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Final Report
ISU–ERI–AMES–74029
Iowa Highway Research Board Project HR-164

A COMPUTERIZED METHOD FOR THE HYDROLOGIC DESIGN OF CULVERTS

ERI Project 999-S

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Submitted to:
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IOWA STATE UNIVERSITY AMES

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PROJECT SUMMARY

Current Engineering Practice

Nationwide, about five cents of each highway construction dollar is spent on culverts. In Iowa, average annual construction costs on the interstate, primary, and federal-aid secondary systems are about \$120,000,000. Assuming the national figure applies to Iowa, about \$6,000,000 are spent on culvert construction annually. For each one percent reduction in overall culvert costs, annual construction costs would be reduced by \$60,000.

One area of potential cost reduction lies in the sizing of the culvert. Determining the flow area and hydraulic capacity is accomplished in the initial design of the culvert. The normal design sequence is accomplished in two parts. The <u>hydrologic</u> portion consists of the determination of a design discharge in cubic feet per second using one of several available methods. This discharge is then used directly in the <u>hydraulic</u> portion of the design to determine the proper type, size, and shape of culvert to be used, based on various site and design restrictions. More refined hydrologic analyses, including rainfall-runoff analysis, flood hydrograph development, and streamflow routing techniques, are not pursued in the existing design procedure used by most county and state highway engineers.

The hydraulic portion of culvert design has been thoroughly researched and published in user manuals for practicing engineers. Although the hydrologic portion of the design has also been the subject of much research, adequate answers have proven more elusive. Lacking basic hydrologic data on small watersheds, since few are actually gaged, the designer has had to use other methods of estimating peak discharges for various recurrence intervals. Two of these analytical methods (use of data and hydrologic factors analyzed from gaged watersheds, and use of rainfall-runoff relationships combined with unit hydrograph techniques) formed the basis for this study.

There is much complexity involved in developing accurate flood hydrographs within the hydrologic variability experienced in nature. This is accompanied by the tediousness of the many calculations required in such studies. These have led engineers away from using refined hydrologic design methods. However, today the digital computer offers a unique opportunity to program a sequential hydrologic design method that easily incorporates all hydrologic variables into the design process. The development and testing of such a complete model, as accomplished in this study, incorporating the refined hydrologic analysis and available hydraulic evaluation has clearly demonstrated the cost reduction potential of such a scheme in the culvert design and construction program.

The computer, however, is not the final answer to any problem. It is only a high-speed calculator with memory capability which allows the designer to use methods which previously were considered to be too time consuming to use economically in a design office. While the computer printout may look impressive, it is only a series of numbers which must be interpreted by the designer. People can be trained to input data to a computer and then record the output. However, the output must be interpreted by someone with education and experience if the final selection of a culvert size is to be made wisely and prudently. The computer can never replace engineering judgment and experience, but it can provide valuable additional information on which to base a final decision.

Purpose and Scope of the Study

The purpose of the project is to develop a comprehensive computer program which includes both current and new innovative design procedures. It should, in a single run and with a minimum of input data, allow the designer to examine several culverts of various sizes and shapes under varying conditions, in order to determine which one size, type, or combination of culverts, is best suited to a particular site. The design is accomplished in three phases: determine inflow to the ponding area upstream of the culvert site, evaluate the storage effect of changes in the temporary pond volume in the flood routing procedure, and discharge the outflow from the pond through the culvert. While the general structure of the program is such that it may be introduced anywhere, the equations used make this specific program form applicable only to the State of Iowa.

Two beneficial results can be achieved using the computer program as developed. First, because a portion of the flood volume is temporarily stored upstream of the culvert, the peak outflow discharge will be less than the peak inflow. Since the culvert needs to be designed only for that discharge which actually flows through it, a smaller culvert frequently can be used. This results in reduced construction costs, since costs are directly related to size.

Second, culverts in all areas of the state would be designed to the same risk of traffic interruption by floods. For example, assume two watersheds are similar except that one has minimal storage available upstream of the proposed culvert and the second has large storage capabilities. Assume the design criterion is to limit the allowable headwater for a particular recurrence interval to an elevation no higher than three feet below the highway grade. If the same size culvert is constructed at both locations, based on existing peak discharge criterion, the second site will have greater protection against overtopping since a flood of a larger recurrence interval is required to pond water to within three feet of the roadway. By using a smaller culvert at the second site and making use of the temporary storage capacity, the same recurrence interval flood would cause water to pond to within three feet of the highway grade at both locations. Therefore, more uniform application of any selected headwater criterion would be achieved.

A basic premise in the development of the program is that the input data be simple and minimal, with all possible calculations contained within the program. In a single run, several alternate sizes of culverts or combination of culverts (such as single or multiple pipes or boxes and/or a drop inlet) at various elevations and for various recurrence intervals and storm durations can be analyzed. For each alternative the designer may determine the reduction in peak discharge, maximum headwater depth, amount of storage used, and length of time the water exceeded any particular elevation. The results will provide the designer with useful information for evaluating the effect of a smaller culvert on headwater depth and risk of overflow. He then can decide which size and type of culvert or culverts would be best suited to a particular site.

The computerized design method includes the following steps: for a given recurrence interval, size of watershed, and location in Iowa, a rainfall is calculated for each of seven storm durations. Each storm duration has a specific rainfall distribution pattern. Each storm also is divided into several equal time increments and the volume of surface runoff from each increment is determined. These runoff increments are converted using unit hydrograph principles into individual triangular hydrographs which are then summed to yield the total inflow hydrograph. The peak discharge of the computed inflow hydrograph is compared with and constrained somewhat to the design discharge obtained from charts presently used by the Preliminary Bridge Section of the Iowa State Highway Commission (ISHC). This hydrograph is then routed mathematically through the upstream temporary storage pond and through the culvert entrance. This temporary ponding occurs as the flood discharge develops sufficient head to flow through the culvert. The hydraulic efficiency of various culvert inlet types is also included in this computerized design method.

Several assumptions are implicit in the development of the computer program. The Peak Rates of Runoff chart used by the ISHC is assumed to yield reasonable estimates of peak discharges for recurrence intervals normally used in culvert design. The rainfall-runoff relationship devised by the Soil Conservation Service (SCS) is used to describe the "losses" of rainfall due to interception, infiltration, and depression storage. Use of the SCS Method in conjunction with unit hydrograph theory and the principles of invariance, superposition, and proportionality then yield inflow hydrographs typical of those which will be experienced by the culvert during its service life.

There are also a few but definite restrictions to the use of the program. The computerized design method is applicable for drainage areas lying between 40 ac. and 16,000 ac. (25 sq mi). Below 40 ac. the minimum culvert sizes permitted by the ISHC in Iowa will generally govern. Its use is also restricted to rural areas throughout Iowa but it may be used in all types of terrain, flat as well as hilly. If used in mixed rural and urban areas, times of concentration and runoff volumes must be adjusted accordingly and good judgment exercised. It assumes also that rainfall floods produce greater peak discharges and flood runoff volumes than snowmelt events, which has been shown to be true for smaller watersheds in Iowa and the midwest in general. Because of the hydraulic equations incorporated in the program, which are based on standard culvert entrances or drop inlets having uncontrolled discharge characteristics, the design procedure should not be used where hydraulic gates or other flow-controlling devices are installed.

General Hydrologic Techniques Used in the Program

The hydrologic cycle is a continuous process which includes precipitation, infiltration, direct surface runoff, groundwater flow, evapotranspiration, and general streamflow. The rate of flow in a stream during a flood is influenced by a combination of many factors which are divided into two major groups: climatic conditions with emphasis on precipitation and the physical characteristics of the drainage basin. A study of forty-five gaged watersheds throughout Iowa indicated that rainfall was the cause of more than eighty percent of the peak annual floods on watersheds less than twenty-eight sq mi in size.

Engineers have devised several methods to estimate peak discharge rates based on rainfall and other factors. These range from simple graphical correlations to complex exponential regression equations. The SCS method of the Soil Conservation Service, US Department of Agriculture was adopted for use in this study since it included most of the variables which affect runoff. These are embodied in a runoff curve number CN. The reliability of these estimates of peak discharge was shown to be within reasonable limits based on our present knowledge.

The concept of using upstream channel and valley storage to reduce the peak discharge required for culvert design is not new. The Iowa Highway Research Board sponsored a study using this concept in 1954 by Howe and Metzler (see Ref. 14). Their results were embodied in culvert design diagrams which showed the diameter of CMP culvert with sharp-edged entrance needed in relation to watershed size, valley configuration, and amount of culvert submergence desired. A more recent study was prepared by Young, et al. for the Federal Highway Administration in 1970 (see Ref. 33). A computer program was developed to perform the necessary calculations; however, the program was restricted to box culverts only, and in addition, the hydrologic data was assumed known and simple traingular hydrographs were used. In the present study, both concrete and corrugated pipe and box culverts are included. In addition, the definition of the inflow hydrograph for various storm durations and recurrence intervals is one major feature of the computerized design method developed.

Program Development

The three phases to the design of a culvert listed previously have been incorporated in the computer program which has been named HDC, hydrologic design of culverts. Technical Paper No. 40, Rainfall Frequency Atlas of the United States (USWB-TP-40), was used as the source for rainfall amounts. The data was reduced to a series of equations for use in the program. Since rain does not fall uniformly throughout a storm, natural storms of varying durations, total rainfall amounts, and time of occurrence and amount of rainfall in individual bursts were included in the computerized design method. These descriptions of naturally occurring storms were obtained from a study made by Huff in Illinois (15) which described the time distribution of rainfall in heavy storms. The rainfall amounts obtained from USWB-TP-40 and variability of rainfall distribution within storms from the study by Huff were combined with the SCS method and unit hydrograph theory to develop the final inflow hydrograph.

The sequence of this development is as follows. The time of concentration of the watershed and then an incremental time period ΔD are calculated. The total storm duration is first made equal to one-half the time of concentration. A particular time distribution of rainfall is selected for storm duration and land use and slope factor. Rainfall for the total storm duration and then the incremental rainfall and runoff amounts for each ΔD time increment are determined. The incremental triangular hydrographs are constructed from these runoff amounts. These are summed to give the final inflow hydrograph for that storm duration. This procedure is repeated for each of seven different storm durations: the first equal to one-half the time of concentration, the second equal to the time of concentration, and the other five equal to some larger multiple of the time of concentration. These seven hydrographs are then used to subject each alternative culvert selected for study to the varying storms and volumes of runoff it undoubtedly will encounter during its service life.

The inflow hydrographs, therefore, represent flood hydrographs typical of those that will occur during the life of the culvert. No presumption is made that by inputing an experienced or observed storm of known time distribution of rainfall, the program will reproduce the observed flood hydrograph caused by the storm, because runoff depends on many factors averaged internally for statistical purposes. In addition, no presumption is made that the peak of the inflow hydrograph will exactly match the peak discharge rate used by the ISHC for culvert design although they will be similar in magnitude. The factors used in the development of the computer program have deliberately been selected such that the peak of the inflow hydrograph will normally be somewhat greater than the design discharge estimate obtained from the present ISHC method or else the latter estimate will be selected by the program internally.

Because each culvert site has unique storage characteristics, the elevation-storage relationship at the site was made an input item to the computerized design method. The amount of storage is determined from available contour maps such as US Geological Survey 7.5-minute quadrangle maps, maps prepared from aerial photographs, or contour maps prepared from field surveys. The maps used should be reasonably accurate because less detailed maps can cause a difference in computed values of the maximum headwater depth of three to four feet in examples tested. While in some cases this may not be critical (such as a roadway grade forty feet above the streambed of a deep gully having no man-made improvements subject to inundation), at many sites this three to four feet variation could mean the difference between using a larger rather than a smaller culvert.

The elevation-storage relationship is used in the routing equations in conjunction with an elevation-outflow relationship. The outflow discharge is calculated within the program for the culvert or combination of culverts being considered. A total of twelve inlet types for pipe and box culverts, drop inlets and overflow weirs have been included in the computerized design method for culverts. There are three inlet types for reinforced concrete pipes, four for corrugated metal pipes, three for box culverts, one for drop inlets, and one for overflow weirs such as water flowing down a side ditch or water overtopping the highway. The equations used for pipe and box culverts were developed by the then Bureau of Public Roads based on research studies by various groups.

The inflow hydrograph, storage, and culvert hydraulics were then combined in a flood routing routine which outputs the following data for each incremental time period: time, inflow rate, outflow rate, amount of storage used, and headwater elevation. This type of output is repeated for each of the seven inflow hydrographs for each alternate culvert being studied. A one-page summary is also output for each culvert size and type studied and includes the following information for each inflow hydrograph: storm duration, total rainfall, total runoff, maximum inflow, maximum outflow, time of maximum outflow, maximum storage used, and maximum headwater elevation.

Program Input

The input requirements have been kept to a minimum and consist of four general types: hydrologic data, stage-storage data, identification data, and hydraulic data. The hydrologic data includes the county number, recurrence interval, drainage area, land use and slope factor, frequency factor, length of main channel, difference in elevation between the watershed divide and the streambed at the culvert site, and number of storage elevations. The stage-storage data consists of a series of elevations and the total storage volumes below those elevations. The identification data includes whatever information the designer wishes to use to identify the culvert site and alternative under consideration. The hydraulic data consists of the culvert type, inlet type, headwater elevation, flowline elevation, size and number of pipes, or size and number of box culverts, or length of weir.

Examples of Program Use

Many sites throughout Iowa were analyzed during the course of the study and potential savings were found in almost all of them. The three examples discussed in detail illustrate the range of applicability of the computerized design method. The first dealt with the bridge inspection program currently underway in Iowa. A sample of eleven inadequate bridges (not capable of being rated for any kind of truck traffic) in Pottawattamie County indicated that new bridges would cost \$210,000 while culverts designed using the computerized method would cost about \$95,000. Culverts designed by the current ISHC method would cost about \$150,000. The second example showed how full use of storage available at a site near Sioux City could reduce the culvert cost from \$37,000 to \$8,300. The third example showed how the use of storage at two adjacent larger watersheds in Webster

County has the potential of reducing culvert costs from a one million dollar level to about $540,000.

Conclusions and Recommendations

A comprehensive computer program has been developed which includes both current and new innovative design procedures for the design of highway culverts. One major factor has come to light in this study. Use of the program clearly shows that the hydrologic portion of the design has a greater influence on the selection of the final culvert size than the hydraulic analysis of the culvert. For instance, reasonable use of the available storage can permit a greater reduction in culvert size then can be obtained by neglecting storage effects but selecting the most efficient culvert inlet shape. The entire hydrologic sequence (from rainfall to runoff to the complete inflow hydrograph and reservoir routing) has been included in the program and is tailored to fit each individual site. A second important conclusion reached is that only a very small temporary storage volume at the culvert site (equivalent to one- to two-tenths of an inch over the watershed) will permit using a smaller size culvert. This essentially means that detailed hydrologic study of all sites should be considered in culvert design. Also, the results show that each culvert site is unique and should be investigated on its own merits using the proposed computerized design method.

Use of the program also has shown that the mathematical equations developed in the study to estimate rainfall amounts for various storm durations and recurrence intervals are within the range of accuracy of published values contained in USWB-TP-40. In addition, the equation developed to represent the design curve of the ISHC Peak Rates of Runoff chart is acceptable as a peak discharge predictor. Results also have shown that the rainfall-runoff relationship devised by the SCS adequately describes the "losses" of rainfall due to interception, infiltration, and depression storage.

Another important finding was the effect of storm duration on the results obtained. Several methods in current use utilize standard storm durations (3 hr, 6 hr, 24 hr, and/or 10 day) for all watershed sizes. This study has shown that much shorter duration storms should be used on the smaller watersheds. In addition, several durations should be analyzed at each site because of runoff volume effects (rather than peak rates) on headwater depths when storage effects delay outflow from the system. Seven storm durations were selected for application in the hydrologic model. Since temporary storage immediately upstream of the culvert site is used in the computerized design method, the volume of runoff becomes as important as the peak discharge. Longer duration storms result in greater volumes of runoff which in turn usually result in increased headwater depths. However, the rate of increase in headwater depth decreases as the storm duration increases. Thus the maximum headwater depth tends to level off or stabilize as storm duration continues to increase.

The sensitivity of headwater depth to other parameters was also investigated. The two most important parameters were culvert size and the amount of storage available at the site. Lesser effects were caused by a change in the SCS curve number for determining runoff amounts. An increase in recurrence interval logically caused an increase in headwater depth (as rainfall and runoff amounts increase), but the amount of increase was more dependent on the other parameters. The efficiency of the culvert inlet had only a negligible effect on headwater depth for pipe culverts and a minor effect for box culverts. The time distribution of rainfall used had a large effect on peak inflow rate but a lesser effect on headwater depth, dependent again on the other parameters. Storage tended to smooth out the peak discharge variations.

Based on studies of changes in the length of main channel and difference in elevation used, the effect on headwater depth was more pronounced for various changes in the length. Therefore, the length of the main channel should be measured as accurately as possible — including the meanders in the lower portion of the watershed.

The numerous surface depressions of the pothole terrain of north central Iowa are capable of temporarily or permanently holding a volume of water equal to one-half to one inch of runoff from the entire watershed. Studies showed that a one-inch reduction in runoff caused a 25 percent reduction in headwater depth, and imply a potential to further reduce culvert sizes. However, the duration of temporary flooding may be more critical and could eventually control the culvert design. These results indicate that more studies should be done to refine our present hydrologic techniques in this pothole region of Iowa.

Loss of storage volume due to sedimentation over a period of years results in increased headwater depths. At sites which have only small volumes available, the increase is minor. However, at sites which have large storage volumes available, the increase can become important enough to influence the final size of culvert used at the site. In these cases, the storage volumes input to the program should be arbitrarily reduced at the time of design to determine the effect that a reduction in storage volume will have on headwater depth.

No computer program is ever complete. The program listed in Appendix D should be regarded as the first major step in developing an improved design method. Several possible improvements to increase the flexibility of the computerized design method are noted below but a possible adverse effect should be noted. As the flexibility is increased, the input requirements usually become more complex. A designer normally would rather be designing than filling out input forms. Therefore, he may not use the program as much if the input forms become too cumbersome. One simple solution is to have two forms of the program, the basic and the more flexible, and then let the designer choose which one he wants to use.

Improvements which could be added to increase the program's flexibility are the following: permit input of a different SCS curve number than the average value currently selected internally within the program for a specific county location; allow the input of a known inflow hydrograph such as an observed one obtained during a recorded flood event; permit arbitrary selection of a specific time distribution of rainfall; input an outflow stage-discharge relation rather than have the program calculate it; include outlet control equations for culverts and sequentially test whether inlet or outlet control governs; and in addition to the hydrologic analysis, calculate and output the design water surface profile through the culvert.

In addition, the program should be used for a period of time and then have the users make recommendations for other use options and output items that would be useful to them. Initial use by the ISHC and the county engineers in Iowa is being planned.

The results confirm the hydrologic routing concepts stating that the outflow discharge is less than the maximum inflow discharge. As this reduced discharge flows downstream, the next downstream structure may also be somewhat reduced in size — depending on additional inflows and use of ponding at the downstream sites. Also, inflow to the site under consideration might also be reduced due to existing structures upstream. This possibility deserves investigation to ascertain if it could be added to the proposed design method.

Iowa's land and water are two of its most valuable resources. The conservation of these two resources is of concern to all. The National Environmental Policy Act (NEPA) of 1969 has given the public a powerful tool to enhance highway planning and design. The use of the proposed computerized design method could have several beneficial effects: reduction in culvert cost, emphasis on the creation of permanent farm ponds upstream of road embankments (which are both economically and esthetically pleasing), and reduction in soil loss through erosion.

One question which should also be investigated and answered is whether or not the volume taken up by the prism of water flowing in the channel should be subtracted from the temporary pond storage used in the flood routing procedure. In deep confined gullies, this may be an important factor.

The ISHC should be encouraged to set up a procedure so that the county engineers and consulting engineers who do work for the county or state may use the program and assist them in its use. The potential savings in culvert construction cost through the use of the proposed computerized design method are sufficiently large that the suggestion can be made to increase the number of personnel in the preliminary bridge section of the ISHC to take full advantage of the possibilities offered by the use of this new method. The additional time and effort required in performing detailed hydrologic analyses is more than offset by the potential cost savings in reduced culvert costs.

As mentioned before, only a very small temporary storage volume at the culvert site will permit using a smaller size culvert. This storage volume is determined in the following manner. First, determine the "maximum allowable" headwater depth at the site. In many cases this could be 20 ft or more. Site conditions and the judgment of the engineer will determine the "maximum allowable" depth. Second, determine the total volume of storage below this depth to the culvert invert. Third, convert this storage volume from acre-feet to inches as follows: multiply the storage in acre-feet by twelve to obtain acre-inches and then divide this result by the drainage area of the watershed in acres to obtain the temporary storage volume in inches over the watershed. If the answer is greater than one- or two-tenths of an inch, the computerized design method proposed in this study should be used because a smaller culvert size could be achieved at the site. The amount of storage available below the "maximum allowable" headwater depth also gives some indication of the amount of reduction in culvert size that can be accomplished. The larger the storage volume in inches over the watershed, the smaller the culvert can be made.

REVIEW OF GENERAL HYDROLOGIC TECHNIQUES

Factors Affecting Runoff

The hydrologic cycle is a continuous process which includes precipitation, infiltration, direct surface runoff, groundwater flow, evapotranspiration, and general streamflow. The culvert designer's primary interest in the hydrologic cycle is with direct surface runoff during flood periods and is concerned with the other portions only to the extent that they affect this direct surface runoff. The effect of these and other factors on peak rates of runoff are well described in the following excerpt from Wisler and Brater (31).

> The flow in any stream is determined by two entirely different sets of factors, the one depending upon the climate with special reference to the precipitation, and the other upon the physical characteristics of the drainage basin. The influence of the first group depends upon:
>
> 1. Type of precipitation
> 2. Rainfall intensity
> 3. Duration of rainfall
> 4. Distribution of rainfall on basin
> 5. Direction of storm movement
> 6. Antecedent precipitation and soil moisture
> 7. Other climatic conditions which affect evaporation and transpiration.
>
> The effect of the second group is determined by the following characteristics of the drainage basin:
>
> 1. Land use
> 2. Type of soil
> 3. Area

4. Shape

5. Elevation

6. Slope

7. Orientation

8. Type of drainage net

9. Extent of indirect drainage

10. Artificial drainage.

> Anyone seeking a simple and convenient equation for determining the maximum flood flow, the minimum flow, or the average flow of a stream will see the difficulty of such a procedure when he realizes that any such equation has to be expressed in terms of all the above variables, and that almost any of the factors may affect the result by one hundred percent or more. Furthermore, if the flow is expressed in terms of only one variable, the result may easily be in error by over a thousand percent. From this it follows that a trustworthy appraisal of any of the several characteristics of streamflow must be based upon a careful consideration of the influence of all the foregoing factors and cannot possibly be determined by the use of a simple equation involving only one, or at best, two or three of those variables.

To add complexity, some of the above variables can be subdivided into categories, these categories can be divided into subsets, and these subsets can be further divided. For instance, land use can be divided into categories of urban and rural land use. Rural land use can be divided into subsets of woodland, pasture, cropland, and farmsteads. Cropland can be further subdivided into several types of crops: row, field, and orchard. Rowcrops can be further subdivided into kinds of rowcrops. Each of these sub, sub, sub, subsets must then be related to the other variables such as rainfall intensity. In a watershed only a few hundred acres in size, the number of possible conditions occurring at the same time can become astoundingly large.

Type of Precipitation

The first variable listed in the previous section on factors affecting runoff was the type of precipitation. For purposes of this study, all precipitation is assumed to be in the form of rain. Flooding from snowmelt is widespread throughout Iowa. However, on the smaller watersheds with which this study is concerned, recorded peak floods are predominately caused by thunderstorms. Time of occurrence of recorded peak annual floods for 45 watersheds, 28 sq mi or less in size, is shown in Table 1. These 45 streamflow gaging stations, 9 recording and 36 crest-stage, have a combined total of 897 station-years of record through the 1972 water year. The data were obtained from records published by the US Geological Survey (30). The discrepancy between total number of occurrences, 783, and the 897 station years of record is accounted for by two factors: sometimes peak floods did not reach the bottom of the gage, so no peak flow was recorded; in other years, no date of occurrence was listed.

Of 783 occurrences, 555 peaks, or 71 percent, were recorded, May through October. Three months, May through July, account for 54 percent. For November through April, no attempt was made to separate which of the remaining 228 peak floods were caused by snowmelt, rainfall, or a combination. If this had been done, the percentage of annual peaks caused by rainfall might have increased to 80 or 90 percent. By adding April and November to the May through October period, the number of annual peaks increases to 623, or 80 percent of the total number of occurrences.

Table 1. Month of occurrence of peak annual floods

Month	Number of occurrences	Month	Number of occurrences
January	12	July	136
February	46	August	78
March	101	September	41
April	54	October	17
May	105	November	14
June	178	December	1

Discharge Relationships Obtained from Gaged Basins

Formulas for the determination of peak discharge rates based on statistical analyses of stream gaging station records have the distinct advantage of including many if not all the variables affecting runoff. A gaging station record includes the hydrograph of each individual storm runoff event. This record of the time rate of runoff is the integrated effect of each variable as it affected each part of the watershed prior to, during, and after each storm. A few of these studies will be commented on later. This section is devoted to a review of the method currently used by the Iowa State Highway Commission (ISHC). It is a combination of empirical and statistical methods, based on a Bureau of Public Roads design procedure published in 1951, known as the BPR Method (2).

The BPR Method is based on work published by Potter in 1950 (22). His study included a statistical analysis of runoff records from experimental drainage basins established by the Soil Conservation Service (SCS). These watersheds are small agricultural basins of less than 1000 ac. with different types of land use in the humid region of the United States, including some in Iowa. Peak rates were plotted against drainage area on log-log paper. Using this curve in conjunction with the probability curves developed at each station yielded the peak rate for any size watershed for any desired recurrence interval. The design peak discharge for a given watershed is computed as the product of four factors: the rainfall factor RF, the land use and shape factor LF, the frequency factor FF, and the peak rate of runoff Q_c for mixed cover in humid regions with a frequency of 25 years and rainfall factor of unity. This relationship is shown as Eq. (1).

$$Q_d = RF \times LF \times FF \times Q_c \tag{1}$$

where Q_d = design discharge in cfs

RF = rainfall factor

LF = land use and slope factor

FF = frequency factor

Q_c = discharge from chart developed by Potter in cfs.

The ISHC has adapted this method to Iowa's conditions and experience using appropriate modifications. The same equation is used except that the rainfall factor is dropped, as it is assumed to be unity for the entire state. The matrix for land use and slope factors has been expanded with the slope as defined below. The frequency matrix and curve have been adjusted to reflect a 50-yr recurrence interval used for the design of culverts on the interstate and primary road systems. A frequency factor of 1.2 is used in Iowa for the 100-yr recurrence interval. The curve has been extended to 10,000 ac. and adjusted downward at the lower end to cover the range of drainage areas used in culvert design. Use of the curve beyond 10,000 ac. is suggested only as a check on other methods. These changes are reflected in Fig. 1.

While the land use categories in Fig. 1 are self-explanatory, the land slope requires the use of some judgment. The following descriptions have been provided by the Preliminary Bridge Section of the ISHC and are intended to assist the designer in making these judgments.

Very hilly land is best typified by the bluffs bordering the Mississippi and the Missouri Rivers. This terrain is practically mountainous in character. Small areas of very hilly land can be found in all parts of the state. Typically, they can be found near the edge of the flood plains of the major rivers.

Hilly land is best typified by the rolling hills of south central Iowa. Interstate 35 in Clark and Warren Counties traverses many hilly watersheds. Small areas of hilly land can be found in all areas of the state.

Rolling land is best typified by the more gently rolling farm lands of central Iowa. Interstate 80 in Cass and Adair Counties

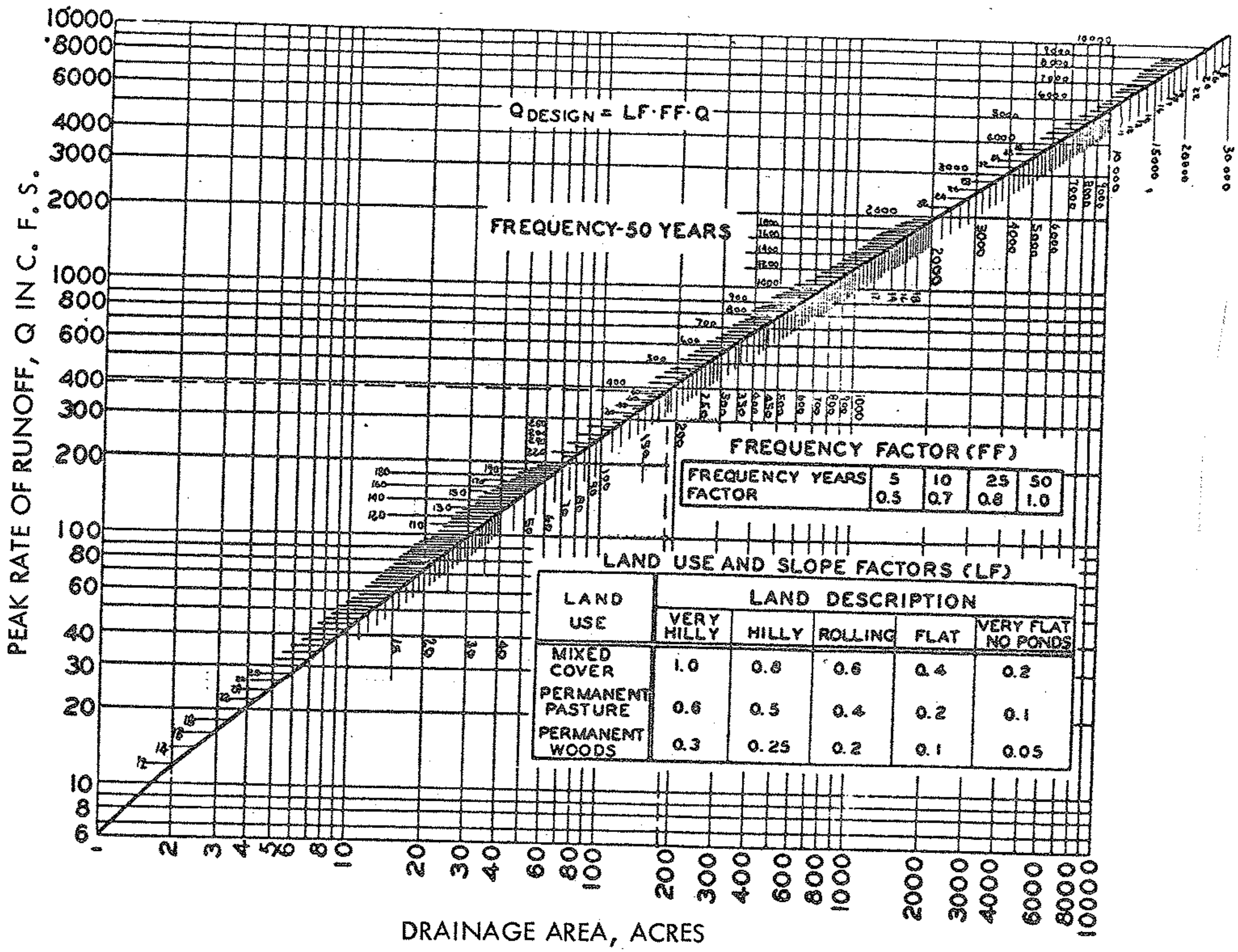

Fig. 1. Peak rates of runoff (ISHC).

traverses many rolling watersheds. Small areas of rolling land also can be found in all parts of the state.

Flat land is best typified by the farm lands of the north central part of the state. US Highway #69 traverses many flat watersheds in Hamilton and Wright Counties. Small areas of flat land can be found in all areas of the state.

Very flat land is best typified by the flood plain of the Missouri River flood plain near the western border of the state. Interstate 29 is located on this type of land for most of its length. Small areas of very flat land also can be found in all parts of the state.

The above descriptions are typical terrain features for the various regions of Iowa. The user should be aware, though, that a small watershed of any land use and any land slope will be found in any and all parts of Iowa.

Rainfall-Runoff Relationships

For culvert-sized drainage areas, streamflow records seldom exist or may be available only for short periods and/or at widely scattered locations. Engineers have devised methods to correlate the more plentiful rainfall records with scarce or incomplete runoff records. The correlations are based on the components of the hydrologic cycle. Both flood runoff volumes and flood hydrograph characteristics have been studied in great detail. Flood volumes of direct surface runoff are computed by the relationship stated in Eq. (2).

$$Q_v = P - L \tag{2}$$

where Q_v = runoff volume expressed in inches depth

P = precipitation in inches

L = losses expressed in inches.

Losses include interception, depression storage, infiltration, and evaporation. Thus, if rainfall is known and losses can be estimated, the amount of runoff can be determined. In the following examples of graphical correlation, the losses are either implicit or are expressed by various parameters.

In the simplest correlation, rainfall is plotted against runoff. There usually is much scatter but a definite trend can be observed as shown in Fig. 2. To reduce scatter and improve the correlation, a third variable can be introduced. This could be season of the year, relative condition of the soil, groundwater flow, number of days to last significant rain, or the antecedent-precipitation index (API). With the coaxial method of graphical correlation, a number of independent factors as well as the dependent variable are included (20). For instance, the dependent variable could be storm runoff with the independent variables being the API, week of the year, amount of precipitation, and/or storm duration. More complex statistical methods using regression analysis also can be employed (11).

An even more comprehensive rainfall-runoff relationship has been developed by the SCS for rural areas. This method was selected for the present study because the SCS Method embodies most of the 17 major factors affecting runoff which were listed previously (8, 25). The SCS rainfall-runoff relationship is defined by Eq. (3).

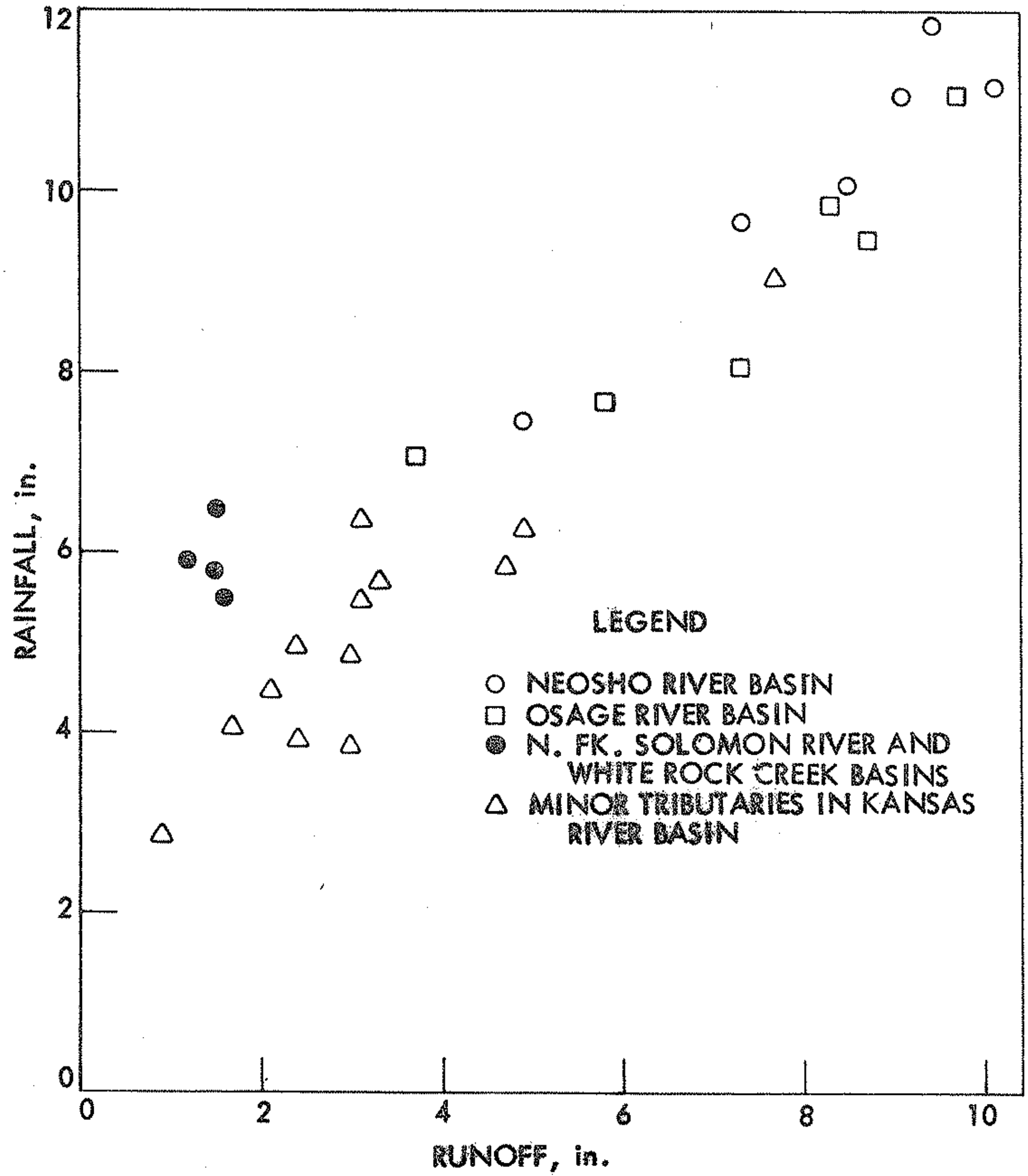

Fig. 2. Relation between rainfall of July 9-13, 1951, and corresponding runoff at selected gaging stations in Kansas.

$$Q_v = \frac{(P - 0.2S)^2}{P + 0.8S} \tag{3}$$

where Q_v = actual runoff amount expressed in inches depth ($Q_v \leq P - I_a$)

P = rainfall in inches

S = maximum potential retention, including the initial abstraction, in inches

I_a = initial abstraction of rainfall before runoff begins in inches = 0.2S in SCS analysis.

Equation (3) is developed from the following conceptual relationship, all of whose parameters are expressed in inches.

$$\frac{F}{S} = \frac{Q_v}{P - I_a} \tag{4}$$

where F = actual retention including infiltration ($F \leq S$)

P = maximum potential runoff

S, Q_v, and I_a as defined above.

The maximum potential runoff in any storm is the amount of precipitation P (assuming $I_a = 0$). The retention S has a specific value for any particular storm; it is the maximum that can occur under the existing conditions. It could be very high in a dry porous soil having little soil moisture or it could be very low for a saturated clay loam with all voids full and having little permeability. The actual retention F varies in a similar manner because it is the difference between $P - I_a$ and Q_v at any point on the mass curve. Substituting this relation for F, Eq. (4) can be rewritten as

$$\frac{(P - I_a) - Q_v}{S} = \frac{Q_v}{P - I_a} \tag{5}$$

Solving Eq. (5) for Q_v produces Eq. (6).

$$Q_v = \frac{(P - I_a)^2}{(P - I_a) + S} \tag{6}$$

Based on studies of rainfall and runoff data from small experimental watersheds, the SCS has developed the following empirical relationship between I_a and S.

$$I_a = 0.2S \tag{7}$$

Substituting this relationship into Eq. (6) yields Eq. (3).

This I_a and S relationship states that 20 percent of the maximum potential retention S is the initial abstraction I_a which is the interception, depression storage, and infiltration occurring before runoff begins. Thus, S is a function of soil-water storage and the infiltration rates of a watershed which in turn are functions of soil types, types and conditions of cover in the watershed, and the antecedent moisture conditions.

These four factors (type of soil, type of cover, condition of cover, and antecedent moisture condition) are included in a curve number CN which is calculated for each watershed. The curve numbers range from 0 to 100 and are a measure of runoff potential. A curve number of 100 means all rainfall appears as runoff. The relationship between curve number CN and maximum potential retention S is

$$CN = \frac{1000}{S + 10} \tag{8}$$

or

$$S = \frac{1000}{CN} - 10 \tag{9}$$

Thus, if CN = 100, S = 0, and Q_v = P. Likewise, as CN approaches 0, S approaches infinity, and Q_v approaches 0. These curve numbers for specific soil and cover conditions were developed by classifying over 4000 soil types into four broad hydrologic soil groups, assuming soil surfaces were bare, maximum swelling had taken place, and rainfall rates exceeded surface intake rates. Each soil grouping indicates the runoff potential of a soil based on the following parameter: the minimum rate of infiltration obtained for a bare soil after prolonged wetting. The definitions for these four soil groups are as follows (25):

A. (Low runoff potential.) Soils having high infiltration rates even when thoroughly wetted and consisting chiefly of deep, well to excessively drained sands or gravels. These soils have a high rate of water transmission.

B. Soils having moderate infiltration rates when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.

C. Soils having slow infiltration rates when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine texture. These soils have a slow rate of water transmission.

D. (High runoff potential.) Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very slow rate of water transmission.

Cover conditions were evaluated by forming several classes: land use, land treatment, and hydrologic condition. Types of land use and land treatment were classified on a flood runoff-producing basis.

Land use is the watershed cover and it includes every kind of vegetation, litter, mulch, and fallow as well as nonagricultural uses such as water and impervious surfaces. Land treatment applies mainly to agricultural land uses and includes mechanical practices, such as contouring and terracing, and management practices, such as grazing control or rotation of crops.

The assignment of curve numbers to hydrologic soil-cover complexes was accomplished as follows. The data literature was searched for watersheds in single complexes (one soil group and one cover). An average curve number for each watershed was obtained using the rainfall-runoff data for the storms which produced the annual floods. These watersheds were generally less than one square mile in size, the storms were of one day or less in duration, and the number of watersheds for a particular complex varied. The data included antecedent precipitation for the 5- and 30-day period preceding the occurrence of the annual flood.

Because of the difficulties of determining antecedent moisture conditions (AMC) from data normally available, the conditions are reduced to the following three cases (25). The total 5-day antecedent rainfall during the growing season for the three moisture conditions is as follows: less than 1.4 in. for AMC-I, between 1.4 in. and 2.1 in. for AMC-II, and over 2.1 in. for AMC-III.

AMC-I. A condition of watershed soils where the soils are dry but not to the wilting point, and when satisfactory plowing or cultivation takes place. (This condition is not considered applicable to the design flood computation methods presented in this text.)

AMC-II. The average case for annual floods, that is, an average of the conditions which have preceded the occurrence of the maximum annual floods on numerous watersheds.

AMC-III. When heavy rainfall or light rainfall and low temperatures have occurred during the 5 days previous to the given storm, and the soil is nearly saturated.

Based on the curve numbers applicable to Iowa, the SCS has developed the generalized curve numbers shown in Fig. 3. The computation sheet used by the SCS is included as Appendix A. The average curve numbers shown in Fig. 3 correspond to watersheds with mixed cover, the condition used for design by the ISHC. Mixed cover is defined as a watershed which includes row crops, pasture, woods, farm buildings, and roads.

Relationship between Runoff and Peak Discharge

Rainfall is usually expressed in inches. A six-inch rain is construed to be an average depth of water on the ground surface equal to six inches. A six-inch rain over a watershed can also be interpreted as a volume — assumed to be or calculated as the average depth of rainfall over a defined watershed. Rainfall can be interpreted as a volume when it is associated with a given watershed size. In like manner, runoff can also be interpreted as a volume: a three-inch runoff from a one square mile watershed.

The time distribution of runoff, or graph of discharge against time, is called a hydrograph. It represents the time rate of runoff at a designated point on the stream in a watershed. A simplified hydrograph takes the form of a triangle, with the peak of the triangle being the peak rate of discharge. The area of the triangle,

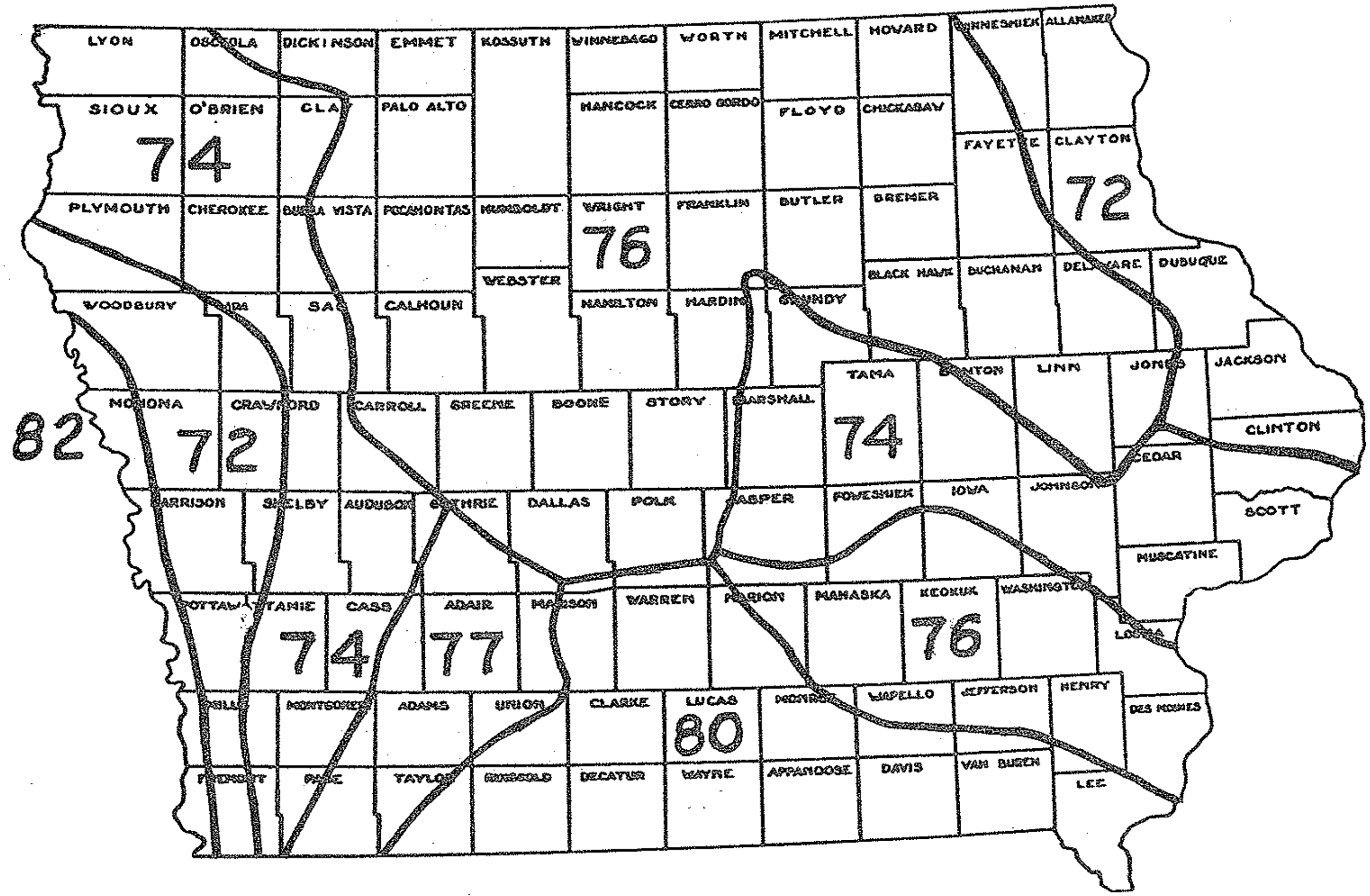

Fig. 3. Generalized estimate of average runoff curve numbers in Iowa (SCS).

i.e., the area under the hydrograph, is equal to the volume of runoff from the storm. Through analysis of the flood hydrograph, the volume of runoff in inches can be related to the peak discharge in cfs.

CURRENT DESIGN PROCEDURE ALTERNATIVES

Peak Discharge Estimates

Highway culverts are normally designed using a peak discharge rate which is associated with a selected recurrence interval. Several decades of research have yielded numerous methods relating peak discharge to watershed and storm characteristics; however, the accuracy of these relationships is still being questioned. No one method gives complete and adequate results; all answers must still be regarded as estimates. Some of these methods are reviewed briefly in the following sections. A thorough discussion of several peak discharge formulas developed over the years is contained in a study by Chow (7).

ISHC Method

A modification of the BPR Method (2), selected by the ISHC, was discussed in a previous section and shown in Fig. 1. The equation for the design discharge is given as Eq. (10).

$$Q_d = FF \times LF \times Q_c \tag{10}$$

where Q_d = design discharge in cfs

FF = frequency factor

LF = land use and slope factor

Q_c = discharge in cfs from chart in Fig. 1.

USGS Method

Another method which can be used in Iowa is based on a statistical analysis of Iowa streamflow records by the US Geological Survey (USGS) in 1966 and known as IHRB Bulletin 28 (24). A combination of the multiple correlation and index flood methods was used to derive two regression equations. The equations were developed for use on drainage areas from 1 to 15,000 sq mi. Experience in Iowa indicates that these equations yield low estimates of discharge on culvert-sized watersheds. Bulletin 28 has recently been updated by the USGS and the Iowa Natural Resources Council as INRC Bulletin No. 11 (18). It is also based on a statistical analysis of streamflow records using the log-Pearson Type III distribution and multiple correlation techniques. No experience has yet been gained in the use of discharge estimates based on these equations. The difficulty lies in the paucity of gage data on small drainage areas; the standard error of estimate using the INRC bulletin is about 30$\pm$ percent.

