Determination of Necessary Spillway Capacity

by

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THESIS

TO

MR. J. H. KIMBALL

Principal Hydraulic Engineer
with the Tennessee Valley Authority
whose technical instruction and guidance
during the past six years have helped to
create a proper appreciation of the
responsibility ofthe engineer,
this thesis is respectfully dedicated

CONTENTS

EXHIBITS

EXHIBITS (Continued)

INTRODUCTION

Purpose

Water has often been referred to as man's greatest friend and his worst enemy. Ample support of the first claim is suggested by the location of every great city on a navigable waterway, while the cold truth expressed by the latter half of this trite phrase has been forcibly demonstrated in recent years by the disastrous floods which swept northeastern United States in 1935 and 1936, and by the great floods on the Ohio and Mississippi Rivers in 1937. In the present movement to conserve the natural resources of the United States, such disasters have given impetus to investigations for the conservation and control of the water resources and have resulted in the construction of a number of storage dams and the planning of a great many more. These projects range in size from the small earthern structures built by farmers to check erosion on their fields to the huge multiple-purpose projects and systems constructed by various governmental agencies to provide integrated development and control of the water resources of an entire drainage basin. A knowledge of the maximum flow which each of these structures may be called upon to pass during extreme rainfall and runoff conditions is vital in assuring the safety of the structure itself and the life of the whole area which it is designed to benefit.

It is the purpose of this paper to present and discuss briefly the more common methods which have been used to determine the spillway capacity that should be provided in such water control projects and to illustrate them by application to points in the Tennessee River basin. The author has assisted in similar investigations made in the Water Control Planning Department of the Tennessee Valley Authority, and fundamental data and methods available with the Authority have been utilized in the preparation of this thesis. Conclusions drawn and opinions expressed, however, are those of the author and not of the Tennessee Valley Authority, although they may be in substantial agreement in some cases.

The Problem

The design of any structure intended for impounding flowing water must include an emergency spillway for releasing flow exceeding that which the structure is designed to store. If the failure of such a structure were to result in only a limited amount of property damage and no loss of life, the spillway might be designed to carry a flow which is likely to be exceeded only once in 25 or 50 years, as it would be more economical to rebuild the structure and pay damages than to provide the expensive construction necessary to make it safe against any flood which might occur. Most of the structures with which we are concerned, however, are not of this type--they are planned to benefit a large area and a large number of people, and failure would result in great property damage and possible loss of life. Thus, they must be designed with spillways sufficiently large to carry with safety any flood which might reasonably be expected. A large spillway is expensive, especially when the limitations of the site are such that it is difficult to find space sufficient for spillway, navigation lock, and powerhouse, and the designing engineer is faced with the problem of providing the necessary spillway capacity and yet keeping the design practical and the costs within reason.

If the spillway provided in a water control structure is subjected to a larger flood than it was designed to carry, the headwater elevation must rise to an elevation above that considered in designing the structure. This is not necessarily serious in a concrete structure, since the design usually provides ample factors of safety which will cover such a contingency, but if the velocities produced below the dam are sufficiently high to cause excessive erosion at the foot of the spillway, the structure may be weakened to the point of failure. In the case of an earthern structure, headwater appreciably above that considered in the design, although below top of the structure, may weaken the structure and cause failure. Failure is sure to occur if the headwater rises sufficiently to overtop any portion of the earth banks, as a slight flow over the top produces sufficient erosion to permit an increased flow which then increases the erosion, and so on until the entire structure fails.

Failure of a small water storage or power dam during the passage of a flood is apt to cause some damage to roads and buildings immediately below it, but its failure may be particularly serious if it creates a sizeable flood wave which may travel downstream to another dam and cause it to be overtopped, and so start a chain of failures which grows progressively worse as the wave proceeds downstream.

The destruction resulting from the failure of a water control structure generally increases greatly with the size of the structure and the amount of water which it impounds, and due to the large volume of water stored and suddenly released, is particularly disastrous in the case of a structure built to control floods. The loss of life may be enormous, especially if a flood control reservoir of considerable storage capacity is built to protect a city which then continues to expand over the river's flood plain formerly considered unsafe for anything but those industries which could stand an occasional flooding, but now considered secure in the protection afforded by the upstream reservoir. The property damage in such a situation would also be enormous, and it would have been better if the structure designed to control floods and to protect the city had never been built at all. Damages to highways and railroads paralleling the river would be far greater for some distance downstream than the damage which might have been produced by the uncontrolled flood itself, and the interruption to business caused by the destruction of the transportation system would be a very real loss, even though it might be difficult to estimate. Although the loss of the dam itself and probably the powerhouse is not to be overlooked, there is also the loss of the use of these structures over the time required for their rebuilding. More important is the loss of the confidence of the people of the surrounding area and possibly throughout the whole country in the organization operating the structure, this lack of confidence probably affecting the success of that organization over several generations in the future.

Another less serious but very important result of a headwater elevation above that considered in the design is the damage to private property along the river above the dam. The height of a water control structure is often fixed to a large extent by the presence of buildings, roads, and other improvements along the edges of the proposed reservoir which would have to be bought or relocated should a higher dam and pool elevation be adopted. A flood which produces a headwater elevation sufficient to damage such properties may result in damage suits and litigation which are expensive in dollars but particularly expensive in the loss of the good will of those concerned.

Spillway capacity can be provided only by increasing substantially the total cost of the project, and in some cases the cost of providing a spillway of proper size may be so great as to make the whole project entirely unjustifiable. It is desirable that the spillway discharge directly into the natural channel of the river, although in many cases this is not possible and a side channel must be provided either through the abutments and thence back into the river or as open channels around the abutments. Small flows may be discharged through gate-controlled openings in the base of the dam itself, but this method is too expensive to be considered in dealing with the discharges that must be handled during a flood.

If power is to be developed at the project, the channel portion of the river must also provide room for a powerhouse. Discharge from the turbines must be carried away as rapidly as possible in order to keep tailwater elevations low and to provide the maximum possible head for power production, and direct discharge into the original river channel is the most satisfactory way of doing this. In the case of a navigable stream, a sizeable portion of the channel is occupied by a navigation lock to lift boats over the dam. Since sufficient depth must be available below the lock for boats to enter even during periods of low flow, the lock should be located in the deeper portion of the channel and must be aligned with the channel below to permit boats to enter with ease. Thus, the natural river channel must provide room for a powerhouse and a navigation lock, and since the damsite was probably chosen because of the narrow valley and river channel at this point, the width remaining for the spillway is apt to be small. To obtain greater spillway width, either the powerhouse or the lock must be moved out of the channel. This is likely to result in both an expensive and an impractical arrangement. The powerhouse might be moved downstream, but a flume or tunnel would be necessary to convey water from the reservoir to the turbines, and unless additional head were to be gained by this arrangement, the cost would be excessive. The navigation lock might be located out of the old river channel, but this would require the removal of enormous quantities of earth and would be likely to result in poor alignment of the lock and the navigation channel below. Other spillway arrangements might be made, such as tunnels through the abutments, open channels around the abutments, sluices and siphon spillways through the dam, etc., but usually these are more expensive than an open spillway.

With the spillway length controlled by the natural width of the river channel and the space that must be alloted to the powerhouse and navigation lock, increased discharge capacity may be obtained at a given headwater elevation only by lowering the elevation of the spillway crest and supplying crest gates or by increasing the height of the gates originally considered. The height of gates is limited by considerations in their design. The 50-foot gates proposed by the Tennessee Valley Authority for Kentucky Dam on the Tennessee River are among the highest in the world. As gate heights are increased, the intensity of pressure at the bottom of the gate increases as well as the total pressure on the gate. The greater gate height results in greater weight, while the increased pressures near the bottom require a stronger design which further adds to the weight. Increased weight of the gate and increased pressure of the gate on its supports both lead to more powerful lifting mechanisms and more complicated designs to reduce friction, while the increased pressures also require more elaborate gate seals. The increased thrust of the gate against its supporting piers requires that the piers be stronger, while the pressure differential on the two sides of a pier created by an open gate on one side and a closed gate on the other side result in another series of forces--all tending to increase the width of the piers and thereby reduce the net length of the spillway. Some of these difficulties are eliminated by using roller or drum gates in place of the usual Tainter or Stoney gates, but these may not be practical for great depth and their costs soon become excessive.

It is apparent, therefore, that the engineer designing a water control project is often faced with the difficult problem of providing a spillway which will carry with safety any flow to which it may be subjected, at the same time keeping his design practical and his costs within reason, realizing all the time the disaster that might result should he underestimate the flood-producing ability of the drainage basin or allow hisjudgment to be influenced by requests for a cheaper design.

COMMON METHODS

Introduction

A consideration of the problem of determining the spillway capacity that should be provided at a water control project indicates that it can be divided into four parts:

(1) Determination of a discharge hydrograph representing the maximum flood reasonably to be expected at the site under the conditions existing prior to construction of the project or any other water control project within the drainage area above. This may be termed the "maximum expected flood, natural conditions" or just the "maximum flood."

(2) Determination of a discharge hydrograph representing the greatest flood reasonably to be expected at the site under the conditions existing just prior to construction of the project. The effect of upstream regulation is here considered. This represents the greatest flood which must be considered in planning the project and may be termed the "project flood." The project flood may be the maximum expected flood as reduced by storage in upstream projects, or it may be the result of a very heavy storm centering over the uncontrolled drainage area immediately above the project.

(3) Determination of the maximum outflow and the maximum headwater elevation that are likely to result under the proposed scheme of operation of the project or under any operation that is likely to take place. This may be termed the "design flood," inasmuch as it represents the maximum flow for which the project must be designed. It will result from routing the project flood through the reservoir, consideration being given to the effect of the natural and controlled storages within the reservoir upon the project flood. It is here that the necessity

of considering a project flood rather than just a project discharge becomes apparent, as the maximum outflow and maximum headwater elevation are dependent upon the total volume of flood flow as well as the maximum rate of flow.

(4) Determination of the spillway dimensions which will most closely fit the functional design of the project. The spillway of a flood control project which does not have sufficient storage capacity to retain the entire flow of a single great flood not only must pass the design flood at a high or maximum headwater elevation, but it also must pass rather large flows when the pool is at a low elevation in order that the pool may be quickly drawn down upon the approach of a great flood to provide additional flood storage. It should also be possible to hold the pool at a comparatively low elevation during the passage of the first portion of a flood wave, thereby reserving considerable controlled storage space for use near the crest of the flood when this storage can be used to the greatest advantage in reducing the flood crest. This investigation involves routing floods of various types through the reservoir to note the effect of various heights of gates and lengths of spillway upon the effectiveness of the flood control operations of the reservoir. Speed with which the reservoir may be emptied after it has been filled by a flood may enter the problem, since it may be necessary sometimes to quickly release the stored flood waters in order to provide storage space for a second flood.

The design flood will have a maximum discharge considerably less than that of the project flood if the controlled storage is large, and there may be some reduction if there is no controlled storage, but the uncontrolled reservoir storage increment is greater than the natural channel storage increment for a given increase in discharge. The term "channel storage increment" as here used represents the volume of water stored temporarily in the natural channel of the river within the limits of the proposed reservoir when a certain change in flow takes place under given conditions of flow. "Uncontrolled reservoir storage increment" or just "uncontrolled storage increment" represents the volume of water stored temporarily within the reservoir when a certain change in flow takes place under given conditions of flow with all spillway gates open. The uncontrolled storage increment is normally greater than the channel storage increment since the reservoir area is generally much greater than the natural stream area, while the spillway usually limits the flow to a certain extent and forces the reservoir surface to rise almost as rapidly as it would rise under natural river conditions to accommodate a certain increase in flow. "Controlled storage" represents the volume of water stored under given conditions of flow in excess of the uncontrolled reservoir storage under the same conditions of flow. This volume may be released or stored at will by opening or closing the spillway gates, whereas the uncontrolled storage cannot be regulated by gate operation.

The design flood may have a maximum discharge equal to or even slightly greater than that of the project flood if there is no controlled storage under project flood conditions and the uncontrolled storage increment is not appreciably greater than the natural channel storage increment. This simplifies the problem to a determination of the project flood, and since flood volume does not enter the study, the problem is further simplified to the maximum flow of the project flood. Since most of the projects built for power alone belong in this class, the practice in the past often has been to compute the maximum flood to be expected at the site, considering any reduction in flow produced by the reservoir or upstream reservoirs as a factor of safety but neglecting the possibility of discharges being increased by the reservoir. Accordingly, a large number of equations and practices have been developed for estimating the maximum expected flow or the flow for which structures should be designed. In the case of flood control reservoirs and multiple purpose reservoirs as constructed in recent years, the large amount of space reserved for retarding or storing flood waters may result in a design flood substantially smaller than the project flood or the maximum expected flood, and more elaborate studies must be made to arrive at the most reasonable outflow for which a spillway must be designed.

Like most other hydraulic investigations, there is no simple method which can be applied to all projects to determine the maximum expected flood or the project flood. The judgment of the designing engineer must always determine the final answer—but there are many methods which he may employ to assist his judgment. The difficulty is tounderstand the advantages and limitations of each method in applying it to a specific project. Certain of the methods cannot be employed because of lack of necessary data, but lack of time should never be an excuse for neglecting any of the methods. There is too much at stake to omit any study which might shed additional light on the problem.

Empirical Equations

The term "empirical equations" is here used to cover the multitude of formulas expressing maximum flood flow as a function of size, or size and shape, of the drainage basin. The most commonly used type considers drainage area alone, and may be written as $Q = C A^n$, where "A" is usually the drainage area in square miles, "n" an exponent varying from about 0.5 to 1.0, and "C" a coefficient which may vary from about 5000 to 10,000, depending upon the various factors influencing flood runoff, such as type of storm, season of the year, topography, shape of the basin, vegetation, valley storage, and channel storage.

Several equations of a slightly different type were formulated by Kuichling from a study of drainage areas up to 5000 square miles on the Mohawk River. Kuichling gives the equations $Q = \frac{127,000}{M + 370} + 7.4$ for rare floods, and $Q = \frac{44,000}{M + 170} + 20$ for occasional floods.

A similar equation is proposed by Murphy for drainage areas up to 10,000 square miles from a study of New England streams: $Q = \frac{46,790}{M + 320} + 15$.

Those equations which attempt to include the effect of factors other than drainage area are generally lengthy and too involved to compare without a detailed plotting. The equation of C. R. Pettis, published privately in 1927, gives $Q = C\,P\,W^{1.25}$ in which "Q" is flood discharge in cubic feet per second, "W" is the average width of the drainage area, "P" is a rainfall coefficient, and "C" is a coefficient representing the combined influence of all the factors not mentioned in the other terms. The rainfall coefficient is the 100-year pluvial index of a 6-day storm as computed in the studies of the Miami Conservancy District for each of the quadrangles into which the eastern half of the United States was divided. The basin width is the average width obtained by dividing drainage area by the length of the main portion of the river.

A similar equation was proposed in 1929 by F. G. Switzer and H. G. Miller, with an adjustment in the rainfall factor to allow for size of drainage area and differences in the time of concentration.

Of the various equations involving frequency, that published by W. E. Fuller in 1914 is the best known. There are really two equations. The first states $Q = Q_{ave} (1 + 0.8 \log_{10} T)$ in which "Q_{ave}" is the average maximum annual flood, and "Q" is the probable maximum annual rate of flow during the period of years "T." The average maximum annual flood is to be obtained from a record of stages at the site, or if this is not available, from the second equation proposed by Fuller, which states $Q_{ave} = C A^{0.8}$ in which "A" is the drainage area in square miles and "C" is a coefficient whose value depends upon the flood producing characteristics of the drainage area. The equation thus reduces to $Q = C A^{0.8} (1 + 0.8 \log_{10} T)$. It should be noted, however, that the discharge given by this equation is the probable maximum annual flood to be expected in a given number of years andnot the flood to be equalled or exceeded in the given number of years. The latter conception is included in most equations involving frequency and in most of the frequency methods to be discussed later.

Another type of equation that should be mentioned here is that in which intensity of rainfall is one of the factors considered. The equations is generally written $Q = C i A$, in which "A" is the drainage area in acres, "i" is the intensity of rainfall in inches per hour, "C" is a coefficient of runoff, and "Q" is the discharge in cubic feet per second. This equation is often spoken of as the "rational method" and is used in computing the runoff for which city storm sewers should be designed. Variations of it are commonly used in computing culvert sizes and the size of openings that must be provided in small bridges, but it is essentially applicable to very small drainage areas and not to the drainage areas involved in reservoirs constructed for flood control or power.

Any of the empirical equations involving size alone or size and other characteristics of the drainage area should be used only after a considerable study of the data on which it is based to make certain that it is applicable to the drainage basin in question, and then great care should be exercised in selecting the proper constants and coefficients.

Maximum Flood of Record

A knowledge of the largest flood that has occurred on the stream being investigated is useful in many water control studies and should always be considered in attempting to fix the maximum flood that is to be expected, even though there can be no fixed relation between the size of these two floods. A record of stages extending back from 50 to 75 years is available at one or more points on most of the large rivers in the eastern portion of the United States, while newspaper accounts and private diaries often mention great floods which occurred in the previous 25 or 50 years. If the greatest known flood occurred prior to the period of continuous stage records, probably little is known about it other than the year of its occurrence and its height above low water or above some other flood, and conflicting stories may make it difficult to determine its height on present gages with any degree of accuracy. If the largest known flood has occurred during the period covered by gage records, however, its height, duration, and volume are available, and such information will be exceedingly useful.

In most cases, information will be lacking as to the height of the maximum flood of record at the particular point on the stream where a water control structure is proposed, but if the height is known at points where the drainage area is not too different or at points above and below the site, some estimate of the size of the flood at the site may be made.

The Miami Conservancy District used the maximum flood of record, that of 1913, as the basis for the design of their entire flood control system. The discharge of the flood of 1913 was increased by 40 percent to obtain the design flood used in planning and designing flood control reservoirs and improving the river channels at critical points. This was done, however, only after a very careful and thorough investigation of all storms which have occurred over eastern United States in an attempt to determine the relation of the storm causing the 1913 flood to the maximum storms that have occurred over areas subjected to similar meteorological conditions.

Similarly in other river basins, the greatest flood which has occurred over a long period of record may be increased by 50 percent, or perhaps more, to obtain some measure of the maximum flood to be expected.

In addition to its use in determining crest discharge, the hydrograph of the maximum flood of record is often useful along with hydrographs of other large floods of record in estimating the shape of the maximum flood hydrograph. The greatest flood of record on a drainage area was probably produced by the same type of storm that is likely to produce the maximum flood to be expected on this drainage area, and if this record storm of the past were reasonably well centered over the area, the shape of the resulting hydrograph should be quite similar to the shape of the hydrograph of the maximum flood to be expected. Thus, if the crest discharge of the maximum flood is fixed by a series of studies, the daily discharges of the maximum flood of record may be increased by the ratio of the two crest discharges to obtain a reasonable hydrograph for the maximum flood.

In making use of the maximum flood of record, it should be remembered that this flood is not the maximum that is reasonably to be expected on the drainage area, nor does it bear any fixed relation to the maximum flood, even though the flood records may cover a century or more. Certain drainage basins have never experienced a flood comparable to the great floods which have occurred on nearby basins having similar flood-producing characteristics, probably due to nothing but the lack of "rhyme and reason" which characterizes storms and floods, and care must be exercised that this lack of great floods during the period of record does not influence too greatly the final estimate of the maximum flood reasonably to be expected on this drainage area.

Frequency of Past Floods at the Site

A number of methods have been developed for determining the maximum flood to be expected in a given period of time from a mathematical study of flood records at the site or on a nearby stream having similar flood-producing characteristics. These methods are discussed extensively in engineering literature and have been followed in the design of many water control structures. The results obtained are to be trusted no farther in the solution of the present problem than is the maximum flood flow shown by the same records. Both are dependent upon the length of record and upon the floods that happened to occur during this period and are completely changed if different portions of the record are used for the study.

The earliest of the frequency methods was that used by W. E. Fuller in developing his equations mentioned previously. In this method, a tabulation is made of the maximum flood occurring in each year of the period of record, and the floods are listed in order of magnitude and numbered serially starting with the largest. Either calendar or water year may be used. The average annual flood is computed and the size of each flood expressed in terms of this average. These ratios are used in all following steps rather than the flood flows themselves, making the average annual flood the unit of flow. The ratios are cumulated, starting with the highest, and each divided by its serial number to obtain the average of all floods (expressed in terms of the average annual) equal to or exceeding each serial number. The total number of years represented by the record is divided by each serial number to find the corresponding period of years. This procedure gives the average of the floods (expressed in terms of the mean flood) occurring in various periods of time up to the length of the record. When plotted on semi-logarithmic paper with ratio to the mean as ordinate and time in years as abscissa, the points form a reasonably straight line which can be extended to read the most probable maximum annual flood to be expected in any given period of time. The line must start with a ratio to the average of 1.0 for a time of one year since the average annual flood must occur once every year on the average. The slope of the line is the coefficient of the log term in Fuller's original equation $Q = Q_{ave} (1 -/ 0.8 \log_{10} T)$.

It is to be noted that the discharge obtained by this method is the most probable maximum annual flood to be expected in any given number of years and not the flood to be equalled or exceeded once in this period of time. The latter conception is involved in most of the other frequency methods.

Statistical methods have been applied to flood records in many ways to determine more accurately the relation between the size and frequency of floods. A special coordinate paper has a vertical scale which is either arithmetic or logarithmic for plotting ratio to the mean annual flood, and a horizontal scale for plotting percent of time, the horizontal graduations appearing close together near the center of the sheet and increasingly farther apart near the edges in accordance with the normal probability curve. If annual flood are studied as in the Fuller method, the ratio of each flood to the average flood may be plotted against the corresponding percent of time during which this flood was equaled or exceeded, the points falling along a reasonably straight line which may be extended to read the size of the flood which is likely to be equaled or exceeded a certain percent of the time or once in any given number of years.

A method was developed by H. A. Foster for using the Pearsonian frequency curves for mathematically extending such a frequency curve, When the ratio of each flood to the average flood is taken, the variation of each from the average was found and used to compute the coefficient of variation and the coefficient of skew. Tables of factors were prepared by Foster for various percentages of time and various coefficients of skew, the factors being multiplied by the coefficient of variation, added to unity, and plotted against the corresponding percentages of time to obtain an extension to the frequency curve plotted as described above. The extension should be a straight line or should curve upward or downward depending upon whether the original flood data follows a normal frequency curve or is skewed.

Annual floods are also used in a similar statistical method presented by Allen Hazen. His tables give factors similar to those of Foster, each factor being multiplied by the coefficient of variation and added to unity before being plotted against the percent of time to which each factor corresponds. Refinements and variations of these statistical methods have been proposed by later investigators, including Goodrich, Slade, and Harris, while another variation was published by the California Department of Public Works.

