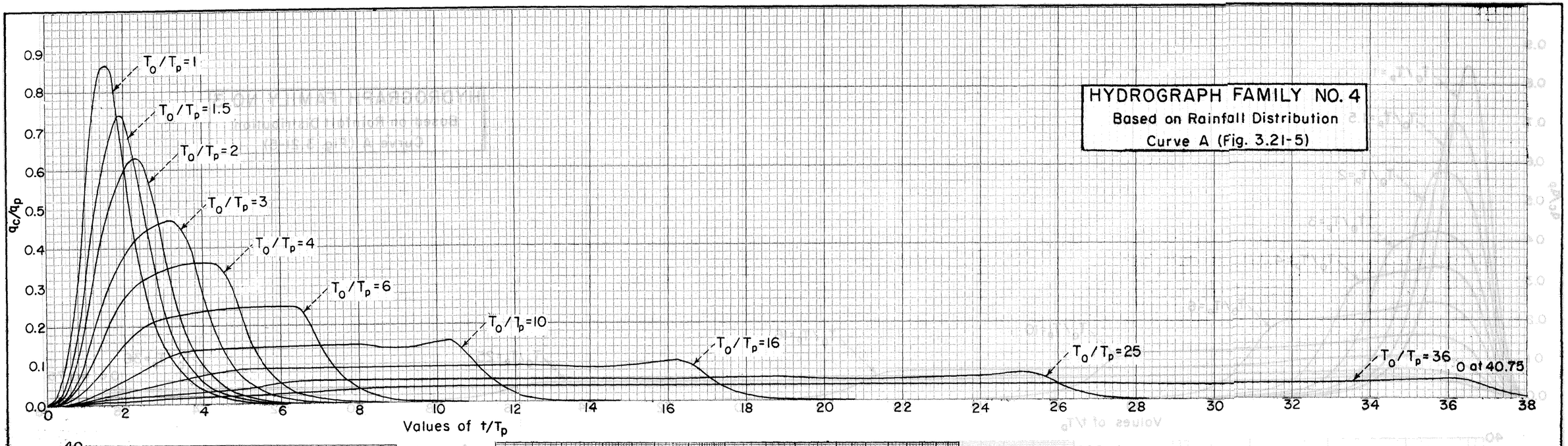
t	Any selected time – hours
T_p	Time of peak of unit hydrograph – hours
T_0	Duration of excess rainfall – hours
q_p	Peak discharge of unit hydrograph – cfs.
q_c	Discharge of the composite hydrograph at any selected time for 1 inch of runoff – cfs.
q_{cp}	Peak discharge of the composite hydrograph for 1 inch of runoff – cfs.

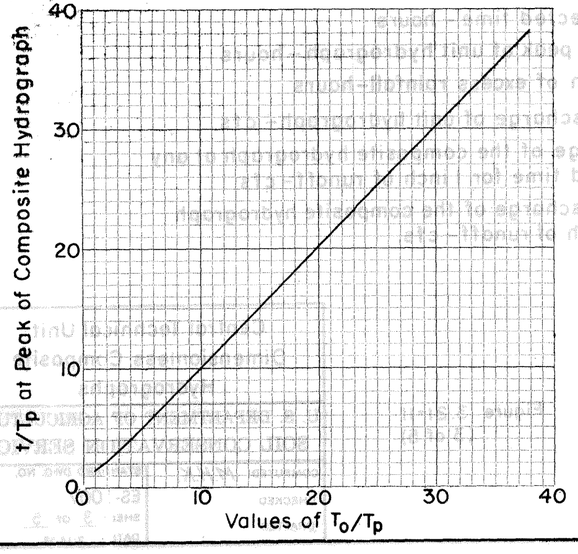
Figure 3.21-11
(3 of 5)

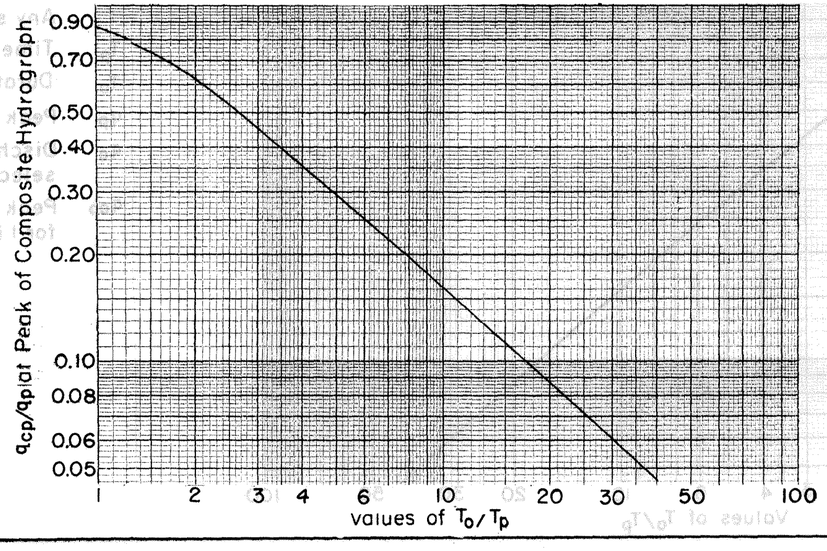
Central Technical Unit
Dimensionless Composite Hydrographs

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

COMPUTED M.H.K.	STANDARD DWG. NO. ES-1007
CHECKED	SHEET 3 OF 5
DRAFTED	DATE 2-14-58







t	Any selected time – hours
T_p	Time of peak of unit hydrograph – hours
T_0	Duration of excess rainfall – hours
q_p	Peak discharge of the unit hydrograph – cfs.
q_c	Discharge of the composite hydrograph at any selected time for 1 inch of runoff – cfs.
q_{cp}	Peak discharge of the composite hydrograph for 1 inch of runoff – cfs.

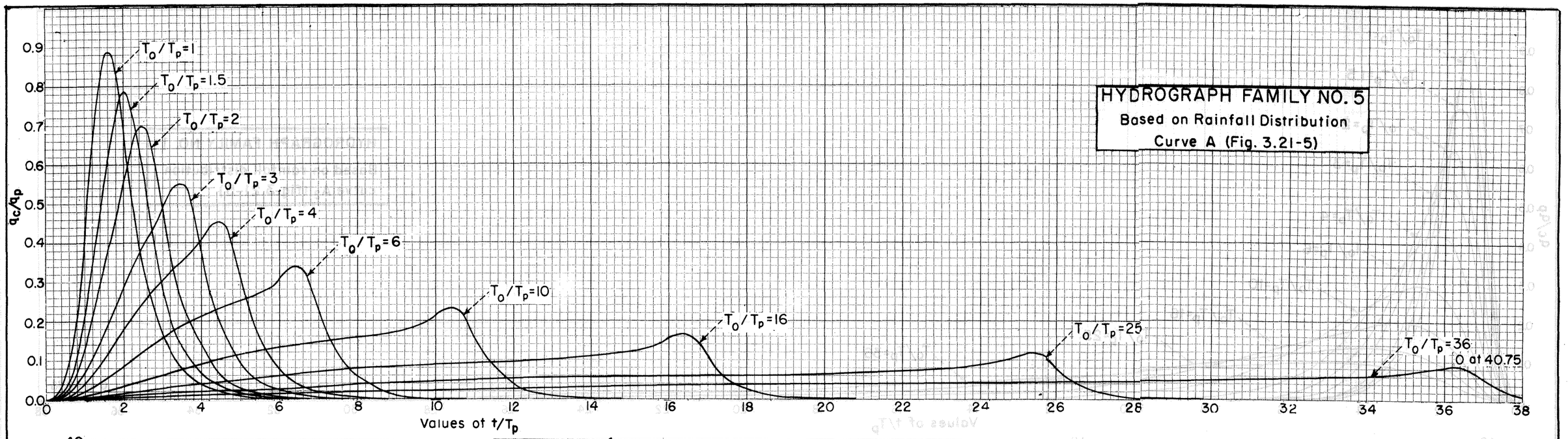
Figure 3.21-11
(4 of 5)

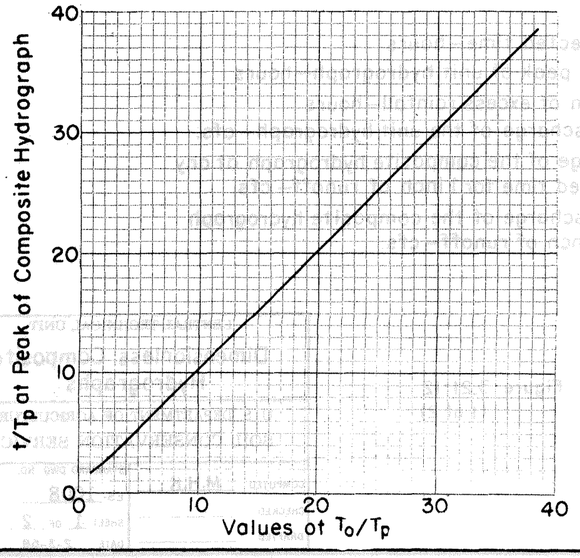
Central Technical Unit
Dimensionless Composite Hydrographs

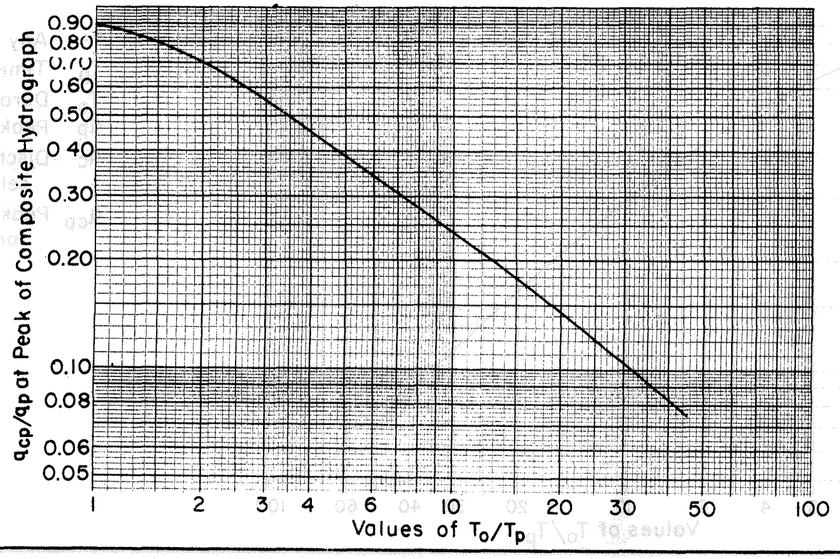
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

COMPUTED M.H.K.
CHECKED
DRAFTED

STANDARD DWG. NO. ES-1007
SHEET 4 OF 5
DATE 2-14-58







t Any selected time - hours

T_p Time of peak of unit hydrograph - hours

T_0 Duration of excess rainfall - hours

q_p Peak discharge of unit hydrograph - cfs.

q_c Discharge of the composite hydrograph at any selected time for 1 inch of runoff - cfs.

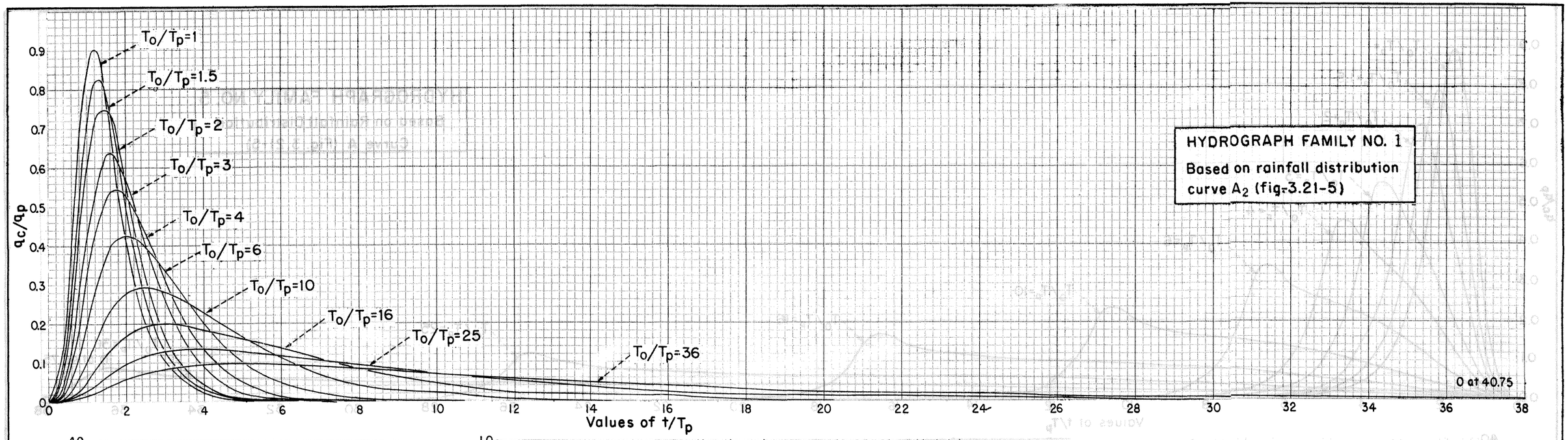
q_{cp} Peak discharge of the composite hydrograph for 1 inch of runoff

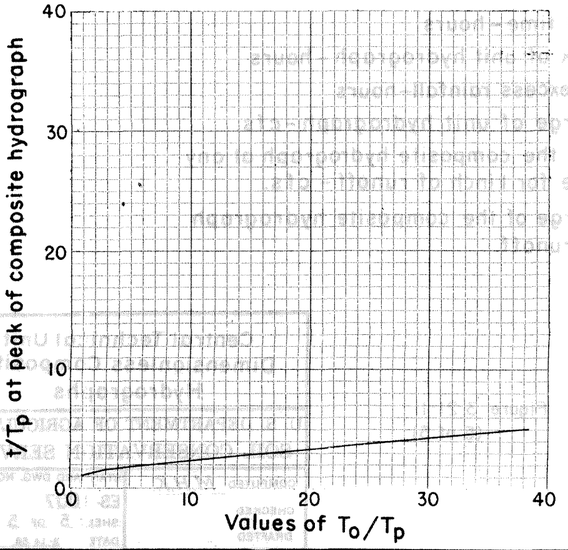
Figure 3.21-11
(5 of 5)

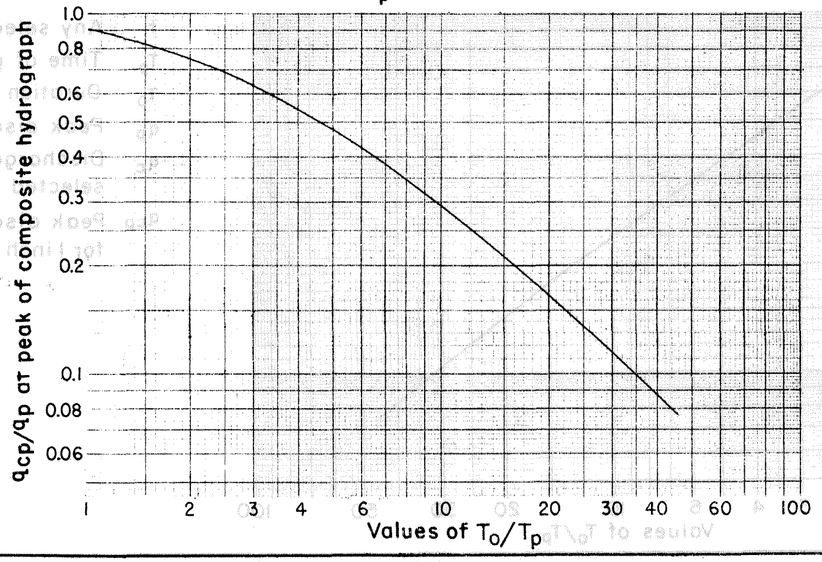
Central Technical Unit
Dimensionless Composite Hydrographs

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

COMPUTED M.H.K.	STANDARD DWG. NO. ES-1007
CHECKED	SHEET 5 OF 5
DRAFTED	DATE 2-14-58







t ..... Any selected time– hours

T_p ..... Time of peak of unit hydrograph– hours

T_o ..... Duration of excess rainfall– hours

q_p ..... Peak discharge of the unit hydrograph– cfs.

q_c ..... Discharge of the composite hydrograph at any selected time for 1 inch of runoff– cfs.

q_{cp} ..... Peak discharge of the composite hydrograph for 1 inch of runoff– cfs.