SCS Method

Another method used in Iowa (and the one adopted for use in the computer program developed in this study) is the method devised by the SCS (25). The peak discharge estimate comes from the hydrograph analysis shown in Fig. 4. The volume of flood runoff equals the area of the triangular hydrograph or one-half the altitude times the base or

$$Q_v = \frac{q_i \times T_p}{2} + \frac{q_i \times T_r}{2}$$

from which

$$q_i = \frac{2Q_v}{T_p + T_r} \qquad (11)$$

where q_i = peak discharge in inches per hour

Q_v = storm runoff in inches

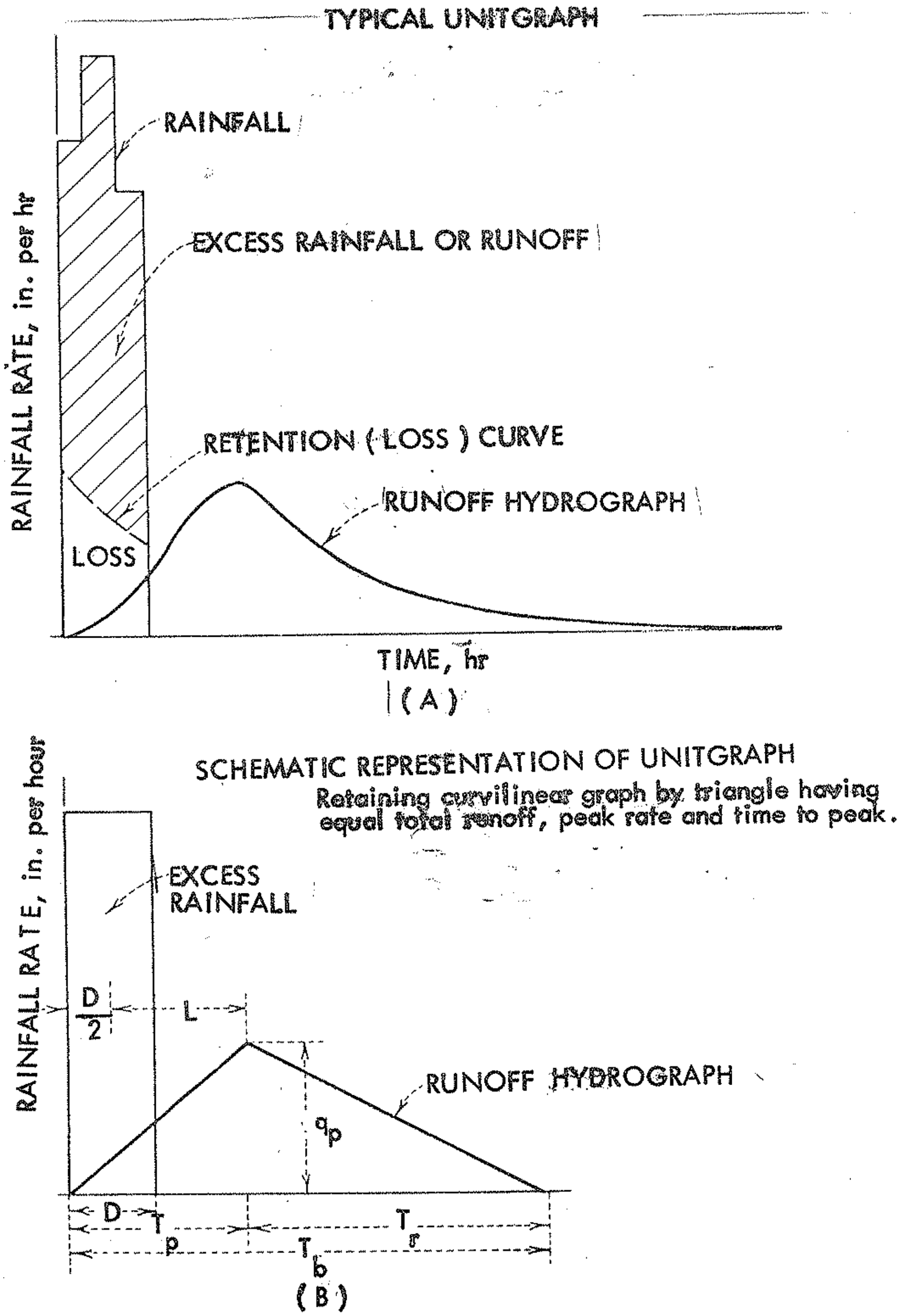

Fig. 4. Triangular hydrograph analysis.

Paper by Victor Mockus, Hydraulic Engineer, Soil Conservation Service, Central Technical Unit, Maryland, 1957.

T_p = time to peak in hours

T_r = time of recession in hours.

Let $T_r = H \times T_p$, where H is a constant to be determined for a particular watershed.

$$q_i = \frac{2Q_v}{T_p + (H \times T_p)}$$

or

$$q_i = \frac{2}{1 + H} \times \frac{Q_v}{T_p} \tag{12}$$

Next, convert inches per hour to cubic feet per second and introduce the drainage area A in square miles. One inch per hour is equivalent to 645.3 cfs per sq mi.

$$q_p = q_i \times 645.3 \times A$$

or

$$q_p = \frac{KAQ_v}{T_p} \tag{13}$$

where q_p = peak discharge in cfs

$$K = \frac{1290.6}{1 + H} . \tag{14}$$

The value of the constant H for a particular stream may be analyzed using observed flood hydrographs. Analyses by the SCS have resulted in their adoption of H = 1.67 as a general average value for ungaged watersheds. Substituting this value for H into Eq. (14) yields

$$K = \frac{1290.6}{1 + 1.67} = 484$$

and substituting this value for K into Eq. (13) yields

$$q_p = \frac{484\ AQ_v}{T_p} \qquad (15)$$

From Fig. 4, $T_p = 0.5D + L$. Based on studies of many watersheds, the SCS has developed the following empirical relationship for lag: $L = 0.6\ T_c$. Substituting these into Eq. (15) yields the final equation for the determination of peak discharge.

$$q_p = \frac{484\ AQ_v}{D/2 + 0.6\ T_c} \qquad (16)$$

where q_p = peak discharge in cfs

A = drainage area in square miles

Q_v = total runoff in inches

D = rainfall excess period in hours

T_c = time of concentration in hours — travel time of the water from the hydraulically most distant point in the watershed to the point of interest.

Potter Method

In 1961, Potter developed a method for determining peak rates of runoff from small watersheds 25 sq mi or less for the Bureau of Public Roads (21). Correlations were established between Q_{10}, the peak rate of runoff for an average recurrence interval of 10 years, and a topographic index T, a precipitation index P, and the watershed area A. The procedure was based on the use of lithological zone and rainfall index maps and a series of correlation nomographs. He cautioned that the results obtained through the procedure should be construed as aids to engineering judgment rather than proven figures.

Illinois Method

In 1968, Ellis developed a method for estimating flood flows from small drainage areas of less than 10 sq mi in Illinois (9). Multiple regression analysis correlated flood discharges of several levels of magnitude with the following basin characteristics: size of drainage area A in square miles, length of stream L in miles, perimeter P in miles, and channel slope S in percent. Nomographs for estimating flood-frequency relations were presented for convenience in solving the exponential equations.

Method of Bock, et al.

In 1972, Bock, et al. developed peak flow estimates for small rural watersheds (less than 25 sq mi) applicable nationally for the National Highway Research Board (4). Three sets of prediction equations for the United States that were similar in predictive capability to each of 31 state methods were presented. Discussion highlighted the designer's responsibility to consider alternatives of design cost and estimation error possibilities.

Reliability of Estimates

Each of these studies discusses results in terms of "estimates" of peak flows or "predictions" of flood flows. None of them claims to have determined the peak flow for a particular watershed, only an estimate of the true value. Bock, et al. indicate that about two-thirds of such predictions may be in error by 25 percent or more, and that some estimates are grossly in error.

The range of predictions from various methods for a given site will indicate variability in the estimation of the true value. Table 2 lists the estimates of the discharge with a 50-yr recurrence interval from several methods for three small gaged watersheds in Iowa.

Table 2. Estimates of Q_{50} based on various methods

Area or method	Estimates of Q_{50}, cfs, for indicated USGS gage number 5-4537	5-4540	5-4550
D.A., ac.	990	15,740	1,930
Methods			
ISHC	900	4,800	1,425
Bulletin 28	800	3,560	1,250
Bulletin 11	1,430	4,940	2,010
Potter	—	4,600	1,580
Bock, C-1	1,256	—	—
Bock, D-3	1,938	—	—

Table 3 lists the location, size, and statistical parameters of 21 selected small gaged streams in Iowa. This list is a portion of the stations used by the USGS to develop flood-frequency equations for Iowa based on the log-Pearson Type III distribution (18). Streamflow data usually is skewed to some extent. By transforming the raw data to logarithms, the data will come closer to a normal distribution

Table 3. Location, size, and statistical parameters of selected small gaged streams in Iowa[a]

Station	Location	Size, ac.	Years of record	Mean, logs	Std. dev., logs	Skew
5-4116.5	Crane Ck. trib. nr. Saratoga	2,600	20	2.7294	0.4593	- 1.5482
5-4144.5	N. Fk. L. Maquoketa R. nr. Richardsville	14,590	20	3.2214	0.3046	0.4325
5-4206	L. Wapsi. R. trib. nr. Riceville	580	20	2.2794	0.3544	- 0.4708
5-4206.2	L. Wapsi. R. nr. Acme	4,970	20	2.7147	0.3260	0.4168
5-4211	Pine Ck. trib. nr. Winthrop	210	17	1.9706	0.3539	0.0124
5-4213	Pine Ck. trib. no. 2 nr. Winthrop	450	17	1.9389	0.6107	- 0.2166
5-4486	E. Br. Iowa R. above Hayfield	1,430	18	1.7691	0.4897	- 0.2015
5-4487	E. Br. Iowa R. nr. Hayfield	5,080	20	2.1639	0.3393	- 0.3927
5-4537.5	Rapid Cr. SW of Morse	9,470	21	2.9879	0.3860	- 0.1507
5-4540	Rapid Ck. nr. Iowa City	15,740	35	3.1770	0.4144	- 0.6314
5-4550	Ralston Ck. at Iowa City	1,930	48	2.6086	0.3879	- 0.4033
5-4552.8	S. Fk. English R. trib. nr. Barnes City	1,610	19	2.5679	0.3245	- 0.6180
5-4553	S. Fk. English R. nr. Barnes City	7,360	20	2.7063	0.3437	- 0.0173
5-4830	E. Fk. Hardin Ck. nr Churdan	15,360	20	2.3282	0.02295	- 0.8581

[a]After Lara (18).

Table 3. Continued

Station	Location	Size, ac.	Years of record	Mean, logs	Std. dev., logs	Skew
5-4956	S. Wyaconda R. nr. West Grove	3,000	18	2.7470	0.4361	- 0.1308
6-4834.4	Dawson Ck. nr. Sibley	2,780	21	2.3765	0.5658	0.7948
6-6105	Indian Ck. at Council Bluffs	5,110	18	2.7851	0.4768	- 0.6612
6-8077.6	Middle Silver Ck. nr. Oakland	16,450	18	2.9458	0.1068	0.3006
6-8090	Davids Ck. nr. Hamlin	16,640	21	2.9196	0.5708	- 0.1788
6-8118	E. Tarkio Ck. nr. Stanton	2,980	16	2.7446	0.4298	0.5406
6-8118.2	Tarkio R. trib. nr. Stanton	430	15	2.2708	0.2487	1.2171

with a skew equal to zero. However, the transformed data in Table 3 still displays some skew.

A statistical method of looking at the range of estimates of the true value is the use of confidence intervals. "Student's" t-distribution is applied to the mean and standard deviation, obtained from an analysis of gaging station records, to determine a lower and upper limit for the true value of the mean with some degree of confidence. "Student's" t-distribution converges to the normal distribution as N, the number of items of data, grows large. Snedecor and Cochran (28) describe the distribution of t as practically normal with $\mu = 0$ and $\sigma = 1$ in large samples. Only when the sample size is less than 30 does the difference become obvious.

Table 4 presents the 95 and 5 percent confidence limits of the data listed in Table 3 and using the "Student's" t-distribution. For example, for gage No. 5-4540, we are 90 percent confident that the true value of the mean annual flood lies between 220 cfs and 10,500 cfs. The mean annual flood is the average of the largest annual floods recorded during the sampling period. These largest annual floods will have varying recurrence intervals. The large confidence interval of 10,280 cfs indicates only the large variability inherent in the annual flooding on Iowa's streams.

Confidence limits associated with the design discharge estimate, such as Q_{100}, make judgment of design adequacy more reliable. Beard (3) has proposed a method to calculate confidence limits for various recurrence intervals and number of years of record. Table 5 presents the 95 and 5 percent confidence limits for the 100-yr flood estimate

Table 4. Ninety-five and 5% confidence limits for the mean annual flood on several small gaged streams in Iowa

Station	Discharge, cfs		
	Q_{maf}	95%	5%
5-4116.5	540	60	4,950
5-4144.5	1,660	380	7,270
5-4206	190	30	1,060
5-4206.2	520	110	2,510
5-4211	90	20	530
5-4213	90	4	1,740
5-4486	60	5	640
5-4487	150	30	750
5-4537.5	970	150	6,250
5-4540	1,500	220	10,500
5-4550	410	70	2,450
5-4552.8	370	80	1,790
5-4553	510	40	5,830
5-4830	210	70	650
5-4956	560	70	4,690
6-4834.4	240	20	3,640
6-6105	540	50	5,570
6-8077.6	880	520	1,490
6-8090	830	50	13,000
6-8118	560	70	4,640
6-8118.2	190	50	640

Table 5. Ninety-five and 5% confidence limits for the 100-yr flood on several small gaged streams in Iowa

Station	Discharge, cfs		
	Q_{100}	95%	5%
5-4116.5	2,030	1,100	5,660
5-4144.5	7,370	4,910	14,600
5-4206	980	610	2,160
5-4206.2	3,950	2,550	8,220
5-4211	610	370	1,500
5-4213	1,970	820	9,250
5-4486	730	370	2,380
5-4487	790	500	1,690
5-4537.5	6,250	3,780	14,400
5-4540	8,620	5,610	16,300
5-4550	2,310	1,620	3,780
5-4552.8	1,330	860	2,850
5-4553	2,820	1,780	6,080
5-4830	510	380	850
5-4956	3,560	1,780	13,500
6-4834.4	7,590	3,640	25,800
6-6105	4,170	2,160	13,200
6-8077.6	1,690	1,460	2,190
6-8090	5,770	2,750	19,800
6-8118	3,640	1,930	11,500
6-8118.2	1,200	830	2,370

for the stations listed in Table 3. The estimates of Q_{100} were obtained from the frequency curve developed for each station based upon the log-Pearson Type III distribution (18). For gage No. 5-4206, we are 90 percent confident that the true value of Q_{100} lies between 610 cfs and 2,160 cfs. The point estimate was 980 cfs based on 20 years of record. If this point estimate had been based on 100 years of record, with all other parameters remaining the same, the confidence limits would have been 770 cfs and 1,310 cfs.

Wycoff in Missouri (32) compared methods of determining peak discharges. Six hydrologic methods were chosen for study and each was applied to several small gaged rural Missouri watersheds which were 100 to 1,000 ac. in size. Results from each method were compared to flood peak values obtained from analysis of existing flood data. Correct prediction was defined as estimation of an observed flow within plus or minus 20 percent. Six categories, three based on watershed size and three on recurrence interval, were used to judge the adequacy of the method. The Potter Curves and BPR Chart ranked last and next to last consistently. The Missouri Geological Survey Regression Equations ranked second, third, or fourth depending on the category. The Rational Method ranked first or second in five out of six categories. The Harbaugh Regression Equations ranked first, second, or third all six times. The Simplified SCS Method ranked first for larger watersheds (greater than 250 ac.) and also ranked first in prediction of the 50-yr flood.

In conclusion, several methods have been developed to determine the design discharge for a specified recurrence interval. No one method can be assumed to yield the true answer, but some methods have been shown to be better than others. For instance, the SCS Method has been shown to be an equal or better predictor than other methods. Also, the range of answers from these various methods are within reasonable limits based on our present knowledge.

Previous Studies Which Incorporated Upstream Storage

Howe and Metzler

The concept of using upstream channel and valley storage to reduce the peak discharge used in the design of culverts is not new. In 1954, Howe and Metzler used this idea in a study for the Iowa Highway Research Board (IHRB) (14). Comments about their study are included herein for two reasons: to summarize previous research sponsored by the IHRB on this subject and to serve as background for the development of the computerized design method detailed in the next section.

County engineers in southwestern Iowa have replaced many small bridges with culverts to halt further channel degradation and erosion of bridge abutments. Three benefits of this change to culverts have been a stabilization of the grade of the channel, a halt to the erosion of the channel banks upstream of the highway, and a reduction in maintenance costs. A fourth benefit occurred when the storage volume in the gullies of this region of Iowa was used to allow reduction in the size of culvert installed.

The peak discharge equations used to compute culvert size do not include the effects of channel and valley storage. To include storage, the designer must use a relatively simple, though sometimes tedious, design procedure. Even though he may have the knowledge to use this procedure, the value of his time in making the lengthy computations may offset some or all of the savings realized by using a smaller culvert. However, while the savings at one site may be small, the large number of culverts installed over a period of a few years may result in a significant overall savings.

The reduction in peak flow through the culvert occurs since part of the water is temporarily stored upstream of the culvert as shown in Fig. 5. This occurred even when the culvert entrance was not submerged. By allowing water to rise above the culvert crown, the amount going into storage was substantially increased with a corresponding decrease in the peak flow through the culvert. The reduction in flow was determined by using a streamflow routing process applicable for reservoirs. Standard inflow hydrographs, dependent on watershed size, were assumed for 5-, 10-, and 25-yr recurrence intervals. These inflows were routed through a series of valley configurations made up of combinations of several channel widths, side slopes, and streambed gradients. The routing results were embodied in culvert design diagrams which showed the diameter of culvert with sharp-edged entrance needed in relation to watershed size, valley form, and amount of culvert submergence desired. The diagrams permitted engineers to take advantage of upstream storage, thereby considerably reducing the required size as the permissible depth of ponding was increased.

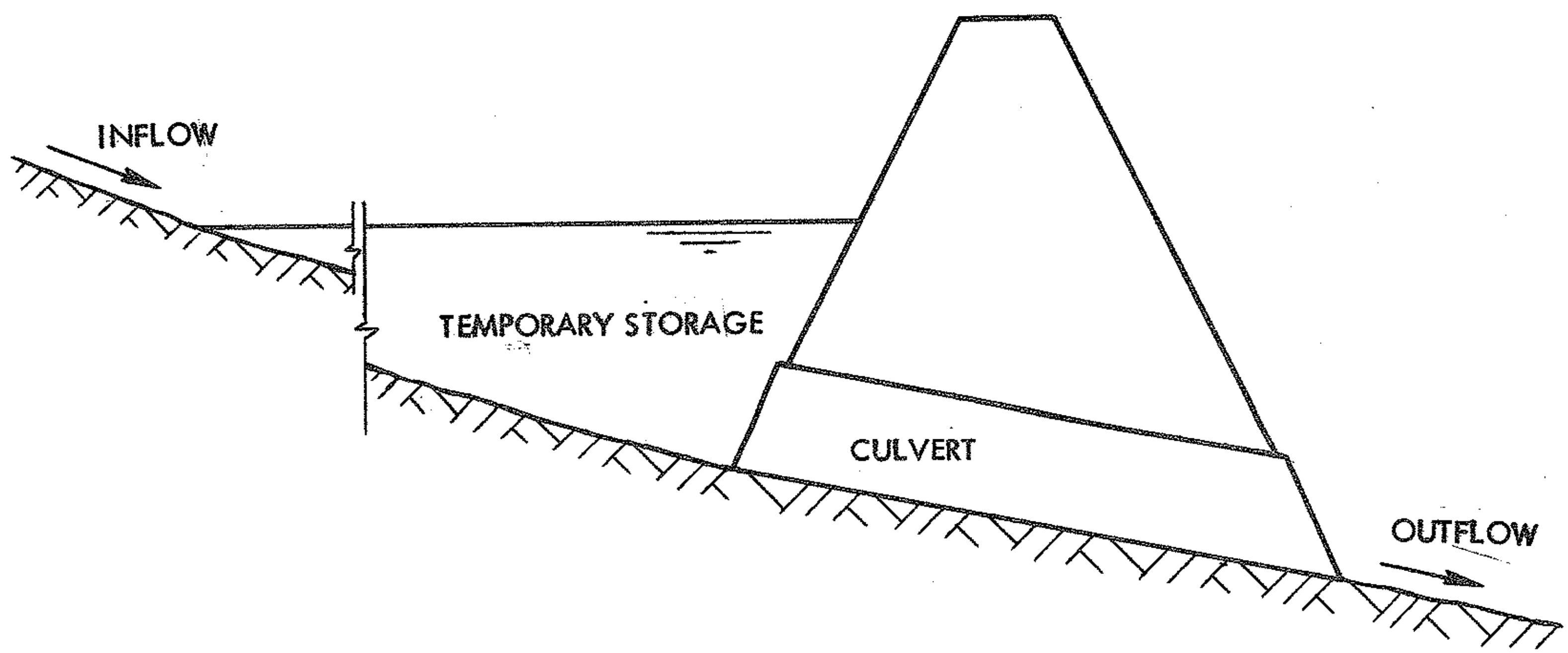

Fig. 5. Temporary storage upstream of a culvert.

Young, et al.

A more recent study which included upstream storage in the design of culverts was prepared by Young, et al. for the Federal Highway Administration in 1970 (33). In addition, their study included the question of economic and social consequences of culvert design. The objective of the study was to develop a procedure to reduce flood-related damage to highways on a sound probabilistic basis, considering hydrologic, hydraulic, and economic factors. Culvert hydraulic computations were an integral part of the analysis and techniques used included ponding, outflow, and headwater prediction as a function of time and analysis of the complete inflow hydrograph.

Two case studies were included to illustrate the use of the computer program Young, et al. developed. Recurrence intervals of five years for the I-85 site and one year for the Glade were required to approximate the optimum solution. The smaller size of the optimal designs, over the conventional 50-yr design, was attributed to permissibility of ponding and acceptance of occasional losses in order to reduce construction costs. Total social costs (sum of construction costs plus expected losses or risks) were lower when some ponding was allowed. For both case studies, optimal designs had 9 percent of their total social costs in the risk category and 91 percent in construction costs.

Improvements available today

The present study differs from the above studies in the following ways. The computer was not readily available as a design tool in 1954 when Howe and Metzler made their study. In addition, they used standard valley configurations rather than the actual storage capacities which are unique to each site. Also, the present study allows multiple box and/or pipe culverts of any size and inlet type to be analyzed rather than single corrugated metal pipes with sharp-edged entrances. While the study by Young, et al. used a computer-based model and a stage-storage function for each site, the hydrologic data were assumed known and simple triangular hydrographs were adopted. Little effort was given to the definition of flood peaks, flood hydrograph shapes, or return-period estimates. In addition, only box culverts could be analyzed. In the present study, the definition of the inflow hydrograph for various storm durations and recurrence intervals is one major feature of the computerized design method developed. Each of these differences makes the present study a more comprehensive tool for the designer of highway drainage structures.

PROGRAM DEVELOPMENT

Introduction

Today, computers in many design offices eliminate the need for tedious calculations. The designer's time plus computer operation represents only a small fraction of the savings effected in culvert construction costs by using the proposed computerized design method. The computer increases the flexibility of design studies by including box culverts as well as pipes (concrete or corrugated metal) plus various entrance types for each, by varying the invert elevation of the culvert to take best advantage of site conditions, by testing the proposed culverts with flows from storms greater than the design storm, and by extending the concept of using upstream storage to all areas of the state.

There are three phases to the passage of water through a highway culvert: inflow to the upstream area, storage changes in the temporary pond volume, and outflow from the pond through the culvert. Each phase is included in the computer program which has been named HDC. After presenting this in detail, the safety and effectiveness of the program in recommending use of a smaller culvert is discussed. While the general structure of the program may be used anywhere, specific equations used make this program form applicable only to the State of Iowa.

Inflow Hydrograph

Amount of rainfall

As described in an earlier section, all precipitation in this study is assumed to be in the form of rain. Over many years, observations from a nationwide network of precipitation gaging stations have been compiled, analyzed, and published by the National Weather Service of the US Department of Commerce (formerly the US Weather Bureau). Technical Paper No. 40, Rainfall Frequency Atlas of the United States (USWB-TP-40), is the source of rainfall amounts used in the present study (23).

Rainfall amounts for total storm durations of 30 minutes and 1, 2, 3, 6, 12, and 24 hours for recurrence intervals of 1, 2, 5, 10, 25, 50, and 100 years were taken from USWB-TP-40. The data are shown as lines of equal rainfall superimposed on a map of the United States. Storm durations of less than 30 minutes are taken as percentages of the 30-minute storm duration as shown in Table 6.

Table 6. Percentages of 30-minute rainfall duration[a]

Duration, minutes	Factor percent
5	37
10	57
15	72

[a]After USWB-TP-40.

In USWB-TP-40, the term reliability is used in the statistical sense to refer to the degree of confidence that can be placed in the accuracy of results. In developing the depth-area relations, data from several dense networks were examined. Examination of data from regions where the physiography could have little or no effect showed, for example, that the standard deviation of point rainfall for the 2-yr return period for a flat area of 300 sq mi was about 20 percent of the mean value. Iowa's rainfall regime is also not influenced locally by orography or bodies of water. Seventy 24-hr stations in Iowa, each with more than 40 yr of record showed a range in the 2-yr, 24-hr isopluvials of from 3.0 to 3.3 in. These deviations must be regarded as a residual error in sampling since there were no assignable causes for these dispersions.

The rainfall amounts obtained from these maps are expressed in partial-duration frequencies and represent point rainfalls. For the recurrence intervals normally used in culvert design, ten years and longer, values for the partial-duration and annual series coincide; so no adjustment was made to the values obtained from USWB-TP-40. Based on analyses of the records, these point rainfall amounts may be used as representing average depths over watersheds up to a few square miles in size.

For watersheds larger than a few square miles, a rainfall ratio must be applied to the rainfall amount obtained from the maps. This rainfall ratio is a function of drainage area and storm duration and is depicted in USWB-TP-40 as a series of curves. A tabular form is shown in Table 7. This ratio is applied to the rainfall amount obtained from the maps by the use of Eq. (17).

$$\text{Rainfall}_{\text{use}} = \text{Rainfall}_{\text{map}} \times \text{Rainfall ratio} \qquad (17)$$

Table 7. Rainfall ratios based on watershed size and storm duration[a]

Duration	Rainfall ratios, percent, for indicated drainage area, sq mi				
	0	25	50	100	150
30 minutes	100	80	69	661	58
1 hour	100	87	80	72	69
3 hours	100	93	90	85	82
6 hours	100	95	92	89	87
24 hours	100	97	95	94	93

[a]After USWB-TP-40.

As shown in Table 7, for a 25-sq mi watershed, there is as much as a 20-percent reduction from the map point value for a storm duration of 30 minutes. As a practical matter, a 30-minute storm duration would not be used on a watershed of 25 sq mi. A longer duration storm is required to develop the peak runoff expected to occur once every 25 or 50 years. Based on a sample of 54 watersheds, ranging from 40 to 17,920 ac., and from flat to very hilly, minimum storm durations for various drainage areas were determined as shown in Table 8.

Minimum rainfall ratios for various watershed sizes are shown in Table 9, a combination from Tables 7 and 8. The maximum reduction is about 6 percent for a watershed of 17,920 ac. Based on this, the decision was made to not reduce the rainfall amounts obtained from

Table 8. Minimum storm durations based on watershed size

Drainage area, ac.	Drainage area, sq mi	Minimum storm duration, hr
100	0.16	0.3
400	0.63	0.4
1,000	1.56	0.8
1,500	2.34	1.2
2,000	3.12	1.8
5,000	7.82	2.0
7,500	11.70	2.2
10,000	15.60	2.5
15,000	23.40	3.8
17,920	28.00	4.5

USWB-TP-40. The higher rainfall amount is within the range of error of the maps and the conservative rainfall depths yield slightly higher headwater depths. From another viewpoint, some agencies use the point rainfall amounts up to 10 sq mi (6,400 ac.) without reduction.

Equations for rainfall amounts

The rainfall amount for a particular storm duration and recurrence interval could have been made an input item to the program. This would have required the designer to look up the rainfall amounts for each design. However, in keeping with the premise that input data be kept to a minimum, USWB-TP-40 was reduced to a series of equations. The only input data required are the recurrence interval and the county

Table 9. Minimum rainfall ratios for watersheds of various sizes

Drainage area, ac.	Drainage area, sq mi	Reduction factor, percent
100	0.16	100
400	0.63	99
1,000	1.56	99
1,500	2.34	98
2,000	3.12	98
5,000	7.82	97
7,500	11.70	96
10,000	15.60	95
15,000	23.40	95
17,920	28.00	94

number obtained from Table 10. The storm duration is determined within the program as will be explained later.

The equations for total rainfall were determined in the following manner. Rainfall amounts for the several durations and recurrence intervals were scaled from the maps in USWP-TP-40 for Story County and plotted on log-log paper as depicted in Fig. 6. The plotted data forms a family of curves which are slightly convex upwards. This family of curves can be described by an equation of the form:

$$P = aRI^{\frac{b}{RI^{c}}}\ Dur^{\frac{d}{Dur^{e}}} \tag{18}$$

Table 10. Iowa county numbers

No.	County	No.	County	No.	County	No.	County
1	Adair	24	Crawford	47	Ida	70	Muscatine
2	Adams	25	Dallas	48	Iowa	71	O'Brien
3	Allamakee	26	Davis	49	Jackson	72	Osceola
4	Appanoose	27	Decatur	50	Jasper	73	Page
5	Audubon	28	Delaware	51	Jefferson	74	Palo Alto
6	Benton	29	Des Moines	52	Johnson	75	Plymouth
7	Black Hawk	30	Dickinson	53	Jones	76	Pocahontas
8	Boone	31	Dubuque	54	Keokuk	77	Polk
9	Bremer	32	Emmet	55	Kossuth	78	Pottawattamie
10	Buchanan	33	Fayette	56	Lee	79	Poweshiek
11	Buena Vista	34	Floyd	57	Linn	80	Ringgold
12	Butler	35	Franklin	58	Louisa	81	Sac
13	Calhoun	36	Fremont	59	Lucas	82	Scott
14	Carroll	37	Greene	60	Lyon	83	Shelby
15	Cass	38	Grundy	61	Madison	84	Sioux
16	Cedar	39	Guthrie	62	Mahaska	85	Story
17	Cerro Gordo	40	Hamilton	63	Marion	86	Tama
18	Cherokee	41	Hancock	64	Marshall	87	Taylor
19	Chickasaw	42	Hardin	65	Mills	88	Union
20	Clarke	43	Harrison	66	Mitchell	89	Van Buren
21	Clay	44	Henry	67	Monona	90	Wapello
22	Clayton	45	Howard	68	Monroe	91	Warren
23	Clinton	46	Humboldt	69	Montgomery	92	Washington

Table 10. Continued

No.	County	No.	County	No.	County	No.	County
93	Wayne	95	Winnebago	97	Woodbury	99	Wright
94	Webster	96	Winneshiek	98	Worth		

where P = total precipitation in inches

RI = recurrence interval in years

Dur = storm duration in hours

a, b, c, d, e = constants.

The maps in USWB-TP-40 show that for a particular storm duration and recurrence interval, there is little if any variation in rainfall in any one county, a minor variation in any one region, and a moderate variation in total rainfall across the state. Since current practice (25) uses rainfall to the nearest tenth of an inch and to reduce the total number of equations required, the state was divided into nine regions as shown in Fig. 7. Equation (19) was developed for the central region of Iowa, Region C, based upon the data for Story County.

$$P = 1.32\ RI^{\frac{0.264}{RI^{0.065}}}\ Dur^{\frac{0.266}{Dur^{0.050}}} \tag{19}$$

P, RI, and Dur are as previously defined. Equation (19) was then multiplied by an adjustment factor for the other regions. The adjustment took the following forms, with the variables as previously defined.

$$\text{All western regions: } Adj. = aRI^{\frac{b}{RI^{c}}}/Dur^{d} \tag{20a}$$

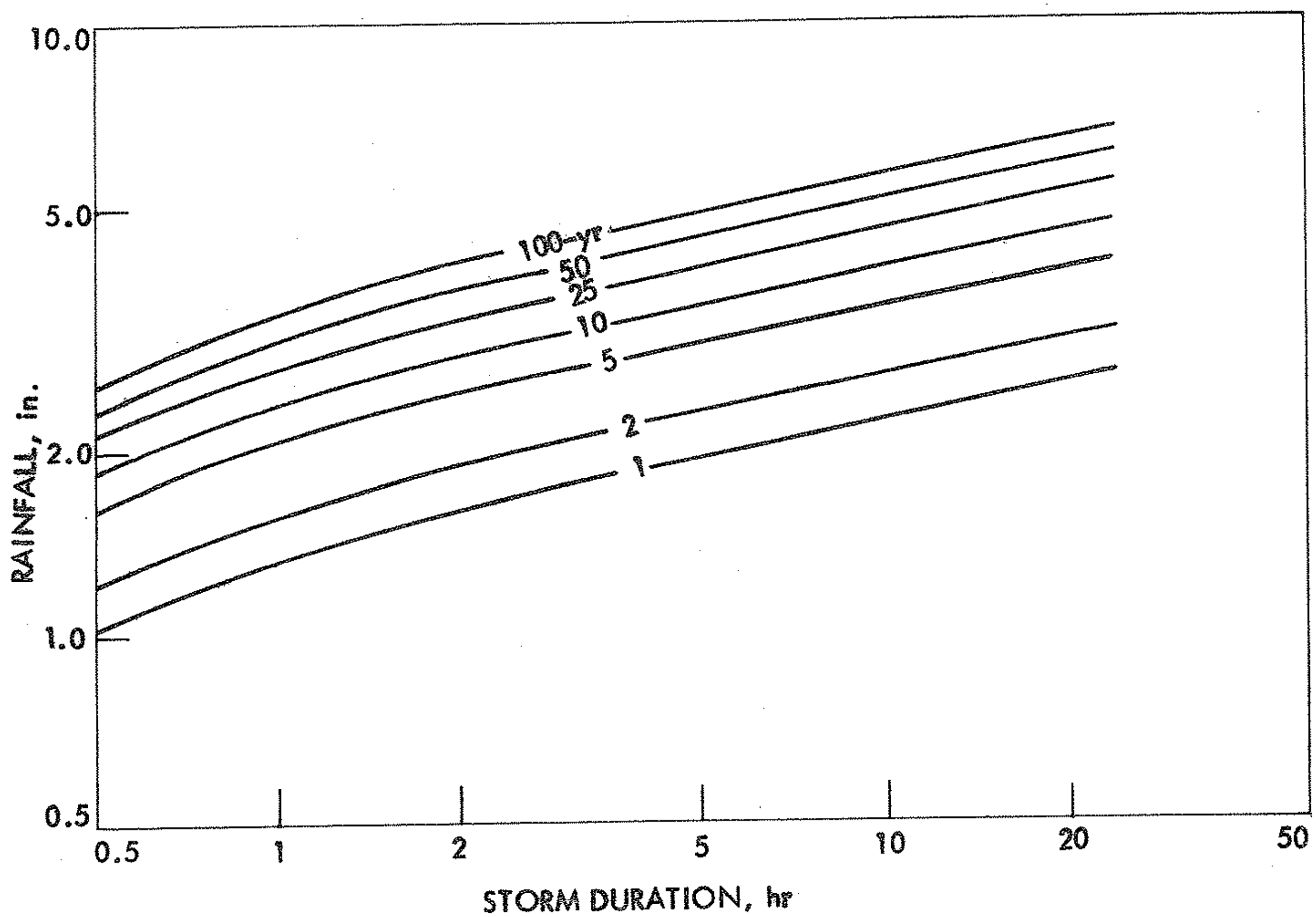

Fig. 6. Rainfall amounts in Story County as obtained from USWB-TP-40.

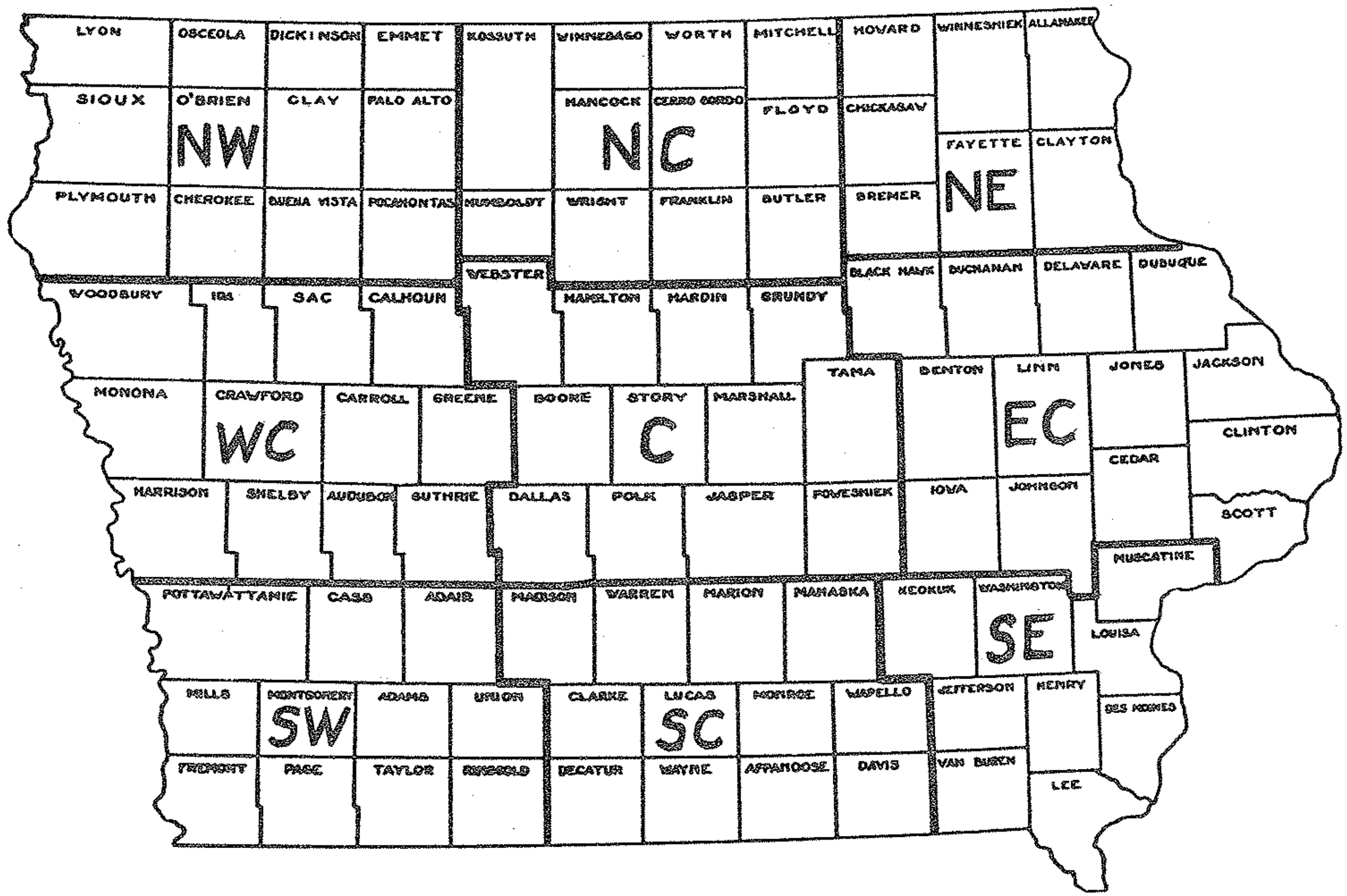

Fig. 7. Regions of similar rainfall in Iowa.

Central: $\text{Adj.} = a = 1$ (20b)

N. and S. central: $\text{Adj.} = a\text{RI}^{\frac{b}{\text{RI}^c}} / \text{Dur}^d$ (20c)

All eastern regions: $\text{Adj.} = a\text{RI}^{\frac{b}{\text{RI}^c}} \ \text{Dur}^d$ (20d)

Rainfall depths for all durations and recurrence intervals listed in USWB-TP-40 for all 99 counties were computed using Eqs. (19) and (20). These estimates were compared to USWB-TP-40 data. The differences for each county, both in inches and in percent, were computed. The coefficient "a" in Eq. (20) was then revised, if necessary, to reduce the difference in rainfall amounts between the equations and USWB-TP-40. These equations can generate rainfall depths for any combinations of storm duration and recurrence interval anywhere in the State of Iowa.

Woodbury County is used as an example since it has the worst fit of the developed equations to the values taken from USWB-TP-40. Table 11 lists the rainfall depths calculated by using the appropriate equations for Woodbury County and those obtained from USWB-TP-40 are listed in Table 12.

The differences between the calculated amounts and USWB-TP-40 data, Table 11 minus Table 12, are shown in Table 13. The percentage difference between the two rainfall amounts is determined by using Eq. (21).

$$\text{Percentage difference} = \frac{P_{\text{eqn.}} - P_{40}}{P_{40}} \times 100 \qquad (21)$$

These percentages are shown in Table 14. In both Tables 13 and 14, the minus sign indicates that the calculated rainfall is less than the

Table 11. Rainfall amounts in Woodbury County calculated using the developed equations

Duration, hr	Rainfall amounts, in., for indicated recurrence interval, yr						
	1	2	5	10	25	50	100
0.5	1.02	1.23	1.63	1.90	2.24	2.49	2.73
1	1.23	1.54	1.96	2.28	2.69	2.99	3.28
2	1.45	1.82	2.32	2.70	3.18	3.54	3.88
3	1.60	2.00	2.55	2.96	3.49	3.88	4.26
6	1.85	2.33	2.96	3.44	4.06	4.51	4.95
12	2.13	2.67	3.40	3.95	4.66	5.18	5.69
24	2.42	3.04	3.87	4.49	5.30	5.90	6.47

Table 12. Rainfall amounts in Woodbury County as obtained from USWB-TP-40

Duration, hr	Rainfall amounts, in., for indicated recurrence interval, yr						
	1	2	5	10	25	50	100
0.5	1.01	1.23	1.62	1.87	2.21	2.46	2.70
1	1.28	1.54	2.05	2.41	2.80	3.12	3.51
2	1.46	1.82	2.35	2.73	3.18	3.60	4.00
3	1.56	1.96	2.62	2.94	3.40	3.80	4.30
6	1.81	2.19	2.92	3.39	3.90	4.44	4.89
12	2.10	2.58	3.33	3.90	4.27	5.00	5.68
24	2.38	2.92	3.75	4.38	5.05	5.77	6.28

Table 13. Differences between rainfall in Woodbury County obtained from the developed equations and USWB-TP-40

Duration, hr	Rainfall differences, in., for indicated recurrence interval, yr						
	1	2	5	10	25	50	100
0.5	0.01	0.00	0.01	0.03	0.03	0.03	0.03
1	- 0.05	0.00	- 0.09	- 0.13	- 0.11	- 0.13	- 0.23
2	- 0.01	0.00	- 0.03	- 0.03	0.00	- 0.06	- 0.12
3	0.04	0.04	- 0.07	0.02	0.09	0.08	- 0.04
6	0.04	0.14	0.04	0.05	0.16	0.07	0.06
12	0.03	0.09	0.07	0.05	0.29	0.18	0.01
24	0.04	0.12	0.12	0.11	0.25	0.13	0.19

Table 14. Differences between rainfall in Woodbury County obtained from the developed equations and USWB-TP-40

Duration, hr	Rainfall differences, percent, for indicated recurrence interval, yr						
	1	2	5	10	25	50	100
0.5	1.0	0.0	0.6	1.6	1.4	1.2	1.1
1	- 3.9	0.0	- 4.9	- 5.4	- 3.9	- 4.2	- 6.5
2	- 0.7	0.0	- 1.3	- 1.1	0.0	- 1.7	- 3.0
3	2.6	2.0	- 2.7	0.7	2.6	2.1	- 0.9
6	2.2	6.4	1.4	1.5	4.1	1.6	1.2
12	1.4	3.5	2.1	1.3	6.8	3.6	1.8
24	1.7	4.1	3.2	2.5	5.0	2.2	3.0

USWB-TP-40 rainfall figure. For the recurrence intervals generally used in the design of culverts, the maximum percentage difference between the two rainfall amounts is normally about 3 percent.

The developed equations are both flexible and complete. Any recurrence interval, such as 43 years, and any duration, such as 12.34 hours, can be used. The accuracy of the equations beyond the range of USWB-TP-40 has been tested and appears to yield satisfactory results.

Distribution of rainfall within the storm

Rain does not fall uniformly throughout a storm. It may begin with drizzle, then rain heavily for some period, fall off into a drizzle, then end in a heavy downpour. Because of this variation and since unit hydrograph theory assumes uniform rainfall intensity, several methods have been devised to overcome this difficulty. The basic idea behind them is to divide the storm into several equal time increments, with the assumption that rainfall is uniform during each of these. The methods differ in how rainfall intensity is assumed to vary throughout the storm.

From SCS analyses come average rainfall distributions for a storm duration of 24 hours applicable to the midwest. Other SCS distributions, for a storm duration of 6 hours, places the period of heaviest rainfall in various sequence locations during the storm.

Brater and Sherrill in Michigan (6) found that the ratio of precipitation occurring during any shorter duration, e.g. one hour, to the 24-hr precipitation of the same frequency was relatively constant.

Hyetographs, or typical rainstorms of various frequencies broken down into time increments as small as thirty minutes, were developed. The order of placement was based on the analysis of many storms with the most intense portion placed before the middle of the total duration. These typical rainstorms were characterized by uniform recurrence intervals during all portions of the storm.

The author made a similar investigation for Iowa using USWB-TP-40. A similar finding was made; the ratio of the rainfall in a shorter duration to the rainfall in a 24-hr duration storm varied over a very narrow range. This is shown graphically in Fig. 8. For example, the ratio of 1-hr to 24-hr rainfall varied from 48.0 to 51.4 percent. Hyetographs similar to those of Brater and Sherrill were also developed. These were compared to those used by the SCS and were found to lie between their B and C type storms.

Neither of the above methods was deemed suitable for use in the present study for several reasons. The use of a basic 6- or 24-hr storm does not provide sufficient variability for the range of watershed sizes and slopes encountered in culvert design. While development of typical rainfall storm data is a step, the assumption that maximum portions of the storm have the same recurrence interval does not reflect reality. Data were needed from a number of observed natural storms to provide the flexibility of allowing storms to be tailored to watersheds of various sizes and slopes.

A comprehensive study by Huff in Illinois (15) describes the time distribution of rainfall in heavy storms which are applicable to the midwest. The study was based on data collected since 1955 on a

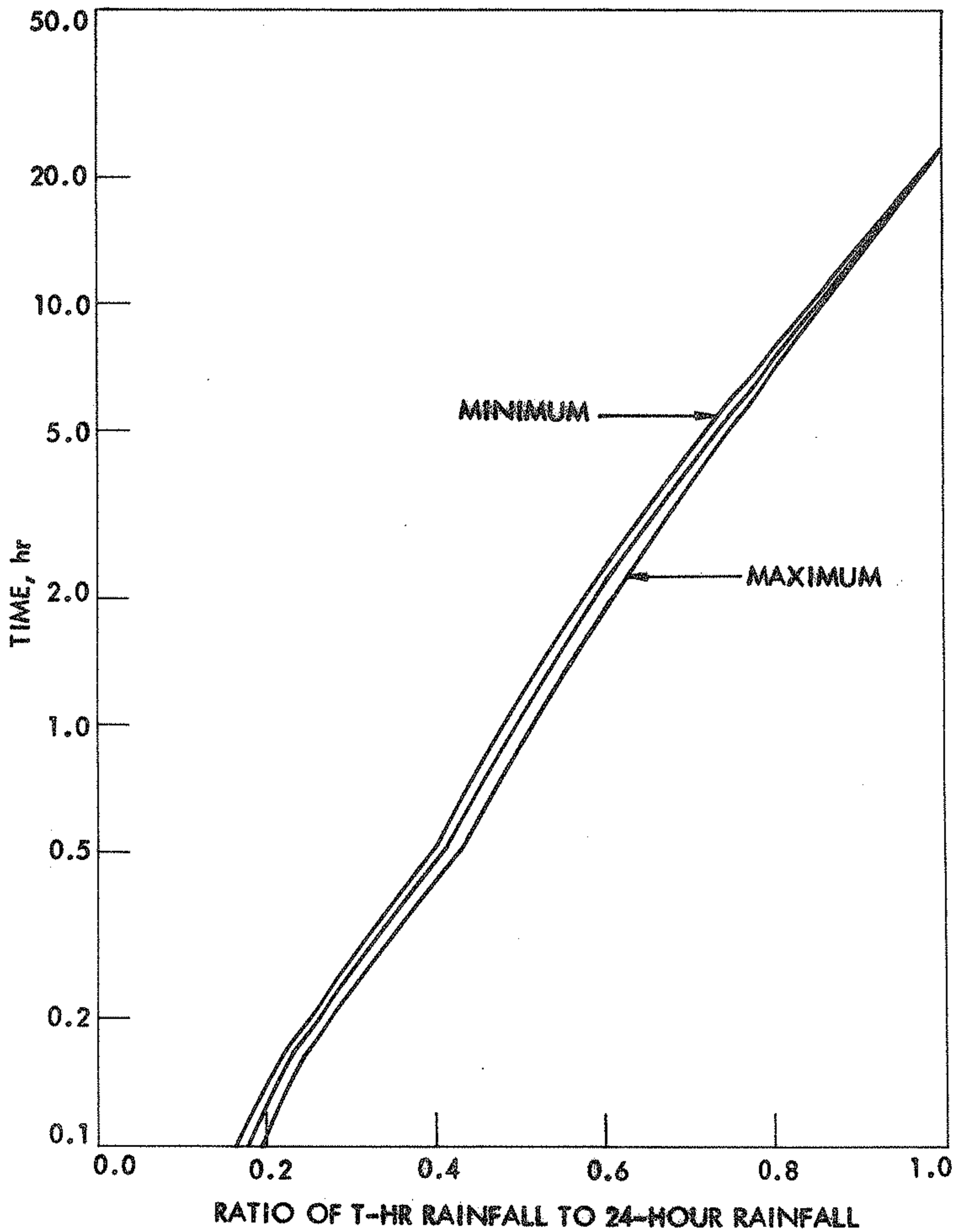

Fig. 8. Variation in ratio of t-hour rainfall to 24-hour rainfall across the State of Iowa.

concentrated network of 49 recording rain gages on 400 sq mi in a rural area of east-central Illinois. Results were presented as probability distributions and provided quantitative measures of both interstorm variability and general characteristics of the time sequence of precipitation in storms. Most rainfall occurs in a small part of the total storm time regardless of storm duration, areal mean rainfall, and total number of showers or bursts in the storm period; therefore, storms were classified into four groups, depending on the quartile in which the heaviest rainfall occurred. Within groups, long-duration storms (over 24 hours) predominated in the fourth-quartile, storms of moderate length (12 to 24 hours) were most frequent with the third-quartile type, and short-duration storms were most common in the first- and second-quartile groups.

This study by Huff (15) provides the variability necessary in the present study. A culvert is constructed to serve for at least fifty years. It will be subjected to all sizes and durations of storms relative to the time of concentration of the watershed. Since the present study includes the effects of temporary storage, the volume of runoff is as important as the peak rate of flow. Using natural storms of varying durations, total rainfall amounts, and time of occurrence and amount of rainfall in individual bursts, the designer can test a proposed culvert for likely conditions and can determine the effects of duration, etc. on headwater depth.

Twenty-six time distributions were selected for study from the thirty-six distributions presented in the study by Huff using a sample of fifty-four watersheds in Iowa which ranged in size from 40 to

17,920 ac. Fourteen of the twenty-six distributions were selected for use in the program on the basis of their ability to reproduce (in conjunction with the SCS Method described previously) the peak discharges used by the ISHC in culvert design. These fourteen time distributions of rainfall are listed in Table 15 and are shown as histograms in Appendix B. Within the program, a specific distribution is selected for each of the seven inflow hydrographs on the basis of two factors: storm duration and land use and slope factor.

Table 15. Selected time distributions of rainfall, percent probability[a]

First quartile	Second quartile	Third quartile	Fourth quartile
10	10	10	10
30	30	30	—
50	50	50	—
60	70	—	—
70	90	—	—

[a]After Huff (15).