A method of sampling which is altogether different from the "annual flood" system used in the methods just discussed may be called the "basic stage" system. Instead of considering only the largest flood occurring in each calendar year and eliminating other floods which may be only slightly smaller than the maximum of the year, this system includes all floods above a certain basic stage, even though a large number of floods may be included in certain years while other years are not represented at all. The method does include, however, all floods which are significant, since the basic stage may be chosen by the investigator. The only difficulty is that a low basic stage increases the length of the computations by increasing the number of floods, and a too high basic stage does not give a fair sample.

In this method, all floods above the chosen basic stage are tabulated in order of magnitude, and the size of each is plotted against the frequency with which each was equaled or exceeded. This frequency is usually expressed in terms of the average number of occurrences per one hundred years, or its reciprocal, the average interval between occurrences. Plotting may be on any of the various forms of arithmetic, logarithmic, or probability paper which will cause the points to fall along a straight line or smooth curve which can be extended to obtain the magnitude of less frequent floods. The curve should be extended by eye, however, and no attempt should be made to apply any of the probability methods, since the floods selected for the study occur at irregular intervals of time and do not represent the average sample necessary for a true probability study.

As a variation of the basic stage method, all daily discharges above the assumed basic stage might be used instead of only the crest discharge of each flood. This is often referred to as the "complete duration series" as contrasted to the "partial duration series" used in the method described above. The occurrences are again listed in order of magnitude and the percent of time that each is equaled or exceeded is computed from the total number of days in the period of record. Since many more items are involved in the complete duration series than in the partial duration series, the procedure becomes laborious and the items are usually grouped into classes and the midpoint of each operated upon as though it represented an item instead of a large number of items. A smooth curve is drawn through the plotted points and extended by eye. Here again, theoretical probability methods cannot be used in obtaining an extension of the frequency curve since the original sample does not represent a complete duration series but only the upper end of such a series.

The relation between frequency and magnitude of past floods as determined by the methods just outlined does not entirely answer the question of necessary spillway capacity until a design frequency is fixed. A structure whose failure would involve only a minor property loss might be designed with a spillway which would pass floods likely to occur every twenty-five or fifty years, as it would be more economical to rebuild than to support an expensive spillway. When great damage or loss of life would be involved in the failure of a water control structure, it has been common practice to design the spillway for floods of 500 to 10,000 year frequency, frequency curves generally becoming so flat at the very high frequencies that there is little increase in magnitude with frequency. The failure of a large storage reservoir in a densely populated are, however, would be so serious that the spillway should be ample to carry any flood which might occur rather than a flood to which a definite frequency can be assigned. It thus becomes important to know the limit of a frequency curve. This is indeterminate in most of the frequency methods, the flood magnitude continuing to increase somewhat with frequency, thereby leaving in the mind of the investigator considerable doubt as to the value of frequency studies in the present problem. Further, when the amount that the original flood data must be extrapolated (by eye or by one of the probability methods) is compared with the length of the available record and the effect of a slight change in the curvature of the extended line is noted, the investigator becomes all the more dubious as to the value of his results. Besides these uncertainties, it is to be remembered that the available records on American streams are generally less than 100 years in length, and more often in the neighborhood of 25 years, a length which is altogether too short to be of any value in predicting the size of the 500-year flood (a flood having one chance in 500 of being equaled or exceeded in a certain year) as has been demonstrated so many times by preparing frequency curves from different portions of the flood record at a single gage and noting the variation in size of the predicted 500-year flood.

Unit Graph

The "unit graph" is a rather new tool of the hydraulician. It made its first public appearance in 1932 in an article by L. K. Sherman and has since been investigated by many others with resulting improvements and refinements but no basic changes. The principal assumption involved is that a uniform rain of a unit duration over a drainage area will produce a hydrograph having a definite length, depending upon the run-off characteristics of the drainage basin, and having ordinates proportional to the total volume of run-off included in the hydrograph. If the ordinates of the hydrograph are modified by the ratio of the volume corresponding to one inch of run-off to the volume under the hydrograph, the resulting hydrograph will be a "unit graph"—a hydrograph representing a unit volume of run-off from a uniform storm of unit duration, the unit of storm duration generally being taken as one day to permit the use of daily rainfall figures. Actually, it is impossible to find a storm which is of uniform intensity over any sizeable drainage area and which at the same time has a duration of exactly one day. Thus it becomes necessary to compute unit graphs from a series of storms which approach these requirements, averaging the results to obtain a reasonably accurate "unit graph."

The unit graph once having been determined for a given drainage basin, run-off coefficients may be computed from a series of storms occurring during different seasons of the year and under various soil conditions and applied to the unit graph to determine hydrographs of run-off from any assumed rainfall. The daily amounts of rainfall are multiplied by estimated run-off coefficients to obtain daily amounts of run-off. The ordinates of the unit graph are multiplied by each of these amounts of daily run-off to obtain a series of overlapping hydrographs of run-off. Ordinates for corresponding days on the different hydrographs are added together to obtain total flow on each day, each day's rainfall producing run-off over a number of days. In this manner it is possible to compute the hydrograph produced by a single storm or various combinations of storms.

In the actual use of the unit graph method, various refinements are made in the method just outlined. Base flow is generally deducted from total flow to arrive at flood run-off in computing the unit graph, and similar estimates of base flow are added to the storm run-off computed from the unit graph when determining stream flow from rainfall. Elaborate studies may also be made to determine the run-off coefficients to be expected for rains of various intensities occurring in different seasons of the year with various amounts of rain on preceding days and various ground-water and soil conditions.

The unit graph provides a means of computing run-off from transposed and superimposed storms, and from a hypothetical storm estimated to represent the maximum rainfall to be expected over the drainage basin. It should be possible to predict the maximum rainfall to be expected over a given area more accurately than it is possible to predict the maximum flood to be expected from this same area. This is partially due to the greater number of rainfall stations and to the longer periods over which they have been operating, but also to the fact that great floods occur less frequently than great storms since each is produced by a great storm (or a series of storms) having a center near the center of the drainage area, a duration comparable to the period of concentration of the area, and a distribution to produce coincidence of floods from the tributary drainage areas, the soil and vegetation conditions being just right over the drainage area to produce a high percentage of run-off from the rainfall. The maximum rainfall to be expected may be estimated and the run-off computed by the unit graph method if the ground is assumed to be frozen or so saturated that the run-off will be nearly 100 percent of the rainfall. In some areas, it is necessary to consider a heavy covering of snow which will be melted by the rains and added to the assumed maximum rainfall. Similarly, large storms which have occurred over other areas having similar meterological characteristics may be transposed to the area under study and the run-off hydrograph computed for assumed run-off coefficients, or the area might be subjected to various combinations of storms which may be reasonably expected to follow each other.

The unit graph method of determining the maximum flood to be expected has several distinct advantages:

(1) The maximum rainfall to be expected over the area can be estimated with some degree of accuracy, and a hydrograph of run-off can be computed for this rainfall and any desired assumption as to soil saturation or amount of snow on the watershed.

(2) If the area is subject to seasonal storms, a maximum storm may be determined for each season and used with the proper run-off coefficients to determine the maximum flood to be expected in the different seasons when reservoirs are held at different levels.

(3) The unit graph is derived from run-off produced by storms which have occurred over the area being investigated and so takes into account the flood producing characteristics of the drainage area.

(4) The method gives the complete maximum hydrograph, not just the peak flow or the volume of run-off.

There are also several disadvantages:

(1) It is difficult to determine a satisfactory unit graph, especially when rainfall records are scanty and the drainage area is so large that storms are neither of uniform intensity over the whole area nor of a well-defined duration.

(2) The method seems to be limited to drainage areas less than several thousand square miles due to the difficulty of finding uniform storms of short duration over the larger areas.

(3) The unit graph must be made from relatively small floods during which the effect of channel storage is proportionally much less than would be expected during the maximum flood.

(4) Application of the unit graph method requires that run-off coefficients be estimated just as in other methods.

Maximum Recorded Rates of Run-off over Similar Basins

The empirical equations discussed previously are based more upon observed rates of run-off than upon theoretical considerations involving rainfall, drainage area, and stream flow. Each equation represents the conclusion reached by an investigator using the data available to him which he considered applicable to a certain river basin or to a certain section of the country. The observed maximum rates of run-off over drainage areas of all sizes were probably plotted in terms of cubic feet per second or cubic feet per second per square mile of contributing drainage area against the drainage area in square miles. Enveloping curves could then be drawn to cover certain ranges in drainage area and to include only certain types of drainage basins and basins subject to certain types of storms. A factor of safety could be added if the investigator thought it necessary, and the resulting enveloping curve could be expressed as an equation involving drainage area and maximum discharge, or length of the river, drainage area, and discharge if the investigator cared to introduce another variable into his plotting.

It is often desirable to prepare similar diagrams to show the maximum observed rates of run-off in the drainage basin selected for a water control project and in adjacent basins having similar rainfall and run-off characteristics. Such a diagram presents to the eye a good picture of the available information on flood flow, its variation with size of drainage area, and the agreement or lack of agreement between flood flows determined from empirical equations, or other methods, and the maximum flows actually observed both in the area being studied and in surrounding regions. The plotting may be either on plain coordinate paper or on logarithmic paper. The later is preferable, since observed maximum flood flows generally fall so that a reasonably straight line represents their upper limit on this paper, while many empirical flood flow equations are of the exponential type and, therefore, represent straight lines on logarithmic paper.

Considerable caution may be necessary in studying such a diagram. If the available data on flood flows in scanty, the upper limit indicated may be too low, and if the discharges assigned to past floods were not carefully determined, the entire diagram may be misleading.

Rational Run-off Method

The so-called "rational" methods of computing flood flows from rainfall are an adaptation and expansion of the method long used in computing the flow for which city storm sewers should be designed. The basic equation involved is that stated previously: $Q = C\,i\,A$, in which "A" is the contributing drainage area, "i" is the intensity of rainfall, "C" is a coefficient expressing the ratio of run-off to rainfall, and "Q" is the resulting discharge. As originally used in sewer design studies and later adapted to slightly larger drainage areas, area is expressed in acres, rainfall intensity in inches per hour, and discharge in cubic feet per second.

The basic assumption involved in the use of this equation is that maximum run-off will be obtained from a given drainage area when it is subjected to a rainfall having a duration equal to the time of concentration of the area, or the time required for water falling at the outermost limits to reach the gaging station or dam site at the lower end of the area. This brings up the corollary assumption that average rainfall intensity over a given area is inversely proportional to duration so that a higher intensity can be expected over a short interval of time than over a longer interval. Thus, in a drainage area of given size and shape and having a given time of concentration, the maximum flood will generally occur when the entire area is contributing to the flow at the lower end, and the run-off from a storm whose duration is less than the time of concentration will be less than the maximum possible run-off since rain has ceased to fall on the lower areas by the time run-off from the farthest limits of the drainage area reaches the lower end. If the duration of the storm is greater than the time of concentration, the entire drainage area may be contributing to the flow at the lower end over a period of time, but the intensity of the rainfall and likewise the rate of run-off would have been greater if the duration were somewhat less and more nearly equal to the time of concentration of the drainage area.

It should be noted, however, that these statements are true only in a general way and that certain areas or certain storms may prove to be exceptions. Drainage areas which are long and narrow may have such a long time of concentration that the intensity is materially reduced and greater flows are produced by a more severe storm of shorter duration covering only a portion of the drainage area. Large drainage areas may have such a long time of concentration that no single storm can be of sufficient duration, and the maximum flood will be produced by one storm over the headwaters and another storm at a later time over the lower end of the basin. Direction of travel of a storm may also enter into the analysis of a large drainage area, a storm which moves downstream so that it continually adds water to the flood crest producing a greater flood than a more severe storm which moves up the drainage area so that rain falling in the headwaters tends to increase the duration of the flood crest without increasing its height.

The basic equation $Q = C\ i\ A$ may be applied directly to small drainage areas, values of "C" and "i" being obtained by estimate or reference to a handbook. Such a procedure can hardly be called a "method" and properly belongs with the empirical equations previously given.

As an improvement over obtaining values of "C" and "i" by estimate, various diagrams have been prepared to give these factors for drainage areas of various sizes and in various locations throughout the United States. Of the two factors, values of "C" can generally be determined with less uncertainty. The winter or spring of the year is the flood season in most sections of this country, and in considering an extreme flood, the ground may be expected to be either frozen or nearly saturated with water at its beginning, thereby giving a run-off coefficient of nearly 100 percent. In those sections where summer storms such as West Indian hurricanes may produce the greatest rainfall, the run-off coefficient is much more indeterminate. The same is true of very small areas which may be subjected to cloudbursts in any season of the year. In his work on the rational run-off method, Merrill Bernard has determined limiting values of the run-off coefficient throughout the eastern portion of the United States, the values being a full 100 percent throughout the Ohio River Basin and decreasing towards the southeast and northwest to a minimum of 60 percent in Florida and 50 percent in the western plain states. These values are considered to represent a frequency of 100 years and may be reduced for less frequent storms.

The intensity of rainfall "i" that must be applied to a drainage area to produce the maximum flood is more difficult to determine since it involves duration, and this in turn involves the time of concentration of the drainage area. Merrill Bernard has expressed the intensity as $i = \frac{K\ T^x}{t^e}$ in which "T" is the frequency and "t" is the time of concentration. Maps of the eastern portion of the United States are prepared to show values of "x" for different localities, one chart applying to duration periods from 5 to 60 minutes and another for periods from 60 to 1440 minutes. Values of "e" are shown in a similar fashion, while values of "K" are shown as dependent upon location alone. The time of concentration still remains to be found, and Merrill Bernard has prepared elaborate charts for estimating this from a knowledge of the length and width of the drainage area, the length, slope, and size of the principal channel, and a series of factors covering other characteristics of the watershed.

This method of determining rainfall intensity and percent run-off appears reasonable and should be superior to any estimate of these coefficients, although values of these coefficients are dependent upon records of somewhat limited length. The method, however, appears too rigid to be practical in studying drainage areas of any size.

The most elaborate of the so-called "rational" methods of computing flood flow divides the drainage area into a series of zones so that rain falling any place within a zone will flow to the lower end of that zone within a unit of time. The basic equation Q = C i A is applied to each zone for each unit of time to obtain a hydrograph of discharge from each zone which can be added to similar hydrographs from the other zones, with proper time of travel allowance, to obtain the hydrograph at any point on the stream. The method is very laborious and requires considerable data on the drainage area and its principal streams, but once the basic information is gathered, rainfall of any magnitude and any duration may be applied to the drainage area and the resulting hydrograph computed without undue effort. Some adjustment of the hydrographs may be necessary, however, since the method does not take into account the flattening effect of channel storage, and this must be estimated or determined by a process of routing the flood flows down the principal channel.

The first and most difficult step in the method is to divide the drainage area into the necessary zones. The time required for water to travel from point to point on the main channel and principal tributaries is computed from a past flood whose profile is known so that average velocities can be computed from known cross sectional areas and discharges or is estimated by means of the Manning equation for open channel flow and the assumption that the hydraulic radius is equal to the flood height above low water, or possibly three quarters of this height, and that the slope is the same as the slope of the low-water profile. These velocity and time-of-travel computations allow the main river and principal tributaries to be divided into time zones, and it remains necessary only to carry these zones to the edges of the basin. The same methods are applied to all the minor tributaries whose profiles are available, velocities being estimated in other streams whose profiles are not known.

The unit of time to be used in laying out the zones requires some consideration. Most records of rainfall have been obtained from gages which were read daily, as automatic rainfall gages are too new and too widely separated to furnish the necessary rainfall data on past storms. With rainfall records limited to daily determinations, the same unit of time becomes convenient in studying run-off. Unless the drainage area is sufficiently large, however, to have a time of concentration of something like ten days, a shorter time unit must be used in order to obtain sufficient points on the final discharge hydrograph, daily rainfall figures being split up into the shorter units of time. A time unit of twelve hours, six hours, or even less may prove successful for the smaller drainage areas, with the obvious disadvantage that the process becomes more laborious as the length of the time unit is decreased.

With the time zones sketched on a map of the drainage basin, each may be planimetered to determine its area. In the application of a certain rainfall to the drainage area, run-off coefficients may be chosen for each zone based upon the season of the year, condition of the ground, amount of cover, etc. The investigator would choose coefficients on the basis of his judgment and knowledge of the run-off characteristics of each zone.

The investigator may apply any rainfall which he thinks reasonable to the drainage basin, transposing past storms from other drainage areas or preparing hypothetical rainfall distributions. He may assume any run-off conditions which appear reasonable and may even add in the effect of melting snows in certain zones and frozen ground in other zones, comparing the resulting hydrographs in terms of crest flow, total volume, duration, and general shape. By neglecting the run-off from certain zones, he may determine the effect of a storage reservoir below those zones or compare the effects of various possible storage reservoirs.

Other difficulties are generally encountered, however, which prevent the problem from being solved so simply. Channel storage usually operates to reduce and broaden flood crests as the floods move downstream, a considerable volume of water entering each length of river before any increase in flow can take place out of the lower end of the length. This volume flows out of the length after the crest has passed, thereby increasing the duration and decreasing the height of the flood. A similar effect is produced by the difference in surface slopes on the rising and falling sides of the flood wave, the steeper slope of the front of the wave causing this water to travel faster than the water at the crest, and the flatter slope at the rear of the wave causing that water to travel slower than the water at the crest.

An elaborate routing scheme might be set up to transfer the water along the main channel from zone to zone or from the mouth of one tributary to the mouth of the next, or an estimate might be made as to the probable crest reduction that is to be expected due to the above factors and the reduction expressed as a percentage of the computed crest discharge. A third possibility is to consider the rational method as resulting in a hydrograph which is representative of the shape and volume of the final hydrograph but is indicative only of the crest discharge; the crest discharge is estimated by some other method and the computed hydrograph reduced to this crest discharge without change in volume and without appreciable change in shape.

The rational run-off method appears to furnish a very flexible and important scheme for estimating the maximum flood to be expected from a drainage area where rainfall is the principal factor in producing floods. The method also provides the basic information necessary in computing flood hydrograph which would result from any rainfall occurring under any conditions of the drainage area. A complete analysis by this method, however, requires very complete data on the drainage area and its principal streams and very lengthy and laborious computations. Estimates might be substituted for some of the computations, the rational method controlling the shape and volume of the final hydrograph while other methods are used to fix the crest discharge. The extensive study of the drainage area required in the method has the decided advantage that hasty conclusions cannot be reached and the investigator must become thoroughly familiar with all parts of the basin and its flood-producing characteristics. The storms or hypothetical rainfall distribution which are to be applied to the area and the run-off rates which are to be used require the careful judgment of one having long experience in this type of work and cannot be attempted by an amateur. The rational method is one of the newer methods of estimating maximum flood hydrographs, and along with the unit graph method, is deserving of a considerable amount of additional investigation.

MAXIMUM FLOOD—TENNESSEE RIVER AT CHATTANOOGA

Introduction

Floods on the Tennessee River at Chattanooga have been studied by a great many investigators. Writers of textbooks and articles on methods of estimating flood flow have often used the records of floods at this point to illustrate their ideas, while more recently the problem has been studied in detail by the Corps of Engineers and the Tennessee Valley Authority in planning a system of reservoirs for the Tennessee River Valley and local flood protection works for the city of Chattanooga.

The city of Chattanooga is located on the Tennessee River in the narrow portion of the Tennessee Valley just above the gorge which the river has cut through Waldren's Ridge and the connecting ridges which divide the basin of the Tennessee River into two parts nearly equal in area, similar in shape, but altogether different in geology and topography. The commercial and industrial portion of the city and a number of the residential sections spread across the river's flood plain, while a substantial share of the residences are located on the surrounding ridges far above the reach of any flood waters. The city has not suffered severely from floods in recent years, but there is no reason to suspect that future floods will not equal or exceed in magnitude the large floods which occurred 50 to 75 years ago. It has been estimated* that a repetition at the present time of the flood of 1867, the maximum of record, would result in a direct loss of $37,6000,000, with an additional intangible loss which might be even more serious. Thus, it is apparent that the city of Chattanooga needs flood protection—and a plan for its protection involves first a determination of the maximum flood against which the city must be protected.

*House Document No. 91, 76 Congress, 1st Session

A knowledge of the maximum flood to be expected was essential for the design of Chickamauga Dam, now nearly completed by the Tennessee Valley Authority across the Tennessee River a few miles above Chattanooga. The determination of this flood at Chattanooga applies with slight modification to Chickamauga Dam.

The record of floods at Chattanooga is one of the longest on the Tennessee River so that flood estimates at other points along the river must be based to a certain extent upon this record. The location of Chattanooga at the natural division between the upper and lower basins of the Tennessee River allows the results of these flood studies to be adjusted to apply for some distance both above and below the city; Chattanooga thus acts as a good reference point to which floods on other portions of the Tennessee River can be related.

The problem of determining the maximum flood to be expected on the Tennessee River at Chattanooga is therefore important in that it affects the design of the flood protection works of the city, the design of Chickamauga Dam immediately above the city, and the determination of the maximum flood to be expected within a considerable distance along the Tennessee River.

Description of the Basin

The drainage area of the Tennessee River at Chattanooga is 21,400 square miles, or a little more than that of the Cumberland River at its mouth and a little less than the Susquehanna River at Harrisburg. The basin is somewhat elongated with a length of 260 miles, a maximum width of 120 miles, and an average width of about 80 miles. The topography is generally mountainous. Mountain ranges of the Appalachian system rise to an average elevation of about 5000 feet on the south, southeast, and north sides, while the Cumberland Mountains form a 2000-

foot divide along the northwest side. Within the basin, the Great Smoky Mountains rise to elevations above 6000 feet in the southern areas, and a series of generally parallel ridges extends over a large portion of the remainder of the basin and reaches elevations between 1000 and 2000 feet. Exhibit 1 contains 1000-foot contours which show the general topographic features of the basin as well as its shape and drainage pattern.

The Tennessee River is formed by the junction of the French Broad and Holston Rivers a few miles above Knoxville and about 190 river miles above Chattanooga to flow generally southwestward parallel to the Ohio and Cumberland Rivers. The French Broad River, with a generally fan-shaped basin, drains the eastern portion of the Great Smoky Mountains. The Holston River flows between straight, parallel ridges throughout most of its length, although it branches into the North and South Forks to make the upper portion of the basin fan-shaped. The Little Tennessee and Hiwassee Rivers enter the Tennessee River from the south to drain most of the remaining portion of the Great Smoky Mountains; the Clinch River parallels the Holston River on the north to enter the Tennessee River between the Little Tennessee and Hiwassee Rivers.