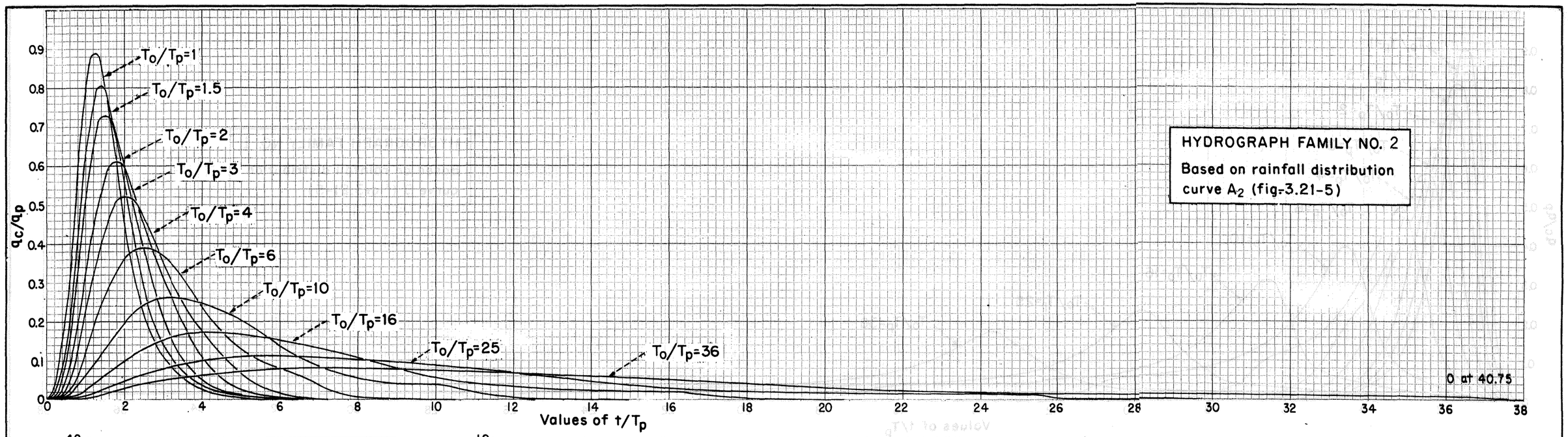
Figure 3.21-12
(1 of 2)

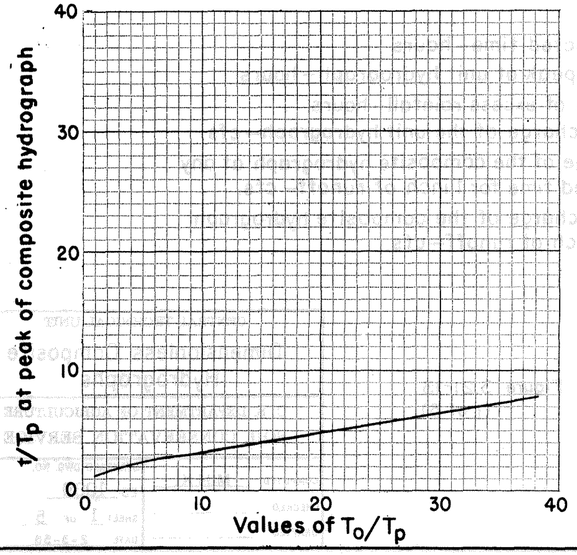
CENTRAL TECHNICAL UNIT

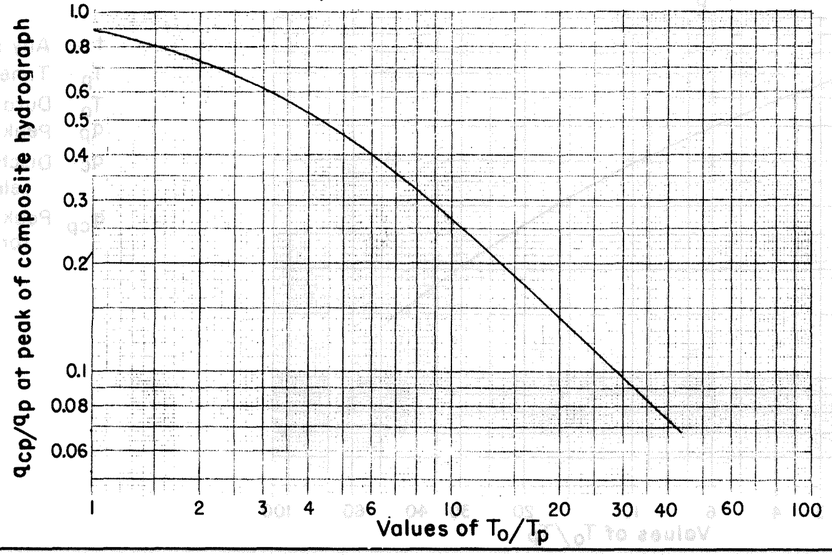
Dimensionless Composite Hydrographs

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

COMPUTED	M.H.K	STANDARD DWG. NO. ES-1008
CHECKED		SHEET 1 OF 2
DRAFTED		DATE 2-3-58







t ..... Any selected time—hours
T_p ..... Time of peak of unit hydrograph—hours
T_o ..... Duration of excess rainfall—hours
q_p ..... Peak discharge of the unit hydrograph—cfs.
q_c ..... Discharge of the composite hydrograph at any selected time for 1 inch of runoff—cfs.
q_{cp} ..... Peak discharge of the composite hydrograph for 1 inch of runoff—cfs.

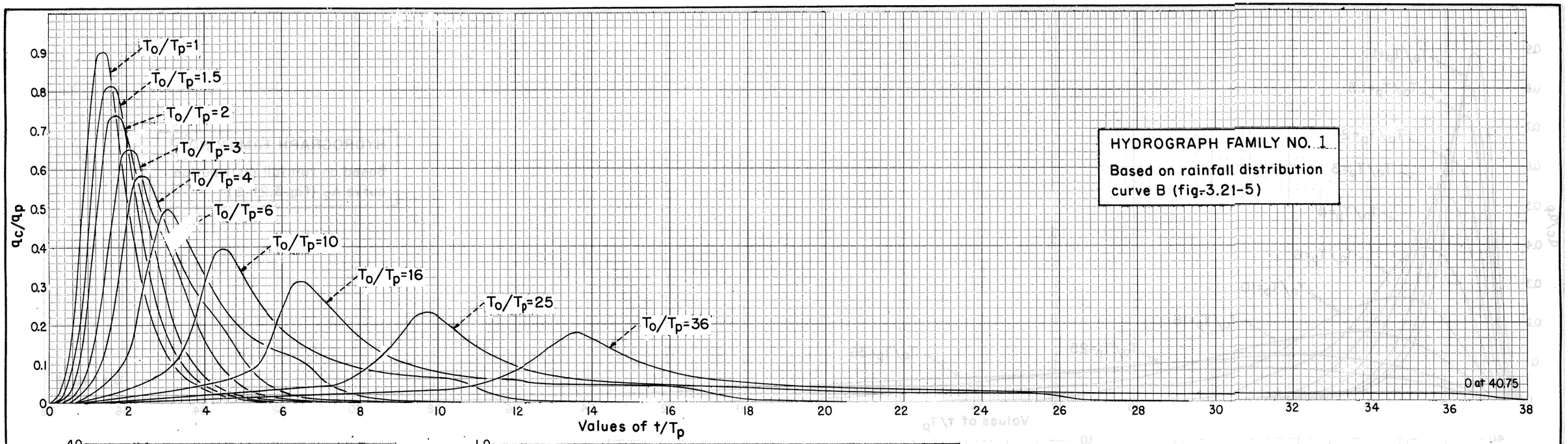
Figure 3.21-12
(2 of 2)

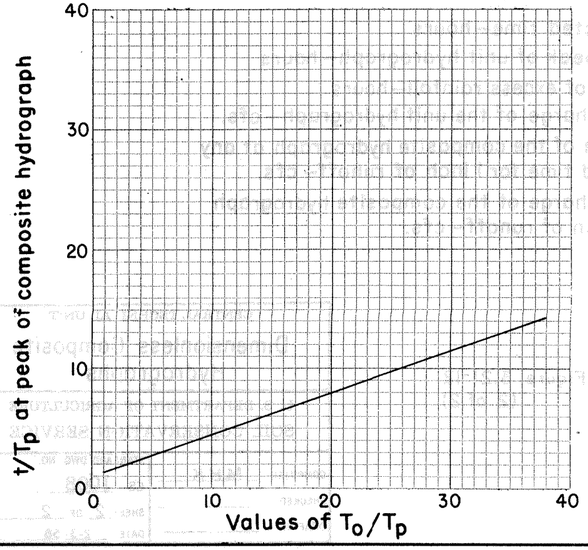
CENTRAL TECHNICAL UNIT

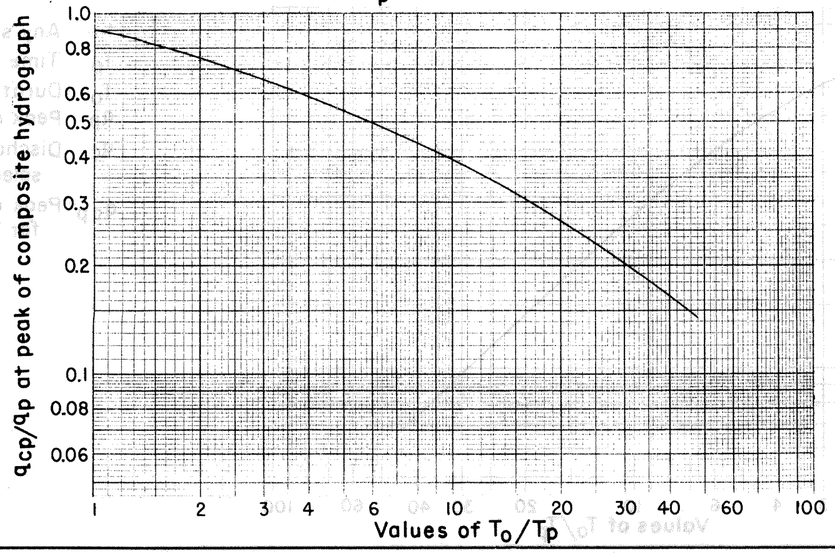
Dimensionless Composite Hydrographs

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

COMPUTED M.H.K.	STANDARD DWG. NO. ES-1008
CHECKED	SHEET 2 OF 2
DRAFTED	DATE 2-3-58







t ..... Any selected time– hours
T_p ..... Time of peak of unit hydrograph– hours
T_o ..... Duration of excess rainfall– hours
q_p ..... Peak discharge of the unit hydrograph– cfs.
q_c ..... Discharge of the composite hydrograph at any selected time for 1 inch of runoff– cfs.
q_{cp} ..... Peak discharge of the composite hydrograph for 1 inch of runoff– cfs.

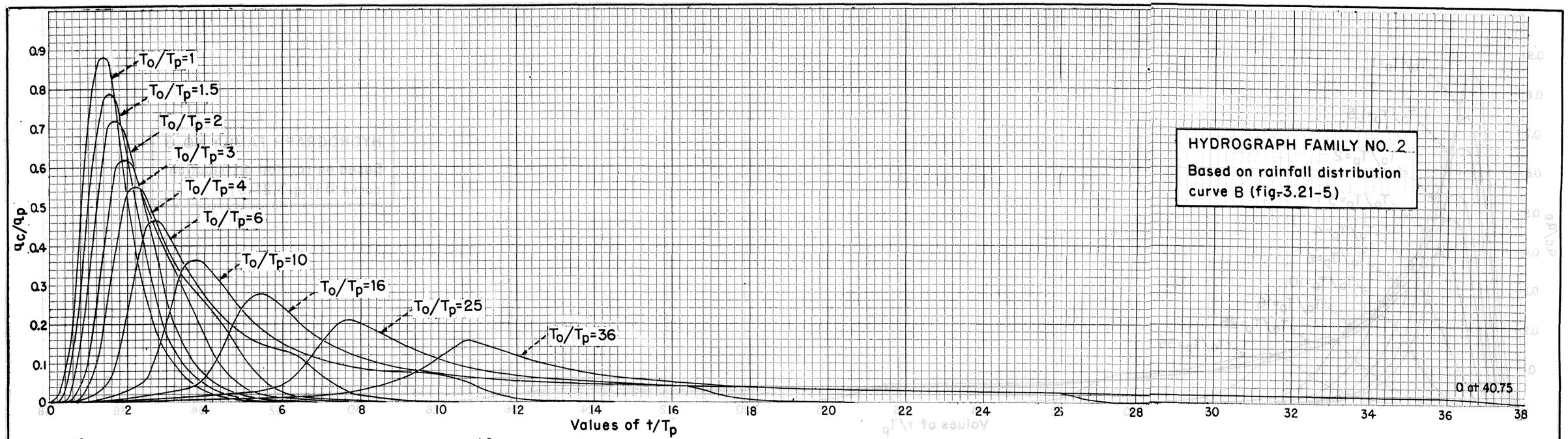
Figure 3.21-13
(1 of 5)

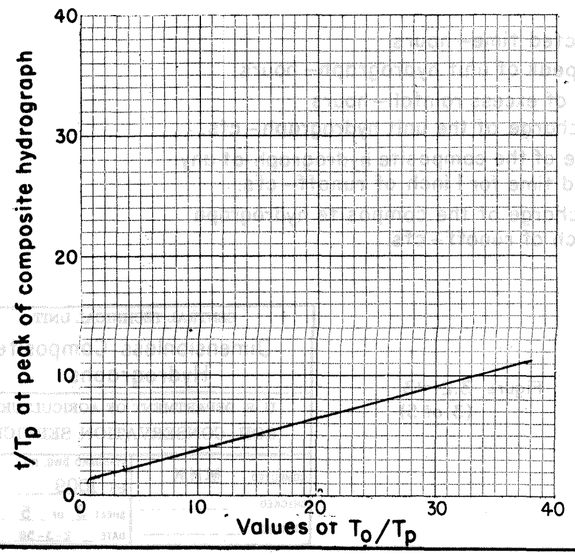
CENTRAL TECHNICAL UNIT

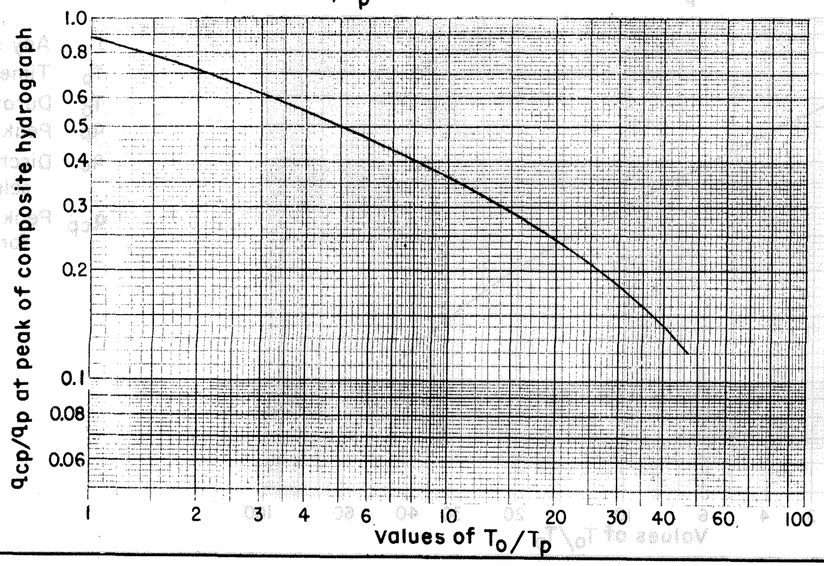
Dimensionless Composite Hydrographs

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

COMPUTED M.H.K.	STANDARD DWG. NO. ES-1009
CHECKED	SHEET 1 OF 5
DRAFTED	DATE 2-3-58







t ..... Any selected time—hours
T_p ..... Time of peak of unit hydrograph—hours
T_o ..... Duration of excess rainfall—hours
q_p ..... Peak discharge of the unit hydrograph—cfs.
q_c ..... Discharge of the composite hydrograph at any selected time for 1 inch of runoff—cfs.
q_{cp} ..... Peak discharge of the composite hydrograph for 1 inch of runoff—cfs.

