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PLANNING MANUAL FOR SYSTEMATIC RIVER BASIN FLOODPLAIN STUDIES

by

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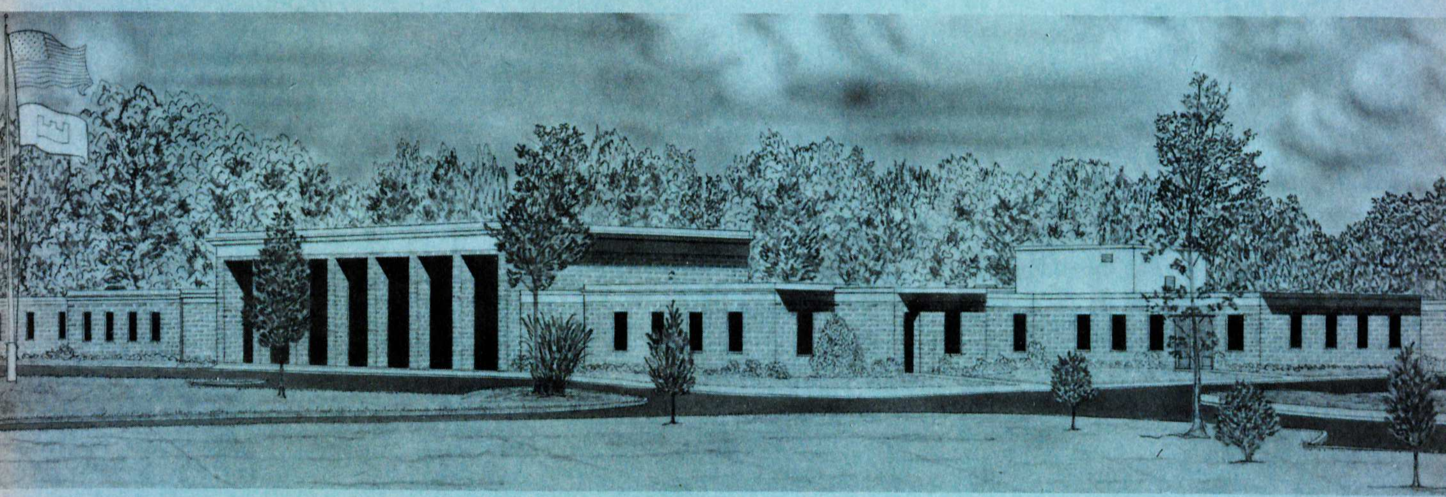
Final Report



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Floodplain management has become a topic of high national priority in recent years. Computer simulation programs, such as HEC-1 and HEC-2, have greatly increased the efficiency with which calculations can be performed once input data are properly formatted. The objective of this study was to develop systematic procedures for the conduct of river basin floodplain studies. Guidelines are presented for standardizing watershed subdivision and the collecting, processing, and storing of stream cross section, elevation, soils, land use, and

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other information in computer data bases. Methodologies for accessing data bases and generating input decks for computer programs HEC-1 and HEC-2 are presented. Also discussed are procedures for interfacing these two programs such that manual input requirements are minimized. Other topics addressed include the coding of bridges, the effect of reach length on computed water surface elevation, the areal and temporal distributions of rainfall, model calibration, the automated mapping of floods, and flood damage estimation. Three software routines and some pertinent reference materials are included in four appendixes.

Preface

This report presents procedures for the systematic planning of large river basin floodplain studies. Work was conducted under Contract No. CW39-77-M-2881 for the U. S. Army Engineer Waterways Experiment Station (WES), Vicksburg, Miss., during the period 20 April 1977 through 30 September 1977. The effort constitutes part of the Wolf River Simulation Study being conducted for the U. S. Army Engineer District, Memphis, Memphis, Tenn., by the WES under Appropriation No. 96X4902.

The study was made and the report prepared by Dr. Gert Aron, Department of Civil Engineering, Pennsylvania State University, University Park, Penn. The contract was monitored technically by Dr. L. E. Link, Chief, Environmental Research Branch (ERB), Environmental Systems Division (ESD), Mobility and Environmental Systems Laboratory (MESL), of WES and Mr. J. G. Collins, ERB, Project Manager, under the general supervision of Messrs. B. O. Benn, Chief, ESD, and W. G. Shockley, Chief, MESL.

The ESD is now part of the Environmental Laboratory of which Dr. John Harrison is Chief.

The Director of WES during the period of this study and the preparation of this report was COL J. L. Cannon. Technical Director was Mr. F. R. Brown.

Contents

Conversion Factors, U. S. Customary to Metric (SI) Units of Measurement

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
acres	4046.856	square metres
cubic feet per second	0.02831685	cubic metres per second
feet	0.3048	metres
inches	25.4	millimetres
miles (U. S. statute)	1.609344	kilometres
square miles	2.589988	square kilometres

PLANNING MANUAL FOR SYSTEMATIC RIVER BASIN FLOODPLAIN STUDIES

Introduction

1. Floodplain management has become a topic of high national priority in the last decade, and billions of dollars will probably be spent in the near future on floodplain delineations and flood level computations.

2. Hydrologic and hydraulic process simulation computer programs such as HEC-1 and HEC-2 have greatly increased the efficiency with which calculations can be performed.* At present, though, most floodplain studies are being conducted in parts, by first performing the hydrologic analysis, then following up with the hydraulic computations. Much time is spent, and possibly wasted, in collecting data from scattered sources, running the hydrologic analysis, then processing the output of this analysis into the appropriate input for the hydraulic computations. Surveying and plotting of stream cross sections and converting these to the proper input format for backwater programs is another highly tedious job subject to many possible human input errors or inconsistencies.

3. The first objective of this manual is to present guidelines for standardizing watershed subdivision and collecting and processing data into unambiguous formats that can be converted automatically into inputs for computer programs HEC-1 and HEC-2. The second objective is to maximize the ease with which these two programs can be interfaced so that a minimum of input data needs to be introduced manually once the main data tapes with information on basin topography, soils, land use, and stream cross sections have been compiled. A uniform treatment of bridge geometry will also be suggested, which, while not necessarily using the most appropriate hydraulic computational procedure in every

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* HEC = Hydrologic Engineering Center.

case, should provide water surface profiles commensurate in precision with presently used backwater computation procedures.

Organizational Step Sequence

4. A systematic hydrologic-hydraulic floodplain analysis should involve the following steps:

- *a.* Selection of Watersheds to be Studied
- *b.* Identification of Primary and Secondary Areas in the Watershed
- *c.* Planning Cross-Section Location and Length
- *d.* Subdivision of Watershed and Subarea Numbering
- *e.* Collection, Coding, and Digitization of Cross-Section and Bridge Data
- *f.* Collection of Soils, Topography, Land Use, and other Runoff-Determining Data
- *g.* Establishment of a Digital Data Bank
- *h.* Collection of Measured Rainfall-Runoff Data
- *i.* Model Calibration
- *j.* Choice of Event Storms
- *k.* Generation of Event Hydrographs
- *l.* Backwater Computations
- *m.* Areal Delineation of Floods
- *n.* Flood Damage Computations

Selection of watersheds to be studied

5. The selection of the watersheds for which floodplain studies should be made depends primarily on the urgency of the situation. Obviously, a densely developed watershed with a history of costly floods merits preference over a wilderness area in which flooding is an entirely acceptable event.

6. There is no lower limit for a watershed size, although it may be too cumbersome to bother about an elaborate systematic scheme for a watershed of one square mile or less. The upper limit of a watershed size to be handled in a single effort depends mostly on computer

storage capacity and available time to complete the project.

7. Whatever the size of the area to be modeled, it should comprise an entire watershed. No streams should cross the boundary except at the outlet section. For example, the method outlined should not be used to study river levels between points A and B in Figure 1 unless reliable inflow hydrographs for point A have been generated in a previous study.

Identification of primary and
secondary areas in the watershed

8. Given sufficient funds, the entire watershed can be considered

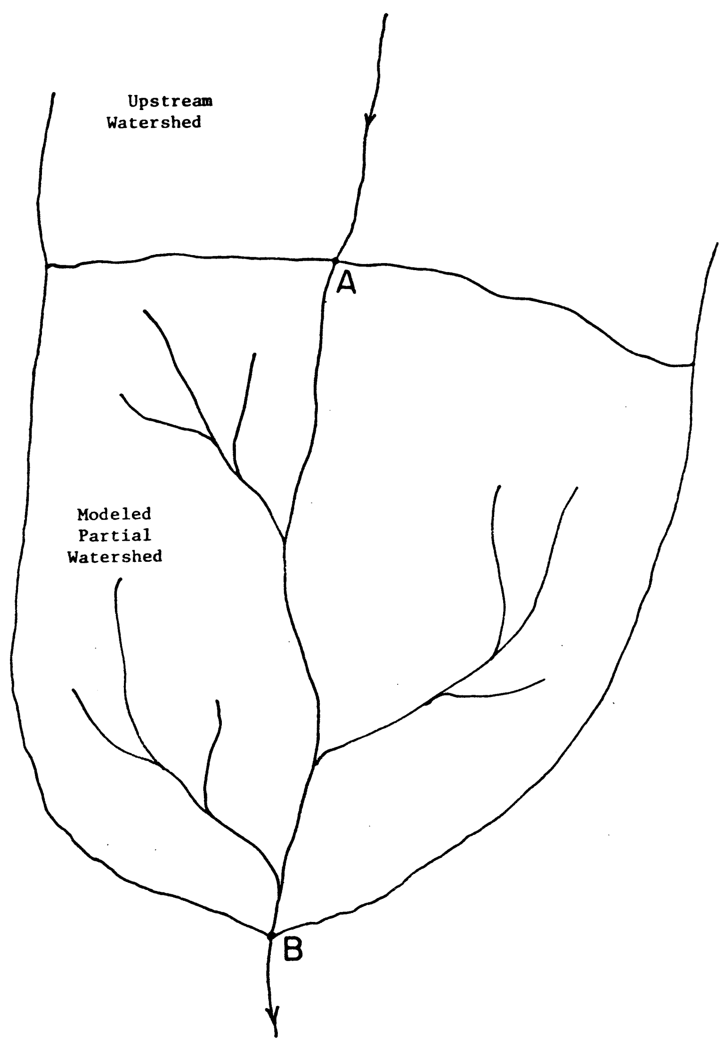

Figure 1. Partial watershed with separately determined inflow

of primary importance and designated for detailed data acquisition and modeling. Even areas which are complete widerness or swamps at present may at some future date become attractive for development.

9. In practice, it will usually be found that a detailed hydrologic-hydraulic modeling effort of an entire watershed of 1000 square miles* or so would become prohibitively expensive. Thus, some areas may be considered of secondary importance and given a more superficial modeling treatment. Subareas may be made larger than in the detailed study area and cross sections may be spaced farther apart. It is important, however, to leave provisions for later conversion of these secondary areas to primary interest areas without causing a major disruption in the model organization. This involves a flexible coding of the subbasins such that future addition of subbasins will not require the sequential recoding of the entire system.

Planning cross-section location and length

10. Once it is determined which portions of the watershed are to be classified as primary and secondary interest areas, the locations and lengths of the cross sections should be planned. Slight changes in cross-section location, or the addition of a few cross sections, can always be made at the time of the survey.

11. First, all streams and tributaries of hydrologic importance must be identified. These are the streams for which event floods of interest and associated flood levels are to be computed. This classification may become a rather subjective task, strongly dependent on the funds available for the flood study. Referring to Figure 2, assume that streams a, b, c, and e have been designated as hydrologically important.

12. At all junctions of important streams two sets of cross-section data must be obtained: one section should be located about one stream width below the junction, another section between one and two

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* A table of factors for converting U. S. customary units of measurement to metric (SI) can be found on page 3.

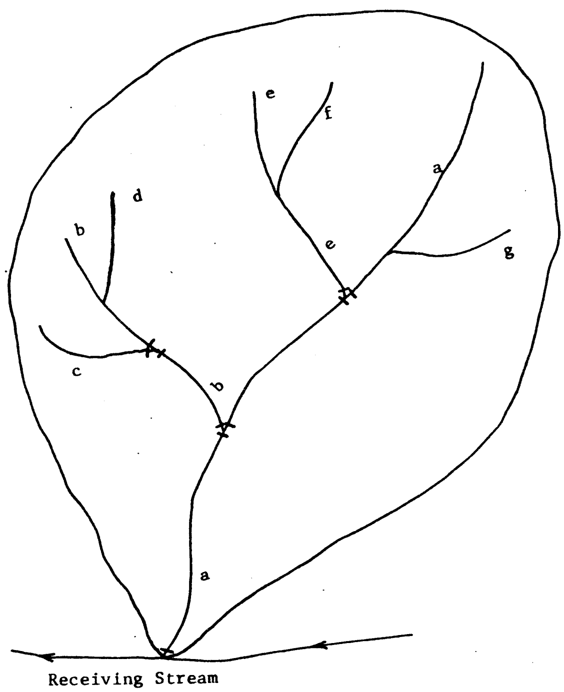

Figure 2. Cross-section location on a
small drainage basin

stream widths above the junction. The upstream cross section must be split at the divide, as shown in Figure 2 at the junctions of streams a and b, b and c, and a and e. This separation of data into two independent upstream sections is absolutely necessary for the correct execution of HEC-2 backwater computations. The cross section should extend through the floodplain on both sides of the stream along a course normal to the flow lines (i.e., approximately perpendicular to the topographic contour lines), except at a bridge crossing where the cross section may follow the road bed.

13. Cross sections are required at all bridges. The number and spacing of these cross sections will be discussed in more detail in the bridge data instructions, step e. Cross sections should also be provided at all control locations, like weirs or other abrupt flow constrictions, at which critical flow is expected to occur for some or all of the discharges of interest. The same holds for locations of stream gages which might be used for the calibration of backwater computations, as well as those points at which the channel and/or overbank

geometry changes appreciably. Control sections are often not apparent on the topographic maps; therefore, it would be extremely useful if a hydrologist or hydraulic engineer accompanies the survey crew, possibly as a crew member, or at least performs a field reconaissance to locate all cross sections.

14. Observance of the above set of guidelines for cross-section location or a similar set provided by HEC[1,2] will usually result in rather close spacing of cross sections in developed areas and large spacings in less developed and hydrologically less important areas. This brings up the problem of <u>maximum allowable reach</u> length, an economically important subject, especially in relatively inaccessible stream reaches, where cross-section surveys are expensive.

15. Barr Engineering Co.[3] cites a rule originating in Minnesota that makes the maximum reach length L a function of stream slope S as follows:

Slope, ft/mile	Maximum Reach Length, ft
<2	2,640
2-3	1,800
>3	1,200

This rule, when applied to some stream data, was found overly conservative, especially in wide and mature rivers like the Mississippi where reach lengths of 2 miles or longer might be quite adequate.

16. Tavener[4] performed an analytical study of backwater computational errors in natural stream channels, which resulted in an equation for maximum reach length as a function of the stream gradient, the Froude Number, wetted cross-section area, and wetted perimeter, as well as the top width of the stream at any section. Aron[5] modified the equation and converted it to a nomograph solution which is less tedious to derive. Both the Tavener equation and the Aron nomograph, however, require dimensions which are not known until after flood profiles have been determined; therefore, both of these methods are suited chiefly to check whether a chosen cross-section spacing was adequate. Appendix D presents formulas for backwater computations.

17. As part of the research leading to this manual, backwater computations were run on a hypothetical channel of parabolic cross section and randomly varying width and slope. Channel width was varied between 20 and 100 ft, depth between 1.5 and 4 ft, slope between 0.0006 and 0.02, and discharge between 100 and 1000 cfs. With any combination of these variables, reach length was varied between 200 and 4800 ft. Using 200-ft reach length results as a standard for comparison, water profile errors occurred sporadically, and no consistent formula for maximum reach length could be found. This was because errors due to expansion and contraction losses interfered randomly with errors due to friction losses. Examining only the largest errors in computed water surface elevation, these seemed to be related to the irregularities of the channel slope. The equation developed from the observations is

$$L_{max} = \frac{5e}{\overline{S} + \Delta S} \tag{1}$$

where

L_{max} = the maximum advisable reach length, ft or m

e = the largest acceptable error in computed water surface elevation, ft or m

$\overline{S}$ = the average channel slope, ft/ft

ΔS = the range of channel slopes, ft/ft

in the vicinity of the intended reach, as determined from a detailed topographic map or a preliminary survey of low water surface profiles. The streambed itself may have local scour holes and irregularities which have negligible effect on the computed flood profiles.

18. For a sample application, take a stream with an average channel slope of 2 ft/mile over the lower 5 miles. According to the Barr rule, cross sections should be taken at about one half mile intervals. If the maximum allowable error was 0.4 ft and the slope varied between 1 and 3 ft/mile, Equation 1 would give the same maximum reach length of one half mile. If, on the other hand, a 1.0-ft error was acceptable and the slope varied only between 1.5 and 2.5 ft/mile, reach lengths as large as 1.7 miles would be acceptable. The use of Equation 1 must,

however, be subject to the previously stated rules for cross-section location. It should also be kept in mind that the computations leading to Equation 1 were made exclusively for subcritical flow conditions.

19. Under certain conditions, reach lengths used may, at least partially, depend on the friction slope average method selected; this topic is discussed herein. A number of equations have been proposed for calculating friction losses, four of which are options in the HEC-2 computer program. These four equations are listed below:

Average Conveyance Equation

$$HL = L\left(\frac{Q_1 + Q_2}{K_1 + K_2}\right)^2 \tag{2a}$$

Average Friction Slope Equation

$$HL = L\,\frac{S_1 + S_2}{2} \tag{2b}$$

Geometric Mean Friction Slope Equation

$$HL = L\,(S_1 S_2)^{0.5} \tag{2c}$$

Harmonic Mean Friction Slope Equation

$$HL = L\,\frac{2\,S_1 S_2}{S_1 + S_2} \tag{2d}$$

where

HL = friction loss for reach

L = discharge-weighted reach length

Q_1 = total discharge at upstream end of reach

Q_2 = total discharge at downstream end of reach

K_1 = total conveyance at upstream end of reach

K_2 = total conveyance at downstream end of reach

S_1 = friction slope at upstream end of reach

S_2 = friction slope at downstream end of reach

20. Equation 2a is used in all HEC-2 source decks dated August 1971 that contain Modification 56. It is presently recommended by HEC for all flow conditions until research shows that another equation is clearly more applicable.

21. Equation 2b represents the arithmetic average of the friction slopes S_1 and S_2 and has been recommended by Tavener[4] as the preferable equation for M-1 curves. Reed and Wolfkill[6] confirmed Tavener's recommendations.

22. Equation 2c represents the geometric average of the friction slopes S_1 and S_2 and is presently used in the U. S. Geological Survey (USGS) computer program for calculating profiles.

23. Equation 2d is equivalent to averaging the inverse values of slopes S_1 and S_2 and has been recommended by Tavener for M-2 profiles.

24. Modification 50 of the computer program HEC-2, dated November 1976, allows the choice among Equations 2a to 2d by specifying a variable IHLEQ in field 1 of input card J6, as listed below:

Value of IHLEQ	Friction Loss Equation
0	Equation 2a is used
1	Program selects equation based on flow conditions
2	Equation 2b is used
3	Equation 2c is used
4	Equation 2d is used

If IHLEQ is set equal to 1, the program selects a friction loss equation for a reach in accordance with criteria below:

Profile Type	Is Friction Slope at Current Cross Section Greater than Friction Slope at Preceding Cross Section?	Equation Used
Subcritical (M1,S1)	Yes	2b
Subcritical (M2)	No	2d
Supercritical (S2)	Yes	2b
Supercritical (M3,S3)	No	2c

25. At present, HEC's stand on friction slope averaging is that the choice of a particular equation makes little difference in the computed water surface profiles, as quoted below.

> Experience to date indicates that application of criteria in Table 1 produces water surface profiles that only rarely differ by more than 0.2 feet from profiles determined with Equation (2a). In a few instances, application of criteria in Table 1 enabled determination of 'balanced' water surface elevations at cross sections for which Equation (2a) could not produce a solution to the energy equation. Any of the alternative friction loss equations will produce satisfactory estimates provided that reach lengths are not too long. The advantage that is sought in alternative friction loss formulations is to be able to maximize reach lengths without sacrificing profile accuracy.

Two specific situations, however, are illustrated in Figures 3 and 4 in which the improper choice of a slope-averaging equation can lead to major errors in computed water surface elevation.

26. For the arithmetic slope-averaging method (Equation 2b), the most critical condition is illustrated in Figure 3; here the downstream slope is abnormally high, due perhaps to a combination of a narrow floodway and high channel roughness. If in this typical M2 profile a few short reaches are used just upstream of the construction, the steep energy slope is gradually reduced to a value which approximates normal flow conditions in the channel. With a long reach, on the other hand, the backwater iterations may converge to a very large upstream depth because the minimum average friction slope used for the reach cannot be smaller than one half of the downstream friction slope, which can be as high as 0.02 at or near critical flow conditions in shallow streams.

27. Equation 2a, presently preferred by HEC, produces adequately accurate flood profiles in the M2 profile situation described above even with reaches of several hundred feet in length, but can cause problems in the M1 profile situation shown in Figure 4, in which the flood water, backed up by a bridge or other obstruction, occupies a wide and deep downstream section resulting in a very large conveyance.