The surface soils of the Tennessee River basin above Chattanooga are generally of a clayey nature, nearly impervious to water, while the rocks underlying the eastern portion of the basin are of the Pre-Cambrian era and therefore impervious and lacking in the sinks, caves, and solution channels which are prevalent in the shale and limestone rocks of the Mississippian era which cover the remainder of this basin. A small amount of surface storage is provided by the limestone sinks, but there are no lakes and swamps to furnish surface storage comparable to that found in most river basins.

The average annual rainfall averages about 52 inches over the basin, from a minimum of around 40 inches at stations in the sheltered portion of the French Broad Valley to a maximum of around 60 inches on the exposed peaks of the higher mountain ridges forming the southeastern boundary of the French Broad area. Exhibit 2 shows the isohyetals depicting variations in the 30-year mean annual rainfall over the basin.

The area is subject to cyclonic storms of the type that cross the eastern portion of the United States from the southwest to the northeast, and some portions of the basin may be subject to intese storms coming from the Gulf of Mexico. The subject of storms and storm rainfall will be discussed in more detail in connection with the rational method of estimating flood runoff.

Available Hydrologic Data

The record of stages of the Tennessee River at Chattanooga is practically complete from April 1, 1874, when the first staff gage was constructed by the U. S. Signal Service, the predecessor of the present U. S. Weather Bureau. Gage records are not available for the flood of 1867, the greatest of record, but the height of this flood has been determined quite accurately from flood marks, and the general shape of the hydrograph has been pieced together from newspaper descriptions of the flood. Great floods are known to have occurred in 1826 and 1847, but neither of these rivaled that of 1867, and it is doubtful if any other great floods occurred in the period of legendary records, 1826 to 1867. Exhibit 3 gives a summary of important data relative to the various river gages operated at Chattanooga; Exhibit 4 shows the principal floods of record.

Discharges of the Tennessee River at Chattanooga have been measured by the Corps of Engineers and by the U. S. Geological Survey, but the stage-discharge relation has been complicated by backwater from Hales Bar Dam constructed by private interests in 1913, at a point about 33 miles below the city. Various stage-discharge curves have been drawn and extended to indicate the discharges reached by high floods of the past and the stages that might be reached by future floods. These rating curves have been reviewed in the Flood Control Section of the Tennessee Valley Authority, and a slightly different curve has been prepared which agrees reasonably well with all discharge measurements, with the discharge indicated for great floods from a study of these floods at other points in the Tennessee River basin, and with backwater curves computed throughout the length of the Tennessee River. The discharges indicated by this rating curve are felt to be reasonably accurate over the complete range of flow to be expected, one section of the curve applying to conditions prior to the construction of Hales Bar Dam and another section to conditions after the construction of this dam.

Stage records have been kept at a number of other points on the Tennessee River besides Chattanooga. The following tabulation shows the date of establishment of these early gages, the river mile, and the drainage area.

Station	River Mile	Drainage Area	Established
Johnsonville	96.4	38,500	October 1, 1875
Riverton	226.5	31,560	May 18, 1891
Florence	256.6	30,810	November 7, 1871
Decatur	304.5	26,900	October 1, 1875
Bridgeport	414.4	22,600	March 23, 1892
Chattanooga	464.2	21,400	April 1, 1874
Kingston	568.2	12,500	October 1, 1874
Knoxville	647.2	8,900	January 1, 1875

Rain gages have been operated at many points in the Tennessee River basin by the U. S. Weather Bureau, some of these records extending farther back than those of the river gages. Most of the rain gages, however, have been located in the larger towns along the rivers and very few records are available from the higher and more inaccessible areas. This makes it difficult to compute average rainfall over the basin for any storm or any period of time. Many additional gages of the automatic type have been installed by the Tennessee Valley Authority in the regions having a deficiency of gages; Exhibit 5 shows the location of both rain gages and stream gages in the basin above Chattanooga.

Application of Empirical Equations

Empirical flood flow equations are of little assistance in estimating the maximum flood to be expected at Chattanooga. While the climatological and flood-producing characteristics of the drainage area appear quite similar to those of other areas in eastern United States, the drainage area of 21,400 square miles is considerably larger than the areas from which flood flow equations have been developed. Consequently, most of these equations must be extrapolated beyond the limits of the data from which they were derived—a process which is very questionable with any data and very dangerous in this case.

Kuichling's equations were derived from flood flow data for drainage areas less than 5000 square miles on the Mohawk River, and an attempt to extrapolate these equations to the drainage area of the Tennessee River at Chattanooga gives the following absurd results:

Rare Floods, $Q = \frac{127,000}{M + 370} + 7.4 = 284,000$ cfs

Occasional floods, $Q = \frac{44,000}{M + 170} + 20 = 470,000$ cfs

The Murphy equation developed for areas up to 10,000 square miles in northeastern United States gives

$Q = \frac{46,790}{M + 320} + 15 = 370,000$ cfs

The Fanning equation for New England streams gives $Q = 200\ M^{5/6} = 812{,}000$ cfs, while the very similar Dickens equation for the Central Provinces of India gives $Q = 500\ M^{\frac{3}{4}} = 885{,}000$ cfs.

The Meyers equation for extreme floods as modified by Jarvis gives $Q = 10{,}000\ M^{\frac{1}{2}} = 1{,}460{,}000$ cfs.

The Fuller equation with "C" estimated at 70 and "T" taken as 1000 years gives $Q = C\ A^{0.8}\ (1 + 0.8 \log_{10} T) = 690{,}000$ cfs.

The general width formula of C. R. Pettis with a 1-day rainfall of 7 inches and an average basin width of 47 miles gives a probable 100-year flood of $Q = 480\ P\ W^{1.25} = 415{,}000$ cfs.

Maximum Observed Rates of Runoff

The record of stages of the Tennessee River at Chattanooga dates back less than 70 years, while the entire period over which there are actual or legendary records of floods is less than twice this length. The most reliable method of supplementing this record in attempting to fix the maximum expected flood is through the collection and comparison of the rates of runoff experienced during major floods which have occurred in drainage areas subjected to similar storms and having similar runoff characteristics.

A study of the storms which have produced the major floods of record in the eastern portion of the United States indicates that these may be divided into two general classes: those of the cyclonic type which move across the United States from west to east or from the southwest to the northeast, and the West Indian hurricanes which move up into the United States from the south or southeast. The mountain ridges which surround the basin of the Tennessee River above Chattanooga are high enough to force any West Indian hurricanes which may travel this far inland to drop most of their moisture along the edge of the basin rather than over the basin itself in a general storm of the duration required to produce the maximum flood at Chattanooga. This leaves only storms of the cyclonic type to be considered as being capable of entering this basin and producing the maximum flood and eliminates from our consideration the floods produced along the Gulf and Atlantic Coasts by West Indian hurricanes. The remaining portion of eastern United States is subject to cyclonic storms and is therefore comparable to the Tennessee Valley in this respect.

The basin of the Tennessee River above Chattanooga is generally mountainous with steep slopes, impervious soils and rocks, and lack of surface storage. Most of the rivers draining the Appalachian Mountains are similar in this respect. Thus, the flood-producing characteristics of the Connecticut, Susquehanna, Delaware, Potomac, Hudson, Cumberland, and upper Ohio Rivers are generally similar to those of the Tennessee River above Chattanooga, and the maximum rates of runoff from these river basins may well be studied in attempting to fix the maximum flood to be expected at Chattanooga.

An examination of the maximum recorded rates of runoff from these rivers draining the Appalachian System discloses that practically all have been produced by one of the following seven great storms:

May 31–June 1, 1889—Susquehanna Basin
March 23-27, 1913—Ohio River Basin
November 2-5, 1927—Connecticut River Basin
March 21-23, 1929—Tennessee and Cumberland River Basins
July 7-8, 1935—Upper Susquehanna River Basin
March 14-22, 1936—Upper Ohio, Susquehanna and New England
January 6-25, 1937—Lower Ohio River Basins

The maximum rates of runoff produced by these storms in the various basins were carefully tabulated from the records of the U. S. Geological Survey and other reliable sources. The entire tabulation fills 62 sheets and is too voluminous to include here, but a sample sheet is reproduced in Exhibit 6.

To provide a visual comparison of these various rates of runoff, runoff in cubic feet per second per square mile of drainage area was plotted against drainage area on log-log paper, different symbols being used for the different floods. The resulting diagram is reproduced in Exhibit 7. It is noted that the highest points on the diagram define rather precisely a line passing through a runoff rate of 50 cfs per square mile for a drainage area of 10,000 square miles and 500 cfs per square mile for a drainage area of 100 square miles, this line having the equation $Q = \frac{5000}{\sqrt{A}}$ in which "Q" is the runoff rate in cfs per square mile, and "A" is the drainage area in square miles.

A similar diagram showing observed maximum rates of runoff was prepared from preliminary studies made in 1934-35 in connection with the determination of the maximum floods to be expected at various points in the Tennessee River basin. These studies were reviewed by a special board of consultants composed of Harrison P. Eddy, Ivan E. Houk, Gerard H. Matthes, and Daniel W. Mead, and a report was submitted by this Board under date of May 29, 1936. In this report the Board recommended the above relation between maximum flood rates and drainage area for general application to drainage areas in the Tennessee River basin of more than 500 square miles. In making this recommendation, however, the Board of Consultants recognized that in dealing with particular drainage areas "each problem should be studied in detail with the exercise of judgment in the light of the local conditions."

For the Tennessee River at Chattanooga with a drainage area of 21,400 square miles, this represents a runoff rate of 34 cfs per square mile, or a total discharge of 730,000 cfs.

It is to be particularly noted that the enveloping curve or limiting line shown on Exhibit 7 is supported in the vicinity of the drainage area at Chattanooga by a point representing a discharge rate of 30.7 cfs per square mile for a drainage area of 24,100 square miles, the symbol indicating that this point belongs to the storm of March 14-22, 1936, covering the northeastern part of the United States. Actually, this point refers to the Susquehanna River at Harrisburg. The flood-producing characteristics of this basin are similar to those of the Tennessee River above Chattanooga, as the watershed of each is principally within the Appalachian Mountains with steep slopes, low infiltration loss, and negligible surface storage. A second point is also noted at the same drainage area but with a runoff rate of 29.0 cfs per square mile. This point represents the 1889 flood on the Susquehanna River at Harrisburg. A study of this flood and the storm which caused it indicates that the greater portion of the rain fell in slightly over 24 hours—too short a period for the entire drainage area to be contributing to the flood crest at Harrisburg. This immediately suggests that the 1889 flood is not the maximum to be expected, a conclusion which has been verified by the 1936 flood.

Frequency of Past Floods

A long record of past occurrences is often the best basis for predicting future events, but care must be exercised to make sure that the available sample is a fair sample and that the data which it presents is reasonably accurate. A record of floods at a single gaging station seldom, if ever, meets both of these requirements, and the record of the Tennessee River at Chattanooga is no exception. This is not seriously objectionable if past floods are to be used in formulating a reservoir operation plan or in estimating how frequently a highway may be flooded, but it becomes very serious if a record having a length of less than a hundred years is to be the basis for fixing the size of the maximum flood for which a water control structure must be designed—a flood which might occur once in 500, 1000, or 10,000 years. Accordingly, the record of past floods at Chattanooga was not used as the basis of a statistical determination of the maximum flood to be expected, but studies were made to show the great inadequacy of this record for any such purpose.

A gage has been operated on the Tennessee River at Chattanooga since 1874, and daily readings have been published by the U. S. Weather Bureau over the entire period. Discharge measurements were first made by the Corps of Engineers and later by the U. S. Geological Survey, the latter organization operating a recording gage for many years and publishing daily discharges over the greater portion of the entire period of record. Although a number of gages have been operated at and near Chattanooga by both the Weather Bureau and the Geological Survey, the Weather Bureau gages have been located relatively close together and have been set to practically the same datum so that gage heights are generally comparable over the period of record. Similarly, although the discharges published by the Geological Survey have been determined from gages located at Hales Bar and Bridgeport as well as at Chattanooga, the drainage area at the various gages is not materially different and the discharge records are generally comparable. The discharges of the great floods which have occurred at Chattanooga, however, are quite uncertain. Most of the Chattanooga discharge measurements were made during recent years, and the highest recent flood is that of 1917 with a discharge of about 341,000 cfs as compared with an estimated discharge of 459,000 cfs for the maximum flood of record, that of 1867. Further, the maximum measured discharge is only 276,000 cfs, and only 7 discharge measurements have been made for flows exceeding 200,000 cfs. The discharge measurements themselves may be in error by appreciable amounts, especially those made during great floods when the river velocities are high, the weather is cold and rainy, and the water is filled with floating and submerged debris of all sorts which is apt to damage or carry away the measuring equipment. The stage-discharge relation probably has changed an appreciable amount between the early floods of record and the time of the discharge measurements upon which flood discharges must be based, stages at Chattanooga being increased somewhat due to the construction of Hales Bar Dam 33 miles below the city and to the encroachment of the city onto the flood plain formerly occupied by the river. Stages may have been reduced somewhat due to improved alignment through the city and clearing of the wooded portions of the overflow area for some distance above and below the city. The discharges published by the U. S. Geological Survey are taken from stage-discharge relations prepared from time to time from the available discharge measurements and are not all taken from the same rating curve. This procedure may take into account some of the major changes in the stage-discharge relation, but the resulting discharges are apt to be somewhat unreliable at high stages because the rating curves are drawn from insufficient discharge measurements. Consequently, for the studies made by the Tennessee Valley Authority, a particular rating curve was adopted for conditions prior to the construction of Hales Bar Dam and another curve for conditions after the construction of the dam, the two curves uniting at a discharge of 340,000 cfs and remaining coincident at all higher flows. These curves are based upon all available discharge measurements, upon a consideration of the crest discharge of past floods as indicated by a study of these floods at other points in the Tennessee River basin, and upon backwater curves computed throughout the length of the Tennessee River for both natural and reservoir conditions.

Although gages have been operated at and near Chattanooga by both the U. S. Weather Buearu and the U. S. Geological Survey and studied by both this agency and the Tennessee Valley Authority, it must be recognized that the estimated discharges of the higher floods may be in error by as much as 10 percent, and this possible error must be kept in mind when the data is used in studying past floods and in predicting possible future floods.

The length of record at Chattanooga which is available for making complete frequency studies is less than 70 years, although considerable information has been gathered pertaining to the flood of 1867, and high floods are known to have occurred in 1826 and 1847. This is hardly sufficient information to make an estimate of the 10,000-year flood or even the 500-year flood, especially when a study of the data shows that it is not a "fair sample."

To show the fallacy of attempting to extrapolate flood discharges from a frequency curve based on the record of past floods at Chattanooga, a series of frequency curves were prepared using different portions of the flood record and assuming that these frequency studies might have been made at various times using the records available at those times. Exhibit 4 shows the magnitude of the past floods at

nooga; the top diagram gives a picture of the distribution by

e lower portion shows the distribution by seasons. It is noted that the three greatest floods of record, those of 1867, 1875, and 1886, all occurred in a period of 20 years, while the 20-year period following the flood of 1917 does not contain a single flood comparable to any of these and only a few floods amounting to more than 50 percent of the highest. Using the basic stage method and considering all winter floods having a discharge of 100,000 cfs or greater, the resulting 500- and 1000-year floods are as follows:

Period Considered	Number of Years	500-year Flood	1000-year Flood
1867 to 1886	20	750,000 cfs	840,000 cfs
1887 to 1935	49	421,000	450,000
1867 to 1935	69	545,000	596,000

This is a variation of 78 percent for the 500-year flood and 87 percent for the 1000-year flood. The discrepancy doubtless would increase to even greater figures if an attempt were made to determine the maximum flood to be expected. The various frequency curves and their extensions are shown on Exhibit 8.

The record of floods on the Tennessee River at Chattanooga has been used by several writers to illustrate the methods they have proposed for estimating the magnitude of the maximum flood to be expected or the magnitude of the flood corresponding to any frequency. A comparison of these figures is interesting in that it shows the great variation in the results obtained by the different investigators. It is to be noted, however, that all of this variation is not due to the method used in handling the data, but to the data that was used—the period of record selected and the discharges adopted for the various floods. The various frequency curves are reproduced on Exhibits 9 to 13, and the resulting floods of 500,1000, and 10,000-year frequencies are shown together with the reference, method, and period of record used. All the frequency curves taken from Water Supply Paper 771 are based on the same period of record and the same discharge data so that variations in the results are wholly due to the method used. In the case of the 500-year flood, the variation is from 361,000 to 405,000 cfs, or about 12 percent, while in the case of the 10,000-year flood, the variation is from 390,000 to 480,000 cfs, or about 25 percent. The 25 percent variation is remarkably small considering that a record of less than 100 years has been extrapolated to 10,000 years, but any sense of accuracy of the results is short-lived when the remainder of the table is examined. As previously explained, the frequency curves shown on Exhibit 8 are based on different periods in the flood record at Chattanooga and show a variation in themselves of nearly 90 percent for the 1000-year flood. The maximum variation considering different periods of record and different determinations of flood flows is from the 840,000 cfs shown on Exhibit 9 for the 1000-year flood using the period 1867 to 1886, to the 375,000 cfs shown on Exhibit 11 for the 1000-year flood as determined by the Goodrich Type V method using the period 1875 to 1951, and discharges as published by the U. S. Geological Survey. This variation amounts to about 150 percent.

Further evidence of the erratic results likely to be obtained by using frequency methods based on relatively short records and discharges which are of questionable accuracy is shown on Exhibit 14. Here the magnitude of the 500-year flood expressed in cfs per square mile of drainage area is plotted against drainage area, the frequencies having been computed by the Corps of Engineers using the Goodrich method. The points scatter rather widely about a mean line having a slope which indicates that the discharge varies in proportion to the square root of the drainage area. There should be some scattering of the points due to the difference in the flood producing characteristics of the different drainage basins, but the variation of several hundred percent seems to indicate that the frequency method cannot be applied to records as short and discharges as uncertain as those in the Tennessee River basin.

Exhibit	Reference	Method	Period	Discharge in 1000 cfs 500 year	1000 year	10,000 year
9	Hydro-Electric Handbook Creager & Justin, 1927	Basic Stage		445	470	570
		Yearly Flood		435	460	525
10	Water Supply Paper 771	Foster type I	1875-1931	371	378	390
		III	"	405	421	476
		Hazen	"	405	424	480
11	"	Goodrich type VI	"	371	376	400
		II	"	380	396	428
		V	"	361	375	394
12	"	Slade	"	416	428	474
13	H.D. 328 Corps of Engineers	Goodrich	1875-1927	410		
8	TVA Records	Foster	1867-1886	750	850	
			1887-1935	405	450	
			1867-1935	511	596	

Rational Runoff Method

The so-called "rational" method of estimating runoff from rainfall was described and some of its variations were discussed under "Common Methods." This method has been used to a considerable extent by the Tennessee Valley Authority in estimating the maximum flood that might occur on the Tennessee River at Chattanooga. As previously explained, the drainage area is divided into a series of zones so that rain falling any place within a zone will flow to the lower end of that zone within a unit of time. The basic equation $Q = C\ i\ A$ is applied to each zone for each unit of time to obtain a hydrograph of discharge from each zone which can be added to similar hydrographs from other zones with proper time of travel allowance to obtain the hydrograph at any point on the stream. Some adjustment must be made in the resulting hydrograph since the method does not consider the flattening effect of channel storage upon flood waves, or else an elaborate routing process must be arranged to transfer the flow down the principal stream from zone to zone or at least between principal tributaries. Attempting to route the flows down the Tennessee River and its principal tributaries is both laborious and of uncertain accuracy at the present stage of development along this line. Therefore, crest discharges determined by the rational method were reduced for the effect of channel storage. The rational method furnishes the volume of runoff and the shape of hydrograph, while the previously discussed diagram of maximum observed runoff rates furnishes the maximum expected flow.

The unit of time to be used in dividing the basin into zones was first taken as 24 hours, most of the available rainfall figures being for 24-hour periods. Later studies, however, indicated that the resulting hydrographs are not well enough defined, due to an insufficient number of points, and 6-hour zones were substituted. The flood of December, 1932, furnished the basic data for computing time of travel, as this flood was of sufficient magnitude to give reasonably high velocities and occurred recently enough to have its profile and discharges rather well defined. Slopes and depths were picked from the profile, and velocities were computed by means of the Manning equation for open channel flow. The river system was divided into reaches sufficiently short so that points at 6-hour intervals of travel along the river system above Chattanooga could be interpolated. These 6-hour travel distances were continued up the Tennessee River, its principal tributaries, and all the minor tributaries on which the necessary data was available. Six-hour travel distances were estimated over the remaining distance to the basin boundary. The points determined in this manner define lines similar to contours except that they represent time above Chattanooga rather than feet above a datum plane. Exhibit 15 shows the resulting 6-hour time of travel zones above Chattanooga.

The maximum flood to be expected at Chattanooga will be produced by a storm having a duration about equal to the concentration period of the drainage area, or about five days. A longer storm would have a somewhat less intensity, while a shorter storm would not cause all the drainage area to contribute to the flood crest. No attempt was made to work out the maximum possible rainfall to be expected over the drainage area, but great storms of the past were transposed over the basin and the resulting hydrographs computed. This required a detailed study of past storms over the Tennessee River basin and adjacent areas to determine what types of storms might be expected to occur in this basin and the magnitude and probable season of occurrence of these storms.

The seasonal distribution of floods at Chattanooga as pictured on Exhibit 4 indicates that the Chattanooga flood season extends from about the middle of December to the Middle of April, the greatest floods occurring in March and the first week of April. This seasonal distribution is explained to a certain extent and supported by the basic meteorological data shown on Exhibit 16. It is observed that the average monthly rainfall as determined from a large number of representative stations in the drainage area above Chattanooga having long periods of record is greatest in March but nearly as great in December, July, and August, there being no great or well-defined seasonal variation in rainfall. The similar curve showing average runoff at Chattanooga is high in January and February but reaches a definite maximum in March, dropping to less than a third of this amount during the summer months. The curve of average percent runoff is still more striking with a maximum of about 70 percent during the winter months of January, February, March, and April, and a minimum of slightly over 25 percent during the summer months of July, August, September, and October.

This low percent runoff during the summer months as compared to the winter months appears logical when the other curves on this chart are considered. The day is seen to be over 50 percent longer (sunrise to sunset) in June than at the first of the year, the curve of average temperature being very similar with a maximum in July. Long days and high temperatures promote drying of the soil which then requires greater rainfall to produce saturation and high rates of runoff.

For the present purpose, storms may be grouped into three classes: winter storms of the cyclonic type, West Indian hurricanes, and summer thundershowers or cloudbursts. The last named storms are accompanied by very intense precipitation but the duration is short and the area covered small so that this type of storm could not possibly produce the maximum flood to be expected from a large drainage area. Isohyetal maps of typical storms of this type which have occurred in the Tennessee Valley region are shown on E xhibit 17. It is noted that the very intense rainfall areas cover only a few square miles while each storm covers a total area less than a hundred square miles.