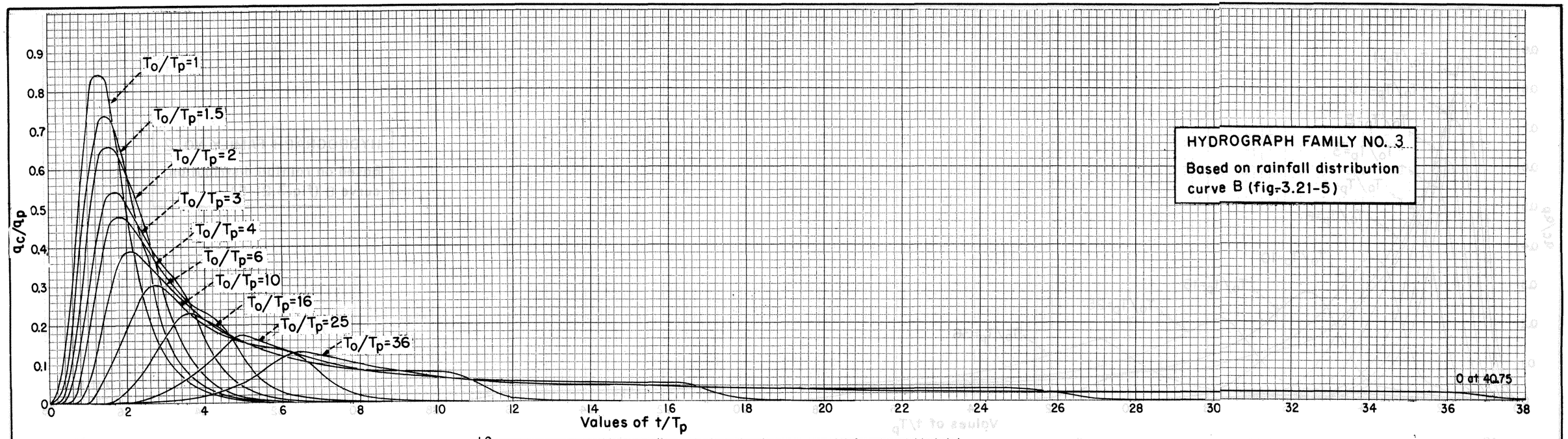
Figure 3.21-13
(2 of 5)

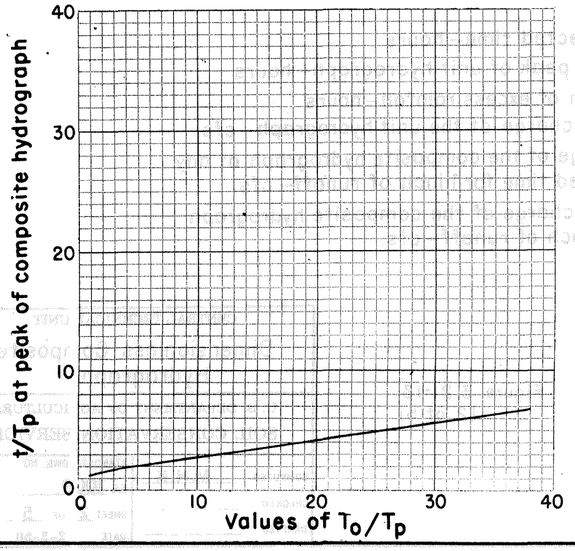
CENTRAL TECHNICAL UNIT

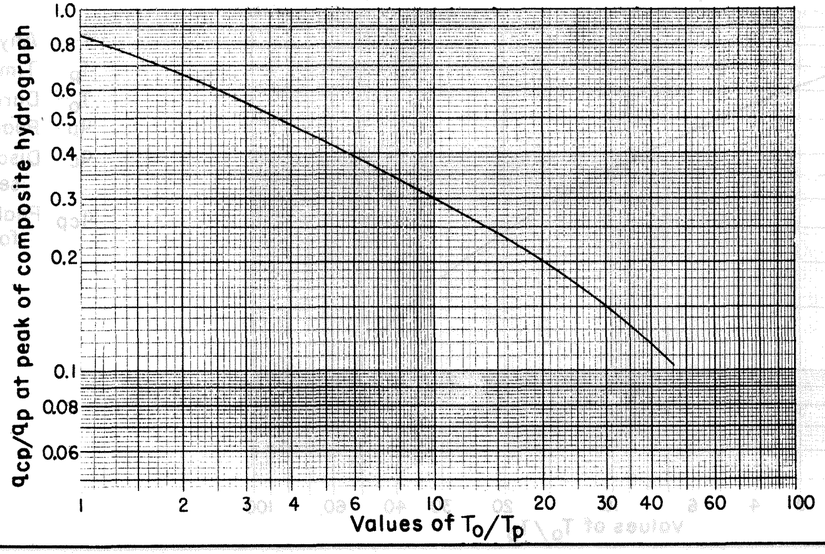
Dimensionless Composite Hydrographs

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

COMPUTED M.H.K.	STANDARD DWG. NO. ES-1009
CHECKED	SHEET 2 OF 5
DRAFTED	DATE 2-3-58







t ..... Any selected time—hours

T_p ..... Time of peak of unit hydrograph—hours

T_o ..... Duration of excess rainfall—hours

q_p ..... Peak discharge of the unit hydrograph—cfs.

q_c ..... Discharge of the composite hydrograph at any selected time for 1 inch of runoff—cfs.

q_{cp} ..... Peak discharge of the composite hydrograph for 1 inch of runoff—cfs.

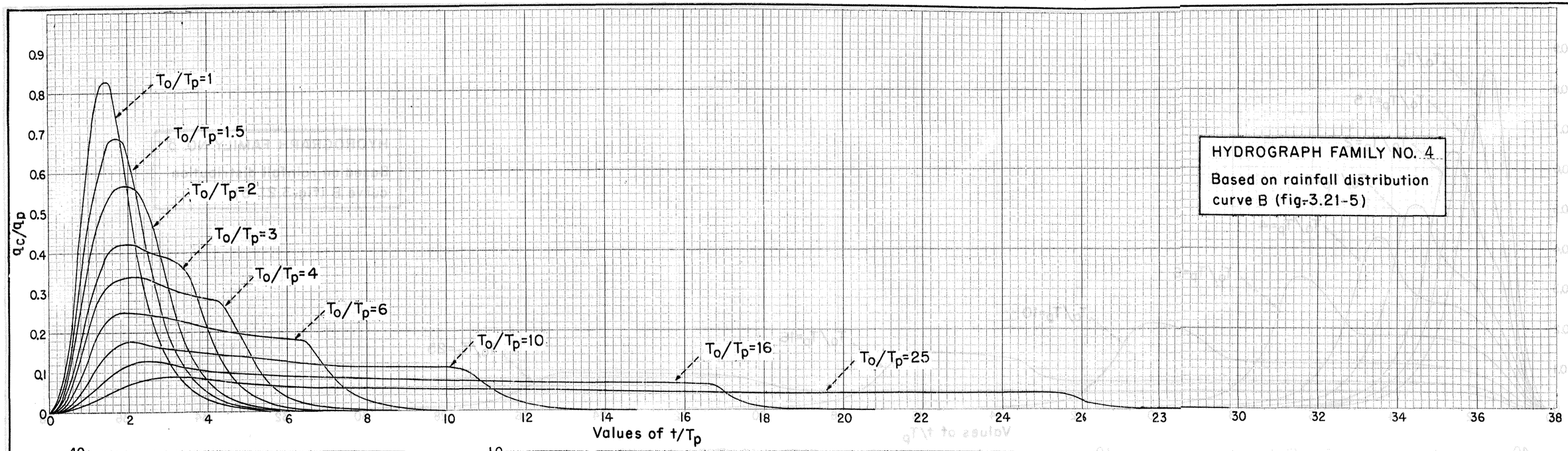
Figure 3.21-13
(3 of 5)

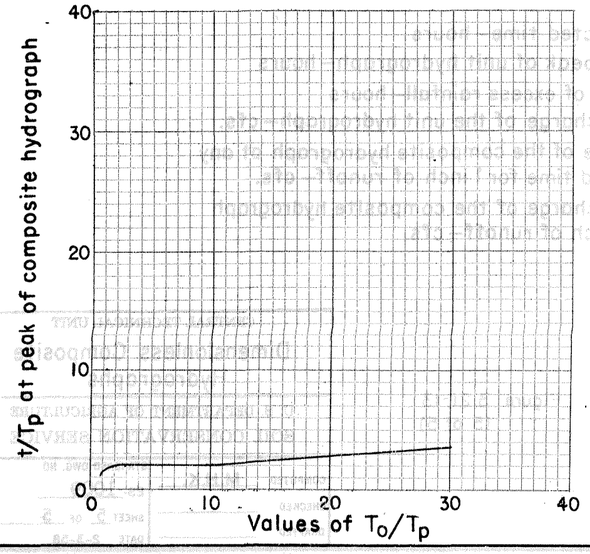
CENTRAL TECHNICAL UNIT

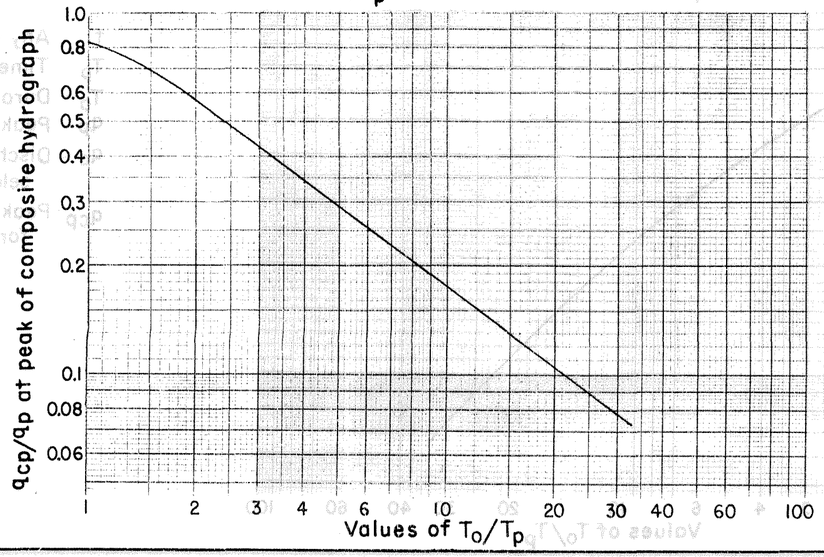
Dimensionless Composite Hydrographs

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

COMPUTED M.H.K.	STANDARD DWG. NO. ES-1009
CHECKED	SHEET 3 OF 5
DRAFTED	DATE 2-3-58







t — Any selected time—hours

T_p — Time of peak of unit hydrograph—hours

T_o — Duration of excess rainfall—hours

q_p — Peak discharge of the unit hydrograph—cfs.

q_c — Discharge of the composite hydrograph at any selected time for 1 inch of runoff—cfs.

q_{cp} — Peak discharge of the composite hydrograph for 1 inch of runoff—cfs.

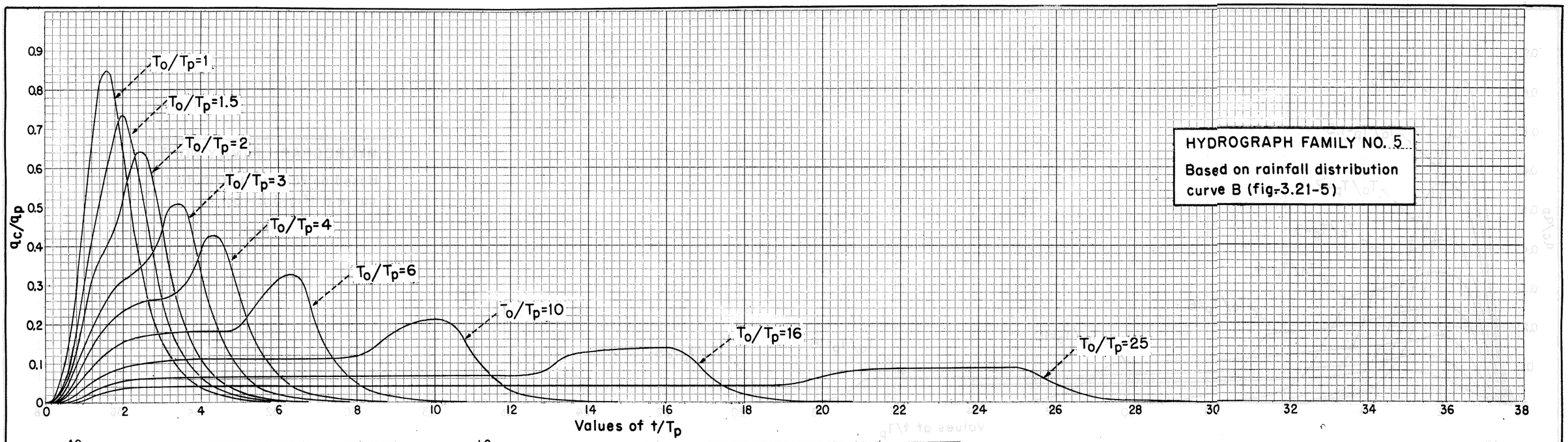
Figure 3.21-13
(4 of 5)

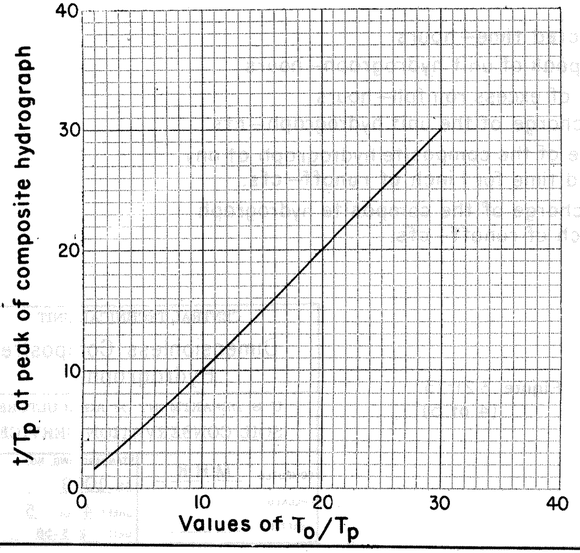
CENTRAL TECHNICAL UNIT

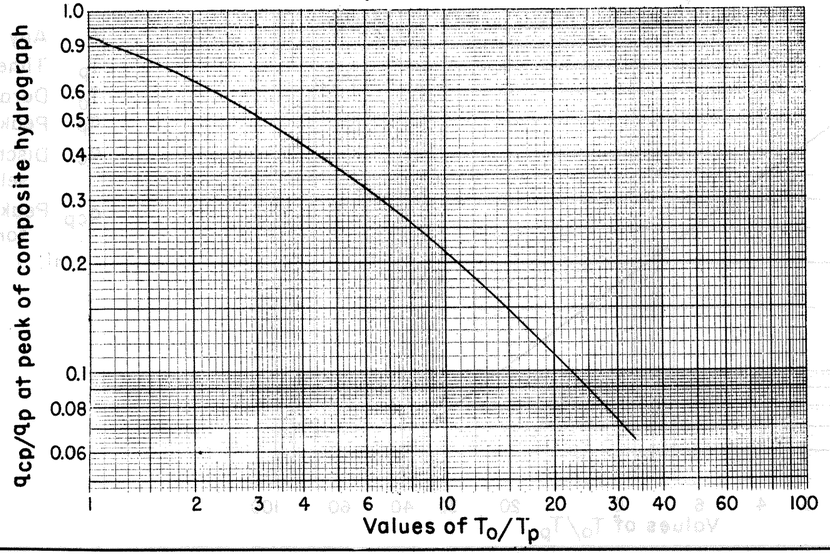
Dimensionless Composite Hydrographs

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

COMPUTED	M.H.K.	STANDARD DWG. NO.	ES-1009
CHECKED		SHEET	4 OF 5
DRAFTED		DATE	2-3-58







t ..... Any selected time—hours
T_p ..... Time of peak of unit hydrograph—hours
T_o ..... Duration of excess rainfall—hours
q_p ..... Peak discharge of the unit hydrograph—cfs.
q_c ..... Discharge of the composite hydrograph at any selected time for 1 inch of runoff—cfs.
q_{cp} ..... Peak discharge of the composite hydrograph for 1 inch of runoff—cfs.