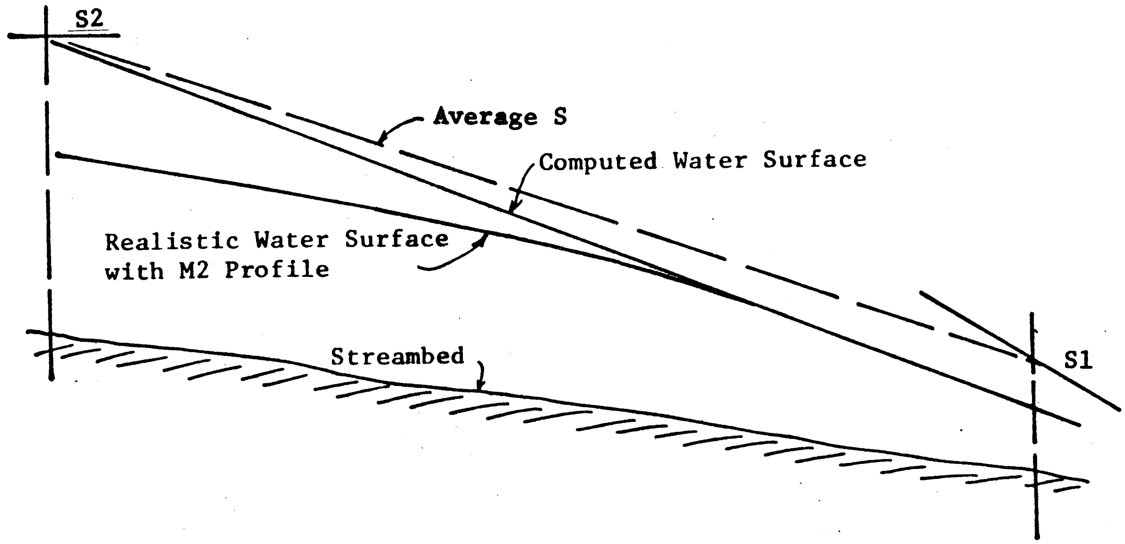

Figure 3. Water profile error due to use of incorrect slope-averaging equation used on M2 profile

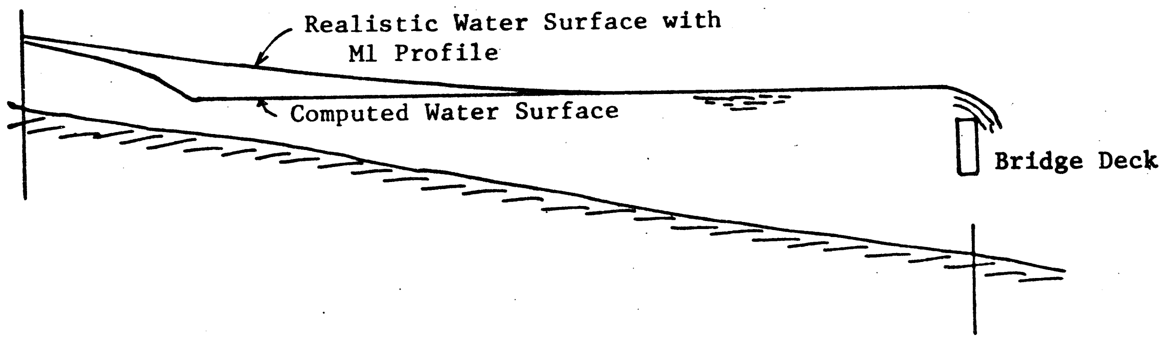

Figure 4. Water profile error due to use of incorrect slope-averaging equation used on M1 profile

This leads to a condition under which, no matter how narrow the upstream section is, the average conveyance cannot be lower than one half the downstream conveyance. The consequence of such a situation is an inescapably low friction slope which forces the HEC-2 program into a critical depth default if the reach length is too long.

28. Because of the observations made in paragraphs 26 and 27, it is recommended that input card J6 be used with variable IHLEQ set equal to 1.

Subdivision of water-
shed and subarea numbering

29. The segmentation or subdivision of a watershed is a common feature of all major storm water runoff models. Ideally, it would be

convenient to have all watershed subareas of equal size, but, as shown in Figure 5, the location and density of subdivision boundaries are primarily dependent on the density of streams important enough for flood level computations.

30. In the watershed subdivision process, a hierarchal system may be used. The upper hierarchy is obviously the watershed to be modeled.

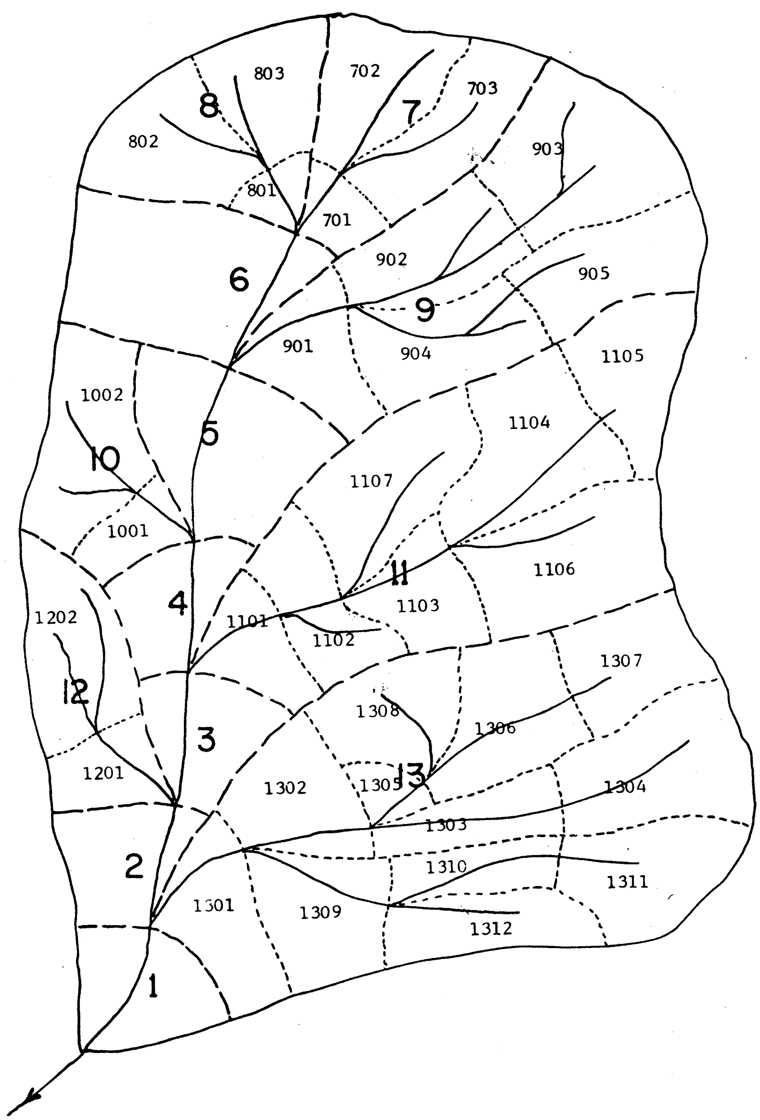

Figure 5. Two-level watershed subdivision into subbasins and minibasins, using numbering sequence alternative No. 1

The watershed may be divided into subbasins and these further into minibasins. In small to medium-size watersheds in which the number of subdivisions is sufficiently small to be processed in one single HEC-1 run, the watershed may be divided directly into minibasins in what will be defined hereafter as a one-level subdivision process. The convenience of dividing a larger watershed in a two-level process into subbasins and minibasins will be discussed below in connection with the subarea number coding. The term subarea will be used in describing properties or processes common to subbasins and minibasins.

31. The first rule in subdividing a watershed must be that no two streams of hydrologic importance can be located in the same minibasin. A logical procedure in watershed subdivision is to follow the main stream of the watershed, starting at the outlet section and proceeding upstream, outlining the watersheds of all the tributaries of hydrologic importance as defined previously. This involves drawing boundary lines along the divides between adjacent streams of hydrologic importance. If a subarea contains a reservoir large enough to be included in the HEC-1 modeling operation, its drainage area should also be separated as a subarea. Typical minibasin boundary lines are shown in Figure 6.

32. As a general rule, a minibasin should not be larger than one square mile in densely developed areas, but in wilderness areas which are highly uniform or for which little hydrologic information is available, subareas can be as large as 10 square miles. If the Soil Conservation Service (SCS) unit hydrograph generating procedure is to be used, however, minibasins should not be excessively large, perhaps less than five square miles, because the SCS has tested their procedure only on areas smaller than 2000 acres. It is also advisable to subdivide areas of highly nonsymetric shape for which travel times of different portions would otherwise vary greatly. When a subbasin must be subdivided, divisions should be made at surveyed cross-section locations if possible. A bridge is a good location for a subdivision line, for example.

33. In labeling the subareas and cross sections, a numerical rather than alphabetic system should be used, because computer programs

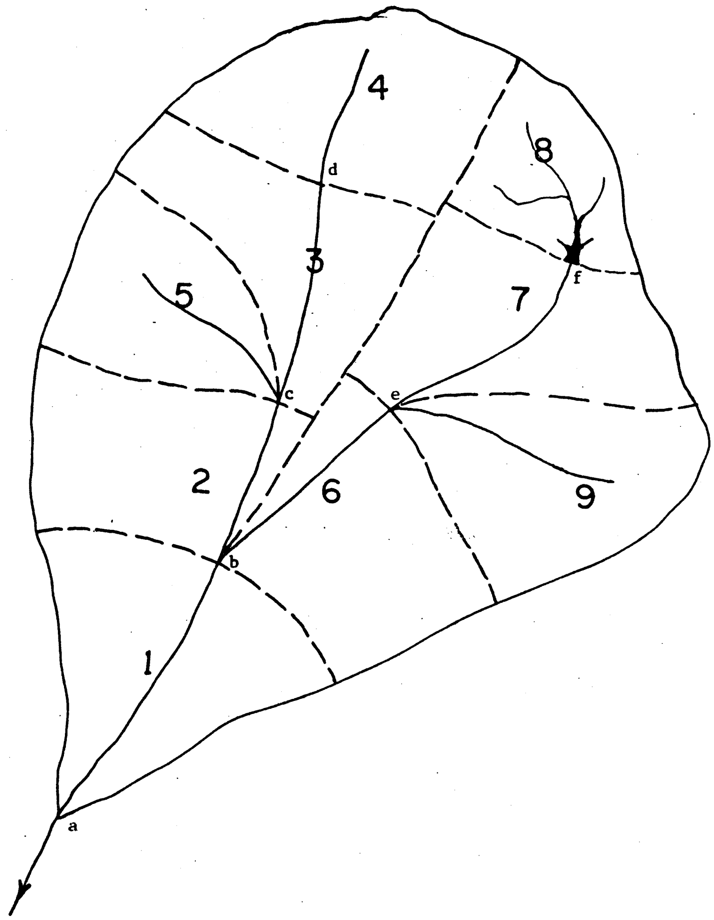

Figure 6. One-level watershed subdivision into minibasins

can sort numerical information with much more ease. Subareas may be numbered in an upstream or downstream sequence. The upstream numbering sequence has the advantage that the downstream limit of a drainage basin is usually the point of outflow into the receiving water body and thus is well defined. At the upstream end, on the other hand, a stream usually fades into a creek and finally into a gully, and the upstream extremity of a stream is thus often difficult to define. During the modeling effort the hydrologist may decide to add or delete one or more upstream sections or basins; if so, fewer corrections in the numbering would be necessary if an upstream sequence is used.

34. The numbering sequence must follow a logic which can be understood by a computer program. In one-level subdivision processes a

single sequence of numbers may be used as illustrated in Figure 6. The largest of the streams is chosen as the main stem, and the minibasin numbering follows the sequence 1 to 4 to the upper minibasin of this stream. Subsequently, the stream is scanned for tributaries, starting at the upper minibasin, namely 4, and moving downstream. At point c the first tributary is found, consisting of only one minibasin, which is labeled 5. Continuing farther down to point b, a tributary flowing through several minibasins is found and the numbering process is repeated, numbering the minibasins 6 to 8, then scanning downstream to pick up one more tributary at point e, which requires only one minibasin, numbered 9.

35. The above described numbering sequence is compatible with the computational sequence of HEC-2 and the input requirements of HYDPAR2, a program described in Appendix B, which converts hydrologic, topographic, and land use data into an HEC-1 input deck.

36. In modeling larger watersheds, it may be advisable to divide the watershed into subbasins and minibasins. The reasons for using a two-level subdivision may be the limit on the number of subareas a program can handle in one run, or the loss in subdivision flexibility due to a large number of sequentially numbered minibasins. With reference to the latter, assume, for example, that the watershed shown in Figure 6 was divided into hundreds of minibasins and that the uppermost minibasin of the main stem was labeled No. 19. After numbering and recording all other subbasins sequentially it was found necessary to divide minibasin 19 into two parts which should logically be renumbered 19 and 20, respectively. This small change then starts a cascade of minibasin number adjustments which can become a rather tedious task and lead to confusion between old and new numbering sequences. Figures 5 and 7 illustrate cases of two-level subdivision and numbering by subbasins and minibasins.

37. The subbasin numbering may be made to follow the same logic as prescribed in Figure 6 for the one-level subdivision. This results in a numbering sequence as shown in Figure 5, following the main stream stem upward to the uppermost subbasin, then numbering all drainage areas

of the tributaries in downstream sequence. This numbering sequence for subbasins which are further subdivided looks awkward; therefore, an alternative sequence is suggested. In Figure 7, drainage areas of tributary streams and areas draining directly into the main stem receive equal subarea status and are numbered consecutively in upstream order. No tributary will occupy more than one subbasin, but any subbasin can be subdivided into minibasins.

38. The minibasin numbering follows the rules laid out in Figure 6

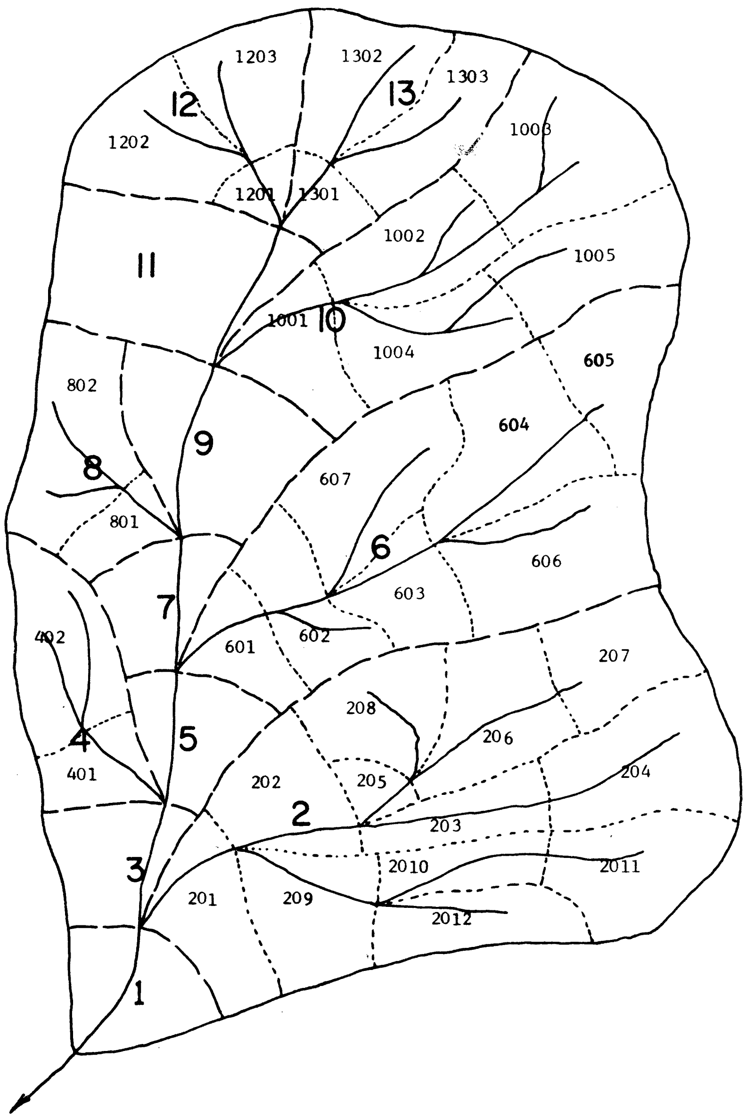

Figure 7. Two-level watershed subdivision into subbasins and minibasins, using numbering sequence alternative No. 2

for single-level subdivision. In this manner each subbasin can be processed individually or as part of a group by program HYDPAR2 into an input deck for the hydrologic program HEC-1. Four digits are reserved for minibasin numbers; the first two of which identify the subbasin, if any, the last two digits identify the minibasin within it.

39. Finally the cross sections within the minibasins must be numbered. This number should be the section number SECNO entered in field 1 of the X1 cards of the HEC-2 input deck. SECNO is restricted to six digits, the first of which may be needed for a minus sign. Therefore, the section numbers consist of the four digits identifying the minibasin, followed by one digit which identifies the cross sections within the minibasin. There will rarely be more than 10 cross sections in a minibasin, except where several bridges cross the stream in close proximity, perhaps requiring a total number of regular plus auxiliary sections larger than 10.

40. Figure 8 is an enlargement of the lower portion of subbasin 2 in Figure 7. It was found convenient to label any cross section immediately above a downstream minibasin boundary with the digit "0" and those immediately below the upstream boundary with the digit "9"; between these boundaries any digits between 1 and 8 may be assigned, which provides the flexibility of later insertion of additional cross sections. Neither the program HEC-2 nor XSEC-PROC,* which generates HEC-2 input decks from cross-sectional data, requires an unbroken sequence of cross section numbers; HYDPAR2, however, recognizes only a section number ending with 9 as legitimate point at which the flows from two upstream tributaries can be combined.

41. Auxiliary sections occur at bridges and other locations at which a surveyed section is repeated upstream or downstream. These auxiliary sections are labeled using the surveyed section number without the subbasin number, followed by a decimal point and decimals 1, 2,... in upstream sequence. As an example, the second auxiliary section at a Section No. 32184 could be numbered 184.2.

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* See Appendix A.

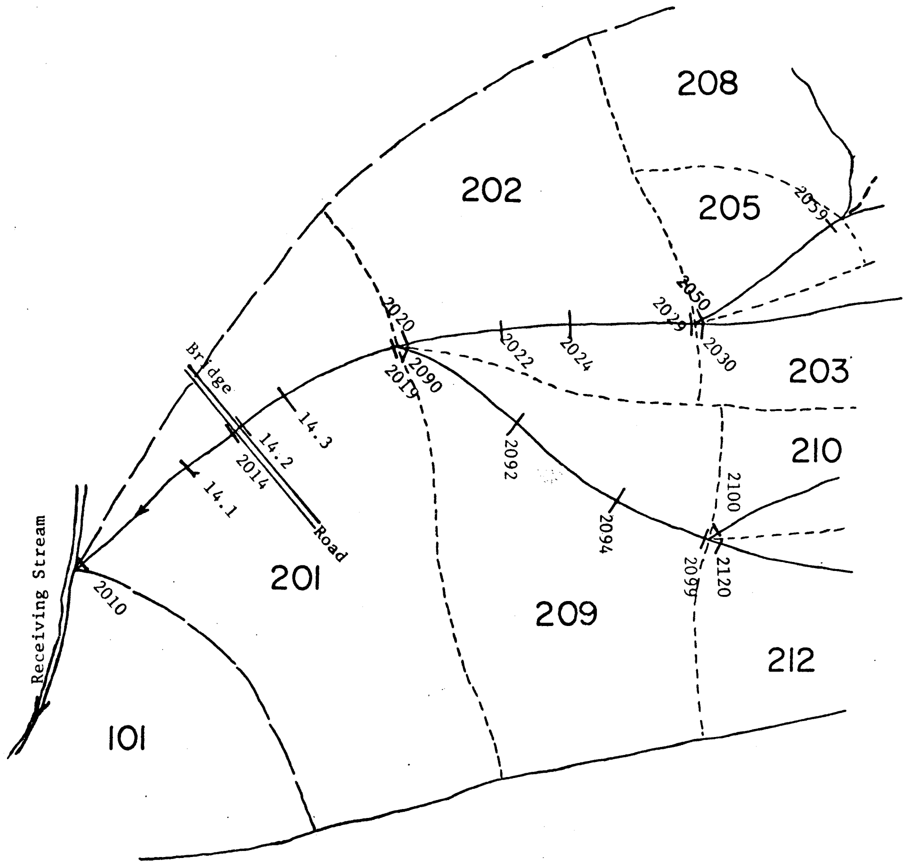

Figure 8. Cross section numbering within minibasins

Collection, coding, and digitization of cross-section and bridge data

42. One of the purposes of this manual is to present a procedure for the automatic conversion of cross-section survey data into HEC-2 input data cards. To accomplish this, a systematic coding not only of the survey data, but likewise of other information and instructions, was developed. Some of these items are supplemental physical measurements or estimates while others are indices of options available for converting the survey and hydraulic data into HEC-2 input cards.

43. The various physical measurements or quantities supplemental to standard cross-section survey data to be included on the cross section data file are as follows:

a. Location of cross section, generally measured as the distance along the stream channel from the stream's outflow point. This distance may be measured in feet, miles, or kilometres.

b. Angle by which a cross section deviates from the normal to the stream channel, expressed in degrees. If this angle is less than 20 deg, it may be ignored.

c. Distance by which the left and right overbank reach lengths between sections differ from the reach length measured along the channel. May be measured from maps and expressed in feet or metres.

d. Number and location of auxiliary cross sections created by transferring surveyed cross sections a stated distance upstream (+) or downstream (-).

e. Discharges as they vary with distance from the stream outlet. These estimates usually come from separate hydrologic analyses and are expressed in cubic feet or cubic metres per second. A set of 1 to 14 design discharges is specified ahead of the first cross section, and these discharges will remain constant until superceded by a new set of discharges entered.

f. Location and height of real or artificial levees. Whether levee instructions should be generated for HEC-2 depends on the ease with which the water can flow into and out of low-lying overbank areas. (See HEC-2 Users Manual for levee card instructions.)

g. Left and right main channel bank stations. These are the limits within which the streamflow is computed with the in-bank roughness factor.

h. Bridge width and height of railings. These data are frequently omitted in cross-section data. The height of the railings should be expressed as the solid height. For example, if the railings consist of three 4-in. horizontal pipes, the height of the railing should be given as 1.0 ft.

i. Stations along a bridge section within which weir flow is expected to occur when the water surface elevation exceeds that of the bridge deck. These are needed only when the Special Bridge routine is to be used. Sometimes, a bridge cross section may be a mile long. Obstructions might well prevent the flow from freely moving to a point one half mile from the stream and cascading across the road. The engineer must make a judgment decision regarding the total width of the weir flow. This width may be expressed as the first and last stations for which BT cards should be created, or simply as a width, in feet or metres, to be entered in field 4 of the SB card of HEC-2.

j. Manning's roughness factors n, as well as flow contraction and expansion coefficients. Wherever a change in n-values is deemed desirable, either three such values, for left overbank, right overbank, and channel may be listed, or more than three values and their respective end stations.

44. Options which can be used include the choice of having all input data printed out, of adding extra comment lines to the input, and of using feet, miles, or kilometres as a measure of the sections' distance from the watershed outlet.

45. A computer program, XSEC-PROC, was written to convert the contents of a cross-section tape file into standard HEC-2 input cards. The processes followed by the program are shown in Figure A1. Detailed input instructions to XSEC-PROC, as well as a program statement listing, a list of definitions of program variables, and a sample problem with input and output lines, are included in Appendix A.

Collection of soils, topography, land use, and other runoff-determining data

46. Physical watershed and subarea data like land slopes, stream lengths, surface roughness factors, and soil properties affecting runoff are needed for the development of design hydrographs.