Runoff from rainfall

The SCS Method uses curve numbers (CN) to determine a volume of runoff from a specific storm. Generalized curve numbers for Iowa were shown in Fig. 3. The author has reworked Fig. 3 so that curve number boundaries fall along county lines. The results are shown in Fig. 9 and these curve numbers are used in the computerized design method developed in this study. With these curve numbers and rainfall amounts,

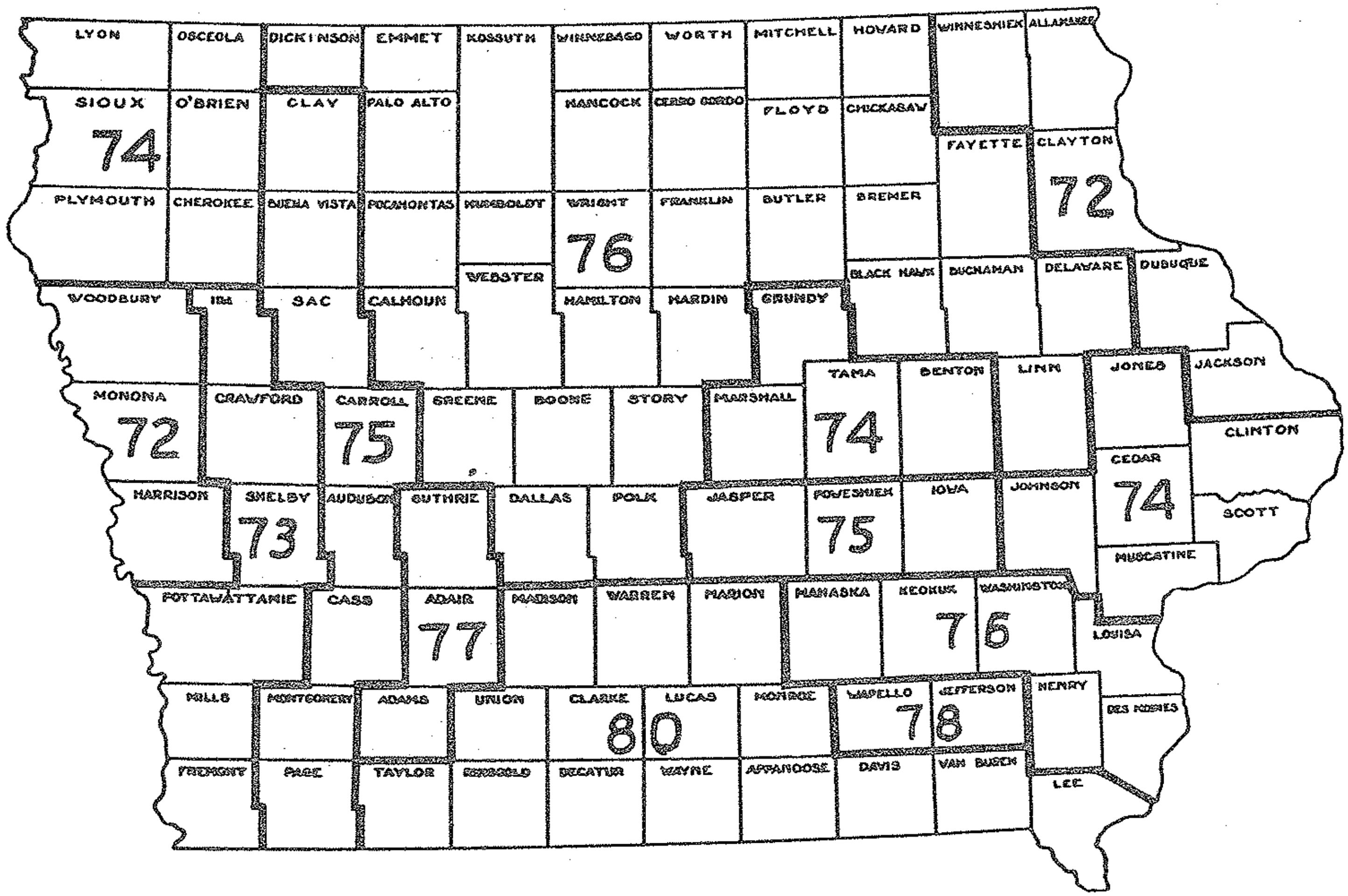

Fig. 9. Generalized runoff curve numbers.

also calculated within the program, runoff volumes for various storm durations and recurrence intervals are determined. The only input requirement for curve number is the county number. These are listed in Table 10.

The final inflow hydrograph is determined by using this SCS rainfall-runoff relation with Sherman's unit hydrograph theory (27). The unit hydrograph is defined as the hydrograph resulting from one inch of direct runoff from a storm of a specified duration. Thus, the area under the hydrograph is equal to a runoff volume of one inch from the basin. Two assumptions are implicit: there is uniform intensity of rainfall for the duration of the storm and there is uniform rainfall coverage over the entire basin. The second assumption can be met to a large extent by restricting the size of watershed to 25 sq mi. The first assumption can be met somewhat by dividing total storm duration into several time increments and developing a hydrograph for each increment.

The equation for peak discharge developed by the SCS (25) is

$$q_p = \frac{484\ AQ_v}{D/2 + 0.6\ T_c} \tag{16}$$

where q_p = peak discharge in cfs

A = drainage area in square miles

Q_v = storm runoff in inches

D = storm duration in hours

T_c = time of concentration in hours.

When $Q_v = 1$, q_p equals the unit peak discharge rate.

To account for the assumption of uniform rainfall intensity, Eq. (16) can be rewritten as:

$$q_p = \frac{484\ A\Delta Q_v}{\Delta D/2 + 0.6\ T_c} \tag{22}$$

where ΔD = incremental storm duration in hours

ΔQ_v = storm runoff in inches during ΔD time

q_p, A, and T_c as defined above.

The SCS (17) suggests using ΔD equal to one-third the time to peak. Figure 10 has been prepared by the SCS (25) as a representative unit hydrograph for ungaged watersheds. The point of inflection occurs at 1.7 times T_p on the curvilinear hydrograph. An equivalent point (same percentage of total runoff) occurs at 1.73 times T_p on the triangular hydrograph. Using the above value and Fig. 9, the following relationship between ΔD and T_c is developed.

$$\Delta D = 0.33\ T_p \quad \text{or} \quad T_p = 3\Delta D \tag{23}$$

$$\Delta D + T_c = 1.73\ T_p$$

$$0.33\ T_p + T_c = 1.73\ T_p$$

$$T_c = 1.73\ T_p - 0.33\ T_p$$

$$T_c = 1.40\ T_p \tag{24}$$

$$T_p = T_c \div 1.40$$

$$T_p = 0.715\ T_c \tag{25}$$

also $3\Delta D = 0.715\ T_c$

$\Delta D = 0.238\ T_c$ with ΔD and T_c in hours

$\Delta D = 14.3\ T_c$ with ΔD in minutes and T_c in hours (26)

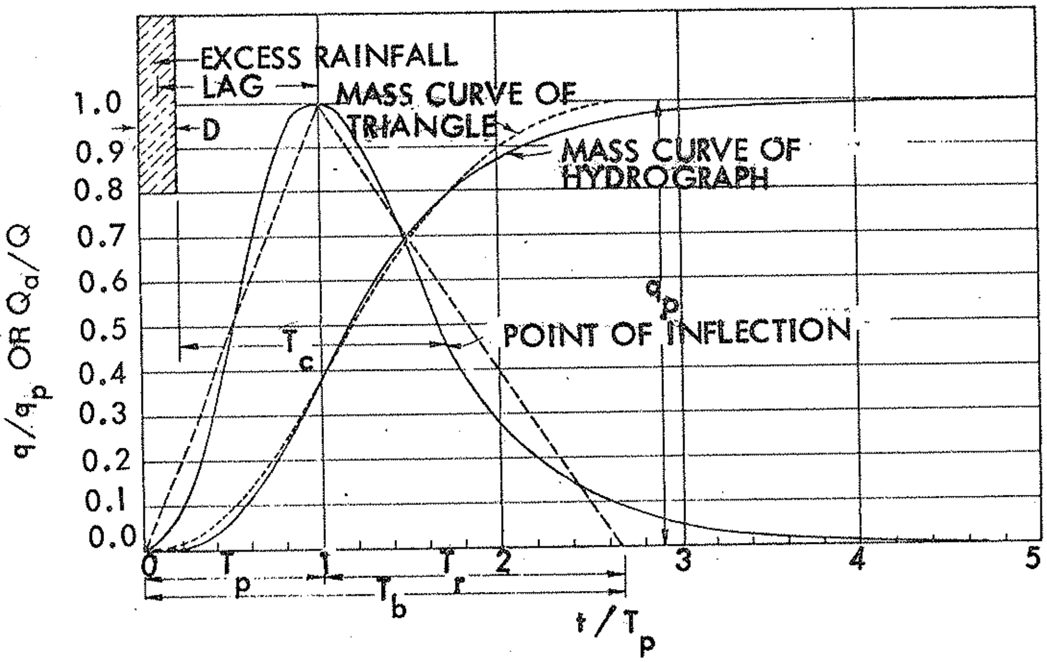

Fig. 10. Dimensionless curvilinear unit hydrograph aid equivalent triangular hydrograph.

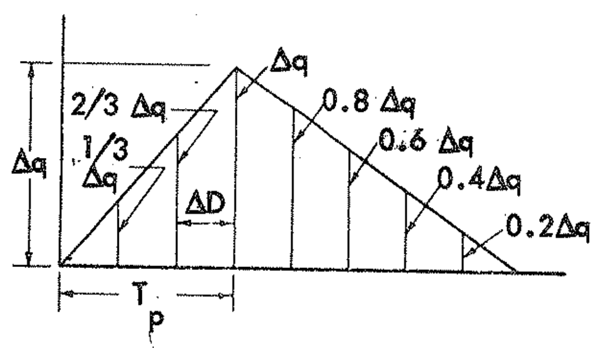

Fig. 11. Triangular hydrograph using ΔD equal to one-third the time to peak.

Also from Fig. 10, 37.5 percent of the total runoff occurs between time equals zero and time equals T_p. The time base of the equivalent triangular hydrograph then becomes:

$$T_B = T_p \div 0.375$$

$$T_B = 2.67\ T_p \tag{27}$$

where T_B = time base of the triangular hydrograph in hours

T_p = time to peak in hours.

Thus, the time to peak is three-eighths of the total time base and the time of recession is five-eighths of the total time base. This relationship allows breaking up the incremental hydrograph into eight equal time increments as shown in Fig. 11.

> "The fundamental principles of invariance and superposition make the unit graph an extremely flexible tool for developing synthetic hydrographs: 1) the hydrograph of surface runoff from a watershed due to a given pattern of rainfall is invariable, and 2) the hydrograph resulting from a given pattern of rainfall excess can be built up by superimposing the unit hydrograph due to the separate amounts of rainfall excess occurring in each unit period. This includes the principle of proportionality by which the ordinates of the hydrograph are proportional to the volume of rainfall excess." (25)

Using these principles, summation of the individual triangular hydrographs (each of which is offset one ΔD time increment from the previous one) yields the final inflow hydrograph for the particular storm duration. This process is illustrated graphically in Fig. 12.

<u>Comparison of peak rate of discharge</u>

The method presently used by the ISHC was shown in Fig. 1. The discharge, from the chart, is a function of drainage area. Thus, to determine a design discharge, three variables are required: a frequency

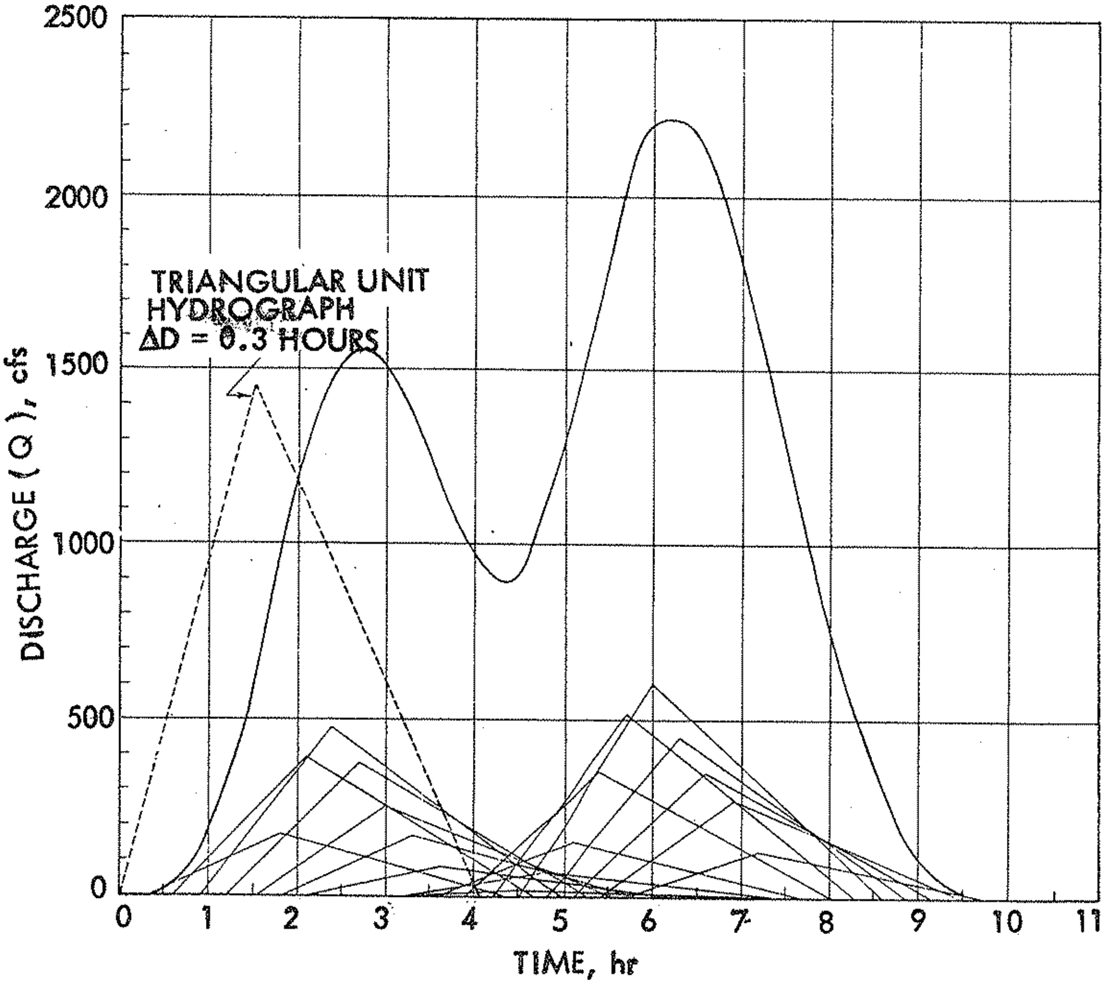

Fig. 12. Composite flood hydrograph from incremental triangular hydrographs.

factor, a land use and slope factor, and the drainage area in acres. These three variables are inputs to the computer program. The discharge calculated from these variables is used as a check on the peak hydrograph discharges calculated by the program.

The log-log plot of discharge versus drainage area shown in Fig. 1 has been included in the computer program as an equation of the form:

$$Q_c = 6.499\, A^{\frac{0.858}{A^{0.0155}}} - \frac{(\ln(0.11\, A))^{1.88} A^{\frac{1.21}{A^{0.05}}}}{75} \qquad (28)$$

The equation is accurate to drainage areas of 20,000 ac. or about 31 sq mi as shown in Table 16.

Summary

To sum up, the inflow hydrographs for a particular drainage area and recurrence interval are determined in the following manner. The time of concentration of the watershed and then ΔD are calculated. The total storm duration is first made equal to one-half the time of concentration. A particular time distribution of rainfall is selected for storm duration and land use and slope factor. Rainfall for the total storm duration and then the incremental rainfall and runoff amounts for each ΔD time increment are determined. The incremental triangular hydrographs are constructed from these runoff amounts. These are summed to give the final inflow hydrograph for that storm duration.

This procedure is repeated for each of seven different storm durations: the first equal to one-half the time of concentration, the second equal to the time of concentration, and the other five equal to some larger multiple of the time of concentration. These

Table 16. Comparison of discharges using the ISHC chart and the equation developed for the ISHC Method

D.A. ac.	Q_{chart}[a] cfs	$Q_{eqn.}$ cfs	D.A. ac.	Q_{chart}[a] cfs	$Q_{eqn.}$ cfs
1	6	6	500	785	779
2	12	12	600	885	882
3	16	16	800	1,060	1,071
4	21	21	1,000	1,240	1,244
5	25	25	2,000	1,960	1,969
6	29	29	3,000	2,540	2,566
8	36	37	4,000	3,080	3,092
10	43	44	5,000	3,540	3,570
15	59	60	6,000	3,970	4,013
20	73	75	8,000	4,800	4,822
25	87	90	10,000	5,520	5,555
30	99	103	11,000	5,900	5,900
40	126	128	12,000	6,200	6,234
50	150	151	13,000	6,550	6,556
60	172	173	14,000	6,900	6,869
80	214	214	15,000	7,200	7,173
100	254	252	16,000	7,450	7,469
150	340	337	17,000	7,750	7,759
200	415	413	18,000	8,050	8,041
300	552	548	19,000	8,300	8,317
400	675	669	20,000	8,600	8,588

[a]Values as interpreted from the chart by the author.

seven hydrographs are then used to subject each alternative culvert selected for study to the varying storms and volumes of runoff it will encounter during its service life.

The inflow hydrographs are meant to be flood hydrographs typical of those that will occur during the life of the culvert. No presumption is made that by inputing an experienced or observed storm of known time distribution of rainfall, the program will reproduce the observed flood hydrograph caused by the storm. In addition, no presumption is made that the peak of the inflow hydrograph will exactly match the peak discharge rate used by the ISHC for culvert design although they will be similar in magnitude. As discussed, no one method can yet be assumed to yield the true value. The decisions made in the development of the computer program have deliberately been made such that the peak of the inflow hydrograph will normally be somewhat greater than the design discharge estimate obtained from the present ISHC method.

Storage

Each culvert site has unique storage capabilities. The earlier study by Howe and Metzler (14) used standardized valley forms with the designer using the configuration closest to his particular situation. In the present study, the elevation-storage relationship at the culvert site is an input item to the program.

This relationship can be determined using one of three available sources. The first is the 7.5 minute quadrangle maps prepared by the US Geological Survey. If this map is not available for a particular culvert site, one of two other maps is useful. These are contour maps prepared by the Kelsh Plotter from aerial photographs and contour maps prepared from actual field surveys. Conversations with the head of the photogrammetry section of the Iowa State Highway Commission indicate that these maps are currently prepared for and used by the road design squads and the preliminary bridge section.

Aerial photographs are presently taken for all projects which include earth moving and culvert construction. Any work needed to close in contours on maps prepared using the Kelsh Plotter can be done at minimal cost according to the head of the ISHC photogrammetry section. His opinion is that maps prepared with the Kelsh Plotter are the most accurate of the three types. A study of sensitivity of headwater elevation to storage capacity based on different types of maps for a certain site is discussed later.

The storage capacity at a site is determined by planimetering the areas enclosed by the contours on the maps and then calculating the storage as shown in Table 17. The input items to the program are columns 1 and 6, the elevation in feet above MSL and the total storage below that elevation in acre-feet, respectively. The elevation-storage curve input to the program should begin with the elevation at which the natural draw flowline crosses the toe of slope of the highway fill. The program then adjusts the elevation-storage curve to the lowest proposed culvert flowline or the elevation of the drop inlet if one is used. Using the contents of Table 17 as an example and assuming a culvert flowline of 1090.0, the curve used within the program would be as shown in Table 18. This method of inputing the

Table 17. Calculation of an elevation-storage curve

1 Elevation, ft	2 Area, ac.	3 Aver. area ac.	4 Δ depth, ft	5 Δ volume, ac. ft	6 Total volume, ac. ft
1,070	0.00				0.0
		0.07	10.0	0.7	
1,080	0.14				0.7
		0.37	10.0	3.7	
1,090	0.60				4.4
		0.89	10.0	8.9	
1,100	1.18				13.3
		1.34	4.0	5.4	
1,104	1.50				18.7
		2.25	6.0	13.5	
1,110	3.00				32.2

Table 18. Elevation-storage curve as used by the program

Elevation, ft	Total volume, ac. ft
1,090	0.0
1,100	8.9
1,104	14.3
1,110	27.8

elevation-storage curve gives flexibility to vary the culvert flowline to determine the effect on headwater elevation by varying storage potential.

A future method of obtaining the elevation-storage curve should be mentioned. This involves use of the digitizer in conjunction with the Kelsh Plotter to produce a deck of punched cards with grid coordinates and an elevation. These cards are presently used to produce a contour map. Additional routines could be added to this existing program to calculate the elevation-storage curve directly. Output from this program could be input to the culvert program and eliminate the need to develop the elevation-storage curve by hand. The ease of adding this capability to the contour map program and the cost of using the program when completed has not been investigated.

Culvert Hydraulics

Equations used

The hydraulics of culvert flow used in the development of the computer program are based on research data used by the Bureau of Public Roads (BPR) in the development of Hydraulic Engineering Circular No. 5 (12). The research data for pipe culverts are contained in two publications (5, 10). Experimental data for box culverts with headwalls and wingwalls were obtained from an unpublished report of the USGS. The data were then reduced to a series of nomographs for easy use by design engineers.

Computer programs were then written by the BPR for design of pipe and box culverts (19, 29). The equations used for determination of headwater were calculated using a least squares polynomial curve fitting computer program. For pipes, the program was used to calculate a 5th degree curve for the data for several culvert models presented in the study by French (10). For box culverts, the coefficients in the equation were fitted to data taken from Chart 12 in Hydraulic

Engineering Circular No. 5. These 5th degree polynomial equations for inlet control conditions are used in the present computerized design method as follows.

For pipes:

$$HW = (DIA)(Y) \tag{29}$$

where

HW = headwater in feet

DIA = pipe diameter in feet

$Y = a + bX + cX^2 + dX^3 + eX^4 + fX^5$

a, b, c, d, e, f = coefficients

$X = Q/(DIA)^{2.5}$

Q = discharge in cfs.

For box culverts:

$$HW = (D)(Y) \tag{30}$$

where

HW = headwater in feet

D = height of box in feet

$Y = a + bX + cX^2 + dX^3 + eX^4 + fX^5$

a, b, c, d, e, f = coefficients

$X = Q/B(D)^{1.5}$

Q = discharge in cfs

B = width of box in feet.

The computer program contains equations for various types of inlets for corrugated metal and reinforced concrete pipes and for reinforced concrete box culverts. The ten options of inlet and culvert type contained in the program are listed in Table 19. These are the types normally used by the ISHC. Pipe arches, either corrugated

Table 19. Inlet types for pipe and box culverts

Number	Inlet type
	Box culverts
1	30 to 75 degree wingwall flare
2	90 or 15 degree wingwall flare
3	Parallel wingwalls
	Reinforced concrete pipe
4	Socket-end projecting
5	Socket-end in a 90 degree headwall
6	Standard end section
	Corrugated metal pipe
7	Projecting from fill
8	Mitered to fill slope
9	90 degree headwall
10	Standard end section
	Weir
11	Drop inlet weir
12	Weir, roadway overtopped

metal or reinforced concrete are not included now, but could be added.

The two weir alternates use an equation of the form:

$$Q = CLH^{1.5} \tag{31}$$

where Q = discharge in cfs

C = coefficient

L = length of weir in feet

H = head on weir in feet.

As presently constructed by the ISHC and SCS, the drop inlet option has a value of 3.7 for the coefficient C. For the roadway overtopped option, the roadway acts as a broad-crested weir and a value of 3.0 is used for C.

Inlet control equations

As stated before, the equations included in the computer program are only for inlet control conditions. No outlet control condition equations have been included for a number of reasons. Experience at the ISHC indicates that outlet control rarely governs. Natural channels in Iowa usually have small in-bank capacities. For most design discharges, the water has overflowed and spread across the valley. In most channels, the tailwater rating curve shows low tailwater depths, even at design discharges.

Two major exceptions are the drainage ditches of north-central Iowa and the draws in the loess region of western Iowa. In both cases, the design discharges normally remain within the channel banks. The tailwater rating curve in these channels is such that the depth of tailwater is greater at the design discharge than for the above cases. However, the channel slopes are steep in western Iowa, which tend to reduce tailwater depth.

The research reported in Hydraulic Engineering Circular No. 5 (12) determined that the governing downstream depth was the larger of either the tailwater depth or the ratio, $(D_c + D)/2$, where D_c is the critical depth and D is either the diameter of the pipe or the height of the box. In most cases in Iowa, the ratio, $(D_c + D)/2$, governs. The most frequent exceptions are confined channels on flat slopes, such as drainage ditches, and the situation of one culvert located just downstream of another. In this case the headwater depth of the downstream culvert becomes the tailwater depth of the upstream culvert. Even in these instances, the fall through the culvert is usually enough to have inlet control govern. Thus, the only instances where inlet control may not govern are those few cases of high tailwater with little or no difference in elevation between the inlet and outlet of the culvert. In these cases, the program results should be used only as an indication of how much the peak inflow might be reduced.

The research report by Young, et al. (33) included two case studies. Methodology included routing floods through the culvert. In both cases, outlet control governed during only a very short initial period of the entire flood which lasted for several hours. The Young study was confined to box culverts. The inlet control equations used in his study were also developed by the BPR. They take a much different form from the fitted polynomial equations used by the BPR in their computer programs. However, for any given size of box culvert, the two sets of equations yield almost identical results.

Thus in almost all cases, highway culvert flow is governed by inlet control. This can also be shown graphically by performance curves, a graph of culvert operation through some range of discharges and barrel slopes for a specific size, type, and length of culvert. Performance curves were developed for several sizes of box culverts (4 x 4 to 20 x 12) and pipe culverts (36 in. to 72 in.). All showed results similar to those for the 6 x 6 box culvert depicted in Fig. 13. Inlet control governs throughout the range of headwater depths and discharges except for culverts on slopes less than one percent with headwater depths less than the height of the culvert.

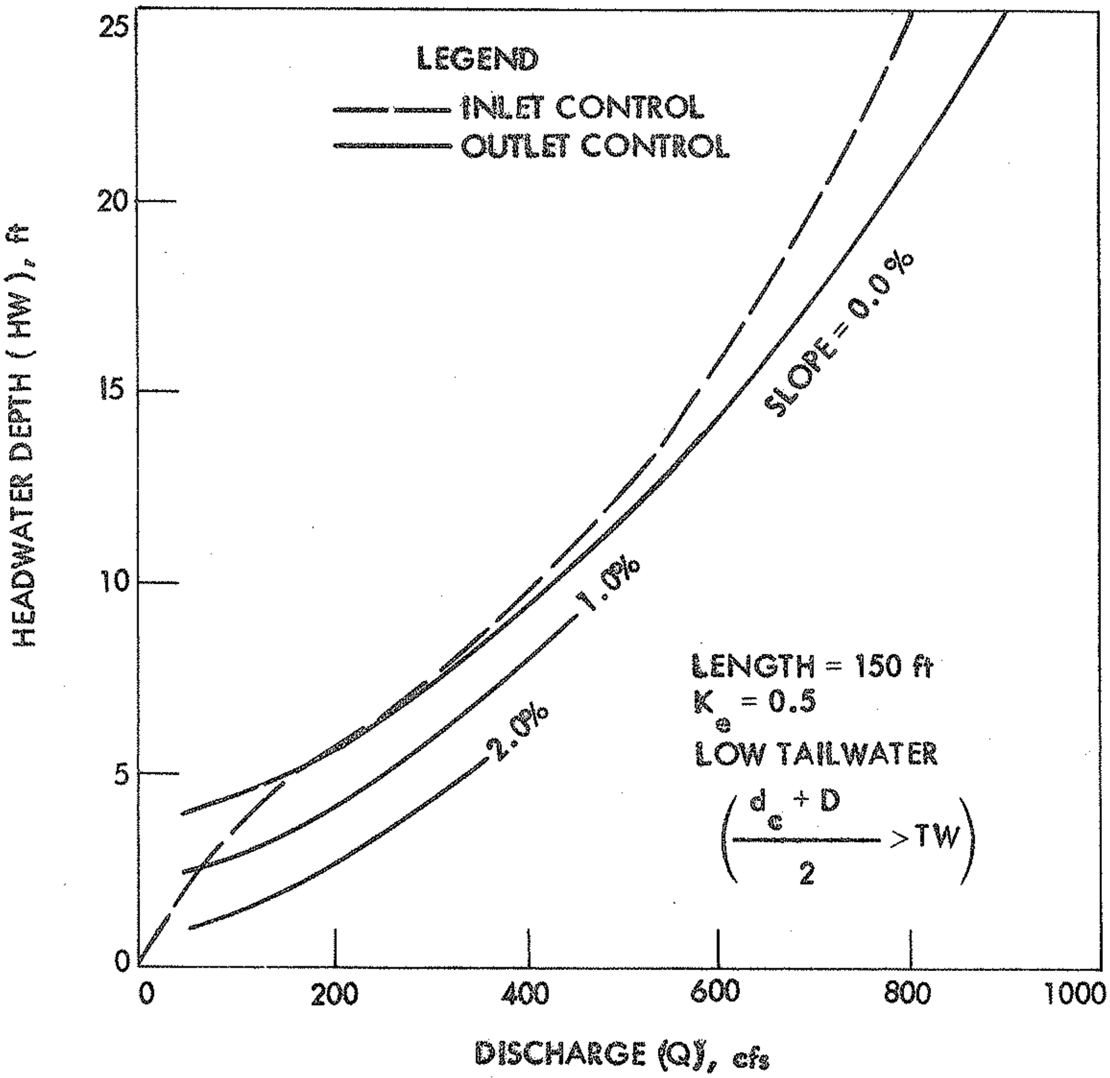

Fig. 13. Hydraulic performance curves for a 6 x 6 box culvert with an ISHC flared headwall.

Scope and capability of computer program

The procedure for determining headwater-discharge relations has been developed to allow the designer a wide choice. Five different outlets of varying types, sizes, elevation, and number can be analyzed at the same time. For instance, a 5 x 5 box culvert at elevation 900 is used in conjunction with a 10 x 5 drop inlet at elevation 912. Additional outflow capacity is provided by twin 42-in. pipes at elevation 918.5. If the water ponds to elevation 925, a side ditch parallel to the roadway will begin to carry water to an adjacent stream. If the water ponds deeper than elevation 930, overtopping of the highway grade will occur.

Assume that the total discharge to elevation 935 is desired. The program begins by calculating the capacity of the 5 x 5 box culvert from elevation 900 to elevation 935 in one foot increments. Then the capacity of the drop inlet is determined from 910 to 935. The discharges for each structure are compared at each elevation and the lower discharge of the two is saved beginning with 0 cfs at elevation 910. Next, the capacity of the twin 36-in. pipes at each elevation from 918.5 to 935 is calculated. Then the discharges flowing down the side ditch from 925 to 935 are determined. Last, the rate the water flows over the highway from elevation 930 to 935 is calculated. At each one foot difference in elevation, beginning at elevation 910 in this example, the total outflow capacity at that elevation is determined by adding together the appropriate discharges from each component weir and/or culvert. Each elevation and total discharge at that elevation is saved in a matrix for future use. A table listing each component discharge and total discharge at each elevation is output.

The more usual situation of a single pipe (60 in. for example) or a twin box culvert (8 x 8 for example) can also be input and calculations made as in the more complex example above. A single run can include as many alternatives as desired, such as varying culvert sizes and types, invert elevations, and/or number of culverts, at as many locations at the culvert site as desired — plus sites in as many different watersheds as desired.

The hydraulic efficiency of various types of inlets and the effects of them on maximum headwater depths will be discussed in a later section.

Flood Routing

The three elements (inflow hydrograph, storage, and culvert hydraulics) are combined in a flood routing routine based on a computer program written by Shearman and Dougal in 1965 (26). The method used is based on two assumptions: the outflow is a function only of the water surface elevation and this water surface is level throughout the temporary pond so that there is a direct relationship between the volume of storage and the water surface elevation.

These three elements are combined in Eq. (32).

$$\text{Outflow} = \text{Inflow} - \text{Change in storage} \qquad (32)$$

For any incremental time period, such as ΔD, this relation satisfies the principle of continuity. If the change in storage is zero (for instance, there is no storage available), then outflow equals inflow. If storage is available, while the inflow is increasing, some of it goes into temporary storage and the outflow is less than the inflow. Later in the flood, the situation is reversed.

The relationship in Eq. (32) can be rewritten as:

Inflow - Outflow = Change in storage

or

$$\bar{I} - \bar{O} = \frac{\Delta S}{\Delta D} \tag{33}$$

where $\bar{I}$ is the average inflow, $\bar{O}$ is the average outflow, and ΔS is the change in storage during some incremental time period ΔD. This is shown pictorially in Fig. 14. Uncontrolled outflow means that the outflow is a function only of the depth of water and the size and shape of the outlet structure. Controlled outflow would involve the incorporation of a movable gate into the outlet structure.

Equation (33) can be rewritten as

$$\frac{I_1 + I_2}{2} - \frac{O_1 + O_2}{2} = \frac{S_2 - S_1}{\Delta D} \tag{34}$$

or

$$\frac{I_1 + I_2}{2} - \frac{O_1 + O_2}{2} = \frac{S_2 K}{\Delta D} - \frac{S_1 K}{\Delta D} \tag{35}$$

where the subscripts 1 and 2 represent the beginning and end of the incremental time period ΔD. Equation (35) is dimensionally correct using the usual units of cfs for the inflow and outflow, hours for time, acre-feet for storage, and K the conversion factor from acre-feet to cfs-hours.

The only unknowns in Eq. (35) are O_2 and S_2, the outflow and storage at the end of the period. The outflow and storage at the

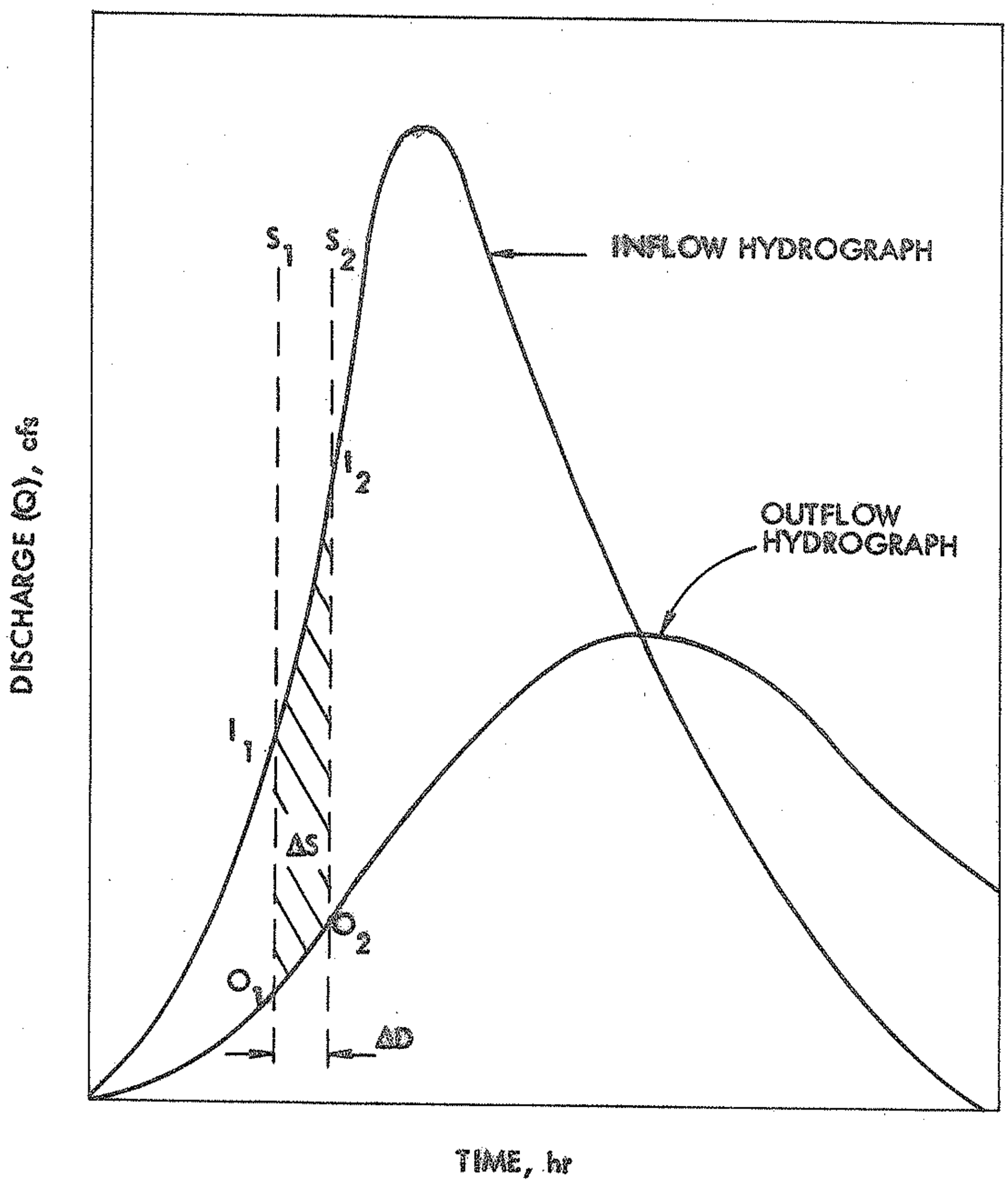

Fig. 14. Reservoir routing with uncontrolled overflow.

beginning are known, as are I_1 and I_2, two adjacent ordinates of the inflow hydrograph. By cross multiplying and arranging all the known elements on the left-hand side, Eq. (35) becomes:

$$(I_1 + I_2) + \left(\frac{2S_1K}{\Delta D} - 0_1\right) = \left(\frac{2S_2K}{\Delta D} + 0_2\right) \tag{36}$$

The right side of Eq. (36) is commonly known as a working curve for end-of-period relationships, and can be related graphically to the known elevation-storage-discharge relationships for a specified outflow scheme.

Thus, for each sequential time period, $\frac{2S_2}{\Delta D} + 0_2$ is obtained using Eq. (36), 0_2 is determined from $\frac{2S}{\Delta D} + 0$ versus 0 relation, and the corresponding reservoir elevation and storage can be determined from the elevation-storage-discharge relation. These relationships and equations are contained within the computer program developed herein. No additional input is required to accomplish the flood routing since the required data have either been input or calculated at an earlier point in the program. The output from this portion of the program includes for each incremental time period: time, inflow rate, outflow rate, amount of storage used, and headwater elevation.

A listing of the computerized design method for culverts, HDC, written in fortran for use on an IBM-360-65 computer is included as Appendix D. A simplified flow chart is included as Appendix F.

PROGRAM INPUT

Previous sections detail the development of the computer program; here, inputing data to the program is described. Again, the input data is kept to a minimum. It consists of four parts: hydrologic data, stage-storage data, identification of alternate data, and hydraulic data. These four sets of data are contained on two input forms which are shown as Fig. 15 and Fig. 16. All items of data are right-justified in their fields and decimal points, where required, are included in the forms.

Hydrologic Data

Required hydrologic data is contained on one input card and consists of eight items. The first five items are record items. The county number is obtained from Table 10. The recurrence interval is given in years, normally 50 for primary and interstate highways and 25 for county highways. A larger number, such as 100, 500, or 1,000, can be input to test the culvert for a larger than design storm. The size of the watershed is input in acres. The land use and slope factor and the frequency factor are obtained from Fig. 1, the chart used by the ISHC to determine peak discharges. The frequency factor assumed for a recurrence interval of 100 years is 1.2 and is 2.0 for a recurrence interval of 1,000 years (based on logarithmic extrapolation).

The eighth item is the number of storage elevations, 1 to 21. This number in column 75 is used only as a flag to the computer to

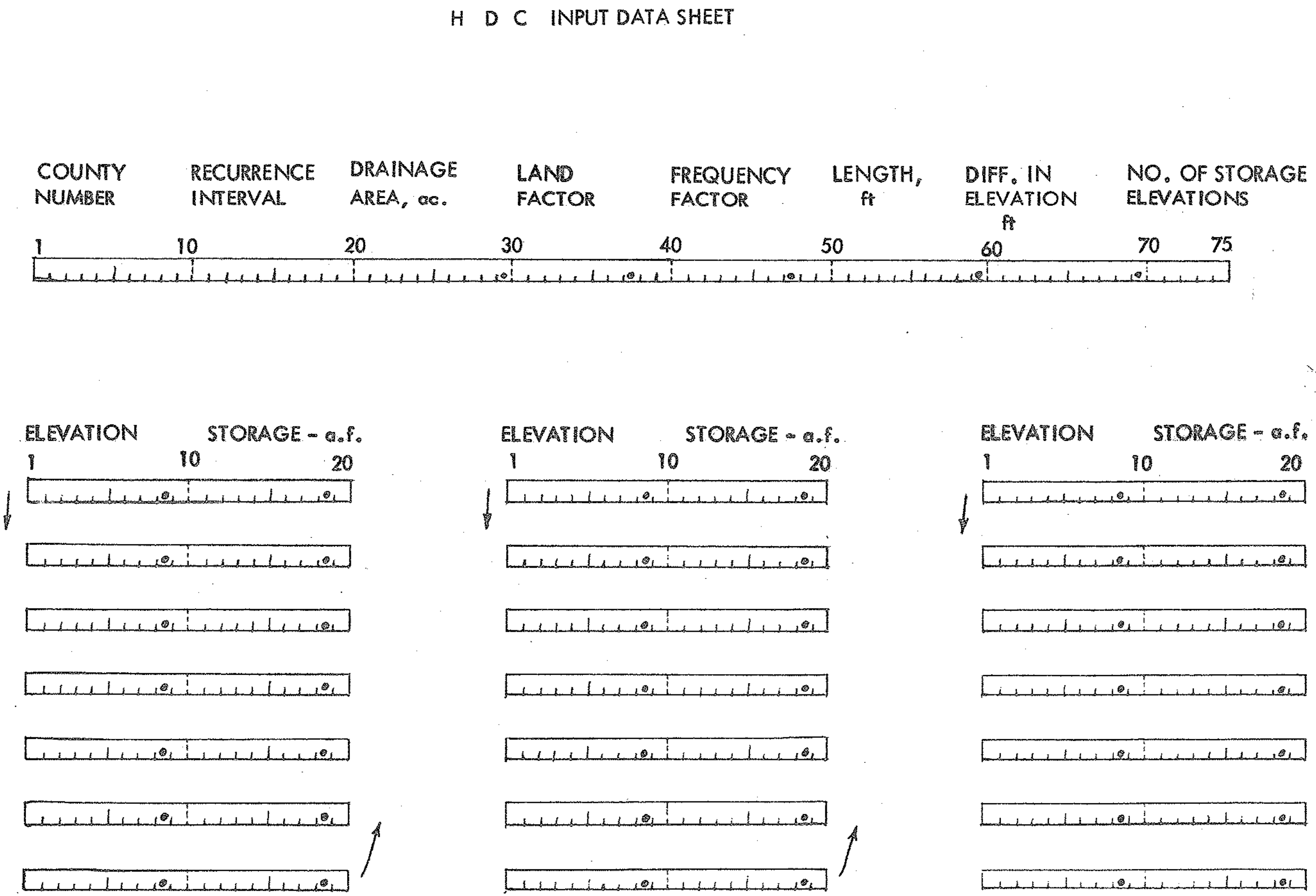

Fig. 15. Program input form - sheet 1.

H D C - INPUT DATA SHEET

COUNTY NUMBER	RECURRENCE INTERVAL	DRAINAGE AREA, ac.	LAND FACTOR	FREQUENCY FACTOR	LENGTH, ft	DIFF. IN ELEVATION, ft	NO. OF STORAGE ELEVATIONS

1 10 20 30 40 50 60 70 75

1 6 10 20 30 40 50 60 65

1 IDENT 1

2 IDENT 2

3 IDENT 3

4 IDENT 4

INLET TYPE	HEADWATER ELEVATION	FLOWLINE ELEVATION	NO. OF PIPES OR BARRELS	BARREL WIDTH, ft	BARREL HEIGHT, ft	PIPE DIAMETER, in.	WEIR LENGTH, ft.

1 5 10 20 30 40 50 60 70 80

1

2

3

4

5

1
☐ DO YOU WANT TO COMPUTE A NEW INFLOW HYDROGRAPH? 0 = NO 1 = YES

1
☐ DO YOU WANT TO PERFORM MORE CALCULATIONS? 0 = NO 1 = YES

Fig. 16. Program input form - sheet 2.

designate how many stage-storage data cards it will be reading. The entry is required only on the first hydrologic data card. If the designer is testing several alternative culverts for the same site, the subsequent hydrologic data cards will have a zero in column 75. This zero acts as another flag to the computer. Its effect is to have the program use the stage-storage data which had previously been input. Typical hydrologic input is shown in Fig. 17.

Items 6 and 7 of the hydrologic data, length and difference in elevation, are the only data requirements not presently used in the culvert design procedure at the ISHC. However, they are required at times in the hydrologic design of bridges. These two items refer to the thalweg of the draw, creek, or stream. Item 6 is the length in feet of the main channel between the culvert site and divide. In the upper reaches of the watershed, the main channel is taken as that branch which has the greatest drainage area. Item 7 is the difference in elevation in feet between that of the divide and the streambed elevation at the culvert site.

These two items of data are obtained from maps currently available in the preliminary bridge section and/or photogrammetry section of the ISHC. Three measures (or numbers) are required for items 6 and 7: streambed elevation at the culvert site, elevation of the divide, and length of the main channel between these two points. The streambed elevation at the culvert site is available from the survey notes for the project, a Kelsh Plotter contour map, or a USGS topographic map. The elevation of the divide is also available from these two types of maps.

H D C - INPUT DATA SHEET

COUNTY NUMBER	RECURRENCE INTERVAL	DRAINAGE AREA, ac.	LAND FACTOR	FREQUENCY FACTOR	LENGTH, ft.	DIFF. IN ELEVATION, ft	NO. OF STORAGE ELEVATIONS
82	50	1140.	0.50	1.00	6200.	65.	6

1 i-ig-280-8(38)294--04-82 des. no. 1569 sta. 486+40	IDENT 1
2 drainage area = 1140ac. - flat to rolling	IDENT 2
3 7.5 min. quad. - davenport west	IDENT 3
4 a single 10 X 10 rcb at elev. 702.0	IDENT 4

	INLET TYPE	HEADWATER ELEVATION	FLOWLINE ELEVATION	NO. OF PIPES OR BARRELS	BARREL WIDTH, ft	BARREL HEIGHT, ft	PIPE DIAMETER, in.	WEIR LENGTH, ft
1	2	1110.	1080.0	1	5.	5.	.	.
2	11	1110.	1090.0		0.	0.	0.	20.
3	6	1110.	1095.0	2	0.	0.	42.	.
4	12	1110.	1099.0		0.	0.	0.	35.
5	12	1110.	1104.0		0.	0.	0.	200.

[0] DO YOU WANT TO COMPUTE A NEW INFLOW HYDROGRAPH? 0 = NO 1 = YES

[1] DO YOU WANT TO PERFORM MORE CALCULATIONS? 0 = NO 1 = YES

Fig. 17. Typical completed input form.

If the drainage area is about 2 sq mi or less, the Kelsh Plotter can be used to obtain all three items by drawing the trace of the stream on a map and listing the two elevations. The stream length is then obtained by measuring the length on the map. Other sources for stream length are USGS quadrangle maps, files of agricultural aerial photos maintained by the photogrammetry section, and various drainage maps prepared by county engineers.

If a USGS quadrangle map is used to determine stream length, then the map scale must be taken into account. The length taken from a 7.5 minute map (a scale of 1:24,000) can be used without correction. As the map scale becomes larger, the meandering which shows on the 7.5 minute map becomes less well defined. The correction factors shown in Table 20 should be used. For example, if the length scaled from a 1:250,000 map is 50,000 ft, then the length input to the computer is 50,000 times 1.3 or 65,000 ft. These factors were determined from the data shown in Table 21.

Table 20. Correction factors for length of stream based on map scale

Size of other map	Scale	Ratio = 7.5 minutes/other
7.5 minutes	1 in. = 2,000 ft	1.00
15 minutes	1 in. = 5,200 ft	1.10
30 minutes	1 in. = 10,400 ft	1.20
1:250,000	1 in. = 20,800 ft	1.30

Table 21. Length ratios for maps other than 7.5 minute quads

1 County	2 Gage	3 Quadrangle Size	4 Length, ft	5 1:250,000 Length, ft	6 Ratio col. 4/col. 5	7 Ratio 7.5/other
Marion	D.A. = 2625	7.5	22,500	17,400	1.29	1.29
Johnson	5-4540	7.5	61,800	47,500	1.30	1.30
Plymouth	6-5998	7.5	28,900	22,600	1.28	1.28
Mills	6-8082	7.5	36,700	28,000	1.31	1.31
Allamakee	5-3884	15	34,200	28,500	1.20	1.08
Greene	5-4830	15	64,200	54,000	1.19	1.09
Allamakee	5-3887	30	9,770	9,200	1.06	1.22

Stage-Storage Data

The method for determining the stage-storage curve is described in a previous section on storage as shown in Table 17. Columns 1 and 6 of Table 17 are entered on the input form shown in Fig. 15. Twenty-one entries for elevation and storage volume are available on the form; normally less than ten entries will adequately describe the storage capability of a site. Only one elevation and the total storage volume below that elevation are listed on each card. Always begin the elevation-storage curve at the elevation at which the natural draw flowline crosses the toe of slope of the highway fill.

Identification Data

Four lines (4 cards) of information to identify the culvert site and alternate under consideration are available in the program as shown in Fig. 16. These four lines are labeled ident1 to ident4 and the information is coded in columns 6 through 65. All four cards must be included. If three lines are sufficient to identify the alternate, then the fourth card need only have the number 4 in column one.

Information that might be included are the designer's name, project number, the design number, the station of the culvert, the drainage area, the type of terrain, the name of the stream if it has one, the type and name of the topo map used, and the type, size, number, and flowline elevation of the culvert, or combination of culverts, used in the particular alternate. An example of identification input is shown in Fig. 17.

Hydraulic Data

Hydraulic data for each alternate is entered on five cards. For twin 48-in. pipes or a single 10 x 10 box culvert, for example, only the appropriate columns on card 1 need be filled in. Then, on cards 2 to 5, a zero in column 10 is required — plus a 2 through 5 on cards 2 to 5 in column 5. The zero in column 10 is a flag to the computer that the card can be bypassed.

Card 2 is reserved for drop inlets. If no drop inlet is used in the alternate, a zero is placed in column 10. If a drop inlet is used, an 11 is placed in columns 9 and 10 and the total length of weir is placed in the proper columns next to the decimal point in column 80. The other columns can be left blank or right-justified zeros can be placed in the other fields.

More complex alternates could require the use of all five cards. For instance, a 5 x 5 box culvert at elevation 1080 is to be used in conjunction with a 10 x 5 drop inlet at elevation 1090. Additional outflow capacity is provided by twin 42-in. pipes at elevation 1095. If the water ponds to elevation 1099, a roadside ditch will begin to carry water to an adjacent stream. If the water ponds deeper than elevation 1104, overtopping of the highway will occur. For this alternate, details of the 5 x 5 box culvert are entered on card 1, the drop inlet on card 2, and the twin 42-in. pipes on card 3. The side ditch will act as a weir and is entered on card 4. The highway itself will also act as a weir and is entered on card 5. These five cards are shown in Fig. 17.

The last two items on the input form are flags to the computer. The first of these questions is whether a new inflow hydrograph is wanted. The usual answer will be no, signified by placing a zero in column 1. The original set of inflow hydrographs is computed from the hydrologic data input to the computer. These hydrographs are stored in memory and are recalled for each alternate analyzed for the site. The only time a yes answer is used is if any of the hydrologic input is changed, or if another watershed in the same or another project is going to be analyzed.

The second question is whether more calculations are to be made. The usual answer will be yes, signified by placing a one in column 1.

The only time a zero is placed in column 1 is if this is to be the last alternate to be analyzed. The zero is a flag to the computer that calculations cease and output terminates following the current alternate.