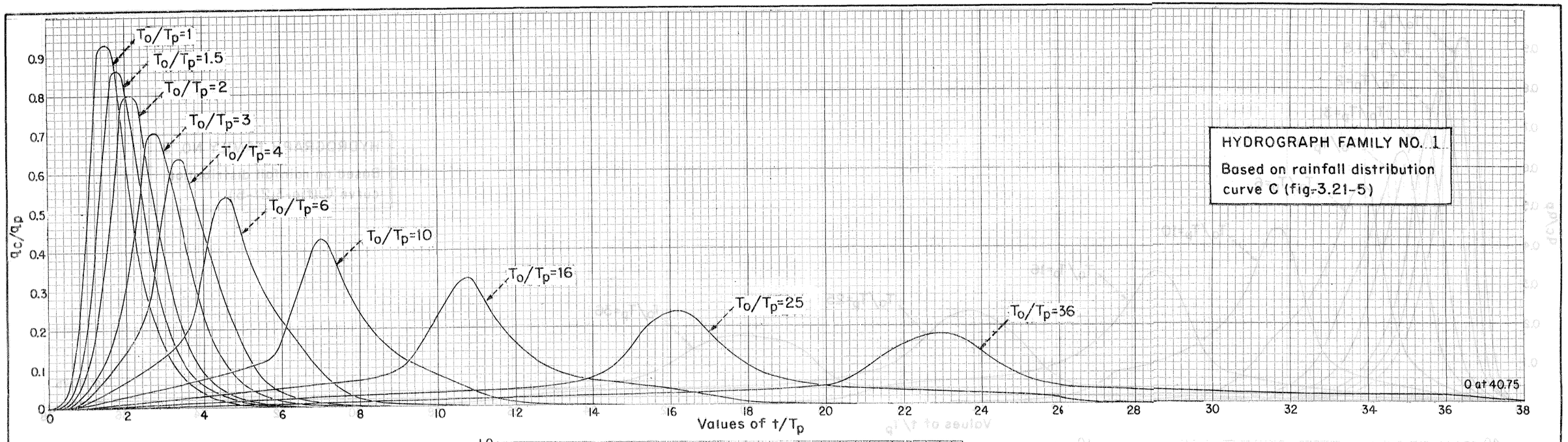
Figure 3.21-13
(5 of 5)

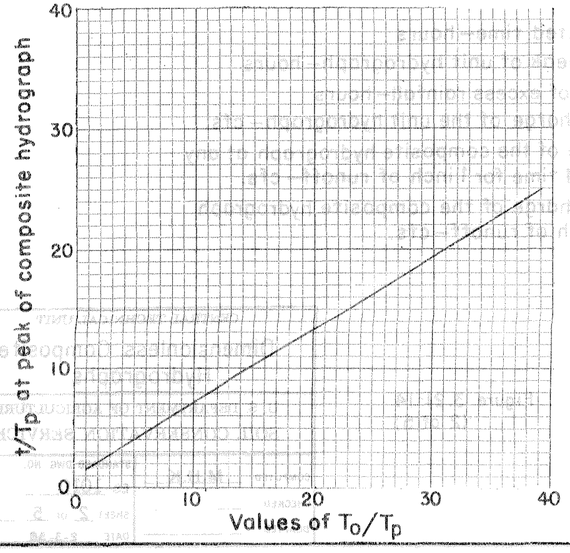
CENTRAL TECHNICAL UNIT

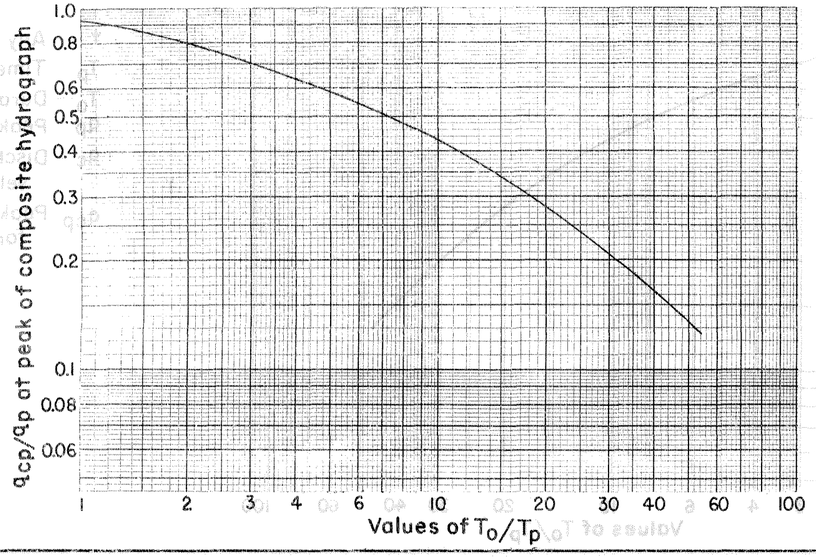
Dimensionless Composite Hydrographs

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

COMPUTED M.H.K.	STANDARD DWG. NO. ES-1009
CHECKED	SHEET 5 OF 5
DRAFTED	DATE 2-3-58







t — Any selected time—hours
T_p — Time of peak of unit hydrograph—hours
T_o — Duration of excess rainfall—hours
q_p — Peak discharge of the unit hydrograph—cfs.
q_c — Discharge of the composite hydrograph at any selected time for 1 inch of runoff—cfs.
q_{cp} — Peak discharge of the composite hydrograph for 1 inch of runoff—cfs.

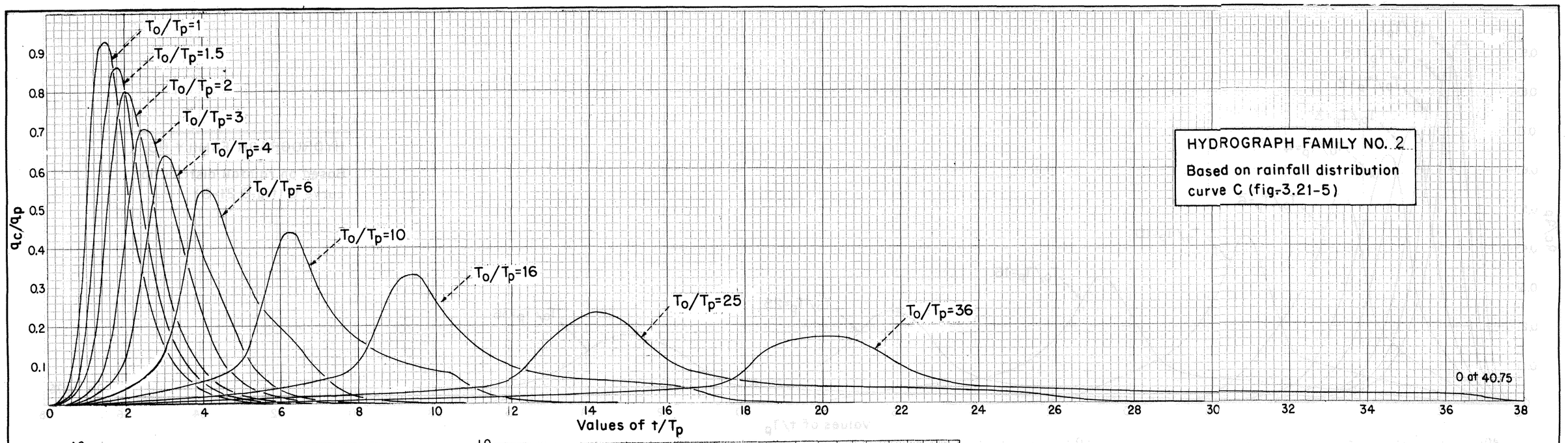
Figure 3.21-14
(1 of 5)

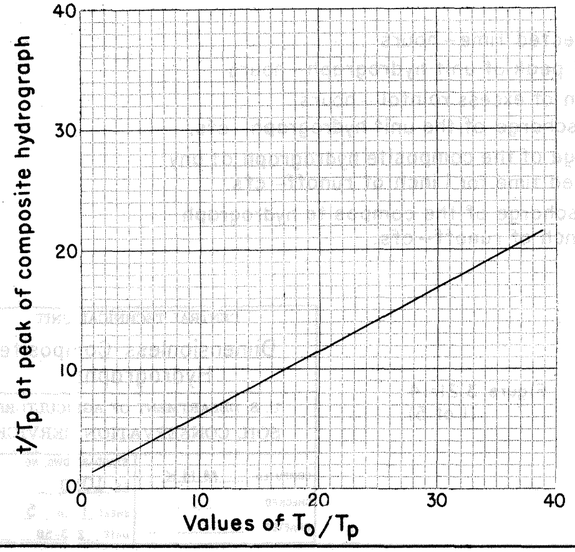
CENTRAL TECHNICAL UNIT

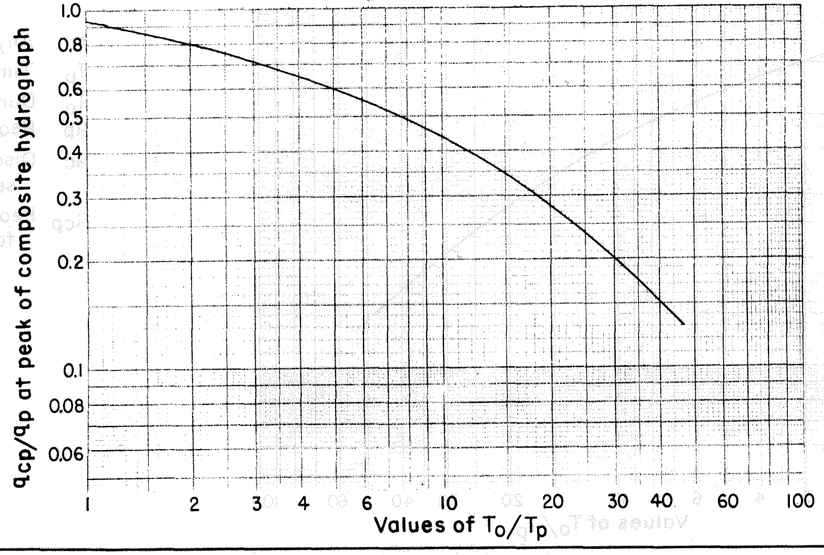
Dimensionless Composite Hydrographs

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

COMPUTED	M.H.K	STANDARD DWG. NO. ES-1010
CHECKED		SHEET 1 OF 5
DRAFTED		DATE 2-3-58







t ..... Any selected time— hours
T_p ..... Time of peak of unit hydrograph— hours
T_o ..... Duration of excess rainfall— hours
q_p ..... Peak discharge of the unit hydrograph— cfs.
q_c ..... Discharge of the composite hydrograph at any selected time for 1 inch of runoff— cfs.
q_{cp} ..... Peak discharge of the composite hydrograph for 1 inch of runoff— cfs.

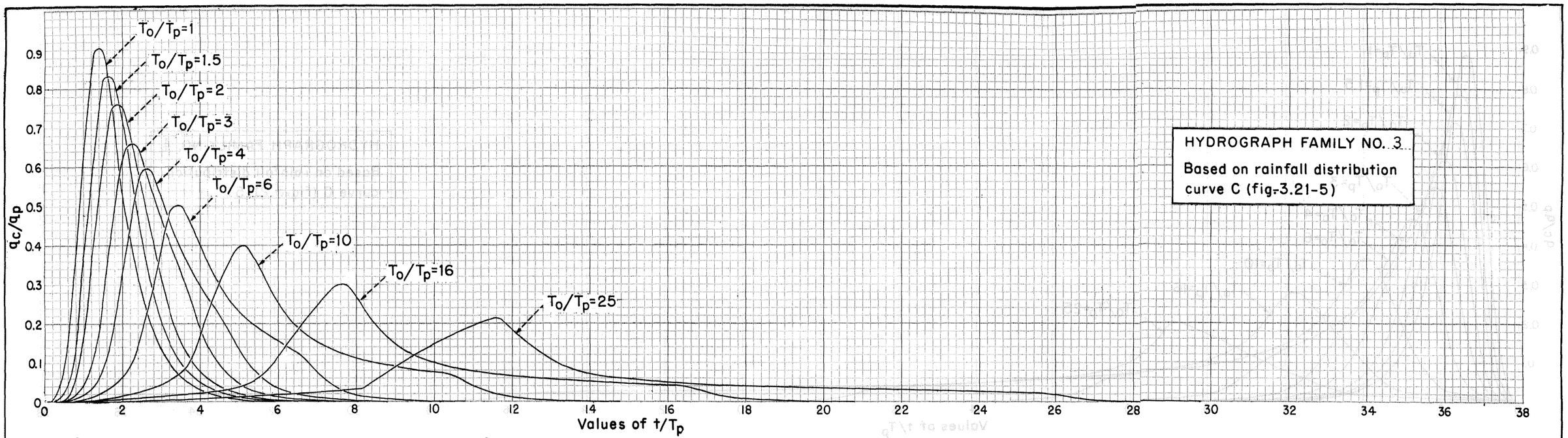
Figure 3.21-14
(2 of 5)

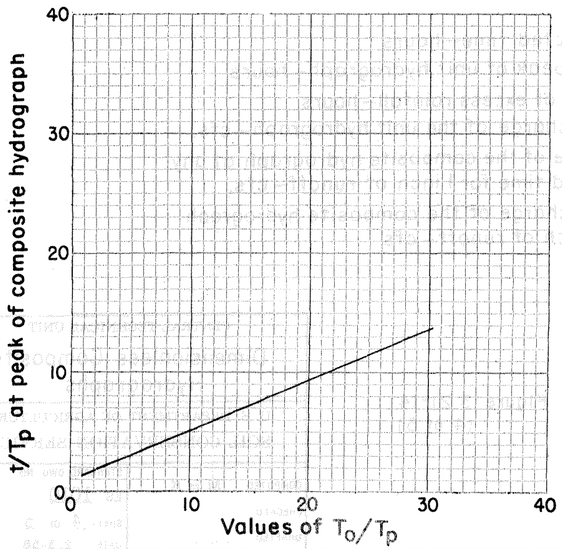
CENTRAL TECHNICAL UNIT

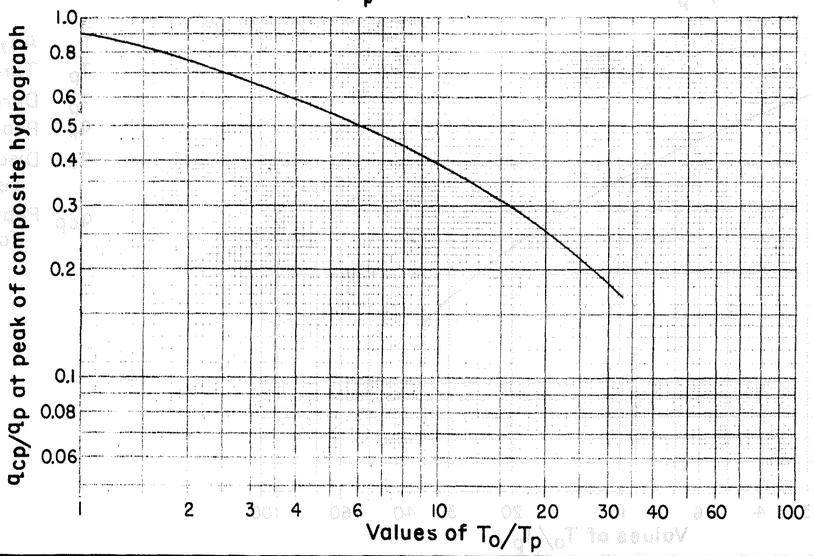
Dimensionless Composite Hydrographs

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

COMPUTED M.H.K	STANDARD DWG. NO. ES-1010
CHECKED	SHEET 2 OF 5
DRAFTED	DATE 2-3-58







t ..... Any selected time—hours
T_p ..... Time of peak of unit hydrograph—hours
T_o ..... Duration of excess rainfall—hours
q_p ..... Peak discharge of the unit hydrograph—cfs.
q_c ..... Discharge of the composite hydrograph at any selected time for 1 inch of runoff—cfs.
q_{cp} ..... Peak discharge of the composite hydrograph for 1 inch of runoff—cfs.

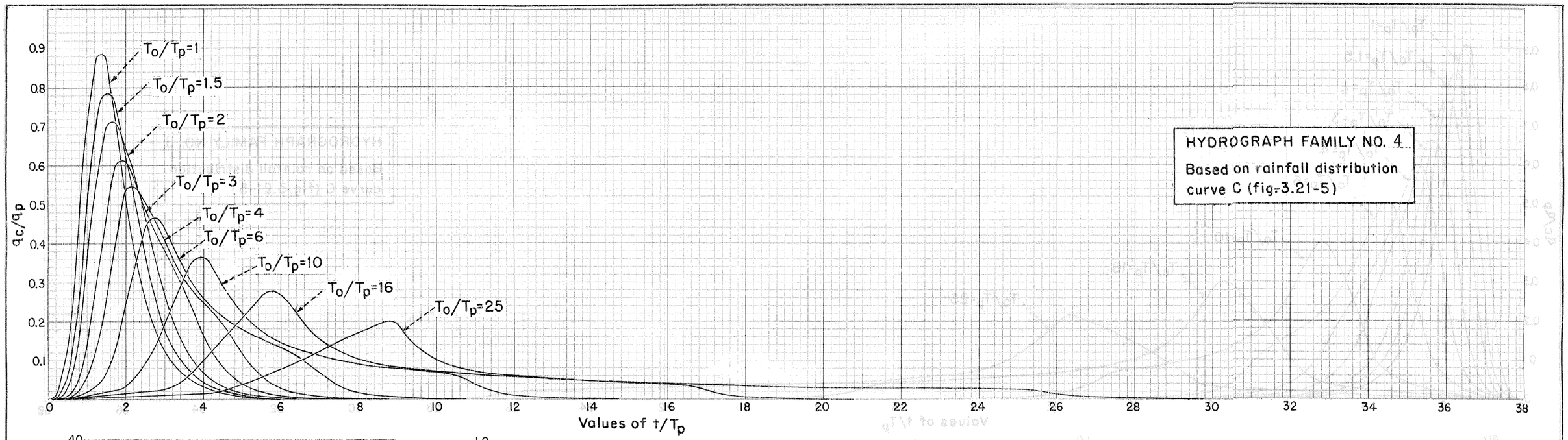
Figure 3.21-14
(3 of 5)

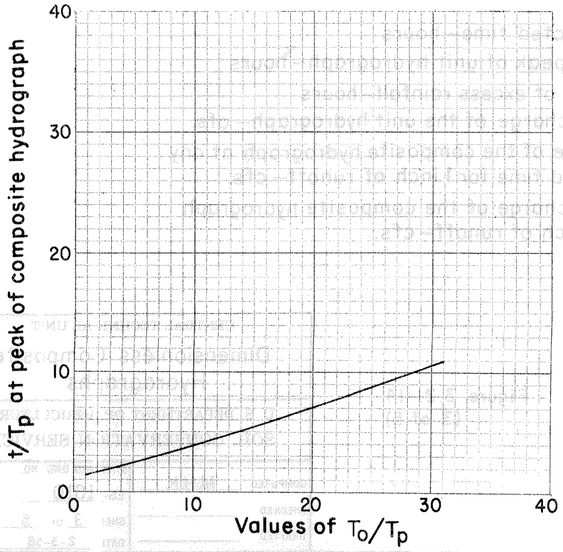
CENTRAL TECHNICAL UNIT

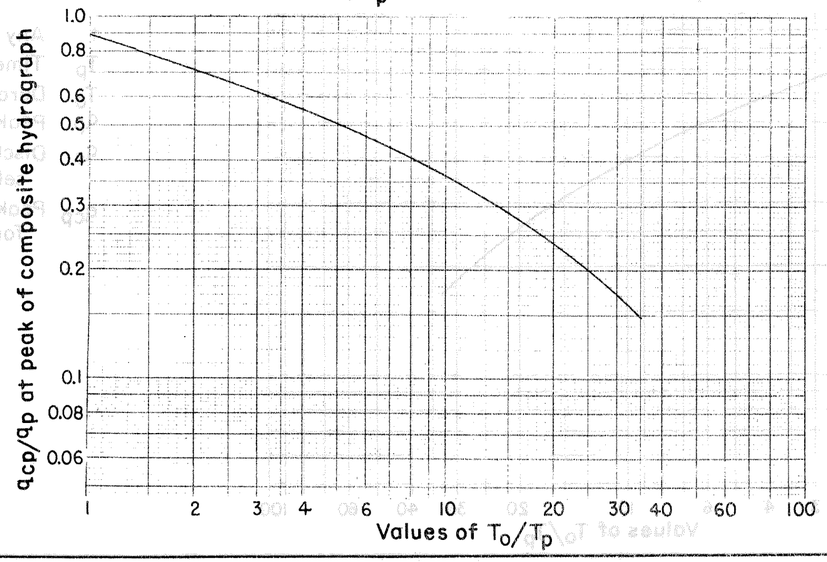
Dimensionless Composite Hydrographs

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

COMPUTED M.H.K	STANDARD DWG. NO. ES-1010
CHECKED	SHEET 3 OF 5
DRAFTED	DATE 2-3-58







t — Any selected time—hours

T_p — Time of peak of unit hydrograph—hours

T_o — Duration of excess rainfall—hours

q_p — Peak discharge of the unit hydrograph—cfs.

q_c — Discharge of the composite hydrograph at any selected time for 1 inch of runoff—cfs.

q_{cp} — Peak discharge of the composite hydrograph for 1 inch of runoff—cfs.