47. Quadrangle maps by USGS, SCS soils and land use maps, and aerial and photographs or infrared imagery are prime sources of basic information. For modeling small watersheds, a manual procedure of extracting watershed properties data and converting them into HEC-1 input is not a difficult task, but for larger watersheds a more automated process of producing these rather voluminous HEC-1 input decks is needed. The input card sequence is highly repetitive and conducive to human error if produced in large quantities, but it follows a simple and consistent logic amenable to computer processing.

48. Automatic interpretation of remote sensing imagery has made considerable progress over the past two decades. The U. S. Army Engineer Waterways Experiment Station (WES), SCS, as well as many universities, have developed techniques to scan imagery and classify the contents by land and topographic parameters.

49. A program HYDPAR has been developed by the HEC[7] for the bookkeeping of physical data and for producing representative average values of HEC-1 input parameters. The program was expanded by WES into the version HYDPAR2, for which a description and a set of input instructions are given in Appendix B. This program scans a digital data bank for subarea boundaries and watershed parameters on a grid-by-grid basis, sorts the information by subbasin or minibasin, and produces an almost complete HEC-1 input deck for a watershed or subbasin.

Establishment of a digital data bank

50. A digital data bank for storing all quantitative data on watershed geometry, soil, and land use is indispensible in any automatic hydrologic and hydraulic modeling process.

51. One or more standard geometric grid sizes should be chosen for the data bank. In presently ongoing research at WES, a 200- by 200-m grid network was established for general watershed data, and a 50- by 50-m network was established for detailed land use data within flood-prone areas.

52. A standard geographic coordinate system should be chosen and available maps rectified to agree with it. In its mapping, WES has adopted the Universal Transverse Mercator (UTM) system. Air photos and other imagery used must also be rectified to agree with this coordinate system so that data from maps and imagery can be stored in the proper grid spaces. Subarea boundaries are digitized and associated identification numbers specified.

53. A program must be written to scan through the grid network, identify each grid space by the subbasin or minibasin to which it belongs, and reproduce, upon request, a map of all areas and subareas with corresponding environmental information. These maps should then be verified by visual comparison with the original maps and other imagery.

54. After verification, the data bank is ready to be accessed by the program HYDPAR2 which combines the watershed data and some separately dictated routing instructions to generate all but the introductory input cards for HEC-1.

Collection of
measured rainfall-runoff data

55. The final objective of a hydrologic and hydraulic flood simulation model is to reproduce the watershed response to storms of varying recurrence frequency under present and future conditions. To verify the model accuracy, however, reliable data on recorded historic storm and runoff events should be used.

56. The nearest rain gages with reliable records should be located. The hyetographs to be used in the model calibration can then be weighted by some appropriate method to reflect the extent to which each rain gage represents rainfall in any subarea. Three available rainfall-weighting procedures are Isohyetal Contour Mapping, Thiessen Diagraming, and Weighting by the Inverse of the Distances to all adjacent gages. These methods are applied to the watershed shown in Figure 9. The Isohyetal Mapping technique is, however, applicable only to total storms and not to hyetographs. Besides, this method is difficult to perform in any computerized form.

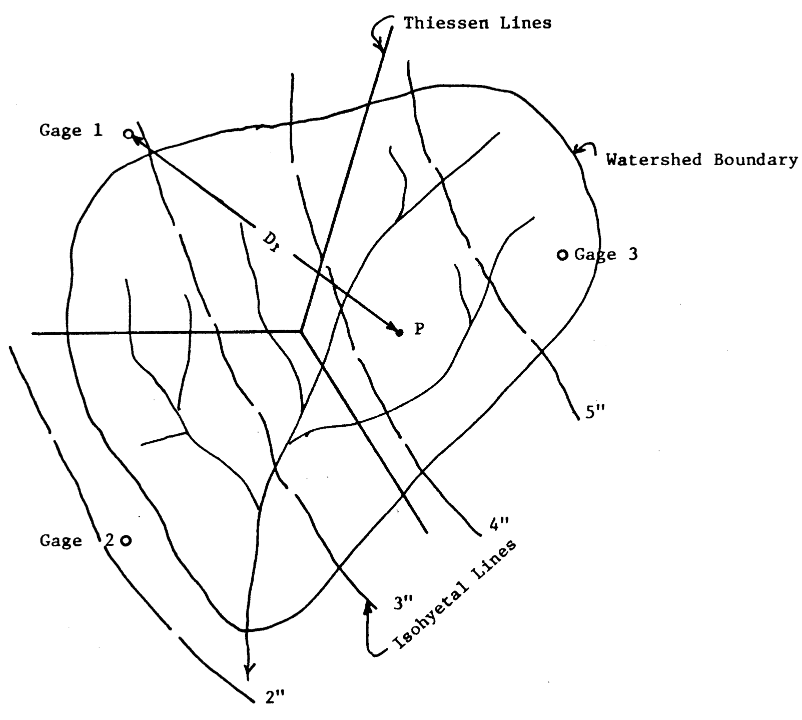

Figure 9. Watershed with three rain gages and
Isohyetal and Thiessen Lines

57. The Thiessen Diagram weights rainfall at any point exclusively in favor of the nearest gage, even if the difference between distances to nearby rain gages is only a few feet. This can lead to abrupt changes in weighted rainfall between two adjacent subareas. If the watershed is divided into a fine grid system for land use mapping as described in paragraph 51, the hyetograph recorded at the rain gage nearest to each grid space can be assigned to that space and the resulting grid space hyetographs averaged over the respective subarea.

58. A rainfall-weighting procedure recently proposed and ideally adapted to computer processing is that of weighting by inverse distances. As an example, let point P in Figure 9 represent the center of a grid space registered in the digital data base. The weighted average rainfall R amount for any given time interval is

$$R = \frac{\sum_{i=1}^{n} \frac{R_i}{D_i^k}}{\sum_{i=1}^{n} \frac{1}{D_i^k}} \tag{3}$$

where

R_i = rainfall recorded at gage i

D_i = distance between gage i and point P

n = the number of gages affecting average rainfall at point P

k = exponent

59. Dean and Synder[8] conclude from the results of recent research that the number n of nearest gages to any point should be three in general, and four in the case of a very dense rainfall network. Dean also recommends a value of $k = 2$ for the exponent in Equation 3, which in his tests gave a slightly greater accuracy in representing rainfall distributions and reduced the discrepancy between weighted rainfall obtained by using the three or four rain gages nearest to a point. A program RAINWEIGHT was written at the Department of Civil Engineering of Pennsylvania State University, which computes weighted average rainfall amount for a small grid system by the inverse distance

formula. The program description, listing, and input instructions are presented in Appendix C.

60. A potentially serious drawback of any weighting method applied to the ordinates of hyetographs has come to light during modeling efforts performed by WES on the Wolf River watershed in Tennessee. Unless the hyetographs recorded at the various rain gages are synchronized and of similar shape, weighted averaging of several hyetographs may lead to a marked attenuation of rainfall intensities. Figure 10 shows hypothetical hyetographs from three stations weighted by 30, 20, and 50 percent, respectively. Each of the three hyetographs has a distinct peak describing a storm which seems to be traveling from gage 1 toward gage 3. The weighted average hyetograph on line d combines all peaks into a single, drawn-out storm of low-to-medium intensity. This should, at least locally, result in highly reduced flood peak estimates.

61. To maintain the peakedness of the recorded hyetographs and still give an appropriate weight to several nearby rain gages, the following procedure is suggested. For each minibasin to be modeled, the three gages nearest to the basin's center of gravity are found, and the three hyetographs' total rainfall amounts as well as their temporal centers of gravity are computed. The basin is then assigned a hyetograph shape like that of the nearest rain gage. Weighting factors determined by the Inverse Distance or Thiessen method are then applied to adjust rainfall ordinates and timing. Line e in Figure 10 demonstrates this procedure; the peakedness of the storm recorded at gage 3 is retained, but its ordinates are reduced and its timing advanced according to the weights assigned to gages 1, 2, and 3. If in addition to some recording rain gages, nonrecording gages are found in the vicinity of the minibasin, the total storm amount from these gages may be used to further weight the hyetograph ordinates without changing its shape and timing.

62. For needed calibration of the hydrologic model, several recorded events of rainfall and runoff should be chosen. If possible, isolated intense storms of short duration should be used because these lend themselves best to unit hydrograph development.

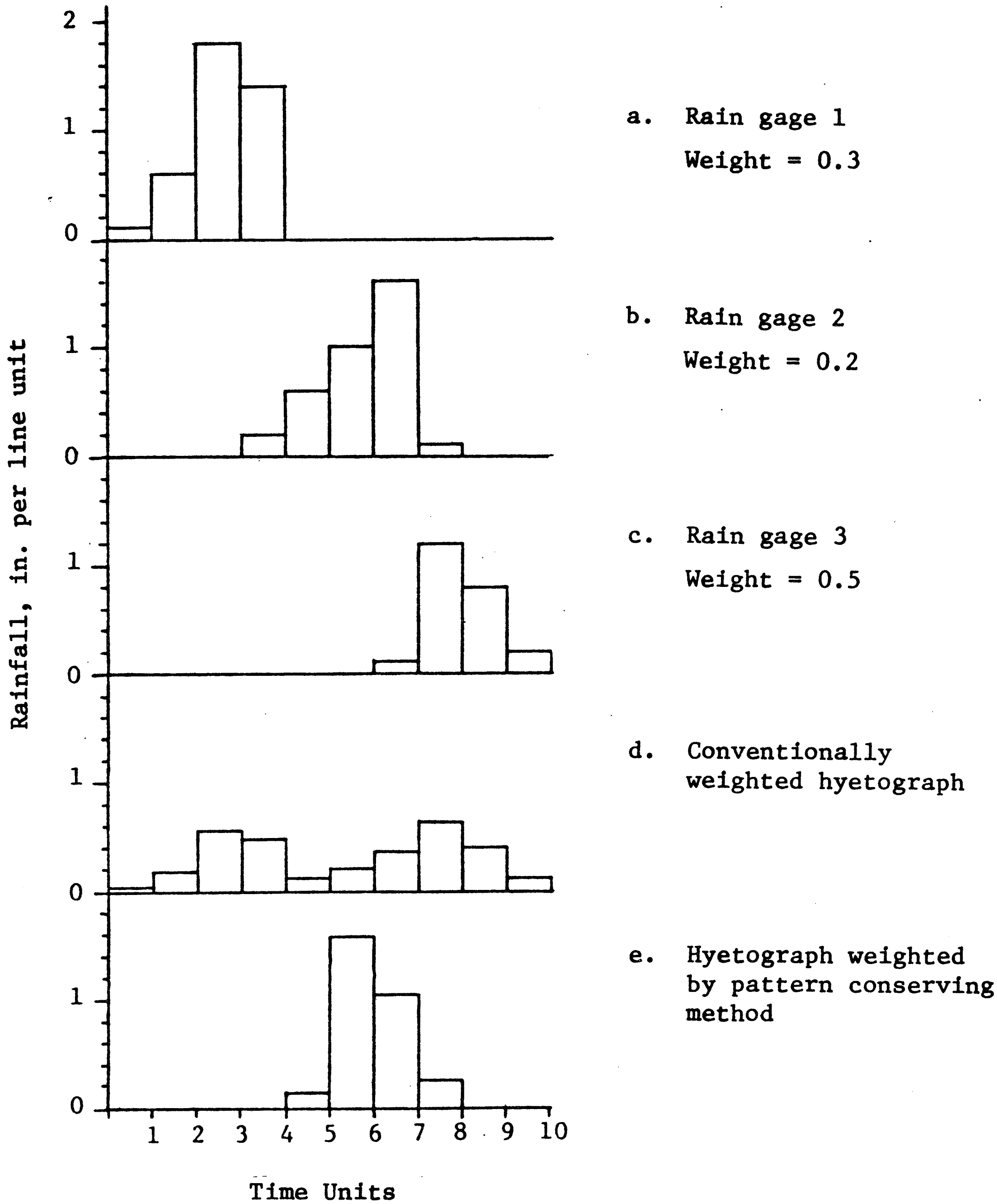

Figure 10. Example of hyetograph weighting methods

63. A major problem in the collection of hydrologic data is usually the lack of streamflow records on a watershed. Ideally, each subbasin in a watershed should have streamflow records in order to calibrate the rainfall-runoff model with any degree of confidence; in reality it is often found that no stream gage record whatsoever is available for an entire watershed.

Model interfacing and calibration

64. Once the watershed subdivision has been accomplished and an adequate amount of rainfall-runoff data collected, calibration modeling can begin. On watersheds with narrow valleys in which floodplain storage has a small attenuation effect on flood hydrographs, the modeling process can proceed along a simple sequence of operational steps without any feedback loops, as shown in Figure 11. The watershed data is processed by HYDPAR and combined with rainfall data to compile the input deck for program HEC-1. The Muskingum routine may be used for hydrograph routing with the proper coefficients determined from hydrologic handbooks. The peak flows of the hydrographs generated at the outflow section of each minibasin are then extracted for input at the appropriate locations of the HEC-2 input deck, which has been largely produced by program XSEC-PROC. Finally, the HEC-2 program is run to compute flood levels; computed flows as well as flood profiles may then be compared with observed flows and flood levels.

65. In a watershed with wide floodplains, storage is an important factor in attenuating hydrographs. This storage, however, cannot be known prior to flood profile computations. The HEC-2 program may be used to compute cumulative floodplain storage and offers the option of punching out storage-outflow cards for HEC-1, but HEC-2 depends on the HEC-1 runs and output for flow values. In this case the looped process sequence shown in Figure 12 should be used. In this sequence, a preliminary run of HYDPAR2 is performed to generate a set of synthetic streamflow distributions based on the equation

$$Q = c\ A^{0.7} \tag{4}$$

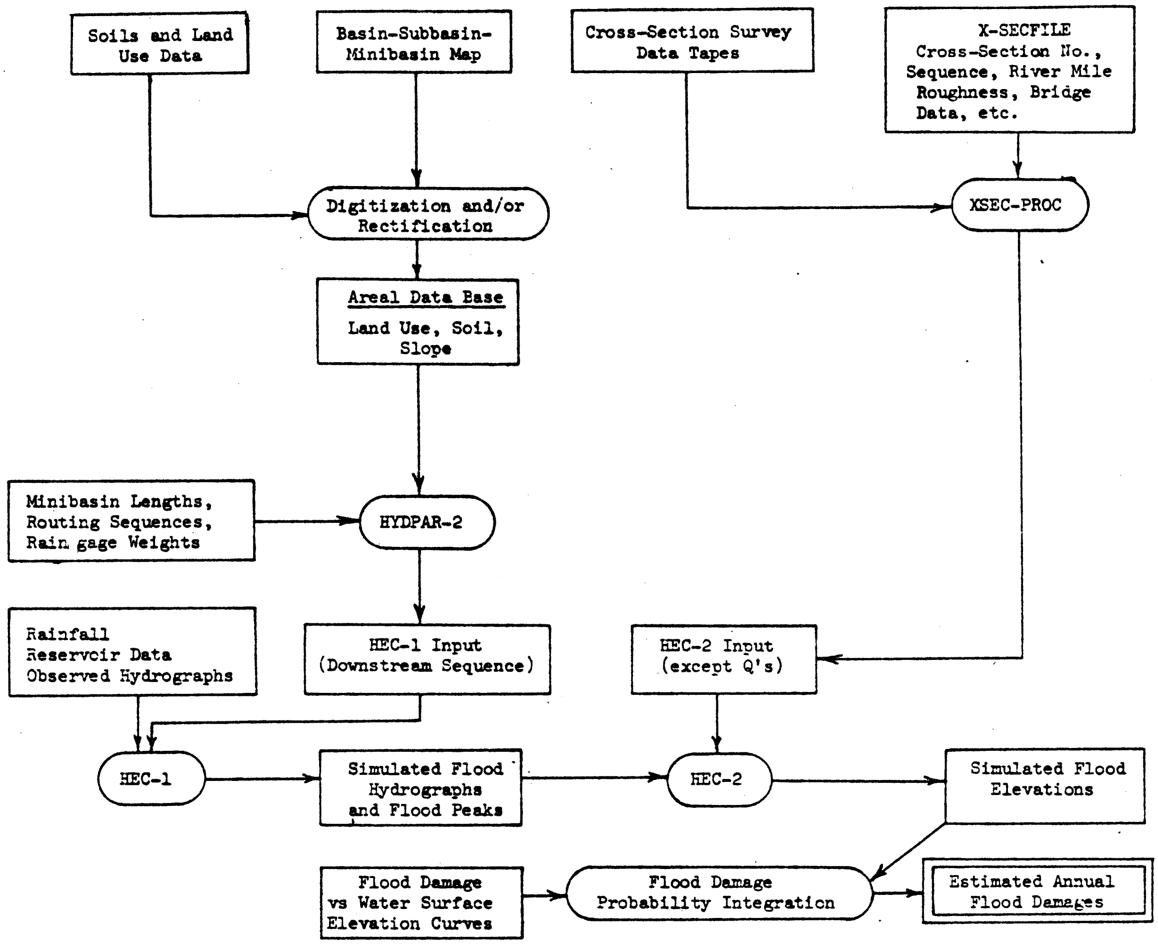

Figure 11. Flow chart of systematic floodplain and damage study (narrow floodplains with small storage effects)

where

Q = peak flow at a given point on the watershed

A = drainage area upstream from this point

c = a constant given 10 values between zero and a value resulting in the maximum flood possibly expected at the watershed outlet, as determined by some chosen conservative empirical formula or available flood frequency data.

66. Program HEC-2 is then run with these synthetic flows to produce a table of storage-outflow values for HEC-1. Subsequently HYDPAR2 is called upon to generate the HEC-1 input deck, HEC-1 is run, and its output used to make the final run of HEC-2.

67. In calibrating the models, the hydrographs generated by HEC-1

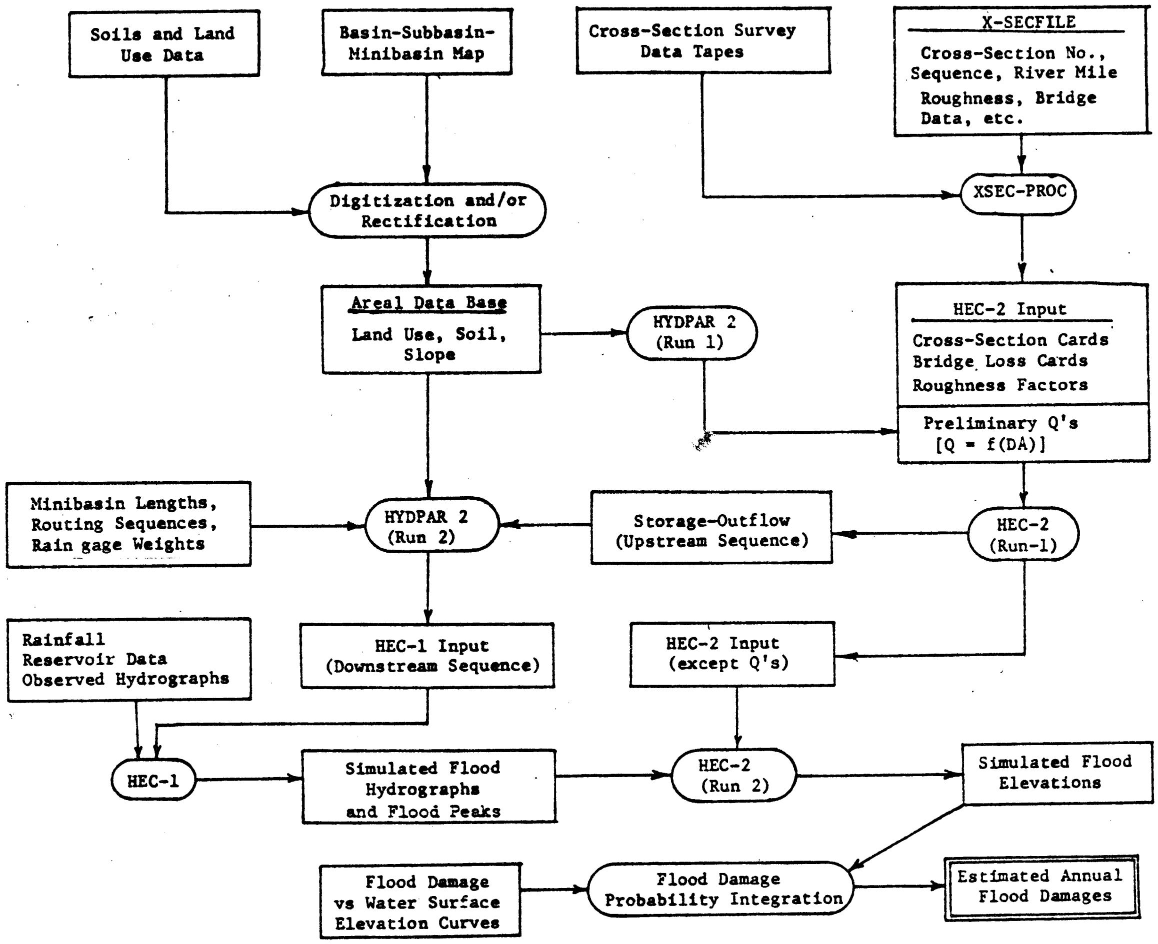

Figure 12. Flow chart of systematic floodplain and damage study (broad floodplains with major storage effects)

should be compared with recorded hydrographs wherever the latter are available. Model parameters may be adjusted as follows: if the computed hydrograph volumes are consistantly smaller or larger than the recorded hydrographs, the error may be in the rainfall data or the runoff loss parameters used in HEC-1. The rainfall data should be examined and possibly the rain amounts from a different weighting routine could be used if the recorded rainfall showed large variations between gages. The adjustment in loss parameters is most likely more effective in changing the hydrograph volume. For example, changing the SCS curve number for an area from 60 to 70, the runoff from a 4-in. rainfall would increase from 0.76 to 1.33 in., or by about 75 percent.

68. Changing the streambed or overbank roughness factors in the HEC-2 input will affect the floodplain storage which in turn will modify the hydrograph attenuation during routing by HEC-1. Use of the roughness factors for hydrograph adjustment, however, precludes the later use of roughness factors to calibrate the stage-discharge relationships computed by HEC-2, if these do not agree with the relationship between recorded flood peaks and observed high watermarks.

Choice of event storms

69. The expected products of a hydrologic-hydraulic flood-level modeling study are the surface water profiles and, possibly, the flood damages resulting from a series of floods of specified frequency (recurrence probability or return period).