EXAMPLES OF PROGRAM USE

The following examples illustrate use of the computer program to help determine the type and size of culvert best suited to a particular site. The three examples portray a variety of situations encountered by the highway culvert and bridge designer: bridge obsolescence in hilly Pottawattamie County, drainage for an urbanizing area in Sioux City, and a combination highway and recreational use proposal in Webster County.

Bridge Replacement in Pottawattamie County

The first example is a case study of eleven small county bridges in Pottawattamie County. The recent national bridge inspection program requires the inspection of all bridges constructed with federal funds. The regulations require that bridges be posted with an allowable load limit. Some will have to be replaced in order to carry their intended traffic, for instance, trucks on the farm-to-market system. The inventory and inspection requirements apply to all bridges carrying and going over federally-aided interstate, primary, and secondary highways in every state. The example shows how the potential strain on the county budget can be eased through the use of the computer program (a savings of somewhat over \$115,000 is possible in the replacement of the 11 bridges).

The magnitude of the total problem can be estimated by looking at the difficulties on the local level. Preliminary information was provided in a private communication from a consulting engineer performing inspection of bridges on the secondary road system in Pottawattamie County. There are 238 bridges included on the farm-to-market system in Pottawattamie County. Of these, 62 bridges, or 26 percent, cannot be rated for any truck traffic under current guidelines. While some of these bridges might be upgraded, many of them will need to be replaced. The 238 bridges represent about one-third of the secondary road bridges in the county. It is estimated that the percentage of inadequate bridges on the remaining local system is likely to run much higher than on the county's farm-to-market system. The percentage of existing bridges on potential culvert-sized watersheds may also be greater.

The 62 inadequate bridges are located throughout the county. They range in length from 15 to 83 ft and drain watersheds which vary in size from 36 ac. to 189 sq mi. Only eight of the 62 watersheds are larger than 25 sq mi. An arbitrary sample of eleven smaller watersheds was selected to provide a variety of locations, storage capabilities, and heights from road grade to streambed. These eleven hilly watersheds range in size from 36 to 960 ac. and the existing bridge lengths vary from 19 to 106 ft as shown in Table 22.

Three types of replacement costs were calculated for these eleven sites: replace with bridges of the same length, replace with culverts using the current ISHC design procedure, and replace with culverts using the computerized design method developed in this study. Highway geometric standards and quantity and cost figures were obtained from the Preliminary Bridge Section of the ISHC.

Table 22. Location and size of eleven small bridges in Pottawattamie County

Number	Location sec., twp., range	Length, ft	Width, ft	Drainage area, ac.
1	33-77-43	24	16	36
2	35-76-43	61	18	83
3	29-77-42	106	16	765
4	09-76-42	22	17	315
5	34-75-42	42	16	330
6	27-74-42	38	20	265
7	10-76-41	23	19	215
8	32-76-41	19	20	465
9	01-75-39	61	17	960
10	27-75-39	34	16	325
11	27-75-39	70	16	315

The first type of replacement cost assumes that each of the eleven bridges would be replaced with a 30-ft wide concrete slab bridge of the same length, although actually some of the bridge lengths would have to be increased due to channel degradation and/or erosion of the banks of the channel. Other bridges would be replaced by culverts in order to stop the deepening of the gully caused by channel erosion. However, this assumption of equal length yields an adequate figure for a minimum estimate of total replacement cost.

The second type assumes that each bridge is replaced with a culvert whose size is determined by using the current ISHC design procedure.

This means determining the design discharge, Q_{25}, by using the Peak Rates of Runoff Chart shown in Fig. 1, then using Q_{25} and Hydraulic Engineering Circular No. 5 (12) to determine the correct size and type of culvert. This method restricts the design headwater to the crown of the culvert or up to two feet above the culvert crown, depending upon the structure.

The third type assumes that each bridge is replaced with a culvert whose size is determined by using the computerized design method developed in this study. Several alternate sizes and types of culverts were analyzed using Q_{25}. Those culverts that were tentatively selected were checked using Q_{100}. The author rejected those culverts in which Q_{100} overtopped the existing highway grade. Pertinent elevations for these eleven sites are shown in Table 23. The difference in headwater elevation between current ISHC and computerized design methods is due to the fact that temporary ponding effects are ignored in the current ISHC method which uses only the peak design discharge.

Required structure sizes of the three replacement methods are shown in Table 24. Other culvert sizes and invert elevations were also investigated. Sizes shown in Table 24 for the computerized design method are minimum sizes in order to show maximum possible savings. Additional site or design restrictions may result in using larger culverts.

Only one headwater elevation for the computerized design method is shown in Table 23; however, the effects of seven storm durations are analyzed in the program. The other six analyses on each alternate yield additional data on the effect different peak discharges and

Table 23. Pertinent elevations and storages used at the eleven bridge sites

Number	Streambed elev.	Proposed culvert elev.	Existing highway elev.	Headwater elev., Q_{25}		Storage used, max. ac. ft
				Current method[a]	Computer method[b]	
1	1064.0	1064.0	1072.0	1068.1	1068.0	1.9
2	1080.0	1085.0	1094.0	1090.6	1090.2	8.1
3	1123.0	1135.0	1150.0	1144.0	1143.3	61.2
4	1114.0	1114.0	1125.0	1120.9	1121.5	9.2
5	1084.0	1084.0	1097.0	1091.0	1092.6	28.1
6	1064.0	1067.0	1080.0	1074.8	1074.9	21.4
7	1187.0	1187.0	1197.0	1193.9	1193.2	11.6
8	1210.0	1210.0	1220.0	1217.1	1215.7	36.5
9	1219.0	1219.0	1229.0	1226.2	1225.8	27.7
10	1199.0	1199.0	1205.5	1204.0	1203.3	12.9
11	1171.0	1177.0	1190.0	1183.9	1187.0	27.9

[a]Hydraulic Engineering Circular No. 5 (12).

[b]Computerized design method, Q_{100} does not overtop existing highway grade.

runoff volumes have on maximum headwater elevation. The program output can also be used to determine the length of time the water surface was above a given elevation. If adjacent cropland is inundated once every 25 or 100 years, this time length of inundation will help determine the probability of crop damage. In all eleven cases, total flood duration for the 100-yr event was less than half a day.

Table 24. Structure replacement sizes at the eleven bridge sites

Number	Replacement bridge size	Current ISHC design method	Computerized design method
1	24 x 30	48 in. CMP	36 in. CMP
2	61 x 30	60 in. CMP	36 in. CMP
3	106 x 30	10 x 8 RCB to 8 x 6 RCB	60 in. CMP
4	22 x 30	8 x 6 RCB	6 x 6 RCB
5	42 x 30	8 x 6 RCB	60 in. CMP
6	38 x 30	6 x 6 RCB	60 in. CMP
7	23 x 30	6 x 6 RCB	72 in. CMP
8	19 x 30	10 x 6 RCB	60 in. CMP
9	61 x 30	2 (8 x 6) RCB	2 (8 x 6) RCB
10	34 x 30	3 (60 in.) CMP	3 (60 in.) CMP
11	70 x 30	8 x 6 RCB to 6 x 6 RCB	54 in. CMP

The replacement of bridges with culverts will stop some erosion of the channel upstream. The highway fill will trap some of the sediment carried by flood waters. This will eventually result in lessening the amount of available storage.

The cost of each of the methods is shown in Table 25. There are additional costs when a bridge is replaced with a culvert. The bridge opening must be filled with embankment material and topped with paving. The fill and paving costs shown in Table 25 are total costs for all eleven sites. Comparing the three methods indicates that culverts designed using the current ISHC method would save about $60,000 over

Table 25. Replacement costs at the eleven bridge sites

Number	Bridge	Current method	Computer method
1	$ 10,100	$ 2,140	$ 1,390
2	25,600	3,260	1,490
3	44,500	16,300	3,810
4	9,200	11,700	10,200
5	17,600	13,300	4,060
6	16,000	11,500	4,060
7	9,700	9,500	4,750
8	8,000	12,200	3,460
9	25,600	17,500	17,500
10	14,300	8,400	8,400
11	29,400	11,900	3,430
Subtotal	$210,000	$117,700	$62,550
Fill	—	12,500	13,500
Paving	—	18,300	18,300
Total	$210,000	$148,500	$94,400

bridge replacement costs. Culverts designed by the computerized design method would save about $115,000. When cost comparisons from this sample of eleven bridges are projected to the entire primary and secondary highway system in Iowa, the total potential saving is impressive.

Small Drainage Area in Sioux City

The second example concerns a small watershed of 340 ac. in Sioux City. Here, the deliberate use of available storage reduces the peak outflow discharge to 6.2 percent of the peak inflow rate (to 47 cfs from 762 cfs). This greatly reduced Q can then be safely handled by the existing culverts downstream. This particular solution has been incorporated into the design plans for US Highway 520 in Woodbury County. Typical program output for this example has been included as Appendix C to illustrate the form and types of information output by the program.

The topography of this urbanizing watershed is typical of the steep loess hills found in western Iowa. The amount of available storage is large with respect to the size of the watershed. While the normal response rate of watersheds in this locale is short, urbanization will further decrease the time required for flow to reach maximum. Urbanization will also increase the magnitude of the peak discharge.

The watershed is located adjacent to the Missouri River flood plain and crosses Freeway No. 520 just east of the Highway 520 interchange with Interstate Highway 129. The proposed alignment of Highway 520 crosses the upstream end of an existing pond located in this portion of the watershed. The outlet of the culvert draining the 340 ac. is to be located just upstream of a proposed letdown structure. This structure drains both the 340 ac. watershed located south of Highway 520 and a 40 ac. watershed on the north side. The combined flow drains into the remainder of the existing pond. If the water gets too deep in the pond, letdown structures paralleling Highway 520 safely convey excess water to the Missouri River flood plain.

The normal structure for this 340 ac. urbanizing hilly watershed would be a 10 x 8 reinforced concrete box culvert. Due to the highway section and grade, local topography, and proposed invert elevation, the normal culvert would be about 185 ft long and would cost about \$37,000. The peak outflow from this culvert would necessitate a similarly sized letdown structure into the existing pond and might overtax the existing outlet structure of the pond. The cost of these structures were not determined.

In order to protect the downstream structures, the decision was made to temporarily pond the water upstream of Highway 520 and release it so the downstream structures and the surrounding area would not be endangered. This regulation was accomplished by reducing the size of the outlet structure through the Highway 520 embankment. Several sizes of reinforced concrete pipe were analyzed using the computer program. The results are shown in Table 26. Dikes upstream and downstream of Highway 520 contain the water within the draw to an elevation of 1170.0. The peak inflow rate of 762 cfs is equivalent to the 100-yr flood.

The size selected for use is a 48-in. RCP with the inlet reduced to a 24-in. opening. The 48-in. barrel was used for ease of inspection and maintenance. This culvert is about 225 ft long and costs about \$8,300. The peak inflow of 762 cfs is reduced to a peak outflow of 47 cfs with a maximum water surface elevation of 1168.6. Even when the peak inflow is increased to 919 cfs, the peak outflow is reduced to 57 cfs with a maximum water surface elevation of 1168.9, still a foot below the top of the dike. The 21 percent increase in peak inflow

Table 26. Pertinent elevations, peak inflow and outflow rates, and storages used for various sizes of pipe culverts for a 340 ac. watershed in Sioux City[a]

Pipe diam., in.	Inflow rate, max., cfs	Outflow rate, max., cfs	Headwater elev., max., ft, MSL	Storage used, max., ac. ft
54	762	224	1165.4	56.1
48	762	187	1165.9	60.0
42	762	151	1166.4	64.4
36	762	117	1167.0	69.1
30	762	85	1167.7	74.3
24	762	47	1168.6	82.1
24	919	57	1168.9	84.1

[a]Existing streambed elev. = 1141.7
Proposed invert elev. = 1155.0
Minimum highway elev. = 1175.4.

is stored in an additional 4-in. depth of water. This reduction in Q is shown graphically in Fig. 18.

Large Drainage Area in Webster County

The third example shows what might be done on two adjacent watersheds (15,000 and 1,800 ac.) on proposed US Highway 520 in Webster County. The cost of the presently proposed culverts for Highway 520 and an adjacent county road is just under $1,000,000. The proposed solution, based on output from the computerized design method, has a culvert, riprap, and land cost of just under $540,000. Thus, there is

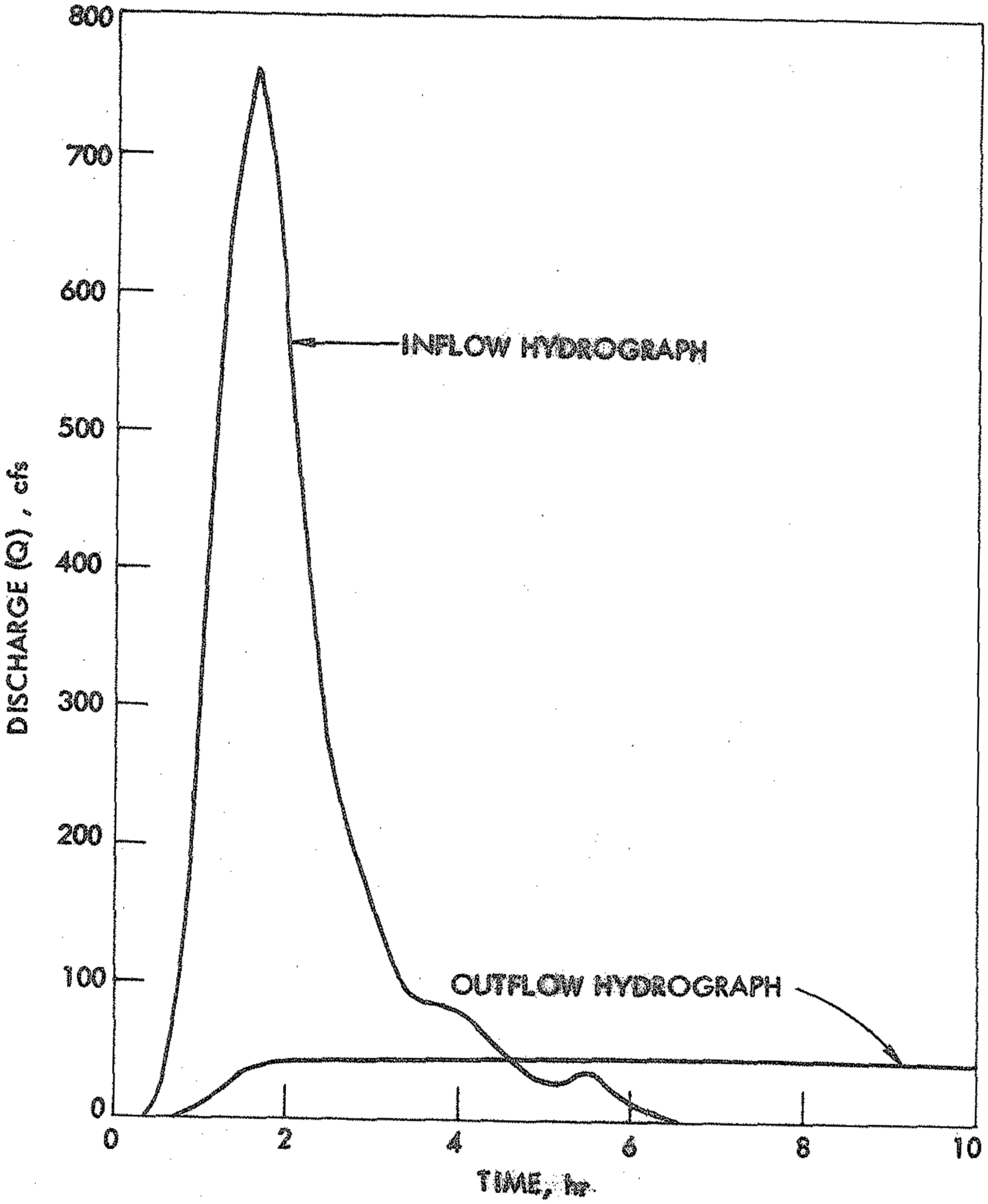

Fig. 18. Inflow and outflow hydrographs for a 340 ac. watershed in Woodbury County.

a potential savings of about \$460,000. The proposed solution creates two small lakes and if used for recreation by the county, other costs would be incurred. These other costs may be in the range of \$50,000 to \$100,000. In addition, before this solution is accepted, the technical, social, environmental, and institutional impacts, both positive and negative, must be assessed.

These two watersheds of 23.4 sq mi and 2.8 sq mi are located mostly in pothole terrain in Webster County. They are different in that their channels have dug into the flat and poorly drained area of north central Iowa in order to meet the streambed of the Des Moines River almost two hundred feet below the upland surface. The proposed alignment of Freeway 520 cuts across these two creeks as they plunge down to the Des Moines River. Farmland here is usually drained by a combination of tile lines and surface drainage ditches. Those portions of the watershed upstream of the proposed highway, 1,000 and 300 ac., consist principally of deeply incised channels and are very hilly. These deep narrow valleys provide little storage volume in relation to the size of the watersheds. An existing county road is located parallel to and just downstream of the proposed Highway 520 alignment.

The proposed culverts for these two watersheds have been designed using current ISHC design procedure. For the 15,000 ac. watershed, a single 24 × 26 reinforced concrete arch culvert has been proposed. It would be 422 ft long through Highway 520 and 164 ft long through the county road. The two sections would be joined by a 46-ft section of open rectangular channel. The invert elevations of the proposed arch culverts match the existing streambed, about 67 ft below the proposed highway grade. For the 1,800 ac. watershed, a 12 × 10 reinforced concrete box culvert tapered from a 16 x 10 inlet has been proposed. This culvert would be 455 ft long through Highway 520 and would have its inlet raised 16 ft above the existing streambed. The proposed highway grade is an average of 65 ft above the existing streambed. The outlet of the culvert would be constructed so that the water would flow down the ditch between Highway 520 and the existing county road. The ditch section between the culvert outlet and the rectangular channel would be armored with riprap. The estimated cost for all of the above structures is just under $1,000,000.

The computerized design method was used to examine this estimated structure cost. Several alternative sizes of pipe and box culverts were analyzed for both of the watersheds with the inlets raised 30 ft above the existing streambed. The inlets were raised for two purposes. First, structure cost would be less because the culverts would be shorter and a lighter section could be used due to the decreased fill height over the structure. Second, if the inlet is raised, more storage volume becomes available for depths of one to two times the height of the culvert.

The initial computer runs showed that each watershed was in effect acting as two separate watersheds. Runoff from the steep portion near the highway would gather quickly and flow through the culvert. This water would recede before the runoff from the flat portion arrived at the culvert. As an example, for a storm duration of ten hours on the 15,000 ac. watershed, the runoff from the 1,000 ac. portion reached a peak of 1,225 cfs two hours after the beginning of the storm, had receded to 133 cfs at time equal to four hours, and to almost zero at ten hours. For the 14,000 ac. flat portion, inflow to the culvert was zero at two hours and 128 cfs at four hours. The peak of 3,630 cfs did not arrive until twelve hours after the beginning of the storm. This ten hour storm had a recurrence interval of 50 years. Total rainfall and runoff were about 5.0 and 2.5 in., respectively.

Summaries of the computer analyses are shown in Tables 27 and 28. The studies indicated that the culverts on both watersheds could be considerably reduced in size without any adverse effects. A tentative selection of final sizes are a 10 x 10 box culvert tapered from a 16 x 12 inlet on the larger watershed and a 4 x 4 box culvert tapered

Table 27. Pertinent elevations, peak inflow and outflow rates, and storages used for various sizes of box culverts for a 15,000 ac. watershed in Webster County[a]

Box culvert size		Inflow rate, max., cfs	Outflow rate, max., cfs	Headwater elev., max., ft, MSL	Storage used, max., ac. ft
Inlet	Barrel				
Twin 20 x 12	Twin 12 x 12	3,939	3,853	1031.2	274
20 x 12	12 x 12	3,939	3,687	1038.9	511
16 x 12	10 x 10	3,939	3,476	1042.6	661
12 x 12	8 x 8	3,939	3,120	1048.3	920
16 x 12	10 x 10	3,386	3,124	1040.0	547
12 x 12	8 x 8	3,386	2,842	1045.0	773

[a]Existing streambed elev. = 990.0
Proposed invert elev. = 1020.0
Minimum highway elev. = 1057.6.

Table 28. Pertinent elevations, peak inflow and outflow rates, and storages used for various sizes of pipe and box culverts on an 1,800 ac. watershed in Webster County[a]

Culvert size		Inflow rate, max., cfs	Outflow rate, max., cfs	Headwater elev., max., ft, MSL	Storage used, max., ac. ft
Inlet	Barrel				
16 x 10	12 x 10	1,273	1,187	1039.4	70
6 x 6	5 x 5	1,273	693	1049.8	176
5 x 5	4 x 4	806	463	1047.9	156
60 in.	48 in.	806	413	1049.6	174
54 in.	42 in.	806	353	1051.0	193
48 in.	36 in.	1,273	319	1056.9	280
5 x 5	4 x 4	1,153	527	1052.1	209

[a]Existing streambed elev. = 1000.0
Proposed invert elev. = 1030.0
Minimum highway elev. = 1062.0.

from a 5 x 5 inlet on the smaller watershed. The total cost of these structures is estimated to be just over $250,000. Because the inlets are each raised 30 ft, two ponds are created. These would be about 21 and 6 ac. in size. To provide land for these ponds, 100 ac. were assumed to be purchased at a cost of $100,000. A unit cost of $1,000 per acre was assumed because this land is somewhat desirable for rural homesites. In order to protect the upstream slope of the highway from erosion due to wave action, a portion of the slope was armored with a 3-ft thickness of riprap. The cost of the riprap was estimated to be $189,000.

Thus, the total cost of this alternate is about $540,000. The estimates for land and riprap were deliberately high to cover other costs which might be involved. The potential savings for these two adjacent watersheds, then, is about $460,000. Before this solution is accepted, however, the technical, social, environmental, institutional, and legal impacts, both positive and negative, must be evaluated. Also, additional computer runs should be made for other invert elevations and culvert sizes to determine the best combination for each of the two sites.

Summary

These three examples show the capability of the computerized design method to reduce culvert costs without adverse effects. Table 29 summarizes the costs of the three examples using the current ISHC design method and the computer program design method. The examples reported here show a total potential saving of about $543,000. Many other sites throughout Iowa were analyzed during the course of the study and potential savings were found in almost all of them.

Table 29. Comparative construction costs of the three examples

Example	Current ISHC design method	Computer program design method
1	$ 148,500	$ 94,400
2	37,000	8,300
3	1,000,000	540,000
Total	$1,185,500	$642,700

SENSITIVITY OF HEADWATER DEPTH TO VARIOUS PARAMETERS

In order to determine how accurately input items need to be measured and how sensitive program results are to several internal items, the sensitivity of headwater depth to various parameters was studied. Headwater depth was selected as the criterion for comparison purposes because in most instances, it is the determining factor on which a specific culvert size and type is either accepted or rejected. The parameters studied were length of main channel, difference in elevation between the watershed divide and the streambed at the culvert site, recurrence interval, culvert inlet efficiency, culvert size, time distribution of rainfall, runoff volume, value of SCS runoff curve number, and volume of storage. Volume of storage is discussed last but it appears to have the greatest effect on headwater depth. Also, at those sites which have a large volume of storage, there is less effect of the other parameters on headwater depth.

Length of Channel and Difference in Elevation

The time of concentration of a watershed is related to the length of main channel and difference in elevation between the watershed divide and the streambed at the culvert site by

$$T_c = \left[\frac{11.9(L/5280)^3}{H}\right]^{0.386} \tag{37}$$

where T_c = time of concentration in hours

L = length of main channel in feet

H = difference in elevation defined above in feet.

This equation was developed by the California Division of Highways (16).

The change in time of concentration T_c caused by a change in the height H or length L is determined by substituting varying percentages of the height and length into Eq. (37). The results from several percentages are shown in Table 30. A 40 percent increase in height results in a 12 percent decrease in time of concentration, while a 40 percent decrease in height results in a 22 percent increase in time of concentration. Conversely, a 40 percent increase in length results in a 48 percent increase in time of concentration, while a 40 percent decrease in length results in a 45 percent decrease in time of concentration. Based on the change in time of concentration, the length requires more accuracy in measurement than the height. If both are in error, the time of concentration for gage 5-3884 is 0.96 hours with $0.6 \times L$ and $1.4 \times H$ and is 3.58 hours for $1.4 \times L$ and $0.6 \times H$.

The effect of these changes in time of concentration on headwater depth was also determined. Maximum headwater depths for various combinations of height and length, based on computer output, are shown in Table 31. The percentage change in depth, using the values from Table 31 and the headwater depth for the L, H combination as the base, is shown in Table 32. Several observations can be made from the data shown in Tables 31 and 32. An over-estimation of height causes an increase in depth while an underestimation causes a decrease in depth. The opposite is true of length. An over-estimation of height has less effect on depth than an underestimation. The same

Table 30. *Time of concentration for several values of height and length*

Location[a]	Time of concentration, hours, for indicated combination								
	L, H	L, 1.4 H	L, 1.2 H	L, 0.8 H	L, 0.6 H	1.4 L, H	1.2 L, H	0.8 L, H	0.6 L, H
5-3884	1.99	1.75	1.85	2.17	2.42	2.94	2.46	1.54	1.10
5-4553	4.56	4.01	4.25	4.97	5.55	6.73	5.63	3.52	2.53
6-8082	2.77	2.43	2.58	3.02	3.38	4.09	3.43	2.14	1.53
5-4540	5.93	5.20	5.52	6.46	7.22	8.75	7.32	4.58	3.28
6-6105	2.00	1.76	1.86	2.18	2.44	2.96	2.47	1.55	1.11
I-380	0.35	0.31	0.33	0.38	0.43	0.52	0.43	0.27	0.19
I-74	0.21	0.18	0.19	0.22	0.25	0.30	0.25	0.16	0.11
F-520	7.01	6.16	6.54	7.64	8.54	10.33	8.67	5.41	3.88

[a]Locations of these gaging stations or projects are listed in Appendix E.

Table 31. Maximum headwater depths for several values of height and length

Location	Maximum headwater depth, feet, for indicated combination								
	L, H	L, 1.4 H	L, 1.2 H	L, 0.8 H	L, 0.6 H	1.4 L, H	1.2 L, H	0.8 L, H	0.6 L, H
5-3884	12.7	12.9	12.8	12.6	12.0	11.1	12.0	14.2	15.3
5-4553	12.5	12.8	12.8	12.6	12.3	11.9	12.3	13.0	13.2
6-8082	13.0	13.1	12.8	12.4	12.4	11.6	12.0	12.9	14.5
5-4540	19.8	20.1	19.7	18.9	18.2	17.0	18.3	20.0	22.2
6-6105	15.4	15.8	15.4	15.4	14.0	14.0	14.0	15.7	17.4
I-380	4.7	4.7	4.7	4.6	4.7	4.7	4.5	4.9	4.7
I-74	3.0	3.0	3.0	3.0	3.0	2.8	3.0	2.9	2.8
F-520	11.7	12.3	12.3	11.5	11.2	11.7	11.3	13.1	16.6

Table 32. Change in headwater depth for several values of height and length

Location	Percent change in headwater depth for indicated combination								
	L, H[a]	L, 1.4 H	L, 1.2 H	L, 0.8 H	L, 0.6 H	1.4 L, H	1.2 L, H	0.8 L, H	0.6 L, H
5-3884	0.0	1.6	0.8	0.8	5.5	12.6	5.5	11.8	20.4
5-4553	0.0	2.4	2.4	0.8	1.6	4.8	1.6	4.0	5.6
6-8082	0.0	0.8	1.5	4.6	4.6	10.8	7.7	0.8	11.5
5-4540	0.0	1.5	0.5	4.5	8.1	14.2	7.6	1.0	12.1
6-6105	0.0	2.6	0.0	0.0	9.1	9.1	9.1	2.0	13.0
I-380	0.0	0.0	0.0	2.1	0.0	0.0	4.3	4.3	0.0
I-74	0.0	0.0	0.0	0.0	0.0	6.7	0.0	3.3	6.7
F-520	0.0	5.1	5.1	1.7	4.3	0.0	3.4	12.0	41.9

[a]Standard for comparison.

is true of length. In almost all cases, the percentage change in depth is less than the percentage change in height or length. At those locations that have a large amount of storage (5-4553, 5-4540, I-380, I-74), the effect of a change in height or length is greatly decreased.

Recurrence Interval

One advantage of the proposed computerized design method is that it can be used to determine the effect on headwater depth of a greater than design flood. If the effects are adverse (water over the roadway, flooding of homes or crops), the designer can weigh the cost of a larger culvert against the cost of additional flood damage. Highway culverts on the primary road system in Iowa are normally designed for a 50-yr recurrence interval, while culverts on the secondary system are designed for a 25-yr recurrence interval. Both 100-yr and 500-yr recurrence intervals were arbitrarily input to the computer program to determine the effect of these larger floods on headwater depth. This two- and ten-fold increase in recurrence interval causes an increase of about 20 and 70 percent, respectively, in the peak discharge rate. The results are shown in Tables 33 and 34.

The percentages shown in Table 34 exhibit great variability. The only general conclusion evident in the results is that the depth of water increases when the recurrence interval increases, a logical result. The variability is due to a number of factors, mainly culvert size and volume of storage. Present design criteria limits headwater depth to a maximum of two feet above the culvert crown for Q_{50}.

Table 33. Headwater depths for various recurrence intervals

County	Drainage area, ac.	Culvert size	Headwater depth, ft, for indicated recurrence interval 50-yr	100-yr	500-yr
Fremont	4,900	2 (12 x 10)	13.0	16.2	21.1
Tama	850	8 x 8	7.6	10.6	13.3
Johnson	38	36 in.	0.8	0.8	1.1
Johnson	223	8 x 6	7.0	8.3	10.7
Scott	43	48 in.	3.0	3.3	4.4
Johnson	600	8 x 8	7.2	8.1	10.1
Black Hawk	7,425	3 (16 x 8)	7.9	9.0	9.7
Pottawattamie	765	5 x 5	9.3	10.5	13.3
Pottawattamie	325	3 (60 in.)	4.7	5.8	7.1
Webster	15,000	3 (16 x 12)	10.4	11.3	13.4
Pottawattamie	36	30 in.	6.0	6.3	7.3
Pottawattamie	83	48 in.	5.2	5.6	6.5
Pottawattamie	330	60 in.	9.2	11.2	14.2
Pottawattamie	265	60 in.	7.8	11.2	12.5
Pottawattamie	465	72 in.	6.5	7.4	9.9
Pottawattamie	960	2 (8 x 6)	7.4	9.2	10.9

Six of the culverts in Table 33 exceed this limit. Advantage is being taken in these six of some of the available storage by using a smaller culvert. The two triple box culverts have extra width introduced deliberately to keep the headwater depth to a minimum. This additional

Table 34. Change in headwater depth for various recurrence intervals

County	Drainage area, ac.	Culvert size	Percent change for indicated recurrence interval		
			50-yr[a]	100-yr	500-yr
Fremont	4,900	2 (12 x 10)	0.0	24.6	62.4
Tama	850	8 x 8	0.0	39.5	75.0
Johnson	38	36 in.	0.0	0.0	37.5
Johnson	223	8 x 6	0.0	18.6	52.9
Scott	43	48 in.	0.0	10.0	46.7
Johnson	600	8 x 8	0.0	12.5	40.3
Black Hawk	7,425	3 (16 x 8)	0.0	13.9	25.3
Pottawattamie	765	5 x 5	0.0	12.9	43.0
Pottawattamie	325	3 (60 in.)	0.0	23.4	51.1
Webster	15,000	3 (16 x 12)	0.0	8.7	28.8
Pottawattamie	36	30 in.	0.0	5.0	21.7
Pottawattamie	83	48 in.	0.0	7.7	25.0
Pottawattamie	330	60 in.	0.0	21.8	54.4
Pottawattamie	265	60 in.	0.0	43.6	60.2
Pottawattamie	465	72 in.	0.0	13.9	52.4
Pottawattamie	960	2 (8 x 6)	0.0	24.4	47.3

[a]Standard for comparison.

culvert capacity helps to reduce the percentage change in headwater depth. The invert of the pipe for the 38 ac. watershed in Johnson County was raised several feet above the existing streambed. The amount of storage available at this elevation is so great that the water gets only a foot deep even at large recurrence intervals. Storage is also being used to good effect at the Scott County culvert.

Culvert Inlet Efficiency

Research has shown that the hydraulic efficiency of the culvert inlet has an effect on flow capacity (5, 10). Ten inlet types for box and pipe culverts are included in the computer program and are listed in Table 19. Table 35 shows the variation in discharge for various types of RCP inlets. Table 36 lists the same information for CMP inlets and box culvert inlets are listed in Table 37. As shown in these three tables, the variation in discharge becomes larger as the depth increases. This variation ranges from 0 to 12 percent for the RCP culverts listed in Table 35, from 4 to 23 percent for the CMP culverts listed in Table 36, and from 0 to 18 percent for the box culverts shown in Table 37.

The overall effect of a change in inlet type on headwater depth is minor. The variation in headwater depth caused by a change in inlet type is shown in Tables 38, 39, and 40 for RCP, CMP, and box culverts, respectively. The greatest percent variation for the culverts studied was 2 percent for the RCP culverts, 10 percent for the CMP culverts, and 13 percent for the box culverts. The greatest change in depth was 0.1 ft for the RCP culverts, 0.6 ft for the CMP culverts, and 1.5 ft for the box culverts. Each of these maximum values occurred

Table 35. Discharge capacity of the RCP culverts studied using different inlet types operating with inlet control

County	Drainage area, ac.	Culvert size	Depth ft	Discharge, cfs, of culvert: Socket-end projecting	Socket-end in 90° hdwl.	Standard end section
Pottawattamie	325	3 (60 in.)	3.0	172	174	174
Pottawattamie	325	3 (60 in.)	5.0	412	416	395
Pottawattamie	325	3 (60 in.)	10.0	847	866	786
Woodbury	340	24 in.	4.0	29	29	27
Woodbury	340	24 in.	8.0	45	47	42
Woodbury	340	24 in.	12.0	56	58	53
Pottawattamie	315	84 in.	4.0	123	123	124
Pottawattamie	315	84 in.	7.0	319	322	305
Pottawattamie	315	84 in.	14.0	655	670	608
Scott	160	60 in.	3.0	58	58	58
Scott	160	60 in.	5.0	137	138	132
Scott	160	60 in.	10.0	282	289	262
Pottawattamie	465	72 in.	6.0	217	219	208
Pottawattamie	465	72 in.	12.0	445	456	413

Table 35. Continued

County	Drainage area, ac.	Culvert size	Depth ft	Discharge, cfs, of culvert		
				Socket-end projecting	Socket-end in 90^{o} hdwl.	Standard end section
Pottawattamie	465	72 in.	18.0	590	614	548
Johnson	151	48 in.	4.0	79	79	75
Johnson	151	48 in.	8.0	162	165	150
Johnson	151	48 in.	11.0	202	209	188

Table 36. Discharge capacity of the CMP culverts studied using different inlet types operating with inlet control

County	Drainage area, ac.	Culvert size	Depth ft	Discharge, cfs, of culvert: Projecting from fill	Mitered to fill slope	90° headwall	Standard end section
Pottawattamie	325	3 (60 in.)	3.0	150	162	171	174
Pottawattamie	325	3 (60 in.)	5.0	334	368	384	395
Pottawattamie	325	3 (60 in.)	10.0	664	690	773	786
Woodbury	340	24 in.	4.0	22	23	26	27
Woodbury	340	24 in.	8.0	35	38	41	42
Woodbury	340	24 in.	12.0	43	47	52	53
Pottawattamie	315	84 in.	4.0	107	115	122	124
Pottawattamie	315	84 in.	7.0	259	285	297	305
Pottawattamie	315	84 in.	14.0	514	533	598	608
Scott	160	60 in.	3.0	50	54	57	58
Scott	160	60 in.	5.0	111	123	128	132
Scott	160	60 in.	10.0	221	230	258	262
Pottawattamie	465	72 in.	6.0	176	194	202	208
Pottawattamie	465	72 in.	12.0	349	363	407	413

Table 36. Continued

County	Drainage area, ac.	Culvert size	Depth ft	Discharge, cfs, of culvert: Projecting from fill	Mitered to fill slope	90^{o} headwall	Standard end section
Pottawattamie	465	72 in.	18.0	457	481	538	548
Johnson	151	48 in.	4.0	64	70	73	75
Johnson	151	48 in.	8.0	127	132	148	150
Johnson	151	48 in.	11.0	157	165	185	188

Table 37. Discharge capacity of the box culverts studied using different inlet types operating with inlet control

County	Drainage area, ac.	Culvert size	Depth ft	Discharge, cfs, of culvert		
				30° to 75° wingwalls	90° or 15° wingwalls	Parallel wingwalls
Webster	15,000	2 (20 × 12)	6.0	1,747	1,481	1,503
Webster	15,000	2 (20 × 12)	12.0	4,834	4,267	4,244
Webster	15,000	2 (20 × 12)	18.0	7,704	7,037	6,727
Webster	1,800	16 × 8	4.0	380	322	327
Webster	1,800	16 × 8	8.0	1,052	929	924
Webster	1,800	16 × 8	16.0	2,140	1,993	1,881
Pottawattamie	960	2 (8 × 6)	3.0	247	209	213
Pottawattamie	960	2 (8 × 6)	6.0	684	603	600
Pottawattamie	960	2 (8 × 6)	12.0	1,390	1,294	1,222
Mills	6,780	3 (10 × 10)	5.0	997	845	858
Mills	6,780	3 (10 × 10)	10.0	2,758	2,435	2,422
Mills	6,780	3 (10 × 10)	15.0	4,396	4,015	3,838
Tama	850	10 × 10	5.0	332	282	286
Tama	850	10 × 10	10.0	919	812	807

Table 37. Continued

County	Drainage area, ac.	Culvert size	Depth ft	Discharge, cfs, of culvert: 30° to 75° wingwalls	Discharge, cfs, of culvert: 90° or 15° wingwalls	Discharge, cfs, of culvert: Parallel wingwalls
Tama	850	10 x 10	15.0	1,465	1,338	1,279
Pottawattamie	765	5 x 5	5.0	163	143	143
Pottawattamie	765	5 x 5	10.0	330	308	290
Pottawattamie	765	5 x 5	15.0	437	413	387
Johnson	1,406	8 x 8	4.0	190	161	164
Johnson	1,406	8 x 8	8.0	526	465	462
Johnson	1,406	8 x 8	11.0	770	698	671

Table 38. Variation in headwater depth using RCP culverts with different inlet types

County	Drainage area, ac.	Culvert size	Headwater depth, ft, for inlet type: Socket-end projecting	Socket-end in 90° hdwl.	Standard end section
Pottawattamie	325	3 (60 in.)	4.8	4.8	4.9
Woodbury	340	24 in.	7.5	7.5	7.6
Pottawattamie	315	84 in.	7.2	7.2	7.3
Scott	160	60 in.	4.6	4.6	4.6
Pottawattamie	465	72 in.	6.3	6.4	6.4
Johnson	151	48 in.	4.6	4.6	4.7

Table 39. Variation in headwater depth using CMP culverts with different inlet types

County	Drainage area, ac.	Culvert size	Headwater depth, ft, for inlet type: Projecting from fill	Mitered to fill slope	90° headwall	Standard end section
Pottawattamie	325	3 (60 in.)	5.4	5.1	5.0	4.9
Woodbury	340	24 in.	7.9	7.8	7.6	7.6
Pottawattamie	315	84 in.	7.9	7.6	7.4	7.3
Scott	160	60 in.	4.7	4.7	4.6	4.6
Pottawattamie	465	72 in.	6.7	6.5	6.4	6.4
Johnson	151	48 in.	4.9	4.8	4.7	4.7

Table 40. Variation in headwater depth using box culverts with different inlet types

County	Drainage area, ac.	Culvert size	Headwater depth, ft, for inlet type: 30° to 75° wingwalls	90° or 15° wingwalls	Parallel wingwalls
Webster	15,000	2 (20 x 12)	10.7	11.7	11.7
Webster	1,800	16 x 8	8.3	9.0	9.0
Pottawattamie	960	2 (8 x 6)	7.4	8.0	8.1
Mills	6,780	3 (10 x 10)	11.9	13.0	13.4
Tama	850	10 x 10	7.9	8.3	8.3
Pottawattamie	765	5 x 5	9.0	9.3	9.4
Johnson	1,406	8 x 8	8.7	9.1	9.1

at sites which had only a small volume of storage. At those sites which had large storage volumes, the effect of inlet type was decreased.

Culvert Size

The size of culvert used at a specific site has a marked effect on headwater depth. Here again, however, at those sites which have a large volume of storage, the effect of a reduction in culvert size on headwater depth is lessened. Table 41 lists the variation and percentage change in headwater depth caused by a change in culvert size at sites with little storage. Table 42 lists the same information at sites with large storage volumes.

Table 41. Variation in headwater depth due to change in culvert size at sites with small storage volumes

County	Drainage area, ac.	Culvert size	Depth ft	Culvert size	Depth ft	Percent change Size	Percent change Depth
Mills	6,780	3 (10 x 10)	14.0	2 (10 x 10)	20.1	50	44
Fremont	4,900	2 (12 x 12)	13.0	2 (8 x 8)	21.3	93	64
Johnson	2,160	2 (10 x 8)	8.3	2 (8 x 8)	9.5	25	14
Johnson	15,740	3 (12 x 12)	16.2	2 (12 x 12)	19.8	50	22
Johnson	1,930	2 (10 x 8)	7.3	2 (8 x 8)	8.2	25	12
Johnson	2,050	2 (12 x 8)	7.4	2 (10 x 8)	8.2	20	11
Pottawattamie	5,110	2 (12 x 10)	15.4	2 (8 x 10)	22.2	50	44
Webster	1,800	16 x 12	9.4	6 x 6	19.8	433	111
Webster	15,000	2 (20 x 12)	11.2	20 x 12	18.9	100	69
Pottawattamie	960	2 (8 x 6)	6.8	8 x 8	9.7	50	43
Pottawattamie	325	3 (60 in.)	4.3	2 (54 in.)	6.2	60	44

Table 42. Variation in headwater depth due to change in culvert size at sites with large storage volumes

County	Drainage area, ac.	Culvert size	Depth ft	Culvert size	Depth ft	Percent change Size	Percent change Depth
Poweshiek	7,360	2 (12 x 10)	11.5	2 (8 x 8)	13.0	88	13
Johnson	990	10 x 8	6.6	6 x 8	7.8	67	18
Johnson	600	8 x 8	7.2	6 x 6	8.1	78	12
Tama	850	10 x 10	7.1	8 x 8	7.6	56	7
Johnson	151	5 x 5	4.3	48 in.	4.8	98	12
Johnson	38	4 x 4	0.8	36 in.	0.8	125	0
Johnson	1,406	12 x 10	7.8	8 x 8	9.1	88	17
Scott	43	4 x 5	2.8	48 in.	3.0	59	7
Scott	160	6 x 6	4.2	60 in.	4.6	84	10
Scott	40	48 in.	3.3	36 in.	3.7	78	12
Scott	173	4 x 5	6.5	36 in.	8.1	182	25
Woodbury	40	48 in.	4.9	24 in.	7.6	300	55
Woodbury	340	54 in.	10.4	24 in.	13.5	406	30
Pottawattamie	765	6 x 6	7.6	60 in.	8.3	84	9
Pottawattamie	330	84 in.	7.3	54 in.	9.3	142	27
Pottawattamie	465	8 x 8	4.9	60 in.	5.7	226	16
Pottawattamie	315	54 in.	10.0	48 in.	11.2	27	12
Pottawattamie	315	84 in.	7.0	60 in.	8.5	96	21
Johnson	223	8 x 6	7.0	5 x 5	8.9	92	27

An inspection of the last three columns of Tables 41 and 42 show quite clearly the reason for the development of the computerized design method for culverts. Substantial decreases in culvert size can be made without excessive increases in headwater depth. And at those sites which have a large amount of storage, using a smaller culvert results in only modest increases in headwater depth. Site conditions are such for all locations listed in Tables 41 and 42 that no adverse effects are created; highways are not overtopped or are residences flooded or crops drowned. The maximum length of crop inundation for any of these sites is less than a day.

Time Distribution of Rainfall

The 14 selected time distributions of rainfall developed by Huff (15) and shown in Appendix B have a large effect on the magnitude of peak discharge. They have a lesser effect on the maximum headwater depth for a given storm because of culvert size and volume of storage. The shorter duration storms (less than 12 hours) predominate for the watershed sizes drained by culverts. The effect of these first- and second-quartile storms, so named because the greatest percentage of the total storm rainfall falls in the first and second quarter of the storm, on the peak discharge of the inflow hydrograph is shown in Table 43. "2nd-10" is interpreted as the 10 percent probability distribution of rainfall of a second-quartile storm. The variation in discharge is almost 100 percent; however, as previously explained,

Table 43. *Variation in peak discharge due to time distribution of rainfall used*

County	Drainage area, ac.	Peak discharge, cfs, for indicated time distribution					
		1st, 30	1st, 50	1st, 70	2nd, 10	2nd, 30	2nd, 50
Scott	40	96	80	57	110	—[a]	—
Scott	92	215	176	125	235	—	—
Johnson	132	313	261	186	358	—	—
Johnson	223	520	423	301	569	—	—
Howard	580	928	798	596	1,035	963	892
Johnson	600	—	—	583	1,093	975	860
Allamakee	700	966	802	586	1,111	1,001	957
Dubuque	970	—	—	850	1,618	1,462	1,347
Poweshiek	1,610	—	—	857	1,507	1,348	1,175
Johnson	1,930	—	—	694	1,253	1,115	958
Marion	2,625	—	—	1,416	2,207	1,988	1,938
Montgomery	2,980	—	—	1,423	2,480	2,227	1,965
Davis	3,000	—	—	1,872	3,178	2,871	2,580
Osceola	4,540	—	—	1,565	2,784	2,500	2,234
Fremont	4,900	—	—	1,898	3,380	3,040	2,544
Howard	4,970	—	—	1,729	2,959	2,753	2,561
Plymouth	5,040	—	—	1,723	3,110	2,807	2,562
Mills	6,780	—	—	2,603	4,641	4,172	3,647
Allamakee	7,620	—	—	2,603	4,641	4,282	3,975
Johnson	9,470	—	—	3,309	5,875	5,277	4,716
Audubon	16,640	—	—	4,690	7,910	7,418	7,033

[a]Blank space indicates no value was calculated for that time distribution.

the specific time distribution of rainfall used for a particular storm duration yields a peak discharge value within acceptable limits.

Another major effect of the time distribution of rainfall is its effect on the shape of the inflow hydrograph. First-quartile storms yield a hydrograph with a steep rising limb and sharp peak, while hydrographs developed from third- or fourth-quartile storms have a low rate of runoff for several hours before the rising limb climbs steeply to a less sharp peak. Typical inflow hydrographs are shown in Fig. 19. Since third- and fourth-quartile storms are indicative of longer duration storms, the volume of runoff is greater and, depending on the volume of storage at the site, may tax the culvert more than shorter duration storms. However, at those sites with little storage, the peak value of the hydrograph has a greater effect on headwater depth.

Runoff Volume

Since storage upstream of the culvert is a component of the computerized design method for culverts, the volume of runoff becomes as important as the peak discharge Q. While peak discharges tend to remain about the same, longer and longer duration storms of the same recurrence interval result in greater and greater volumes of runoff. These larger runoff volumes usually result in increased headwater depths; for this reason, each culvert alternate is subjected to seven storms of increasing duration. The variation in headwater depth due to an increase in storm duration is shown in Table 44. An analysis of

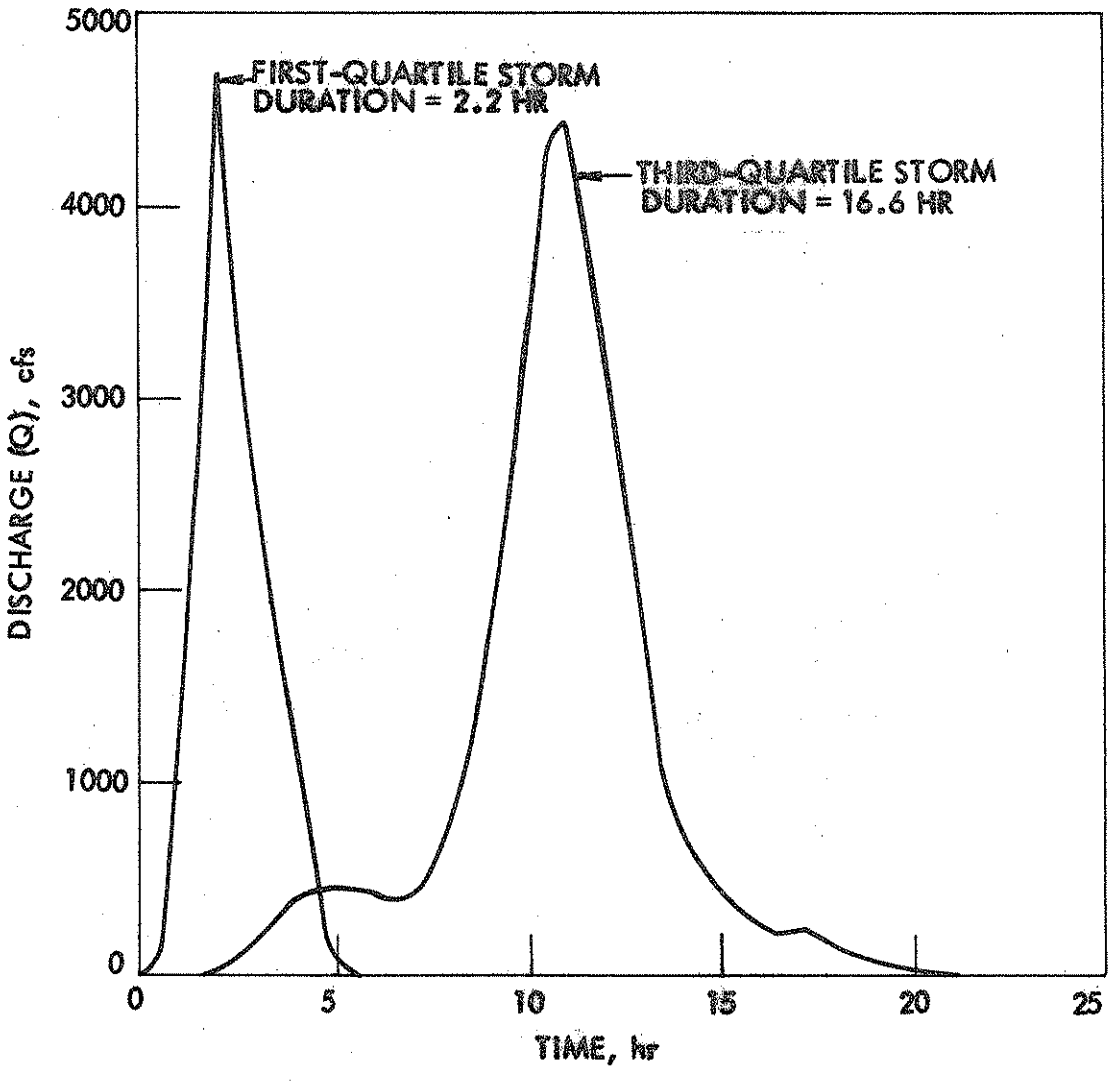

Fig. 19. Typical inflow hydrographs for a 7,620 ac. watershed in Allamakee County.