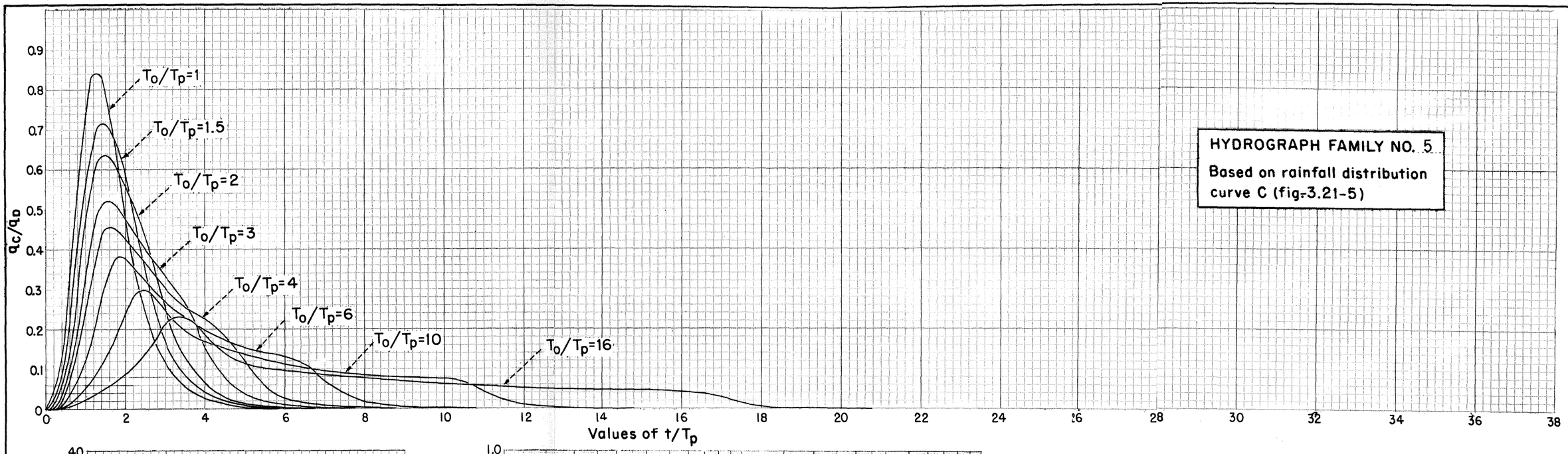
Figure 3.21-14
(4 of 5)

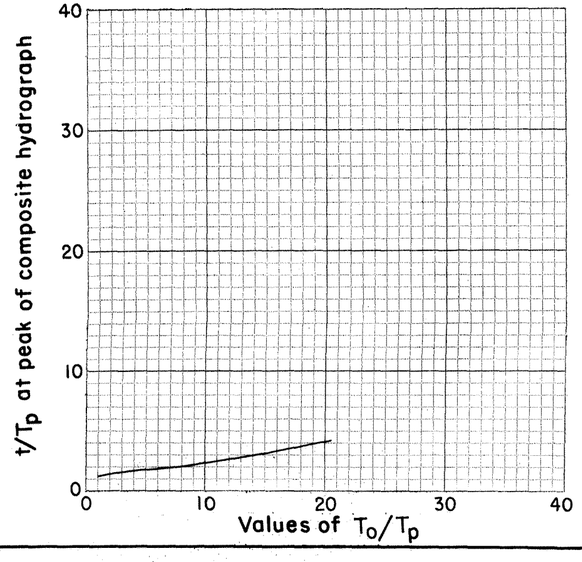
CENTRAL TECHNICAL UNIT

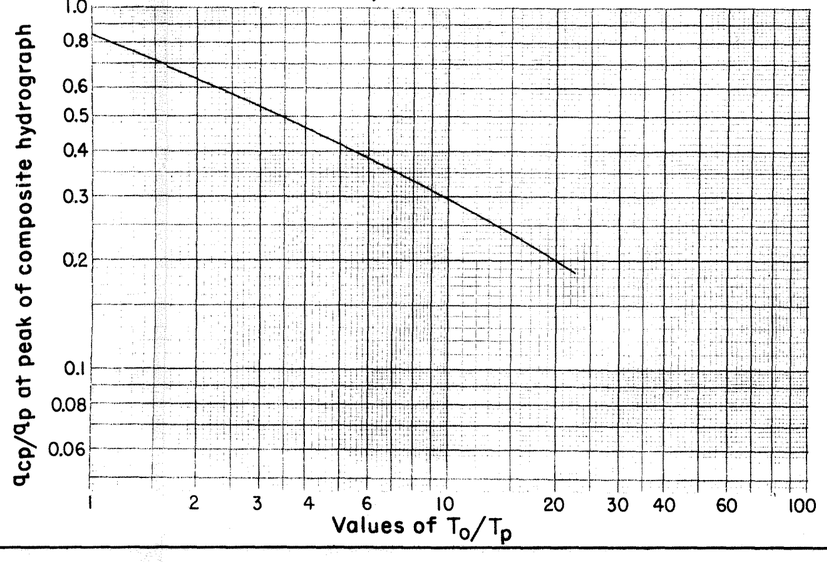
Dimensionless Composite Hydrographs

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

COMPUTED M.H.K.	STANDARD DWG. NO. ES-1010
CHECKED	SHEET 4 OF 5
DRAFTED	DATE 2-3-58







t ..... Any selected time—hours

T_p ..... Time of peak of unit hydrograph—hours

T_o ..... Duration of excess rainfall—hours

q_p ..... Peak discharge of the unit hydrograph—cfs.

q_c ..... Discharge of the composite hydrograph at any selected time for 1 inch of runoff—cfs.

q_{cp} ..... Peak discharge of the composite hydrograph for 1 inch of runoff—cfs.

Figure 3.21-14
(5 of 5)

CENTRAL TECHNICAL UNIT

Dimensionless Composite Hydrographs

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

COMPUTED M.H.K.	STANDARD DWG. NO. ES. 1010
CHECKED	SHEET 5 OF 5
DRAFTED	DATE 2-3-58

PART 4. APPLICATIONS IN NATURAL WATERSHEDS

This part is to contain examples illustrating the application of the methods presented in Parts 2 and 3 to hydrology problems studied by Service personnel. Time limitations have permitted the inclusion of only a few examples. Additional examples, primarily those submitted by Engineering and Watershed Planning Unit hydrologists, will be added later to illustrate further the various methods and procedures presented in this Guide.

426963 O - 57 - 20

4.1 Water surface profiles; variation of Leach's method

This variation of Leach's method was developed by William J. Owen, of the Lincoln Engineering & Watershed Planning Unit. It is a trial-and-error method, but a relatively simple one. Changes in velocity head are ignored, so that the method is not suitable where those changes are likely to be important.

The data are from Silver Creek Watershed, South Dakota. Cross section A is at the lowermost end of the watershed. Other sections are lettered progressively upstream.

The following information is used:

1. Channel and valley cross section data. (Elevations at all sections must be referenced to the same datum plane.)

2. Drainage area above each section.

3. Channel and valley distances between sections.

The method is as follows:

1. Plot the cross sections on arithmetic graph paper. (The Silver Creek sections for this example are not shown.)

2. Compute K_d values at selected elevations of each section. In this case, elevations at one-foot intervals were used because of the wide, flat floodplain.

(Note: The K_d equations are:

$$K_d = \frac{1.486\ a\ r^{2/3}}{n} \qquad (1)$$

$$K_d = \frac{q}{s^{1/2}} \qquad (2)$$

where a = cross section area in square feet
r = hydraulic radius
n = Manning's n
q = discharge in cfs
s = slope

Drawing ES-76 (NEH-5) may be used to determine F, with:

$$K_d = \frac{F}{n} \qquad (3)$$

Table 1 on sheet 4 of ES-83 (NEH-5) shows typical computations, with total K_d values given in column 11. (Column 12 is not used in this method.)

3. Plot K_d values versus elevation and draw curves, as shown on figure 4.1-1. Semilogarithmic paper is generally the most suitable paper on which to draw the curves.

4. Check for critical and supercritical flow if it appears that such flow is likely to occur in a reach for most of the discharges required in the watershed project evaluations. The method described on sheet 4 of ES-83 (NEH-5) may be used for the check. At this time, bridges and other possible constrictions on flow may also be investigated, keeping in mind, however, that such constrictions must operate during the important discharges before it is worthwhile using special methods for this evaluation. See Sec. 3.14 for methods in these special cases.

5. Determine or estimate depths of flow for the needed discharges at the lowermost cross section. In this case, the elevation versus discharge relation for cross section A was estimated by using the slope downstream from A, and multiplying the K_d values at A by the square root of the slope. When closer estimates of depth at section A are needed, more cross sections downstream from A can be included in the field survey, and the computations started at a section farther downstream. The converging effect of the profiles as they are computed in the upstream direction is discussed below.

6. Prepare table 4.1-1. The discharge and elevation is listed for the lowest cross section (section A). The K_d value is read from figure 4.1-1. For the next section upstream (B), tabulate the discharge and channel length from the downstream section. The first estimate for the elevation at B may be made by computing the elevation at B using a K_d value that is about the same as the one for A. The square root of the slope equals q divided by K_d. H_1 is the length multiplied by the slope. E_2 is H_1 plus the elevation at the downstream section (E_1). The assumed elevation (E_1) is "correct" when it is about equal to the computed elevation (E_2). At cross section B, E_1 was estimated to be 27.45 and E_2 was computed to be 27.05. The required slope is less than the actual slope, and the desired elevation is between 27.05 and 27.45. A second estimate of 27.35 is made for E_1, and E_2 is computed to be 27.40. The required slope is more than the actual slope, and the desired elevation is between 27.35 and 27.45. Since the E_2 value of 27.40 is very close to the E_1 value of 27.35, the final E_1 value of 27.37 is used for station B.

The K_d curve of section C crosses that for section B at the same elevation for K_d values less than 25,000. This may sometimes be an error in surveying or in computing K_d values, and should be checked. In this case, E_1 could not be estimated by using the same K_d value for B as the elevation decrease at C. The elevation at B was computed as 27.37. The elevation at C will be some higher value. The slope will be flat as the K_d value for section C will be much higher. The preliminary value of 27.87 was estimated by arbitrarily adding 0.50 to 27.37.

Table 4.1-2 shows the computations using a 100-csm discharge. Enough points to plot stage-versus-discharge curves were obtained by computing 10-csm, 40-csm, 100-csm, and 250-csm discharges.* In other watersheds, computation of additional discharges may be required to define an adequate stage-discharge curve.

Table 4.1-3 shows that a large difference in starting elevations at cross section A becomes negligible at upstream sections since the profiles tend to converge. The E_1 value at A was assumed to be 29.50, instead of 25.60 as on table 4.1-1. The resulting elevation at D is almost identical for both conditions. When closer estimates at A are required, several cross sections should be obtained downstream from A, and the calculations started at a downstream cross section.

If the lowermost cross section in the stream is near the watershed juction with a major stream, the depth of flow at that cross section will be dependent on the water-surface elevation of the major stream. Similar computations on the major stream may be required to determine the stage-discharge relation at the lowermost cross section.

In computing the water surface profiles, the distance between cross sections (reach length) is an important factor. In natural channels having considerable meander, the actual length will change in overbank flows as the stage changes. If only the actual length of the channel is used with high flood flows, considerable error may result. In Silver Creek Watershed, the channel and the flood-plain lengths were determined from 8-inch aerial photos. These lengths are tabulated in column 4 on table 4.1-2. The flood-plain and channel lengths were the same from cross section A to D because the channel has been straightened in those reaches. The adjusted flood-plain lengths were used when computing the high rates of flow (100 to 250 csm). The channel length was used for the 10- and 40-csm rates. Discharge rates of 40 csm produced overbank flow, but for that case it was assumed that the majority of the flow would follow the channel path. This type of adjustment for channel length is not exact, but often adequate.

*Four points and the bottom elevation were adequate for drawing the stage-discharge curves.

Table 4.1-1. Water surface profile computations.
Silver Creek Watershed, South Dakota

q = 10 csm

Section	A	q	L 1/	Elevations E_1	K_d	$s^{1/2}$	s	H_1	Elevations E_2
	(sq.mi.)	(cfs)	(ft.)	(ft.)				(ft.)	(ft.)
A	24	240	-	25.60	9900				
B	23	230	2740	27.45	10000	0.023	0.0053	1.45	27.05
				27.35	9000	.0256	.0065	1.80	27.40
				27.37					
C	22	220	3040	27.87	21000	.0105	.00011	.33	27.70
				27.80	20200	.0109	.00012	.36	27.73
				27.77					
D	21	210	5940	31.17	15000	.014	.00020	1.16	28.93
				30.50	8000	.0262	.00069	4.07	31.84
				30.75	10300	.0204	.00042	2.05	29.82
				30.60	9000	.0233	.00054	3.22	30.99
				30.65					
E	20	200	5020	32.15	10300	.0194	.00038	1.89	32.54
				32.25	12200	.0164	.00027	1.35	32.00
				32.20					
F	19	190	8390	32.90	12000	.0158	.00025	2.10	34.30
				33.30	16700	.0114	.00013	1.09	33.29
				33.30					
G	18	180	3960	33.70	16050	.0112	.00013	.50	33.80
				33.73	17000	.0106	.00011	.44	33.74
				33.74					
H	17	170	5610	36.54	16100	.0106	.00011	.63	34.37
				35.60	9900	.0172	.00030	1.65	35.39
				35.50	9300	.0183	.00033	1.88	35.62
				35.54					

1/ Low flow channel length only

Table 4.1-2. Water surface profile computations.
Silver Creek Watershed, South Dakota

q = 100 csm

Section	A	q	L 1/	Elevations E_1	K_d	$s^{1/2}$	s	H_1	Elevations E_2
	(sq.mi.)	(cfs)	(ft.)	(ft.)				(ft.)	(ft.)
A	24	2400	-	26.75	85000	-	-	-	-
B	23	2300	2740	29.06	84000	0.0274	0.00075	2.06	28.81
			(2740)	29.00	78000	.0294	.00087	2.38	29.13
				29.03					
C	22	2200	3040	30.13	85000	.0259	.00067	2.04	31.07
			(3040)	30.35	114000	.0193	.00037	1.13	30.16
				30.32					
D	21	2100	5940	32.80	113000	.0186	.00035	1.56	31.88
			(4520)	32.60	91000	.0231	.00053	2.41	32.73
				32.65					
E	20	2000	5020	33.65	98000	.0204	.00042	1.60	34.25
			(3830)	33.75	110000	.0182	.00033	1.27	33.92
				33.80					
F	19	1900	8390	35.10	122000	.0156	.00024	1.53	35.33
			(6300)	35.15	128000	.0148	.00022	1.39	35.19
				35.16					
G	18	1800	3960	35.70	130000	.0139	.00019	.68	35.84
			(3530)	35.75	135000	.0133	.00018	.63	35.79
				35.77					
H	17	1700	5610	39.20	147000	.0116	.00013	.57	36.34
			(4290)	38.50	72000	.0236	.00056	2.39	38.16
				38.40	67000	.0254	.00064	2.77	38.54
				38.43					

1/ Upper figure, low flow channel length. Lower figure (in parenthesis) flood plain or valley length.