70. If hydrologic modeling is applied, the floods must be generated from appropriate frequency storms. Even though it is recognized that a 100-year flood event, for example, may be produced by a storm of larger or smaller return period, the hydrologist has little choice in the construction of his model but to assume that the return periods of a causative storm and the resulting flood are identical. The basic rainfall data needed as hydrologic input are then the magnitudes of these storms of specified return periods, the appropriate rainfall duration, and time intervals to be employed in the hydrograph synthesis and routing processes.

71. The set of return periods is usually specified by the client or the agency requesting the project. Frequently, however, an unnecessarily large number of return periods is specified, resulting in unmanagably large stacks of computer output which do more to confuse than to clarify the modeling results. The requesting agency may be more interested in only examining summary tables which can be output to user's requirements or may choose to examine alternative methods of analyzing multiple return periods based on data obtained from a prescribed set of return periods. For example, flood peaks, crest elevations, and damages at chosen points on the watershed could well be calculated for return periods of 2, 10, 25, and 100 years and plotted on logarithmic or probability graph papers and the results for any intermediate return period

read from the graphs with as much reliability as when the modeling studies are repeated for all return periods. Occasionally, a one or even one half year flood and/or the standard project flood may be added to the above list of four basic return periods. A 500-year event flood is often generated for flood insurance studies. It should be clearly understood, though, that results are highly suspect because rainfalls for recurrence intervals exceeding 100 years are subject to large uncertainties. It is the responsibility of the user to select a set of return periods and an output format on the basis of cost, time, and level of accuracy.

72. The rainfall duration chosen should be at least as long as the estimated travel time through the watershed. In contrast to the Rational Formula for which the storm duration chosen is equal to the travel time and applied uniformly not only over the drainage area but also over the storm duration, the rainfall in a HEC-1 hydrograph simulation is applied over up to 150 time intervals, usually as a storm of time-varying intensity.

73. Storms of a desired return period and duration can be obtained from U. S. Weather Bureau Publications[9,10,11] or in many states from more detailed maps prepared for regional rainfall studies. To obtain a time-varying rainfall input it is suggested that hyetographs be constructed from 24-hr design storms using the dimensionless temporal storm distribution developed by SCS,[12] or similar rainfall duration-intensity curves developed by other agencies.

74. If hydrographs are to be generated for largely varying areas, it may be advisable to adjust rainfall as a function of watershed area as prescribed in U. S. Weather Bureau TP 40[9] or apply an elliptical storm distribution pattern as prescribed by the U. S. Army Engineers[13] for the standard project storm. If the inverse distance rainfall weighting program (Appendix C) is used in the model calibration with recorded hyetographs, the elliptical storm distribution can be superimposed on the watershed map and a rainfall assigned to each rain gage, which is then converted to a hyetograph for each gage. The weighting program can then be used to compute a weighted hyetograph for each minibasin.

Generation of event hydrograph

75. After adjustment of watershed parameters by model calibration and the construction of input hyetographs by procedures described earlier under "Choice of Event Storms" associated event hydrographs can be generated with the use of the HEC-1 program. As described earlier, the storage-outflow relationships for the Y-card input may have to be generated by a preliminary run of HEC-2, but this will probably be done already during the model calibration process.

76. The HEC-1 input deck supplied by HYDPAR2 and HEC-2 should be checked and supplemented where necessary, using the most up-to-date HEC-1 input instructions.

77. Upon examining the HEC-1 output hydrograph, the earlier suggestion to keep the number of modeled return periods small may begin to make sense. The HEC-1 output is rather voluminous even for just one subbasin with 40 minibasins; the output for a large watershed and 10 return periods could fill an entire room. The HEC-1 option to print only the most essential information, namely the flood peaks, may be useful in reducing output volume as well as providing the input to the final HEC-2 run.

Backwater computations

78. Backwater, or flood profile computations, can be performed by the HEC-2 or a similar program. The program XSEC-PROC can provide the entire input deck except for the title cards, the J cards for initiating the program and certain options, and the job ending cards EJ and ER. Some special optional cards such as X4 or C cards will not be provided by XSEC-PROC either.

79. A fair amount of visual checking and comparing of the HEC-2 output is necessary to remove occasional inconsistencies like decreasing flood levels with increasing discharges. The most up-to-date HEC-2 version and input instructions should be used.

Areal delineation of floods

80. The flood levels computed by HEC-2 must be extended laterally to plot the flood boundaries. As long as the stream as well as the topographic contour lines are reasonably smooth, the flood level

boundary mapping should be a relatively easy task, even for a computerized plotting process used in conjunction with a digitized topographic data base. Considerable technical judgment is necessary, however, to plot flood level boundaries along channel bends and across road or railroad embankments.

81. For two separate reasons, it is strongly suggested that the energy line be used instead of the computed water surface for flood plotting. First, the computed water surface elevation fluctuates with velocity, tending to be low in swift flow reaches of the stream and high in slow flow reaches. These undulations do not properly represent overbank conditions where water surface profiles tend to be much smoother. The computed energy line changes much more gradually than the computed water surface. Second, the computed water surface represents the average water surface in the stream, disregarding any waves which may be created by the stream velocity and carried over into the floodplain. The energy line represents the average water surface elevations plus the kinetic energy or velocity head. The energy line thus constitutes the upper limit of the wave height which can be generated in the stream and sent out across the overbanks.

Flood damage computations

82. The estimate of potential flood damages is often the most difficult and controversial task in a flood study. Damages suffered during previous floods may depend more on the amount of advance warning than on the stage of the flood crest. It is also very difficult to estimate the relationship between flood depth and damages, particularly in commercial or industrial buildings. Managers or owners of industrial-commercial enterprises are sometimes reluctant to talk about flood damages suffered, or they may exaggerate the estimates in order to improve the likelihood of having flood-control projects approved.

83. If several substantial floods have been experienced over the past few years, a flood discharge-flood damage curve can be generated. In a systematic modeling project, discharge damage would best be tabulated or drawn for each minibasin separately.

84. Frequently, however, too few floods have been experienced to

establish a reliable curve, and potential damages are estimated by tabulating the cumulative value of perishable property located at or below given topographic elevations. This method results in a curve of cumulative damages versus flood elevation which is not very useful because the peak water surface elevation of any flood event varies with its location relative to the stream (see paragraph 81). Thus the potential flood estimates should be computed individually by grids small enough that the flood level within the grid may be considered horizontal. Damages can then be related to discharges by interpolating between water surface elevations or energy grade lines at the nearest upstream and downstream cross sections computed with HEC-2. The damages resulting from a given flood may finally be added for all blocks within a minibasin.

85. In conjunction with flood frequency analyses the discharge-damage relationships can be converted into curves of damage versus flood exceedence probability, as shown in Figure 13. Total annual expected damages are determined by integrating the damages of the full exceedence probability range. Figure 13 illustrates this integration graphically. The total damages TD

$$TD = \int_{0}^{P_o} D \, dP \tag{5}$$

are represented by the area under the damage-probability curve. In Equation 5, D is the damage caused by a flood of exceedence probability P and P_o is the exceedence probability of the largest flood which causes zero damages.

86. In Figure 13 it will be noticed that the flood damages corresponding to an exceedence probability close to zero are large and unknown. This is a common dilemma in flood damage analyses. Flood peaks for 500- or 1000-year return periods could be computed, but results would be highly uncertain; a manual curve extension could be made, but it would probably be equally unreliable (see paragraph 71). Thus, both

exceedence probability and damage estimates are highly unreliable for extremely large floods. This, in turn, also renders the determination of average annual flood damages a highly speculative process.

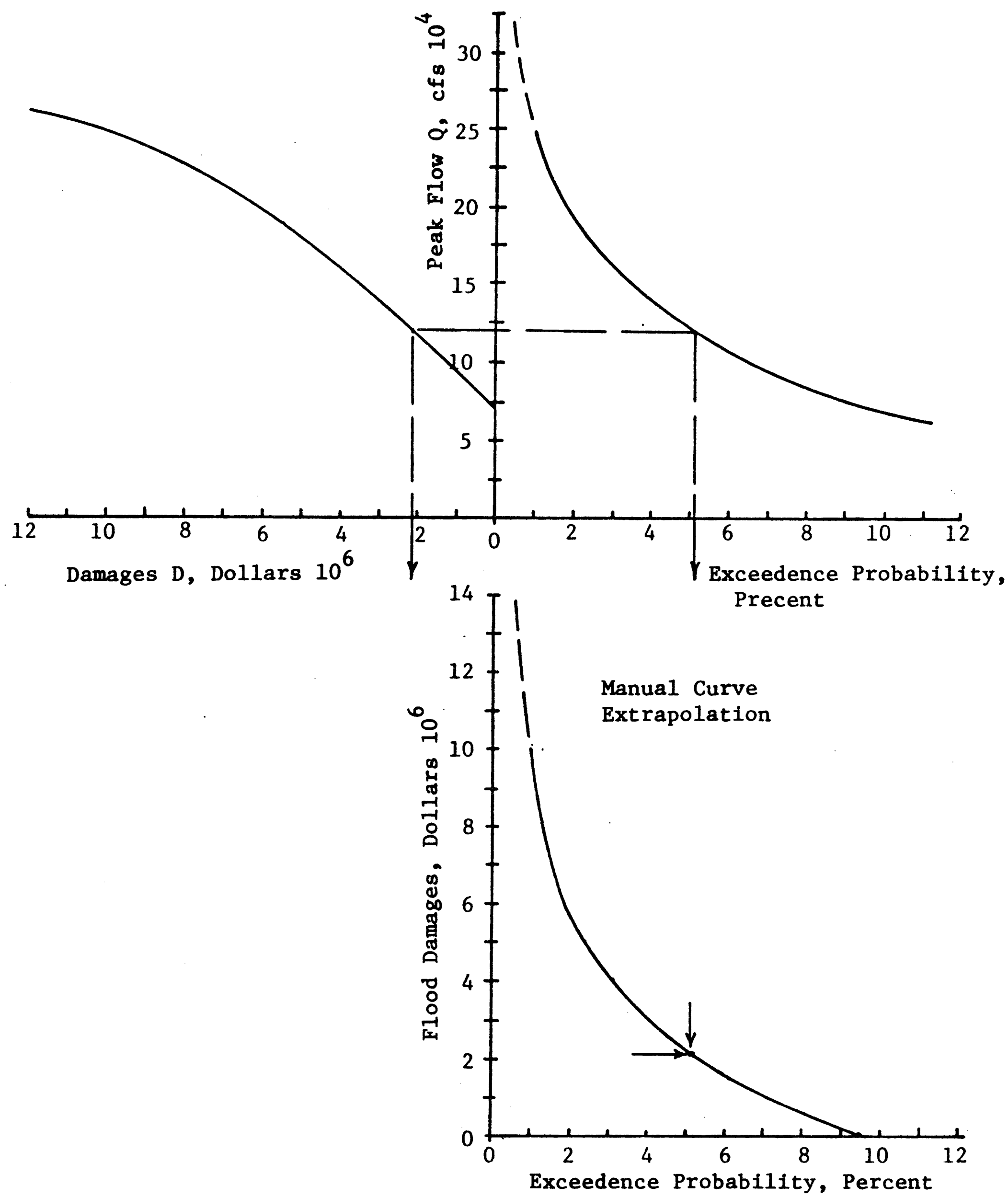

Figure 13. Damage-probability curve generated from flood-damage and flood-probability curves

References

1. HEC, "Training Course on Flood Plain Hydrology," 1977.
2. HEC, "Water Surface Profiles," IDH Vol 6, Draft, 1974.
3. Barr Engineering Co., "The Effect of Cross Section Spacing and Cross Section Data Errors on Water Surface Profile Determination," Unpublished Report for the Minnesota Department of Natural Resources, 1972, Minneapolis, Minn.
4. Tavener, G. F., "Stability and Reach Length in Water Surface Profile Determination," AWRA Water Resources Bulletin, Oct 1973, pp 950-962.
5. Aron, G., "Optimal Energy Slope Averaging and Reach Length Formulas in Backwater Computations," Short course notes on Flood Plain Hydraulics and Use of HEC-2, 1976, Pennsylvania State University.
6. Reed, J. R. and Wolfkill, D., "Evaluation of Friction Slope Models Proceedings, Symposium on Inland Waterways for Navigation and Flood Control," Vol 2, Aug 1976, pp 1159-1178, Colorado State University.
7. HEC, "HYDPAR Users Manual," U. S. Army Hydrologic Engineering Center, 1976, Davis, Calif.
8. Dean, J. D. and Snyder, W. M., "Temporally and Areally Distributed Rainfall," ASCE Journal, Vol 103, No. IR2, Jun 1977, pp 221-229.
9. U. S. Weather Bureau, "Rainfall Frequency Atlas of the United States," Tech Paper 40, May 1961.
10. U. S. Weather Bureau, "Two- to Ten-Day Precipitation for Return Periods of 2 to 100 years in the Contigious United States," Tech Paper 49, 1964.
11. U. S. Weather Bureau, "Five- to 60-Minute Precipitation Frequency for the Eastern and Central United States," HYDRO-35, Jun 1977.
12. U. S. Soil Conservation Service, "A Method for Estimating Volume and Rate of Runoff in Small Watersheds," SCS-TD-149, Apr 1973.
13. U. S. Dept. of the Army, Corps of Engineers, "Standard Project Flood Determination," CE Bulletin 52-8, 1952.

APPENDIX A: COMPUTER PROGRAM XSEC-PROC

1. A computer program was written to convert data from a cross-section data file to standard HEC-2 input cards. The program as written takes its input from a tape and produces punched cards as output. Simple control instructions, however, can be used to change the input and/or output type. The processes followed in the XSEC-PROC program are shown in Figure A1.

2. The advantages of using the computer program XSEC-PROC over conventional procedures are:

- a. Field cross-section data (i.e., rod readings) can easily be converted to HEC-2 input data decks (i.e., BT and GR cards) provided field crews are aware of some simple coding criteria.
- b. Coding for either normal or special bridge routines is done by the program; the operator has only to decide which routine is most applicable for a given situation.
- c. Many of the variables required on the SB card (i.e., trapezoidal area, bottom width, side slaps, etc.) used in the special bridge routine are computed within the program and reduce the time required for these factors to be determined manually.
- d. Repeated sections are automatically raised or lowered depending of the slope of the stream bottom between two measured sections.
- e. The probability of human error in coding HEC-2 data decks is greatly reduced.

3. The input to XSEC-PROC consists of the regular cross-section survey data, preceded by supplemental data lines labeled A to K as described below. The identification letters, entered in column 1 of each line, are not mandatory, but they make it easier to read the coded information.

Input

Line A. - Title or Comment Line. Format (17A4, I4)

Columns	Variable	Value	Description
1-68	COMM (I)		Any alphanumeric information.
69-72	IC	0 or 1	If IC=1, read another A line.

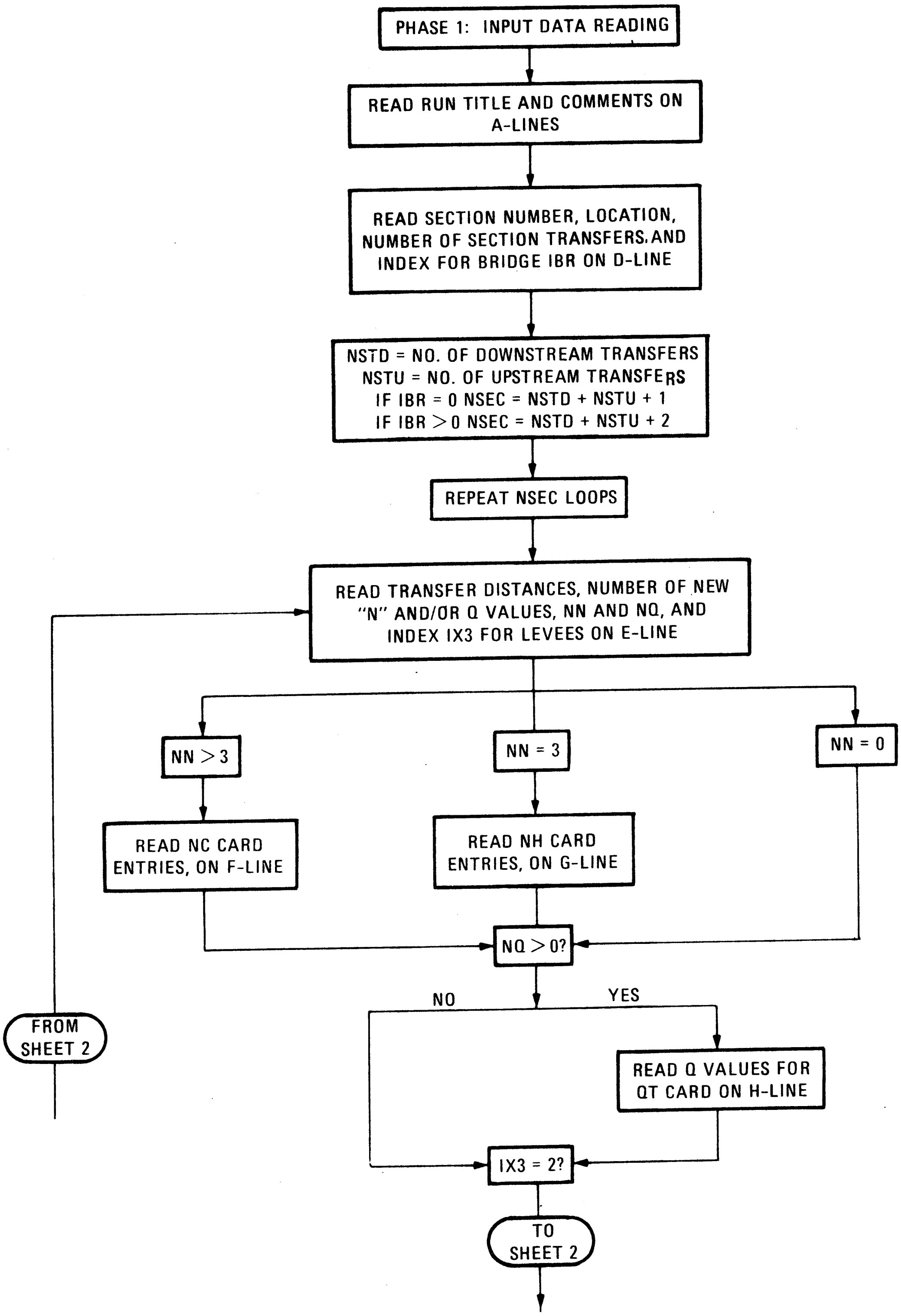

Figure A1. Flow chart of XSEC-PROC (sheet 1 of 4)

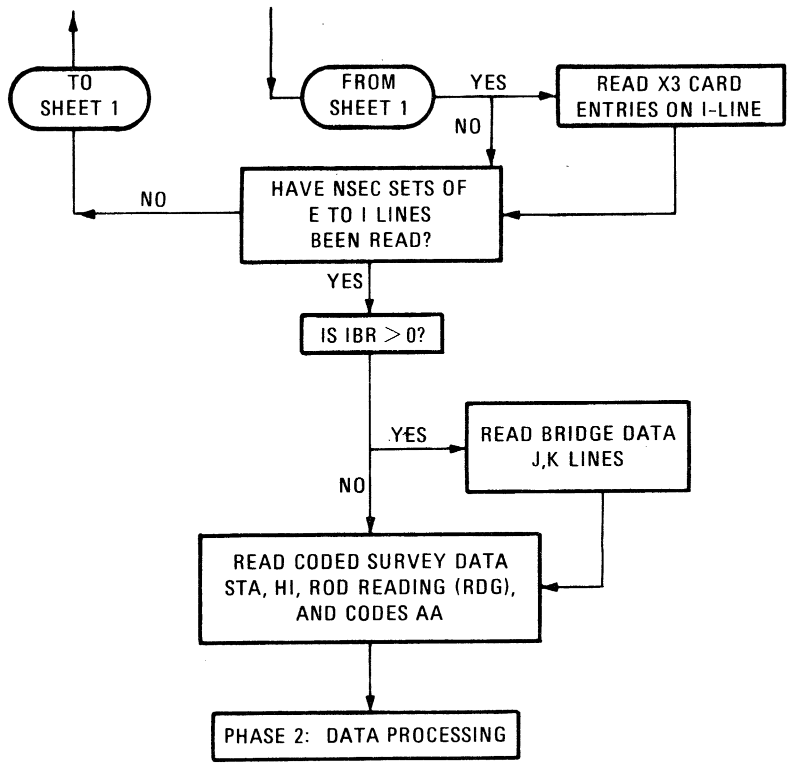

Figure A1 (sheet 2 of 4)

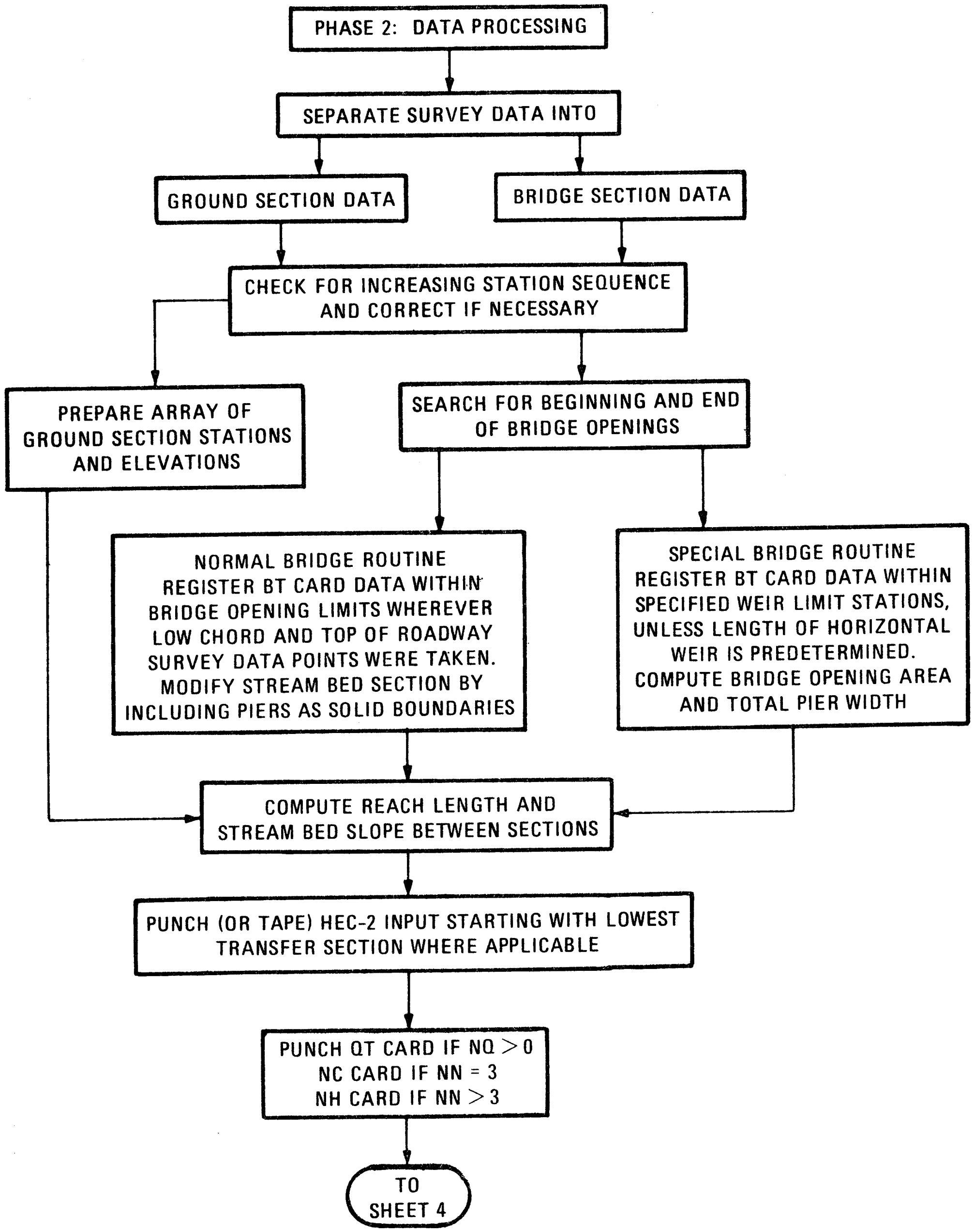

Figure A1 (sheet 3 of 4)