Table 44. Variation in headwater depth as storm duration increases

County	Drainage area, ac.	Headwater depth, ft, for indicated storm duration number						
		1	2	3	4	5	6	7
Woodbury	340	2.1	4.2	5.0	6.2	6.9	7.3	7.6
Woodbury	40	0.6	2.3	3.1	4.1	4.7	5.1	5.3
Pottawattamie	765	4.1	5.6	6.5	7.3	7.7	7.9	8.1
Pottawattamie	36	0.4	1.4	2.8	3.8	4.2	4.5	4.7
Pottawattamie	83	0.6	1.4	3.5	4.3	4.6	4.7	4.7
Pottawattamie	465	1.8	3.0	3.9	4.8	5.2	5.4	5.5
Tama	850	3.7	4.9	6.0	6.9	7.4	7.5	7.6
Johnson	38	0.0	0.1	0.3	0.5	0.6	0.7	0.8
Johnson	223	2.9	5.2	7.0	7.1	7.1	7.1	7.1
Poweshiek	1,610	7.6	9.7	11.0	12.8	13.2	13.2	13.2
Poweshiek	7,360	8.4	10.2	10.8	11.0	11.2	11.4	11.5
Johnson	990	3.7	4.9	5.4	6.3	6.5	6.6	6.6
Johnson	1,406	4.6	6.3	7.3	8.5	8.9	9.1	9.1

these depths indicate that the rate of increase in headwater depth decreases as the storm duration increases. Thus there tends to be a leveling off of maximum headwater depth as storm duration continues to increase. This is shown graphically in Fig. 20.

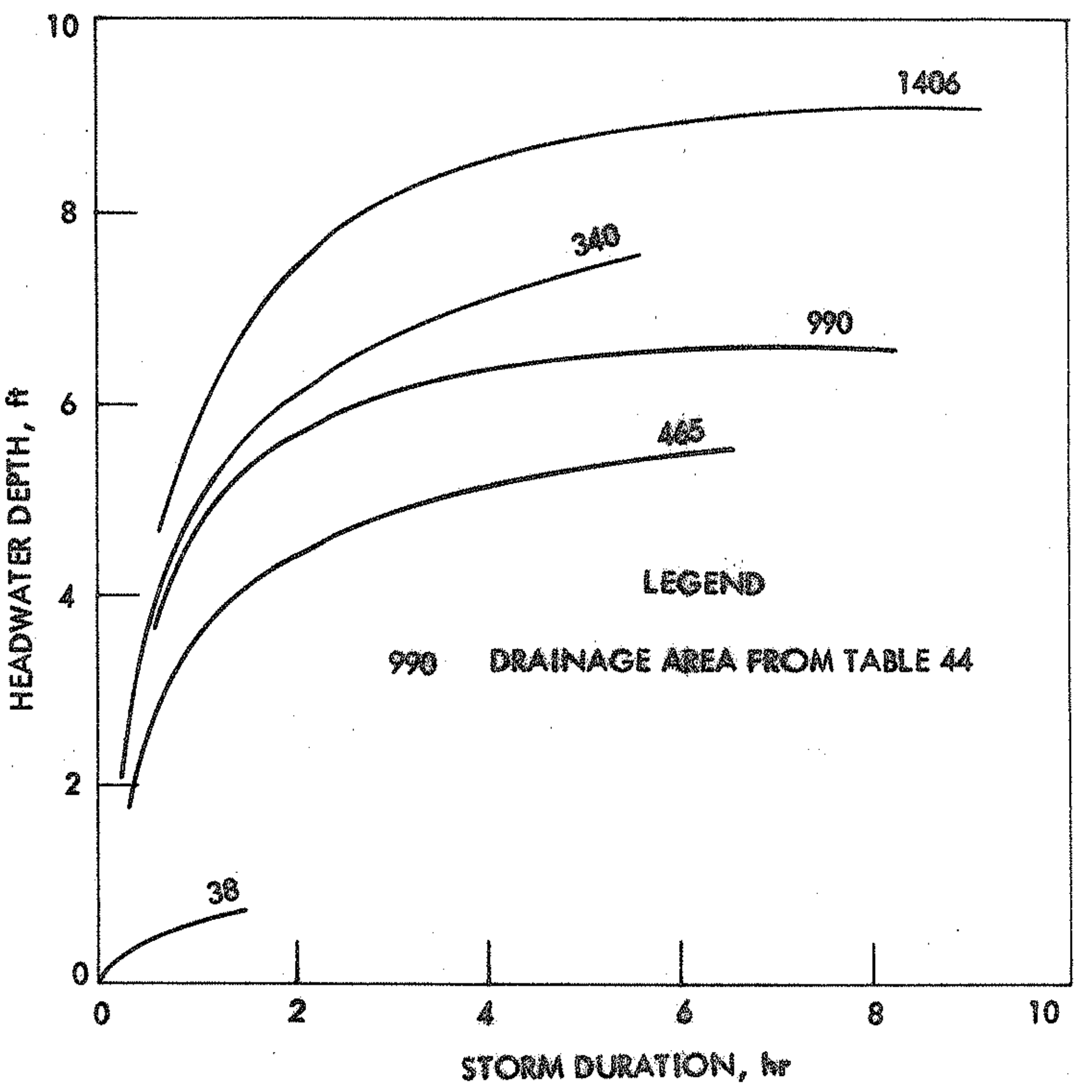

Fig. 20. Increase in headwater depth as storm duration increases.

Value of SCS Runoff Curve Number

Figure 2 shows that the runoff curve number CN developed by the SCS for Iowa vary over a narrow range, 72 to 82. The sensitivity of headwater depth to value of curve number used was studied by varying the value of the curve number. In addition, the effect of a change in curve number, along with a change in culvert size and amount of storage available, on headwater depth was also studied. The results are shown in Table 45.

When the curve number is changed, the volume of runoff from a storm changes. If the curve number is increased, the runoff volume increases; if the curve number is decreased, the headwater depth decreases also. The greatest change in average curve number value across the State of Iowa is 14 percent. However, the percentage change in headwater depth normally is greater than the percentage change in average curve number. In all cases studied, using a smaller culvert size accentuated the change in headwater depth caused by using a different curve number. Again, at those sites which have a large volume of storage, the effect of a change in curve number is decreased.

The pothole terrain of north-central Iowa presents a special problem to the highway drainage designer. The numerous surface depressions are capable of holding a volume of water equal to one-half to one inch of runoff from the entire watershed. This additional reduction of runoff can be accounted for by adjusting the curve number. The normal curve number for this region of Iowa is 76. By reducing the curve number to 73, runoff volume is reduced by about one-quarter inch.

Table 45. Variation in headwater depth due to a change in runoff curve number

County	Drainage area, ac.	Culvert size	Normal CN		Changed CN		Percent change	
			CN	Depth, ft	CN	Depth, ft	CN	Depth
Black Hawk	7,425	3 (16 x 8)	76	9.0	72	8.6	5	4
Black Hawk	7,425	16 x 8	76	11.8	72	11.2	5	5
Marion	2,625	2 (10 x 12)	77	11.7	80	11.7	4	0
Marion	2,625	10 x 10	77	15.8	80	16.6	4	5
Pottawattamie	765	5 x 5	73	8.1	82	10.5	12	30
Pottawattamie	765	60 in.	73	8.3	82	10.9	12	31
Pottawattamie	325	3 (60 in.)	73	4.2	82	5.7	12	36
Pottawattamie	325	3 (54 in.)	73	4.4	82	6.2	12	41
Webster	15,000	3 (16 x 12)	76	10.4	72	10.2	5	2
Webster	15,000	16 x 12	76	24.3	72	23.3	5	4
Webster	1,800	12 x 12	76	10.5	72	9.6	5	9
Webster	1,800	60 in.	76	23.5	72	20.3	5	14
Woodbury	40	48 in.	72	4.9	82	5.9	14	20
Woodbury	40	24 in.	72	7.9	82	9.9	14	25

Likewise, a value of 71 reduces runoff by about one-half inch; a value of 68 reduces runoff by about three-fourths inch; a value of 65 reduces runoff by about one inch; a value of 60 reduces runoff by about one and one-half inches; and a value of 55 reduces runoff by about two inches. This effect of a reduction in curve number on runoff volume for seven storm durations is shown in Table 46 for two

Table 46. Variation in runoff volume with a change in curve number on two watersheds in Webster County

Item	Runoff volume, in., for indicated inflow hydrograph number[a]						
	1	2	3	4	5	6	7
Watershed 1							
Duration, hr	3.51	7.01	14.02	21.03	28.05	35.06	42.07
Rainfall, in.	4.02	4.70	5.44	5.90	6.23	6.49	6.71
Runoff, in.							
CN = 76	1.76	2.29	2.90	3.29	3.58	3.81	4.00
CN = 73	1.54	2.05	2.63	3.00	3.28	3.50	3.69
CN = 71	1.41	1.90	2.46	2.82	3.08	3.30	3.48
CN = 68	1.22	1.67	2.20	2.54	2.80	3.00	3.18
CN = 65	1.04	1.46	1.95	2.28	2.52	2.71	2.88
CN = 60	0.77	1.13	1.57	1.85	2.07	2.25	2.40
CN = 55	0.54	0.84	1.21	1.46	1.65	1.81	1.94
Watershed 2							
Duration, hr	1.21	2.43	3.64	7.29	10.93	14.57	18.21
Rainfall, in.	3.09	3.69	4.06	4.74	5.17	5.48	5.73
Runoff, in.							
CN = 76	1.08	1.50	1.78	2.33	2.68	2.94	3.15
CN = 73	0.92	1.31	1.57	2.08	2.41	2.67	2.87
CN = 71	0.82	1.18	1.43	1.92	2.25	2.49	2.69
CN = 68	0.68	1.01	1.24	1.70	2.00	2.23	2.42
CN = 65	0.55	0.85	1.06	1.49	1.77	1.98	2.16

[a]Inflow hydrograph number varies according to increasing storm duration.

Table 46. Continued.

Item	Runoff volume, in., for indicated inflow hydrograph number						
	1	2	3	4	5	6	7
CN = 60	0.37	0.61	0.79	1.15	1.40	1.59	1.75
CN = 55	0.22	0.41	0.55	0.86	1.07	1.23	1.37

watersheds in Webster County, one 15,000 ac. in size and the other 1,800 ac. in size.

The reduction in CN affects the peak discharge Q as well as the volume of runoff which results in lesser headwater depths. This reduction in depth is shown in Table 47 for the two watersheds in Webster County. A one-inch reduction in runoff causes about a 25 percent reduction in headwater depth. This result indicates that more work should be done to refine our present hydrologic techniques in this pothole region of Iowa.

Volume of Storage

The volume of storage at a site, along with culvert size, was found to have the greatest effect on headwater depth of all parameters studied. The greater the volume of storage at a site, the less effect a change in culvert size had on headwater depth. This fact allows a much smaller culvert to be used at a site (which in turn reduces culvert cost) without causing adverse effects due to an increased depth of water. The sensitivity of headwater depth to a change in the

Table 47. Variation in headwater depth with a change in curve number on two watersheds in Webster County

Curve number	Headwater depth, ft, for indicated inflow hydrograph number						
	1	2	3	4	5	6	7
Watershed 1							
76	10.2	12.0	13.7	13.6	12.4	12.1	11.7
73	9.4	11.2	12.8	12.8	11.9	11.7	11.3
71	8.9	10.9	12.2	12.3	11.5	11.4	11.0
68	8.1	10.4	11.4	11.5	11.0	10.9	10.5
65	7.4	10.0	10.5	10.7	10.5	10.4	10.0
60	6.1	9.2	9.1	9.3	9.5	9.4	9.2
55	4.8	8.7	7.7	8.0	8.4	8.4	8.2
Watershed 2							
76	6.3	7.9	8.5	9.3	8.9	9.3	9.0
73	5.6	7.1	7.7	8.6	8.3	8.8	8.5
71	5.1	6.6	7.2	8.1	7.9	8.4	8.2
68	4.4	6.3	6.5	7.4	7.3	7.9	7.8
65	3.9	5.9	5.9	6.7	6.7	7.4	7.3
60	3.0	5.4	4.8	5.6	5.7	6.5	6.5
55	2.1	4.9	3.7	4.6	4.8	5.5	5.6

amount of storage at a site is shown in Table 48. The percentage change in headwater depth is shown in Table 49. This variation in storage volume could be caused by a number of things: lack of good data, inaccurate maps, faulty calculation of storage volume, or siltation of

Table 48. Variation in headwater depth due to a change in storage volume

County	Drainage area, ac.	Culvert size	Headwater depth, ft, for indicated reduction in storage			
			0	1/6	1/3	1/2
Black Hawk	7,425	3 (16 x 8)	9.1	9.2	9.4	9.5
Pottawattamie	36	30 in.	4.7	5.1	5.6	6.0
Pottawattamie	83	48 in.	4.7	5.0	5.2	5.4
Pottawattamie	330	60 in.	7.5	7.9	8.4	9.2
Pottawattamie	265	60 in.	7.5	7.9	8.3	8.4
Pottawattamie	465	72 in.	5.4	6.0	6.7	7.8
Pottawattamie	960	2 (8 x 6)	6.6	6.8	7.0	7.1
Marion	2,625	2 (8 x 8)	13.5	13.8	14.2	14.8
Woodbury	340	24 in.	6.9	7.8	9.2	11.1
Woodbury	40	42 in.	5.3	5.4	5.5	5.8
Webster	1,800	8 x 8	14.1	14.7	15.5	16.3

the ponding area over a period of time. The greatest reduction in storage due to sedimentation is estimated as one-half, assuming a rectangular storage area with vertical side walls. For normal valleys, the reduction in storage over a period of time due to sedimentation is assumed to be about one-sixth or less.

The most important conclusion to be reached from the data shown in Tables 48 and 49 is that the change in headwater depth due to decreasing amounts of storage is minimal at those sites which have little storage volume. Conversely, at those sites which have large

Table 49. Percent change in headwater depth due to a change in storage volume

County	Drainage area, ac.	Culvert size	Percent change in headwater depth for indicated reduction in storage			
			0[a]	1/6	1/3	1/2
Black Hawk	7,425	3 (16 x 8)	0.0	1.1	3.3	4.4
Pottawattamie	36	30 in.	0.0	8.5	19.1	27.7
Pottawattamie	83	48 in.	0.0	6.4	10.6	14.9
Pottawattamie	330	60 in.	0.0	5.3	12.0	22.7
Pottawattamie	265	60 in.	0.0	5.3	10.7	12.0
Pottawattamie	465	72 in.	0.0	11.1	24.1	44.5
Pottawattamie	960	2 (8 x 6)	0.0	3.0	6.1	7.6
Marion	2,625	2 (8 x 8)	0.0	2.2	5.2	9.6
Woodbury	340	24 in.	0.0	13.1	33.4	60.9
Woodbury	40	42 in.	0.0	1.9	3.8	9.4
Webster	1,800	8 x 8	0.0	4.3	9.9	15.6

[a]Standard for comparison.

storage volumes, the change in headwater depth is much greater. The reasoning behind these results is as follows. When little storage volume is available, only a minor portion of the incoming flood can be stored below each successive foot of depth; so the depth of water increases rapidly. This rapid increase in depth creates sufficient head so that water flows through the culvert at about the same rate as it is flowing to the culvert. As the inflow rate rises to a peak, sufficient head develops immediately (due to the lack of storage) to pass this increased rate of flow through the culvert. Thus the outflow Q is almost equal to the inflow Q. Since the culvert must be designed for the peak Q that flows through it, little or no reduction in culvert size is possible. Since the inflow and outflow rates are almost equal already, reducing the amount of storage by one-sixth or one-half has little effect on headwater depth.

Conversely, at sites with large storage volumes, much of the water can be stored below each successive foot of depth; so the depth of water increases more slowly. Since the rate of flow through a culvert is directly proportional to the head on the culvert, the outflow Q is much less than the inflow Q. The difference in the two flows during any incremental time period is stored temporarily upstream of the culvert. The increase in depth during this time period is a function of the area of the pond. The volume of flow during the incremental time period is roughly equal to the length of time period multiplied by the average of the inflow and outflow Q. This volume of water is spread uniformly over the pond. Thus, the greater the area of the pond, the smaller the increase in depth will be during that time period. The amount of reduction in outflow rate is dependent on the amount of storage available. Since the outflow Q is lower, a smaller culvert can be used. Reducing the amount of storage by one-sixth or one-half at these sites means less water can be stored below each successive foot of depth; so the depth of water increases more rapidly and the percentage change in depth becomes greater as the storage is decreased.

The amount that the peak Q is reduced as it flows through the temporary pond and culvert is an indication of whether there is a small or large amount of storage at a site. Table 50 shows the reduction in Q that is effected at sites which have little storage. Table 51 shows the reduction in Q that takes place at sites which have a large volume of storage. The difference in the percentage reduction in Q between the two tables is significant. A graphical method of showing the reduction in peak Q is by plotting the inflow and outflow hydrographs. Hydrographs at a site with a large volume of storage were shown in Fig. 16. Inflow and outflow hydrographs at a site which has little storage is shown in Fig. 21.

The data shown in Tables 50 and 51 indicate the expected results. At those sites which have small storage volumes, the reduction in Q is slight, ranging from 0 to 13 percent. However, at those sites which have large storage volumes, the reduction in Q is much greater, ranging from 35 to 93 percent. An interesting occurrence is also shown in these two tables. Three sites (40, 1,800, and 2,625 ac. watersheds) are listed in both tables. The only difference is the size of culvert used. Simply by using a smaller culvert, a site which appears to have a small storage volume becomes a site with a large storage volume. The additional storage comes from an increase in headwater depth. In most cases studied, this increase in headwater depth had no adverse effects. In those instances where a smaller culvert did cause adverse effects, that size of culvert was simply rejected from further consideration.

Table 50. Reduction in peak discharge at sites which have "small" storage volumes

County	Drainage area, ac.	Culvert size	Maximum inflow, cfs	Maximum outflow, cfs	Percent change	Storage used, ac. ft
Poweshiek	330	6 x 6	350	329	6.0	3.9
Mills	6,780	3 (10 x 10)	3,852	3,735	3.0	97.4
Fremont	4,900	2 (12 x 12)	2,773	2,751	0.8	34.8
Johnson	5,020	3 (12 x 6)	2,748	2,519	8.3	184.1
Johnson	223	8 x 6	384	374	2.6	3.3
Pottawattamie	5,110	2 (12 x 10)	3,305	3,296	0.3	34.3
Webster	1,800	16 x 12	1,273	1,187	6.8	69.5
Webster	15,000	2 (20 x 12)	3,939	3,853	2.2	274.4
Woodbury	40	48 in.	108	96	11.1	1.8
Pottawattamie	960	2 (8 x 6)	826	717	13.2	27.7
Marion	2,625	2 (10 x 12)	2,182	2,062	5.5	73.9

Table 51. Reduction in peak discharge at sites which have "large" storage volumes

County	Drainage area, ac.	Culvert size	Maximum inflow, cfs	Maximum outflow, cfs	Percent change	Storage used, ac. ft
Poweshiek	7,360	2 (12 x 10)	3,761	2,358	37.3	659.0
Johnson	990	10 x 8	789	439	44.4	73.4
Johnson	1,406	12 x 10	1,193	673	43.6	103.1
Scott	160	60 in.	222	117	47.3	19.3
Tama	850	10 x 10	934	486	48.0	59.9
Johnson	151	5 x 5	220	114	48.2	19.1
Johnson	38	36 in.	75	5	93.3	6.2
Scott	43	48 in.	81	47	42.0	4.0
Webster	1,800	6 x 6	1,273	693	45.6	175.6
Woodbury	40	24 in.	108	41	62.0	4.3
Woodbury	340	24 in.	762	47	93.9	82.1
Pottawattamie	765	5 x 5	728	255	65.0	58.2
Pottawattamie	330	60 in.	402	234	41.8	28.1
Pottawattamie	465	72 in.	479	178	63.0	34.8
Pottawattamie	315	48 in.	389	190	51.2	32.3

Table 51. Continued

County	Drainage area, ac.	Culvert size	Maximum inflow, cfs	Maximum outflow, cfs	Percent change	Storage used, ac. ft
Black Hawk	7,425	16 x 8	4,170	1,500	64.0	1137.2
Marion	2,625	10 x 10	2,182	1,414	35.2	160.7

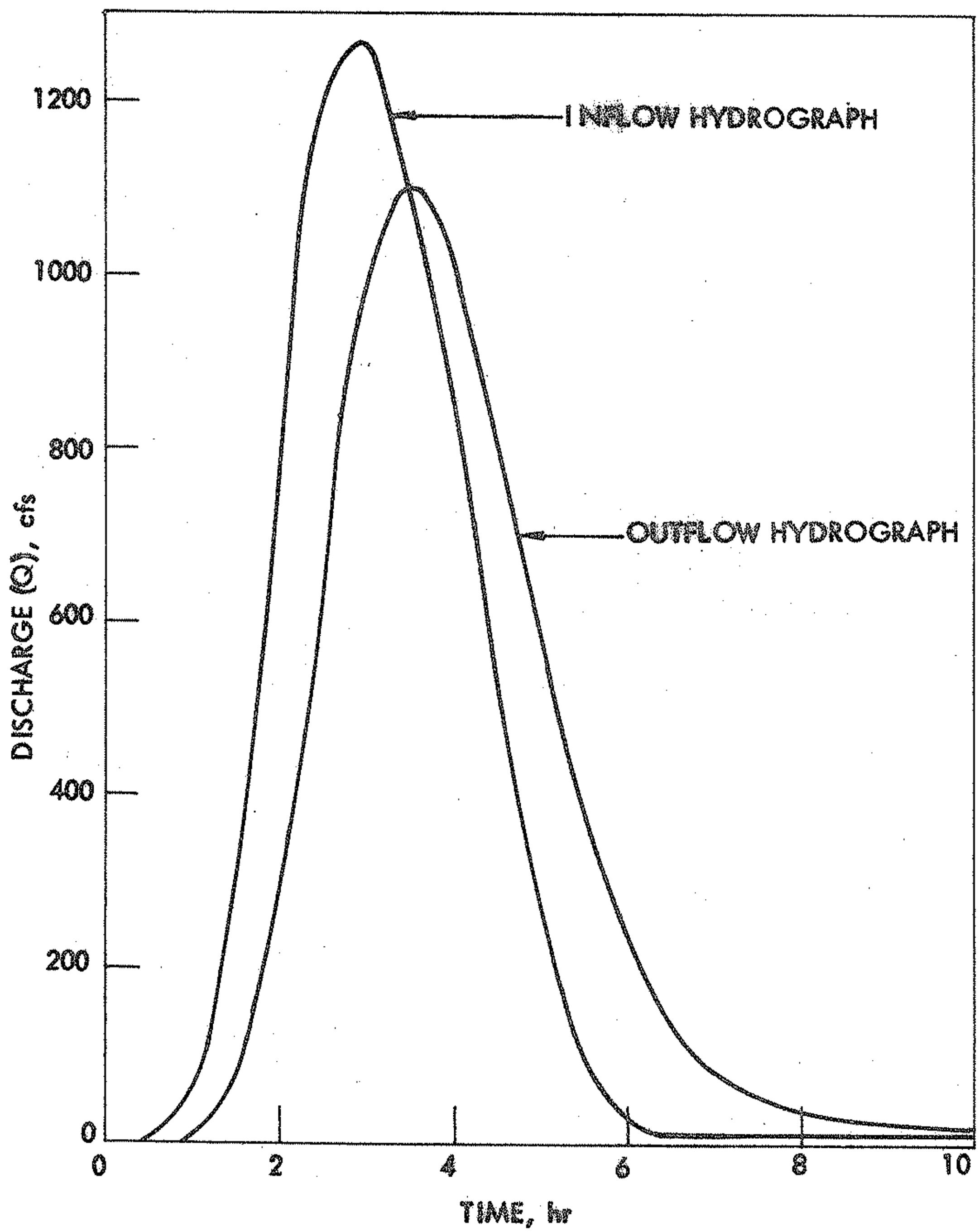

Fig. 21. Inflow and outflow hydrographs for a 2,160 ac. watershed in Johnson County.

Another interesting occurrence is that the words "small" and "large" are relative terms. At one site 274.4 ac. ft of storage produced only a 2 percent reduction in peak Q, while at another site, just 6.2 ac. ft of storage produced a 93 percent reduction in peak Q. In the first case, 274.4 ac. ft was "small" and in the second case, 6.2 ac. ft was "large." The drainage area is 15,000 ac. at the first site and 38 ac. at the second site; so it would appear that the size of watershed needs to be taken into consideration. Further investigation revealed that both the size of culvert and the volume of runoff during some increment of time were also important. This led to an attempt to find some simple method of determining whether or not a specific site had a "large" volume of storage and was, therefore, a good candidate for using a smaller culvert.

At first, the percent reduction in Q was used as the basis for determining whether or not a site has a "large" or "small" volume of storage so that a smaller culvert could be used. This approach was abandoned when several sites showed only a small percent reduction in Q but a one-third to one-half reduction in culvert size. The reason for this was increased headwater depths. Even with a much smaller culvert, the reduction in Q was slight because of the increased head available. The basis finally used was the volume of storage in inches over the watershed below the "maximum allowable" headwater depth. The results of this are shown in Fig. 22.

The data shown in Fig. 22 indicate that any site with a temporary storage volume greater than one- or two-tenths of an inch is a candidate for using a smaller culvert. The storage is determined in the following

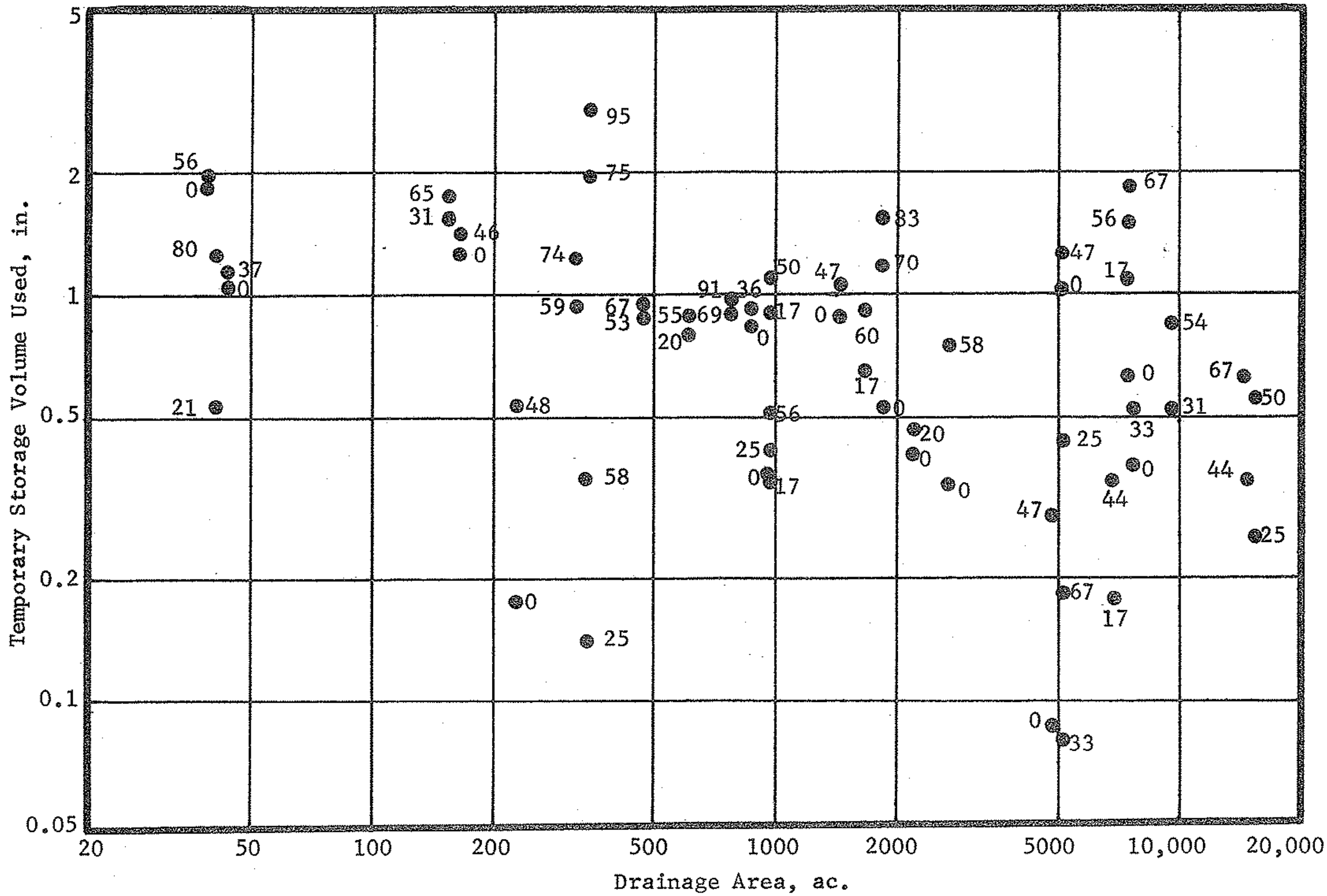

Fig. 22. Percent reduction in culvert size using temporary storage at various sites.

manner. First, determine the "maximum allowable" headwater depth at the site. In many cases this could be 20 ft or more. Site conditions and the judgment of the engineer will determine the "maximum allowable" depth. Second, determine the total volume of storage below this depth (to the culvert invert) using the method shown in Table 17. Third, convert this storage volume from acre-feet to inches as follows: multiply the storage in acre-feet by twelve to obtain acre-inches and then divide this result by the drainage area of the watershed in acres to obtain the temporary storage volume in inches over the watershed. If the answer is greater than one- or two-tenths of an inch, the computerized design method proposed in this study should be used because a smaller culvert size could be achieved at the site. Each point shown in Fig. 22 is listed in Table 52. The amount of storage available below the "maximum allowable" headwater depth also gives some indication of the amount of reduction in culvert size that can be accomplished. The larger the storage volume in inches over the watershed, the smaller the culvert can be made.

All of the culverts listed in Table 50 were reanalyzed using a smaller culvert. Each of them then used a storage volume of about 0.2 in. or more. The five sites which had originally plotted below the 0.2 in. line in Fig. 22 had headwater depths between two and three times the height of the culvert. However, this did not cause any adverse effects at any of the five sites. These results indicate again, though, that at those sites which have minimal storage available, using a smaller culvert will cause larger increases in headwater depth.

Table 52. Culvert size, headwater depth, storage used, and percent change in culvert size using the computerized design method

County	Drainage area, ac.	Culvert size current method	Computerized design method				Percent change in size
			Culvert size	Depth ft	Storage used, ac. ft	Storage used, in.	
Johnson	38	4 × 4	4 × 4	0.8	6.0	1.90	0
Johnson	38	4 × 4	36 in.	0.8	6.2	1.96	56
Woodbury	40	54 in.	48 in.	4.9	1.8	0.54	21
Woodbury	40	54 in.	24 in.	7.6	4.3	1.29	80
Scott	43	4 × 5	4 × 5	2.8	3.9	1.09	0
Scott	43	4 × 5	48 in.	3.0	4.0	1.12	37
Johnson	151	6 × 6	5 × 5	4.3	19.1	1.52	31
Johnson	151	6 × 6	48 in.	4.8	22.2	1.76	65
Scott	160	6 × 6	6 × 6	4.2	17.2	1.29	0
Scott	160	6 × 6	60 in.	4.6	19.3	1.45	46
Johnson	223	8 × 6	8 × 6	7.0	3.3	0.18	0
Johnson	223	8 × 6	5 × 5	8.9	9.5	0.51	48
Pottawattamie	315	8 × 6	60 in.	6.7	25.0	0.95	59
Pottawattamie	315	8 × 6	48 in.	11.2	32.3	1.23	74

Table 52. Continued

County	Drainage area, ac.	Culvert size current method	Computerized design method				Percent change in size
			Culvert size	Depth ft	Storage used, ac. ft	Storage used, in.	
Poweshiek	330	8 x 6	6 x 6	8.0	3.9	0.14	25
Poweshiek	330	8 x 6	4 x 5	8.7	9.8	0.36	58
Woodbury	340	8 x 8	54 in.	10.4	56.1	1.98	75
Woodbury	340	8 x 8	24 in.	13.5	80.9	2.86	95
Pottawattamie	465	10 x 6	72 in.	5.4	34.8	0.90	53
Pottawattamie	465	10 x 6	60 in.	5.7	36.5	0.94	67
Johnson	600	10 x 8	8 x 8	7.2	40.2	0.80	20
Johnson	600	10 x 8	6 x 6	8.1	45.3	0.91	55
Pottawattamie	765	10 x 8	5 x 5	8.1	58.2	0.91	69
Pottawattamie	765	10 x 8	60 in.	8.3	61.2	0.96	91
Tama	850	10 x 10	10 x 10	7.1	59.9	0.84	0
Tama	850	10 x 10	8 x 8	7.6	65.7	0.93	36
Pottawattamie	960	2 (8 x 6)	2 (8 x 6)	6.8	27.7	0.35	0
Pottawattamie	960	2 (8 x 6)	12 x 6	8.0	33.5	0.42	25

Table 52. Continued

County	Drainage area, ac.	Culvert size current method	Computerized design method				Percent change in size
			Culvert size	Depth ft	Storage used, ac. ft	Storage used, in.	
Dubuque	970	12 x 12	12 x 10	9.4	27.9	0.34	17
Dubuque	970	12 x 12	8 x 8	11.4	41.0	0.51	56
Johnson	990	12 x 8	10 x 8	6.6	73.4	0.89	17
Johnson	990	12 x 8	6 x 8	7.8	90.9	1.10	50
Johnson	1,406	12 x 10	12 x 10	7.8	103.1	0.88	0
Johnson	1,406	12 x 10	8 x 8	9.1	122.3	1.04	47
Poweshiek	1,610	12 x 10	10 x 10	10.4	86.0	0.64	17
Poweshiek	1,610	12 x 10	6 x 8	13.2	122.0	0.91	60
Webster	1,800	12 x 10	12 x 10	10.5	79.6	0.53	0
Webster	1,800	12 x 10	6 x 6	19.8	175.6	1.17	70
Webster	1,800	12 x 10	60 in.	23.5	229.3	1.53	83
Johnson	2,160	2 (10 x 8)	2 (10 x 8)	8.3	71.4	0.40	0
Johnson	2,160	2 (10 x 8)	2 (8 x 8)	9.5	83.4	0.46	20
Marion	2,625	2 (10 x 12)	2 (10 x 12)	11.7	73.9	0.34	0

Table 52. Continued

County	Drainage area, ac.	Culvert size current method	Computerized design method				Percent change in size
			Culvert size	Depth ft	Storage used, ac. ft	Storage used, in.	
Marion	2,625	2 (10 × 12)	10 × 10	15.8	160.7	0.74	58
Fremont	4,900	2 (12 × 10)	2 (12 × 10)	13.0	34.8	0.08	0
Fremont	4,900	2 (12 × 10)	2 (8 × 8)	21.3	116.6	0.29	47
Johnson	5,020	3 (12 × 8)	3 (12 × 6)	10.2	184.1	0.44	25
Plymouth	5,040	2 (12 × 10)	2 (12 × 10)	9.9	419.7	1.00	0
Plymouth	5,040	2 (12 × 10)	2 (8 × 8)	12.4	523.1	1.25	47
Pottawattamie	5,110	3 (12 × 10)	2 (12 × 10)	15.4	34.3	0.08	33
Pottawattamie	5,110	3 (12 × 10)	2 (8 × 10)	22.2	77.0	0.18	67
Mills	6,780	3 (10 × 12)	3 (10 × 10)	14.0	97.4	0.17	17
Mills	6,780	3 (10 × 12)	2 (10 × 10)	20.1	192.9	0.34	44
Poweshiek	7,360	2 (12 × 12)	2 (12 × 10)	11.5	659.0	1.07	17
Poweshiek	7,360	2 (12 × 12)	2 (8 × 8)	13.0	912.3	1.49	56
Black Hawk	7,425	3 (16 × 8)	3 (16 × 8)	9.0	385.1	0.62	0
Black Hawk	7,425	3 (16 × 8)	16 × 8	11.8	1137.2	1.84	67

Table 52. Continued

County	Drainage area, ac.	Culvert size current method	Computerized design method: Culvert size	Depth ft	Storage used, ac. ft	Storage used, in.	Percent change in size
Allamakee	7,620	3 (12 x 12)	3 (12 x 12)	12.7	233.8	0.37	0
Allamakee	7,620	3 (12 x 12)	2 (12 x 12)	15.9	323.0	0.51	33
Johnson	9,470	3 (12 x 12)	3 (10 x 10)	15.0	412.4	0.52	31
Johnson	9,470	3 (12 x 12)	2 (10 x 10)	19.1	662.1	0.84	54
Webster	15,000	3 (12 x 12)	2 (10 x 12)	15.7	438.2	0.35	44
Webster	15,000	3 (12 x 12)	12 x 12	25.0	773.3	0.62	67
Johnson	15,740	3 (16 x 12)	3 (12 x 12)	16.2	331.5	0.25	25
Johnson	15,740	3 (16 x 12)	2 (12 x 12)	19.8	718.4	0.55	50

What can be done at sites with large storage volumes is also shown in Fig. 22. Several sites used storage volumes between about 0.2 and 1.0 in. with zero percent reduction in culvert size (that is, the same size culvert was input to the computerized design method as was determined using the current ISHC design method). When smaller culverts were input, only small additional storage volumes were required to achieve substantial reductions in culvert size. The depths recorded in Table 52 for these two culvert sizes indicate that the headwater depth was usually less than the culvert height for the larger size and somewhat greater or much greater for the smaller culvert depending on the amount of reduction in size.

A temporary storage volume of one- to two-tenths of an inch over the watershed is rather small and essentially means that all sites should be considered for using a smaller culvert. The shotgun pattern shown in Fig. 22 indicates that each culvert site is unique and should be investigated on its own merits using the computerized design method.

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APPENDIX A.

SCS CURVE NUMBER COMPUTATION SHEET

UNITED STATES DEPARTMENT OF AGRICULTURE
Soil Conservation Service
Iowa

HYDROLOGIC CURVE NUMBER COMPUTATION SHEET

Watershed________________ Site________________

Computed by__________ Date__________ Checked by__________ Date__________

Cover	Practice	Condition or Rota.	Acres Per Practice	Curve Numbers Moisture Cond. II B Soils	(1)	C Soils	Product
Fallow	Straight Row	--		86		91	
Row Crops	Straight Row	Poor		81		88	
	Straight Row	Good		78		85	
	***Contoured	Poor		79		84	
	***Contoured	Good		75		82	
	* C and T	Poor		74		80	
	* C and T	Good		71		78	
Small Grain	Straight Row	Poor		76		84	
	Straight Row	Good		75		83	
	***Contoured	Poor		74		82	
	***Contoured	Good		73		81	
	* C and T	Poor		72		79	
	* C and T	Good		70		78	
Legumes or Rotation Meadow	Straight Row	Poor		77		85	
	Straight Row	Good		72		81	
	***Contoured	Poor		75		83	
	***Contoured	Good		69		78	
	* C and T	Poor		73		80	
	* C and T	Good		67		76	
Pasture		Poor		79		86	
		Fair		69		79	
		Good		61		74	
Meadow (Permanent)		Good		58		71	
Woods (Farm)		Poor		66		77	
		Fair		60		73	
		Good		55		70	
Farmsteads		--		74		82	
**Roads	Dirt	--		82		87	
	Hard Surface	--		84		90	

Total D.A. = ac.; Product total = ______________

Weighted Runoff Curve No. $\frac{\text{Product Total}}{\text{Total Acres}}$ = __________ = ______________ for Mois. Cond. II

(1) For other Hydrologic Soil types or Moisture Conditions, as determined by Section 3.21 of Hydrology Guide or Intermediate curve numbers for mixed areas.

* Contoured and graded terraces

** Includes right-of-way

*** Includes level terraced areas (runoff corrected by volume).

APPENDIX B.

HISTOGRAMS OF TIME DISTRIBUTIONS OF RAINFALL

The histograms of the fourteen time distributions of rainfall shown in this appendix were selected from the thirty-six distributions presented in the study by Huff (15). Each histogram is labeled with a quartile and a percent probability. These should be interpreted as follows.

The quartile refers to that part of the storm in which the largest percentage of the total rainfall occurs. Thus, a first-quartile storm has the heaviest rainfall occurring in the first quarter of the storm. Likewise, a fourth-quartile storm has the largest percentage of total rainfall occurring in the last quarter of the storm. The quartile listing is also an indication of the duration of the storm. First- and second-quartile storms are normally associated with shorter duration storms (less than 12 hr). Third-quartile storms are generally associated with moderate length storms (12 to 24 hr) and fourth-quartile storms are normally associated with the longer duration storms (greater than 24 hr).

The histograms are also expressed in probability terms because of the great variability in the characteristics of the time distribution of the rainfall from storm to storm within the same quartile grouping. Thus, the 50 percent probability represents the average time distribution of rainfall for all storms in that quartile. The 10 percent and 90 percent probability levels should be interpreted as time distributions of rainfall that will occur in ten percent or less of all storms. Likewise, the 30 percent and 70 percent probability levels are interpreted as time distributions of rainfall that will occur in thirty percent or less of all storms.

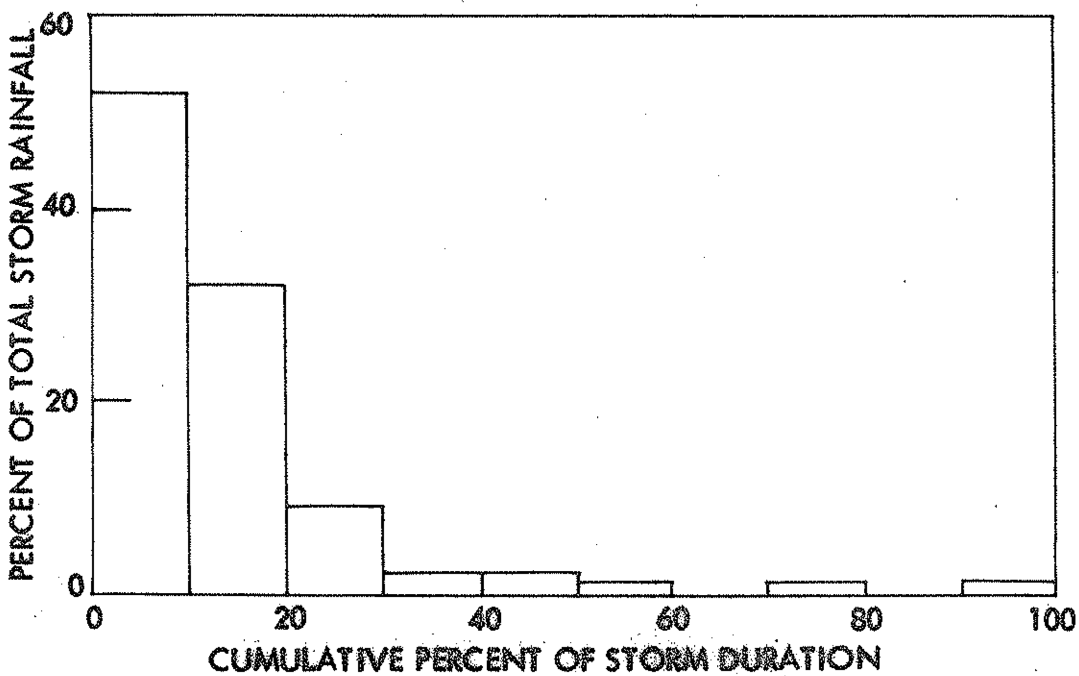

Fig. B-1. Histogram of first-quartile, 10 percent probability time distribution of rainfall.

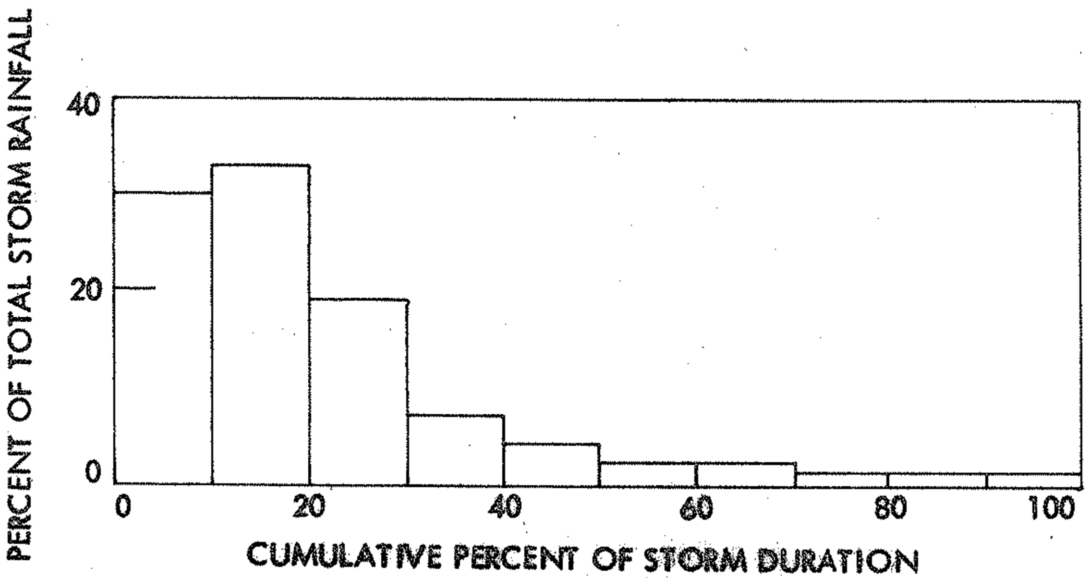

Fig. B-2. Histogram of first-quartile, 30 percent probability time distribution of rainfall.

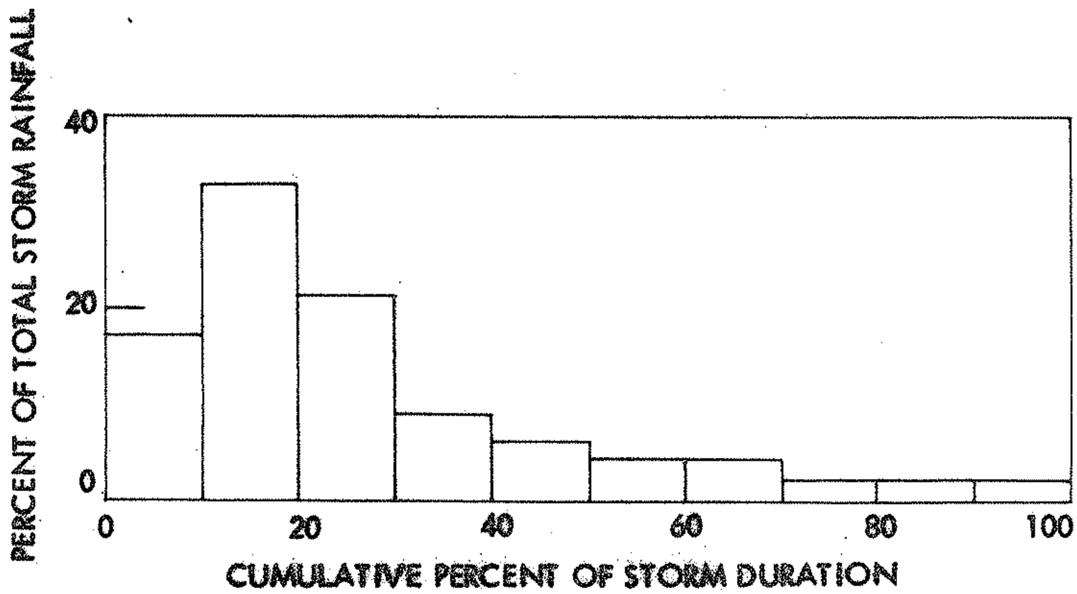

Fig. B-3. Histogram of first-quartile, 50 percent probability time distribution of rainfall.

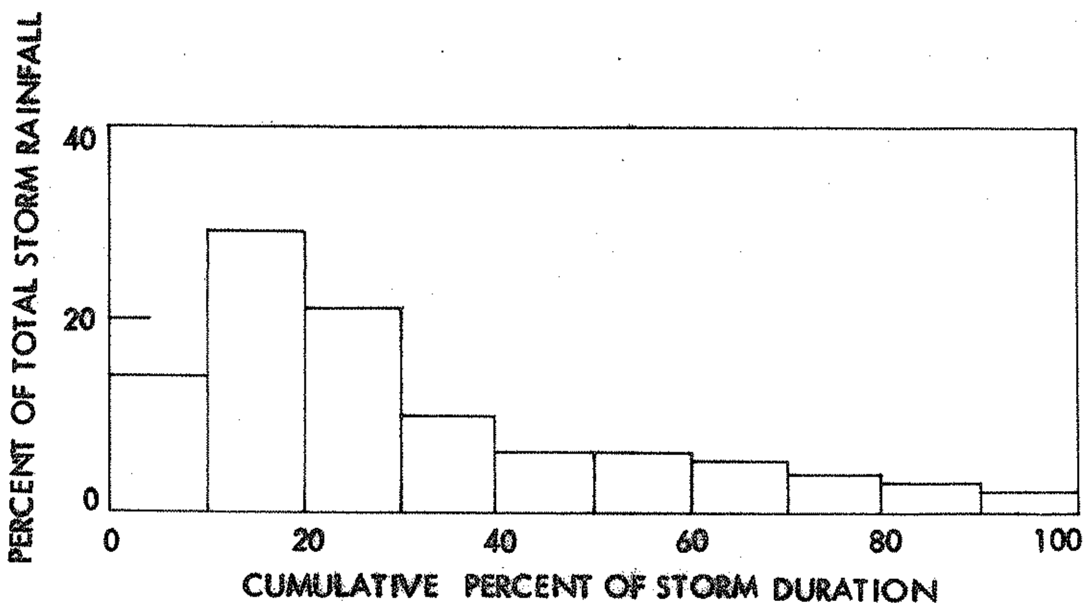

Fig. B-4. Histogram of first-quartile, 60 percent probability time distribution of rainfall.

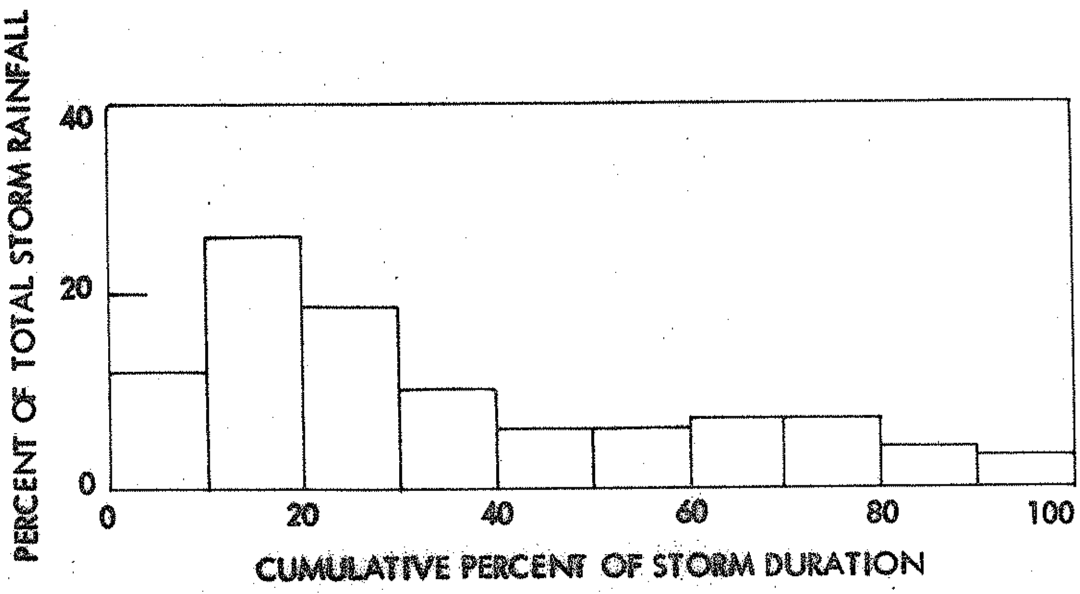

Fig. B-5. Histogram of first-quartile, 70 percent probability time distribution of rainfall.