West Indian hurricanes are tropical storms which originate near the equator in the vicinity of the West Indies to travel northward through the Gulf of Mexico and across the southern states, generally curving towards the Atlantic Ocean and seldom coming very far inland. They produce intense precipitation over short periods of time and over relatively large areas, but generally occur in the late summer months when runoff is a small percentage of rainfall. Storms of this type are likely to produce maximum floods in the areas subject to such storms. There is considerable doubt, however, that such a storm could travel this far inland and cross the mountains surrounding the Tennessee River basin and still retain sufficient moisture to produce a maximum flood on a large drainage area. The storm of July 14-16, 1916, which produced the greatest flood of record on the upper portions of the French Broad River was of this type. The maximum rainfall was recorded at two points in North Carolina on the eastern boundary of the Tennessee River basin, these points being on opposite sides of a mountain gap about midway between Grandfather Mountain with an elevation of 5964 and Mount Mitchell with an elevation of 6711. Altapass on the east slope recorded a rainfall of 23.77 inches in a period a little over 24 hours, while Altapass Inn on the west slope had 22.25 inches. Rainfall stations were too few in this area to give a complete picture of the intensity and duration of this storm, but an isohyetal map was prepared by the Miami Conservancy District in connection with their storm studies and is reproduced along with other storms on Exhibit 23. The studies of the Tennessee Valley Authority indicate that a somewhat different rainfall pattern can be deduced from the same rainfall data; the results are shown on Exhibit 18. Consideration is here given to the effect of topography on rainfall, resulting in a greatly reduced volume of rainfall within the Tennessee River basin. Gaging stations in operation at this time were few and discharge estimates none too accurate, but runoff studies on the French Broad River indicate that most of the rain must have fallen on the southeast side of the mountains rather than in the Tennessee River basin on the northwest side of the mountains. Exhibit 19 shows the paths of low pressure areas accompanying great hurricanes which have approached or crossed the Tennessee River basin while shaded strips indicate the areas of greatest rainfall.

The most important storms as far as Chattanooga is concerned are those of the cyclonic type which travel across this country during the winter season in a generally west to east or southwest to northeast direction. Storms of this type have produced most of the great floods of record in eastern United States as shown under "Maximum Observed Rates of Runoff" and are likely to produce the maximum floods to be expected on the Tennessee River. Exhibit 20 shows the paths followed by a number of these storms passing near the Tennessee River basin while Exhibit 21 shows low pressure movements and corresponding rainfall areas of a number of cyclonic storms which have produced heavy rains in the Tennessee River basin. Exhibits 22, 23, and 24 show isohyetal maps of the great storms of eastern United States, including the storm of July 14-16, 1916, as well as the storms of the cyclonic type. In order to get a better picture of the comparative intensity of the various great storms occurring over portions of eastern United States subject to meteorological conditions similar to those of the Tennessee Valley, a series of time-area-depth curves are plotted on Exhibit 25; the storms are separated into those having a duration of 2 days, of 3 and 4 days, and of 5 days or over.

Storms having a duration of about 5 days are of greatest importance in the present study since the drainage area above Chattanooga has a concentration time of about this length, but it is well to consider also storms of slightly greater or less duration. Examining the time-area-depth curves for 3 and 4-day periods, it is noted that for the drainage area above Chattanooga (21,400 square miles), the 4-day storm of January 21-24, 1937 with an average rainfall of about 10 inches is the most severe; the 3-day storm of October 4-6, 1910 is less intense by only half an inch. The 1937 storm covered a portion of the Tennessee River basin and centered so close to the basin that there is little question but that a repetition of this storm might reasonably be expected in this area. Further, this storm occurred during the early part of the flood season at Chattanooga, while the 1910 storm occurred outside the Chattanooga flood season, even though it also occurred within a reasonable distance of the Tennessee River basin.

Examining the chart showing time-area-depth curves for periods of 5 days and longer, it is seen that for a drainage area the size of that above Chattanooga, the 5-day storm of July 6-10, 1916, with an average rainfall of about 15 inches, is far above all others. Exhibit 19, however, shows this storm to be a West Indian hurricane which moved northward from the Gulf of Mexico to drop most of its moisture before crossing into the Tennessee River basin. The 5-day storm of November 17-21, 1906, and the 6-day storm of January 20-25, 1937, appear next on the chart, each with an average rainfall of about 11.8 inches over an area the size of that above Chattanooga. Here again, the 1937 storm was so located and occurred at such a time that its repetition over the area above Chattanooga during the flood season appears reasonable. The 1906 storm occurred at some distance from the Tennessee River basin and outside the Chattanooga flood season so that there may be some doubt as to the possibility of its recurrence over the Tennessee Valley.

Thus, in considering all the past storms which have visited eastern United States, the 1937 storm appears to be the most severe that might reasonably be expected to recur over the Tennessee River basin above Chattanooga at a time of the year favorable to producing a great flood. It is also important to consider that this storm did produce the greatest flood of record on the Lower Ohio River and that the occurrence was recent enough so that a large mass of relatively accurate rainfall and runoff data is available for further study.

The 5-day storm of March 23-27, 1913, falls next on the chart with an average intensity of about 9 inches over 21,400 square miles. It occurred at the very height of the Chattanooga flood season and covered an area not far removed from the Tennessee River basin. It produced the maximum flood of record on the Miami River and a flood which equalled the 1884 flood on the Lower Ohio River, the then maximum flood of record although it has since been exceeded by that of January, 1937. The great damage caused by this flood to industrial cities in Ohio led to the formation of the Miami Conservancy District and the construction of a system of flood control reservoirs. Thus, this storm also produced a great flood on a nearby drainage area and has been studied in detail by engineers of the Miami Conservancy District so that a large amount of authentic information on both rainfall and runoff is available.

In the application of the rational runoff method, the percentage of storm rainfall which runs off as flood flow must be estimated. Since the maximum flood must occur in the winter season when evaporation and transpiration are very low and is likely to be preceded by rains which will leave the ground saturated, the percentage of rainfall that finds its way immediately to the streams is likely to be large. Exhibit 26 contains hydrographs of the principal floods at Chattanooga with mass curves for each showing the cumulative rainfall and runoff, runoff here being storm runoff or the difference between the recorded flows and an estimated base flow or ground water flow. The total runoff for each storm is seen to be a large proportion of the storm rainfall. The following tabulation shows the total rainfall and runoff of large floods at Chattanooga and the percent of the storm rainfall which appeared as flood runoff:

Storm	Rainfall		Runoff		Percent
	Duration Days	Inches (Total)	Duration Days	Inches (Total)	Runoff
Feb.-Mar. 1875	14	11.4	32	7.6	67
Mar.-Apr. 1886	22	9.9	31	6.9	70
Feb.-Mar. 1917	10	5.5	30	3.8	70
Mar.-Apr. 1920	16	6.0	30	3.7	62
Dec.-Jan. 1927	11	6.0	30	3.8	63
Mar.-Apr. 1929	9	3.1	30	2.3	74
Mar.-Apr. 1936	29	11.2	45	8.0	71

These percentages appear rather low when compared with those given in the following table extracted from "Surface Waters of Tennessee," Bulletin 40 of the Division of Geology, State of Tennessee, by Warren R. King, District Engineer of the U. S. Geological Survey:

River	Location	Drainage area (sq.mi.)	Average Rainfall (inches)	Total Rainfall (acre-ft.)	Total Runoff (acre-ft.)	Percent Runoff
Cumberland	Barbourville	962	4.55	238,000	226,000	95
Cumberland	Cum. Falls	2,010	4.9	525,000	505,000	96
Cumberland	Burnside	4,890	5.0	1,300,000	1,190,000	91.5
Cumberland	Celina	7,320	4.56	1,780,000	2,315,000	*130
Cumberland	Carthage	10,700	5.23	3,000,000	3,544,000	*118
Cumberland	Nashville	12,800	5.12	3,480,000	3,840,000	*110
Rockcastle	Rockcastle Spg.	746	4.2	162,000	150,000	93
New River	New River	312	8.0	133,000	125,000	94.5
S.Fork Cu.	Nevelsville	1,260	6.06	408,000	386,000	94.6
Collins	McMinnville	624	7.50	249,000	226,000	91
Obey	Byrdstown	416	5.28	117,000	114,000	97.5
Stones	Smyrna	552	5.90	174,000	165,000	95
Caney Fork	Rock Island	1,640	8.45	740,000	694,000	93.7
Caney Fork	Silver Point	2,100	8.39	938,000	912,000	97.3
Emery	Harriman	793	9.00	324,000	307,000	94.8

*Rainfall figures for Celina, Carthage and Nashville do not include ¾-inch of rain March 26-27 and 1.30 inches of rain March 30, but runoffs from these rains are included in runoff figures.

This tabulation refers to the floods of March 23-31, 1929, on the Cumberland and Tennessee River basins. The storm producing these floods centered over the Cumberland Mountains which form the divide between these two watersheds. Maximum floods of record were produced on some of these streams, but the storm did not cover a large enough area on either the Tennessee or Cumberland Rivers to produce maximum floods on these streams. On the basis of these figures, it would appear that 90 percent of the rain falling during a storm period might appear as runoff in the resulting flood period if the storm occurred in the winter season and was preceded by minor storms which left the ground well saturated.

The working data necessary for computing flood runoff from rainfall by the rational method has been assembled, and this data must now be applied to the computation of the maximum flood to be expected at Chattanooga. The storm of March 22-27, 1913, was transposed to the

Tennessee River basin above Chattanooga, and a hydrograph was computed by the rational method for an assumed runoff of 90 percent. Several locations of this storm over the drainage area were investigated in order to determine the one producing the greatest discharge at Chattanooga. In every case the storm was transposed without changing the direction of its axis or the shape of the isohyetal lines—that is, the storm was not rotated nor were the isohyetal lines shifted in order to make the storm fit the shape of the drainage area more closely. The position finally chosen is given by the solid isohyetals on Exhibit 27.

Previous studies have showed, however, that the 1913 storm had an average rainfall of only 9 inches over a drainage area equivalent to that above Chattanooga, while the 1937 storm averaged 11.8 inches over a similar area. The duration of the 1913 storm is generally given as five days, or about equal to the minimum estimate for the drainage area above Chattanooga; various longer periods of rainfall may be chosen for the 1937 storm. It is therefore reasonable to suppose that the 1913 storm is not quite the maximum to be expected over a drainage area of this size and that it might be preceded or followed by a secondary storm which would bring the total rainfall up to an amount comparable to that of the 1937 storm. Accordingly, a hypothetical storm of maximum intensity of 2.5 inches was assumed to occur on March 29 and to be so located that it would contribute to the crest of the flood at Chattanooga. The isohyetals of this storm are shown as dashed lines on Exhibit 27. With this added rainfall and a runoff of 90 percent, the rational method resulted in a flood at Chattanooga having a crest discharge of 850,000 cfs, a total volume of 5,340,000 day-second-feet, and a duration of 12 days.

The reduction in crest discharge caused by channel storage has not been considered up to this point. If this is assumed to amount to 14 percent of the crest discharge, the crest is reduced to the 750,000 cfs given by the diagram of Maximum Observed Runoff Rates. A crest reduction of this amount due to channel storage may not be unreasonable, although it appears somewhat small, and the resulting discharge is well supported by points on the diagram of observed rates of runoff. A new hydrograph having this crest discharge was sketched from the computed hydrograph, the volume and duration being made the same and the shape similar to the computed hydrograph. Care was exercised to make the crest of the new hydrograph occur on the falling side of the computed hydrograph so that the area between the two hydrographs (representing the volume of water held in channel storage) is a maximum at the time of the crest of this adjusted hydrograph. The final hydrograph is reproduced on Exhibit 28.

The storm of January 12-25, 1937, was also transposed to several positions over the drainage area above Chattanooga, and the resulting hydrographs were computed by the rational method with a runoff factor of 90 percent. The position which results in the greatest discharge is shown on Exhibit 29. The computed discharge of 930,000 cfs was considered to be reduced to the previously fixed 750,000 cfs for the effect of channel storage, and the hydrograph shown on Exhibit 30 was drawn to have a similar shape and the same volume and duration.

Maximum Flood Hydrograph

In order to bring together for comparison various possible flood hydrographs at Chattanooga, a mass-duration curve was prepared for each and plotted as shown on Exhibit 31, the daily flows being summed in order of magnitude downward from the crest of each hydrograph. Mass-duration curves were first computed for the floods produced by the transposed storms of 1913 and 1937. It is noted that these curves are coincident for the first three days, the mass of the 1913 flood being considerably smaller than that of the 1937 flood for longer durations. This appears reasonable inasmuch as the 1913 storm was much shorter than the 1937 storm and contained considerably less volume when the entire 1937 storm period is considered.

Mass-duration curves were added for the four great floods of record at Chattanooga. These curves fall far below those of the 1913 and 1937 storms, inasmuch as the crest discharges were much smaller even though the durations were about the same. The daily flows of the great floods at Chattanooga were then increased in the ratio of the estimated maximum expected discharge (730,000 cfs) to the actual discharge of each of these floods. This is merely an application of the basic principle of the unit graph: a storm of given duration over a drainage area produces a flood hydrograph of definite length regardless of the magnitude of the storm. The added assumption here is that great floods are likely to be caused by great storms whose durations (duration of the principal rainfall period) are approximately equal to the time of concentration of the drainage area. The duration curves of these enlarged floods all fall below that of the 1937 transposed, the enlarged 1875 and 1886 crossing the 1937 at a duration of 11 days and continuing to rise somewhat above it. The mass-duration curve of the flood resulting from the 1937 transposed storm thus forms an enveloping curve which contains the mass-duration curves of all the other floods over a ten-day period surrounding the day of maximum discharge, the enveloping curve being exceeded on the 11th day only because base flow was not taken out of the actual flood hydrographs and so was increased by the same ratio as the crest flow to give unusually great volumes for long durations.

The picture presented by the mass-duration curves is rather remarkable. The crest portions of the hydrographs produced by the 1913 and 1937 storms transposed to the Tennessee River basin are very similar in shape, although the 1937 hydrograph is broader--that is, the length is greater at low discharges. The mass-duration curves of the four great floods of record at Chattanooga fall generally between these two curves when the daily flows of these floods are increased to bring their crest discharges up to the estimated maximum discharge of 730,000 cfs. The mass-duration curve of the 1937 storm can be considered as the enveloping curve of all these individual curves.

To summarize, it has been shown that the storm producing the flood of January, 1937, on the lower Ohio and Mississippi Rivers is the greatest storm of record that has occurred under conditions such that its repetition over the Tennessee Valley above Chattanooga at a season favorable to producing a great flood at Chattanooga is a reasonable possibility. This storm has been transposed to the drainage area above Chattanooga in the position which produces the greatest possible flood at Chattanooga, and the resulting hydrograph has been computed by the rational method. The crest discharge has been decreased for the effect of channel storage to 730,000 cfs, the maximum discharge indicated for this drainage area by the diagram of Maximum Observed Runoff Rates, and a new hydrograph has been estimated which would have the same volume and duration as the computed hydrograph, a similar shape, and the proper treatment of channel storage. A similar hydrograph has been prepared for the 1913 storm transposed to the basin with an added secondary storm. Mass-duration curves were prepared for the floods produced by these transposed storms and for the four greatest floods of record at Chattanooga with their daily discharges increased in the ratio of the maximum expected flood to the actual floods. The mass-duration curve of the flood produced by the 1937 storm transposed forms an enveloping curve for the other curves. It is thus concluded that the hydrograph produced by the 1937 storm transposed to the valley above Chattanooga is a reasonable determination of the maximum flood to be expected under natural conditions at this point, and Exhibit 32 reproduces this hydrograph under the title "Hydrograph, Maximum Assumed Flood, Tennessee River at Chattanooga, Tennessee."

MAXIMUM FLOOD--HIWASSEE RIVER AT HIWASSEE DAM SITE

Introduction

Hiwassee Dam is the second major tributary project to be undertaken by the Tennessee Valley Authority. It is located on the Hiwassee River 75.8 miles above its mouth and about 113 river miles above the city of Chattanooga. A knowledge of the maximum flood to be expected at this point is of importance in planning the entire project and in determining the design flood on which the size of the spillway must be based. The methods which may be used in arriving at a reasonable estimate of the hydrograph of this flood are similar to those discussed in connection with the determination of the maximum flood on the Tennessee River at Chattanooga and so need not be repeated in detail. However, the drainage area is much smaller, the topography much more rugged, and the available rainfall and streamflow data less abundant and probably less dependable than at Chattanooga, so that the two situations are not altogether similar.

Description of the Basin

The Hiwassee River is the first major tributary to enter the Tennessee River above Chattanooga, and it is therefore important that one or more storage reservoirs be constructed on this stream to assist in controlling floods and that these structures be made capable of withstanding any floods which may be reasonably expected to occur. The river rises in the mountain ridges along the southern end of the Great Smoky Mountains to flow northwestward to the town of Murphy where it is joined by the Nottely and Valley Rivers to form a fan-shaped drainage basin. Below Murphy, the river continues to flow through a mountainous country until it is joined by the Ocoee River, also rising in the mountains and flowing roughly parallel to and west of the Hiwassee River. Below the mouth of the Ocoee River, the mountain ridges become less frequent and the river flows through a farming country to join the Tennessee River about 37 miles above Chattanooga.

While the lower one-third of the basin is generally rolling and suitable for farming, the upper two-thirds is very rugged with sharp ridges and deep valleys. The river cuts through a number of ridges to form deep canyons which have been long studied as potential dam sites. The ridges forming the basin boundary on the south rise to elevations between 2000 and 5000 feet, while elevations as high as 6000 feet are reached along the eastern divide between this basin and that of the Little Tennessee River.

The gradient of the Hiwassee River is generally steep. The slope averages slightly over a foot permile over the lowest third of its length, reaches a maximum of 500 feet in 10 miles where the river passes through the more rugged country, and averages over 15 feet per mile over 60 miles of its length.

The average annual rainfall over the Hiwassee River basin is about 10 percent greater than that over the Tennessee River basin above Chattanooga, about 57 inches as compared with 52 inches. The average runoff is slightly over 50 percent of the rainfall.

The site of Hiwassee Dam is in the mountainous portion of the basin where the stream profile is steep and the valley narrow. The selection of a site far enough downstream to control a sizeable drainage area and yet not too far downstream to be out of the mountainous section with its narrow and steep valleys and its good dam sites places this dam about 20 miles below Murphy and therefore immediately below the fan-shaped drainage area formed by the junction of the Valley and Nottely Rivers with the Hiwassee River. The drainage area at this point is 977 square miles.

Available Data

A number of streamflow stations have been operated on the Hiwassee River and its principal tributaries by the U. S. Geological Survey. Of particular importance are the gages on the Hiwassee River in the vicinity of Reliance. The drainage area at this point is 1180 square miles, or only 12 percent greater than the drainage area at the dam site. The first gage was installed at this point in 1900, and stage records are continuous to date, although the gage has been moved several times without obtaining a sufficient number of discharge measurements at each location to adequately determine a rating curve and without obtaining overlapping records of sufficient length to determine the stage relation existing between gages at the different locations. A gage was established on the Hiwassee River at Murphy in 1897 and records have been kept to date. The drainage area at this point is only 419 square miles, since the gage is above both the Valley and Nottely Rivers, but the records are of particular value in studying the backwater protection which must be provided through the town of Murphy.

A third gage of importance was established at Charleston in 1898 on the lower portion of the river below the mouth of the Ocoee River, with a drainage area of 2296 square miles. The period of record is not complete, but stages from a staff gage read by the U. S. Weather Bureau in this vicinity are useful in filling out the record. Flood discharges determined from the recorded stages must be used with caution, however, since Charleston is below the steep portion of the river and stages at this point are affected by backwater from the Tennessee River.

Prior to the establishment of a large number of rainfall stations in the Hiwassee River basin by the Tennessee Valley Authority, the station at Murphy was the only one within the drainage area above the dam site. Only six stations were operated at any one time within or close enough to the basin to be of much assistance in estimating storm rainfall over the area. Considering the great variation in elevation throughout the basin and the effect of the mountain ridges upon moisture-laden winds, these stations are far from satisfactory in any storm studies where it is necessary to determine average rainfall over the drainage area. The many additional gages installed by the Tennessee Valley Authority are a great help, but the records from these gages are too short at this time to be of much assistance.

<u>Application of Empirical Equations</u>

Although empirical flood flow equations were found to be of little assistance in estimating the maximum flood to be expected on the Tennessee River at Chattanooga, they should be more successful on the Hiwassee River since the drainage area involved is more within the limits of the data from which these equations were originally developed. The mountainous drainage area above Hiwassee Dam Site, however, might be expected to produce floods somewhat greater than those given by most empirical equations.

Kuichling's equations derived from the Mohawk River give

Rare floods, $Q = \frac{127,000}{M + 370} + 7.4 = 100,000$ cfs

Occasional floods, $Q = \frac{44,000}{M + 170} + 20 = 57,000$ cfs

The Murphy equation derived from northeastern United States gives $Q = \frac{46,790}{M + 320} + 15 = 54,000$ cfs.

The Meyers equation for extreme floods as modified by Jarvis gives $Q = 10,000\ M^{\frac{1}{2}} = 310,000$ cfs.

The Fuller equation with "C" estimated at 70 and "T" taken as 1000 years gives $Q = C\ A^{0.8}\ (1 + 0.8\ \log_{10}\ T) = 58,000$ cfs.

The general width formula of C. R. Pettis with a 1-day rainfall of 7 inches and an average basin width of 14 miles gives a probable 100-year flood of $Q = 480\ P\ W^{1.25} = 91,000$ cfs.

Frequency of Past Floods

Stage and discharge records covering about 40 years are available on the Hiwassee River at Murphy, Reliance, and Charleston. The record at Reliance is of particular interest since this gage is nearest to the dam site in both mileage and drainage area.

Frequency curves were prepared by the Corps of Engineers for the gages at Murphy and Reliance using the Goodrich method, and the results were published in H. D. 328 as previously mentioned. The 500-year flood at each of these gages is plotted along with corresponding floods at other gages within the Tennessee River basin on Exhibit 15, and an average line is drawn to represent the relation between drainage area and the computed 500-year flood. This line passes directly through the point plotted for Reliance and comes very close to the point for Murphy but falls considerably below the points representing long periods of record at Knoxville, Chattanooga, and Florence.