Table 4.1-3. Water surface profile comparative computations. Silver Creek Watershed, South Dakota

q = 100 csm

Section	A	q	L	Elevations E_1	K_d	$s^{1/2}$	s	H_1	Elevations E_2
	(sq.mi.)	(cfs)	(ft.)	(ft.)				(ft.)	(ft.)
A	24	2400	-	29.50 1/	960000	-	-	-	-
B	23	2300	2740	29.80	172000	0.0134	0.000179	0.49	29.99
				29.87	188000	.01223	.000150	.41	29.91
				29.89					
C	22	2200	3040	30.55	148000	.0149	.000221	.67	30.56
				30.55					
D	21	2100	5940	32.70	102000	.0206	.000424	1.92	32.47
			(4520)	32.65	97000	.0216	.000468	2.12	32.67
				32.66					
E 2/									
F 2/									
G 2/									
H 2/									

1/ E_1 value of 29.50 assumed for comparing results with those in table 4.1-2 where an E_1 value of 26.75 was used at this section. See figure 4.1-2 for the plotted water surface profiles.

2/ E_2 value same as determined in table 4.1-2.

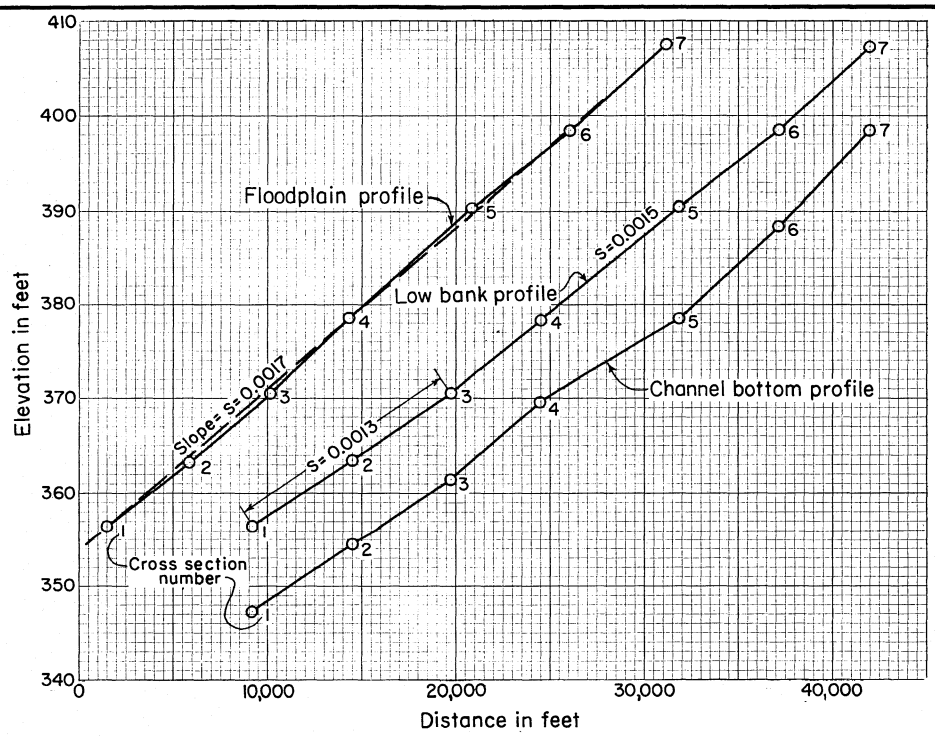


Figure 4.2-1

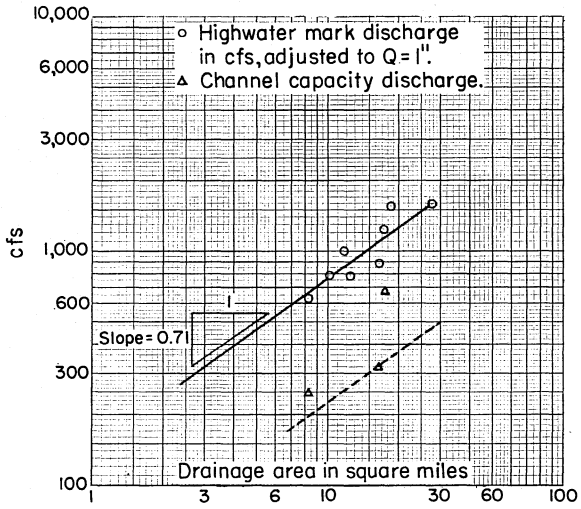


Figure 4.2-2

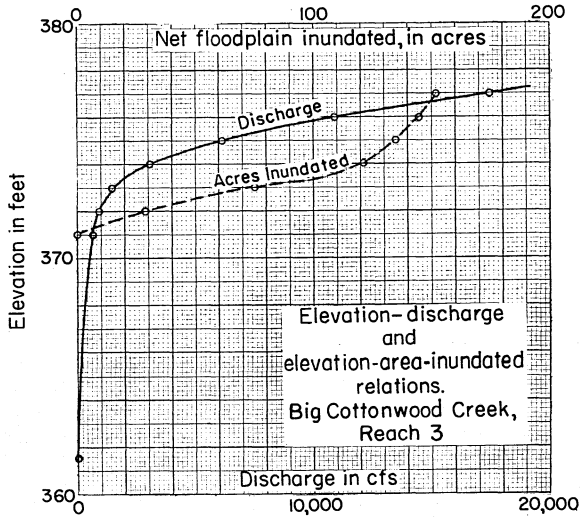


Figure 4.2-3

4.2 Concordant flow

This illustration was submitted by R. G. Andrews, and shows how the concordant flow method was used on the Big Cottonwood subwatershed of Cedar Creek Watershed, Texas. Section 2.7 and the flow chart, figure 2.7-1, should also be consulted.

Figure 4.2-1 shows the stream and valley profiles of the subwatershed which indicate no unusual profile conditions.

Table 4.2-1 gives the computation of weighted rainfall of the selected recent flood, and the corresponding estimated runoff.

Table 4.2-2 shows the computations needed in the adjustment of high-water mark discharges. The adjustment is made by plotting drainage area (col. 2) versus the discharge for Q = 1" (col. 5), as shown on figure 4.2-2. A straight line is drawn through the plotted points, following a general trend, or, as sometimes done, by fitting a line with an average slope for all watershed points to a subwatershed plotting so that an equal number of points falls on both sides of the line. In this case, the slope of the line is 0.707 or 0.71. This is determined, after the line is drawn, by using the equation:

$$s = \frac{\log\left[\frac{q_2}{q_1}\right]}{\log\left[\frac{A_2}{A_1}\right]} \qquad (1)$$

where

s = computed slope
q_1 = discharge on line at area 1
q_2 = discharge on line at area 2
A_1 = area 1 (the lesser area)
A_2 = area 2 (the greater area)

and usually logarithms are to base 10. The slope can often be determined closely enough by measuring with an engineer's scale, as shown on figure 4.2-2. This measuring method can be used on log and on arithmetic coordinate paper, but not on semilog paper.

Table 4.2-1. Computation of weighted rainfall, for selected large flood, Big Cottonwood subwatershed.

Section number	Accumulated drainage area	Incremental areas	Rainfall over incremental areas	Col. 3 times Col. 4, accumulated	Weighted rainfall (Col. 5 ÷ Col.2)	Runoff for weighted rainfall 1/
	(sq. mi.)	(sq.mi.)	(in.)	(in. x mi.)	(in.)	(in.)
		8.32	8.30			
DS-1	8.32			69.06	8.30	4.37
		1.99	7.25			
7	10.31			83.48	8.10	4.22
		1.30	6.70			
6	11.61			92.19	7.94	4.10
		.75	6.00			
5	12.36			96.69	7.82	4.02
		3.93	6.25			
4	16.29			121.26	7.44	3.74
		.95	5.30			
3	17.24			126.29	7.33	3.65
		1.50	5.15			
2	18.74			134.02	7.15	3.52
		9.00	6.00			
1	27.74			188.02	6.78	3.25

(Note: Computation of weighted runoff can also be made by getting runoff for rainfalls of column 4 and weighting by drainage area as above.)

1/ Using figure 3.10-1.

Table 4.2-2. Adjustment of high-water mark discharges for selected recent flood, Big Cottonwood subwatershed.

Section number	Drainage area	Discharge at HWM	Runoff 1/	Discharge 2/ (Q - 1") (Col. 3 ÷ Col. 4	Adjusted discharges 3/ (Q = 1")	Adjusted HWM discharge (Col.4 x Col.6)
	(sq. mi.)	(cfs)	(in.)	(cfs)	(cfs)	(cfs)
DS-1	8.32	2,750	4.37	630	670	2,930
7	10.31	3,320	4.22	790	785	3,310
6	11.61	4,100	4.10	1,000	855	3,510
5	12.36	3,190	4.02	790	920	3,690
4	16.29	3,300	3.74	880	1,080	4,030
3	17.24	4,310	3.65	1,180	1,130	4,130
2	18.74	5,510	3.52	1,560	1,200	4,220
1	27.74	5,135	3.25	1,580	1,580	5,130

1/ From column 7 of table 4.2-1.

2/ These discharges are plotted versus drainage area (col. 2) as shown on figure 4.2-2.

3/ Using the relation of figure 4.2-2 as described in the text.

The computations for stage-discharge curves were previously made on a preliminary basis, as shown in table 4.2-3. The HWM discharge at cross section 3 is 4.310 cfs for Q = 3.65" (table 4.2-2), and 4,310/3.65 = 1180 cfs, which is plotted versus 17.24 square miles on figure 4.2-2. After the line is drawn, an adjusted discharge of 1130 cfs is obtained, and (1130) (3.65) = 4,130 cfs, which is the adjusted HWM discharge. The adjusted "n" value in table 4.2-3 is obtained using the relation:

$$n_a = \frac{n_p \, q_p}{q_a} \qquad (2)$$

where n_a = adjusted n value

n_p = preliminary n value

q_p = preliminary discharge

q_a = adjusted discharge

In this case,

$$n_a = \frac{(0.070)(4,310)}{(4,130)} = 0.073$$

at the HWM. This "n" value was used as a guide in adjusting "n" values for higher and lower elevations, as shown in table 4.5-3, and new discharges were computed as shown. In the usual case, the "n" values for bankfull discharges at all sections would be similarly adjusted to provide another concordant flow agreement. In this case, the bankfull "n" value was not adjusted, though a line parallel to the one shown on figure 4.2-2 was drawn to envelop the lowest bankfull discharge in the reach, and from that line, to find the equivalent at the reference cross section. This discharge was used later as a minimum base for selection of storms in the series.

The historical storm series was developed next by selecting all storms with rainfall greater than that needed to start flooding at any point in the reach. Although the storm series was developed for use over the whole watershed, its method of development is shown using only Big Cottonwood subwatershed data: (1) channel capacities in cfs were plotted on figure 4.2-2. (2) The concordant flow line was brought down parallel and fitted to the lowermost point, that for section 4. (3) The discharge on this line at the drainage area for the reference section (section 3) was read to be 335 cfs. (4) This discharge was divided by the discharge for Q = 1", to find the amount of runoff at which flooding begins (Q_{fb}), or:

$$Q_{fb} = \frac{335}{1130} = 0.296" \ .$$

Table 4.2-3. Stage-discharge computation work sheet

Watershed Cedar Creek
Subwatershed
Section 3

	CHANNEL SECTION: n = .050, s = .0015, $s^{1/2}$ = .03873						OVERFLOW SECTION: s = .0017, $s^{1/2}$ = .04123							TOTAL			
Elevation of water surface (feet)	Area of section (sq.ft.)	Width of water surface (feet)	Wetted Perimeter (feet)	Hydraulic radius (a/p)		$\frac{1.486\ ar^{2/3}\ s^{1/2}}{n}$	Area of section (sq.ft.)	Width of water surface (feet)	Wetted Perimeter (feet)	Hydraulic radius (a/p)		Coefficient of roughness	$\frac{1.486\ ar^{2/3}\ s^{1/2}}{n}$	Overflow and channel section combined	Total acres floodplain and channel combined (Ac./ft. w.)	Acres in channel (Ac./ft.w.)	Net flood plain acres
M.S.L.	a	L	p	r	$r^{2/3}$	Q	a	L	p	r	$r^{2/3}$	n	Q	Q	.0973	.1151	
361.5	Bottom of channel																
371.0	210	35	45	4.67	2.794	675						*	0	675	5	5	0
372.0	248	40	50	4.96	2.908	830	170	350	310	0.55	.671	.075	93	923	34	5	29
373.0							830	820	780	0.94	.960	.075	650	1480	80	5	75
374.0							2000	1290	1250	1.60	1.368	.075	2235	3065	126	5	121
375.0							3375	1440	1400	2.41	1.797	.070	5310	6140	140	5	135
376.0							4870	1540	1500	3.23	2.185	.065	10,030	10,860	150	5	145
377.0							6455	1610	1570	4.01	2.524	.060	16,636	17,466	157	5	152
374.4 (high-water mark)							2550	1350	1310	1.95	1.561	.073	3,300	4,130			

*n values adjusted to bring Q to concordant flow line. See figure 4.2-2.

Flood plain length of reach = 4240 ft.

$\frac{4240}{43560}$ = 0.0973 acre per foot of average width

Channel length in reach = 5015 ft.

$\frac{5015}{43560}$ = 0.1151 acre per foot of average width

(5) The watershed rainfall-runoff curves for conditions I, II, and III were entered with Q_{fb} to find the rainfall (P_{fb}) at which flooding begins. (6) The historical rainfall series was selected using rainfalls greater than the minima in each watershed condition. (7) Rainfalls that would not be used in the series because of proximity or season were discarded. Table 4.2-4 shows a sample of the storm series, together with estimated runoff. The estimated runoffs should then be used to check the historical series for adequacy of length of record by the method of Section 3.18. In this particular case, figure 3.18-3 was not yet available, and the historical series was taken between dates previously used as bracketing a sufficiently long period.

Curves of runoff versus area inundated were next developed: (1) A series of runoff amounts was selected to cover the range of usefulness from the smallest to the largest damaging floods in the evaluation series. (2) A computation, as shown by table 4.2-5, was made for each runoff amount. (3) The results were used to prepare a graph, figure 4.2-4, of runoff amounts versus area inundated.

The storm series runoffs for present conditions (column 3 of table 4.2-4) were used with figure 4.2-4 to get and tabulate the acres inundated for each storm for each depth increment.

The storm series runoffs for conditions with future expected land treatment measures in place (column 4 of table 4.2-4) were used with figure 4.2-4 to get and tabulate the acres inundated for each storm for each depth increment.

Curves of runoff versus area inundated for the condition "with land treatment and structural measures in place" were developed. Table 4.2-6 shows a computation for a given runoff amount in the storm series. This is an application of equation 10a of section 3.17. A similar table was prepared for eight additional runoff amounts, covering the needed range from no inundation to maximum for the storm series. The curves were plotted (figure 4.2-5) and used with the runoffs in the storm series to get a tabulation of area inundated.

The hydrologic portion of the project evaluation was completed when the three acre-inundated tabulations (1) Present conditions, (2) With land treatment measures in place, and (3) With land treatment and structural measures in place, were given to the economist.

Table 4.2-4. Estimated runoffs for the historical storm series.

Storm date			Estimated Runoff		
			Storm rainfall	Present conditions	With future land treatment measures
			(inches)	(inches)	(inches)
1923	June	2-3	7.10	4.85	4.30
	Sept.	15-16	6.60	2.37	2.11
1925	Feb.	22	1.78	.51	.37
	Nov.	5-7	2.49	.81	.64
1927	Oct.	2	6.71	2.43	2.17
1928	June	27-28	2.34	.83	.62
1929	May	13-14	1.96	.58	.40
1930	May	7-9	6.15	3.77	3.32
1932	May	15-16	5.80	3.47	3.04
1934	May	5-7	2.05	.64	.45
1941	May	4-6	4.35	2.28	1.94

426963 O -57 -21

Table 4.2-5. Computation of inundated area for a given runoff amount, Big Cottonwood subwatershed. For (1) Present, and (2) With land treatment conditions

Section number 1/	Discharge when Q = 1" 2/	Discharge 3/	Given runoff (Q) = 4.85 inches Elevation 4/	Area inundated 5/ Total	0-1'	1'-3'	3'+
	(cfs)	(cfs)	(ft.)	(acres)	(acres)	(acres)	(acres)
1	1600	7760		94	9	85	0
2	1200	5820		172	30	137	5
3 6/	1130	5480	374.6	130	18	93	19
4	1080	5240		197	40	120	37
5	880	4270		157	12	145	0
6	855	4150		157	19	138	0
7	785	3810		171	37	134	0
DS-1	670	3250		86	20	66	0
Totals:				1164	185	918	61

1/ Each section represents a reach.
2/ Column 2 from column 6 of table 4.2-2.
3/ Column 3 equals column 2 times 4.85.
4/ Column 4 shows elevation only for the reference section.
5/ From graphs like figure 4.2-3 for each section or reach.
6/ Used as the reference section for the subwatershed.