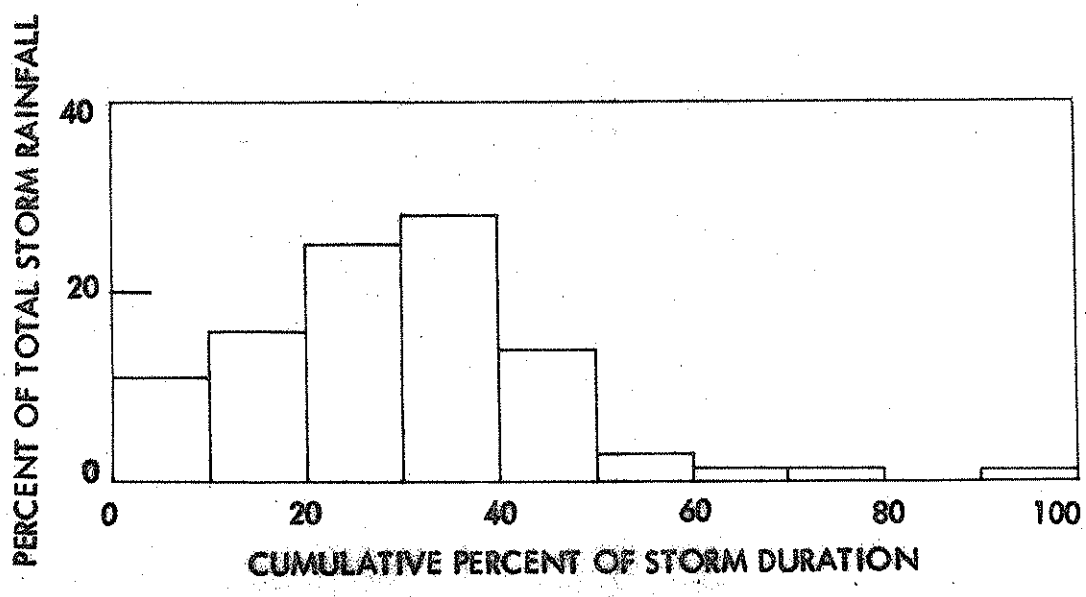

Fig. B-6. Histogram of second-quartile, 10 percent probability time distribution of rainfall.

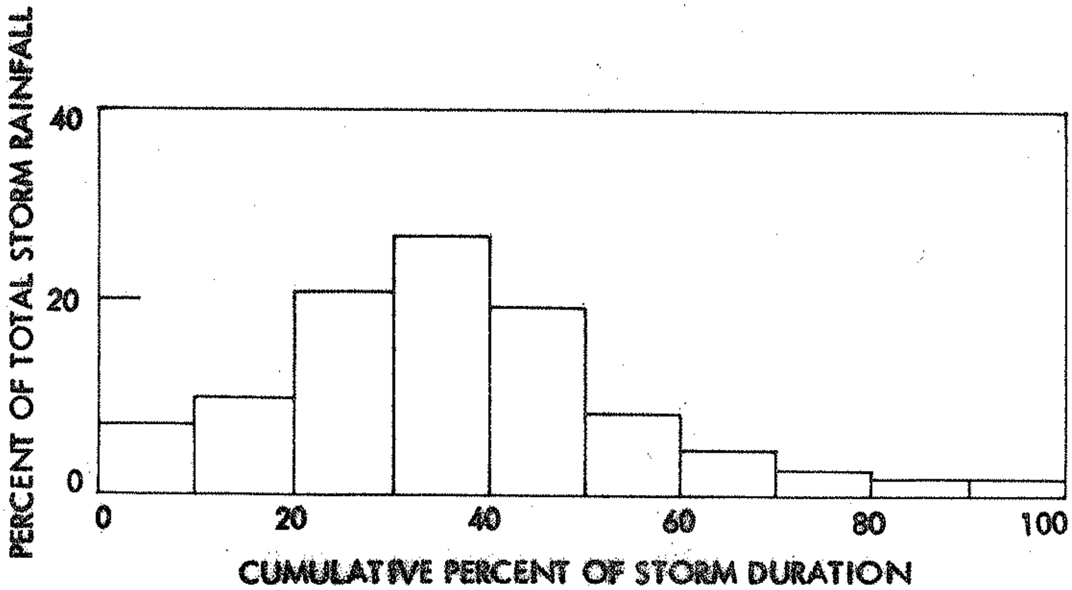

Fig. B-7. Histogram of second-quartile, 30 percent probability time distribution of rainfall.

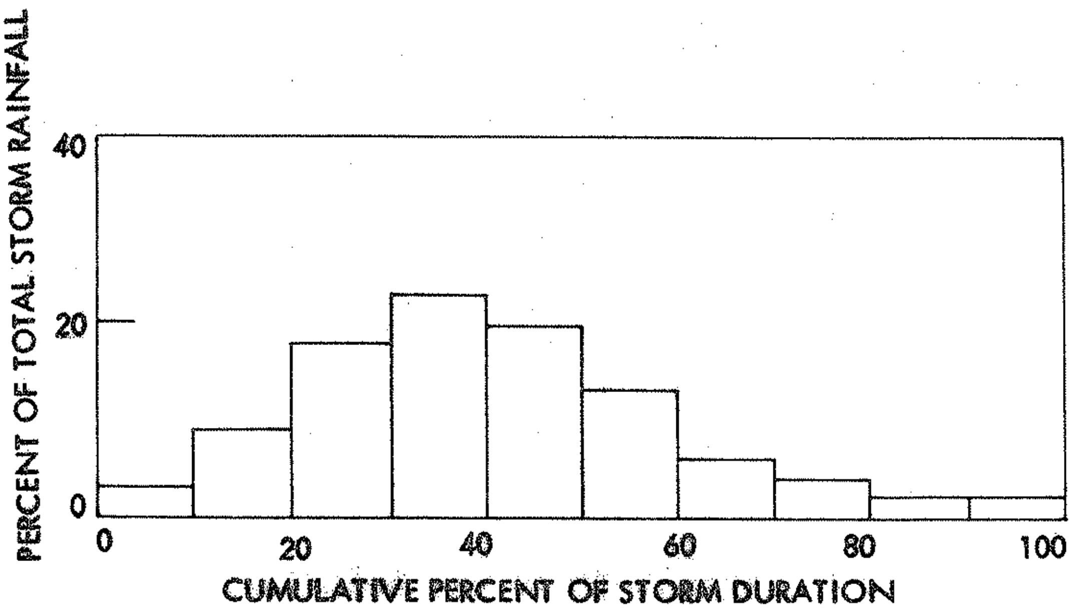

Fig. B-8. Histogram of second-quartile, 50 percent probability time distribution of rainfall.

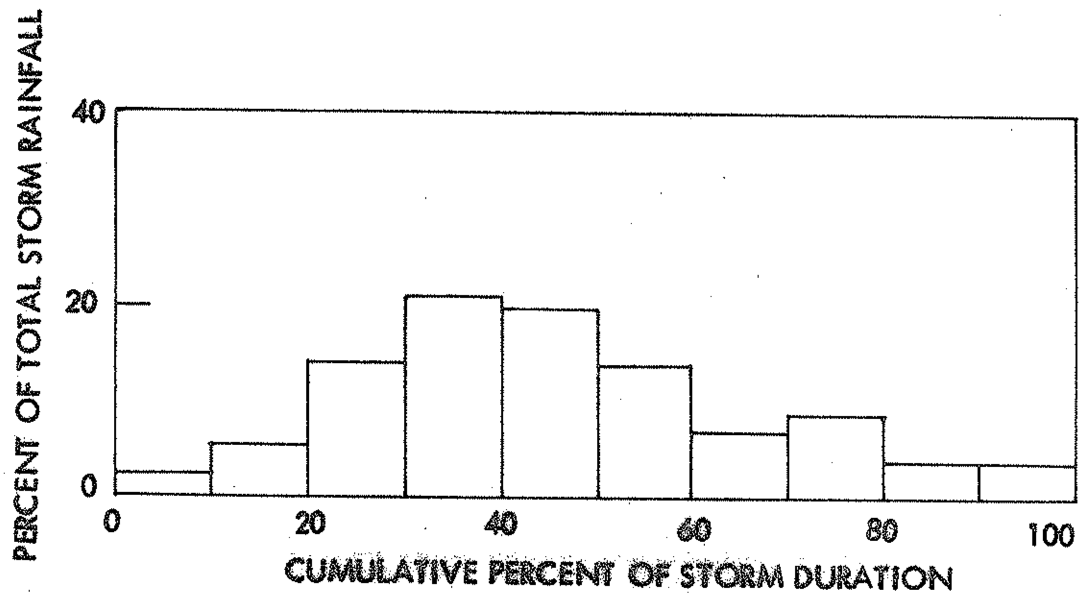

Fig. B-9. Histogram of second-quartile, 70 percent probability time distribution of rainfall.

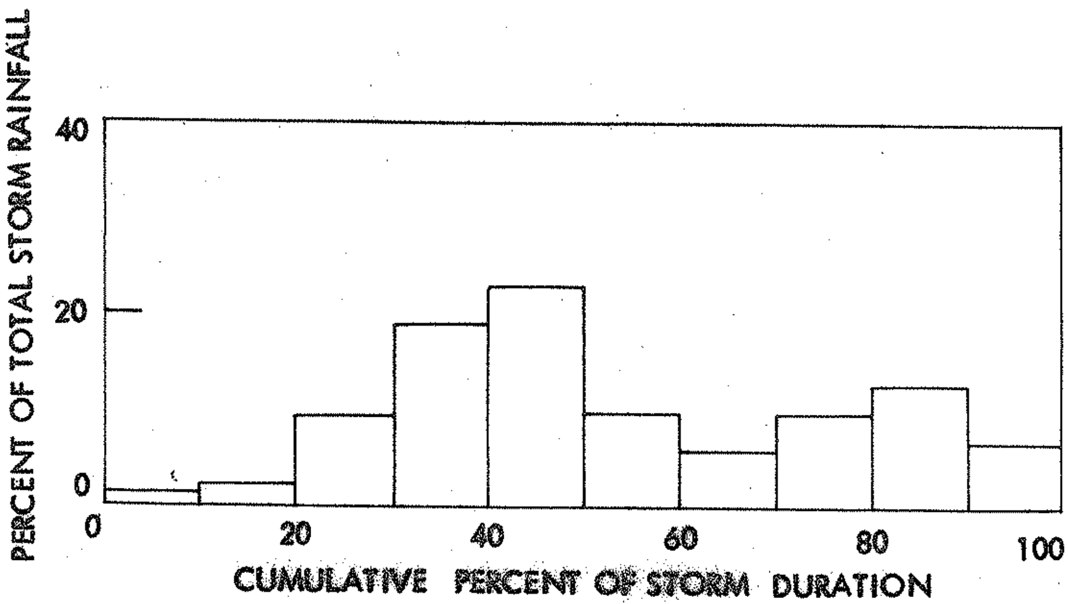

Fig. B-10. Histogram of second quartile, 90 percent probability time distribution of rainfall.

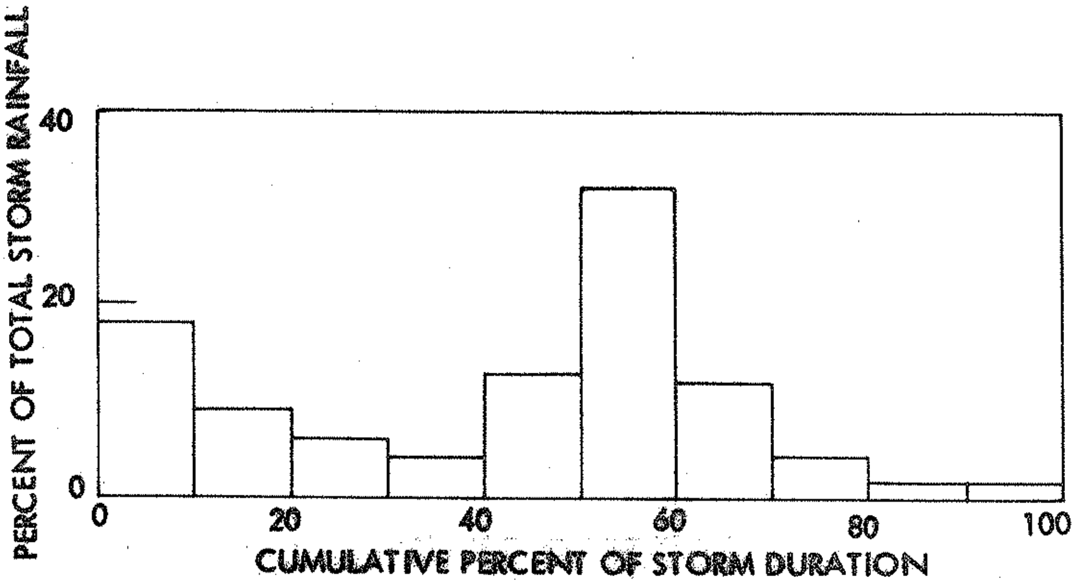

Fig. B-11. Histogram of third-quartile, 10 percent probability time distribution of rainfall.

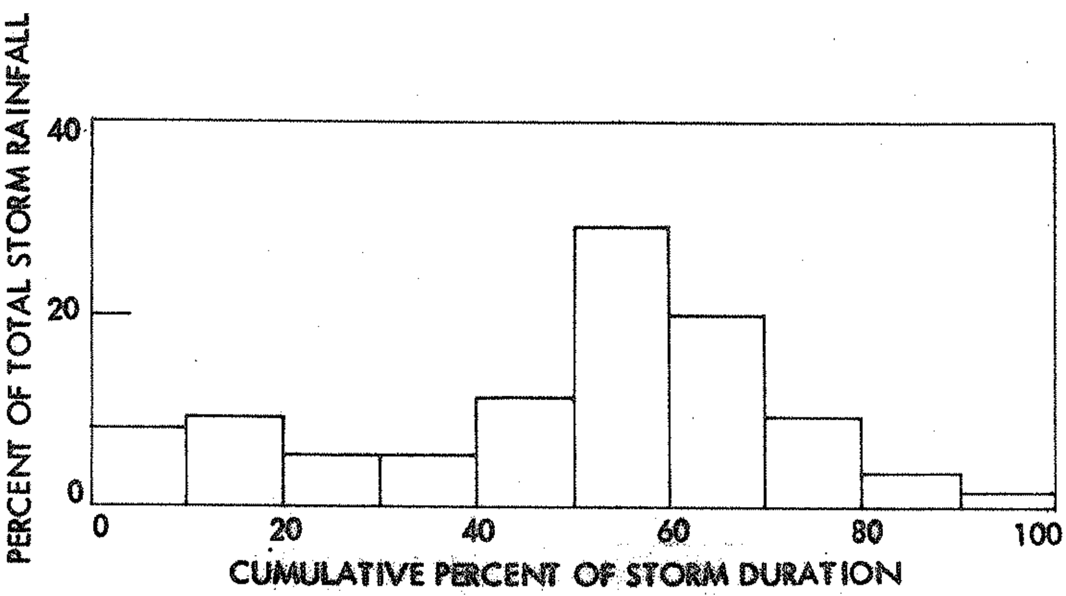

Fig. B-12. Histogram of third-quartile, 30 percent probability time distribution of rainfall.

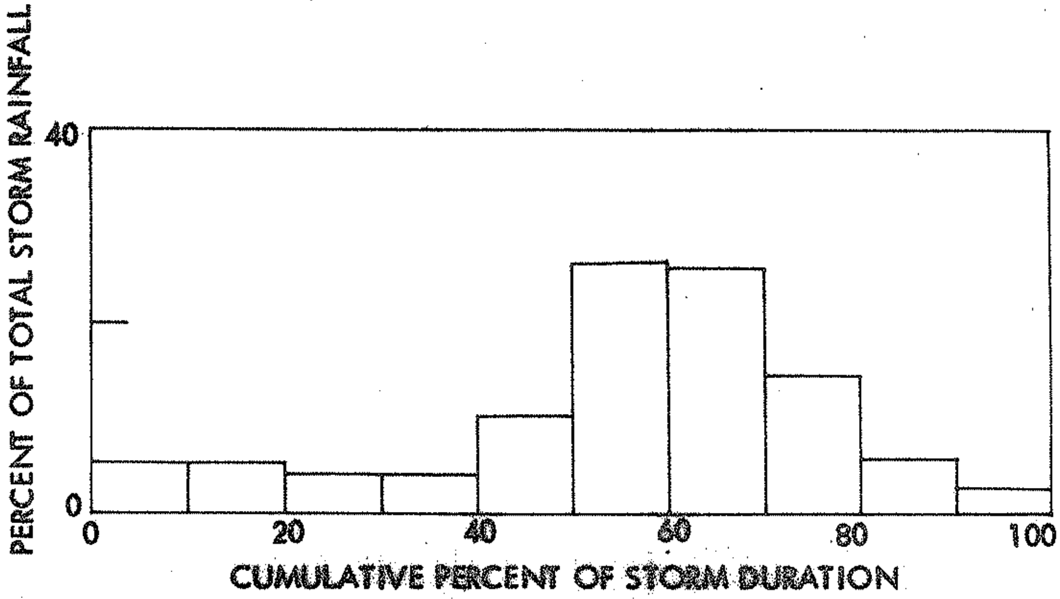

Fig. B-13. Histogram of third-quartile, 50 percent probability time distribution of rainfall.

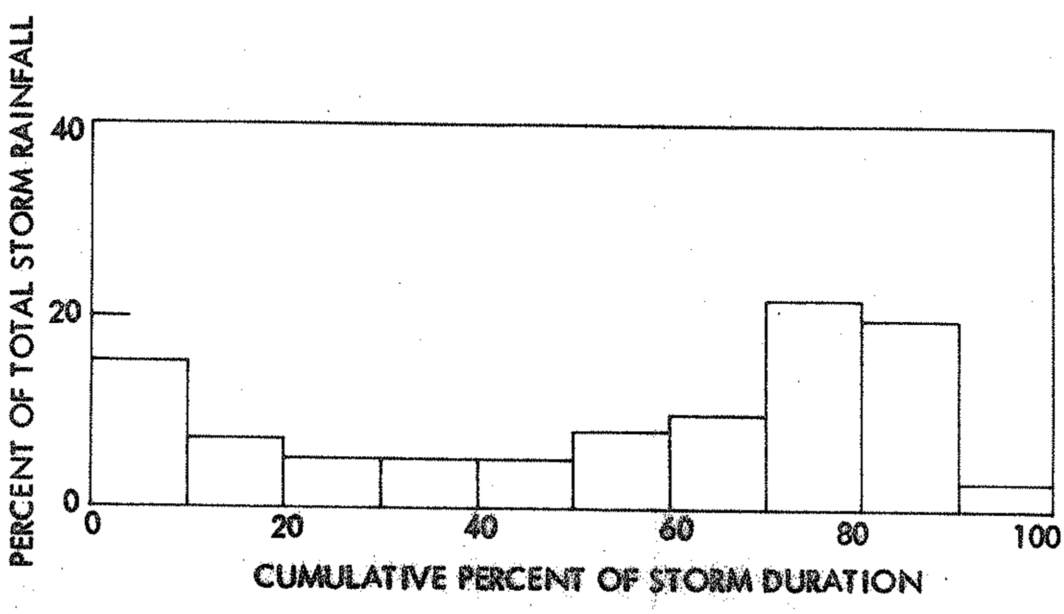

Fig. B-14. Histogram of fourth-quartile, 10 percent probability time distribution of rainfall.

APPENDIX C.

TYPICAL PROGRAM OUTPUT

I N P U T   D A T A

INFLOW DATA

WOODBURY COUNTY

RECURRENCE INTERVAL = 50 YEARS

DRAINAGE AREA = 340. ACRES

LAND FACTOR = 0.90

FREQUENCY FACTOR = 1.00

LENGTH OF STREAM = 7200. FEET

DIFF. IN ELEVATION = 167. FEET

STAGE-STORAGE DATA

ELEV	AC-FT
1141.0	0.0
1150.0	9.9
1155.0	23.1
1160.0	44.6
1165.0	75.8
1170.0	116.5
1175.0	170.0
1180.0	241.8

OUTFLOW DATA

I-IG-129-6(3)145--04-97   POND AT STA. 477+60
DRAINAGE AREA = 340 AC. - VERY HILLY
7.5 MIN. QUAD - SERGEANT BLUFF (93-A)
SINGLE 24 INCH CMP AT ELEV. 1155.0

INLET TYPE	HDWATER ELEV.	FLOLINE ELEV.	NO.	WIDTH FEET	HEIGHT FEET	DIAM. IN.	WEIR L FEET
7	1175.0	1155.0	1	0.0	0.0	24.0	0.0
0	0.0	0.0	0	0.0	0.0	0.0	0.0
0	0.0	0.0	0	0.0	0.0	0.0	0.0
0	0.0	0.0	0	0.0	0.0	0.0	0.0
12	1175.0	1170.0	0	0.0	0.0	0.0	40.0

WOODBURY COUNTY D.A. = 340. ACRES

INFLOW HYDROGRAPH CALCULATIONS

TIME OF CONCENTRATION = 0.52 HR.

DELTA DURATION = 5. MIN.

TIME TO PEAK = 15. MIN.

TIME BASE = 40. MIN.

UNIT PEAK Q = 1029. CFS

STORM DURATION = 0.52 HR.

TIME MIN.	RAINFALL INCHES	RUNOFF INCHES	DELTA SRO INCHES	DELTA Q CFS
0.	0.00	0.00		
			0.00	0.
5.	0.53	0.00		
			0.13	134.
10.	1.56	0.13		
			0.33	338.
15.	2.36	0.46		
			0.05	52.
20.	2.46	0.51		
			0.01	13.
25.	2.49	0.52		
			0.01	13.
30.	2.51	0.54		

TIME MIN.	TIME HR.	Q CFS
0.	0.00	0.
5.	0.08	0.
10.	0.17	45.
15.	0.25	202.
20.	0.33	377.
25.	0.42	538.
30.	0.50	416.
35.	0.58	320.
40.	0.67	217.
45.	0.75	107.
50.	0.83	24.
55.	0.92	8.
60.	1.00	3.
65.	1.08	1.
3200.	53.33	1.

MAX. Q = 484. CFS DES. Q = 538. CFS

WOODBURY COUNTY D.A. = 340. ACRES

I-IG-129-6(3)145--04-97 POND AT STA. 477+60
DRAINAGE AREA = 340 AC. - VERY HILLY
7.5 MIN. QUAD - SERGEANT BLUFF (93-A)
SINGLE 24 INCH CMP AT ELEV. 1155.0

ELEV	Q1	Q2	Q1 OR Q2	Q3	Q4	Q5	TOTAL Q
1155.0	0.	0.	0.	0.	0.	0.	0.
1156.C	4.	0.	4.	0.	0.	0.	4.
1157.C	11.	0.	11.	0.	0.	0.	11.
1158.0	18.	0.	18.	0.	0.	0.	18.
1159.C	22.	0.	22.	0.	0.	0.	22.
1160.C	26.	0.	26.	0.	0.	0.	26.
1161.0	29.	0.	29.	0.	0.	0.	29.
1162.0	32.	0.	32.	0.	0.	0.	32.
1163.0	35.	0.	35.	0.	0.	0.	35.
1164.C	37.	0.	37.	0.	0.	0.	37.
1165.0	39.	0.	39.	0.	0.	0.	39.
1166.C	41.	0.	41.	0.	0.	0.	41.
1167.0	43.	0.	43.	0.	0.	0.	43.
1168.0	45.	0.	45.	0.	0.	0.	45.
1169.0	47.	0.	47.	0.	0.	0.	47.
1170.C	49.	0.	49.	0.	0.	0.	49.
1171.0	51.	0.	51.	0.	0.	120.	171.
1172.0	53.	0.	53.	0.	0.	339.	392.
1173.0	55.	0.	55.	0.	0.	624.	678.
1174.0	56.	0.	56.	0.	0.	960.	1016.
1175.0	58.	0.	58.	0.	0.	1342.	1400.

FLOOD ROUTING OUTPUT DATA---PAGE 1

WOODBURY COUNTY D.A. = 340. ACRES

I-IG-129-6(3)145--04-97 POND AT STA. 477+60
DRAINAGE AREA = 340 AC. - VERY HILLY
7.5 MIN. QUAD - SERGEANT BLUFF (93-A)
SINGLE 24 INCH CMP AT ELEV. 1155.0

DUR = 0.26 HR. RAIN = 2.07 IN. RUNOFF = 0.32 IN.

TIME HOURS	INFLOW CFS	OUTFLOW CFS	STORAGE AC-FT	ELEVATION FT
0.00	0.	0.	0.0	1155.00
0.08	20.	0.	0.1	1155.02
0.17	124.	0.	0.6	1155.13
0.25	234.	2.	1.8	1155.42
0.33	313.	3.	3.6	1155.85
0.42	256.	6.	5.6	1156.30
0.50	190.	9.	7.1	1156.64
0.58	124.	10.	8.1	1156.88
0.67	58.	11.	8.6	1157.01
0.75	4.	12.	8.8	1157.04
0.83	1.	11.	8.7	1157.02
0.92	1.	11.	8.6	1157.01
1.00	1.	11.	8.6	1156.99
1.08	1.	11.	8.5	1156.97
1.17	1.	11.	8.4	1156.96
1.25	1.	11.	8.4	1156.94
1.33	1.	11.	8.3	1156.93
1.42	1.	11.	8.2	1156.91
1.50	1.	10.	8.2	1156.90
1.58	1.	10.	8.1	1156.88
1.67	1.	10.	8.0	1156.87
1.75	1.	10.	8.0	1156.85
1.83	1.	10.	7.9	1156.84
1.92	1.	10.	7.8	1156.82
2.00	1.	10.	7.8	1156.81
2.08	1.	10.	7.7	1156.79
2.17	1.	10.	7.7	1156.78
2.25	1.	10.	7.6	1156.77
2.33	1.	9.	7.5	1156.75
2.42	1.	9.	7.5	1156.74
2.50	1.	9.	7.4	1156.73
2.58	1.	9.	7.4	1156.71
2.67	1.	9.	7.3	1156.70
2.75	1.	9.	7.3	1156.69
2.83	1.	9.	7.2	1156.68
2.92	1.	9.	7.2	1156.66
3.00	1.	9.	7.1	1156.65
3.08	1.	9.	7.1	1156.64

FLOOD ROUTING OUTPUT DATA---PAGE 2

WOODBURY COUNTY D.A. = 340. ACRES

I-IG-129-6(3)145--04-97 POND AT STA. 477+60
DRAINAGE AREA = 340 AC. - VERY HILLY
7.5 MIN. QUAD - SERGEANT BLUFF (93-A)
SINGLE 24 INCH CMP AT ELEV. 1155.0

DUR = 0.26 HR. RAIN = 2.07 IN. RUNOFF = 0.32 IN.

TIME HOURS	INFLOW CFS	OUTFLOW CFS	STORAGE AC-FT	ELEVATION FT
3.17	1.	8.	7.0	1156.63
3.25	1.	8.	7.0	1156.62
3.33	1.	8.	6.9	1156.61
3.42	1.	8.	6.9	1156.59

RUNOFF VOLUME = 113. CFS-HOURS

FLOOD ROUTING OUTPUT DATA---PAGE 1

WOODBURY COUNTY D.A. = 340. ACRES

I-IG-129-6(3)145--04-97 POND AT STA. 477+60
DRAINAGE AREA = 340 AC. - VERY HILLY
7.5 MIN. QUAD - SERGEANT BLUFF (93-A)
SINGLE 24 INCH CMP AT ELEV. 1155.0

DUR = 0.52 HR. RAIN = 2.51 IN. RUNOFF = 0.54 IN.

TIME HOURS	INFLOW CFS	OUTFLOW CFS	STORAGE AC-FT	ELEVATION FT
0.00	0.	0.	0.0	1155.00
0.08	0.	0.	0.0	1155.00
0.17	45.	0.	0.2	1155.04
0.25	202.	1.	1.0	1155.23
0.33	377.	3.	3.0	1155.69
0.42	538.	7.	6.1	1156.42
0.50	416.	12.	9.3	1157.16
0.58	320.	16.	11.7	1157.73
0.67	217.	18.	13.5	1158.13
0.75	107.	19.	14.4	1158.36
0.83	24.	20.	14.8	1158.43
0.92	8.	20.	14.7	1158.43
1.00	3.	20.	14.6	1158.40
1.08	1.	19.	14.5	1158.38
1.17	1.	19.	14.4	1158.35
1.25	1.	19.	14.3	1158.32
1.33	1.	19.	14.1	1158.29
1.42	1.	19.	14.0	1158.26
1.50	1.	19.	13.9	1158.23
1.58	1.	19.	13.8	1158.20
1.67	1.	19.	13.7	1158.18
1.75	1.	18.	13.5	1158.15
1.83	1.	18.	13.4	1158.12
1.92	1.	18.	13.3	1158.09
2.00	1.	18.	13.2	1158.07
2.08	1.	18.	13.1	1158.04
2.17	1.	18.	12.9	1158.01
2.25	1.	18.	12.8	1157.98
2.33	1.	17.	12.7	1157.96
2.42	1.	17.	12.6	1157.93
2.50	1.	17.	12.5	1157.91
2.58	1.	17.	12.4	1157.88
2.67	1.	17.	12.3	1157.86
2.75	1.	17.	12.2	1157.83
2.83	1.	16.	12.1	1157.81
2.92	1.	16.	12.0	1157.78
3.00	1.	16.	11.9	1157.76
3.08	1.	16.	11.8	1157.73

FLCOD ROUTING OUTPUT DATA---PAGE 2

WOODBURY COUNTY D.A. = 340. ACRES

I-IG-129-6(3)145--04-97 POND AT STA. 477+60
DRAINAGE AREA = 340 AC. - VERY HILLY
7.5 MIN. QUAD - SERGEANT BLUFF (93-A)
SINGLE 24 INCH CMP AT ELEV. 1155.0

DUR = 0.52 HR. RAIN = 2.51 IN. RUNOFF = 0.54 IN.

TIME HOURS	INFLOW CFS	OUTFLOW CFS	STORAGE AC-FT	ELEVATION FT
3.17	1.	16.	11.7	1157.71
3.25	1.	16.	11.5	1157.69
3.33	1.	16.	11.4	1157.66
3.42	1.	15.	11.4	1157.64

RUNOFF VOLUME = 191. CFS-HOURS

FLOOD ROUTING OUTPUT DATA---PAGE 1

WOODBURY COUNTY D.A. = 340. ACRES

I-IG-129-6(3)145--04-97 POND AT STA. 477+60
DRAINAGE AREA = 340 AC. - VERY HILLY
7.5 MIN. QUAD - SERGEANT BLUFF (93-A)
SINGLE 24 INCH CMP AT ELEV. 1155.0

DUR = 1.03 HR. RAIN = 3.01 IN. RUNOFF = 0.82 IN.

TIME HOURS	INFLOW CFS	OUTFLOW CFS	STORAGE AC-FT	ELEVATION FT
0.00	0.	0.	0.0	1155.00
0.17	47.	0.	0.3	1155.07
0.33	237.	2.	2.3	1155.52
0.50	475.	9.	7.1	1156.64
0.67	657.	20.	14.7	1158.41
0.83	624.	27.	23.1	1160.26
1.00	524.	31.	30.6	1161.46
1.17	392.	33.	36.5	1162.40
1.33	237.	35.	40.3	1163.02
1.50	97.	35.	42.2	1163.31
1.67	44.	36.	42.6	1163.39
1.83	18.	36.	42.6	1163.38
2.00	5.	35.	42.2	1163.33
2.17	1.	35.	41.8	1163.25
2.33	1.	35.	41.3	1163.18
2.50	1.	35.	40.9	1163.10
2.67	1.	35.	40.4	1163.03
2.83	1.	35.	39.9	1162.96
3.00	1.	34.	39.5	1162.88
3.17	1.	34.	39.0	1162.81
3.33	1.	34.	38.6	1162.74
3.50	1.	34.	38.1	1162.66
3.67	1.	34.	37.7	1162.59
3.83	1.	33.	37.2	1162.52
4.00	1.	33.	36.8	1162.45

RUNOFF VOLUME = 561. CFS-HOURS

FLOOD ROUTING OUTPUT DATA---PAGE 1

WOODBURY COUNTY D.A. = 340. ACRES

I-IG-129-6(3)145--04-97 POND AT STA. 477+60
DRAINAGE AREA = 340 AC. - VERY HILLY
7.5 MIN. QUAD - SERGEANT BLUFF (93-A)
SINGLE 24 INCH CMP AT ELEV. 1155.0

DUR = 2.07 HR. RAIN = 3.56 IN. RUNOFF = 1.16 IN.

TIME HOURS	INFLOW CFS	OUTFLOW CFS	STORAGE AC-FT	ELEVATION FT
0.00	0.	0.	0.0	1155.00
0.17	1.	0.	0.0	1155.00
0.33	82.	0.	0.6	1155.13
0.50	295.	3.	3.1	1155.73
0.67	575.	12.	9.0	1157.10
0.83	763.	23.	18.0	1159.18
1.00	765.	29.	28.1	1161.06
1.17	673.	34.	37.6	1162.58
1.33	534.	37.	45.4	1163.83
1.50	367.	39.	51.1	1164.74
1.67	234.	40.	54.7	1165.24
1.83	161.	40.	56.8	1165.51
2.00	116.	41.	58.2	1165.67
2.17	84.	41.	59.0	1165.77
2.33	64.	41.	59.4	1165.83
2.50	38.	41.	59.6	1165.84
2.67	21.	41.	59.4	1165.83
2.83	11.	41.	59.1	1165.78
3.00	5.	41.	58.6	1165.73
3.17	1.	41.	58.1	1165.66
3.33	1.	41.	57.6	1165.60
3.50	1.	40.	57.0	1165.53
3.67	1.	40.	56.5	1165.46
3.83	1.	40.	55.9	1165.40
4.00	1.	40.	55.4	1165.33
4.17	1.	40.	54.9	1165.27
4.33	1.	40.	54.3	1165.20
4.50	1.	40.	53.8	1165.14
4.67	1.	39.	53.3	1165.07
4.83	1.	39.	52.8	1165.01
5.00	1.	39.	52.2	1164.93
5.17	1.	39.	51.7	1164.84
5.33	1.	39.	51.2	1164.76
5.50	1.	39.	50.7	1164.68
5.67	1.	38.	50.2	1164.59
5.83	1.	38.	49.7	1164.51
6.00	1.	38.	49.1	1164.43
6.17	1.	38.	48.6	1164.35

FLOOD ROUTING OUTPUT DATA---PAGE 2

WOODBURY COUNTY D.A. = 340. ACRES

I-IG-129-6(3)145--04-97 POND AT STA. 477+60
DRAINAGE AREA = 340 AC. - VERY HILLY
7.5 MIN. QUAD - SERGEANT BLUFF (93-A)
SINGLE 24 INCH CMP AT ELEV. 1155.0

DUR = 2.07 HR. RAIN = 3.56 IN. RUNOFF = 1.16 IN.

TIME HOURS	INFLOW CFS	OUTFLOW CFS	STORAGE AC-FT	ELEVATION FT
6.33	1.	38.	48.1	1164.27
6.50	1.	37.	47.6	1164.19
6.67	1.	37.	47.1	1164.11
6.83	1.	37.	46.6	1164.03
7.00	1.	37.	46.1	1163.95
7.17	1.	37.	45.7	1163.87
7.33	1.	37.	45.2	1163.79
7.50	1.	36.	44.7	1163.71
7.67	1.	36.	44.2	1163.64
7.83	1.	36.	43.7	1163.56
8.00	1.	36.	43.2	1163.48
8.17	1.	36.	42.8	1163.41
8.33	1.	35.	42.3	1163.33
8.50	1.	35.	41.8	1163.25
8.67	1.	35.	41.3	1163.18
8.83	1.	35.	40.9	1163.10
9.00	1.	35.	40.4	1163.03
9.17	1.	35.	40.0	1162.96
9.33	1.	34.	39.5	1162.88
9.50	1.	34.	39.0	1162.81
9.67	1.	34.	38.6	1162.74
9.83	1.	34.	38.1	1162.67
10.00	1.	34.	37.7	1162.59
10.17	1.	33.	37.2	1162.52
10.33	1.	33.	36.8	1162.45

RUNOFF VOLUME = 806. CFS-HOURS

FLOOD ROUTING OUTPUT DATA---PAGE 1

WOODBURY COUNTY D.A. = 340. ACRES

I-IG-129-6(3)145--04-97 POND AT STA. 477+60
DRAINAGE AREA = 340 AC. - VERY HILLY
7.5 MIN. QUAD - SERGEANT BLUFF (93-A)
SINGLE 24 INCH CMP AT ELEV. 1155.0

DUR = 3.10 HR. RAIN = 3.91 IN. RUNOFF = 1.40 IN.

TIME HOURS	INFLOW CFS	OUTFLOW CFS	STORAGE AC-FT	ELEVATION FT
0.00	0.	0.	0.0	1155.00
0.17	0.	0.	0.0	1155.00
0.33	17.	0.	0.1	1155.03
0.50	108.	1.	1.0	1155.23
0.67	301.	3.	3.8	1155.87
0.83	552.	13.	9.5	1157.21
1.00	737.	23.	18.1	1159.22
1.17	793.	30.	28.3	1161.09
1.33	735.	34.	38.4	1162.70
1.50	612.	37.	47.1	1164.11
1.67	460.	40.	54.0	1165.15
1.83	325.	41.	58.8	1165.75
2.00	230.	42.	62.0	1166.15
2.17	173.	42.	64.2	1166.42
2.33	142.	43.	65.8	1166.61
2.50	119.	43.	67.0	1166.76
2.67	102.	43.	67.9	1166.87
2.83	82.	43.	68.6	1166.96
3.00	67.	43.	69.0	1167.01
3.17	54.	43.	69.3	1167.04
3.33	45.	44.	69.4	1167.05
3.50	41.	44.	69.3	1167.04
3.67	29.	43.	69.2	1167.03
3.83	18.	43.	69.0	1167.00
4.00	11.	43.	68.6	1166.95
4.17	6.	43.	68.1	1166.89
4.33	1.	43.	67.5	1166.82
4.50	1.	43.	66.9	1166.75
4.67	1.	43.	66.4	1166.68
4.83	1.	43.	65.8	1166.61
5.00	1.	42.	65.2	1166.54
5.17	1.	42.	64.7	1166.47
5.33	1.	42.	64.1	1166.40
5.50	1.	42.	63.5	1166.33
5.67	1.	42.	63.0	1166.26
5.83	1.	42.	62.4	1166.19
6.00	1.	42.	61.8	1166.12
6.17	1.	41.	61.3	1166.06

FLOOD ROUTING OUTPUT DATA---PAGE 2

WOODBURY COUNTY D.A. = 340. ACRES

I-IG-129-6(3)145--04-97 POND AT STA. 477+60
DRAINAGE AREA = 340 AC. - VERY HILLY
7.5 MIN. QUAD - SERGEANT BLUFF (93-A)
SINGLE 24 INCH CMP AT ELEV. 1155.0

DUR = 3.10 HR. RAIN = 3.91 IN. RUNOFF = 1.40 IN.

TIME HOURS	INFLOW CFS	OUTFLOW CFS	STORAGE AC-FT	ELEVATION FT
6.33	1.	41.	60.7	1165.99
6.50	1.	41.	60.2	1165.92
6.67	1.	41.	59.6	1165.85
6.83	1.	41.	59.1	1165.78
7.00	1.	41.	58.5	1165.72
7.17	1.	41.	58.0	1165.65
7.33	1.	40.	57.4	1165.58
7.50	1.	40.	56.9	1165.52
7.67	1.	40.	56.4	1165.45
7.83	1.	40.	55.8	1165.38
8.00	1.	40.	55.3	1165.32
8.17	1.	40.	54.8	1165.25
8.33	1.	40.	54.2	1165.19
8.50	1.	40.	53.7	1165.12
8.67	1.	39.	53.2	1165.06
8.83	1.	39.	52.6	1164.99
9.00	1.	39.	52.1	1164.91
9.17	1.	39.	51.6	1164.82
9.33	1.	39.	51.1	1164.74
9.50	1.	38.	50.6	1164.66
9.67	1.	38.	50.1	1164.58
9.83	1.	38.	49.5	1164.49
10.00	1.	38.	49.0	1164.41
10.17	1.	38.	48.5	1164.33
10.33	1.	38.	48.0	1164.25
10.50	1.	37.	47.5	1164.17
10.67	1.	37.	47.0	1164.09
10.83	1.	37.	46.5	1164.01
11.00	1.	37.	46.0	1163.93
11.17	1.	37.	45.6	1163.85
11.33	1.	36.	45.1	1163.78
11.50	1.	36.	44.6	1163.70
11.67	1.	36.	44.1	1163.62
11.83	1.	36.	43.6	1163.54
12.00	1.	36.	43.1	1163.47
12.17	1.	36.	42.7	1163.39
12.33	1.	35.	42.2	1163.31
12.50	1.	35.	41.7	1163.24

FLOOD ROUTING OUTPUT DATA---PAGE 3

WOODBURY COUNTY D.A. = 340. ACRES

I-IG-129-6(3)145--04-97 POND AT STA. 477+60
DRAINAGE AREA = 340 AC. - VERY HILLY
7.5 MIN. QUAD - SERGEANT BLUFF (93-A)
SINGLE 24 INCH CMP AT ELEV. 1155.0

DUR = 3.10 HR. RAIN = 3.91 IN. RUNOFF = 1.40 IN.

TIME HOURS	INFLOW CFS	OUTFLOW CFS	STORAGE AC-FT	ELEVATION FT
12.67	1.	35.	41.2	1163.16
12.83	1.	35.	40.8	1163.09
13.00	1.	35.	40.3	1163.02
13.17	1.	34.	39.9	1162.94
13.33	1.	34.	39.4	1162.87
13.50	1.	34.	38.9	1162.80
13.67	1.	34.	38.5	1162.72
13.83	1.	34.	38.0	1162.65
14.00	1.	34.	37.6	1162.58
14.17	1.	33.	37.1	1162.51
14.33	1.	33.	36.7	1162.44

RUNOFF VOLUME = 970. CFS-HOURS

FLOOD ROUTING OUTPUT DATA---PAGE 1

WOODBURY COUNTY D.A. = 340. ACRES

I-IG-129-6(3)145--04-97 POND AT STA. 477+60
DRAINAGE AREA = 340 AC. - VERY HILLY
7.5 MIN. QUAD - SERGEANT BLUFF (93-A)
SINGLE 24 INCH CMP AT ELEV. 1155.0

DUR = 4.13 HR. RAIN = 4.17 IN. RUNOFF = 1.58 IN.

TIME HOURS	INFLOW CFS	OUTFLOW CFS	STORAGE AC-FT	ELEVATION FT
0.00	0.	0.	0.0	1155.00
0.17	0.	0.	0.0	1155.00
0.33	3.	0.	0.0	1155.00
0.50	45.	0.	0.3	1155.08
0.67	159.	1.	1.7	1155.40
0.83	359.	5.	5.2	1156.22
1.00	574.	16.	11.5	1157.68
1.17	732.	25.	20.2	1159.70
1.33	786.	30.	30.3	1161.40
1.50	744.	35.	40.3	1163.02
1.67	632.	38.	49.3	1164.45
1.83	496.	40.	56.5	1165.47
2.00	373.	42.	61.9	1166.13
2.17	279.	43.	65.8	1166.61
2.33	215.	43.	68.6	1166.96
2.50	169.	44.	70.7	1167.21
2.67	137.	44.	72.2	1167.39
2.83	116.	44.	73.3	1167.53
3.00	105.	45.	74.2	1167.64
3.17	96.	45.	75.0	1167.74
3.33	84.	45.	75.6	1167.81
3.50	74.	45.	76.1	1167.87
3.67	62.	45.	76.4	1167.91
3.83	54.	45.	76.6	1167.93
4.00	40.	45.	76.6	1167.93
4.17	35.	45.	76.5	1167.92
4.33	36.	45.	76.3	1167.91
4.50	37.	45.	76.2	1167.89
4.67	24.	45.	76.0	1167.87
4.83	17.	45.	75.7	1167.82
5.00	11.	45.	75.3	1167.77
5.17	5.	45.	74.8	1167.71
5.33	1.	45.	74.2	1167.64
5.50	1.	45.	73.6	1167.57
5.67	1.	44.	73.0	1167.49
5.83	1.	44.	72.4	1167.42
6.00	1.	44.	71.8	1167.35
6.17	1.	44.	71.2	1167.27

FLOOD ROUTING OUTPUT DATA---PAGE 2

WOODBURY COUNTY D.A. = 340. ACRES

I-IG-129-6(3)145--04-97 POND AT STA. 477+60
DRAINAGE AREA = 340 AC. - VERY HILLY
7.5 MIN. QUAD - SERGEANT BLUFF (93-A)
SINGLE 24 INCH CMP AT ELEV. 1155.0

DUR = 4.13 HR. RAIN = 4.17 IN. RUNOFF = 1.58 IN.

TIME HOURS	INFLOW CFS	OUTFLOW CFS	STORAGE AC-FT	ELEVATION FT
6.33	1.	44.	70.6	1167.20
6.50	1.	44.	70.0	1167.13
6.67	1.	44.	69.4	1167.06
6.83	1.	43.	68.9	1166.99
7.00	1.	43.	68.3	1166.91
7.17	1.	43.	67.7	1166.84
7.33	1.	43.	67.1	1166.77
7.50	1.	43.	66.6	1166.70
7.67	1.	43.	66.0	1166.63
7.83	1.	43.	65.4	1166.56
8.00	1.	42.	64.8	1166.49
8.17	1.	42.	64.3	1166.42
8.33	1.	42.	63.7	1166.35
8.50	1.	42.	63.1	1166.28
8.67	1.	42.	62.6	1166.21
8.83	1.	42.	62.0	1166.15
9.00	1.	42.	61.5	1166.08
9.17	1.	41.	60.9	1166.01
9.33	1.	41.	60.4	1165.94
9.50	1.	41.	59.8	1165.87
9.67	1.	41.	59.3	1165.81
9.83	1.	41.	58.7	1165.74
10.00	1.	41.	58.2	1165.67
10.17	1.	41.	57.6	1165.60
10.33	1.	40.	57.1	1165.54
10.50	1.	40.	56.5	1165.47
10.67	1.	40.	56.0	1165.41
10.83	1.	40.	55.5	1165.34
11.00	1.	40.	54.9	1165.27
11.17	1.	40.	54.4	1165.21
11.33	1.	40.	53.9	1165.14
11.50	1.	39.	53.3	1165.08
11.67	1.	39.	52.8	1165.01
11.83	1.	39.	52.3	1164.93
12.00	1.	39.	51.8	1164.85
12.17	1.	39.	51.2	1164.77
12.33	1.	39.	50.7	1164.68
12.50	1.	38.	50.2	1164.60

FLOOD ROUTING OUTPUT DATA---PAGE 3

WOODBURY COUNTY D.A. = 340. ACRES

I-IG-129-6(3)145--04-97 POND AT STA. 477+60
DRAINAGE AREA = 340 AC. - VERY HILLY
7.5 MIN. QUAD - SERGEANT BLUFF (93-A)
SINGLE 24 INCH CMP AT ELEV. 1155.0

DUR = 4.13 HR. RAIN = 4.17 IN. RUNOFF = 1.58 IN.

TIME HOURS	INFLOW CFS	OUTFLOW CFS	STORAGE AC-FT	ELEVATION FT
12.67	1.	38.	49.7	1164.52
12.83	1.	38.	49.2	1164.44
13.00	1.	38.	48.7	1164.36
13.17	1.	38.	48.2	1164.28
13.33	1.	37.	47.7	1164.20
13.50	1.	37.	47.2	1164.12
13.67	1.	37.	46.7	1164.04
13.83	1.	37.	46.2	1163.96
14.00	1.	37.	45.7	1163.88
14.17	1.	37.	45.2	1163.80
14.33	1.	36.	44.7	1163.72
14.50	1.	36.	44.2	1163.64
14.67	1.	36.	43.8	1163.57
14.83	1.	36.	43.3	1163.49
15.00	1.	36.	42.8	1163.41
15.17	1.	35.	42.3	1163.34
15.33	1.	35.	41.9	1163.26
15.50	1.	35.	41.4	1163.19
15.67	1.	35.	40.9	1163.11
15.83	1.	35.	40.5	1163.04
16.00	1.	35.	40.0	1162.96
16.17	1.	34.	39.5	1162.89
16.33	1.	34.	39.1	1162.82
16.50	1.	34.	38.6	1162.75
16.67	1.	34.	38.2	1162.67
16.83	1.	34.	37.7	1162.60
17.00	1.	33.	37.3	1162.53
17.17	1.	33.	36.8	1162.46

RUNOFF VOLUME = 1096. CFS-HOURS

FLOOD ROUTING OUTPUT DATA---PAGE 1

WOODBURY COUNTY D.A. = 340. ACRES

I-IG-129-6(3)145--04-97 POND AT STA. 477+60
DRAINAGE AREA = 340 AC. - VERY HILLY
7.5 MIN. QUAD - SERGEANT BLUFF (93-A)
SINGLE 24 INCH CMP AT ELEV. 1155.0

DUR = 5.17 HR. RAIN = 4.37 IN. RUNOFF = 1.73 IN.

TIME HOURS	INFLOW CFS	OUTFLOW CFS	STORAGE AC-FT	ELEVATION FT
0.00	0.	0.	0.0	1155.00
0.17	0.	0.	0.0	1155.00
0.33	0.	0.	0.0	1155.00
0.50	18.	0.	0.1	1155.03
0.67	84.	1.	0.8	1155.19
0.83	217.	2.	2.9	1155.67
1.00	399.	8.	7.0	1156.63
1.17	576.	18.	13.5	1158.15
1.33	710.	26.	22.1	1160.09
1.50	762.	31.	31.8	1161.65
1.67	732.	35.	41.6	1163.23
1.83	636.	38.	50.5	1164.65
2.00	518.	41.	57.9	1165.64
2.17	405.	42.	63.7	1166.35
2.33	313.	43.	68.0	1166.89
2.50	246.	44.	71.3	1167.28
2.67	201.	45.	73.8	1167.59
2.83	168.	45.	75.7	1167.82
3.00	139.	45.	77.2	1168.01
3.17	116.	46.	78.3	1168.15
3.33	101.	46.	79.2	1168.25
3.50	91.	46.	79.9	1168.34
3.67	85.	46.	80.4	1168.41
3.83	83.	46.	80.9	1168.47
4.00	78.	46.	81.4	1168.53
4.17	68.	47.	81.8	1168.57
4.33	57.	47.	82.0	1168.60
4.50	50.	47.	82.1	1168.61
4.67	45.	47.	82.1	1168.61
4.83	33.	47.	82.0	1168.60
5.00	25.	46.	81.8	1168.57
5.17	28.	46.	81.5	1168.54
5.33	33.	46.	81.3	1168.51
5.50	32.	46.	81.1	1168.49
5.67	24.	46.	80.8	1168.46
5.83	17.	46.	80.5	1168.41
6.00	11.	46.	80.0	1168.36
6.17	4.	46.	79.5	1168.29

FLOOD ROUTING OUTPUT DATA---PAGE 2

WOODBURY COUNTY D.A. = 340. ACRES

I-IG-129-6(3)145--04-97 POND AT STA. 477+60
DRAINAGE AREA = 340 AC. - VERY HILLY
7.5 MIN. QUAD - SERGEANT BLUFF (93-A)
SINGLE 24 INCH CMP AT ELEV. 1155.0

DUR = 5.17 HR. RAIN = 4.37 IN. RUNOFF = 1.73 IN.