In the frequency studies described in connection with the determination of the maximum flood to be expected at Chattanooga, 20 and 50-year periods selected from the 70-year record of floods at this gage resulted in 500 and 1000-year floods of considerably different magnitudes. The 40-year periods of record on the Hiwassee River can not be expected to produce frequency curves which might be extrapolated to 500 years with any degree of certainty, although they might be very useful in studying the smaller floods that are likely to occur every few years. Further, an examination of the storm rainfall in the Hiwassee River basin indicates that this area has never been subjected to storms as severe as those which have hit surrounding areas in the Tennessee River basin and which might be expected to occur over this drainage basin. Thus, the period of available records is not a fair sample of conditions because it is short and does not contain storms as great as have occurred over adjacent drainage areas.

The uncertainty of the magnitude of the crest discharges of the high floods of record at both Reliance and Charleston must also be considered in attempting frequency computations based on available knowledge of past floods. A discharge of 56,000 cfs has been estimated for the flood of April 2, 1920, at Reliance, the highest flood of record at this gage, while 55,200 cfs is reported for the flood of November, 1906. Both figures are subject to correction in the future as more discharge measurements are made at high flows. The flashy nature of the stream makes flood peaks very short so that it is quite difficult for an engineer to reach the gaging point and make a discharge measurement while a flood is near its crest.

Unit Graph Method

The "unit graph" and its various modifications have been described as one of the methods available to the hydraulician for converting rainfall into runoff, and computation of the flood produced when a maximum storm is transposed over the drainage area is one of the possible applications of this method.

As previously explained, it is often difficult to find floods which were produced by storms of uniform intensity and duration throughout the whole drainage area for use in preparing a unit graph. This proved to be the problem in studying the Hiwassee River basin. Rainfall records prior to the establishment of the Tennessee Valley Authority gages were found to be of little value, since only one station was located within the basin and the great variation in elevations would not permit stations some distance outside the basin to be used in ascertaining the uniformity of the intensity and duration of the rainfall over the drainage area. In the short periods of record available since Tennessee Valley Authority rain gages were established, only a few floods of any size have occurred, and the rainfall producing these floods did not have the required uniformity. Consequently, the unit graphs which were determined from these data did not have a great deal of similarity; variations in peak discharge ranged as high as 50 percent.

An average unit graph constructed from these individual unit graphs could hardly be increased the amount necessary to obtain the maximum flood, nor could the unit graph having the maximum crest discharge be increased in this manner without obtaining results which might be expected to be in error by something like 25 or 50 percent because of the inherent difficulty of obtaining a reliable unit graph.

The hydrographs of these floods and the resulting unit graphs were useful, however, in checking the shape of the hydrographs obtained by the rational method and in drawing the final hydrograph of the maximum flood.

Maximum Observed Rates of Runoff

The maximum recorded runoff at all gages in the Hiwassee River basin is shown on Exhibit 55. The short periods of record at these gages and the apparent lack of any great storms over the basin within this period make it advisable to consider the maximum rates of runoff that have occurred in other basins having similar flood producing characteristics. As previously discussed, Exhibit 7 represents recorded rates of runoff throughout eastern United States, with the exception of areas along the southeastern and southern coasts where heavy runoff is produced by tropical hurricanes. The Hiwassee River basin is well shielded by high mountain ridges so that these tropical storms are not likely to produce heavy rainfall over large areas within the basin. The low percentage of rainfall which appears as runoff during the summer months when storms of this type occur will further tend to produce low runoff rates.

The enveloping line shown on this diagram was recommended by the Consulting Board for general application to drainage areas in the Tennessee River basin of more than 500 square miles. As previously noted, however, the Board recognized that in dealing with particular drainage areas "each problem should be studied in detail with the exercise of judgment in the light of the local conditions" and added further: "In some cases it may be necessary to materially increase or decrease the values so determined in order to allow for special local conditions."

It is noted on the diagram of Maximum Runoff Rates that the storm of March 22-23, 1929, over the Tennessee and Cumberland basins resulted in a point representing a discharge of 193 cfs per square mile from a drainage area of 800 square miles, this point lying somewhat above the enveloping line. This flood occurred on the Emory River at Harriman, a drainage basin similar in shape and topography to that of the Hiwassee River basin above Hiwassee Dam Site and not far distant from this area. This rate of runoff is based largely upon discharge measurement made under difficult conditions and is therefore subject to some doubt. It may be either too high or too low. However, when this high rate of runoff is considered along with the fact that the Hiwassee River basin has steep slopes, no surface storage, a fan-shaped drainage pattern, and a soil and rock cover which is highly impervious to water, it is reasonable to expect this basin to produce floods greater than those occurring on most basins of this size in eastern United States, and the enveloping curve on Exhibit 7 should be raised to include every point rather than just the great majority of points. An enveloping line having the equation $Q = \frac{6000}{\sqrt{A}}$ rather than $Q = \frac{5000}{\sqrt{A}}$ is considered to represent a reasonable estimate of the maximum discharge to be expected from drainage areas in the upper portion of the Hiwassee River basin. This gives a runoff rate of 193 cfs per square mile at the dam site of a discharge of 188,000 cfs.

Rational Runoff Method

The so-called "rational" method of computing flood flow from rainfall may be applied to the Hiwassee River basin in much the same way that it was used in determining the maximum flood on the Tennessee River at Chattanooga. The general principles of this method were discussed previously and need not be repeated.

Time zones were computed at 6-hour intervals over the Tennessee River basin above Chattanooga, and these zones on the Hiwassee River may also be used in studying floods above the Hiwassee Dam Site. As shown on Exhibit 15, there are only three time zones above the dam site, indicating a period of concentration of between 12 and 18 hours. The 6-hour zones are rather large for this small drainage area, but smaller zones would probably give an appearance of refinement which could not be attained with the available data.

Of the three types of storms previously discussed as occurring within the Tennessee River basin, cloudbursts result in intense precipitation over only a few square miles and generally spread over less than a hundred square miles so that they need not be considered as possible producers of a maximum flood at the dam site with its drainage area of nearly a thousand square miles. Both cyclonic storms and West Indian hurricanes must be considered.

The annual and seasonal distribution of floods at the Reliance gage of the U. S. Geological Survey is pictured on Exhibit 34. The record is complete from 1901 to 1937, with the exception of the period from 1914 through 1918 when the gage was located at Apalachia, 16 miles upstream, and stages could not be related to the present gage with reasonable accuracy for comparison with other floods. The seasonal distribution of floods is seen to be similar to that of the Tennessee River at Chattanooga as pictured on Exhibit 4, although the flood season appears to be somewhat longer, starting a little earlier in the fall and continuing a little later in the spring. It is noted also that the maximum flood shown occurred on November 19, or very early in the flood season, while the second highest flood occurred near the end of the flood season, on April 2. There are only a few summer floods and none of any magnitude. This diagram suggests, therefore, that winter storms of the cyclonic type are the principal producers of floods at Reliance just as at Chattanooga but does not prove that great floods might not be produced by summer storms of the West Indian hurricane type.

An examination of the tabulation of the maximum floods of record at the various gaging stations in the Hiwassee River basin as given on Exhibit 33 shows that the West Indian hurricane of July 6-10, 1916, produced the maximum flood of record on the Ocoee River at both Emf and Parksville, even though all the maxima on the Hiwassee River itself were produced by other storms. The path followed by this hurricane is pictured on Exhibit 19. Here it is noted that this storm had an intensity of about 15 inches just before it entered the Tennessee River basin, dropping to only 6 inches in Tennessee. It was explained previously that the West Indian hurricane of July 14-16, 1916, also resulted in the greatest rainfall just outside the Tennessee River basin, although the rain which fell upon the upper portion of the French Broad River was great enough to produce the maximum flood of record at many points on this stream.

An examination of the contours on Exhibit 1 discloses a gap in the divide along the southern edge of the Ocoee River basin; the elevation drops below 2000 feet for a short distance along this divide, whereas the divide generally ranges from 3000 to 5000 feet above sea level. There is some possibility that a West Indian hurricane might move up through this gap to cause excessive precipitation in the valley, but the narrowness of the gap and the high and rugged ridges surrounding it would doubtless prevent such a storm from covering an area the size of that above the dam site with rainfall of sufficient intensity and duration to produce the maximum flood at this point.

It must also be considered that West Indian hurricanes occur in the summer and early fall months when the percentage of rainfall that appears as runoff is low, even in the case of intense storms of short duration. To produce the same runoff, a West Indian hurricane must therefore have an intensity almost twice as great as that of a winter storm, and it is not reasonable to expect a hurricane of this intensity to enter the Hiwassee River basin and to cover an area the size of that above the dam site.

Storms of the cyclonic type are the remaining class to be considered as being capable of producing the maximum flood at the dam site. As shown on Exhibit 20, storms of this type seem to follow up the basins of the Ohio, Cumberland, and Tennessee Rivers, and it is reasonable to expect that one may center over almost any portion of the Tennessee River basin.

To be applicable to the present case, such a storm must have a duration of less than one day, as the time of concentration for this drainage area is between 12 and 18 hours. The storm of March 22-23, 1929, previously referred to, which centered over the divide between the Cumberland and Tennessee River basins is a good example. This storm produced the maximum flood of record on the Emory River, a tributary of the Tennessee River system, and on various tributaries of the Cumberland River system, causing property damage approaching $5,000,000 and drowning at least 22 persons.

A considerable amount of data was collected on this flood and the storm which produced it by W. H. King of the U. S. Geological Survey. No recording rain gages were in operation within the storm area, but immediate field investigations were undertaken and the observations of the regular Weather Bureau stations supplemented by the measurement of rain collected in cans and tanks. The storm began on the morning of March 22 and continued until the next morning, but local observers reported that most of the rain fell within 18 hours and about half of it fell in something like two hours.

The isohyetals of this storm are drawn on Exhibit 23, and the computed time-area-depth curve is shown on Exhibit 25. It is noted that the time-area-depth curve for this storm falls very close to curves for the storms of May 31-June 1, 1889, and November 3-4, 1927, and is exceeded only by that of October 3-4, 1869, the maximum rainfall being about 11 inches while nearly 9 inches was averaged over an area of a thousand square miles.

The storm of March 22-23, 1929, appears to be a reasonable storm to transpose to the Hiwassee River basin to produce the maximum flood. The time-area-depth curves show it to have been exceeded only by the storm of October 3-4, 1869, which occurred outside of the flood season. The flood discharges caused by this storm plot very high on the diagram of maximum runoff rates. The storm had about the proper duration, and it occurred on a similar drainage area located very close to the Hiwassee River basin. Further, a considerable amount of reliable data has been collected on this storm and is available for study and use.

Since no recording rain gages were in operation within the area covered by this storm, some assumption has to be made as to the time of occurrence of the rainfall before the storm can be applied to the Hiwassee River basin. Based on the available information regarding the storm, it is assumed that all of the rain fell in a 24-hour period: 20 percent within the first six hours, 40 percent in the second six, 30 percent in the third, and the remaining 10 percent in the fourth six-hour period.

The percentage of rainfall which is likely to appear as runoff during a major flood on the Hiwassee River cannot be estimated from available records of rainfall and streamflow in this basin. Rainfall stations are too scattered prior to the establishment of a large number of stations by the Tennessee Valley Authority to give an accurate picture of the average rainfall over the drainage area, and a sufficient number of floods of any size have not occurred since these stations were established. The investigations made in connection with the application of the rational runoff method to the Tennessee River at Chattanooga indicated that an average runoff of 90 percent during a flood period is reasonable for that area, and the same figure appears reasonable for the Hiwassee River basin above the dam site.

The storm of March 22-23, 1929, was transposed to the basin of the Hiwassee River above the dam site with the area of maximum rainfall located immediately above the dam site, care being taken not to rotate the axis of the storm in transposing it to this location. The rain was all assumed to occur in a 24-hour period with the distribution previously explained, a runoff factor of 90 percent being applied in computing the flow at the dam site for each 6-hour period. The resulting hydrograph does not contain the effect of channel storage in reducing and broadening flood peaks but gives a reasonable picture of the hydrograph that might be produced by a storm like that of 1929 if it were to occur over the Hiwassee River basin in the position assumed.

Maximum Flood Hydrograph

The crest discharge of the maximum flood hydrograph has been estimated at 188,000 cfs from a study of the maximum rates of runoff observed on streams having similar flood producing characteristics, but it remains to estimate the volume and duration of this flood and to sketch a hydrograph having a reasonable shape.

The hydrograph computed by the rational method for the storm of March 22-23, 1929, transposed to the area above Hiwassee Dam Site may have its crest discharge reduced by channel storage to the 188,000 cfs previously determined without change in duration or total flood volume.

The resulting hydrograph appears generally similar in duration and shape to many of the unit graphs computed for the Hiwassee River at Reliance. This hydrograph, therefore, has a crest discharge determined from maximum floods which have occurred over similar drainage areas on other rivers in eastern United States, a duration equal to that produced by transposing the 1929 storm over the drainage area and a shape similar to the unit graphs at Reliance and the computed hydrograph of the 1929 storm transposed, and so represents a reasonable estimate of the maximum flood.

DETERMINATION OF DESIGN FLOOD

General

In discussing the methods commonly used to estimate necessary spillway capacity, the problem was divided into four parts: Determination of (1) maximum flood hydrograph, (2) project flood, (3) design flood, and (4) planning the spillway.

The various methods commonly employed in estimating the discharge hydrograph of the maximum flood reasonably to be expected at the site under the conditions existing prior to construction of the project or any other water control project within the contributing drainage area were reviewed in a general way and then illustrated by reference to the Tennessee River at Chattanooga and the Hiwassee River at the Hiwassee Dam Site, the first representing a drainage area of 21,400 square miles, while the second represents less than 1000 square miles.

The "project flood" differs from the "maximum flood" in that it is the greatest flood reasonably to be expected at the site under the conditions existing prior to construction of the project and thus includes the effect of any upstream regulation, while the maximum flood is based on natural conditions without any upstream regulation. The project flood may be determined from the maximum flood by computing the effect of upstream storage, or it may be the result of a storm centering over the uncontrolled drainage area between the upstream projects and the point under study. Both possibilities must be investigated. The rational method of estimating runoff from rainfall may be used with hydrographs computed at each of the upstream projects and these routed through the various reservoirs and finally down the river channels to the site being studied.

The "design flood" results from routing the project flood through the proposed reservoir, consideration being given to the effects of natural and controlled storages within the reservoir upon the original flood hydrograph. Since the natural channel storage existing within the length of river covered by the reservoir is displaced by the reservoir storage, the effect of this natural storage must be taken out of the hydrograph of the project flood or it will be considered twice in the routing computations. This requires another routing under natural channel conditions to determine the hydrograph produced by the project flood at the upper end of the reservoir under the natural channel conditions existing prior to construction of the project. In case the next upstream project is immediately above the proposed project, its outflow hydrograph will be the required inflow hydrograph, and in case the natural channel storage is small, the project flood may be taken as the inflow hydrograph without serious error. The hydrograph of the local inflow which is discharged directly into the reservoir is to be added to the inflow hydrograph to obtain the total inflow. This hydrograph is then equal in volume to that of the project flood but has a shorter duration and higher crest discharge since the flattening effect of the displaced channel storage has been eliminated. It is this total inflow hydrograph which must be routed through the reservoir to obtain the design flood.

In routing the total inflow hydrograph through the reservoir to obtain the design flood, the most severe conditions that might be reasonably expected are assumed to exist. The project flood is generally assumed to occur with the reservoir at the highest level at which it can be maintained with all the gates closed. The gates remain closed until the storm has struck and then are opened, at first only sufficiently to hold the pool elevation constant by making outflow equal to inflow and then fully as the seriousness of the flood becomes apparent, in order to lower the pool and provide additional capacity for storing at the time of the flood crest. Some allowance should be made, however, for difficulty and delay in gate operations. It is often assumed that the power plant is closed down so that no flow is discharged through the turbines and that several, or all, of the sluices have become clogged with debris or the gates stuck so that they cannot assist in discharging the flood waters. If there are a large number of gates to be opened, it may also be reasonable to assume that some of these gates are jammed with debris and cannot be operated.

The routing procedure generally followed in determining the effect of a reservoir upon a flood or in computing the design flood from the total inflow hydrograph is a step process using inflows at equal intervals of time and computing outflows at similar intervals, the difference between inflow and outflow over the time interval considered being equal to change in reservoir storage. If inflows at the beginning and end of a time interval "t" are denoted by "I_1" and "I_2" with "Q_1" and "Q_2" representing the corresponding rates of outflow, and "S_1" and "S_2" representing the reservoir storages, the equality may be written:

$$\frac{(I_1 + I_2)t}{2} - \frac{(Q_1 + Q_2)t}{2} = S_2 - S_1$$

If the time interval is taken as 24 hours and the flows expressed in cubic feet per second, the terms expressing average inflow and average outflow will be in day-second-feet, or since the flows are divided by 2, each term represents acre feet. If the storages are also expressed in acre feet, the equality reduces to $I_1 + I_2 - Q_1 - Q_2 = S_2 - S_1$.

In the routing process, the inflows are always known while the storage and outflow at the beginning of each time interval are obtained from the computations for the preceeding interval, and these values are assumed for the beginning of the first interval--that is, the gates are usually considered closed so that there is no outflow at the beginning of the flood period, and the pool is considered at the top of the gates to fix the storage. Thus Q_2 and S_2 are the only unknowns in the equality. When these unknowns are transposed to the right side of the equation, there results: $I_1 + I_2 + S_1 - Q_1 = Q_2 + S_2$.

The storages here considered must be the total amount of water temporarily stored in the reservoir at the instant of time being considered. On a large river where the reservoir is long, the slopes very flat, and the flow through the reservoir relatively large, the storage used in the calculations cannot be that under a level water surface but must be that under a backwater curve for the headwater and flow conditions existing at the time. In the case of a tributary project, the steeper slope and smaller flow produce a smaller amount of backwater over a shorter distance and then only in the upper end of the reservoir where the pool covers the river channel only, thus reducing backwater storage to an amount which can generally be neglected.

If backwater storage can be neglected, both outflow and storage become a function of discharge when the spillway gates are opened, and the right hand side of the equation, or the "outflow-storage factor," may be plotted against headwater elevation or against either outflow or storage. Such a curve enables both outflow and storage to be read directly after their sum has been calculated by adding together the known quantities on the left side of the equation. This gives a direct solution of the storage equation and permits outflows to be computed by successive steps without the cut-and-try computations that would otherwise be necessary to balance the difference between inflow and outflow against change in storage.

If backwater storage is too large to be neglected, numerous backwater curves must be computed for various headwater elevations and various inflows and outflows. Storage then becomes a function of the three variables: headwater elevation, inflow, and outflow. If the reservoir is not too long, backwater storage will be dependent largely upon headwater elevation and inflow, and the effect of outflow may be neglected, thereby eliminating one of the variables and permitting outflow-storage to be plotted against outflow in a family of curves, each curve representing a different inflow. This again gives a direct solution of the equation of storage and permits outflows to be computed in successive steps.

If outflow cannot be neglected in computing backwater storage, the problem becomes one of plotting four variables, and additional families of curves must be prepared. In the case of very long reservoirs, storages become dependent upon flows and elevations at various points along the length of the reservoir and a cut-and-try process must be resorted to in arriving at changes in storage.

The storage equation may be handled by many other methods in routing floods through a reservoir, but the methods are basically the same and the results generally identical.

The outflow hydrograph produced by applying the project flood to the reservoir under the most severe conditions to be reasonably expected has been termed the "design flood." If the elevation to which the pool rises in these computations exceeds that previously fixed as the highest allowable pool elevation, the dimensions of the spillway must be increased by either lowering the crest or increasing its length. New routing curves must be prepared for the modified spillway, and the process must be repeated until the most satisfactory and economical combination of spillway length and gate size is found which will safely pass the design flood.

Further spillway investigations must be made in the fourth step of the process if the reservoir is to be used for flood control and the storage capacity is not sufficiently great to retain the entire flow of a single great flood with the headwater at the beginning of the flood at the elevation at which the project will normally be operated during the flood season.

<u>Hiwassee River at Hiwassee Dam</u>

The hydrograph of the maximum flood reasonably to be expected at the Hiwassee Dam Site has been decided upon, and it remains to determine the project flood and the design flood before finally fixing the size of the spillway.

At the present time there are several hydro-electric developments within the drainage area above the Hiwassee Dam Site, but these developments are all small and their storage capacities negligible under flood conditions. These would, therefore, have little effect upon the maximum flood hydrograph at Hiwassee D am Site, and the project flood is identical with the maximum flood.

The natural channel of the Hiwassee River is very narrow and steep with practically no flood plain along a great portion of its length, particularly in the mountainous section of the valley where the river passes through a series of ridges. This is equally true of the portion of the river which will be flooded by the proposed Hiwassee Dam so that the amount of channel storage which will be displaced by the reservoir is very small. If only flat pool storage is considered in the flood routing computations, the neglected storage increments between flat pool and the backwater curves produced by the flows may be considered to offset the increments of displaced channel storage and the inflow hydrograph will be the same as the hydrograph of the project flood.

It is proposed that flood flows at Hiwassee Dam be passed by a spillway having a clear length of 260 feet and by four sluices having a total capacity of about 20,000 cfs under normal heads. The spillway crest is proposed to be at elevation 1503.5 and the top of gates at elevation 1526.5, the pool being held just below the top of the gates during the summer season but much lower during the winter flood season. The pool elevation is not to exceed 1532 in order to prevent excessive upstream damage. These spillway dimensions provide a discharge capacity of 130,000 cfs, or a total capacity for spillway and sluices of about 150,000 cfs.

A series of routing curves were prepared for these and other proposed spillway and sluice combinations, and the project flood was routed through the reservoir under various assumed conditions to determine the maximum discharge and the pool elevation reached by the design flood. Sample routing curves are shown on Exhibit 38 for a three-hour time interval. A longer time interval would not furnish sufficient points to define the design flood hydrograph with sufficient accuracy. In this diagram, the abscissae represents reservoir outflow in terms of 1000 day-second-feet per 3-hour period of time, while the ordinates are outflow-storage factor, or the sum of outflow and storage. An additional curve is added to show the relation of storage to reservoir elevation.

As previously discussed, the maximum flood is considered to result from a winter storm of the cyclonic type, and the reservoir is to be held at a rather low elevation during the winter season to provide storage capacity for controlling floods of this type. If the sluice gates are all assumed to be closed or the sluices clogged with debris, routing computations indicate that the reservoir elevation might be as high as 1510 (6.5 feet above the top of the spillway) at the time of occurrence of the project flood and still enable the spillway to pass this flood without exceeding the maximum permissible elevation of 1552. Since it is proposed to draw the pool down to an elevation of about 1415 by the beginning of the flood season, the storage between this elevation and elevation 1510 may be considered as a factor of safety.