Table 4.2-6. Computation of inundated area for a given runoff amount, Big Cottonwood Subwatershed. For condition with land treatment and structural measures in place.

Given runoff (Q) = 4.85 inches

Section number 1/	Drainage area 2/	Area controlled 2/	Area not controlled 2/	Ratio 3/	Unmodified discharge 4/	Uncontrolled discharge 5/	Release discharge 6/	Modified discharge 7/	Modified elevation	Total	Area Inundated 8/ 0 - 1'	1'-3'	3' +
	(sq.mi.)	(sq.mi.)	(sq.mi.)		(cfs)	(cfs)	(cfs)	(cfs)	(feet)	(acres)	(acres)	(acres)	(acres)
1	27.74	10.57	17.17	0.6190	7660	4740	53	4793		88	22	66	0
2	18.74	10.57	8.17	.4360	5820	2540	53	2593		142	67	75	0
3 9/	17.24	10.57	6.67	.3869	5480	2120	53	2173	373.4	104	61	43	0
4	16.29	8.32	5.72	.3511	5260	1850	42	1892		150	53	97	0
5	12.36	8.32	4.04	.3269	4460	1460	42	1502		137	137	0	0
6	11.61	8.32	3.29	.2834	4150	1180	42	1222		133	123	10	0
7	10.31	8.32	1.99	.1930	3810	735	42	777		97	97	0	0
DS-1	8.32	8.32	0	0	3250	0	42	42		0	0	0	0
									Totals	851	560	291	0

1/ Each section represents a reach.
2/ At the section.
3/ Column 4 divided by column 2.
4/ Same as column 3 of table 4.2-5.
5/ Column 5 times column 6.
6/ Previously determined by hydrologist as desirable total release rate from structures. This rate is usually a bankfull discharge or less. Average rate from structures usually used. Average rate taken as 0.80 of rate with head at full pool elevation.
7/ Column 7 plus column 8.
8/ From graphs like figure 4.2-3 for each section.
9/ Used as a reference section for the subwatershed.

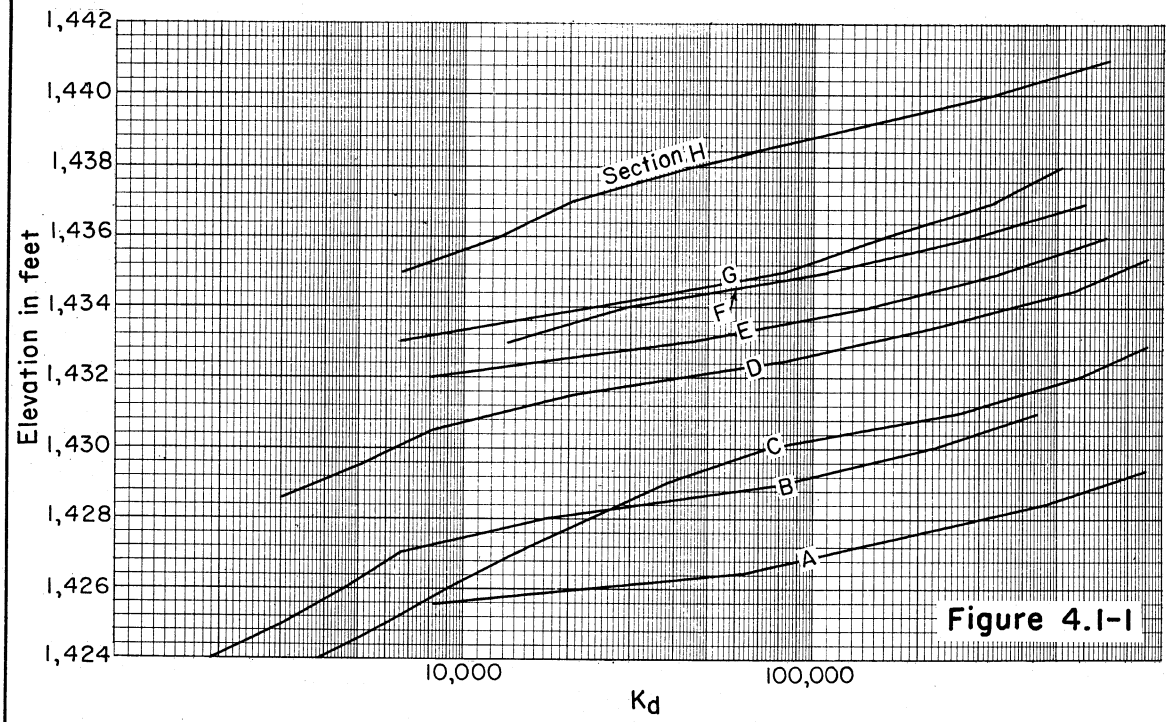
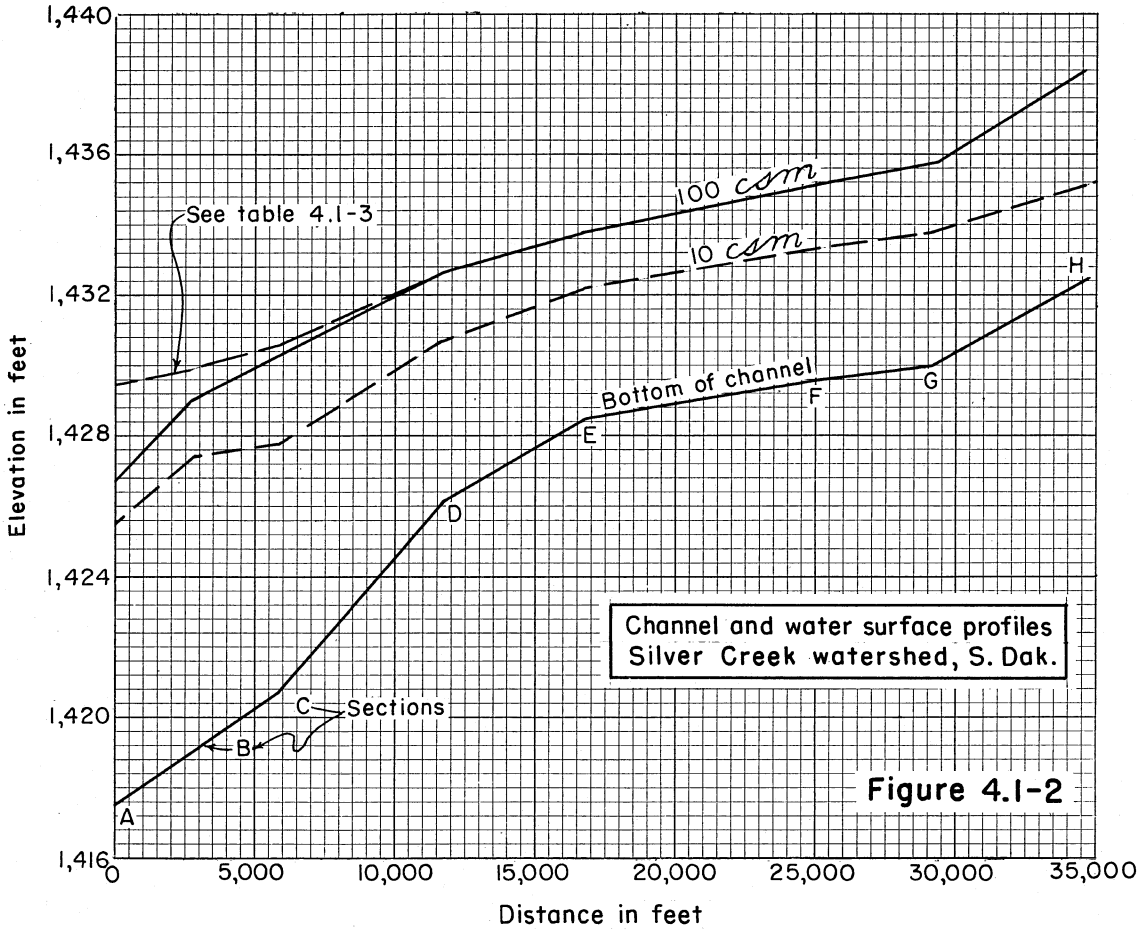


Figure 4.1-1



Channel and water surface profiles
Silver Creek watershed, S. Dak.

Figure 4.1-2

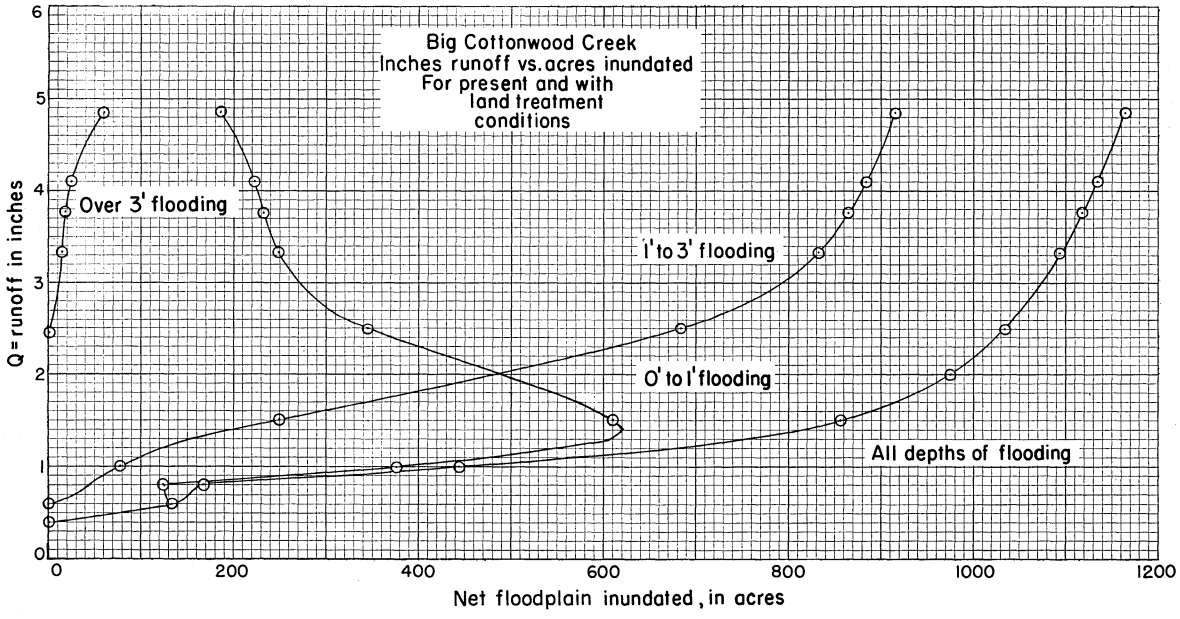


Figure 4.2-4

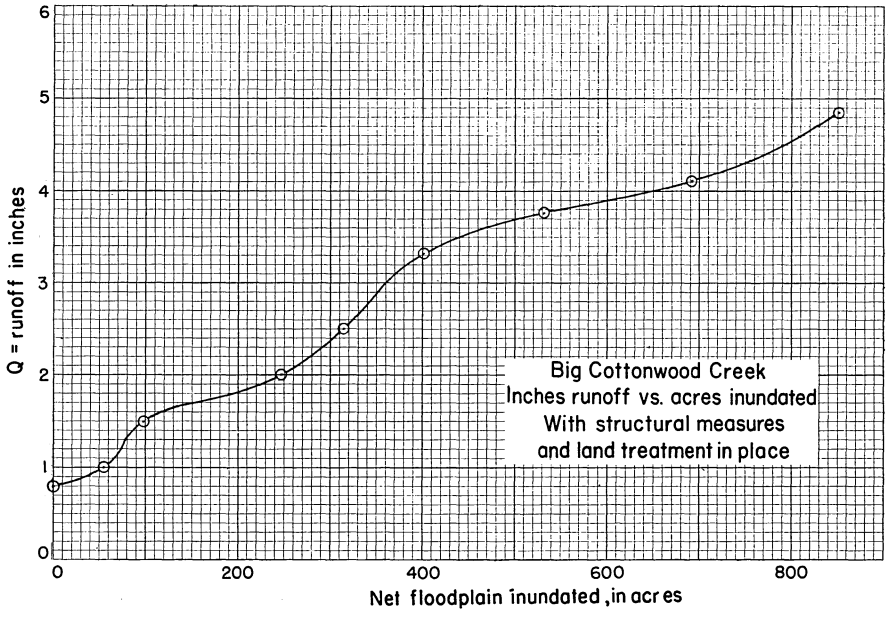


Figure 4.2-5

PART 5. DEFINITIONS AND CONVERSIONS

5.1 Definitions

A selected list of definitions of words and terms used in hydrologic evaluations of watershed projects is given. Other useful definitions are given in:

> Soil and Water Conservation Glossary, Soil Conservation Society of America, Des Moines, Iowa. 1952
>
> Weather Glossary, U. S. Department of Commerce, Weather Bureau, 1946. (For sale by Superintendent of Documents, U. S. Government Printing Office, Washington, D. C. for $1.00.)
>
> National Catalog of Practices and Measures Used in Soil and Water Conservation, Section IV, National Records and Reports Handbook, U. S. Department of Agriculture, Soil Conservation Service, November 1954.

Underlined words and terms in a definition are defined elsewhere in the list except for those terms preceded by a number. (Example: 3.4 Precipitation data), in which case the reference is to a section in the Hydrology Guide.

acre-foot -- The amount of water that will cover 1 acre to a depth of 1 foot. Equals 43,560 cubic feet. Abbreviated AF.

AF -- Abbreviation for acre-foot or acre-feet.

annual flood -- The highest rate of flow of a stream during a year. Used in flood frequency analysis.

annual runoff -- The total natural discharge of a stream for a year, usually expressed in inches depth or AF. See water yield.

annual series -- A frequency series in which only the largest value in each year is used, such as the annual floods.

annual yield -- The total amount of water obtained in a year from a stream, spring, artesian well, etc. Usually expressed in inches depth, AF, millions of gallons, or cubic feet.

antecedent moisture condition -- The degree of wetness of a watershed at the beginning of a storm. See 3.4 Precipitation data.

area rainfall -- The average rainfall over an area, usually as derived from, or discussed in contrast with, point rainfall.

base flow -- Stream discharge derived from groundwater sources. Sometimes considered to include flows from regulated lakes or reservoirs. Fluctuates much less than storm runoff.

cfs -- Abbreviation for cubic feet per second. A unit of water flow. Sometimes called "second-feet".

cfs day -- A unit of water volume. One cfs flowing for 12 days, or 12 cfs flowing for 1 day, make 12 cfs days. Also called "second-foot days".

closed-end level terrace -- A level terrace having ends of the ridge blocked to prevent loss of stored runoff. Built on soils with high infiltration rates and does not require terrace outlet. Used for water conservation and erosion control.

concordant flow method -- A simple form of regional analysis, in which flood peaks are related only to size of drainage area. See 2.7 Concordant flow.

consumptive use -- A term used mainly by irrigation engineers to mean the amount of water used in crop growth plus evaporation from the soil. See evapotranspiration.

contour farming -- The practice of farming on the contour, or by following terrace grades, for soil and water conservation.

contour furrow -- Furrow plowed during contour farming; or a separate furrow placed on the contour, generally on pasture or range land, for soil and water conservation.

cover -- The vegetation, or vegetational debris such as mulch, that exists on the soil surface. In some classification schemes, such as table 3.9-1, fallow or bare soil is taken as the minimum cover class.

cross section (stream or valley) -- The shape of a channel, stream, or valley, viewed across the axis. In watershed investigations it is determined by a line approximately perpendicular to the main path of water flow, along which measurements of distance and elevation are taken to define the cross-sectional area.

cross-slope farming -- Farming so that plowing, planting, tillage, etc., are done across the general slope of the land, but not on the contour as in contour farming.

csm -- Abbreviation for cubic feet per second per square mile of drainage area. A flow of 23 csm from a drainage area of 80 square miles is equal to 23 x 80 or 1840 cfs.

damage reach -- A length of floodplain or valley selected for damage evaluation. See 3.6 Stream reaches and cross sections.

degree-day -- As used in snowmelt studies, a day with an average temperature one degree above 32° F. The average is usually obtained by averaging the maximum and minimum for the day. A day with an average of 40° F. gives 8 degree-days.

depth-area curve -- A graph showing the change in average rainfall depth as size of area changes.

design storm -- A given rainfall amount, areal distribution, and time distribution, used to estimate runoff. The rainfall amount is either a given frequency (25-, 50-year, etc.) or a special large value. See 3.21 Design hydrographs.

direct runoff -- The water that enters the stream channels during a storm or soon after, forming a runoff hydrograph. May consist of rainfall on the stream surface, surface runoff, and seepage of infiltrated water (rapid subsurface flow).

double-mass curve -- A graph in which accumulated amounts of item X are plotted versus accumulated amounts of item Y, the amounts for given times being used. See NEH-4, 4.3.3 Double mass analysis, etc.

drainage area -- The area draining into a stream at a given point. The area may be of different sizes for surface runoff, subsurface flow, and base flow, but generally the surface runoff area is used as the drainage area. See watershed.

drop inlet -- An L-shaped conduit placed in an earth-fill dam, used to drop water from one level to another for gradient control and channel stabilization.

dry dam -- A floodwater retarding structure with no permanent storage. The principal spillway is placed at the bottom of the reservoir so that all or most of the reservoir area can be farmed or grazed between flood periods.

effective duration -- The time in a storm during which the water supply for direct runoff is produced. Also used to mean the duration of excess rainfall.

effective rainfall -- Another term for direct runoff. Usually not the same quantity on upland streams as on downstream rivers because of variability of seepage flows.

emergency spillway -- A rock or vegetated earth waterway around a dam, built with its crest above the normally used principal spillway. Used to assist the principal spillway in conveying extreme amounts of runoff safely past the dam.