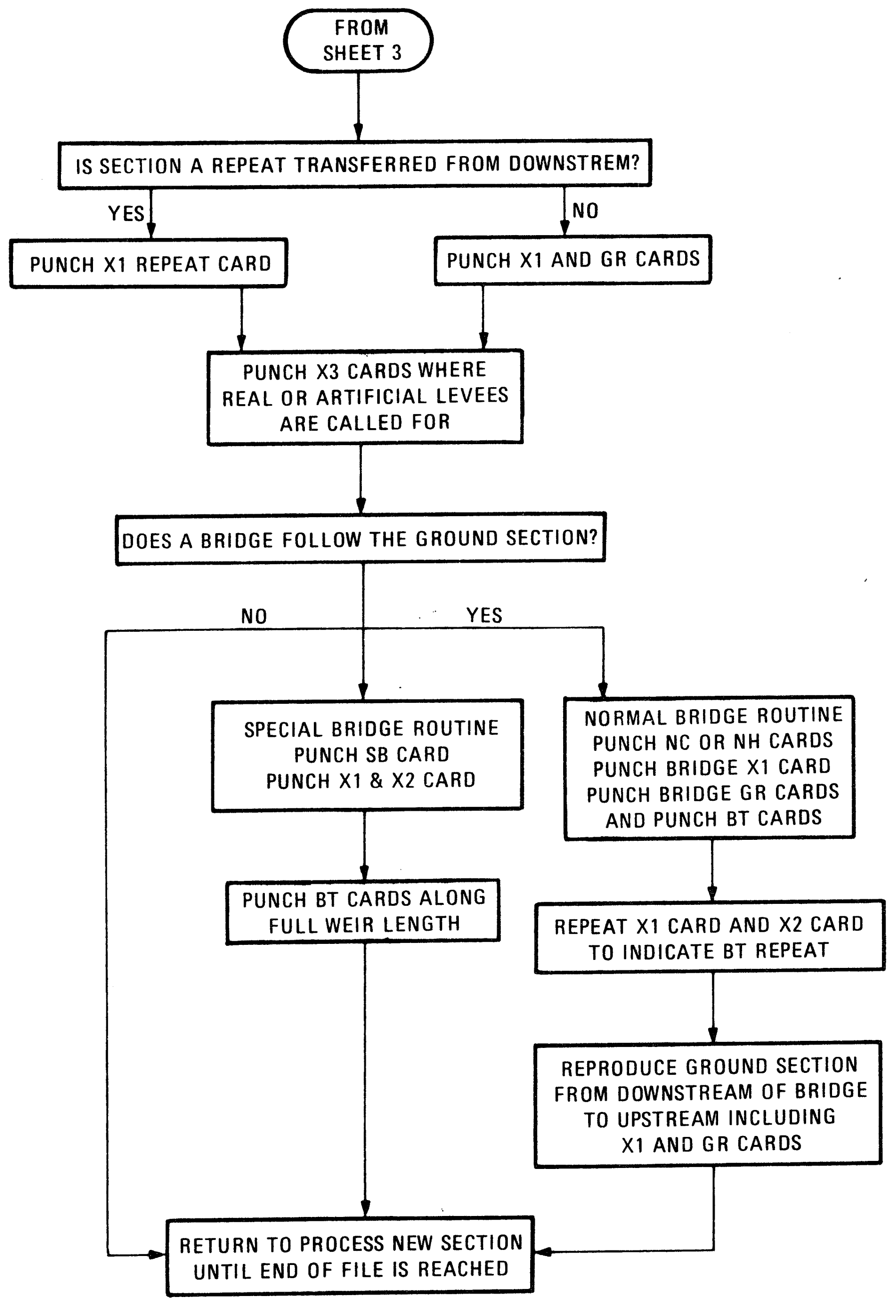

Figure A1 (sheet 4 of 4)

Line B. - Printing and Units Codes. Format (A1, I3, I4)

Columns	Variable	Value	Description
1	A (2)	B	
2-4	IPRI	0 or +	Printing instruction. If IPRI>0, XSEC-PROC input data are printed out.
5-8	IUN	0, 1, or 2	Measurement units index for distances between sections and watershed outlet.
			If IUN=0 distance is in feet.
			If IUN=1 distance is in miles.
			If IUN=2 distance is in kilometres and all other dimensions in metres.

Lines C to K, where applicable, and survey data lines are repeated for each section.

Line C. - Initializing and end-of-file line. Format (A1, 8I2, 4X, A3, 24F2.0)

Columns	Variable	Value	Description
1	A (3)	C	
2-21	Misc.	blank	Various integers which must be set to zero prior to processing a new section.
22-24	AA	blank	Read section data.
		EOF	End of file is reached. Exit.
25-72	Misc.	blank	Various real variables which must be set to zero prior to processing a new section.

Line D. - General Section Information. Format (A1, I7, 4I4, 4F6.0, 23X, I1)

Columns	Variable	Value	Description
1	A (4)	D	
2-8	ISEC (J)	+	Cross-section identification number.
		-	Identification number of first cross section of a tributary. ISEC must be the same as ISEC of cross section just below the junction.

Line D. (Continued)

Columns	Variable	Value	Description
9-12	NSTD	+	Number of copied cross sections transfered downstream. Minimum of 1 at a bridge.
13-16	NSTU	+	Number of copied cross sections transfered upstream. Minimum of 2 at a bridge.
17-20	IBR	0	No bridge at this section.
		1	Normal bridge.
		2	Special bridge.
21-24	ITR	0 1	At all times, except if last transfered cross section is located just below a tributary. ISEC will be increased for this transfer section.
25-30	XRIV	+	Distance from section to watershed outlet, in feet, miles, or kilometres, depending on the value of IUN on line B.
31-36 37-42	STCH (1) STCH (2)	+	Left and right channel boundary stations.
43-48	THETA	+	Angle by which the section deviates from the normal to the channel, in degrees.
72	IC	0 or 1	If IC=1 an A-Line is read after this line, containing verbal descriptions.

Lines, E, F, G, H, and I are repeated, where applicable, a number of times equal NSTD + NSTU + 1.

Line E. Transfer Section Line. Format (A1, I3, 2IA, 5F6.0)

Columns	Variable	Value	Description
1	A (5)	E	
2-4	NN (I)	0	No new n values for transfer section I.
		3	3 new n values and possibly new contraction and expansion coefficients will be read on F line for NC card.

Line E. (Continued)

Columns	Variable	Value	Description
		>3	NN new values will be read on G line for NH card.
5-6	NQ (I)	0	No QT card needed.
		+	NQ discharge values to be read on H line for QT card.
7-8	IX3 (I)	0	No X3 card needed.
		1	Standard X3 card to be punched.
		2	Levee values will be read on I line for X3 card.
9-14	DXTR (I)	- or +	Distance of transfer section from surveyed section (- for downstream, + for upstream transfer section).
15-20	DYTR (I)	- or +	Drop or lift of transfer section I with respect to surveyed section, for entry in field 9 of X1 card. If DYTR = 0, drop or lift will be computed from slope and reach length. DYTR must not be 0 in first section.
21-26	XW (I)	0	No section width multiplier needed.
		+	Section width multiplier for field 8 of X1 card.
27-32	DXL (I)	0	Overbank reach length equals channel length.
33-38	DXR (I)	+ or -	Left and right overbank reach length increments, respectively.

Line F. (If NN (I) = 3) - NC Card Entries. Format (A1, F5.0, 11F6.0)

Columns	Variable	Value	Description
1	A (6)	F	
2-6	VNC (I, II)	0 or +	Entries for NC card.
7-12	II = 1, 5		
13-18			
etc.			

Line G. (If NN (I) > 3) - NH Card Entries. Format (A1, F5.0, 11F6.0)

Columns	Variable	Value	Description
1	A (7)	G	

Line G. (Continued)

Columns	Variable	Value	Description
2-6	VNH (I, 1)	+	First n value for NH card.
7-12	XNH (I, 1)	+	End station of first n value, for NH card.
13-18	VNH (I, 2)	+	Second n value.
19-24	XNH (I, 2)	+	End station of second n value.
etc.	etc.		
	etc.		
	repeat NN (i) times		

Line H. (If NQ (I) > 0) - QT Card Entries. Format (A1, F5.0, 11F6.0)

Columns	Variable	Value	Description
1	A (8)		
2-6	QT (I, 1)	+	Flow 1 for QT card in Auxiliary Section I.
7-12	QT (I, 2)		Flow 2 for QT card in Auxiliary Section I.
13-18	QT (I, 3)		Flow 3 for QT card in Auxiliary Section I.
etc.	etc.		etc.

Line I. (If IX3 = 2) - X3 Card Entries. Format (A1, F7.0, 8F8.0)

Columns	Variable	Value	Description
1	A (9)	I	
2-8	X3 (I, 1)	0 or +	HEC-2 Entries in X3 cards for levee and encroachment data.
9-16	X3 (I, 2)		
17-24	X3 (I, 3)		
etc.	etc.		

Line J. (If IBR > 0) - Bridge Data. Format (A1, F5.0, 11F6.0)

Columns	Variable	Value	Description
1	A (10)	J	
2-6	BRW	+	Bridge width, in feet or metres.
7-12	PW	+	Pier width, in feet or metres.
13-18	SHR	+	Solid height of railings.
19-24	STCHB (1)	+	Left channel bank station in bridge section.

Line J. (Continued)

Columns	Variable	Value	Description
25-30	STCHB (2)	+	Right channel bank station in bridge section.

Line K. (If IBR = 2) - Additional Special Bridge Data. Format (A1, F5.0, 11F6.0)

Columns	Variable	Value	Description
1	A (11)	K	
2-6	SB (1)	+	3 special bridge coefficients for fields 1-3 in SB card. (See HEC-2 manual for suggested values.)
7-12	SB (2)		
13-18	SB (3)		
19-24	WL	0	Weir length will be determined by limits WSTL, WSTR of BT cards.
		+	Special bridge effective weir length. Weir should be horizontal.
25-30	SS	0 or +	Abutment side slope $\frac{\Delta X}{\Delta Z}$.
31-36	WSTL*	+	Starting station of weir described by BT cards.
37-42	WSTR*	+	Ending station of weir described by BT cards.

* WSTL and WSTR should be left blank if WL > 0, and vice versa.

Cross-Section Survey Data - Format (3F6.0, 3X, A3, 2F6.0)

These lines do not have identification letter in column 1.

Columns	Variable	Value	Description
1-6	ST	0 or +	Station, in feet or metres, of a survey observation.
7-12	HJ	0	Previous height of instrument line of sight will be used.
		+	New height of instrument line of sight.
13-18	RDG	+	Rod reading at ground or road.
22-24	AA*	A	Alphabetic code word, to be matched against registered codes which direct the program into different branches of operations. See below for code words.

Cross-Section Survey Data (Continued)

Columns	Variable	Value	Description
25-30	YLCD	+	Rod reading at low chord of bridge.
31-36	RWRD	+	Rod reading at top of roadway above bridge opening.

* Codes CODE (I) against which readings AA will be matched.

i	CODE (i)	Description
1	3 blanks	Normal ground station.
2	BPR	Bridge pier station.
3	BSB	Streambed station under bridge, no pier.
4	CLR	Center line of road outside bridge.
5	EBR	Edge of bridge. More than one bridge opening is acceptable, and an EBR line should be coded at the beginning and end of each bridge.
6	END	End of section stations.
7	EWS	Edge of water surface at normal (fairly dry-weather flow). Only one edge of the stream needs to be coded and the reading will be used to estimate normal stream slope.
8	EOF	End of file. Causes program to terminate data reading and processing.

Definition of Variables

A (I) = Array of identification letters (from A to J) to be entered into Column 1 of XSEC-2 input.

AA = Alphabetic variable to be read as code from cross-section data lines in input, columns 22 to 24.

A1, A (2), A (3) = Arrays of 1, 2, and 3 constants set to zero, to be called on when wanted for output.

ABO = Area of bridge opening, computed internally from the survey data by the trapezoidal rule.

B1 = Constant set equal to 1, to be called on when wanted for output.

BT (I, II) = Matrix of output values for BT cards, for up to 50 stations per section.

BT2, BT3 = Variables for temporary assignment of BT (i, 2) and BT (i, 3) values.

CODE (I) = Alphabetic code entered in data block. Variables AA read from tape are compared with CODE (i) to identify cross-section stations as roadbed, bridge pier, and other features. CODE (i) from i = 10 to 22 is used for writing HEC-2 codes X1, GR, etc., and for printing out messages.

COMM (I) = Array of 17 comment words (alphabetic) read in subroutine COMMNT and printed out but not punched into HEC-2 input.

DXL (I), DXR (I) = Array of distances by which channel reach lengths should be lengthened or shortened for left and right overbanks.

DXTR (I) = Array of distances (- = downstream, + = upstream) by which surveyed cross sections should be transferred to create artificial sections.

DYTR (I) = Array of y-increments by which a transferred cross section should be moved up or down relative to surveyed cross section.

DY, DZ = Left and right height of each trapezoidal section of bridge opening. Used to compute area ABO of bridge opening. See Figure A2.

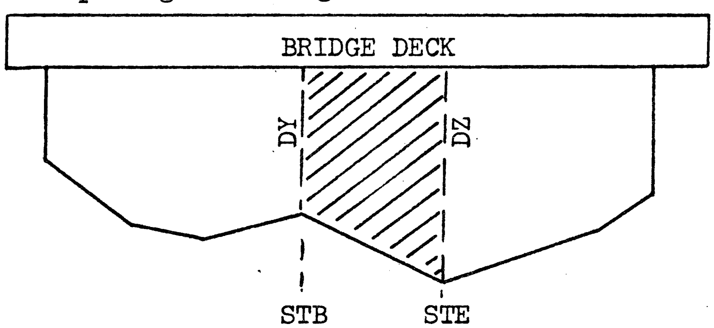

Figure A2. Bridge opening representation

EL (I) = Array of elevations for GR cards of ground sections.

ELB (I) = Array of elevations for GR cards of bridge sections.

HI = Surveying height of instrument.

HJ = HI recorded in surveying notes. As long as HJ = 0, previous HI prevails. When HJ > 0, HI is set equal to HJ.

IBR = Bridge type identification (1 = normal, 2 = special) in input data.

IBT = Counter to indicate whether a station lies within limits of BT statements in special bridge routine.

IEB = Counter for bridge abutment stations.

IO, I1O	= Constants set equal to 0 and 10, respectively, for use in output.
I	= Do Loop parameter for total number of transfer sections, and several other do loops.
II	= Minor Do Loop parameter.
IC	= Indication that input statement will be followed by a comment statement (if IC > 0).
ISEC (I)	= I.D. number of cross-section I.
INR	= Indicator to send routine back to statement 45 to generate new NC or NH cards before or after bridge opening.
IOP	= Counter for stations inside bridge openings.
IPRI	= Instruction to print back input data if IPRI > 0.
ISCN	= Temporary section I.D. number. If an ISEC < 0 is found, indicating the start of a tributary, ISCN is set = -ISEC so that the corresponding downstream section of I.D. No. ISCN can be located.
ITR	= Indicator that a tributary joins above an upstream transfer of this section.
IX1	= Entry for field 1 in X1 card.
IX3 (I)	= Instruction to produce an X3 card for transfer section I. If IX3 = 1 produce standard X3 card. If IX3 = 2 produce X3 card as read from input tape.
IUN	= Instruction for distance conversion from miles to feet if IUN = 1, kilometres to metres if IUN = 2.
J	= Counter for cross sections. Used as subscript for section data.
K	= Counter for stations in ground section.
KK	= Counter for stations in bridge sections.
KBT	= Counter for BT card stations.
NN (I)	= Number of new roughness n values (if any) for transfer section I to be punched onto NC or NH cards. New n values may be dictated for up to 10 transfer sections per surveyed section.
NNT	= Temporary substitute for NN (I).
NQ (I)	= Number of Q values to be read at transfer section I from data and punched onto QT card.
NQT	= Temporary substitute for NQ (I).
NSTD	= Number of downstream transfer sections.
NSTU	= Number of upstream transfer sections.

NSEC	= NSTD + NSTU + 1 = total number of X1 cards to be generated for cross-section J.
PW	= Pier width.
QT (I,II)	= Flows to be entered into QT card.
RDG	= Rod reading in survey notes.
RL (I)	= Reach lengths for left bank, right bank, and channel.
RWRD	= Bridge deck rod reading.
R1, R2, R3	= Standard reach lengths used on bridge sections.
SB (I)	= Entries for SB card.
SHR	= Solid height of bridge railing.
SL (J)	= Stream slope at section J.
SS	= Side slopes of bridge abutments (for SB card).
ST	= Station distances in feet (or metres) read from survey data.
STA (K)	= ST value at station K in ground section.
STAB (KK)	= ST value at station KK in ground section.
STB, STE	= Beginning and ending stations of trapezoidal sector or bridge opening. See Figure A2.
STCH (1), STCH (2)	= Left and right channel stations, for X1 card.
STCHB (1), STCHB (2)	= Left and right channel stations for bridge section.
THETA	= Angle of section and with respect to a normal stream.
VNC (I, II), VNH (I, II)	= Entries in field II of NC or NH cards.
WBO	= Width of bridge opening (entry for SB (5)).
WL	= Weir length (entry for SB (4)).
WPR	= Total pier width (entry for SB (6)).
WS	= Water surface elevation at normal stream flow.
WSTL, WSTR	= Left and right stations to be used for weir length in special bridge routing if WL = 0 and BT cards are used.
XEB	= Indicator of whether a station is inside or outside a bridge opening.
XKB	= Floating point definition of KBT.
XNN	= Floating point definition of NN.

XQT	= Floating point definition of NQT.
XNH (I, J)	= Station in NH card.
XRIV	= River mile or kilometres from survey notes for section J.
XR (J)	= XRIV converted to feet or metres.
XXR (I)	= XR values for transfer sections.
XX8, XX9	= Entries for fields 8 and 9 in X3 card downstream from bridge.
X3 (I, II)	= Entries for X3 cards, if specified in input data.
X8, X9	= Entries for fields 8 and 9 in X1 card.
YY8, YY9	= Same as XX8, XX9, but upstream from bridge.
Y (IOP)	= Low chord elevation in bridge opening.
YLCD	= Low chord rod reading.