If the project flood should occur (due to some peculiar and unforseen combination of circumstances) with the pool at the highest elevation at which it can be maintained, elevation 1526, routing computations show that by opening all spillway and sluice gates this flood could be safely passed with a discharge not exceeding 158,000 cfs and a headwater elevation not exceeding 1530.5. With a reasonably reliable forecasting system, the spillway gates might be operated to reduce this crest discharge somewhat, by holding the outflow to 116,000 cfs without allowing the reservoir to rise above elevation 1552.

The proposed spillway has been found to be sufficiently large to pass the discharges resulting from the project flood, but many additional studies must be made in connection with part 4 of the investigation: Planning the spillway. These studies must make certain that the proposed spillway and sluices are the proper combination to provide the greatest possible flood control benefits (at reasonable cost) from the available storage capacity of the reservoir. The combination of sluices and spillway must permit the reservoir to be drawn down rapidly upon the approach of a general flood over the drainage area above the points to be protected and between twin floods that might occur in quick succession. Such studies present many new problems in the choice of twin floods, the possible spacing of these floods, and the proposed schemes of the flood control operation of this reservoir and other reservoirs in the system for the flood control benefits to local areas and to the Ohio and Mississippi Rivers. Such problems are beyond the scope of this discussion.

GENERAL OBSERVATIONS

The methods commonly used in estimating the maximum flood discharge for which the spillway of a water control structure must be designed and the application of these methods to points within the Tennessee River basin serve to demonstrate the danger of a hasty and incomplete investigation of such a problem and point out the need for additional hydrologic data and improved methods of analyzing such data.

Engineers engaged in hydraulic investigations are generally appreciative of the immense amount of damage that can result from water which breaks away from the controls under which it has been placed and agree that all reasonable precautions must be exercised to prevent such an occurrence. The present day tendency towards the construction of reservoirs of all sizes as a means of creating jobs for the unemployed, however, is apt to result in hasty planning of these structures in an attempt to provide immediate employment and in the selection of designing engineers on the basis of political affiliations rather than ability and experience. The studies and discussions here presented demonstrate that necessary spillway capacity cannot be properly estimated from a single method of study nor from the application of handbook equations but must result from the experienced judgment of a competent engineer who has made a thorough study of the hydrological and meteorological characteristics of the particular drainage basin.

Rainfall and streamflow records are the fundamental data of the hydraulician engaged in water control studies, and although the need for a greater quantity and better quality of these data have been expressed by many investigators, it can bear repetition. Many additional rain gages were installed by the Tennessee Valley Authority throughout the

Tennessee River basin as the existing gages were too widely separated to provide a true picture of storm rainfall, particularly in a mountainous area where elevation and exposure influence rainfall to a high degree. In a similar way, our knowledge of storm rainfall in other drainage basins may be increased by the installation of additional gages in those areas where gages are scarce, particularly if the topography is such as to produce a considerable variation in rainfall over short distances. Automatic rain gages are particularly useful as they may be installed in isolated areas with only occasional inspections. An accurate record of rainfall intensities over short periods can thus be obtained as well as the times of beginning and ending of rainfall periods.

Many additional recording stream gages have been installed by the U. S. Geological Survey in recent years, and an ever greater number would be useful. It is important, however, that each gage installation be so arranged that stages will be recorded by the gage over the complete range in water surface elevation that may be reasonably expected. It is sometimes impractical to build the float well of a recording gage of sufficient height to allow the gage to record the maximum flood to be expected, but every effort should be made towards this result.

Stream gages are generally only a means to an end, the results sought being a record of stream discharge. Accordingly, each gage should be rated as accurately as possible and daily discharge measurements (oftener on flashy streams) made during the passage of a great flood. It is often very difficult to make an accurate discharge measurement during a flood, as the cableway or bridge from which the measurements are made may be isolated by the high water, the river filled with floating debris which is apt to damage or carry away the current meter, and the weather cold and disagreeable. An accurate knowledge of flood discharges, however, is of prime importance in any flood control investigations, and the actual measurement of these discharges must be accomplished regardless of the difficulties.

The U. S. Geological Survey is to be particularly commended for the mass of rainfall and streamflow information which it has collected and published on the floods of 1936 in northeastern United States and the flood of 1937 on the Ohio and Mississippi Rivers. This practice should be continued, even though a glance at the volumes of figures already collected may suggest that such a great mass of information has been accumulated that more is unnecessary.

Improved methods of analyzing rainfall and streamflow data are continually being developed. It is well that these be given circulation among engineers to ascertain the advantages and weaknesses of each. The theories of air mass analysis as developed by Norwegian meteorologists appear to be a means of determining the validity of storm transpositions as well as indicating the maximum amount of rainfall that might be expected over an area. The unit graph method of estimating runoff from rainfall presents possibilities for determining crest discharge as well as shape of the maximum flood hydrograph. Methods of routing floods through reservoirs and down natural channels to obtain the effect of changes in surface slopes and storages is subject to a great deal of improvement and simplification. The determination of runoff coefficients from a knowledge of preceding weather conditions and ground water elevations presents interesting possibilities; but these are only a few of the many phases of the determination of necessary spillway capacity in which the existing methods of analyzing the basic data are being improved and new methods originated.

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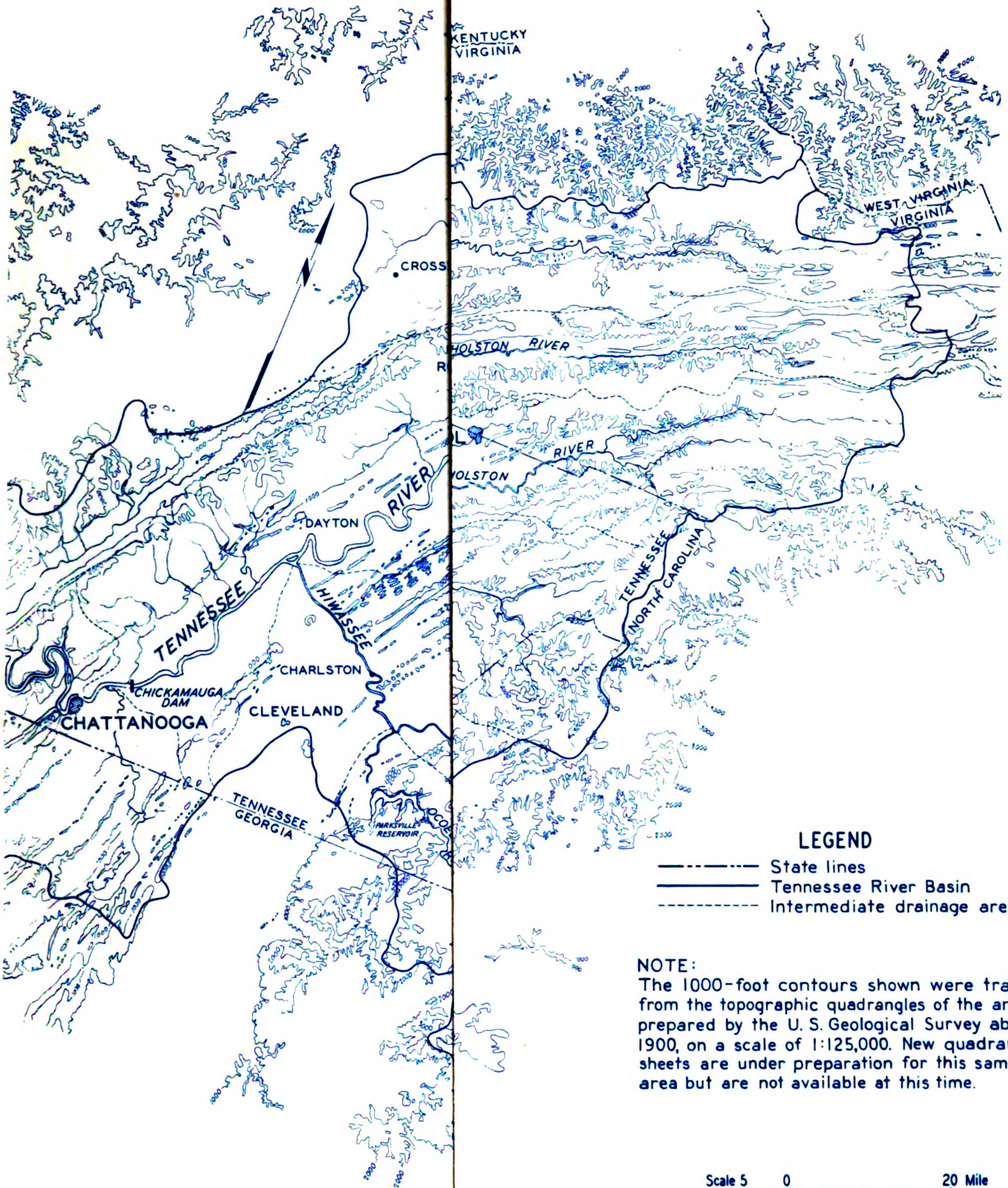
ACKNOWLEDGMENTS

The author wishes to gratefully acknowledge the suggestions and constructive criticisms of Mr. J. H. Kimball, Principal Hydraulic Engineer with the Tennessee Valley Authority; the courtesy of the Tennessee Valley Authority in permitting the use of unpublished data from their files; and the assistance of Mrs. Wylie Bowmaster in editing this thesis.

EXHIBITS



LEGEND

- State lines
- Tennessee River Basin
- Intermediate drainage areas

NOTE:
The 1000-foot contours shown were traced from the topographic quadrangles of the area prepared by the U. S. Geological Survey about 1900, on a scale of 1:125,000. New quadrangle sheets are under preparation for this same area but are not available at this time.

Scale 5 0 20 Mile

TOPOGRAPHIC MAP

UPPER TENNESSEE RIVER BASIN

FLOOD CONTROL INVESTIGATIONS

TENNESSEE VALLEY AUTHORITY
WATER CONTROL PLANNING DEPARTMENT

SUBMITTED	RECOMMENDED	APPROVED
David E. Donley	J. Kimball	

KNOXVILLE	9-27-38	W	PP	0	IK59R

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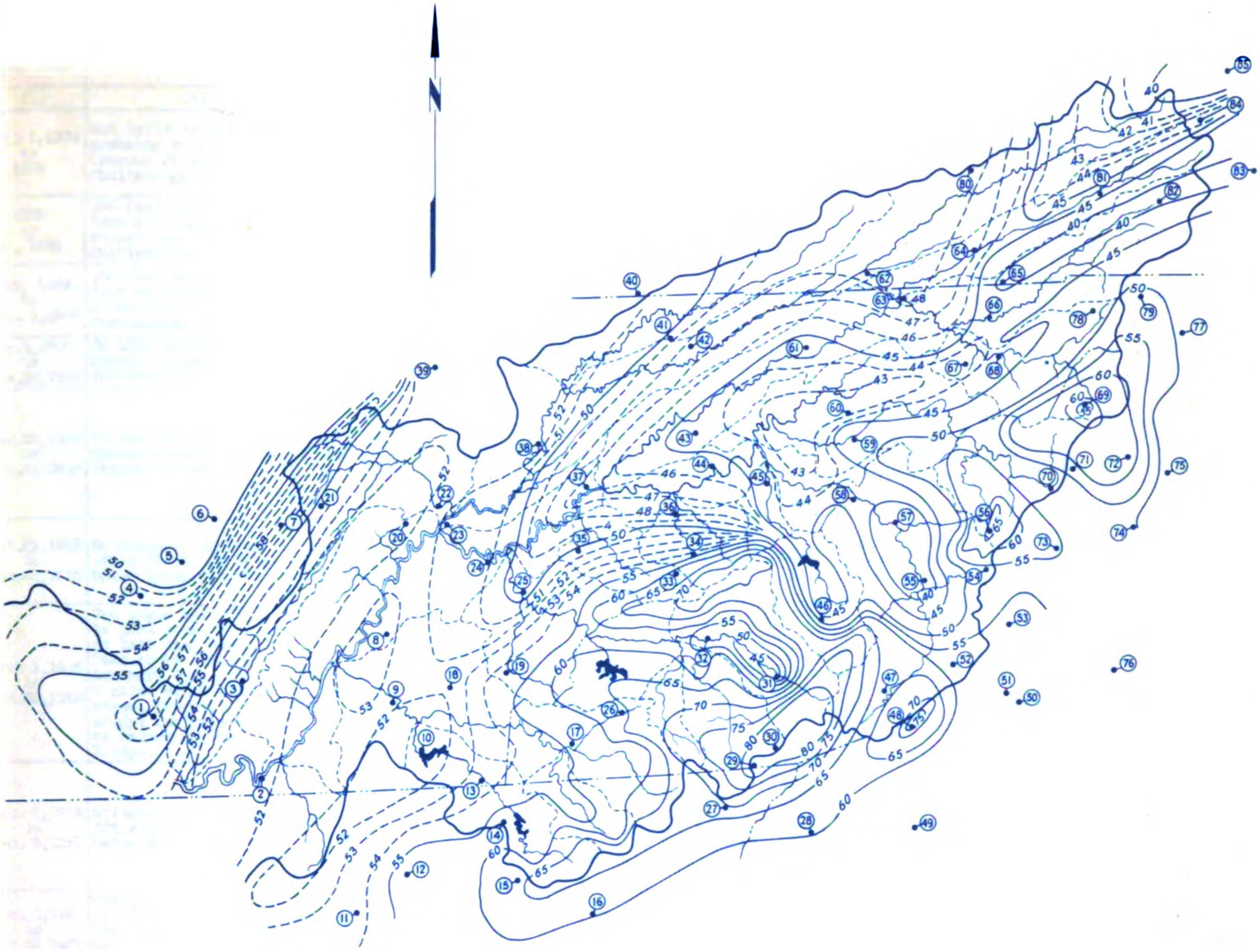
DRWN. L.H.D. COMPUTED
TRCD.
CHKD. ENGINEER David E. Donley



RAINFALL STATIONS

1. Tracy City
2. Chattanooga
3. Dunlap
4. McMinnville
5. Rock Island
6. Sparta
7. Erasmus
8. Decatur
9. Charleston
10. Parksville
11. Resaca
12. Ramhurst
13. Copperhill
14. Blue Ridge
15. Diamond
16. Dahlonega
17. Murphy
18. Etowah
19. Tellico Plains
20. Rockwood
21. Crossville
22. Harriman
23. Kingston
24. Loudon
25. Mc Ghee
26. Andrews
27. Clayton
28. Walhalla
29. Rock House
30. Highlands
31. Cullowhee
32. Bryson
33. Elkmont
34. Gatlinburg
35. Maryville
36. Sevierville
37. Knoxville
38. Clinton
39. New River
40. Middlesboro
41. Tazewell
42. Springdale
43. Jefferson City
44. Dandridge
45. Newport
46. Waynesville
47. Brevard
48. Caesars Head
49. Liberty
50. Landrum
51. Tryon
52. Hendersonville
53. Chimney Rock
54. Montreat
55. Asheville
56. Mt. Mitchell
57. Marshall
58. Hot Springs
59. Birds Bridge
60. Greeneville
61. Rogersville
62. Speers Ferry
63. Kingsport
64. Mendota
65. Bristol
66. Bluff City
67. Johnson City
68. Elizabethton
69. Banners Elk
70. Altapass
71. Linville Falls
72. Gorge
73. Marion, N.C.
74. Morganton
75. Lenoir
76. Caroleen
77. Jefferson
78. Mountain City
79. Parker
80. Dante
81. Saltville
82. Marion, Va.
83. Wytheville
84. Burke's Garden
85. Bluefield

Scale 20 0 20 40 Miles

NOTE:
The isohyetals shown represent the average annual rainfall for the period 1904 to 1933, inclusive, and are based on the actual and computed 30-year values for the stations listed.

MEAN ANNUAL RAINFALL
UPPER TENN RIVER BASIN

FLOOD CONTROL INVESTIGATIONS
TENNESSEE VALLEY AUTHORITY
WATER CONTROL PLANNING DEPARTMENT

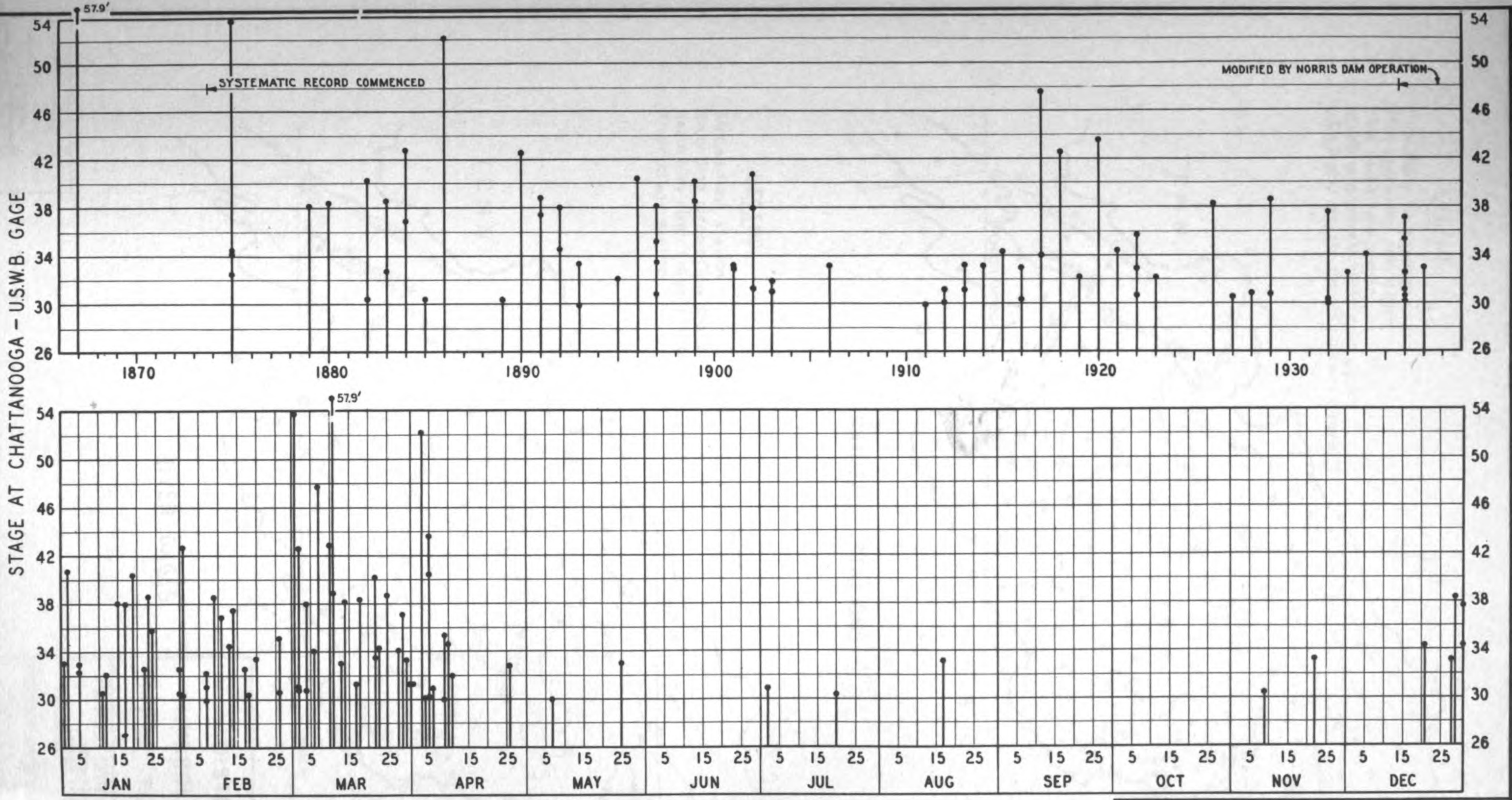
SUBMITTED	RECOMMENDED	APPROVED
David E. Donley	J. L. Kimball	T. B. Parker