ET -- Abbrevation for evapotranspiration.

evaluation series -- A list of floods or storms that produced floods during a representative period, and used in water project evaluation to obtain estimates of flood damages.

evapotranspiration -- Plant transpiration plus evaporation from the soil. Difficult to determine separately, therefore used as a unit for study. See consumptive use.

excessive precipitation -- Standard USWB term for "Rainfall in which the rate of fall is greater than certain adopted limits, chosen with regard to the normal precipitation (excluding snow) of a given place or area". Not the same as excess rainfall.

excess rainfall -- Direct runoff at the place where it originates.

fallow -- Cropland kept free of vegetation during the growing season. May be a normal part of the cropping system for weed control, water conservation, soil conditioning, etc.

f_c -- Symbol for the low, almost uniform, infiltration rate obtained after prolonged wetting of the soil.

field-size watershed -- A small watershed, usually between 1 and 20 acres in size, with its shape defined by the topography, with usually only one cover or land use at a time, and farmed as the farmer would a field using animal- or tractor-drawn equipment. Used in hydrology to distinguish data from that of small plots, where planting, cultivation, etc., must be done by hand.

flood -- In common usage, an event where a stream overflows its normal banks. In frequency analysis it means an annual flood that may not overflow the banks.

flood routing -- Determining the changes in a flood wave as it moves downstream through a valley or through a reservoir (then sometimes called reservoir routing). Graphic or numerical methods are used. The process may be reversed to route a flood wave upstream and find its original appearance.

flood pool -- Floodwater storage in a reservoir. In a floodwater retarding reservoir, the temporary storage between the crests of the principal and emergency spillways.

floodwater retarding structure -- A dam, usually with an earth fill, having a flood pool where incoming flood water is temporarily stored and slowly released downstream through a principal spillway. The reservoir contains a sediment pool and sometimes storage for irrigation or other purposes.

flood wave -- The rise and fall in streamflow during and after a storm.

frequency -- In common usage, "how often something happens". In hydrology, "how often something this large or larger will occur". Thus, a 50-year frequency flood will be equaled or exceeded in size, on the average, only once in 50 years. Similarly in other hydrologic usage. However, in drought or deficiency studies frequency is "how often you'll get this much or less", on the average, and usually in terms of years. See percent chance. See 3.18 frequency methods.

frequency line -- The line on probability paper that represents a series of events and their frequencies.

frequency series -- A sequence or array of actual events (floods, etc.) suitable for use in frequency analysis; or, a sequence or array of hypothetical events obtained from a frequency analysis.

graded terrace -- Terrace with its channel having a relatively flat slope toward (usually) one end. Flow drains into a terrace outlet. Most important use is erosion control.

ground water -- The water in the saturated zone beneath the water table. A source of base flow in streams.

Hazen equation -- $F_a = (2n - 1) /2y$. Used to obtain plotting positions for plotting flood values on log-normal paper. See 3.18 Frequency methods.

Hazen method -- As considered in the Hydrology Guide, it consists of using the Hazen equation and log-normal paper (or Hazen paper) to obtain frequencies. More generally, it consists also of skewness computations described by Allen Hazen in his book, "Flood Flows", published in 1930 by John Wiley and Sons, Inc., New York, N. Y.

historical series -- A list of all actual storms (or floods) that caused flood damage in a watershed, in a given period of years, with the date of each storm or flood being known.

hydrograph -- A graph showing changes (especially the rise and fall) in water discharge of a stream during a period of time. See 3.18 Hydrographs for special types, use, etc.

hydrologic soil-cover complex -- A combination of a hydrologic soil group and a type of cover.

hydrologic soil group -- A group of soils having the same runoff potential under similar storm and cover conditions.

hydrology -- The science that deals with the occurrence and behavior of water in the atmosphere, on the ground, and underground. Rainfall intensities, rainfall interception by trees, effects of crop rotations on runoff, floods, droughts, the flow of springs and wells, are some of the topics studied by a hydrologist.

initial abstraction (I_a) -- When considering surface runoff, I_a is all the rainfall before runoff begins. When considering direct runoff, I_a consists of interception, evaporation, and the soil-water storage that must be exhausted before direct runoff may begin. Sometimes called "initial loss", about which see loss.

infiltration -- Rainfall minus interception, evaporation, and surface runoff. The part of rainfall that enters the soil.

interception -- Precipitation retained on plant or plant residue surfaces and finally absorbed, evaporated, or sublimated. That which flows down the plant to the ground is called "stemflow" and not counted as true interception.

irrigation pool -- Reservoir storage used to store water for release as needed in irrigation.

isohyet -- A line on a map, connecting points of equal rainfall amounts.

lag (or lag time) -- In general, the delay in time between a period of excess rainfall and the appearance of resulting runoff at a downstream point. See 3.15 Time of concentration and lag.

land treatment measure -- A tillage practice, a pattern of tillage or land use, or any land improvement, with a substantial effect of reducing runoff and sediment production or of improving use of drainage and irrigation facilities. Examples are contouring, improved crop rotations, controlled grazing, land leveling, field drainage. In hydrologic computations, nonbeneficial measures (such as straight-row, poor-rotation corn) are included for convenience in evaluation. See table 3.9-1. In general conservation work "land treatment measure" has a broader meaning that includes measures to improve the soil, control sheet erosion, increase soil fertility.

land use -- A land classification. Cover, such as row crops or pasture, indicates a kind of land use. Roads may also be classified as a separate land use. For a classification scheme, see table 3.9-1.

log paper -- Short for "full-logarithmic graph paper", which is a graph paper (available commercially) that has logarithmic scales on both horizontal and vertical axes. Sometimes called "log-log paper". The scales may be any number of cycles, but usually in combinations like 1x1, 2x2, 3x3, 3x5, 4x7, etc.

log-normal -- Short for "logarithmic-normal probability distribution".

log-normal paper -- Graph paper used in estimating frequencies of floods, etc. Has a logarithmic scale for the flood (or other) amounts, and a cumulative distribution scale (also called frequency or percent chance scale) for the probability plotting positions.

loss -- In hydrology, a loss for one purpose is usually a gain for another, so that the net effect may be more important than the loss. At various times, evapotranspiration, initial abstraction, infiltration, surface storage, direct runoff, seepage, etc. have been called losses according to the aims of a water user. See water loss.

Manning's n -- A coefficient of roughness, used in a formula for estimating the capacity of a channel to convey water. Generally, "n" values are determined by inspection of the channel. See 3.14 Stage-discharge relations.

mean daily -- The average or mean discharge of a stream for one day. Usually given in cfs.

NEH-4 -- National Engineering Handbook, Section 4, Hydrology.

NEH-5 -- National Engineering Handbook, Section 5, Hydraulics.

normal -- As used in reference to hydrological periods, data, etc., means the 30-year period (or an arithmetical average for that period) defined by the World Meteorological Organization as presently 1921-50 inclusive, and up-dated every 10 years so that in 1961 the normal period is 1931-60, and so on. See "Minutes of the 56-1 meeting, Subcommittee on Hydrology, Interagency Committee on Water Resources, February 7, 1956".

open-end level terrace -- A level terrace with one or both ends open so that runoff may flow out. Terrace outlets are required. This type of terrace may eventually become a graded terrace.

partial duration series -- A frequency series in which an arbitrary base is selected, usually the smallest annual flood, and all values in the record equal to or greater than the base are used to prepare a frequency line.

percent chance -- A name often given to the probability scale on log-normal paper. A 2-percent chance flood is a 50-year frequency flood (see frequency) since

$$\frac{100}{\text{percent chance}} = \text{frequency in years}$$

It is also sometimes stated: In any given year, you have a 2-percent chance of having a 50-year frequency (or larger) flood; and so on.

plotting position -- Computed by an equation and used to locate given data on probability paper. See 3.18 Frequency methods.

point rainfall -- Rainfall at a single rain gage.

principal spillway -- A concrete or metal pipe or conduit used with a drop inlet dam or floodwater retarding structure. It conveys, in a safe and nonerosive manner, all ordinary discharges coming into a reservoir and all of an extreme amount that does not pass through the emergency spillway.

probability paper -- Any graph paper prepared for a plotting of magnitudes of events versus their frequencies or probabilities. See log-normal paper.

reach -- A length of stream or valley, selected for convenience in a study. See damage reach, stream reach.

recession curve -- The receding portion of a hydrograph, occurring after excess rainfall has stopped.

recurrence interval -- The average number of years within which a given event will be equaled or exceeded. A 50-year frequency flood has a 50-year recurrence interval; and so on.

regional analysis -- Flood frequency lines for gaged watersheds in a similar area or region are used to develop a flood frequency line for an ungaged watershed in that region. Also used with other types of hydrologic data. Method is a simple (usually graphical and freehand) form of "regression analysis" used by statisticians.

reservoir routing -- Flood routing through a reservoir.

row crop -- A crop planted in straight or contour rows, and tilled for weed control. Examples: Corn (maize), soybeans, cotton, some types of sorghum.

s. d. -- Abbreviation for standard deviation.

second-foot -- See cfs.

sediment pool -- Reservoir storage provided for sediment, thus prolonging the usefulness of floodwater or irrigation pools.

semilog paper -- Short for "semilogarithmic graph paper", which is graph paper having an arithmetic scale along one axis and a logarithmic scale along the other. Either scale is used for the independent variable, as the data require. Commercially available paper has various divisions (5, 6, 7, 10 to the inch) for the arithmetic scale, and various cycles (1, 2, 3, 4, 5) for the logarithmic side.

setback time -- Equal to half the travel time. Used in construction of a hydrograph for a subdivided complex watershed. See 3.16 Hydrographs.

skew -- When data plot in a curve on log-normal paper, the curvature is skewness. See 3.18 Frequency methods.

small grains -- Wheat, oats, barley, flax, rice, and other close-drilled or broadcast grain crops.

soil-cover complex -- See hydrologic soil-cover complex.

soil-water-storage -- The amount of water the soils (including geologic formations) of a watershed will store at a given time. Amounts vary from watershed to watershed. The amount for a given watershed is continually varying as rainfall or ET takes place.

spillway -- See principal spillway and emergency spillway.

standard deviation -- Statisticians' name for an important measure of dispersion, abbreviated s.d. Data grouped closely about their mean have a small s.d.; grouped less closely, they have a larger s.d. See table 3.18-3 for calculation of s.d.

standard rain gage -- Also "standard gage". The USWB nonrecording rain gage, having an opening 8 inches in diameter, and a holding capacity of 24 inches of rainfall. The gage is usually examined once daily at a regular time, and the catch (if any) measured by depth in inches and hundredths of an inch.

storage-indication method -- Name often given to a flood-routing method also often called the Puls method (after Louis G. Puls) though it is actually a variation of the method devised by Puls.

straight-row farming -- Operating farm equipment in straight rows, without regard to the lay of the land.

stream reach -- A length of stream channel selected for use in hydraulic or other computations.

strip cropping -- Farming with fields in relatively long strips instead of in wide rectangles. The strips may be on the contour, for water erosion control, or in straight lines, for wind erosion control. Systematic arrangement of strips, usually a good rotation with erosion-resisting crops in alternate strips, is generally followed.

structural measure -- For flood prevention work, any form of earthwork (dam, ditch, levee, etc.) or installation of concrete, masonry, metal or other material (drop spillway, jetties, riprap, etc.); or installation for forest fire protection (firetowers, roads, firebreaks); or, in some cases, a special planting for nonfarm purposes (stabilization of critical sediment-producing area, etc.)

subsurface runoff -- Water that infiltrates the soil and reappears as seepage or spring flow, and forms part of the flood hydrograph for that storm. Difficult to determine in practice and seldom worked with separately. See direct runoff.

subwatershed -- A watershed that is part of a larger watershed. It is worked on separately when necessary in order to improve computational accuracy for results on a whole watershed basis, or to get results for that area only.

surface runoff -- Total rainfall minus interception, evaporation, infiltration, and surface storage, and which moves across the ground surface to a stream or depression.

surface storage -- Natural or man-made roughness of a land surface, which stores some or all of the surface runoff of a storm. Natural depressions, contour furrows, and terraces are usually considered as producing surface storage, but stock ponds, reservoirs, stream channel storage, etc. are generally excluded.

synthetic series -- A storm or flood series obtained by taking selected values from a frequency line based on historical data.

terrace -- An earth embankment or a broad channel and embankment built on a slight grade or on the contour, to control runoff and minimize erosion, or for water conservation. See closed-end level t., graded t., open-end level t., and t. outlet.

terrace outlet -- A grassed waterway designed to convey runoff from graded or open-end level terraces, at nonerosive velocities, downhill to a stream or waterway.

time of concentration (T_c) -- The time it takes water from the most distant point (hydraulically) to reach a watershed outlet. T_c varies, but often used as constant. See 3.15 Time of concentration and lag.

transmission loss -- A reduction in volume of flow in a stream, canal, or other waterway, due to infiltration or seepage into the channel bed and banks. Evaporation is also a transmission loss, but it is ordinarily neglected under the assumption that it is small.

travel time -- The average time for water to flow through a reach or other stream or valley length that is less than the total length. A travel time is part of a T_c but never the whole T_c.

unit hydrograph -- A discharge hydrograph coming from 1 inch of direct runoff distributed uniformly over the watershed, with the direct runoff generated at a uniform rate during the given storm duration. A watershed may have 1-hour, 2-hour, etc. unit hydrographs.

USGS -- United States Department of the Interior, Geological Survey.

USWB -- United States Department of Commerce, Weather Bureau.

water equivalent -- The depth of water, in inches, that results from melting a given depth of snow.

426963 O-57 -22

water loss -- Variable meaning, depending on personal interest of water user. Farmers and ranchers usually think of flood runoff as a water loss; many river engineers think of infiltration as a water loss. In Hydrology Guide, the meaning is apparent from the context. See loss.

watershed -- The area contributing direct runoff to a stream. Usually it is assumed that base flow in the stream also comes from the same area. However, the groundwater watershed may be larger or smaller.

watershed measures -- Any vegetative or structural means (including earthwork) of directly improving or conserving the soil and water resources of a watershed. See land treatment measure and structural measure.

water table -- The upper surface of ground water. Below the w.t. the soil is saturated.

water year -- The year taken as beginning October 1. Often used for convenience in streamflow work, since in many areas streamflow is at its lowest at that time. Used by USGS in their WSP.

water yield -- The actual streamflow, at a given place, from a watershed. This is natural annual runoff that may be affected by irrigation uses, reservoir losses, diversions into or out of the watershed, etc.

WSP -- Water-Supply Paper. An annual publication of the USGS, in which streamflow for the water year is given for all gaged streams in a subdivision of the United States or in Hawaii.

5.2 Conversions FACTORS

A selected number of conversions is given in table 5.2-1. The conversion constants are shown to four significant figures, but the answer, in practice, is generally rounded off to three significant figures. Some common conversions are often rounded off even farther. (Examples: 1 cfs day equals 2 AF; 1 inch per hour equals 1 cfs per acre). When this is done, a footnote or some other notice should be given to alert the reader.

Table 5.2-1 Conversions FACTORS

THIS:	TIMES THIS:	GIVES YOU THIS:
cfs days	1.983	AF
cfs days	0.03719	inches depth on 1 square mile
cfs days per square mile	0.03719	inches depth
cfs hours	0.08264	AF
cfs hours per square mile	0.001550	inches depth
cfs	1.983	AF per day
cfs	724.0	AF per year (365 days)
cfs	448.8	U. S. gallons per minute
cfs	0.6463	million U. S. gallons per day
csm	0.03719	inches depth per day
csm	13.57	inches depth per year (365 days)
inches per hour	645.3	csm
inches per hour	1.008	cfs per acre
inches depth	53.33	AF per square mile
inches depth on 1 sq. mi.	53.33	AF
AF	0.5042	cfs days
AF	12.10	cfs hours
AF	0.01875	inches depth on 1 square mile
AF	0.3258	million U. S. gallons
AF per day	0.5042	cfs
AF per square mile	0.01875	inches depth
U. S. gallons per minute	0.002228	cfs
million U.S. gallons per day	1.547	cfs
million U.S. gallons per day	3.069	AF
feet per second	0.6818	miles per hour
centimeters	0.3937	inches
hectares	2.471	acres
liters	0.2642	U.S. gallons
kilograms	2.205	pounds
cubic feet	7.480	U.S. gallons
imperial gallons	1.200	U.S. gallons

U. S. GOVERNMENT PRINTING OFFICE : 1957 O - 426963