XSEC-PROC Listing

```
C   PROGRAM XSEC-PROC TO CREATE INPUT DECK FOR HEC-2
      DIMENSION A(12), A3(3), BT(50,3), CODE(30), DXL(10), DXR(10),
     1 DXTR(10), DYTR(10), EL(100), ELB(100), ISEC(500), IX3(10), NN(10)
     2,NQ(10), QT(10,15), RL(3), R1(3), R2(3), R3(3), SB(10), SL(500),
     3 STA(100), STAB(100), STCH(2), STCHB(2), VNC(10,5), VNH(10,20),
     4 XNH(10,20), XR(500), XW(10), XXR(500), X2(10), X3(10,9), Y(100),
     5 WS(500)
      DATA CODE  /3H   ,3HBPR,3HBSB,3HCLR,3HEBR,3HEND,3HEWS,3HEOF,3HDUM,
     12HNC,2HNH,2HOT,2HX1,2HX2,2HX3,2HGR,2HBT,2HSB,3HGRO,3HUND,3HBRI,3HD
     2GE,3H  F,3HEET,3H MI,3HLES,3H KI,3HLOM,3H   ,3H   /
C   READ PRINT,PUNCH OR TAPE INSTRUCT. AND ENGL.-METR. UNIT OPTIONS
C   IUN=0 - FEET,   IUN=1 - MILES,    IUN=2 - KILOMETERS
      READ (2,90)A(1), IPRI,IUN
      IF(IPRI.GT.0)PRINT 99
      L1 = 23 + 2*IUN
      L2 = 24 + 2*IUN
      IF(IPRI.GT.0)PRINT 89,  A(1), IPRI, IUN,CODE(L1), CODE(L2)
   89 FORMAT(' ',A1,2I4,'  CROSS SECTION LOCATION MEASURED IN',2A3)
      CALL COMMNT(IPRI)
      J = 0
      I0 = 0
      I10 = 10
   20 J = J+1
C   READ BLANK VARIABLES
      READ (2,91) A(3),K,KK,KBT,IBT,IEB,AA,ABO,WPR,RL,WBO,A3
      IF(AA.EQ.CODE(8))GO TO 61
      DO 1 I=1,10
      X2(I) = 0.
      DO 1 II=1,9
    1 X3(I,II) = 0.
C   READ SECTION NUMBER AND CHARACTERISTICS
      READ (2,92) A(4), ISEC(J), NSTD, NSTU, IBR,ITR,XRIV,STCH,THETA,IC
      IF(IPRI.GT.0)PRINT 101, A(4), ISEC(J), NSTD, NSTU, IBR, ITR, XRIV,
     1STCH,THETA
      THETA = THETA*3.1416/180.
      IF(IC.GT.0)CALL COMMNT(IPRI)
      NSEC = NSTD + NSTU + 1
C   CHECK FOR TRANSFER SECTIONS, READ DATA
      DO 2 I = 1,NSEC
      READ (2,93) A(5), NN(I), NQ(I),IX3(I), DXTR(I), DYTR(I), XW(I),
     1DXL(I), DXR(I)
      IF(IPRI.GT.0)PRINT 102,A(5), NN(I), NQ(I), IX3(I), DXTR(I), DYTR(I
     1), XW(I), DXL(I), DXR(I)
      IF(J.EQ.1)SL(J)=DYTR(1)/DXTR(1)
      IF(XW(I).EQ.0.)XW(I)=1.
      NOT = NQ(I)
      NNT = NN(I)
      IF(NNT-3)17,18,19
   18 READ (2,94) A(6),(VNC(I,II),II=1,5)
      IF(IPRI.GT.0)PRINT 103,A(6), (VNC(I,II),II=1,5)
      GO TO 17
   19 READ (2,94) A(7), (VNH(I,II),XNH(I,II), II=1,NNT)
      IF(IPRI.GT.0)PRINT 103, A(7),(VNH(I,II),XNH(I,II),II=1,NNT)
   17 IF(NOT.EQ.0)GO TO 21
```

```
      READ (2,94) A(8), (QT(I,II).II=1,NQT)
      IF(IPRI.GT.0)PRINT 104, A(8), (QT(I,II),II=1,NQT)
   21 IF(IX3(I).LT.2)GO TO 2
      READ (2,96) A(9), (X3(I,II),II=1,9)
      IF(IPRI.GT.0)PRINT 104, A(9), (X3(I,II),II=1,9)
    2 CONTINUE
C     READ BRIDGE DATA IF ANY
      IF(IBR.EQ.0) GO TO 30
      READ (2,94) A(10),BRW, PW, SHR, STCHB
      IF(IPRI.GT.0)PRINT 104, A(10),BRW, PW, SHR, STCHB
      IF(IBR.EQ.2)READ (2,94) A(11),(SB(II),II=1,3), WL,SS,WSTL,WSTR
      IF(IPRI.EQ.1.AND.IBR.EQ.2)PRINT 104, A(11),(SB(I),I=1,3), WL,SS,
     1WSTL, WSTR
C     BEGIN READING CROSS SECTION SURVEY DATA
   30 READ (2,95) ST, HJ, RDG, AA, YLCD, RWRD
      IF(IPRI.GT.0)PRINT 105, ST, HJ, RDG, AA, YLCD, RWRD
      IF(HJ.GT.0.)HI=HJ
      IF(AA.EQ.CODE(6))GO TO 41
      IF(AA.EQ.CODE(1).OR.AA.EQ.CODE(7))GO TO 40
      KK = KK+1
      STAB(KK) = ST
      ELB(KK) = HI - RDG
      DO 5 I=1,3
      R1(I) = 1.
      R2(I) = BRW
    5 R3(I) = BRW + 2.
C     BRANCHOFF FOR SPECIAL BRIDGE
      IF(IBR.EQ.2)GO TO 34
      IF(AA.EQ.CODE(3).OR.AA.EQ.CODE(4))GO TO 30
      IF(RWRD.EQ.0.)GO TO 32
      KBT = KBT+1
      BT(KBT,1) = ST
      BT(KBT,2) = HI - RWRD + SHR
      BT(KBT,3) = HI - YLCD
      IF(AA.NE.CODE(2))GO TO 30
      BT(KBT,1) = ST - PW/2.
   32 STAB(KK) = ST - PW/2.
      KK = KK+1
      STAB(KK) = STAB(KK-1)
      ELB(KK) = HI - YLCD
      KK = KK+1
      STAB(KK)= STAB(KK-1) +PW
      ELB(KK) = ELB(KK-1)
      KK = KK+1
      STAB(KK) = STAB(KK-1)
      ELB(KK) = ELB(KK-3)
      GO TO 30
   34 IF(STAB(KK).EQ.WSTL)IBT=1
      IF(WL.GT.0.)GO TO 28
      IF(IBT.EQ.0)GO TO 30
      BT2 = ELB(KK)
      BT3 = 0.
   28 IF(AA.EQ.CODE(5))IEB=IEB+1
      XEB = FLOAT(IEB)/2. - FLOAT(IEB/2)
      IF(XEB.EQ.0..AND.AA.NE.CODE(5))IOP=0
      IF(XEB.EQ.0.AND.AA.NE.CODE(5))GO TO 36
      IF(AA.EQ.CODE(2))WPR=WPR+PW
      IOP = IOP + 1
      STE = ST
      Y(IOP) = HI - YLCD
```

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      IF(RWRD.GT.0.) Y1=HI-RWRD+SHR
      IF(YLCD.EQ.0.)Y(IOP)=Y(IOP-1)
      YB = Y(IOP)
      DZ = Y(IOP)-ELB(KK)
      IF(DZ.LT.0.)DZ=0.
      IF(IOP.EQ.1)GO TO 35
      ABO = ABO + (DY+DZ)*(STE-STB)/2.
      WBO = WBO + STE - STB
   35 STB = STE
      DY = DZ
      IF(WL.GT.0.)GO TO 30
      IF(RWRD.EQ.0.)GO TO 37
      BT2 = HI - RWRD + SHR
      BT3 = HI - YLCD
   36 IF(WL.GT.0.)GO TO 30
      KBT = KBT + 1
      BT(KBT,1) = ST
      BT(KBT,2) = BT2
      BT(KBT,3) = BT3
      IF((IEB*IOP).EQ.1)XX8=BT(KBT,3)
      IF((IEB*IOP).EQ.1)YY8=BT(KBT,2)
      IF(RWRD.GT.0.)XX9 = BT(KBT,3)
      IF(RWRD.GT.0.)YY9 = BT(KBT,2)
   37 IF(STAB(KK).EQ.WSTR)IBT=0
      GO TO 30
   40 K = K+1
      STA(K) = ST
      EL(K) = HI - RDG
      IF(AA.EQ.CODE(7))WS(J)=EL(K)
      GO TO 30
C     ESTABLISH REACH LENGTHS AND NORMAL WATER SLOPES
   41 XR(J) = XRIV
      IF(IUN.EQ.1)XR(J)=XR(J)*5280.
      IF(IUN.EQ.2)XR(J)=XR(J)*1000.
      XSN  =  ABS((FLOAT(ISEC(J))/1000.-FLOAT(ISEC(J)/1000))*1000.)
      IF(J.GT.1)SL(J)=(WS(J-1)-WS(J))/(XR(J)-XR(J-1))
C     CHECK FOR INCREASING ORDER OF STATIONS
      DO 6 I=2,K
    6 IF(STA(I).LT.STA(I-1))PRINT 108,CODE(19),CODE(20),CODE(16)
      IF(IBR.EQ.0)GO TO 42
      XKB = KBT
      DO7 I=2,KK
    7 IF(STAB(I).LT.STAB(I-1))PRINT 108, CODE(21),CODE(22),CODE(16)
C     CONSTRUCT X1 CARDS
   42 IF(J.GT.1)II=J-1
      IF(ISEC(J).GT.0)GO TO 44
      ISCN = -ISEC(J)
      DO 3 I=1,J
    3 IF(ISCN.EQ.ISEC(I))II=I
      XRP = XXR(II)
   44 IF(J.GT.1)SL(J)=(WS(J)-WS(II))/(XR(J)-XR(II))
      Z9 = 0.
      I = 0
   38 I = I + 1
      XXR(J) = XR(J)+DXTR(I)
      IF(I.GT.1)GO TO 39
      IF(J.EQ.1)GO TO 43
   39 RL(3) = XXR(J) - XRP
   43 RL(1) = RL(3)+DXL(I)
      RL(2) = RL(3)+DXR(I)
```

```
      X8 = XW(I)*COS(THETA)
      IF(I.GT.1)X8 = XW(I)/XW(I-1)
      X9 = DYTR(I) - Z9
      IF(DYTR(I).EQ.0.)X9=SL(J)*DXTR(I)-Z9
      Z9 = Z9 + X9
      IF(NQ(I).EQ.0) GO TO 31
      NQT = NQ(I)
      XQT = NQ(I)
      WRITE (7,110) XQT, (QT(I,L),L=1,NQT)
   31 INR = 0
      IF(IBR.EQ.1.AND.I.EQ.(NSTD+3))GO TO 48
   45 IF(NN(I)-3)48,46,47
   46 WRITE (7,111) (VNC(I,II),II=1,5)
      GO TO 48
   47 XNN = NN(I)
      MN = NN(I)
      WRITE (7,112) XNN, (VNH(I,II),XNH(I,II),II=1,MN)
   48 IF(INR.EQ.1)GO TO 65
      IF(INR.EQ.2)GO TO 66
      IF(I.GT.1)GO TO 49
      XSN = XSN + 0.1
      IF(NSTD.EQ.0.OR.ISEC(J).LT.0)GO TO 51
      WRITE (7,113)XSN,K,STCH,RL,X8,X9
      GO TO 52
   51 WRITE(7,114)ISEC(J),K, STCH,RL,X8,X9
   52 IF(IX3(I).EQ.1)WRITE (7,115) I10
      IF(IX3(I).EQ.2)WRITE(7,116)I10,(X3(I,II),II=2,9)
      WRITE (7,117) (EL(II),STA(II),II=1,K)
      GO TO 4
   49 XSN = XSN + 0.1
      IF(I.EQ.(NSTD+1).AND.ISEC(J).GE.0)GO TO 55
      IF(I.EQ.NSEC.AND.ITR.EQ.1)GO TO 53
      WRITE (7,118) XSN,RL,X8,X9
      GO TO 54
   53 J = J+1
      ISEC(J) = ISEC(J-1) + 1
      SL(J) = SL(J-1)
      XXR(J) = XXR(J-1)
      XR(J) = XXR(J)
      WS(J) = WS(J-1) + SL(J)*(XR(J)- XR(J-1))
   55 WRITE(7,119)ISEC(J),RL,X8,X9
   54 IF(IX3(I).EQ.2)WRITE(7,116)I10,(X3(I,II),II=2,9)
      IF(I.EQ.(NSTD+1))GO TO 50
   58 IF(IX3(I).EQ.1)WRITE (7,115) I10
      GO TO 4
   50 IF(IBR-1)58,56,57
   56 XX8 = BT(1,3)
      XX9 = BT(KBT,3)
      YY8 = BT(1,2)
      YY9 = BT(KBT,2)
      IF(IX3(I).EQ.1)WRITE (7,116)I10,A3,A3,XX8,XX9
      XSN = XSN + 0.1
      X8 = COS(THETA)
      X9 = 0.
      I = I + 1
      INR = INR + 1
      GO TO 45
   65 WRITE (7,113) XSN,KK, STCHB,R1
      IF(IX3(I).EQ.1)WRITE (7,116)I10,A3,A3,XX8,XX9
      IF(IX3(I).EQ.2)WRITE(7,116)I10,(X3(I,II),II=2,9)
```

```
      WRITE (7,117) (ELB(II),STAB(II),II=1,KK)
      WRITE(7,121) XKB, ((BT(II,M),M=1,3),II=1,KBT)
      XXR(J) = XXR(J) + R1(3)
C   GOING THROUGH BRIDGE OPENING
      XSN = XSN + 0.1
      X8 = 0.
      X9 = SL(J)*BRW
      WRITE (7,118) XSN,R2,X8,X9
      X2(7) = 1.
      WRITE (7,120) X2
      IF(IX3(I).EQ.1)WRITE (7,116)I10,A3,A3,YY8,YY9
      IF(IX3(I).EQ.2)WRITE(7,116)I10,(X3(I,II),II=2,9)
      XXR(J) = XXR(J)+R2(3)
      XSN = XSN + 0.1
      X8 = 0.
      X9 = SL(J)*R3(3)
      INR = INR + 1
      I = I + 1
      GO TO 45
   66 WRITE(7,113) XSN,K,STCH,R1,X8,X9
      I = I - 1
      IF(IX3(I).EQ.1)WRITE (7,116)I10,A3,A3,YY8,YY9
      WRITE (7,117) (EL(II),STA(II),II=1,K)
      XXR(J) =XXR(J) + R1(3)
      GO TO 4
   57 IF(IX3(I).EQ.1)WRITE (7,116)I10,A3,A3,XX8,XX9
      SB(4) = WL
      SB(9) = 0.
      SB(10) = 0.
      WRITE (7,122) (SB(II),II=1,4), WBO, WPR, ABO, SS
      XSN = XSN + 0.1
      I = I + 1
      X8 = 0.
      X9 = SL(J)*R3(3)
      WRITE (7,118) XSN,R3,X8,X9
      X2(3) = 1.
      IF(WL.EQ.0.)GO TO 59
      X2(4) = YB
      X2(5) = YT
   59 WRITE (7,120) X2
      IF(IX3(I).EQ.1)WRITE (7,116)I10,A3,A3,YY8,YY9
      IF(IX3(I).EQ.2)WRITE(7,116)I10,(X3(I,II),II=2,9)
      XXR(J) =XXR(J) + R3(3)
      IF(WL.GT.0.)GO TO 4
      WRITE(7,121) XKB, ((BT(II,M),M=1,3),II=1,KBT)
    4 XRP = XXR(J)
      IF(I.EQ.NSEC)GO TO 20
      GO TO 38
   90 FORMAT(A1,I3,4I4)
   91 FORMAT(A1,5I2,10X,A3,24F2.0)
   92 FORMAT(A1,I7,4I4,4F6.0,I24)
   93 FORMAT(A1,I3,2I4,5F6.0)
   94 FORMAT(A1,F5.0,11F6.0)
   95 FORMAT(3F6.0,3X,A3,2F6.0)
   96 FORMAT(A1,F7.0,8F8.0)
   99 FORMAT('1'//'   INPUT PRINTOUT'///)
  101 FORMAT(' ',A1,5I8,4F8.2)
  102 FORMAT(' ',A1,3I8,5F8.2)
  103 FORMAT(' ',A1,10F8.3)
  104 FORMAT(' ',A1,10F8.1)
```

```
105 FORMAT(' ',3F8.1,A4,2F8.1)
108 FORMAT('  ',2A3,1X,A3,' STATIONS ARE NOT IN INCREASING ORDER')
110 FORMAT('OT',F6.0,9F8.0)
111 FORMAT('NC',F6.3,9F8.3)
112 FORMAT('NH',F6.0,F8.3,F8.0,F8.3,F8.0,F8.3,F8.0,F8.3,F8.0,F8.3)
113 FORMAT('X1',F6.1,I8,5F8.0,3F8.2)
114 FORMAT('X1',I6,I8,5F8.0,3F8.2)
115 FORMAT('X3',I6)
116 FORMAT('X3',I6,9F8.1)
117 FORMAT('GR',F6.1,F8.0,F8.1,F8.0,F8.1,F8.0,F8.1,F8.0,F8.1,F8.0)
118 FORMAT('X1',F6.1,24X, 3F8.0,2F8.2)
119 FORMAT('X1', I6, 24X, 3F8.0,2F8.2)
120 FORMAT('X2',F6.1,9F8.1)
121 FORMAT('BT',F6.1,9F8.1)
122 FORMAT('SB',F6.1,9F8.1)
 61 STOP
    END
    SUBROUTINE COMMNT(IPRI)
    DIMENSION COMM(17)
 80 READ (2,91) COMM, IC
    IF(IPRI.GT.0)PRINT 100,COMM
 91 FORMAT(17A4,I4)
100 FORMAT(' ',17A4)
    IF(IC.GT.0)GO TO 80
    RETURN
    END
```

Test Problem

4. As a demonstration of generating a HEC-2 input deck from stream cross-section data, a creek sector was hypothesized containing several regular sections as well as auxiliary or transfer sections, three bridges, a tributary, and several locations with changing n values. The creek sector was assumed to be located in minibasins 12 to 14 of subbasin 40, as shown in Figure A3.

5. Cross-section 40121 is a regular ground section with two auxiliary sections located 1000 and 500 ft downstream from the main section and one auxiliary section 400 ft upstream. The downstream section starts with three roughness values on an NC card superceded by a set of five new roughness values at the main section to be punched on an NH card.

6. Section 40124 cuts along the center line of a bridge designated for the Normal Bridge Routine. Downstream and upstream auxiliary sections are located at distances -200 and 100 ft from the main section, respectively. A new NC card should be entered at the main section, then changed back to the old NC card upstream from the bridge. X3 cards for artificial levees are called for downstream and upstream from the bridge.

7. Section 40128 is a regular ground section 200 ft downstream from the inaccessible junction with a tributary. An auxiliary section is to be placed 20 ft above the junction and labeled 40129.

8. Section 40132 is a "Special Bridge," located 1020 ft upstream from the junction. An auxiliary section is to be placed 1000 ft downstream from the main section and two more auxiliary sections are needed 100 ft upstream and downstream from the main section. Again, changes in NC cards at the bridge and X3 cards will be needed. The bridge has a weir flow over a roadbed of variable profile; therefore, the limits and profile of the weir are to be entered onto BT cards.

9. Section -40129 is a section on the tributary immediately above the junction. The surveyed section is located 70 ft upstream from the junction, and an auxiliary section is to be established 50 ft downstream from the surveyed section and is to be labeled -40129.

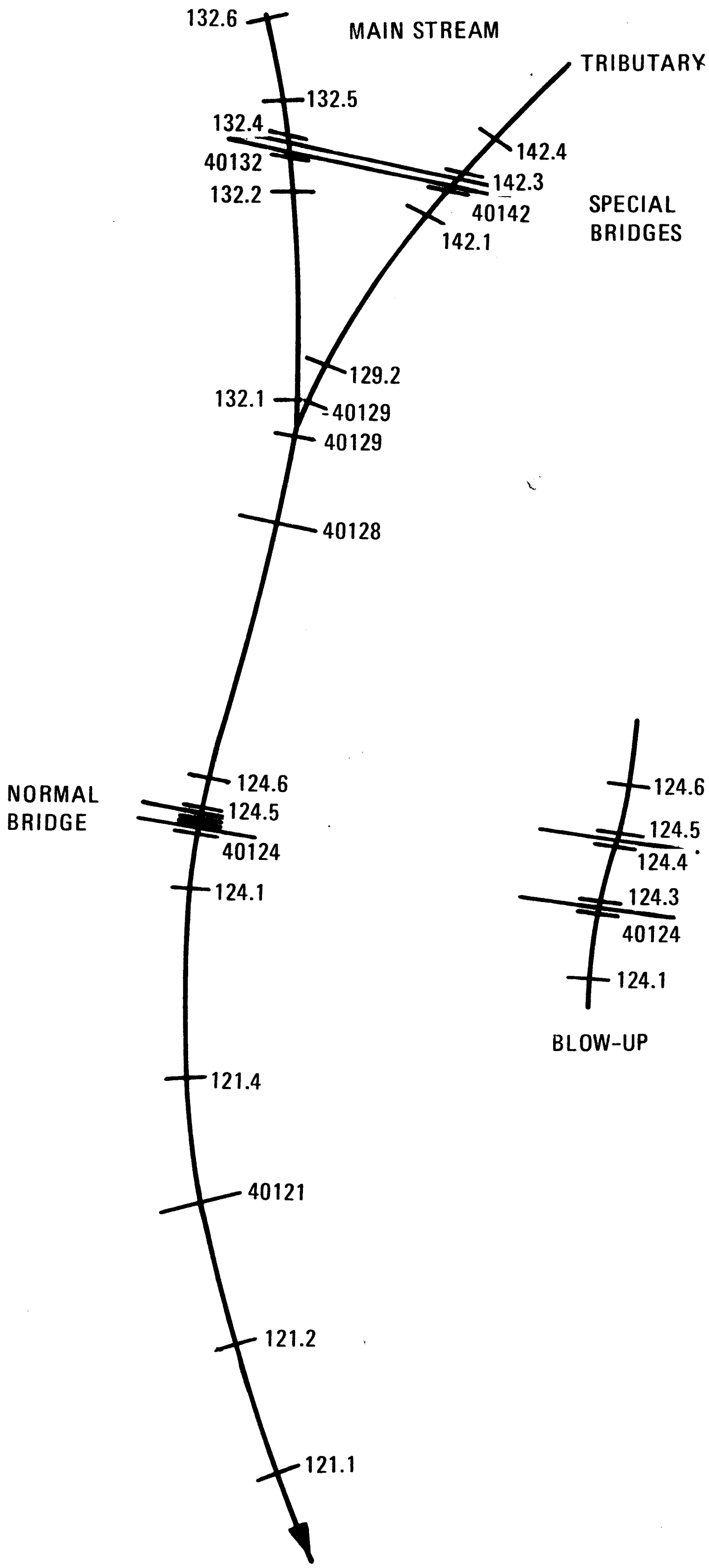


Figure A3. Test creek for XSEC-PROC demonstration

10. Section 40142 is a special bridge section with weir flow over a horizontal profile length of 300 ft. This length will have to be indicated on field 4 of the SB card, and the low chord as well as top of the road, including railing, are to be entered into fields 4 and 5 of the X2 card following the first X1 cards above the bridge. Two auxiliary sections are to be placed 100 ft downstream and upstream from the bridge.

11. Tables A1 and A2 are listings of the XSEC-PROC input and output decks. It can be seen that the output constitutes a complete HEC-2 input deck with the exception of T and J cards.

Table A1

Input to Test Problems