KNOXVILLE	9-2-38	W-PR	0	1A37R0

DRWN. H.A.P.	COMPUTED
TRCD.	ENGINEER
CHKD. H.H.B.	W.A. Ammaster

TENNESSEE RIVER GAGES AT AND NEAR CHATTANOOGA, TENNESSEE

Records Available	Location	Drainage Area Sq. Mi.	Gage	Regulation	Channel and Control	Accuracy
(1)	(2)	(3)	(4)	(5)	(6)	(7)
April 1,1874 to 1875	Not definitely known; probably at foot of Lookout Street below Chattanooga Island.	21,400	Established by U.S.Army Engineers and turned over to Signal Service (U.S.Weather Bureau) Dec. 1, 1876.	None	Same as 1902-1913	No discharge measurements until 1891. Gage in charge of U. S. Weather Bureau.
1875 to Oct. 1884	150 feet above the foot of Lookout Street just below Chattanooga Island.	21,400	Wooden gage established after high water of 1875 by U.S.Army Engineers and reset in 1881.	None	Same as 1902-1913	No discharge measurements until 1891. Gage in charge of U.S. Weather Bureau.
Oct. 1884 to Dec. 1,1900	150 feet above the foot of Lookout Street just below Chattanooga Island.	21,400	Inclined T-rail bolted to rock and vertical timber on cliff of left bank.	None	Same as 1902-1913	No discharge measurements until 1891. Gage in charge of U.S. Weather Bureau.
Dec.1,1900 to Dec.30,1902	On pier of Hamilton County highway bridge. (Walnut Street)	21,400	Vertical brass scale bolted to pier.	None	Same as 1902-1913	Not reliable. Projecting base of pier influences accuracy.
Dec.30,1902 to Oct.21,1913	On pier of Hamilton County highway bridge. (Walnut Street)	21,400	Automatic recording gage installed. The gage of 1884 is considered standard.	Construction of Hales Bar Dam (33 miles downstream) began in 1905. In 1910 hydrograph comparisons indicate no backwater effect. Ocoee No.1 completed in 1911.	Bed composed of loose sand,rock,and gravel. Fairly constant. Right bank is high and overflows at flood stages. Hales Bar Dam is control after Oct.22,1913.	Station rating curve has remained practically constant During 1911 automatic gage records questionable and should be used with caution.
Oct.22,1913 to Feb.28,1915	Bridgeport, Alabama (Mile 414.0, 17 miles below Hales Bar Dam)	22,650 at gage	U. S. Weather Bureau gage established in 1896.	Flow during low stages regulated to some extent by the operation of power plant at Hales Bar.	Gravel and Rock shoal about one-half mile below gage. Permanent.	U. S. Weather Bureau readings used. Station not rated for discharge. Discharges published by State Geologist in Bulletin 34.
March 1,1915 to Sept.30,1918	Two gages used. No.1 is 1884 gage. No.2 is on left bank about 100 feet above Cincinnati Southern R.R. bridge, 7 miles above Chattanooga. Hamilton County bridge now referred to as Walnut Street Bridge.	21,400 at Gage No. 1	No.1 is 1884 gage. No.2 is vertical staff gage in three sections fastened to trees on left bank. Both have same datum.	Operation of power plant changes slope of water surface.	Channel practically permanent. Control is Hales Bar Dam and power plant.	Results considered good,although some error in estimates for individual days,especially during low flow. Rating curve well defined between 11,500 and 363,000 cfs. Records fair but means subject to error. Discharge determined by slope method.
Oct. 1,1918 to Jan. 5,1921	Bridgeport, Alabama (Mile 414.0, 17 miles below Hales Bar Dam)	22,650 at gage	U. S. Weather Bureau.	Flow during low stages regulated to some extent by operation of power plant at Hales Bar.	Gravel and rock shoal about one-half mile below gage. Permanent.	Station not rated for discharge. Rating curve constructed by plotting Chattanooga discharge measurements against Bridgeport gage height, allowing one day lag. Monthly discharges within 10%. Daily discharges show greater differences.
Jan. 6,1921 to Sept.30,1921	Two gages used. No.1 is 1884 gage. No.2 installed on the Cincinnati Southern R.R. bridge near right bank.	21,400 at Gage No. 1	No.1 is gage installed during 1884,used with Fulton recorder. No.2 is chain gage set to same datum as No.1	Flow during low stages regulated to some extent by operation of power plant at Hales Bar.	Channel practically permanent. Control is Hales Bar Dam and power plant.	Three-foot flashboards used at Hales Bar Dam during low flows. Rating curve for this condition poorly defined. Records fair for other periods.
Oct. 1,1921 to Sept.30,1923	Four gages used. No.1 and No.2 unchanged. Gage No.3 on lower lock wall on right bank at Hales Bar Dam. Gage No.4 located on upper lock wall on right bank at Hales Bar Dam.	21,400 at Gage No. 1 (21,800 at Gages 3 & 4)	Gage No.3 is a Bristol 7-day recorder. Gage No.4 is a Bristol 24-hour recorder.	Flow during low stages regulated to some extent by operation of power plant at Hales Bar.	Control for No.1,2 and 4 is Hales Bar Dam. Control for Gage No.3 is a rock and gravel shoal one-fourth mile below dam, probably permanent.	Slope method used except during flashboard use at Hales Bar Dam. Gage No.3 used during low flows. Rating curve for No.3 is well defined between 8,000 and 17,000 cfs. Daily discharges corrected for changes in pool level as determined by midnight readings of Gage No.4. Records fair.
Oct. 1,1921 to Jan.16,1925	Locations unchanged.	Un-changed	Gurley 7-day recorder after August 23,1924.	Flow during low stages regulated to some extent by operation of power plant at Hales Bar.	Same as 1921-1923.	Gage heights for Nos.3 and 4 furnished by Tennessee Electric Power Company since they were installed.
Jan.17,1925 to Aug.14,1925	Gage No.4 bolted to retaining wall on left bank just above Hales Bar power house.	Un-changed	Gage No.4 changed to Gurley 24-hour recorder in a corrugated iron stilling well.	Flow during low stages regulated to some extent by operation of power plant at Hales Bar.	Control for Gage No.3 after August 14,1925 formed by Widows Bar Dam, 22 miles below.	Same as Jan.to Sept. 1921.
Aug.15,1925 to Sept.30,1925	(See Column 7) (See accuracy)	21,400	(See Column 7)	(See Column 7)	(See Column 7)	Discharge estimated on basis of discharges of Tennessee River at Loudon; Clinch River at Clinton; Emery River at Deermont; and Hiwassee River at Charleston.
Oct. 1,1925 to Sept.30,1926	Two gages used. No.1 is at same site as inclined and vertical staff gage of 1884. (On left bank 150 feet above Walnut Street bridge) No.2 is on upper lock wall at Hales Bar Dam.	21,400 at Gage No. 1 (21,800 at Gage No. 2)	Gage No.1 staff gage to June 15,1926, after that date an automatic 60-day recorder in timber shelter over 24-inch corrugated iron stilling well. Gage installed June 16,1926. Gage No.2 is a vertical staff gage.	Flow during low stages regulated to some extent by operation of power plant at Hales Bar.	Channel practically permanent. Control is compound. Channel control modified by backwater caused by Hales Bar Dam and Lock,and further affected by water wheels at power house.	Two curves used; one a slope-velocity curve, and the other a stage-area curve. Both curves well defined. Records good.
Oct. 1,1926 to June 30,1930	Locations unchanged.	Un-changed	Gage No. 1 unchanged. Gage No.2 is a water-stage recorder.	Santeetlah completed in 1928. Waterville completed in 1930.	Same as 1925-1926.	Records good. Discharges estimated at various periods.
July 1, 1930 to Feb.12, 1932	Gage just below Hales Bar Dam,6 miles above mouth of Sequatchie River and 33 miles downstream from Chattanooga.	21,800	Water-stage recorder.	Flow regulated to some extent by operation of Hales Bar power plant above gage.	Control is Widows Bar Dam.	Records good.
Feb.13,1932 to Date	Gage at new state highway bridge one-half mile below Hales Bar Lock and Dam,5-1/2 miles above mouth of Sequatchie River and 33-1/2 miles downstream from Chattanooga.	22,000	Water-stage recorder.	Flow regulated to some extent by operation of Hales Bar power plant above gage. Norris Dam affects stream flow after June 10,1935.	Control is Widows Bar Dam.	Records good.



Gage Zero :
Elevation 621.12 (C & GS 1929)
USWB Flood Stage = 30 feet

DRWN K.W.K.	COMPUTED K.W.K.	
TRCD G.L.C.	ENGINEER W.A.B.	
CHKD A.B.		

DISTRIBUTION OF FLOODS TENNESSEE RIVER AT CHATTANOOGA, TENN

FLOOD CONTROL INVESTIGATIONS
TENNESSEE VALLEY AUTHORITY
WATER CONTROL PLANNING DEPARTMENT

SUBMITTED	RECOMMENDED	APPROVED
David E. Donley	J. H. Kimball	T. B. Parker
KNOXVILLE	5-2-38 G GP 0	IA136 R0

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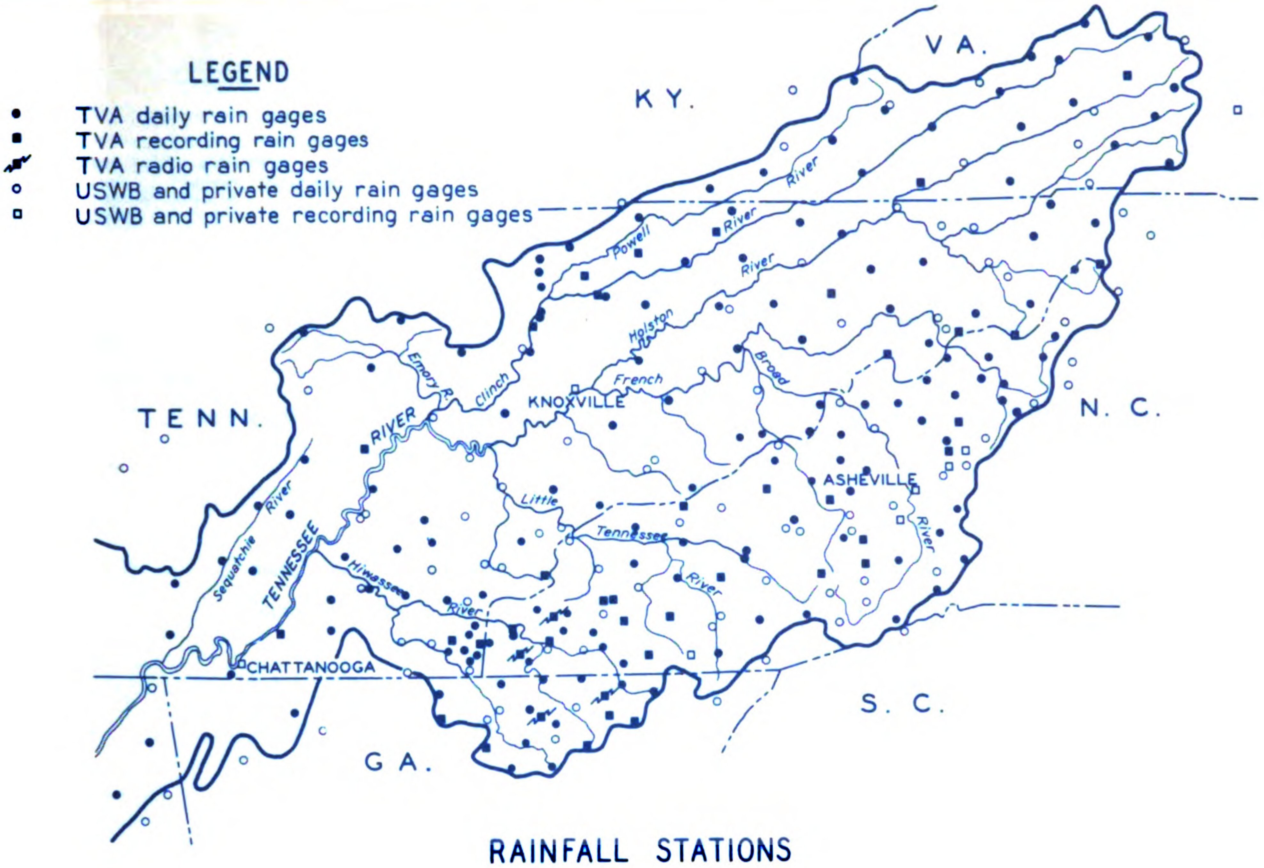
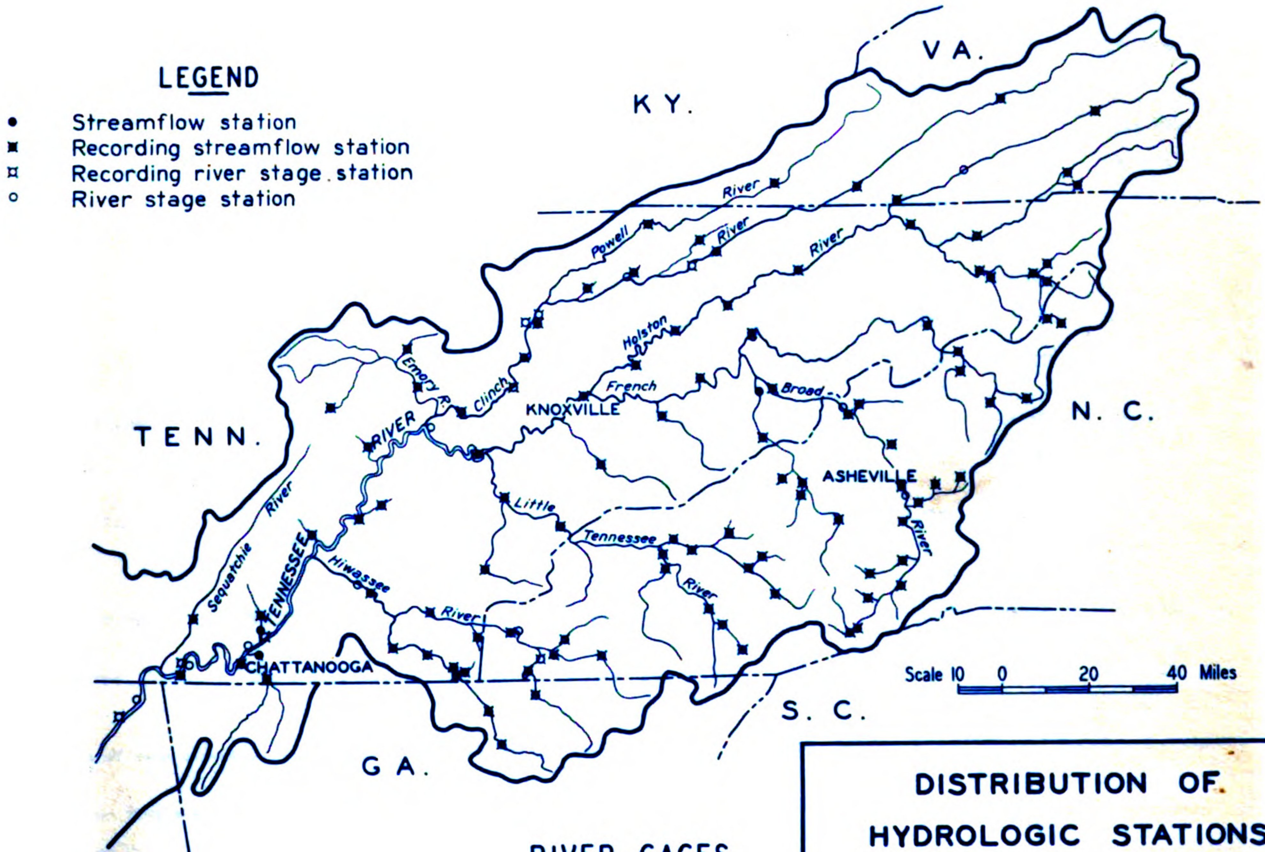


RAINFALL STATIONS



RIVER GAGES

DISTRIBUTION OF HYDROLOGIC STATIONS UPPER TENN. RIVER BASIN

FLOOD CONTROL INVESTIGATIONS
TENNESSEE VALLEY AUTHORITY
WATER CONTROL PLANNING DEPARTMENT

SUBMITTED	RECOMMENDED	APPROVED
David B. Donley	J. H. Kimball	T. B. Parker

KNOXVILLE	10-5-38	W	PP	O	1A70 R0

DRWN. W.A.B.	COMPUTED
TRCD. L.H.D.	ENGINEER
CHKD.	

Tennessee Valley Authority
Water Control Planning Department
Flood Control Section
August 1939

MAXIMUM OBSERVED FLOOD DISCHARGE RATES - SELECTED AREAS IN EASTERN UNITED STATES

OHIO RIVER BASIN

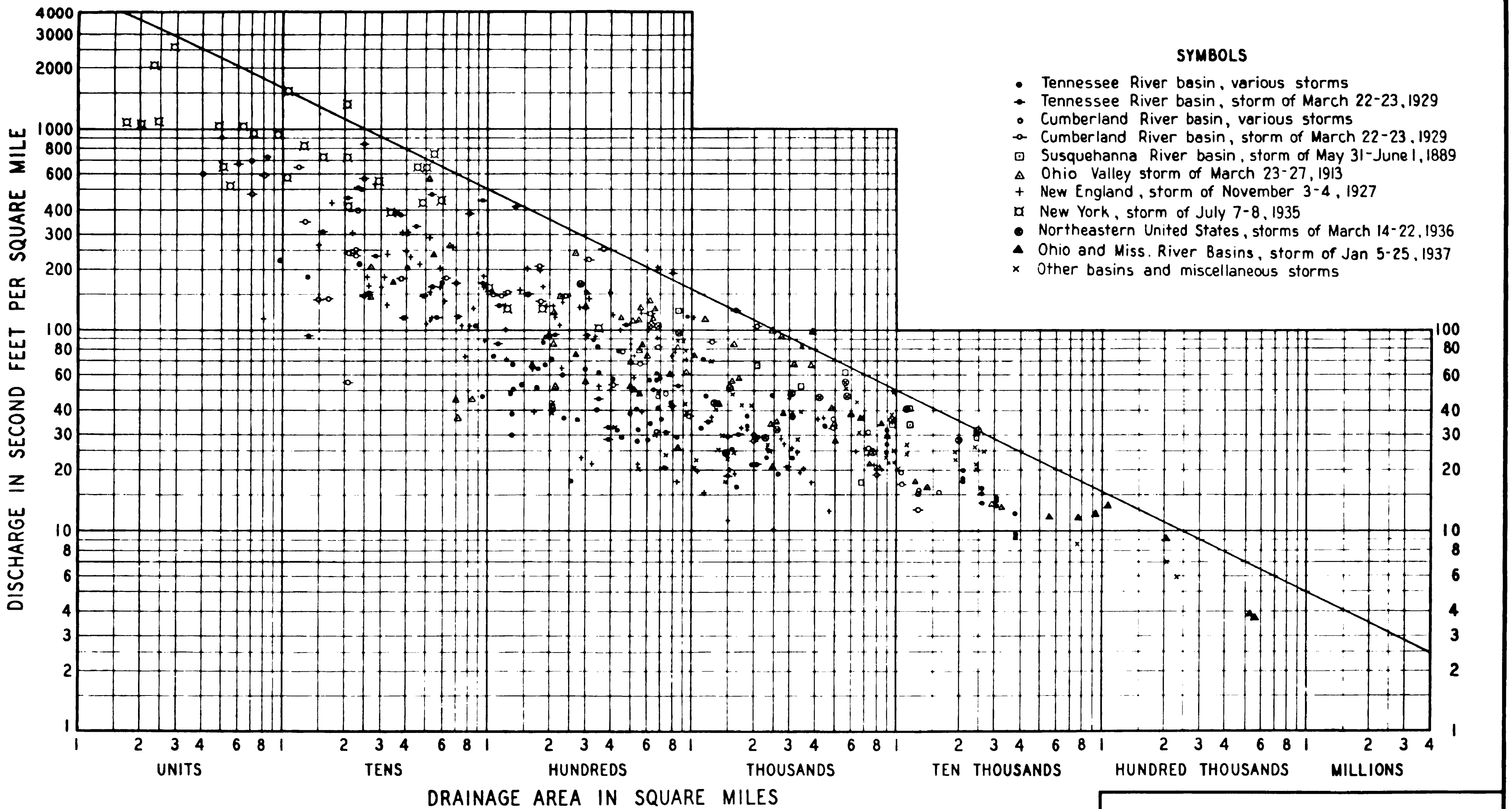
River	Station	D. A. Sq. Mile	Date	Stage Feet	Discharge in cfs Total	Discharge in cfs Per Sq. Mile	Reference
WABASH RIVER BASIN							
Wabash	Bluffton, Ind.	470[d]	Mar. 26, 1913	20.0[b]	45,000	95.7**	H.D. 100
	"	470	Jan. 16, 1937	15.02	10,800	23.0*	WSP 823
Wabash	Wabash, Ind.	1,670	Mar. 26, 1913	28.7	60,000	35.9*	WSP 838
	"	"	Jan. 16, 1930	22.65	31,100	18.6*	WSP 823
Wabash	Peru, Ind.	2,810	Mar. 1913	—	110,000	39.2**	H.D. 100
Wabash	Logansport, Ind.	3,760	Mar. 26, 1913	25.5	116,000	30.9*	WSP 823
	"	"	Feb. 27, 1936	17.85	63,700	16.9*	"
Wabash	Delphi, Ind.	4,570	Mar. 1913	—	135,000	29.6*	H.D. 100
Wabash	Lafayette, Ind.	7,250	Mar. 26, 1913	32.9	144,000	19.9*	WSP 823
	"	"	Feb. 27, 1936	25.50	78,600	10.9*	"
Wabash	Williamsport, Ind.	8,180	Mar. 1913	—	170,000	20.8*	H.D. 100
Wabash	Covington, Ind.	8,430	Mar. 1913	35.1[b]	175,000	20.8*	H.D. 100
Wabash	Montezuma, Ind.	11,000	Mar. 27, 1913	34.9	192,000	17.5*	WSP 838
	"	"	Mar. 14, 1933	28.19	101,000	9.2*	WSP 823
Wabash	Terre Haute, Ind.	12,200	Mar. 27, 1913	33.0	200,000	16.4*	WSP 823
	"	"	May 15, 1933	26.53	106,000	8.7*	"
Wabash	Palestine, Ill.	13,370	Mar. 1913	—	225,000	16.8*	H.D. 100
Wabash	Vincennes, Ind.	13,700	Mar. 29, 1913	24.3	200,000	14.6*	WSP 823
	"	"	Jan. 17, 1930	25.25	114,000	8.3*	"
Wabash	Mt. Carmel, Ill.	28,600	Mar. 30, 1913	27.65	428,000	15.0**	WSP 823
	"	"	Jan. 22, 1937	25.10	285,000	10.0*	"
Wabash	New Harmony, Ind.	29,460	Mar. 1913	—	405,000	13.7**	H.D. 100
Wabash	At mouth	33,100	Mar. 1913	—	440,000	13.3**	H.D. 100
Mississinewa	Marion, Ind.	740	May 12, 1933	15.59	20,600	27.8*	WSP 823
	"	"	Jan. 15, 1937	14.64	18,700	25.3*	"
Vermillion	Danville, Ill. (Near)	1,280	Feb. 27, 1936	22.76	22,000	17.2*	WSP 823
	"	"	Nov. 3, 1936	21.69	19,800	15.4*	"
Raccoon Creek	Bridgeton, Ind.	285	Jan. 1937	5.06	9,700	34.0*	WSP 838
Embarrass	St. Marie, Ill.	1,540	May 30, 1927	24.3	39,000	25.3*	WSP 823
	"	"	Jan. 16, 1937	20.79	17,300	11.2	"

* Runoff rate exceeding $500/(D.A.)^{1/2}$ but less than $2000/(D.A.)^{1/2}$.
** " " " $2000/(D.A.)^{1/2}$ " " " $4000/(D.A.)^{1/2}$.

b Taken from a source other than the one indicated.

d Taken from a source more recent than the reference indicated.

g Head on crest of dam.



NOTE:
Drainage areas over 500,000 sq mi apply to the Mississippi River Basin from which the arid area in the northwest part of the Missouri Basin is excluded.
This diagram indicates available peak flood rates recorded for streams in Eastern United States except those flowing through the South Atlantic and Gulf States.
The straight line indicates a basic relation of peak flood rate to drainage area for general application in the Tennessee Valley but is subject to modification for individual cases.