TIME HOURS	INFLOW CFS	OUTFLOW CFS	STORAGE AC-FT	ELEVATION FT
6.33	1.	46.	78.9	1168.22
6.50	1.	46.	78.3	1168.14
6.67	1.	46.	77.7	1168.07
6.83	1.	45.	77.1	1167.99
7.00	1.	45.	76.5	1167.92
7.17	1.	45.	75.9	1167.84
7.33	1.	45.	75.3	1167.77
7.50	1.	45.	74.6	1167.70
7.67	1.	45.	74.0	1167.62
7.83	1.	45.	73.5	1167.55
8.00	1.	44.	72.9	1167.48
8.17	1.	44.	72.3	1167.40
8.33	1.	44.	71.7	1167.33
8.50	1.	44.	71.1	1167.26
8.67	1.	44.	70.5	1167.19
8.83	1.	44.	69.9	1167.11
9.00	1.	44.	69.3	1167.04
9.17	1.	43.	68.7	1166.97
9.33	1.	43.	68.2	1166.90
9.50	1.	43.	67.6	1166.83
9.67	1.	43.	67.0	1166.76
9.83	1.	43.	66.4	1166.69
10.00	1.	43.	65.9	1166.62
10.17	1.	42.	65.3	1166.55
10.33	1.	42.	64.7	1166.48
10.50	1.	42.	64.1	1166.41
10.67	1.	42.	63.6	1166.34
10.83	1.	42.	63.0	1166.27
11.00	1.	42.	62.5	1166.20
11.17	1.	42.	61.9	1166.13
11.33	1.	41.	61.3	1166.06
11.50	1.	41.	60.8	1165.99
11.67	1.	41.	60.2	1165.93
11.83	1.	41.	59.7	1165.86
12.00	1.	41.	59.1	1165.79
12.17	1.	41.	58.6	1165.72
12.33	1.	41.	58.0	1165.66
12.50	1.	40.	57.5	1165.59

FLOOD ROUTING OUTPUT DATA---PAGE 3

WOODBURY COUNTY D.A. = 340. ACRES

I-IG-129-6(3)145--04-97 POND AT STA. 477+60
DRAINAGE AREA = 340 AC. - VERY HILLY
7.5 MIN. QUAD - SERGEANT BLUFF (93-A)
SINGLE 24 INCH CMP AT ELEV. 1155.0

DUR = 5.17 HR. RAIN = 4.37 IN. RUNOFF = 1.73 IN.

TIME HOURS	INFLOW CFS	OUTFLOW CFS	STORAGE AC-FT	ELEVATION FT
12.67	1.	40.	57.0	1165.52
12.83	1.	40.	56.4	1165.46
13.00	1.	40.	55.9	1165.39
13.17	1.	40.	55.3	1165.32
13.33	1.	40.	54.8	1165.26
13.50	1.	40.	54.3	1165.19
13.67	1.	40.	53.8	1165.13
13.83	1.	39.	53.2	1165.06
14.00	1.	39.	52.7	1165.00
14.17	1.	39.	52.2	1164.92
14.33	1.	39.	51.7	1164.83
14.50	1.	39.	51.1	1164.75
14.67	1.	38.	50.6	1164.67
14.83	1.	38.	50.1	1164.58
15.00	1.	38.	49.6	1164.50
15.17	1.	38.	49.1	1164.42
15.33	1.	38.	48.6	1164.34
15.50	1.	38.	48.1	1164.26
15.67	1.	37.	47.6	1164.18
15.83	1.	37.	47.1	1164.10
16.00	1.	37.	46.6	1164.02
16.17	1.	37.	46.1	1163.94
16.33	1.	37.	45.6	1163.86
16.50	1.	36.	45.1	1163.78
16.67	1.	36.	44.6	1163.71
16.83	1.	36.	44.1	1163.63
17.00	1.	36.	43.7	1163.55
17.17	1.	36.	43.2	1163.47
17.33	1.	36.	42.7	1163.40
17.50	1.	35.	42.2	1163.32
17.67	1.	35.	41.8	1163.25
17.83	1.	35.	41.3	1163.17
18.00	1.	35.	40.8	1163.10
18.17	1.	35.	40.4	1163.02
18.33	1.	35.	39.9	1162.95
18.50	1.	34.	39.4	1162.87
18.67	1.	34.	39.0	1162.80
18.83	1.	34.	38.5	1162.73

FLOOD ROUTING OUTPUT DATA---PAGE 4

WOODBURY COUNTY D.A. = 340. ACRES

I-IG-129-6(3)145--04-97 POND AT STA. 477+60
DRAINAGE AREA = 340 AC. - VERY HILLY
7.5 MIN. QUAD - SERGEANT BLUFF (93-A)
SINGLE 24 INCH CMP AT ELEV. 1155.0

DUR = 5.17 HR. RAIN = 4.37 IN. RUNOFF = 1.73 IN.

TIME HOURS	INFLOW CFS	OUTFLOW CFS	STORAGE AC-FT	ELEVATION FT
19.00	1.	34.	38.1	1162.66
19.17	1.	34.	37.6	1162.58
19.33	1.	33.	37.2	1162.51
19.50	1.	33.	36.7	1162.44

RUNOFF VOLUME = 1199. CFS-HOURS

SUMMARY OF FLOOD ROUTINGS

WOODBURY COUNTY D.A. = 340. ACRES

I-IG-129-6(3)145--04-97 POND AT STA. 477+60
DRAINAGE AREA = 340 AC. - VERY HILLY
7.5 MIN. QUAD - SERGEANT BLUFF (93-A)
SINGLE 24 INCH CMP AT ELEV. 1155.0

INFLOW HYDROGRAPH NUMBER	STORM DURATION HR	TOTAL RAINFALL IN	TOTAL RUNOFF IN
1	0.26	2.07	0.32
2	0.52	2.51	0.54
3	1.03	3.01	0.82
4	2.07	3.56	1.16
5	3.10	3.91	1.40
6	4.13	4.17	1.58
7	5.17	4.37	1.73

INFLOW HYDROGRAPH NUMBER	MAX INFLOW CFS	MAX OUTFLOW CFS	TIME MAX OUTFLOW HR	MAX STORAGE AC-FT	MAX ELEV HEADWATER FT
1	313.	12.	0.75	8.8	1157.04
2	538.	20.	0.83	14.8	1158.43
3	624.	36.	1.67	42.6	1163.39
4	765.	41.	2.50	59.6	1165.84
5	793.	44.	3.33	69.4	1167.05
6	786.	45.	4.00	76.6	1167.93
7	762.	47.	4.67	82.1	1168.61

APPENDIX D.

PROGRAM LISTING

```
C
C
C         HDC  -  HYDROLOGIC DESIGN OF CULVERTS
C
C
C         A COMPUTER PROGRAM FOR THE HYDROLOGIC AND HYDRAULIC
C         DESIGN OF CULVERTS UTILIZING THE TEMPORARY PONDING
C         AT THE INLET OF THE CULVERT
C
C
      COMMON I,CO(99,14),A,ID1(17),ID2(17),ID3(17),ID4(17),
     1 NOG
      COMMON /RR1/ KD(99),NP,PCD(21),PCP(21,14),ARI,LF,FF,
     1 LEN,H
      COMMON /RR2/ MIT(5),THW(5),TFL(5),MBR(5),TBA(5),TD(5),
     1 TDI(5),TWL(5)
      COMMON /RR3/ NSL,EST(75),STR(75),BGL,ENL
      INTEGER*4 ARI
      REAL*4 LF,LEN
      NOG=1
C
C          READS IN COUNTY NUMBERS, RAINFALL ADJUSTMENT
C          FACTOR, COUNTY NAME, SCS RUNOFF CURVE NUMBER,
C          AND RAINFALL EQUATION EXPONENTS
C
      DO 1 J=1,99
      READ (5,900) I,KD(I),(CO(I,M),M=1,9)
  900 FORMAT (I2,I4,4A4,F4.0,F4.2,3F4.3)
    1 CONTINUE
C
C          READS IN PERCENT OF TOTAL STORM DURATION AND
C          PERCENT OF TOTAL STORM RAINFALL FOR FOURTEEN
C          TIME DISTRIBUTIONS OF RAINFALL (ISD=1,...,14)
C
      NP=21
      DO 2 J=1,21
      READ (5,901) M,PCD(M),(PCP(M,K),K=1,14)
  901 FORMAT (I5,15F5.2)
    2 CONTINUE
C
C          READS IN HYDROLOGIC DATA
C
C                I = COUNTY NUMBER
C              ARI = RECURRENCE INTERVAL
C                A = DRAINAGE AREA
C               LF = LAND USE AND SLOPE FACTOR
C               FF = FREQUENCY FACTOR
C              LEN = LENGTH OF MAIN CHANNEL
C                H = DUFFERENCE IN ELEVATION OF MAIN CHANNEL
C              NST = NUMBER OF STORAGE ELEVATIONS TO BE READ
C
    5 READ (5,902) I,ARI,A,LF,FF,LEN,H,NST
  902 FORMAT (2I10,F10.0,2F10.2,2F10.0,I5)
C
C          READS IN STAGE-STORAGE DATA
C
C              EST = ELEVATION
C              STR = TOTAL VOLUME OF STORAGE
C                    BELOW THAT ELEVATION
```

```
C
      IF (NST.EQ.0) GO TO 4
      NSL=NST
      DO 3 J=1,NSL
      READ (5,954) EST(J),STR(J)
  954 FORMAT (2F10.1)
    3 CONTINUE
C
C          READS IN IDENTIFICATION DATA
C
    4 READ (5,990) (ID1(J),J=1,17)
      READ (5,990) (ID2(J),J=1,17)
      READ (5,990) (ID3(J),J=1,17)
      READ (5,990) (ID4(J),J=1,17)
  990 FORMAT (5X,17A4)
C
C          READS IN HYDRAULIC DATA
C
C               MIT = INLET TYPE
C               THW = HEADWATER ELEVATION
C               TFL = INVERT ELEVATION
C               MBR = NUMBER OF PIPES OR BARRELS
C               TBA = WIDTH OF BARREL
C                TD = HEIGHT OF BARREL
C               TDI = DIAMETER OF PIPE
C               TWL = LENGTH OF WEIR
C
      DO 9 J=1,5
      READ (5,919) IL,MIT(IL),THW(IL),TFL(IL),MBR(IL),
     1 TBA(IL),TD(IL),TDI(IL),TWL(IL)
  919 FORMAT (2I5,F10.0,F10.1,I10,4F10.0)
C
C          EST(NSL) = HIGHEST STORAGE ELEVATION
C            THW(1) = HEADWATER ELEVATION
C
C          HIGHEST STORAGE ELEVATION MUST BE EQUAL TO
C          OR GREATER THAN HEADWATER ELEVATION
C
C          IF NOT, EXECUTION CEASES AND A MESSAGE IS OUTPUT
C
      IF (EST(NSL).LT.THW(1)) GO TO 1300
      GO TO 1310
 1300 WRITE (6,1305)
 1305 FORMAT ('1',9X,'REVISE INPUT SO THAT THE HIGHEST'/10X,
     1 'STORAGE ELEVATION IN THE INPUT DATA IS EQUAL'/10X,
     2 'TO OR GREATER THAN THE HEADWATER ELEVATION'/10X,
     3 'LISTED IN THE HYDRAULIC INPUT DATA')
      GO TO 1315
C
C          BGL = BEGINNING ELEVATION OF FLOOD ROUTING
C
 1310 IF (IL.EQ.1) BGL=TFL(1)
      IF (IL.EQ.2) GO TO 6
      GO TO 9
    6 IF (MIT(2).GT.0) BGL=TFL(2)
    9 CONTINUE
C
C          ENL = ENDING ELEVATION OF FLOOD ROUTING
C
      ENL=BGL+2.5
```

```
C
C          OUTPUTS ALL INPUT DATA
C
           WRITE (6,191)
       191 FORMAT ('1'////40X,'I  N  P  U  T     D  A  T  A'//20X,
         1 'INFLOW CATA')
           WRITE (6,192) (CO(I,M),M=1,4),ARI,A,LF,FF
       192 FORMAT ('0',21X,4A4,'  COUNTY'//22X,'RECURRENCE ',
          1 'INTERVAL =',I7,' YEARS'//22X,'DRAINAGE AREA',7X,
          2 '=',F7.0,' ACRES'//22X,'LAND FACTOR',9X,'=',F7.2,//
          3 22X,'FREQUENCY FACTOR     =',F7.2)
           WRITE (6,315) LEN,H
       315 FORMAT ('0',21X,'LENGTH OF STREAM     =',F7.0,
          1 ' FEET'//22X,'DIFF. IN ELEVATION  =',F7.0,' FEET')
           WRITE (6,193)
       193 FORMAT ('0',19X,'STAGE-STORAGE CATA'//33X,'ELEV',
          1 '      AC-FT'/)
           DO 194 J=1,NSL
           WRITE (6,195) EST(J),STR(J)
       195 FORMAT (' ',F37.1,F9.1)
       194 CONTINUE
           WRITE (6,196)
       196 FORMAT ('0',19X,'OUTFLOW DATA')
           WRITE (6,298) (ID1(J),J=1,17)
       298 FORMAT ('0',21X,17A4)
           WRITE (6,299) (ID2(J),J=1,17)
           WRITE (6,299) (ID3(J),J=1,17)
           WRITE (6,299) (ID4(J),J=1,17)
       299 FORMAT (' ',21X,17A4)
           WRITE (6,199)
       199 FORMAT ('0',21X,'INLET  HDWATER  FLOLINE  NO.  WIDTH',
          1 '  HEIGHT  DIAM.  WEIR L'/23X,'TYPE   ELEV.     ',
          2 'ELEV.',9X,'FEET    FEET     IN.     FEET')
           DO 197 J=1,5
           WRITE (6,198) MIT(J),THW(J),TFL(J),MBR(J),TBA(J),
          1 TC(J),TCI(J),TWL(J)
       198 FORMAT ('0',I24,F10.1,F9.1,I5,F8.1,2F7.1,F8.1)
       197 CONTINUE
C
C          GO TO 55 IF SAME INFLOW HYDROGRAPHS ARE TO BE USED
C
           IF (NOG.EQ.0) GO TO 55
C
C          FLD - SUBROUTINE FOR CALCULATING THE
C                INFLOW HYDROGRAPH
C
           CALL FLD
C
C          HYD - SUBROUTINE FOR CALCULATING THE
C                CULVERT HYDRAULICS (STAGE-OUTFLOW CURVE)
C
        55 CALL HYD
C
C          RTG - SUBROUTINE FOR PERFORMING THE FLOOD ROUTING
C
           CALL RTG
C
C          ARE SAME INFLOW HYDROGRAPHS TO BE USED?
C
           READ (5,56) NOG
```

```
   56 FORMAT (I1)
C
C          ARE MCRE CALCULATIONS TO BE PERFORMED?
C
      READ (5,995) MOR
  995 FORMAT (I1)
      IF (MOR.EQ.1) GO TO 5
      WRITE (6,918)
  918 FORMAT ('1',9X,'END  OF  JOB')
 1315 STOP
      END
C
C

      SUBROUTINE FLD
C
C
C          THIS PORTION WILL COMPUTE THE INFLOW HYDROGRAPH
C          FOR DRAINAGE AREAS LESS THAN 25 SQUARE MILES
C          FOR ANY DESIRED RECURRENCE INTERVAL
C
C
      COMMON I,CO(99,14),A,ID1(17),ID2(17),ID3(17),ID4(17),
     1 NOG
      CCMMON /RR1/ KC(99),NP,PCD(21),PCP(21,14),ARI,LF,FF,
     1 LEN,H
      CCMMON /RR4/ DRN(7),DLT(7),PRP(7),QI(75,7),TI(75,7),
     1 SR(7),NIT(7),BGT,ENT
      DIMENSICN PX(75,7),SRO(75,7),DSR(75,7),DPQ(75,7),
     1 CN(75),TIM(75,7)
      INTEGER*4 ARI
      REAL*4 LF,LEN
C
C          TC = TIME CF CONCENTRATION
C          DD = DELTA DURATION OF STORM
C               EQUAL TIME INCREMENTS FCR ROUTING
C
      TC=(11.9*(LEN/5280.)**3/H)**.386
    7 DD=14.3*TC
C
C          ROUNDS OFF DC TO AN EVEN TIME INCREMENT
C
      IF (DC.LE.7.5) DD=5.
      IF (DD.GT.7.5.AND.DD.LE.12.5) DD=10.
      IF (DD.GT.12.5.AND.DD.LE.17.5) DD=15.
      IF (DD.GT.17.5.AND.DD.LE.22.5) DD=20.
      IF (DD.GT.22.5.AND.DD.LE.27.5) DD=25.
      IF (DC.GT.27.5.AND.DD.LE.32.5) DD=30.
      IF (DD.GT.32.5.AND.DD.LE.37.5) DD=35.
      IF (DD.GT.37.5.AND.DD.LE.42.5) DD=40.
      IF (DD.GT.42.5.AND.DD.LE.47.5) DD=45.
      IF (DD.GT.47.5.AND.DD.LE.52.5) DD=50.
      IF (DD.GT.52.5.AND.DD.LE.57.5) DD=55.
      IF (DD.GT.57.5.AND.DD.LE.62.5) DD=60.
      IF (DD.GT.62.5.AND.DD.LE.75.) DD=70.
      IF (DD.GT.75.0.AND.DD.LE.85.) DD=80.
      IF (DD.GT.85.0.AND.DD.LE.95.) DD=90.
      IF (DD.GT.95.0.AND.DD.LE.105.) DD=100.
      IF (DD.GT.105.0.AND.DD.LE.115.) DD=110.
      IF (DD.GT.115.0.AND.DD.LE.130.) DD=120.
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      IF (DD.GT.130.0.AND.DD.LE.150.) DD=140.
      IF (DD.GT.150.0.AND.DD.LE.170.) DD=160.
      IF (DD.GT.170.) DD=180.
C
C          TP = TIME TO PEAK OF TRIANGULAR HYDROGRAPH
C          TB = TIME BASE OF TRIANGULAR HYDROGRAPH
C         UPQ = UNIT PEAK Q
C               PEAK DISCHARGE OF A TRIANGULAR HYDROGRAPH
C               WITH ONE INCH OF RUNOFF
C
      TP=3.*DC
      TB=8.*DC
      UPQ=45.4*A/TP
C
C         DUR = TOTAL STORM DURATION
C               ONE OF SEVEN STORM DURATICNS
C
C         ISD = TIME DISTRIBUTION OF RAINFALL PATTERN
C             = 1,...,14
C
      DO 50 L=1,7
      IF (L.EQ.1) DUR=.5*TC
      IF (L.EQ.2) DUR=TC
      IF (L.GT.2) GO TO 70
      IF (TC.LE.4.2) ISD=6
      IF (TC.GT.4.2) ISD=10
      GO TO 98
   70 IF (TC.LE.0.5) DUR=3.*(L-2.)*TC
      IF (TC.GT.0.5.AND.TC.LE.1.0) DUR=2.*(L-2.)*TC
      IF (TC.GT.1.0.AND.TC.LE.3.0) DUR=1.5*(L-2.)*TC
      IF (TC.GT.3.0) DUR=1.0*(L-1.)*TC
      IF (TC.LE.0.6)  DD=10.
C
C         DUR
C         FIRST OF TWO STEPS TO DETERMINE THE
C         TIME DISTRIBUTION OF RAINFALL PATTERN
C
      IF (DUR.GT.13.0.AND.DUR.LE.25.0) GO TO 94
      IF (DUR.GT.25.0) GO TO 97
      IF (DUR.LE.0.3) GO TO 72
      IF (DUR.GT.0.3.AND.DUR.LE.0.325) GO TO 74
      IF (DUR.GT.0.325.AND.DUR.LE.0.6) GO TO 76
      IF (DUR.GT.0.6.AND.DUR.LE.1.0) GO TO 79
      IF (DUR.GT.1.0.AND.DUR.LE.2.5) GO TO 80
      IF (DUR.GT.2.5.AND.DUR.LE.3.0) GO TO 84
      IF (DUR.GT.3.0.AND.DUR.LE.5.0) GO TO 90
      IF (DUR.GT.5.0) GO TO 92
C
C         LF
C         SECCND OF TWC STEPS TC DETERMINE THE
C         TIME DISTRIBUTICN OF RAINFALL PATTERN
C
C         ISD   QUARTER OF      PERCENT
C         NO.   TOTAL STORM   PROBABILITY
C                DURATION
C
C          1         1             10
C          2         1             30
C          3         1             50
C          4         1             60
```

```
C             5            1             70
C             6            2             10
C             7            2             30
C             8            2             50
C             9            2             70
C            10            2             90
C            11            3             10
C            12            3             30
C            13            3             50
C            14            4             10
C
   72 IF (LF.LE.0.8) ISD=3
      IF (LF.GT.0.8.AND.LF.LE.0.88) ISD=2
      IF (LF.GT.0.88) ISD=1
      GO TO 98
   74 IF (LF.LE.0.8) ISD=4
      IF (LF.GT.0.8.AND.LF.LE.0.88) ISD=2
      IF (LF.GT.0.88) ISD=1
      GO TO 98
   76 IF (LF.LE.0.8) GO TO 77
      GO TO 78
   77 ISD=5
      GO TO 98
   78 IF (LF.GT.0.8.AND.LF.LE.0.92) ISD=2
      IF (LF.GT.0.92) ISD=1
      GO TO 98
   79 IF (LF.LE.0.66) ISD=3
      IF (LF.GT.0.66.AND.LF.LE.0.88) ISD=2
      IF (LF.GT.0.88) ISD=1
      GO TO 98
   80 IF (LF.GT.0.56.AND.LF.LE.0.92) GO TO 82
      GO TO 88
   82 ISD=8
      GO TO 98
   84 IF (LF.GT.0.56.AND.LF.LE.0.92) GO TO 86
      GO TO 88
   86 ISD=9
      GO TO 98
   88 IF (LF.LE.0.56) ISD=7
      IF (LF.GT.0.92) ISD=1
      GO TO 98
   90 IF (LF.LE.0.92) ISD=8
      IF (LF.GT.0.92) ISD=1
      GO TO 98
   92 IF (LF.LE.0.92) ISD=9
      IF (LF.GT.0.92) ISD=1
      GO TO 98
   94 IF (TC.GT.3.0) GO TO 96
      IF (LF.GT.0.56.AND.LF.LE.0.92) GO TO 96
      ISD=11
      GO TO 98
   96 ISD=13
      GO TO 98
   97 ISD=14
C
C          TMI = NUMBER OF TIME INCREMENTS
C          DRN = STORM DURATION FOR EACH OF SEVEN STORMS
C          DLT = DELTA DURATION FOR EACH OF SEVEN STORMS
C
   98 TMI=60.*DUR/DD
```

```
          DRN(L)=DUR
          DLT(L)=DD/60.
          TMI=TMI-.5
C
C              INC = NUMBER OF TIME INCREMENTS
C                    CALCULATION OF NUMBER OF TIME
C                    INCREMENTS AS AN INTEGER
C
          DO 20 J=1,100
          AK=J
          IF (TMI.LT.0.0) GO TO 25
          TMI=TMI-1.
       20 CONTINUE
       25 INC=AK-1.
C
C              P = PRECIPITATION
C                  CALCULATION OF PRECIPITATION
C                  FOR TOTAL STORM DURATION
C
          AA=CO(I,6)
          AB=CO(I,7)
          AC=CO(I,8)
          AD=CO(I,9)
          KK=KD(I)
          GO TO (33,34,35),KK
       33 ADJ=CO(I,6)
          GO TO 36
       34 ADJ=AA*ARI**(AB/ARI**AC)/DUR**AD
          GO TO 36
       35 ADJ=AA*ARI**(AB/ARI**AC)*DUR**AD
       36 P=1.32*ARI**(.264/ARI**.065)*DUR**(.266/DUR**.05)*ADJ
          PRP(L)=P
          NTI=INC+1
C
C              INITIALIZE VARIABLES
C
C               PX = ACCUMULATIVE PRECIPITATION FOR PERCENTAGES
C                    OF TOTAL STORM DURATION
C              SRO = ACCUMULATIVE SURFACE RUNOFF
C              DSR = INCREMENTAL SURFACE RUNOFF
C              DPQ = PEAK Q OF TRIANGULAR HYDROGRAPH
C                    FOR EACH DELTA TIME INCREMENT
C               QN = ORDINATE OF INCREMENTAL
C                    TRIANGULAR HYDROGRAPH
C               QI = ORDINATE OF FINAL INFLOW HYDROGRAPH
C              TIM = TIME IN MINUTES
C               TI = TIME IN HOURS
C
          DO 101 J=1,75
          PX(J,L)=0.0
          SRO(J,L)=0.0
          DSR(J,L)=0.0
          DPQ(J,L)=0.0
          QN(J)=0.0
          TIM(J,L)=0.0
          QI(J,L)=0.0
          TI(J,L)=0.0
      101 CONTINUE
          S=1000./CO(I,5)-10.
C
```

```
C           COMPUTES CUMULATIVE RUNOFF AMOUNTS FOR
C           PERCENTAGES OF TOTAL STORM DURATION
C
      RNT=NTI-1
      DO 110 J=2,NTI
      RJ=J-1
      RAT=RJ/RNT
      PCT=VL2(RAT,NP,PCD,PCP,ISC)
      PX(J,L)=P*PCT
      SRO(J,L)=(PX(J,L)-.2*S)**2/(PX(J,L)+.8*S)
      IF (PX(J,L)-.2*S) 102,102,110
  102 SRO(J,L)=0.0
  110 CONTINUE
C
C           COMPUTES SURFACE RUNOFF AND PEAK OF TRIANGULAR
C           HYDROGRAPH FOR EACH DELTA TIME INCREMENT
C
      SR(L)=SRO(NTI,L)
      DO 130 J=1,INC
      DSR(J+1,L)=SRO(J+1,L)-SRO(J,L)
      DPQ(J+1,L)=DSR(J+1,L)*UPQ
  130 CONTINUE
C
C           COMPUTES ORDINATES OF INCREMENTAL
C           TRIANGULAR HYDROGRAPHS
C
      DO 140 J=2,NTI
      QN(J-1)=0.
      QN(J)=.333*DPQ(J,L)
      QN(J+1)=.667*DPQ(J,L)
      QN(J+2)=DPQ(J,L)
      QN(J+3)=.8*DPQ(J,L)
      QN(J+4)=.6*DPQ(J,L)
      QN(J+5)=.4*DPQ(J,L)
      QN(J+6)=.2*DPQ(J,L)
      QN(J+7)=0.
C
C           COMPUTES ORDINATES OF FINAL INFLOW HYDROGRAPHS
C
      QI(J-1,L)=QI(J-1,L)+QN(J-1)
      QI(J,L)=QI(J,L)+QN(J)
      QI(J+1,L)=QI(J+1,L)+QN(J+1)
      QI(J+2,L)=QI(J+2,L)+QN(J+2)
      QI(J+3,L)=QI(J+3,L)+QN(J+3)
      QI(J+4,L)=QI(J+4,L)+QN(J+4)
      QI(J+5,L)=QI(J+5,L)+QN(J+5)
      QI(J+6,L)=QI(J+6,L)+QN(J+6)
      QI(J+7,L)=QI(J+7,L)+QN(J+7)
  140 CONTINUE
C
C           COMPUTES CUMULATIVE TIMES OF FINAL INFLOW
C           HYDROGRAPHS IN MINUTES AND HOURS
C
      INB=INC+8
      DO 150 J=1,INB
      TIM(J+1,L)=TIM(J,L)+DD
      TI(J+1,L)=TIM(J+1,L)/60.
  150 CONTINUE
C
C           TIM,TI = TIME IN MINUTES AND HOURS OF BASE FLOW
```

```
C                    RATE AT END OF INFLOW HYDROGRAPHS
C               QI = BASE FLOW RATE FOR INFLOW HYDROGRAPHS
C              BGT = BEGINNING TIME FOR FLOOD ROUTING
C              ENT = ENDING TIME FOR FLOOD ROUTING
C
      NIT(L)=INB+1
      NLT=NIT(L)
      TIM(NLT,L)=80.*TB
      TI(NLT,L)=TIM(NLT,L)/60.
      ENT=TIM(NLT,L)/960.
      BGT=0.0
      QI(NLT,L)=A/320.
      IF (QI(NLT,L).LT.1.) QI(NLT,L)=1.
      IF (QI(NLT,L).LT.QI(INB,L)) GO TO 155
      QI(INB,L)=QI(NLT,L)
C
C         QCH = Q FROM PEAK RATES OF RUNOFF CHART OF ISHC
C         QDS = DESIGN Q OF ISHC
C         QIM = MAXIMUM INFLOW RATE
C
  155 QCH=6.499*A**(.858/A**.0155)-
     1(ALOG(.11*A))**1.88*A**(1.21/A**.05)/75.
      QDS=LF*FF*QCH
      IND=INC+5
      QIM=0.0
C
C         COMPARE QIM WITH QDS
C         USE THE LARGER OF THE TWO VALUES
C
      DO 160 J=1,IND
      IF (QIM.GT.QI(J,L)) GO TO 160
      KL=J
      QIM=QI(J,L)
  160 CONTINUE
      IF (QIM.GT.QDS) GO TO 170
      QI(KL,L)=QDS
  170 CONTINUE
C
C         OUTPUTS DETAILS OF INCREMENTAL UNIT HYDROGRAPH,
C         INCREMENTAL RAINFALLS AND RUNOFFS, AND FINAL
C         INFLOW HYDROGRAPH FOR THE STORM DURATION
C         EQUAL TO THE TIME OF CONCENTRATION
C
      IF (L-2) 50,175,50
  175 WRITE (6,910) (CC(I,M),M=1,4),A
  910 FORMAT ('1'//20X,4A4,'  COUNTY',6X,'D.A. =',F6.0,
     1 ' ACRES'//29X,'INFLOW  HYDROGRAPH  CALCULATIONS')
  176 WRITE (6,911) TC,DD,TP,TB,UPQ
  911 FORMAT ('0'/20X,'TIME OF CONCENTRATION =',F6.2,
     1 ' HR.'//20X,'DELTA DURATION =',F4.0,' MIN.'//20X,
     2 'TIME TO PEAK =',F5.0,' MIN.'//20X,'TIME BASE =',
     3 F5.0,' MIN.'//20X,'UNIT PEAK Q =',F6.0,' CFS')
  177 WRITE (6,297) DUR
  297 FORMAT ('0'/20X,'STORM DURATION =',F6.2,' HR.')
  178 WRITE (6,912)
  912 FORMAT ('0'/23X,'TIME    RAINFALL   RUNOFF   DELTA',
     1 ' SRO  DELTA Q'/23X,'MIN.     INCHES    INCHES',
     2 '    INCHES      CFS'/)
      DO 180 J=1,NTI
      WRITE (6,913) TIM(J,L),PX(J,L),SRO(J,L)
```

```
  913 FORMAT (' ',F26.0,2F10.2)
      IF (J.EQ.NTI) GO TO 185
      WRITE (6,914) DSR(J+1,L),DPQ(J+1,L)
  914 FORMAT (' ',F56.2,F10.0)
  180 CONTINUE
  185 WRITE (6,915)
  915 FORMAT (' '//33X,'TIME        TIME         Q'/33X,
     1 'MIN.',7X,'HR.',7X,'CFS'/)
      DO 187 J=1,NLT
      WRITE (6,916) TIM(J,L),TI(J,L),QI(J,L)
  916 FORMAT (' ',F36.0,F10.2,F10.0)
  187 CONTINUE
  190 WRITE (6,917) QIM,QDS
  917 FORMAT ('0',24X,'MAX. Q =',F6.0,' CFS',6X,
     1 'DES. Q =',F6.0,' CFS')
   50 CONTINUE
      RETURN
      END

      FUNCTION VL2 (X,NPT,BX,BY,NZ)
C
C
C   INTERPOLATION ROUTINE FOR CASES WHERE THE KNOWN
C   VARIABLE IS IN A ONE-DIMENSIONAL ARRAY AND THE
C   UNKNOWN VARIABLE IS IN A TWO-DIMENSIONAL ARRAY.
C
C
      DIMENSION BX(21),BY(21,14)
      DO 1 K=2,NPT
      IF(X-BX(K)) 2,2,1
    1 CONTINUE
    2 J=K-1
      VL2=BY(J,NZ)+(BY(K,NZ)-BY(J,NZ))*(X-BX(J))/
     1 (BX(K)-BX(J))
      RETURN
      END
C
C

      SUBROUTINE HYD
C
C
C        THIS PORTION WILL COMPUTE THE DISCHARGES AT
C        VARYING HEADWATER DEPTHS FOR SINGLE, TWIN, AND
C        TRIPLE BOX AND PIPE CULVERTS OF ANY SIZE
C        OPERATING WITH INLET CONTROL - PLUS DROP INLETS
C        AND FLOW OVER THE HIGHWAY GRADE
C
C        BOX CULVERTS   HEC NO. 5 - PROGRAM HY-3
C             KTP=1 - 30 TO 75 DEGREE WINGWALL FLARE
C             KTP=2 - 90 OR 15 DEGREE WINGWALL FLARE
C             KTP=3 - PARALLEL WINGWALLS
C        PIPE CULVERTS   HEC NO. 5 - PROGRAM HY-1
C          REINFORCED CONCRETE PIPE
C             KTP=4 - SOCKET-END PROJECTING
C             KTP=5 - SOCKET-END IN A 90 DEGREE HEADWALL
C             KTP=6 - STANDARD END SECTION
C          CORREGATED METAL PIPE
C             KTP=7 - PROJECTING FROM FILL
C             KTP=8 - MITERED TO FILL SLOPE
```

```
C                 KTP=9 - IN A 90 DEGREE HEADWALL
C                 KTP=10- STANDARD END SECTION
C             KTP=11 - DROP INLET WEIR
C             KTP=12 - WEIR, ROADWAY OVERTOPPED
C
C
      COMMON I,CO(99,14),A,ID1(17),ID2(17),ID3(17),ID4(17),
     1 NOG
      COMMON /RR2/ MIT(5),THW(5),TFL(5),MBR(5),TBA(5),TD(5),
     1 TDI(5),TWL(5)
      COMMON /RR5/ EOU(75),QOU(75),ELO(75),QO(75),HWL,NOL
      DIMENSION QM(75,6),QEQ(450),FL(5),Q(75),HW(75),Y(75)
      EQUIVALENCE (QM(1,1),QEQ(1))
C
C           INITIALIZE VARIABLES
C
      DO 997 J=1,6
      QM(75,J)=0.
  997 CONTINUE
      DO 998 J=1,75
      EOU(J)=0.
      QOU(J)=0.
      ELO(J)=0.
      QO(J)=0.
  998 CONTINUE
      DO 999 J=1,450
      QEQ(J)=0.
  999 CONTINUE
      ITT=1
      DO 1100 IE=1,5
      IT=MIT(IE)
      HWE=THW(IE)
      FL(IE)=TFL(IE)
      BNR=MBR(IE)
      B=TBA(IE)
      D=TD(IE)
      DIA=TDI(IE)
      WL=TWL(IE)
      IF (IT.GT.0) GO TO 1090
      GO TO (1100,1095,1100,1100,1100),IE
 1090 LFL=FL(IE)
      DEC=FL(IE)-LFL
      AHW=HWE-FL(IE)
      IF (IE.GT.1) GO TO 1092
      LF1=LFL
      DC1=DEC
      AH1=AHW
      HWL=HWE
 1092 LAD=LFL-LF1
      DO 1105 J=1,74
      Q(J)=0.
 1105 CONTINUE
      GO TO (410,420,430,710,720,730,740,750,760,730,
     1 1001,1002),IT
C
C           KTP=1 - 30 TO 75 DEGREE WINGWALL FLARE
C
  410 DO 200 J=1,74
      HW(J)=J-DEC
      Y(J)=HW(J)/D
```

```
      X=5.
      DO 204 K=1,100
      G=.0724927+.507087*X-.117474*X**2+.0221702*X**3-
     1 .00148958*X**4+.0000380126*X**5-Y(J)
      IF (ABS(G).LE.0.0005) GO TO 202
      H=.507087-.234948*X+.0663406*X**2-.00595832*X**3+
     1 .000190063*X**4
      DX=G/H
      X=X-DX
  204 CONTINUE
  202 Q(J)=BNR*X*B*D**1.5
      IF (Q(J).LT.0.0) Q(J)=2.*B*BNR
      LEV=LAD+J+1
      QM(LEV,IE)=Q(J)
      TRY=AHW-HW(J)-1.
      IF (TRY.LT.0.0) GO TO 1100
  200 CONTINUE
      GO TO 1100
C
C         KTP=2 - 90 OR 15 DEGREE WINGWALL FLARE
C
  420 DO 210 J=1,74
      HW(J)=J-DEC
      Y(J)=HW(J)/D
      X=5.
      DO 214 K=1,100
      G=.122117+.505435*X-.10856*X**2+.0207809*X**3-
     1 .00136757*X**4+.0000345642*X**5-Y(J)
      IF (ABS(G).LE.0.0005) GO TO 212
      H=.505435-.21712*X+.0623427*X**2-.00547028*X**3+
     1 .000172821*X**4
      DX=G/H
      X=X-DX
  214 CONTINUE
  212 Q(J)=BNR*X*B*D**1.5
      IF (Q(J).LT.0.0) Q(J)=2.*B*BNR
      LEV=LAD+J+1
      QM(LEV,IE)=Q(J)
      TRY=AHW-HW(J)-1.
      IF (TRY.LT.0.0) GO TO 1100
  210 CONTINUE
      GO TO 1100
C
C         KTP=3 - PARALLEL WINGWALLS
C
  430 DO 220 J=1,74
      HW(J)=J-DEC
      Y(J)=HW(J)/D
      X=5.
      DO 224 K=1,100
      G=.144138+.461363*X-.0921507*X**2+.0200028*X**3-
     1 .00136449*X**4+.0000358431*X**5-Y(J)
      IF (ABS(G).LE.0.0005) GO TO 222
      H=.461363-.1843014*X+.0600084*X**2-.00545796*X**3+
     1 .0001792155*X**4
      DX=G/H
      X=X-DX
  224 CONTINUE
  222 Q(J)=BNR*X*B*D**1.5
      IF (Q(J).LT.0.0) Q(J)=2.*B*BNR
```

```
      LEV=LAD+J+1
      QM(LEV,IE)=Q(J)
      TRY=AHW-HW(J)-1.
      IF (TRY.LT.0.0) GO TO 1100
  220 CONTINUE
      GO TO 1100
C
C        KTP=4 - SOCKET-END PROJECTING
C
  710 DO 800 J=1,74
      HW(J)=J-DEC
      Y(J)=12.*HW(J)/DIA
      X=5.
      DO 804 K=1,100
      G=.108786+.662381*X-.233801*X**2+.0579585*X**3-
     1 .0055789*X**4+.000205052*X**5-Y(J)
      IF (ABS(G).LE.0.0005) GO TO 802
      H=.662381-.467602*X+.1738755*X**2-.0223156*X**3+
     1 .00102526*X**4
      DX=G/H
      X=X-DX
  804 CONTINUE
  802 Q(J)=BNR*X*(DIA/12.)**2.5
      IF (Q(J).LT.0.0) Q(J)=BNR*DIA/12.
      LEV=LAD+J+1
      QM(LEV,IE)=Q(J)
      TRY=AHW-HW(J)-1.
      IF (TRY.LT.0.0) GO TO 1100
  800 CONTINUE
      GO TO 1100
C
C        KTP=5 - SOCKET-END IN A 90 DEGREE HEADWALL
C
  720 DO 810 J=1,74
      HW(J)=J-DEC
      Y(J)=12.*HW(J)/DIA
      X=5.
      DO 814 K=1,100
      G=.114099+.653562*X-.233615*X**2+.0597723*X**3-
     1 .00616338*X**4+.000242832*X**5-Y(J)
      IF (ABS(G).LE.0.0005) GO TO 812
      H=.653562-.46723*X+.1793169*X**2-.02465352*X**3+
     1 .00121416*X**4
      DX=G/H
      X=X-DX
  814 CONTINUE
  812 Q(J)=BNR*X*(DIA/12.)**2.5
      IF (Q(J).LT.0.0) Q(J)=BNR*DIA/12.
      LEV=LAD+J+1
      QM(LEV,IE)=Q(J)
      TRY=AHW-HW(J)-1.
      IF (TRY.LT.0.0) GO TO 1100
  810 CONTINUE
      GO TO 1100
C
C        KTP=6,10 - STANDARD END SECTION
C
  730 DO 820 J=1,74
      HW(J)=J-DEC
      Y(J)=12.*HW(J)/DIA
```

```
      X=5.
      DO 824 K=1,100
      G=.120659+.630768*X-.218423*X**2+.0591815*X**3-
     1 .00599169*X**4+.000229287*X**5-Y(J)
      IF (ABS(G).LE.0.0005) GO TO 822
      H=.630768-.436846*X+.1775445*X**2-.02396676*X**3+
     1 .001146435*X**4
      DX=G/H
      X=X-DX
  824 CONTINUE
  822 Q(J)=BNR*X*(DIA/12.)**2.5
      IF (Q(J).LT.0.0) Q(J)=BNR*DIA/12.
      LEV=LAC+J+1
      QM(LEV,IE)=Q(J)
      TRY=AHW-HW(J)-1.
      IF (TRY.LT.0.0) GO TO 1100
  820 CONTINUE
      GO TO 1100
C
C        KTP=7 - PROJECTING FROM FILL
C
  740 DO 830 J=1,74
      HW(J)=J-DEC
      Y(J)=12.*HW(J)/DIA
      X=5.
      DO 834 K=1,100
      G=.187321+.56771*X-.156544*X**2+.0447052*X**3-
     1 .00343602*X**4+.000089661*X**5-Y(J)
      IF (ABS(G).LE.0.0005) GO TO 832
      H=.56771-.313088*X+.1341156*X**2-.01374408*X**3+
     1 .000448305*X**4
      DX=G/H
      X=X-DX
  834 CONTINUE
  832 Q(J)=BNR*X*(DIA/12.)**2.5
      IF (Q(J).LT.0.0) Q(J)=BNR*DIA/12.
      LEV=LAC+J+1
      QM(LEV,IE)=Q(J)
      TRY=AHW-HW(J)-1.
      IF (TRY.LT.0.0) GO TO 1100
  830 CONTINUE
      GO TO 1100
C
C        KTP=8 - MITERED TO FILL SLOPE
C
  750 DO 850 J=1,74
      HW(J)=J-DEC
      Y(J)=12.*HW(J)/DIA
      X=5.
      DO 854 K=1,100
      G=.107137+.757789*X-.361462*X**2+.1233932*X**3-
     1 .01606422*X**4+.00076739*X**5-Y(J)
      IF (ABS(G).LE.0.0005) GO TO 852
      H=.757789-.722924*X+.3701796*X**2-.06425688*X**3+
     1 .00383695*X**4
      DX=G/H
      X=X-DX
  854 CONTINUE
  852 Q(J)=BNR*X*(DIA/12.)**2.5
      IF (Q(J).LT.0.0) Q(J)=BNR*DIA/12.
```

```
      LEV=LAC+J+1
      QM(LEV,IE)=Q(J)
      TRY=AHW-HW(J)-1.
      IF (TRY.LT.0.0) GO TO 1100
  850 CONTINUE
      GO TO 1100
C
C        KTP=9 - IN A 90 DEGREE HEADWALL
C
  760 DO 870 J=1,74
      HW(J)=J-DEC
      Y(J)=12.*HW(J)/DIA
      X=5.
      DO 874 K=1,100
      G=.167433+.538595*X-.149374*X**2+.0391543*X**3-
     1 .00343974*X**4+.000115882*X**5-Y(J)
      IF (ABS(G).LE.0.0005) GO TO 872
      H=.538595-.298748*X+.1174629*X**2-.01375896*X**3+
     1 .00057941*X**4
      DX=G/H
      X=X-DX
  874 CONTINUE
  872 Q(J)=BNR*X*(DIA/12.)**2.5
      IF (Q(J).LT.0.0) Q(J)=BNR*DIA/12.
      LEV=LAC+J+1
      QM(LEV,IE)=Q(J)
      TRY=AHW-HW(J)-1.
      IF (TRY.LT.0.0) GO TO 1100
  870 CONTINUE
      GO TO 1100
C
C        KTP=11 - DROP INLET WEIR
C
 1001 DO 1020 J=1,74
      HW(J)=J-DEC
      Q(J)=3.4*WL*HW(J)**1.5
      LEV=LAC+J+1
      QM(LEV,IE)=Q(J)
C
C         COMPARE Q FOR DROP INLET WITH Q FOR CULVERT AT
C         BOTTOM OF DROP INLET - USE SMALLER Q
C
      DIF=QM(LEV,1)-QM(LEV,2)
      IF (DIF.LT.0.0) GO TO 1012
      QM(LEV,6)=QM(LEV,2)
      GO TO 1015
 1012 QM(LEV,6)=QM(LEV,1)
 1015 TRY=AHW-HW(J)-1.
      IF (TRY.LT.0.0) GO TO 1100
 1020 CONTINUE
      GO TO 1100
C
C        KTP=12 - WEIR, ROADWAY OVERTOPPED
C                         OR SIDE DITCH
C
 1002 DO 1040 J=1,74
      MTT=HWE-LFL
      IF (MTT.LT.0) GO TO 1100
      HW(J)=J-DEC
      Q(J)=3.0*WL*HW(J)**1.5
```

```
            LEV=LAD+J+1
            QM(LEV,IE)=Q(J)
            TRY=AHW-HW(J)-1.
            IF (TRY.LT.0.0) GO TO 1100
       1040 CONTINUE
            GO TO 1100
       1095 ITT=2
       1100 CONTINUE
      C
      C          QM(J,6) IS MADE EQUAL TO QM(J,1)
      C          WHEN NO DROP INLET IS USED
      C
            GO TO (1210,1200),ITT
       1200 DO 1205 J=1,74
            HW(J)=J-DC1
            QM(J,6)=QM(J,1)
            TRY=AH1-HW(J)
            IF (TRY.LT.0.0) GO TO 1210
       1205 CONTINUE
       1210 DO 1215 J=2,75
            HW(J)=J-DC1-1
            EOU(J)=LF1+J-1
            QOU(J)=QM(J,3)+QM(J,4)+QM(J,5)+QM(J,6)
            NOL=J
            TRY=AH1-HW(J)-1.
            IF (TRY.LT.0.0) GO TO 1250
       1215 CONTINUE
      C
      C          OUTPUTS DISCHARGES FOR EACH OUTLET TYPE
      C          AND TOTAL STAGE-OUTFLOW CURVE
      C
       1250 WRITE (6,903) (CC(I,M),M=1,4),A
        903 FORMAT ('1'//20X,4A4,'  COUNTY',6X,'D.A. =',F6.0,
           1 ' ACRES')
            WRITE (6,294) (ID1(J),J=1,17)
        294 FORMAT ('0',19X,17A4)
            WRITE (6,992) (ID2(J),J=1,17)
            WRITE (6,992) (ID3(J),J=1,17)
            WRITE (6,992) (ID4(J),J=1,17)
        992 FORMAT (' ',19X,17A4)
            WRITE (6,993)
        993 FORMAT ('0'/21X,'ELEV',6X,'Q1',6X,'Q2  Q1 OR Q2',
           1 '    Q3      Q4      Q5   TOTAL Q'/)
            EOU(1)=FL(1)
            WRITE (6,994) EOU(1),QM(1,1),QM(1,2),QM(1,6),QM(1,3),
           1 QM(1,4),QM(1,5),QOU(1)
        994 FORMAT (F26.1,3F8.0,3F7.0,F8.0)
            DO 1150 J=2,75
            WRITE (6,994) EOU(J),QM(J,1),QM(J,2),QM(J,6),QM(J,3),
           1 QM(J,4),QM(J,5),QOU(J)
            HW(J)=J-DC1-1
            TRY=AH1-HW(J)-1.
            IF (TRY.LT.0.0) GO TO 22
       1150 CONTINUE
         22 RETURN
            END
      C
      C

            SUBROUTINE RTG
```

```
C
C
C          THIS PORTION PERFORMS THE ROUTING OF THE
C          INFLOW HYDROGRAPH THROUGH THE TEMPORARY POND
C          AND THE CULVERT(S)
C
C
      COMMON I,CO(99,14),A,ID1(17),ID2(17),ID3(17),ID4(17),
     1 NCG
      COMMON /RR3/ NSL,EST(75),STR(75),BGL,ENL
      COMMON /RR4/ DRN(7),DLT(7),PRP(7),QI(75,7),TI(75,7),
     1 SR(7),NIT(7),BGT,ENT
      COMMON /RR5/ EOU(75),QOU(75),ELO(75),QO(75),HWL,NOL
      DIMENSION ELS(75),ST(75),SCV(75),TG(300),QG(300),
     1 OG(300),S2G(300),E2G(300),QIX(7),QOX(7),TOX(7),
     2 SOX(7),EOX(7)
C
C
C    BLOCK 10 -- DETERMINES 'CON' FOR THE UNITS OF HOURS,
C                'NOE', 'NSE', AND COMPUTES THE 'WORKING
C                CURVE' ARRAY
C
C
C          CON = CONSTANT
C          NOE = NUMBER OF OUTFLOW ELEVATIONS
C          NSE = NUMBER OF STORAGE ELEVATIONS
C
      DO 60 L=1,7
      LBG=BGL
      LHW=HWL
      DCL=(LHW-LBG)-(HWL-BGL)
      IF (DCL.GT.0.0) GO TO 17
      NOE=HWL-BGL+1.
      GO TO 18
   17 NOE=HWL-BGL+2.
   18 NBO=NOL-NOE+1
      DO 19 J=1,NSL
      NSE=NSL-J+1
      NBS=J
      IF (BGL-EST(J)) 21,23,19
   21 NSE=NSE+1
      NBS=J-1
      GO TO 23
   19 CONTINUE
   23 CON=24.2424/DLT(L)
      STM=VL1(BGL,NSL,EST,STR)
      STR(NBS)=STM
      EST(NBS)=BGL
      K=0
      DO 24 J=NBS,NSL
      K=K+1
      ELS(K)=EST(J)
      ST(K)=STR(J)-STM
   24 CONTINUE
      K=0
      DO 26 J=NBO,NOL
      K=K+1
      ELO(K)=EOU(J)
      QO(K)=QOU(J)
   26 CONTINUE
```

```
      K=0
      DO 28 J=1,NOE
      EL=ELC(J)
      K=K+1
      SCV(K)=CCN*VL1(EL,NSE,ELS,ST)+VL1(EL,NOE,ELO,QC)
   28 CONTINUE
C
C
C   BLOCK 11 -- COMPUTES AND OUTPUTS INITIAL ROUTING VALUES
C
C
C
C             T1 = TIME AT BEGINNING OF INITIAL PERIOD
C            EL1 = ELEVATION AT BEGINNING OF INITIAL PERIOD
C             Q1 = INFLOW AT BEGINNING OF INITIAL PERIOD
C             S1 = STORAGE AT BEGINNING OF INITIAL PERIOD
C             O1 = OUTFLOW AT BEGINNING OF INITIAL PERIOD
C            TRL = LEFT-HAND TERM OF ROUTING EQUATION
C
      IPG=1
      T1=BGT
      EL1=BGL
      Q1=VL3(T1,NIT(L),TI,QI,L)
      S1=VL1(EL1,NSE,ELS,ST)
      O1=VL1(EL1,NOE,ELO,QO)
      TRL=CCN*S1-O1
      WRITE (6,293) IPG
  293 FORMAT ('1'////30X,'FLOOD ROUTING OUTPUT',
     1 ' DATA---PAGE',I3)
      WRITE (6,904) (CC(I,M),M=1,4),A
  904 FORMAT ('0',19X,4A4,'  COUNTY',6X,'D.A. =',F6.0,
     1 ' ACRES')
      WRITE (6,294) (ID1(J),J=1,17)
  294 FORMAT ('0',19X,17A4)
      WRITE (6,992) (ID2(J),J=1,17)
      WRITE (6,992) (ID3(J),J=1,17)
      WRITE (6,992) (ID4(J),J=1,17)
  992 FORMAT (' ',19X,17A4)
      WRITE (6,296) DRN(L),PRP(L),SR(L)
  296 FORMAT ('0',19X,'DUR =',F6.2,' HR.',4X,'RAIN =',
     1 F6.2,' IN.',4X,'RUNOFF =',F6.2,' IN.')
      WRITE (6,295)
  295 FORMAT ('0',23X,'TIME    INFLOW    OUTFLOW   ',
     1 'STORAGE  ELEVATION'/23X,'HOURS',6X,'CFS',7X,
     2 'CFS',6X,'AC-FT',7X,'FT'/)
      WRITE (6,292) T1,Q1,O1,S1,EL1
  292 FORMAT (F28.2,F9.0,F11.0,F10.1,F11.2)
      NOO=1
      TG(1)=T1
      QG(1)=Q1
      OG(1)=O1
      S2G(1)=S1
      E2G(1)=EL1
      LIN=19
C
C
C   BLOCK 12 -- COMPUTES AND OUTPUTS VALUES FOR THE
C               END OF EACH TIME PERIOD AND CHECKS
C               FOR END OF ROUTING
C
```

```
C
C
C              T2 = TIME AT END OF PERIOD
C              Q2 = INFLCW AT END OF PERIOD
C              O2 = OUTFLOW AT END OF PERIOD
C             EL2 = ELEVATION AT END OF PERIOD
C              S2 = STORAGE AT END OF PERIOD
C             TRR = RIGHT-HAND TERM OF ROUTING EQUATION
C
   42 T2=T1+DLT(L)
      Q2=VL3(T2,NIT(L),TI,QI,L)
      TRR=Q1+Q2+TRL
      O2=VL1(TRR,NOE,SCV,QO)
      EL2=VL1(TRR,NOE,SCV,ELO)
      S2=VL1(EL2,NSE,ELS,ST)
      WRITE (6,292) T2,Q2,O2,S2,EL2
      NOO=NOO+1
      TG(NOO)=T2
      QG(NOO)=Q2
      OG(NOO)=O2
      S2G(NOO)=S2
      E2G(NOO)=EL2
      IF (T2-ENT) 44,43,43
   43 IF (EL2-ENL) 47,47,44
C
C
C     BLOCK 13 -- INITIALIZES TRL,T1, AND Q1 FOR
C                 THE NEXT TIME PERIOD
C
C
   44 LIN=LIN+1
      IF (LIN-56) 46,45,45
   45 IPG=IPG+1
      WRITE (6,293) IPG
      WRITE (6,904) (CC(I,M),M=1,4),A
      WRITE (6,294) (ID1(J),J=1,17)
      WRITE (6,992) (ID2(J),J=1,17)
      WRITE (6,992) (ID3(J),J=1,17)
      WRITE (6,992) (ID4(J),J=1,17)
      WRITE (6,296) DRN(L),PRP(L),SR(L)
      WRITE (6,295)
      LIN=18
   46 TRL=TRR-2.*O2
      T1=T2
      Q1=Q2
      GO TO 42
   47 VOL=0.
      NTD=NOO-1
      JJ=0
      DO 48 J=1,NTD
      JJ=J+1
      VOL=VOL+(QG(J)+QG(JJ))*DLT(L)/2.
   48 CONTINUE
      WRITE (6,290) VOL
  290 FORMAT ('0',29X,'RUNOFF VOLUME =',F10.0,' CFS-HOURS')
      IF (L.EQ.1) JX=1
      DO 51 J=JX,NTD
      JY=J
      IF (QG(J).GT.QG(J+1)) GO TO 52
   51 CONTINUE
```

```
   52 QIX(L)=QG(J)
      JX=J
      DO 53 J=JY,NTD
      IF (QG(J).GT.QG(J+1)) GO TO 54
   53 CONTINUE
   54 QOX(L)=QG(J)
      TOX(L)=TG(J)
      SOX(L)=S2G(J)
      EOX(L)=E2G(J)
   60 CONTINUE
      WRITE (6,300)
  300 FORMAT ('1'////38X,'SUMMARY OF FLOOD ROUTINGS')
      WRITE (6,904) (CC(I,M),M=1,4),A
      WRITE (6,994) (ID1(J),J=1,17)
      WRITE (6,992) (ID2(J),J=1,17)
      WRITE (6,992) (ID3(J),J=1,17)
      WRITE (6,992) (ID4(J),J=1,17)
      WRITE (6,301)
  301 FORMAT ('0'//32X,'INFLOW     STORM     TOTAL     ',
     1 'TOTAL'/30X,'HYDROGRAPH DURATION  RAINFALL   ',
     2 'RUNOFF'/32X,'NUMBER',6X,'HR',8X,'IN',8X,'IN'/)
      DO 310 J=1,7
      WRITE (6,302) J,DRN(J),PRP(J),SR(J)
  302 FORMAT ('0',I35,F12.2,F9.2,F10.2)
  310 CONTINUE
      WRITE (6,303)
  303 FORMAT ('0'//22X,'INFLOW',6X,'MAX',7X,'MAX     ',
     1 'TIME MAX      MAX    MAX  ELEV'/20X,'HYDROGRAPH  ',
     2 'INFLOW   OUTFLOW   OUTFLOW   STORAGE  HEADWATER'/
     3 22X,'NUMBER',6X,'CFS',7X,'CFS',7X,'HR',7X,'AC-FT',
     4 6X,'FT'/)
      DO 320 J=1,7
      WRITE (6,304) J,QIX(J),QOX(J),TOX(J),SOX(J),EOX(J)
  304 FORMAT ('0',I25,F11.0,F11.0,F10.2,F10.1,F11.2)
  320 CONTINUE
      RETURN
      END

      FUNCTION VL1 (X,NPT,AX,AY)
C
C
C   INTERPOLATION ROUTINE FOR CASES WHERE BOTH THE
C   KNOWN VARIABLE AND THE UNKNOWN VARIABLE ARE IN A
C   ONE-DIMENSIONAL ARRAY.
C
C
      DIMENSION AX(75),AY(75)
      DO 1 K=2,NPT
      IF (X-AX(K)) 2,2,1
    1 CONTINUE
    2 J=K-1
C
C         IF ELEVATION CORRESPONDING TO UNKNOWN VALUE OF
C         OUTFLOW Q WILL BE HIGHER THAN HEADWATER ELEVATION,
C         EXECUTION CEASES AND A MESSAGE IS OUTPUT
C
      IF (J.EQ.NPT) GO TO 1320
      GO TO 1340
 1320 WRITE (6,1325)
 1325 FORMAT ('1',9X,'THE VALUE OF OUTFLOW Q REQUIRES'/10X,
```

```
     1 'A HEAD GREATER THAN THE HEADWATER ELEVATION'/10X,
     2 'LISTED IN THE HYDRAULIC INPUT DATA')
      WRITE (6,1330)
 1330 FORMAT ('0',9X,'THIS MIGHT MEAN THAT THE CULVERT'/10X,
     1 'SIZE BEING INVESTIGATED IS TOO SMALL FOR THE'/10X,
     2 'STORAGE AVAILABLE AT THE SITE')
      WRITE (6,1335)
 1335 FORMAT ('0',9X,'IF YOU WISH TO CONTINUE'/10X,
     1 'INVESTIGATING THIS CULVERT SIZE, INCREASE THE'/10X,
     2 'HEADWATER ELEVATION LISTED IN THE HYDRAULIC'/10X,
     3 'INPUT DATA.  ALSO MAKE SURE THAT THE HIGHEST'/10X,
     4 'STORAGE ELEVATION IS AT LEAST EQUAL TO THE'/10X,
     5 'NEW HEADWATER ELEVATION.')
      STOP
 1340 VL1=AY(J)+(AY(K)-AY(J))*(X-AX(J))/(AX(K)-AX(J))
      RETURN
      END

      FUNCTION VL3 (X,NPT,AX,AY,NZ)
C
C
C     INTERPOLATION ROUTINE FOR CASES WHERE BOTH THE
C     KNOWN VARIABLE AND THE UNKNOWN VARIABLE ARE IN A
C     TWO-DIMENSIONAL ARRAY.
C
C
      DIMENSION AX(75,7),AY(75,7)
      DO 1 K=2,NPT
      IF(X-AX(K,NZ)) 2,2,1
    1 CONTINUE
    2 J=K-1
      VL3=AY(J,NZ)+(AY(K,NZ)-AY(J,NZ))*(X-AX(J,NZ))/
     1 (AX(K,NZ)-AX(J,NZ))
      RETURN
      END
```