```
A    1  1
B    TEST CREEK WITH NORMAL AND SPECIAL BRIDGES
B    3 FLOW PROFILES WITH DIFFERENT DISCHARGES
C
D    40121      2       1               1.0     200     300     15
E    3  3               -1000   -2.2    1.5
F    0.08       0.06    0.03    0.1     0.3
H    300        600     900
E                       -500    -1.2    1.2     -100    80
E    5          1                               -50     100
G    0.2        100     0.1     200     0.03    300     0.15    400     0.18    500
E    3                  400     1.0     0.9     -100    60.
F    0.10       0.08    0.03
     0          200.0   2.4
     100                2.8
     200                2.9
     210                4.0     EWS
     220                6.8
     280        202.5   9.9
     300                4.8
     400                5.0
     500                4.0
                                END
C
D    40124      1       2  1            1.8     200     280
B    NORMAL BRIDGE
E    3                  -200                    -100    150
H    250        500     750
E               1
E    3          1
F    0.05       0.05    0.02    0.3     0.6
E    3                  100
F    0.08       0.06    0.03    0.1     0.3
J    30         2       2.5     220     280
     0          215.2   1.8
     100                2.2
     200                2.3
     210                5.2     EWS
     0                  1.2     CLR
     80                 1.2     CLR
     220                1.0     EBR     3.0     1.0
     220                6.2     BSB
     220                6.0
     230                6.4     BSB
     240                6.6     BPR
     260                6.5     BPR
     250                7.0
     270        216.2   7.5
     280                3.0
     280                5.0     BSB
     280                1.5     EBR     3.5     1.5
     400                1.2     CLR
     400                2.2
                                END
C
D    40128              1       1       2.2     220     270
E    6
G    0.1        100     0.08    200     0.06    220     0.03    260     0.05    270     0.2     400
E                       180             1.2     40      -60
     -50        216.2   2.0
     100                2.2
     200                2.4
     220                2.4
     220                2.9
     228                4.0     EWS
     238                6.2
     260                6.8
     270                2.6
     400                2.2
                                END
C
D    40132      2       3  2            2.45    150     190     20
E    3  3               -1100           1.1
F    0.06       0.05    0.03    0.10    0.30
H    120        240     360
E                       -200
E               2
I    10                                                         147     185             216.    216.
E    3          1
F    0.05       0.08    0.02    0.3     0.7
E                       100
E    3                  500             0.8
H    80         160     240
J    36         1.8     2.0     147     185
K    1.0        1.0     2.5             0.1     100     225
```

(Continued)

	0	219.0	2.2										
	0		2.0		CLR								
	100		2.5										
	100		2.2		CLR								
	140		3.2										
	147		3.0		EBR	4.5	3.0						
	150		4.0										
	155		5.0										
	155		5.0		EWS								
	160		7.0										
	148		4.8		BSB								
	160		7.0		BSB								
	166		7.2		BPR								
	184		5.4		BSB								
	180		6.0										
	185		3.5		EBR	5.0	3.5						
	190		4.0										
	225		3.6		CLR								
	300		3.4		CLR								
	300		3.4										
					END								
C													
D	-40129	1				2.25	200	240					
E	3 3		-50				50	-20					
F	0.1	0.1	0.04		0.2	0.4							
H	100	200	300										
E	0	218.0	2.0										
	200		2.0										
	205		4.0		EWS								
	210		6.0										
	230		6.0										
	240		2.0										
	400		2.0										
					END								
C													
D	40142	1	2	2		2.40	200	240					
E		1	-100										
E		2											
I	10											217.2	217.2
E	5	1											
G	0.1	200	0.03		210	0.18	230	0.02	240	0.1	400		
E			100										
J	20	1.0	1.5		200	240							
K	1.0	1.0	2.5		120	0.1							
	0	220.0	4.0										
	100		4.2										
	200		4.4										
	210		6.0		EWS								
	215		7.0										
	230		7.5										
	240		4.0										
	400		3.2										
	0		3.0		CLR								
	200		2.8		EBR	4.0	2.8						
	200		7.0		BSB								
	210		7.0		BSB								
	210		2.8		EBR								
	230		2.9		EBR	4.1	2.9						
	230		7.5		BSB								
	240		7.5		BSB								
	240		3.0		EBR	4.2	3.0						
	400		2.8		CLR								
					END								
C					EOF								

Table A2

<u>Output from Test Problem</u>

QT	3.	300.	600.	900.						
NC	0.080	0.060	0.030	0.100	0.300					
X1	121.1	9	200.	300.	0.	0.	0.	1.45	-2.20	
GR	197.6	0.	197.2	100.	197.1	200.	196.0	210.	193.2	220.
GR	192.6	280.	197.7	300.	197.5	400.	198.5	500.		
X1	121.2				400.	580.	500.	0.80	1.00	
NH	5.	0.200	100.	0.100	200.	0.030	300.	0.150	400.	0.180
NH	500.									
X1	40121				450.	600.	500.	0.83	1.20	
X3	10									
NC	0.100	0.080	0.030	0.000	0.000					
X1	121.4				300.	460.	400.	0.90	1.00	
QT	3.	250.	500.	750.						
X1	124.1	9	200.	280.	3524.	3774.	3624.	1.00	-0.66	
GR	213.4	0.	213.0	100.	212.9	200.	210.0	210.	209.2	220.
GR	208.2	250.	208.7	270.	213.2	280.	214.0	400.		
X1	40124				200.	200.	200.	1.00	0.66	
X3	10	0.0	0.0	0.0	0.0	0.0	0.0	212.2	212.7	
NC	0.050	0.050	0.020	0.300	0.600					
X1	124.3	16	220.	280.	1.	1.	1.			
X3	10	0.0	0.0	0.0	0.0	0.0	0.0	212.2	212.7	
GR	214.0	0.	214.0	80.	214.2	220.	209.0	220.	208.8	230.
GR	208.6	239.	215.2	239.	215.2	241.	208.6	241.	208.7	259.
GR	215.2	259.	215.2	261.	208.7	261.	211.2	280.	214.7	280.
GR	215.0	400.								
BT	2.0	220.0	216.7	212.2	280.0	217.2	212.7			
X1	124.4				30.	30.	30.	0.00	0.10	
X2	0.0	0.0	0.0	0.0	0.0	0.0	1.0	0.0	0.0	0.0
X3	10	0.0	0.0	0.0	0.0	0.0	0.0	216.7	217.2	
NC	0.080	0.060	0.030	0.100	0.300					
X1	124.5	9	200.	280.	1.	1.	1.	0.00	0.11	
X3	10	0.0	0.0	0.0	0.0	0.0	0.0	216.7	217.2	
GR	213.4	0.	213.0	100.	212.9	200.	210.0	210.	209.2	220.
GR	208.2	250.	208.7	270.	213.2	280.	214.0	400.		
X1	124.6				68.	68.	68.	1.00	0.33	
NH	6.	0.100	100.	0.080	200.	0.060	220.	0.030	260.	0.050
NH	270.	0.200	400.							
X1	40128	10	220.	270.	2012.	2012.	2012.	1.00	0.00	
GR	214.2	-50.	214.0	100.	213.8	200.	213.8	220.	213.3	220.
GR	212.2	228.	210.0	238.	209.4	260.	213.6	270.	214.0	400.
X1	40129				220.	120.	180.	1.20	0.19	
QT	3.	120.	240.	360.						
NC	0.060	0.050	0.030	0.100	0.300					
X1	132.1	10	150.	190.	40.	40.	40.	1.03	-1.56	
GR	216.8	0.	216.5	100.	215.8	140.	215.0	150.	214.0	155.
GR	214.0	155.	212.0	160.	213.0	180.	215.0	190.	215.6	300.
X1	132.2				900.	900.	900.	0.91	1.27	
X1	40132				200.	200.	200.	1.00	0.28	
X3	10	0.0	0.0	0.0	0.0	147.0	185.0	216.0	216.0	
SB	1.0	1.0	2.5	0.0	38.0	1.8	65.4	0.1		
X1	132.4				38.	38.	38.	0.00	0.05	
X2	0.0	0.0	1.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
X3	10	0.0	0.0	0.0	0.0	0.0	0.0	218.0	217.5	
BT	4.0	100.0	216.8	0.0	147.0	218.0	214.5	185.0	217.5	214.0
BT	225.0	215.4	0.0							
X1	132.5				62.	62.	62.	1.00	0.14	
QT	3.	80.	160.	240.						
X1	132.6				400.	400.	400.	0.80	0.57	
QT	3.	100.	200.	300.						
NC	0.100	0.100	0.040	0.200	0.400					
X1	-40129	7	200.	240.	84.	14.	34.	1.00	-0.96	
GR	216.0	0.	216.0	200.	214.0	205.	212.0	210.	212.0	230.
GR	216.0	240.	216.0	400.						
X1	129.2				50.	50.	50.	1.00	0.96	
X1	142.1	8	200.	240.	692.	692.	692.	1.00	0.00	
X3	10									
GR	216.0	0.	215.8	100.	215.6	200.	214.0	210.	213.0	215.
GR	212.5	230.	216.0	240.	216.8	400.				
X1	40142				100.	100.	100.	1.00	0.00	
X3	10	0.0	0.0	0.0	0.0	0.0	0.0	217.2	217.2	
SB	1.0	1.0	2.5	120.0	40.0	0.0	64.0	0.1		
X1	142.3				22.	22.	22.	0.00	0.00	
X2	0.0	0.0	1.0	215.8	218.5	0.0	0.0	0.0	0.0	0.0
X3	10	0.0	0.0	0.0	0.0	0.0	0.0	218.0	217.5	
X1	142.4				78.	78.	78.	1.00	0.00	

APPENDIX B: COMPUTER PROGRAM HYDPAR2 MANUAL

Background

1. Computer program HYDPAR2 is an amplification of the HYDPAR computer program developed by the Hydrologic Engineering Center (HEC). Whereas HYDPAR is essentially a bookkeeping program which computes many of the input values for HEC-1, HYDPAR2 is designed to produce, in card form, 90 to 95 percent of the input to HEC-1, in the specified sequence, as well as QT and J4 cards for the preliminary run of HEC-2.

2. HYDPAR2 is written to distinguish between subbasins and minibasins and identifies the outflow section of a minibasin by a 5-digit number. The first two digits identify the subbasin, the third and fourth digit the minibasin, and the fifth digit is reserved for cross sections within the minibasins. Subbasins and minibasins should be numbered in upstream (HEC-2) sequence, and the section number at the minibasin outlet is zero. Minibasin inlet points, with section number 9, are identified by HYDPAR2 only when this inlet is a junction of flow from two or more minibasins. For example, if the subbasin shown in Figure B1 is labeled No. 19, HYDPAR2 will compute drainage areas at the inlet and outlet points of minibasin 2, labeling the locations 19029 and 19020, respectively. Since minibasin 1 receives inflow

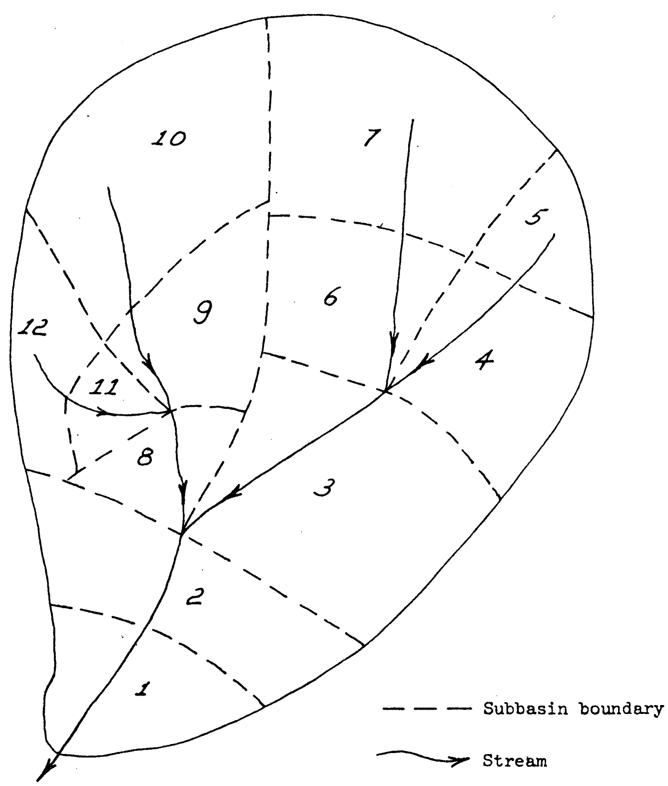


Figure B1. Example watershed with 12 subbasins only from minibasin 2, no section 19019 is required.

3. The program can handle up to 5 subbasins with up to 50 minibasins each. It will scan the master data file for grid areas belonging to any one of the designated subbasins, store all pertinent data in its memory, then proceed to process the data from each subbasin separately into printed and/or punched output, as described below.

4. HYDPAR2 can perform two routines, labeled IRD = 1 and 2, respectively. In IRD = 1, the program adds up grid points belonging to all designated minibasins and determines not only the areas of the minibasins, but also the drainage area contributing to each minibasin outlet. In Figure B1, for example, the drainage basin computed for minibasin 3 includes the areas for minibasins 3, 4, 5, 6, and 7. From these drainage areas a set of preliminary discharges are computed by the formula

$$Q_j = Cj^{1.5}(DA)^{0.7} \tag{1}$$

where

Q_j = preliminary discharges

C = an arbitrary constant to be chosen on the basis of the desired range of flows

j = 1, 2,...NSQ, a counter which produces NSQ flow rates up to a maximum of nine flows

DA = disposal area, acres

The 1.5 exponent creates a parabolically increasing set of flow rates, allowing a high upper bound. In the Wolf River Project, C was made equal to 2.0. With these values, for example, a 2000-acre drainage basin can be assigned preliminary discharges varying between 400 and 10,800 cfs roughly corresponding to 0.2 and 5 in./hr of runoff. These preliminary flows are used by HEC-2 to produce a storage-outflow table for each minibasin. In addition to the QT cards which dictate the discharges used by HEC-2, HYDPAR also produces the J4 cards which designate all minibasins for which storage outflow tables are requested.

5. In routine IRD = 2, the input variables for HEC-1 are read from instruction cards or computed and punched out in the sequence required by HEC-1. As one possible option, rainfall cards comprising a design

hyetograph can be read and multiplied by up to 10 factors to produce multiples of this hyetograph. From land use, hydrologic soil group and land slopes read in the master data file, weighted average Soil Conservation Service (SCS) curve numbers, and unit hydrograph lag times are computed.

6. The hydrologic effect of CN, namely runoff or runoff peak, is not proportional to CN. Although commonly done, the arithmetic average curve number, $\overline{CN}$, can be shown to underestimate runoff, especially for small rainfalls. A more acceptable and accurate method of averaging the CN values of grids within a runoff area (minibasin, in this case) should involve weighting on the basis of runoff effects. A routine to accomplish this was incorporated in HYDPAR2.

7. According to the SCS formulas:

$$S = \frac{1000}{CN} - 10 \qquad (2)$$

and

$$Q = \frac{(P - xS)^2}{P + (1 - x)S} \qquad (\text{If } P<xS,\ Q=0)$$

which, through an exchange of terms, becomes

$$S = \frac{(P - xS)^2}{(1 - X)\ Q} - \frac{P}{1 - x} \qquad (3)$$

where

S = storage capacity of the soil (inches).

P = cumulative rainfall since start of event (inches).

Q = cumulative runoff since start of event (inches).

x = portion of storage capacity S to be used as initial abstraction of rainfall P (SCS uses x = 0.2).

8. Procedures for computing runoff-weighted CN values are as follows. For a runoff area containing n values of CN from n grids of equal area and the approximate total rainfall P , Q is determined

for each of the n grids and an arithmetic average value, $\overline{Q}$, computed. A runoff-weighted curve number, CN*, is then determined through an iterative solution of Equation 3. The iterative process is initiated by substituting $S_1 = 1000/\overline{CN} - 10$ on the right side and solving for S_2 on the left side. If $|S_2 - S_1/S_1| < 0.01$, S_2 is accepted as the solution. If not, a new S_1 is set equal to $S_2 + S_1/2$ and solution of Equation 3 is repeated. When an acceptable S has been found, a runoff-weighted average curve number is computed, i.e., CN* = 1000/S+10.

9. A routine was added to recode the SCS erosion-slope index, listed in column 6 of the master data file into separate erosion and slope parameters for HEC input. The SCS erosion-slope index is converted to individual indices by the following code:

<u>a</u>. The first digit is the erosion index.

<u>b</u>. The last digit is the slope index, except when the erosion index is greater than 2, in which case the slope index is the last digit plus 1.

<u>c</u>. The slope index is further converted to a percent slope according to the following code:

Slope index:	1	2	3	4	5	6	7
Percent Slope:	1.0	4.0	8.0	12.5	20.0	35.0	8.0

Example:

SCS Erosion-Slope Index	Erosion Index	Slope Index	Slope Percent
201	2	1	1
302	3	3	8

<u>d</u>. The input to the HEC-1 K cards is produced, which provides the operational direction in the HEC-1 program. To obtain these K-cards in the proper sequence, the following rules should be observed:

(1) Starting with the longest streambed, number the subbasins consecutively in upstream direction (1 to 5 in Figure B1).

(2) Pick up the most upstream tributary and label the basins again in upstream order (6 to 7 in Figure B1).

(3) Repeat step 2 with the next lower tributary, numbering the basins as before (8 to 10 in Figure B1). If the tributary has subtributaries itself, continue

numbering these basins on the subtributaries (11 and 12 in Figure B1) before moving down to the next tributary of the main stream.

Input Directions

10. For run 1, the input must consist of one A1, J1, J2, and J3 card each, followed by an A2, H, and one or more K cards for each subbasin.

11. For run 2, the full sequence of input cards shown on the following pages (which include all of the run 1 input cards) must be entered.

12. Values may be entered in I or F type except where I type is specifically indicated under "Value."

<u>A1 Card</u> Title Card, Format (2X, A4, 11A6)

13. A1 may be punched (optionally) in columns 1, 2, followed by a title of alphanumeric type in columns 3-72.

<u>J1 card</u> Operational Instructions, Format (2X, FG.0, 8F8.0)

Field	Columns	Variable	Value	Description
0	1-2		J1	Card Code
1	3-8	NSUB	+	Number of subbasins (max 5)
2	9-16	NLUSE	+	Number of land uses (max 25)
3	17-24	NSQ	+	Number of storage -Q values (max 9)
4	25-32	SIZE	9.87	Grid cell size in acres
5	33-40	ILURPT	-	No printout of minibasin sizes, slopes, CN values, etc., is wanted
			0 or +	Normal printout requested
6	41-48	IP	0	No punched cards requested
			1	Punched cards from run 1 (HEC-2 QT cards) or from run 2 (HEC-1 input) requested
7	49-56	IRD	1	Run 1 requested
			2	Run 2 requested
8	57-64	LLN	0	CN values will be computed by HYDPAR

J1 Card (Continued)

Field	Columns	Variable	Value	Description
			1	CN values are read from Data Base
9	65-72	NRNG	0	No rainfall will be read
			+	No. of Hyetographs to be read
10	73-80	NXRN	0	If NRNG = 0
			+	No. of multipliers to be read in R1 card. Also No. of new hyetographs to be created

J2 Card Data Base Access, Format (2X, F6.0, 8F8.0)

14. The values in the J2 card are the fields in the Data Base under which the indicated variables can be found.

Field	Columns	Variable	Value	Description
0	1-2		J2	Card Code
1	3-8	ISD	+	Subbasin number
2	9-16	IBS	+	Minibasin number
3	17-24	ISL	+	Soil group
4	25-32	IHYO	+	Hydrologic soil group
5	33-40	ISLP	+	Slope-erosion index
6	41-48	ILD	+	Land use category
7	49-56	ICN	0	No CN values will be read from data base
			+	Data base field where CN values are listed

J3 Card Format (2X, F6.0, F8.9, 5I8)

Field	Columns	Variable	Value	Description
0	1-2		J3	Card Code
1	3-8	XQ	+	Coefficient for preliminary design flows (see Equation 1)
2	9-16	P	+	Precipitation in inches used in weighted CN averaging; recommended value = 3
3	17-24	ISUB(1)	I	First subbasin number
4	25-32	ISUB(2)	I	Second subbasin number

J3 Card (Continued)

Field	Columns	Variable	Value	Description
5-7		Repeat for total of NSUB, given in J1 card		
8	57-64	IRTIME	I	Rainfall and routing time interval in minutes
9	65-72	NRNI		No. of rainfall intervals in each hyetograph

R1 Card Only used when IRD = 2 (run 2) and NRNG > 0 (J1-9)

Field	Columns	Variable	Value	Description
0	1-2		R1	Code
1	3-8	XRN(1)	F	First Multiplier to create new hyetograph
2	9-16	XRN(2)		Second multiplier
etc. repeat		NXRN times (J1-10)		

R2 Cards Only used when R1 cards are used

Field	Columns	Variable	Value	Description
0	1		2	Card Code. (Note that only column 1 is used for the code, in order to allow the use of the same rainfall data cards as input to program HEC-1)
1	2-8	PREC(1)	F	Rainfall, in units to be used in HEC-1 (usually inches) falling during first interval.
2	7-16	PREC(2)	F	Rainfall in second interval
etc., repeat		NRNI times (J3-9)		

LU Cards Land Uses, Format (4A6, 4F8.0)

15. Only used when IRD = 2 (run 1) Repeat NLUSE times as given on J1 card.

Field	Columns	Variable	Value	Description
1-3	1-24	LU	A-N	Land use name
4	25-32	CVNM(1)	+	CN for hyd. group A
5	33-40	CVNM(2)	+	CN for hyd. group B
6	41-48	CVNM(3)	+	CN for hyd. group C
7	49-56	CVNM(4)	+	CN for hyd. group D

16. The following set of cards, A2 to K, will have to be repeated for each subbasin.

A2 Card Title Card for subbasin described same format as A1 card

H1 Card Specific Instruction for HEC-1 Input, Format (2X, I2, 9I4, 4F4.0)

Field	Columns	Variable	Value	Description
0	1-2		H1	Card Code
1	3-4	NMIN	I	Number of minibasins
2	5-8	NBR	I	Number of branches
3	9-12	NJ4	I	Number of sections for which storage-outflow cards are generated by HEC-2
4	13-16	M1	I	Entry for HEC-1 card M, field 1
5	17-20	M2	I	Entry for HEC-1 card M, field 2
				M2 = 2 if SCS hydrographs are generated
6	21-24	IQ1	I	Entry for HEC-1 card Q, field 1
			0	If M1 ≠ 2
			-	Number of nonrecording rain gages if weighting factors are constant over the subbasin
			+	Number of nonrecording rain gages if weighting factors must be read for all minibasins
7	25-28	IQ2	I	Entry for HEC-1 card Q, field 2 Same as Q1, but for recording rain gages
8	29-32	JY4	I	Entry for HEC-1 card Y, field 4
9	33-36	JY11	I	Entry for HEC-1 card Y1, field 1
10	37-40	JY17	I	Entry for HEC-1 card Y1, field 7
11	41-44	X2	F	Entry for HEC-1 card X, field 2
12	45-48	X3	F	Entry for HEC-1 card X, field 3
13	49-52	Y1	F	Entry for HEC-1 card Y, field 1
14	53-56	Y2	F	Entry for HEC-1 card Y, field 2

J4 Cards Locations for Storage-Outflow Data

17. This is the same card, or cards, which are submitted as HEC-2 input. The first three fields on the first J4 card contain data specified by HEC-2, and the following fields in a 10F8.0 format contain NJ4 section numbers, on as many cards as needed.

Storage-Outflow Cards

18. When NJ4 section numbers are specified on the J4 cards, HEC-2 will produce storages for all reaches between successive sections and outflows at the corresponding downstream sections. Together with the Y and Y1 cards, which are not related to the storages and outflows on the Y2 and Y3 cards, HEC-2 will thus cause (NJ4-1) sets of four cards to be punched out. This entire deck is entered into HYDPAR2 after the J4 cards. HYDPAR2 will then reproduce the storage-output cards, but enter zero values in field 1, and place the cards into the proper place in the HEC-1 input deck.

J5 Card(s) Only use if IQ1 < 0, indicating nonrecording gages with constant weighting factors. Format (2X, I2, F4.0, 7(I4, F4.0))

Field	Columns	Variable	Value	Description
0	1-2		J5	Card Code
1	3-4	JDNR(1)	I	I.D. No. of first nonrec. gage
2	5-8	VTNR(1)	F	Weighting factor for gage
3	9-12	JDNR(2)	I	Same as above, for second gage
4	13-16	VTNR(2)	F	

Continue for IQ1 gages.

J6 Card Only used if IQ2 < 0

19. Same as J5 card, but for recording gages.

J7 Card Only if IQ1 > 0 (Variable weighting)

Field	Columns	Variable	Value	Description
0	1-2		J7	Card Code
1	3-4	IDNR(1)	I	I.D. number of nonrec. rain gage
2-20		blank		

Unlabeled cards following J7 card

Field	Columns	Variable	Value	Description
1	1-4	WTNR(1,I)	F	Weighting factor for gage IDNR(1) applicable on minibasin 1
2	5-6	WTNR(2,1)	F	Weighting factor for some gage but minibasin 2

etc., for all minibasins, even if weighting factor is zero.

Repeat J7 and weighting factor cards for all nonrecording gages.

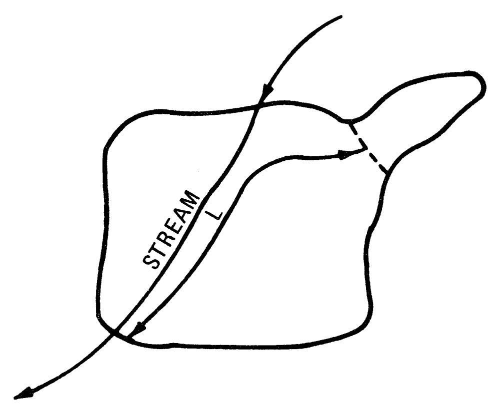
Cards J8 Only if IQ2 > 0 (Variable weighting)

20. Same as J7 card(s), but for recording gages.

L Cards Flow Path Lengths, Format (2X, P6.0, 9F8.0)

21. These cards provide the flow path lengths for all minibasins within the subbasin under consideration. These lengths are used to compute the lag time for the unit hydrograph derivation in each minibasin and should thus be estimates of the longest path from the minibasin outlet to any point on the basin except that narrow appendices of a minibasin should be ignored (see sketch).

22. For minibasins 1 to NMIN, the flow path lengths, in feet, should be entered in fields 1 to 10 (8 columns each) on as many cards as needed.



K Cards Routing Instructions to produce HEC-1 K cards, Format (20I4)

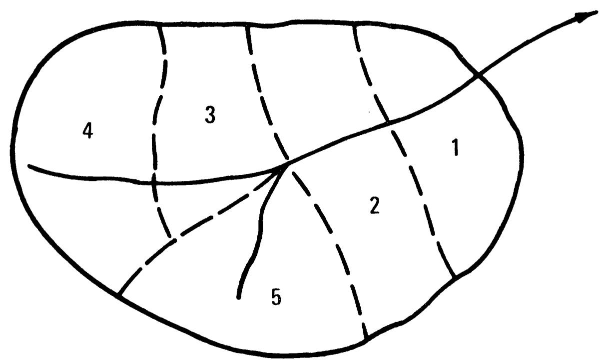
23. These cards contain minibasin numbers arranged in branch order required to generate the proper sequence of K cards for hydrograph, generation, routing, and combining in HEC-1.

24. Consider the stream with tributary in the sketch below. HEC-1 will proceed generating and routing hydrographs from minibasin 4 to the top of basin 2, then from basin 5 to the top of basin 2, combine hydrographs and continue down to the bottom of basin 1 (or top of basin 0).

25. The tree of branches is $4 \rightarrow 2$, $5 \rightarrow 2$, $2 \rightarrow 0$ which is entered in the K card as

4 2 5 2 2 0

in consecutive fields of four columns each.



APPENDIX C: COMPUTER PROGRAM RAINWEIGHT

Weighting Techniques

1. Typical conventional rainfall weighting techniques are averaging, Thiessen, and Isohyetal contours.

2. Recently, it has become popular to weight rainfall readings at neighboring stations by the inverse of the distance to any point under consideration. If we have, for example, three rainfall readings R at gages 1 to 3 and want a weighted rainfall at point P this rainfall is

$$R = \frac{\sum_{i=1}^{n} \frac{R_i}{D_i^{\,k}}}{\sum_{i=1}^{n} \frac{1}{D_i^{\,k}}} \qquad (1)$$

where D_i is the distance from point P to gage i. If there are more than three gages, often only the three closest gages are considered (Figure C1).

3. If a weighted average rainfall is wanted for an entire watershed, the watershed is divided into a grid network for at least 50 squares, as shown in Figure C2.

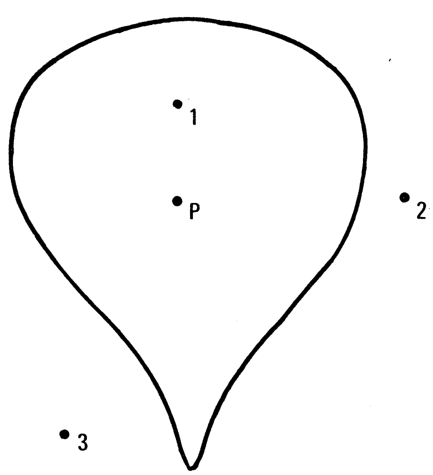


Figure C1. Watershed with three rain gages

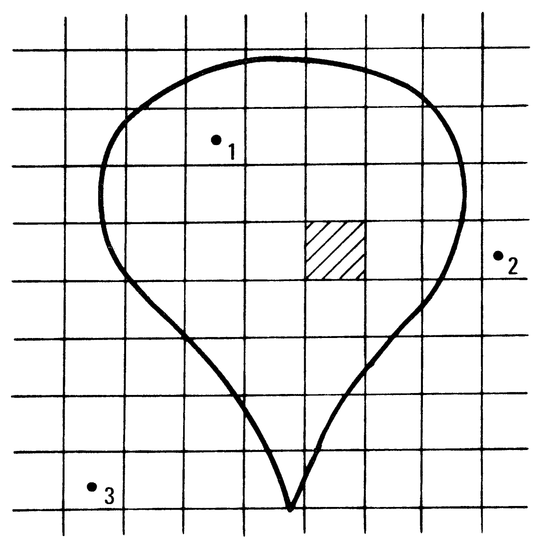


Figure C2. Watershed divided in grid network

4. Each grid area, including the boundary areas larger than one half square, is given a set of coordinates, (4,5) for example for the cross-hatched area. The rain gages are given the coordinates of the square they are located in. Now the distance from any grid area to a neighboring gage is given by $\sqrt{(\Delta X)^2 + (\Delta y)^2}$. The distance from the shaded area to gage 1, for example, is $\sqrt{2^2 + 2^2} = 2.83$.

5. The weighted rainfall for each grid space is then computed by Equation 1, and the weighted rainfall amounts for all grid spaces are averaged. Any grid space in which a gage is located is automatically weighted 100 percent in favor of that gage, and all other gages are ignored.

6. Dean and Snyder[8*] suggest using the inverse of distance squared (i.e., k = 2) as a weighting factor, but differences in resulting weighted rainfalls are usually small.

7. The procedure is obviously very tedious unless a computer program is used.

Program Description

8. For weighted rainfall computing by program RAINWEIGHT, it is only necessary to specify the number of rain gages, their coordinates, the number of grid columns, and the range of rows covered by each grid column. The input directions are as follows:

Card 1: NG, NC, IPR, XN Format (3I4, F4.1)

Card 2: X(1), Y(1), X(2), Y(2) .... X(NG), Y(NG) Format (20F4.0)

Card 3: YT(1), YB(1), YT(2), YB(2), ... YT(NC), YB(NC) Format (20F4.0)

Card 4: PR(1), PR(2) .... PR(NG) Format (20F4.1)

in which

NG = Number of gages

NC = Number of grid columns

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* See References at the end of the main text.

IPR = Printing instruction. Set IPR = 1 if a printout is wanted of the three gages nearest to any grid space.

XN = Exponent of distance to be used in weighting.

X(i), Y(i) = x and y coordinates of gage i

YT(j), YB(j) = Top and bottom row number in column 1

PR(i) = Rainfall reading at gage i

Computer Program