APPENDIX E.

LOCATION OF GAGING STATIONS AND HIGHWAY PROJECTS USED IN THE STUDY

Table E-1. Number, size, and location of USGS gaging stations used in the study

Station	Size ac.	Section	Township	Range	County	Location
5-3884	7,620	25	98 N	3 W	Allamakee	Wexford Ck. nr. Harpers Ferry
5-3887	700	1	97 N	4 W	Allamakee	L. Paint Ck. trib. nr. Waterville
5-4116.5	2,600	21	99 N	13 W	Howard	Crane Ck. trib. nr. Saratoga
5-4144.5	14,590	28	90 N	1 E	Dubuque	N. Fk. L. Maquoketa R. nr. Richardsville
5-4146	970	11	89 N	2 E	Dubuque	L. Maquoketa R. trib. at Dubuque
5-4206	580	27	99 N	14 W	Howard	L. Wapsi. R. trib. nr. Riceville
5-4206.2	4,970	10	98 N	14 W	Howard	L. Wapsi. R. nr. Acme
5-4211	210	27	89 N	8 W	Buchanan	Pine Ck. trib. nr. Winthrop
5-4213	450	2	88 N	8 W	Buchanan	Pine Ck. trib. No. 2 nr. Winthrop
5-4486	1,430	4	96 N	24 W	Hancock	E. Br. Iowa R. above Hayfield
5-4487	5,080	35	97 N	24 W	Hancock	E. Br. Iowa R. nr. Hayfield
5-4536	5,020	21	80 N	5 W	Johnson	Rapid Ck. below Morse
5-4537	990	22	80 N	5 W	Johnson	Rapid Ck. trib. No. 4 nr. Oasis
5-4537.5	9,470	21	80 N	5 W	Johnson	Rapid Ck. SW of Morse

Table E-1. Continued

Station	Size ac.	Section	Township	Range	County	Location
5-4539	600	33	80 N	5 W	Johnson	Rapid Ck. trib. nr. Oasis
5-4539.5	2,160	31	80 N	5 W	Johnson	Rapid Ck. trib. nr. Iowa City
5-4540	15,740	36	80 N	6 W	Johnson	Rapid Ck. nr. Iowa City
5-4550	1,930	11	79 N	6 W	Johnson	Ralston Ck. at Iowa City
5-4550.1	2,050	14	79 N	6 W	Johnson	S. Br. Ralston Ck. at Iowa City
5-4552.8	1,610	21	78 N	14 W	Poweshiek	S. Fk. English R. trib. nr. Barnes City
5-4553	7,360	34	78 N	14 W	Poweshiek	S. Fk. English R. nr. Barnes City
5-4553.5	330	11	78 N	14 W	Poweshiek	S. Fk. English R. trib. No. 2 nr. Montezuma
5-4641.33	850	33	86 N	15 W	Tama	Half Mile Ck. nr. Gladbrook
5-4830	15,360	5	84 N	30 W	Greene	E. Fk. Hardin Ck. nr. Churdan
5-4956	3,000	5	68 N	14 W	Davis	S. Wyaconda R. nr. West Grove
6-4834.4	2,780	20	99 N	41 W	Osceola	Dawson Ck. nr. Sibley
6-4834.5	4,540	35	99 N	42 W	Osceola	Wagner Ck. nr. Ashton
6-5998	5,040	12	91 N	47 W	Plymouth	Perry Ck. nr. Merrill

Table E-1. Continued

Station	Size ac.	Section	Township	Range	County	Location
6-6105	5,110	18	75 N	43 W	Pottawattamie	Indian Creek at Council Bluffs
6-8077.6	16,450	4	75 N	41 W	Pottawattamie	Middle Silver Ck. nr. Oakland
6-8080	6,780	19	71 N	41 W	Mills	Mule Ck. nr. Malvern
6-8082	4,900	31	71 N	41 W	Fremont	Spring Valley Ck. nr. Tabor
6-8090	16,640	9	79 N	34 W	Audubon	Davids Ck. nr. Hamlin
6-8118	2,980	34	73 N	37 W	Montgomery	E. Tarkio Ck. nr. Stanton
6-8118.2	430	16	72 N	37 W	Montgomery	Tarkio R. trib. nr. Stanton

Table E-2. County, size, and location of sites on county and state highway projects used in the study

County	Size ac.	Section	Township	Range	Station	Project
Black Hawk	7,425	7	78 N	13 W	—	FU-UG-520-6(9)--44-07
Johnson	38	15	81 N	7 W	1629 + 70	I-IG-380-6(7)247--04-52
Johnson	132	9	81 N	7 W	1702 + 75	I-IG-380-6(7)247--04-52
Johnson	151	3	80 N	7 W	1406 + 50	I=IG-380-6(7)247--04-52
Johnson	223	16	81 N	7 W	1664 + 22	I-IG-380-6(7)247--04-52
Johnson	1,406	9	81 N	7 W	1716 + 90	I-IG-380-6(7)247--04-52
Marion	2,625	14	75 N	19 W	1123 + 00	F-92-6(5)--20-63
Pottawattamie	36	33	77 N	43 W	—	County Road G-20
Pottawattamie	83	35	76 N	43 W	—	County Road G-36
Pottawattamie	215	10	76 N	41 W	—	County Road, 1 mi west of L-66
Pottawattamie	265	27	74 N	42 W	—	County Road off G-66
Pottawattamie	315	9	76 N	42 W	—	County Road off L-52
Pottawattamie	315	27	75 N	39 W	—	County Road M-37

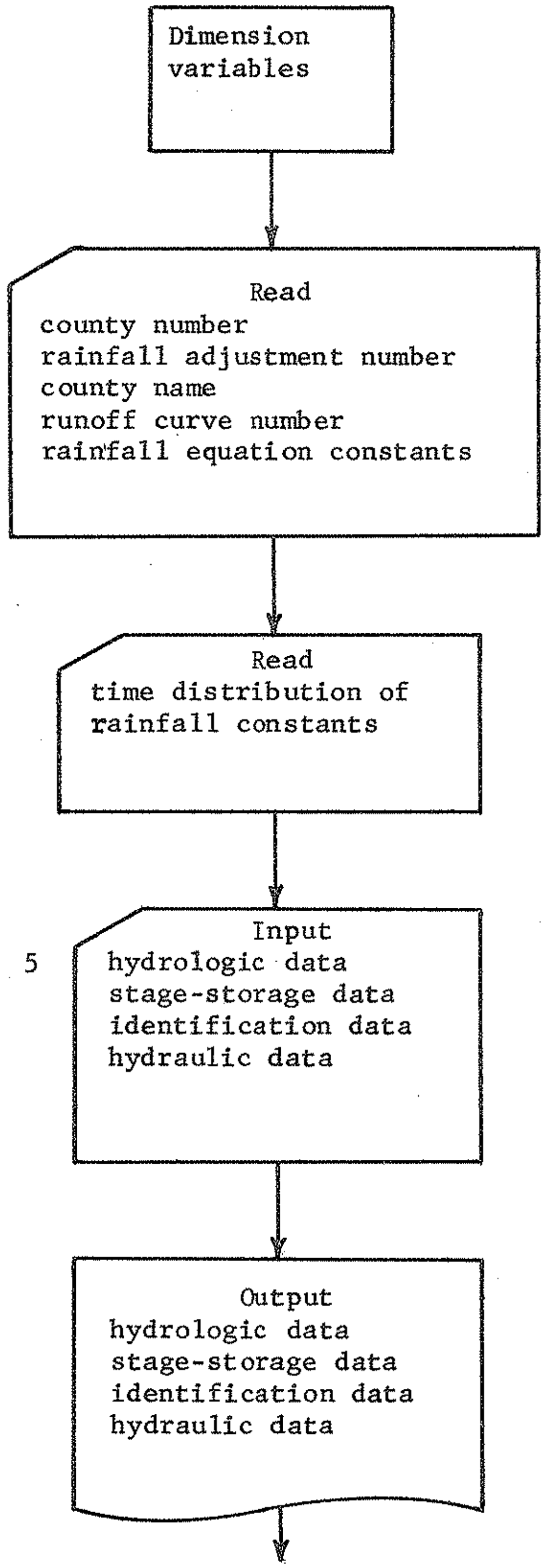
Table E-2. Continued

County	Size ac.	Section	Township	Range	Station	Project
Pottawattamie	325	27	75 N	39 W	—	County Road M-37
Pottawattamie	330	34	75 N	42 W	—	County Road L-52
Pottawattamie	465	32	76 N	41 W	—	County Road L-55
Pottawattamie	765	29	77 N	42 W	—	County Road G-20
Pottawattamie	960	1	75 N	39 W	—	County Road, 2 mi east of M-37
Scott	40	8	78 N	4 E	2272 + 00	I-74-1(6)3--01-82
Scott	43	17	78 N	4 E	2242 + 21	I-74-1(6)3--01-82
Scott	92	25	78 N	2 E	343 + 40	I-IG-280-8(38)294--04-82
Scott	160	8	78 N	4 E	2264 + 41	I-74-1(6)3--01-82
Scott	173	8	78 N	4 E	2300 + 40	I-74-1(6)3--01-82
Webster	1,800	10	88 N	28 W	859 + 40	F-520-3(5)--20-94
Webster	15,000	10	88 N	28 W	853 + 66	F-520-3(5)--20-94
Woodbury	40	7	88 N	47 W	480 + 00	I-IG-129-6(3)145--04-97
Woodbury	340	7	88 N	47 W	477 + 60	I-IG-129-6(3)145--04-97

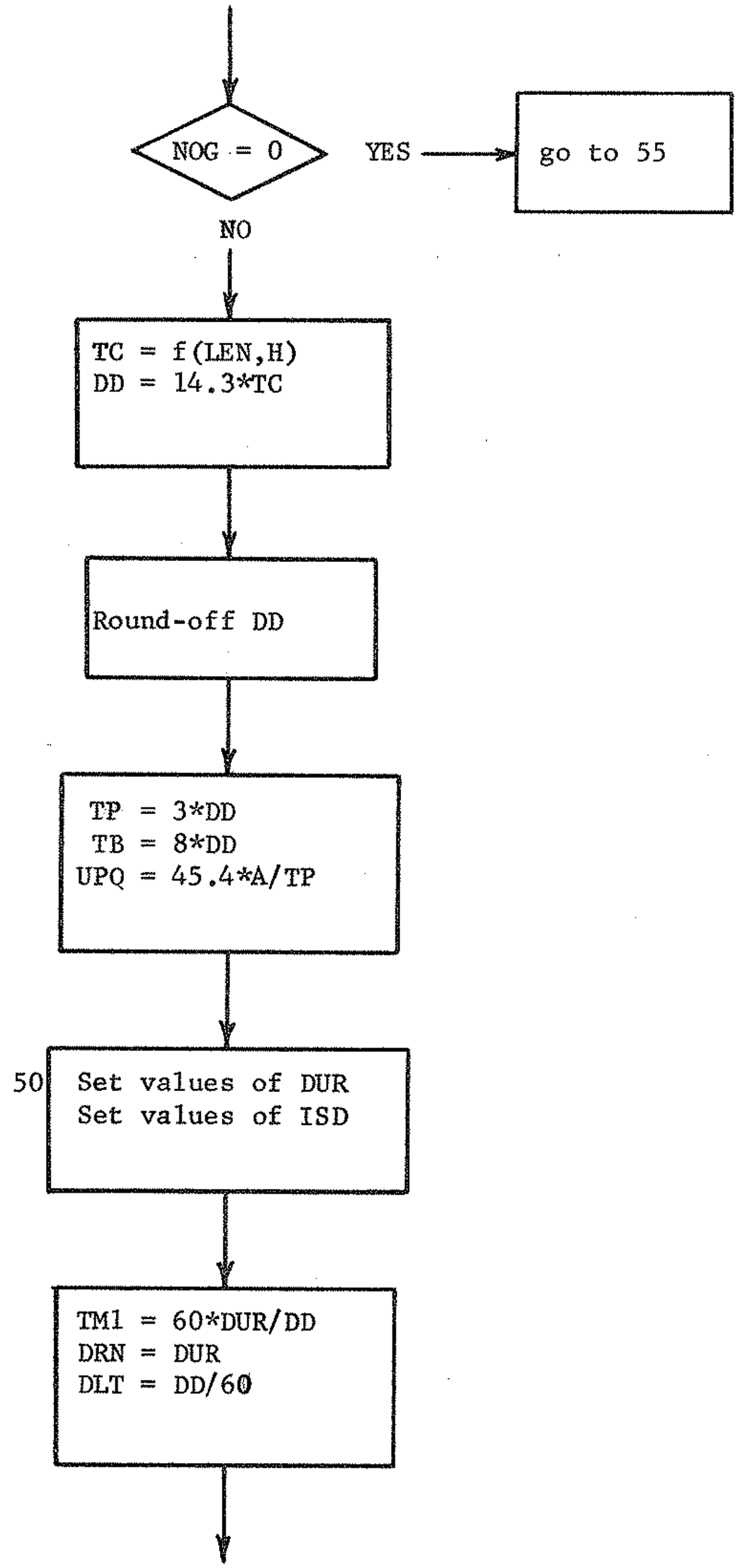
APPENDIX F.

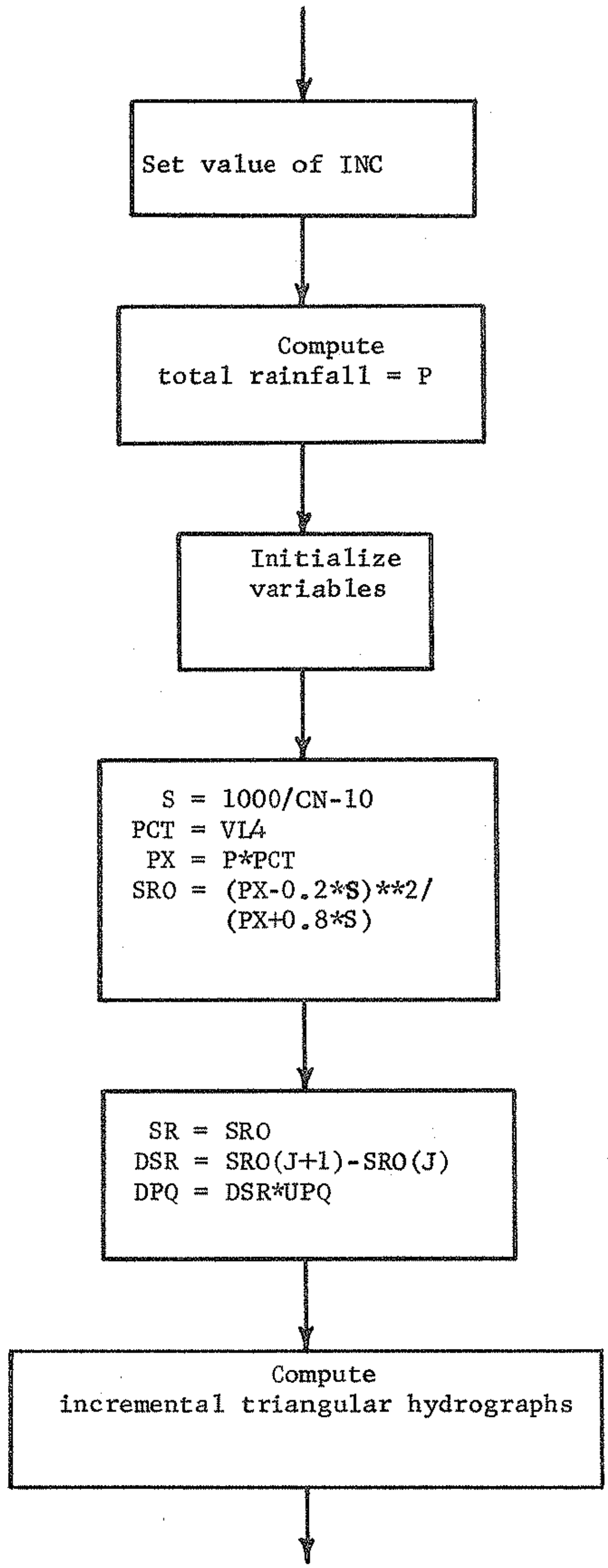
SIMPLIFIED FLOW CHART

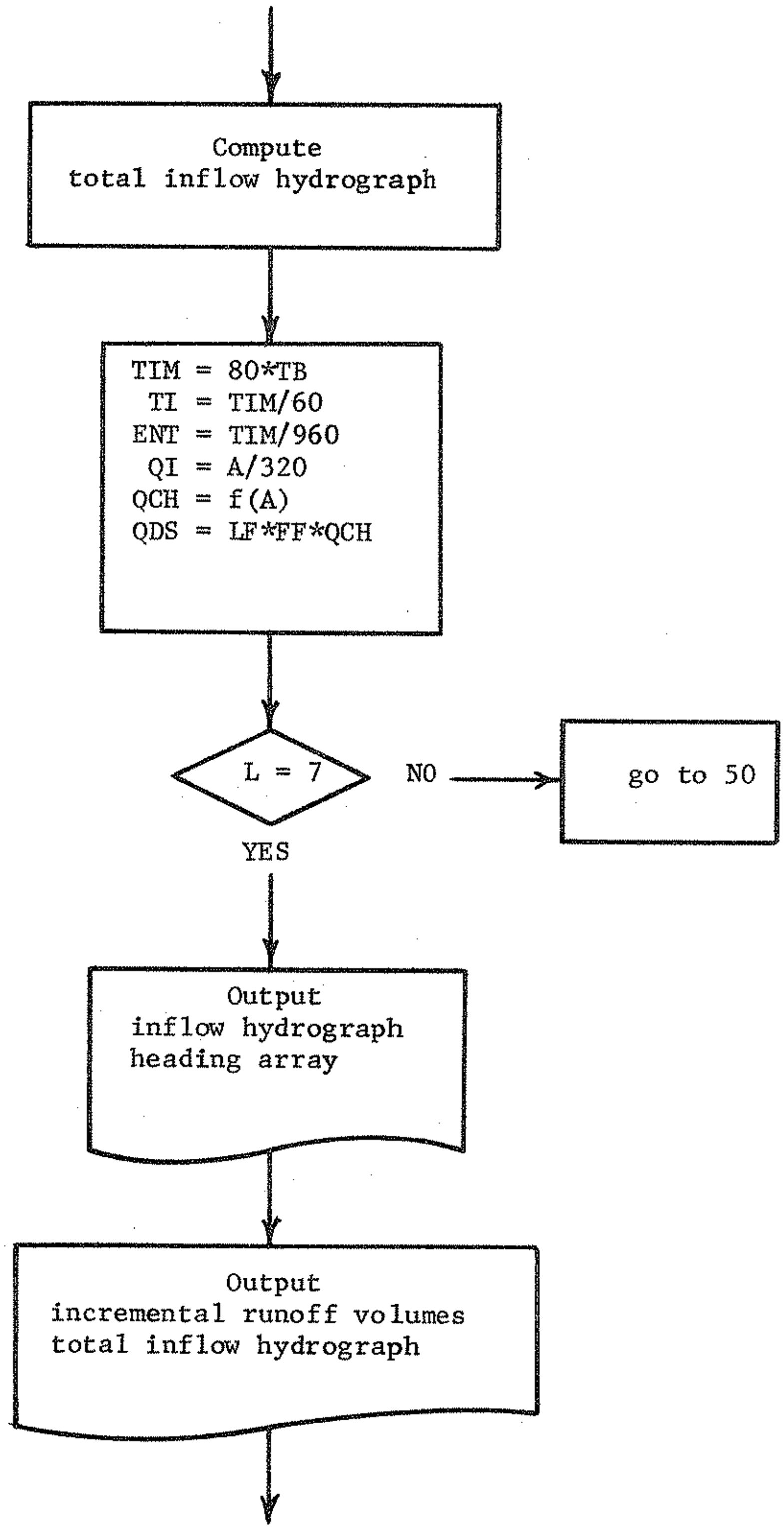
INPUT DATA


Dimension
variables
Read
county number
rainfall adjustment number
county name
runoff curve number
rainfall equation constants
Read
time distribution of
rainfall constants
5
Input
hydrologic data
stage-storage data
identification data
hydraulic data
Output
hydrologic data
stage-storage data
identification data
hydraulic data

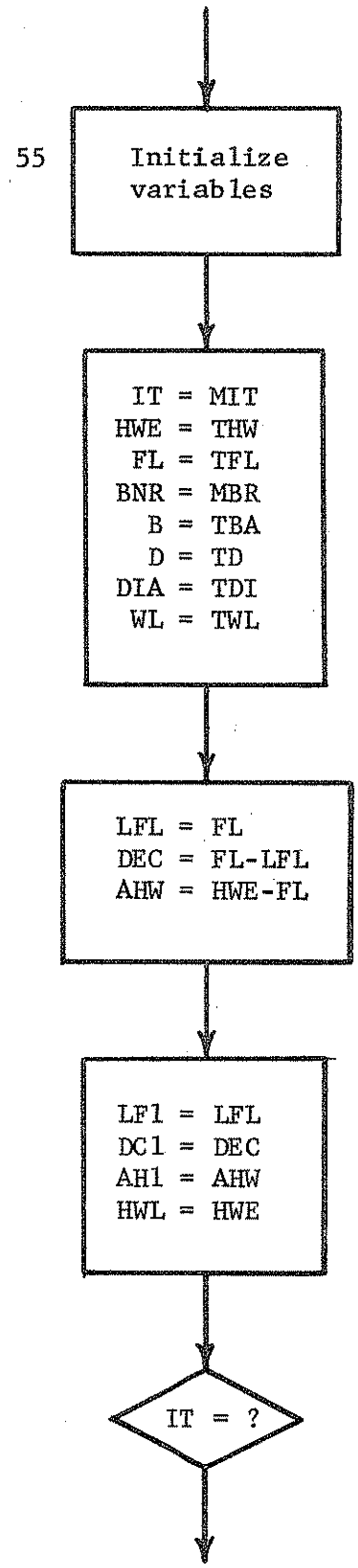
INFLOW HYDROGRAPH

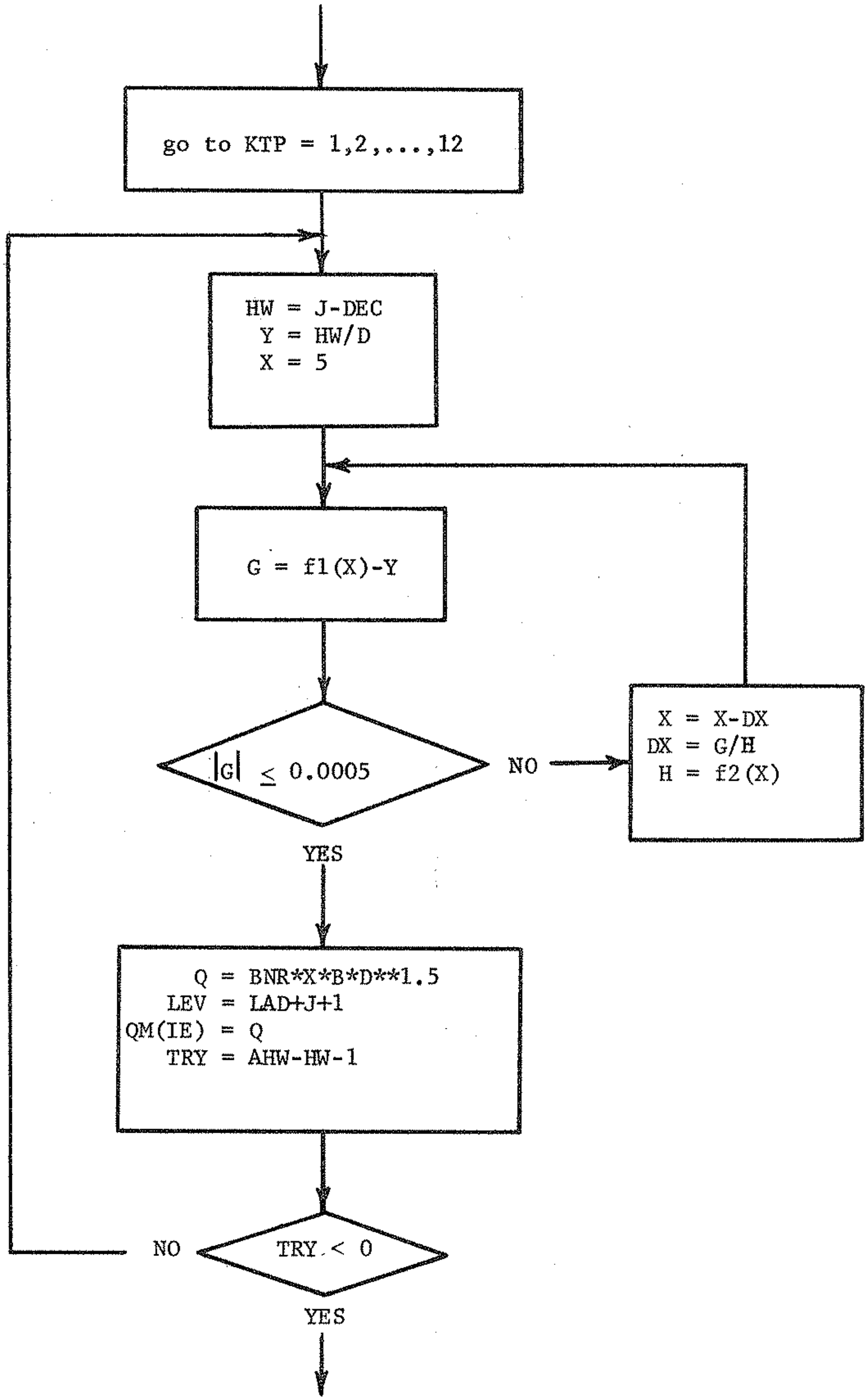



Set value of INC
Compute
total rainfall = P
Initialize
variables
S = 1000/CN-10
PCT = VL4
PX = P*PCT
SRO = (PX-0.2*S)**2/
(PX+0.8*S)
SR = SRO
DSR = SRO(J+1)-SRO(J)
DPQ = DSR*UPQ
Compute
incremental triangular hydrographs

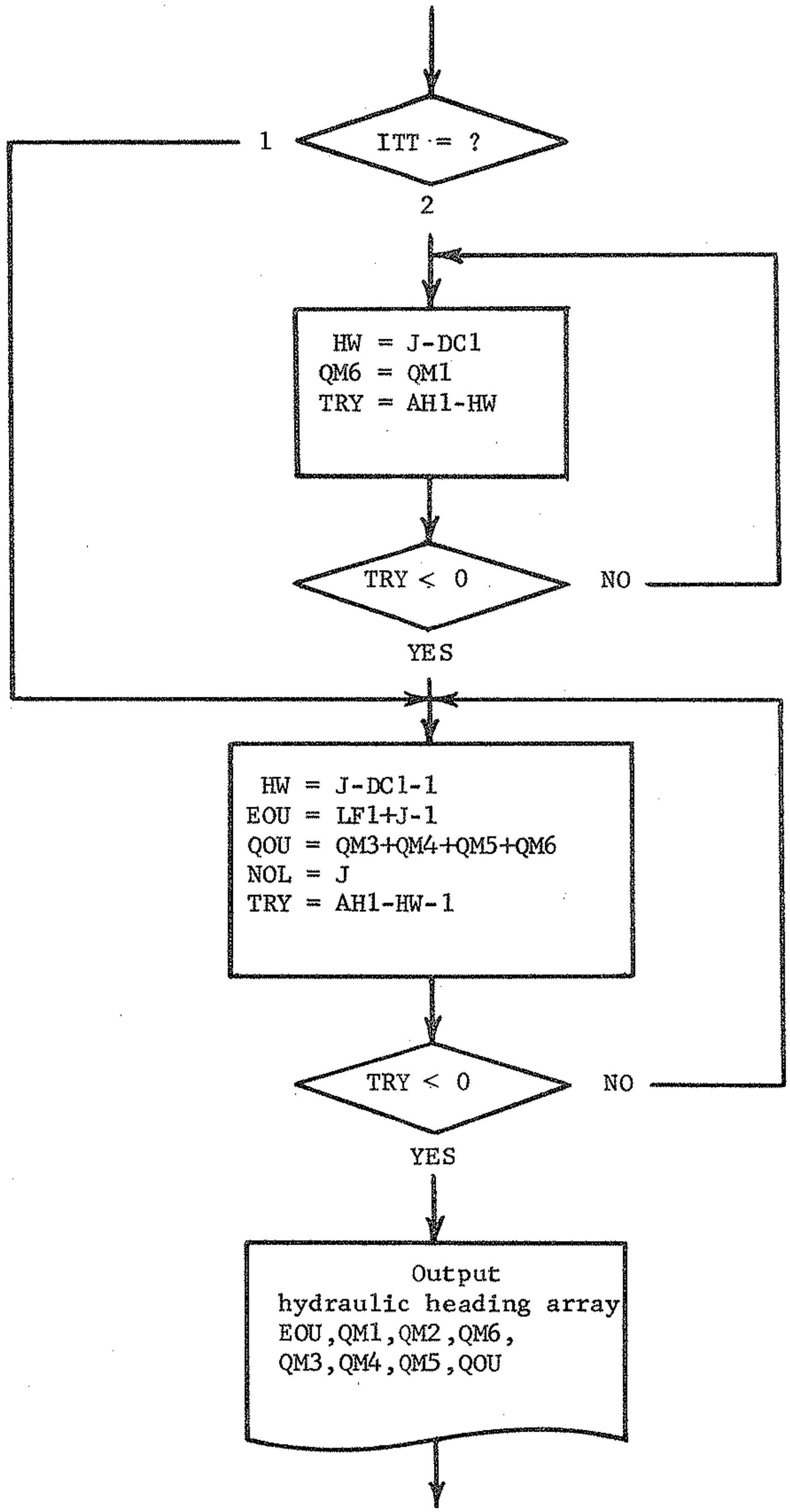

Compute
total inflow hydrograph
TIM = 80*TB
TI = TIM/60
ENT = TIM/960
QI = A/320
QCH = f(A)
QDS = LF*FF*QCH
L = 7
NO
go to 50
YES
Output
inflow hydrograph
heading array
Output
incremental runoff volumes
total inflow hydrograph

HYDRAULIC CALCULATIONS

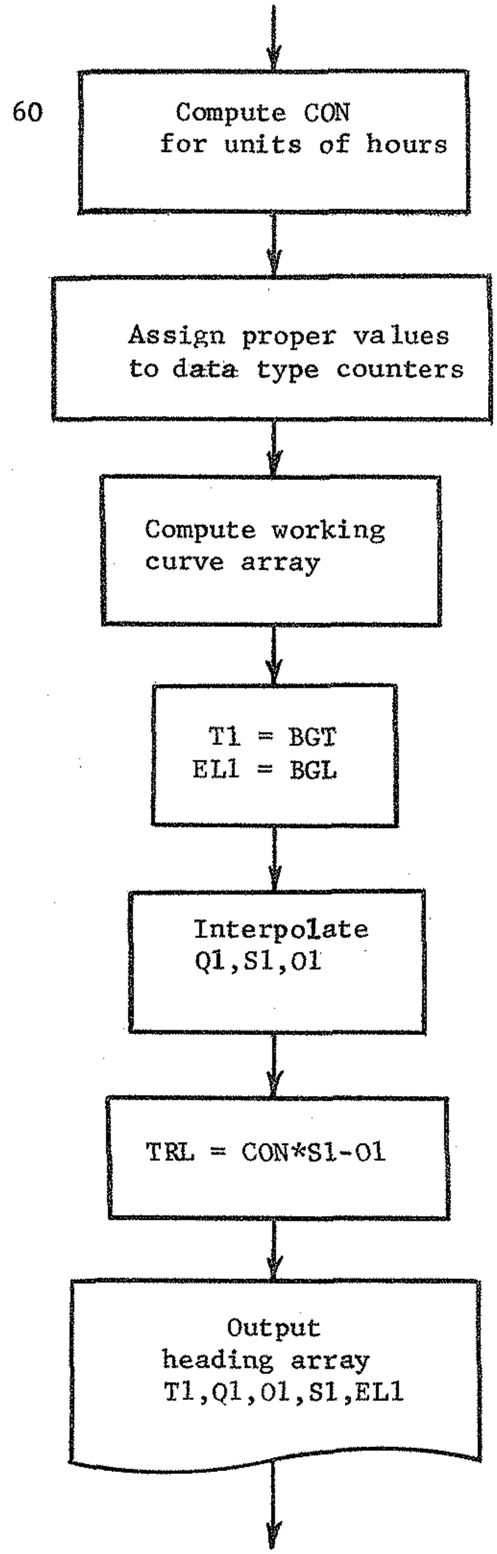


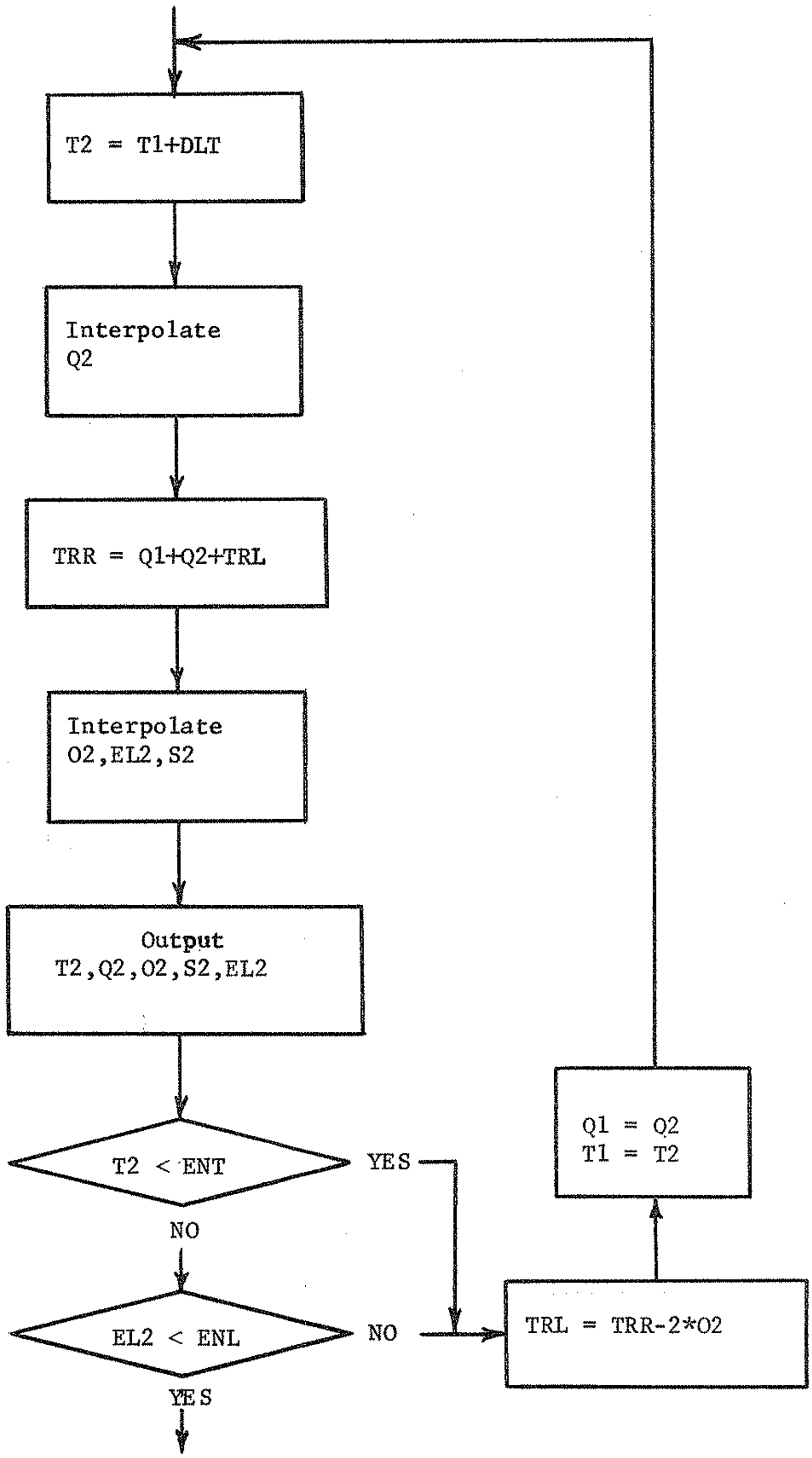


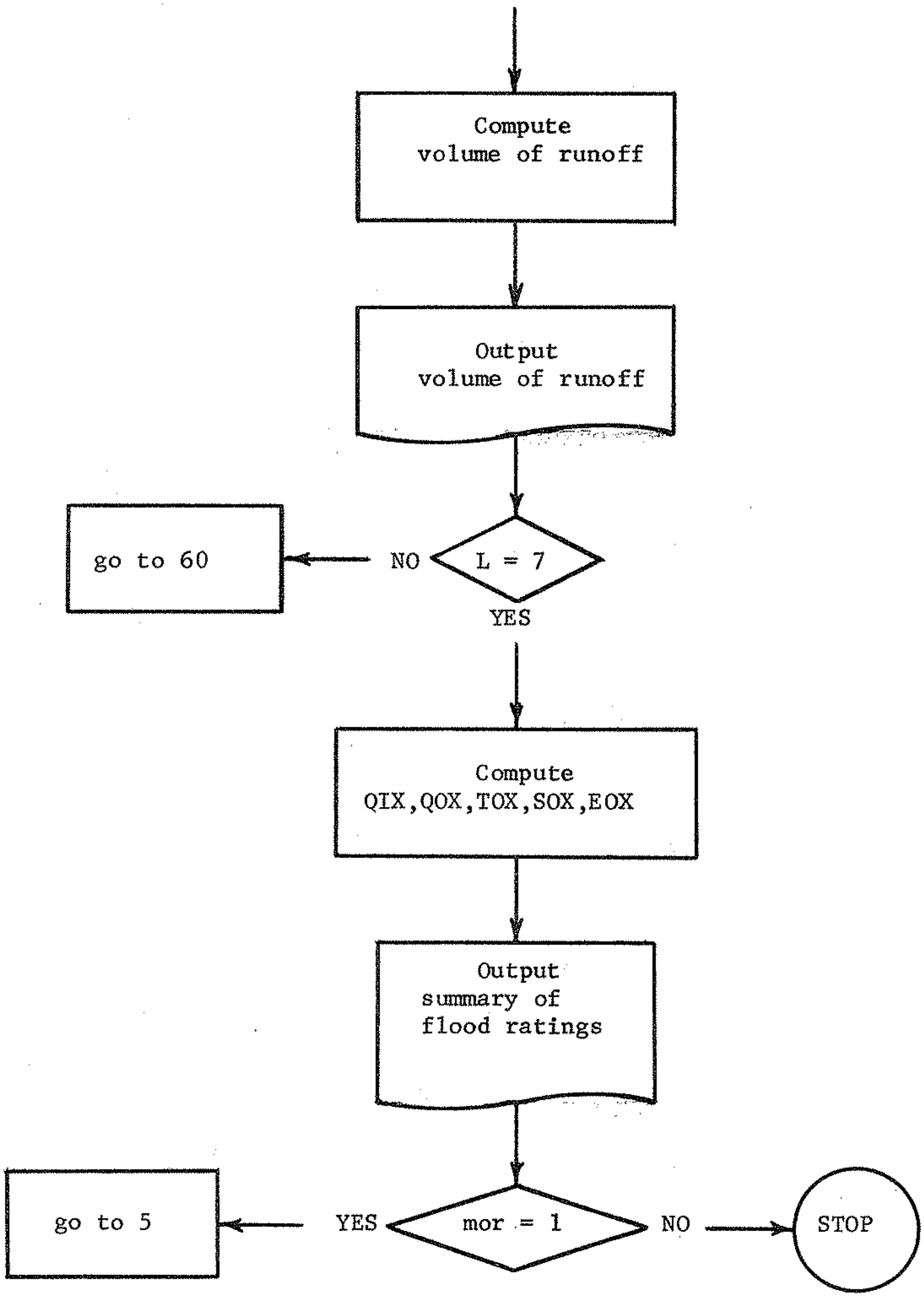
go to KTP = 1,2,...,12
HW = J-DEC
Y = HW/D
X = 5
G = f1(X)-Y
|G| ≤ 0.0005
NO
X = X-DX
DX = G/H
H = f2(X)
YES
Q = BNR*X*B*D**1.5
LEV = LAD+J+1
QM(IE) = Q
TRY = AHW-HW-1
NO
TRY < 0
YES


ITT = ?
1
2
HW = J-DC1
QM6 = QM1
TRY = AH1-HW
TRY < 0
NO
YES
HW = J-DC1-1
EOU = LF1+J-1
QOU = QM3+QM4+QM5+QM6
NOL = J
TRY = AH1-HW-1
TRY < 0
NO
YES
Output
hydraulic heading array
EOU,QM1,QM2,QM6,
QM3,QM4,QM5,QOU

HYDROGRAPH ROUTING




T2 = T1+DLT
Interpolate Q2
TRR = Q1+Q2+TRL
Interpolate O2,EL2,S2
Output T2,Q2,O2,S2,EL2
T2 < ENT
YES
NO
EL2 < ENL
NO
YES
TRL = TRR-2*O2
Q1 = Q2
T1 = T2


Compute
volume of runoff
Output
volume of runoff
go to 60
NO
L = 7
YES
Compute
QIX,QOX,TOX,SOX,EOX
Output
summary of
flood ratings
go to 5
YES
mor = 1
NO
STOP