```
C  PROGRAM RAINWEIGHT, AVERAGING RAINFALL BY INVERSE DISTANCE WEIGHTING
C  WEIGHTED RAINFALL IS COMPUTED FOR EACH GRID SPACE FROM NEAREST N GAGES
C  AND AVERAGED.
      DIMENSION X(10), Y(10), YT(20), YB(20),PR(20),ND(10), D(10)
      READ 90, NG,NC, IPR, XN
      READ 91, (X(I),Y(I)  ,I=1,NG)
      READ 91,  (YT(J),YB(J) ,J=1,NC)
      READ 92, (PR(I),I=1,NG)
   90 FORMAT(3I4,F4.1)
   91 FORMAT(20F4.0)
   92 FORMAT(20F4.1)
      PRINT 110, NC, NG, XN
  110 FORMAT(' '////'  AREA WITH',I3,' SUBAREA COLUMNS AND',I3,' RAIN GA
     1GES, USING A DISTANCE EXPONENT =',F4.1/)
      DO 10 I=1,NC
      IYT = YT(I)
      IYB = YB(I)
   10 PRINT 111, I, IYT, IYB
  111 FORMAT(' COLUMN',I3,' RUNS FROM ROW',I3,' TO ROW',I3)
      DO 11 I=1,NG
      IX = X(I)
      IY = Y(I)
   11 PRINT 112, I, IX, IY, PR(I)
  112 FORMAT(' GAGE',I3,' IS LOCATED AT X =',I3,', Y =',I3,'    PRECIP ='
     1',F4.1)
      PRINT 115
  115 FORMAT(' '///)
      IF(IPR.GT.0)PRINT 113
  113 FORMAT(' FIELD  LOCATION    NEAREST GAGES'/'    NO.     X     Y
     1N1    N2    N3'/)
      K=0
      SP = 0.
      DO 1 J=1,NC
      K1 = YT(J)
      K2 = YB(J)
      DO 2 M=K1,K2
      K=K+1
      DO 3 N=1,NG
      NN = N
      YG = M
      XG = J
      DSQ = (YG-Y(N))**2 + (XG-X(N))**2
      IF(DSQ.EQ.0.)GO TO 20
      EX = 0.5*XN
      D(N) = DSQ**EX
      ND(N) = N
      IF(NG.EQ.1) GO TO 20
      IF(NG.LT.4)GO TO 3
      IF(N.EQ.1) GO TO 3
      DO 4 I=2,N
      IF(D(N).GE.D(I-1))GO TO 4
      ND(I-1) = ND(I-1)+1
      ND(N) = ND(N) - 1
    4 CONTINUE
```

```
    3 CONTINUE
      GO TO 21
   20 P = PR(NN)
      GO TO 2
   21 DO 5 N=1,NG
      IF(ND(N).EQ.1)N1=N
      IF(ND(N).EQ.2)N2=N
      IF(ND(N).EQ.3)N3=N
    5 CONTINUE
      IF(NG.GT.2) GO TO 22
      P = (PR(N1)/D(N1) + PR(N2)/D(N2))/(1./D(N1) + 1./D(N2))
      GO TO 2
   22 P =(PR(N1)/D(N1) + PR(N2)/D(N2) + PR(N3)/D(N3))/(1./D(N1) + 1./D(N
     12) + 1./D(N3))
      IF(IPR.GT.0)PRINT 114, K,J, M, N1, N2, N3
  114 FORMAT(6I6)
    2 SP =SP + P
    1 CONTINUE
      XK = K
      PAVE = SP/XK
      PRINT 115
      PRINT101, K,NG,PAVE
  101 FORMAT('1WEIGHTED PRECIP. ON',I3,' GRID SPACES WITH',I3,' GAGES ='
     1,F7.4,' INCHES')
      STOP
      END
//DATA.INPUT DD *
    5    9    1   1.
   1.   2.   3.   8.   4.   6.   7.   9.   8.   3.
   4.   5.   3.   6.   3.   7.   2.   8.   1.   9.   1.  10.   2.  10.   2.   9.   4.   8.
 3.0 5.0 1.0 8.0 2.0
/*
```

APPENDIX D: REFERENCE MATERIAL ON OPTIMAL ENERGY SLOPE AVERAGING AND REACH LENGTH

1. One of the major sources of error in open channel backwater computations lies in the determination of average energy gradients over relatively long reaches of river channel. The two questions the hydraulic engineer is confronted with in this regard are:

   a. What reach lengths should be used?

   b. Given the downstream and upstream energy slopes within a reach, how should these slopes be averaged to determine head losses?

2. In all likelihood, the accuracy of backwater computations increases with increasing number of cross sections or decreasing reach length. The cost-conscious engineer must keep in mind, however, that the surveying of cross sections is a fairly costly task and that it is, therefore, desirable to maximize the reach lengths in a channel without introducing excessive errors in the water surface profile computations.

3. Secondly, most engineers have at one time or another seen several formulas for averaging slopes. Most manuals propose simple arithmetic averaging of upstream and downstream energy gradients, S_u and S_d , respectively, according to the formula

$$\bar{S} = \frac{S_u + S_d}{2} \tag{1}$$

where $\bar{S}$ is the average energy gradient slope. Others advocate geometric averaging by the formula

$$\bar{S} = \sqrt{S_u \times S_d} \tag{2}$$

or the averaging of conveyances, $1.49/n\ AR^{2/3}$ which is equivalent to averaging the slopes taken to the (-1/2) power (A = cross-sectional area; R = hydraulic radius).

4. Tavener presents some analytical answers to these questions.[4*] In the first place he suggests distinguishing between M-1 and M-2 surface profiles, depending on whether the energy slope is flatter or steeper, respectively, than the channel slope. He then makes the interesting observation that the M-1 curve resembles a parabola with a vertical axis while the M-2 curve resembles a parabola with a horizontal axis. On the basis of this observation, Tavener proceeds to show that for an M-1 curve the arithmetic slope averaging equation (Equation 1) is the most correct one, while for an M-2 curve the formula

$$\bar{S} = \frac{2\,S_u S_d}{(S_u + S_d)} \tag{3}$$

should be used.

5. Through a semianalytical and experimental process Tavener then derived a formula for the maximum allowable reach length L_o, namely

$$L_o = \frac{1 - \mathbb{F}^2}{S_d\left(\frac{5T_d}{3A_d} - \frac{2B}{3P}\right)} \tag{4}$$

where

$\mathbb{F}$ = Froude Number, dimensionless

S = hydraulic gradient, ft/ft

T = top width of the wetted area, ft

A = cross-sectional wetted area, ft^2

P = wetted perimeter, ft

B = change in P with respect to depth, ft/ft

and the subscript d denotes that the above dimensions correspond to the downstream section of a reach.

6. By comparison of the results from Equation 4 with his experimental results, Tavener found that the equation slightly overestimated the allowable reach length ALOWL and suggested the design formula

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* See References at the end of the main text.

$$\text{ALOWL} = \frac{L_o}{c} \tag{5}$$

in which c equals 2.0 for M-1 curves and 1.2 for M-2 curves.

7. Equation 5 cannot be applied to channels prior to the backwater computations because all of the dimensions used in the formula depend on downstream depth. The equation is rather used as a check after each backwater step to see whether additional measured or interpolated cross sections should be inserted in the following reach.

8. It may be noted that Equation 4 is somewhat cumbersome to perform after each backwater step. While A_d , P_d, and S_d are usually listed in the backwater tabulations and the top width can be easily obtained, $\mathbb{F}_d^2$ and B_d would have to be computed separately. Furthermore, B_d , the change in P with depth, could assume unreasonably high values if the bank slope changes abruptly with depth. By testing a large range of parabolic cross sections, the value of $2B/3P$ was found to vary between 12 and 20 percent of $5T/3A$. The writer, therefore, decided to elimate the term $2/3\ B/P$ by setting $3/2\ B/P \approx 0.2$ $5/3\ T/A$ and to replace $\mathbb{F}^2$ by Q^2T/gA^3 . With these substitutions Equation 5 is changed to

$$L = \frac{1 - \dfrac{Q^2}{gA_d^2}\ \dfrac{T_d}{A_d}}{\dfrac{5}{3} \cdot c\ \bar{S}_d\ \dfrac{T_d}{A_d}} \tag{6}$$

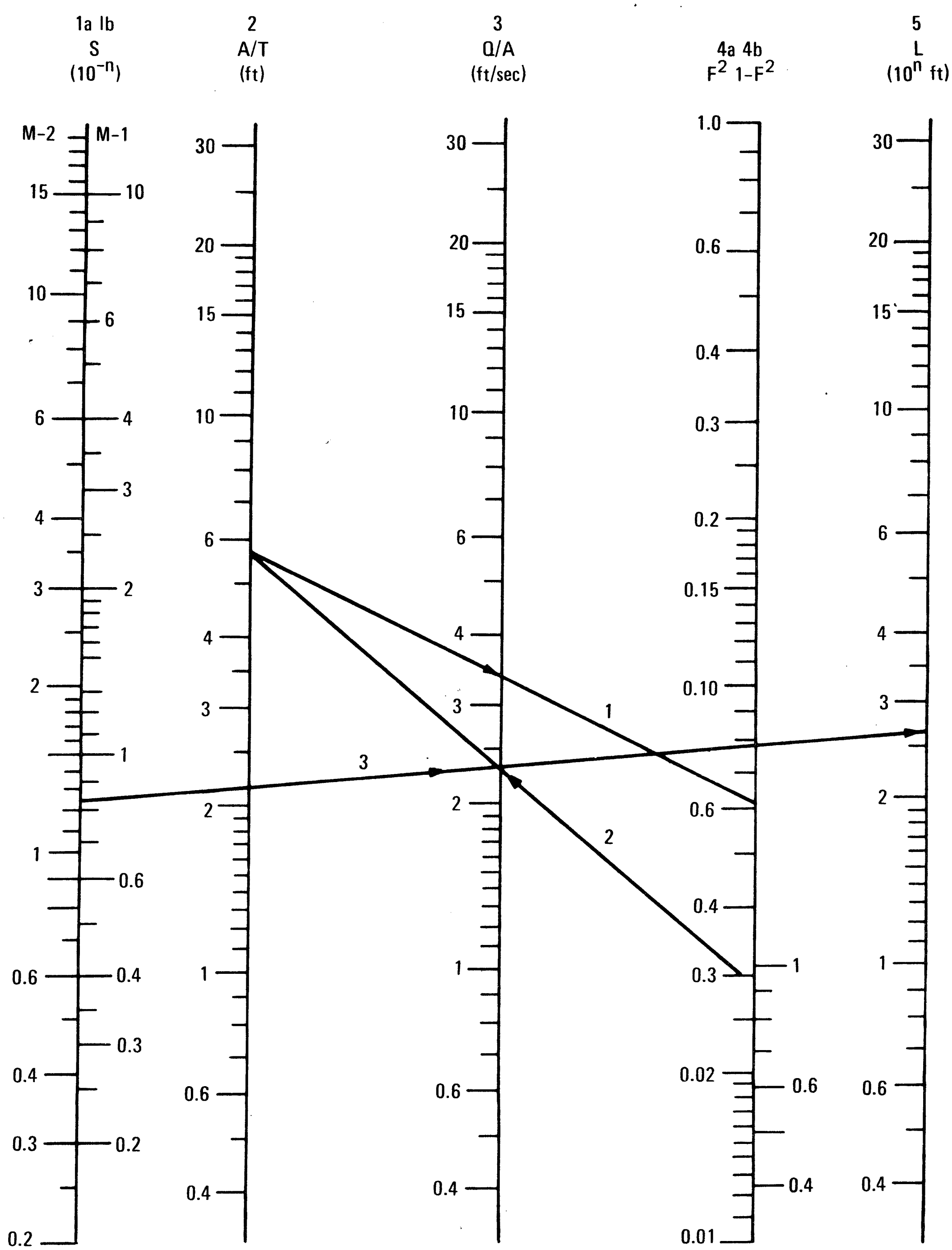
in which g is the gravitational constant ($32.2\ ft/sec^2$) and c equals 1.5 for M-1 curves and 1.0 for M-2 curves.

9. Equation 6 looks even more complicated than Equation 4, but lends itself very well to nomograph solution. The nomograph shown in Figure 1 was constructed and is used as follows:

- a. Find Q/A and A/T for the downstream section of a new reach and draw line 1 from scale 2 through scale 3 to scale 4a, where the F^2 is read.
- b. Subtract F^2 from unity and draw a line 2 from $1-F^2$ on scale 4b to A/T on scale 2.

c. Using the point P where line 2 crosses scale 2 as a pivot point, draw a line from S_d on scale 1 through point P to obtain the allowable reach length L . The scales 1a or 1b are chosen depending on whether the water profile is that of an M-1 or M-2 curve, or, in practical terms, whether S_d is flatter or steeper than the average channel bed slope over the new reach.

10. In Figure D1, the values of Q/A , A/T , and S from Tavener's example were used to determine reach length. It should be noted that the scales for S and L are labeled 10^{-n} and 10^{n} ft. This allows the length computation over a large range of S without the need for several log cycles on the S and L scales. In the example, the hydraulic gradient was 0.82×10^{-3}. If line 3 is started from the value 0.82 on the S scale, the exponent n equals 3 and $L = 2.6 \times 10^{3} = 2{,}600$ ft. If, on the other hand, S had been defined as being equal to 8.2×10^{-4} and line 3 had started from the value 8.2, L would have been read from the extended scale 5 as 0.26 and L would have been $0.26 \times 10^{4} = 2{,}600$ ft, the same distance merely read from a different scale.



EXAMPLE 1 FROM TAVENER:

Q/A = 3.4 FT/SEC, A/T = 5.7 FT, S = 0.82 X 10^{-3}, M-1 CURVE,
MAX. ALLOWABLE REACH LENGTH L = 2.4 X 10^3 = 2400 FT.

Figure D1. Nomograph for determining maximum allowable reach length[4]