MAXIMUM RUNOFF RATES
EASTERN UNITED STATES

FLOOD CONTROL INVESTIGATIONS
TENNESSEE VALLEY AUTHORITY
WATER CONTROL PLANNING DEPARTMENT

SUBMITTED	RECOMMENDED	APPROVED
David E. Donley	J. L. Kimball	T. B. Parker

KNOXVILLE	1-11-37	7	PP	0	000A9R1

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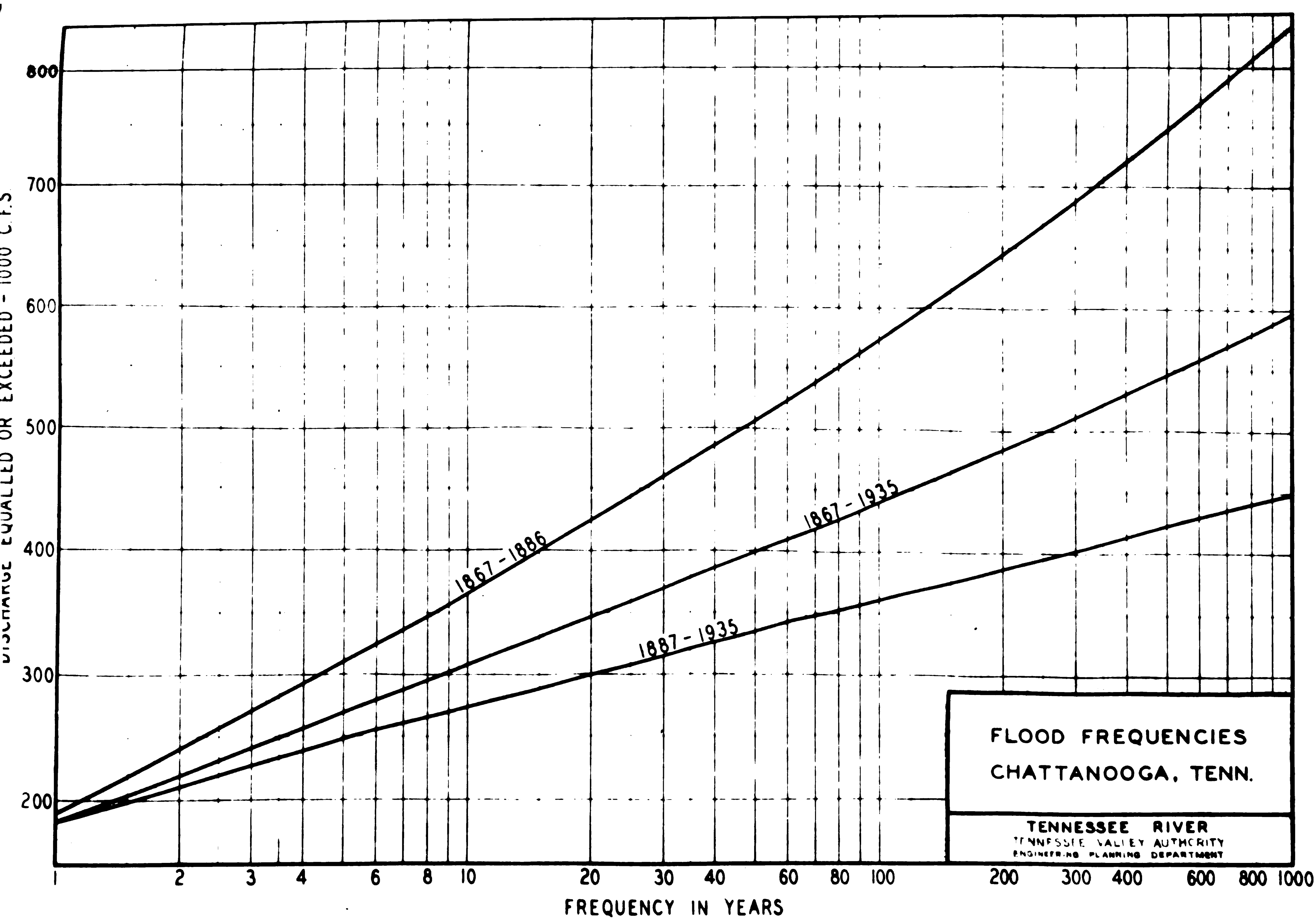
ENGINEER B. J. Buehler


FLOOD FREQUENCIES
CHATTANOOGA, TENN.
TENNESSEE RIVER
TENNESSEE VALLEY AUTHORITY
ENGINEERING PLANNING DEPARTMENT
1867 - 1886
1867 - 1935
1887 - 1935
DISCHARGE EQUALLED OR EXCEEDED - 1000 C.F.S.
800
700
600
500
400
300
200
FREQUENCY IN YEARS
1
2
3
4
6
8
10
20
30
40
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400
600
800
1000

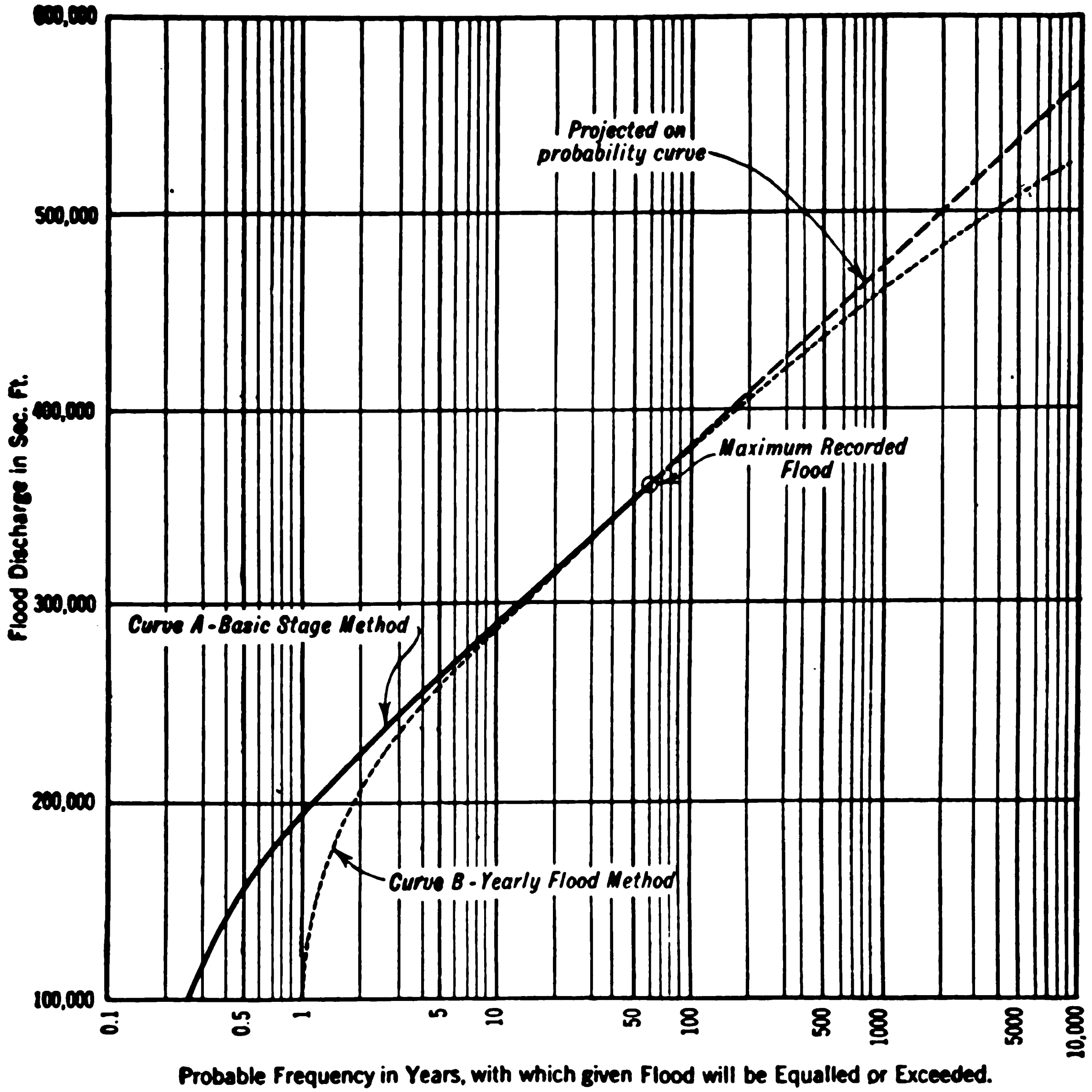
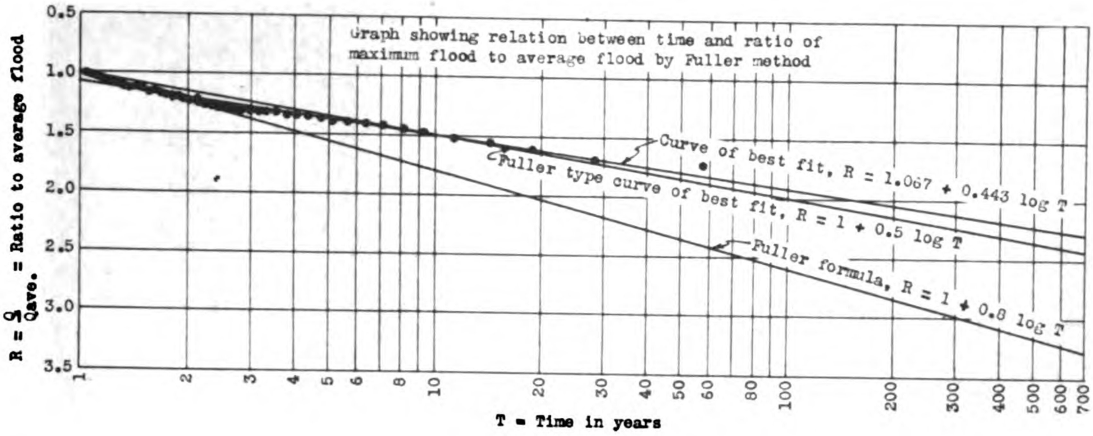
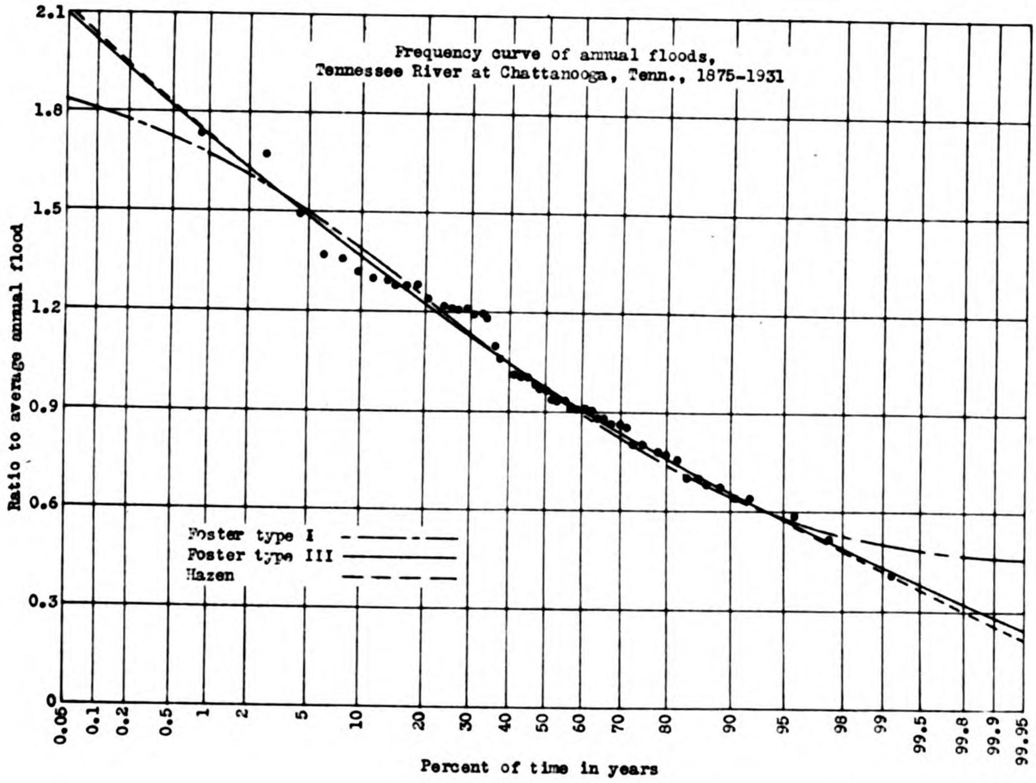


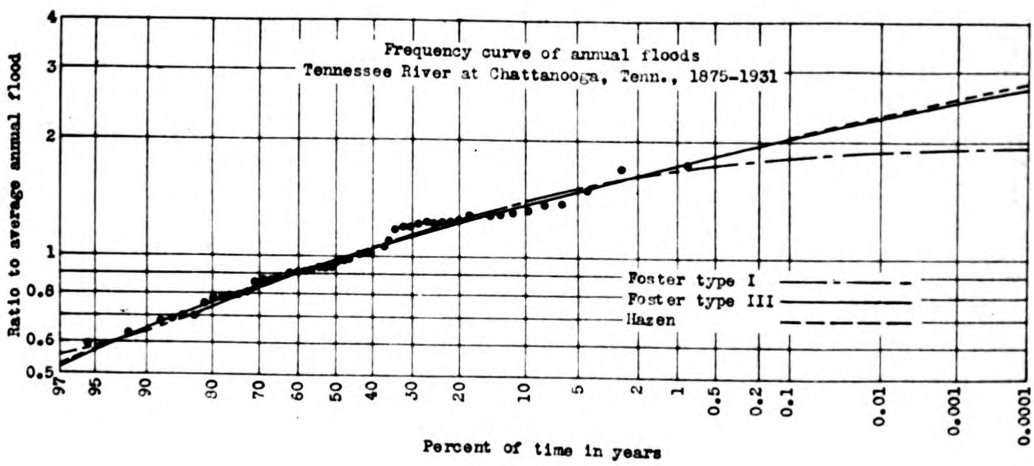
FIG. 27.—Frequency Curve, Tennessee River, at Chattanooga, Tenn.

From Hydro-Electric Handbook
by
Creager and Justin

1927

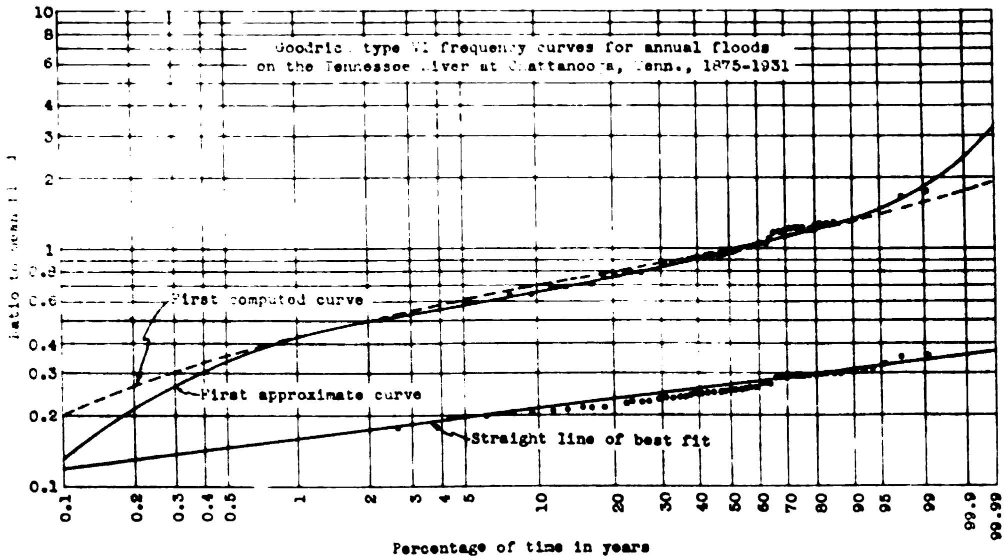


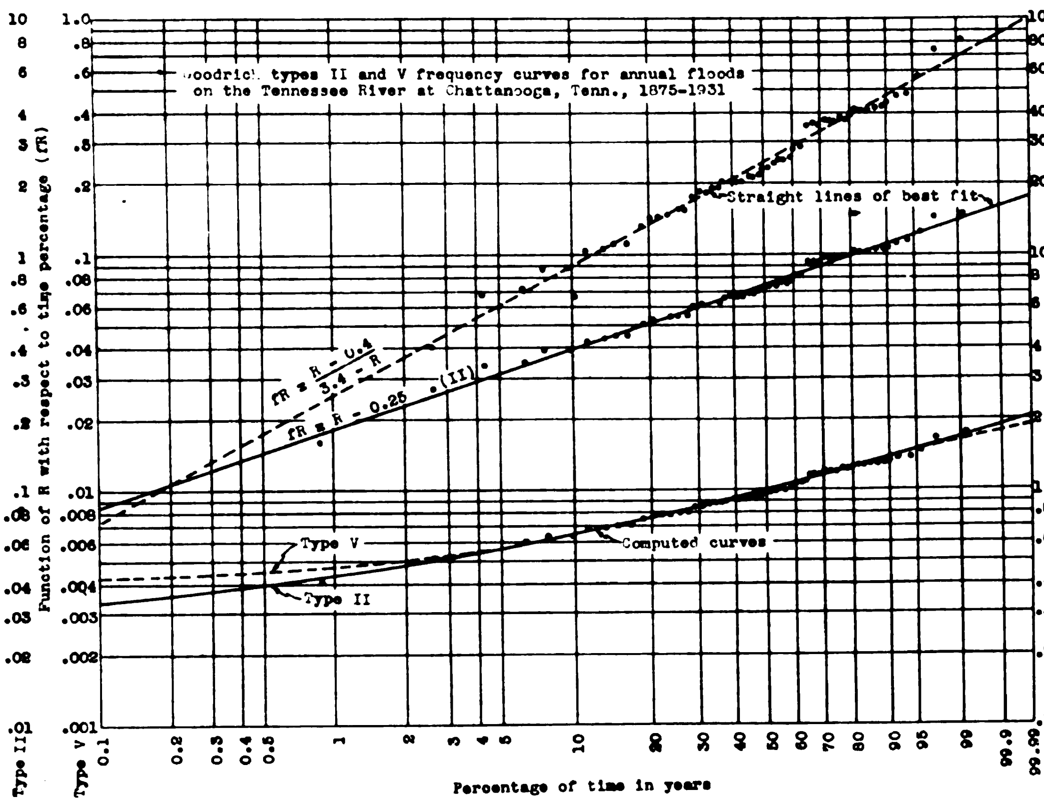




From "Floods in the United States"

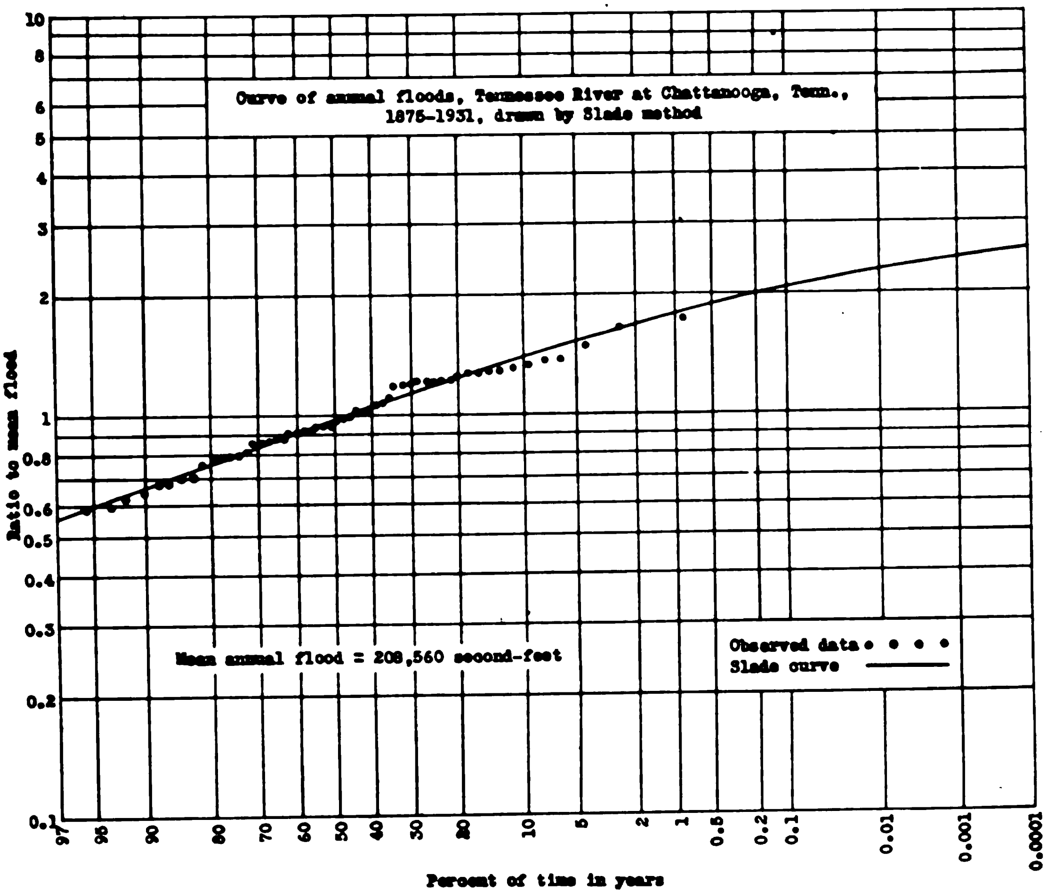
Water Supply Paper 771

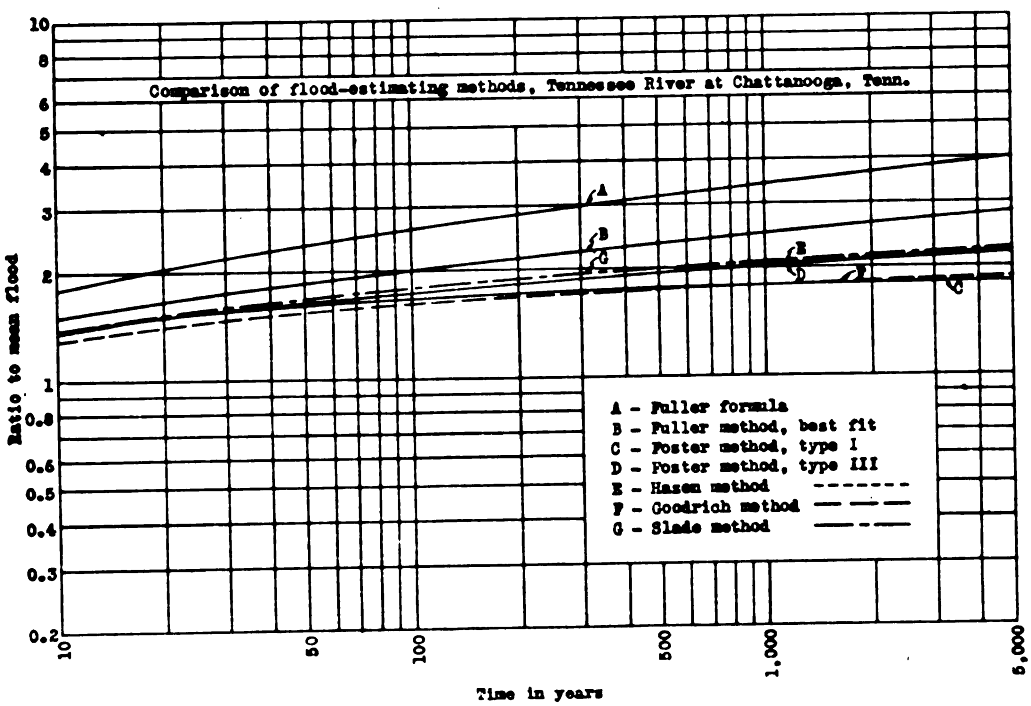




From "Floods in the United States"

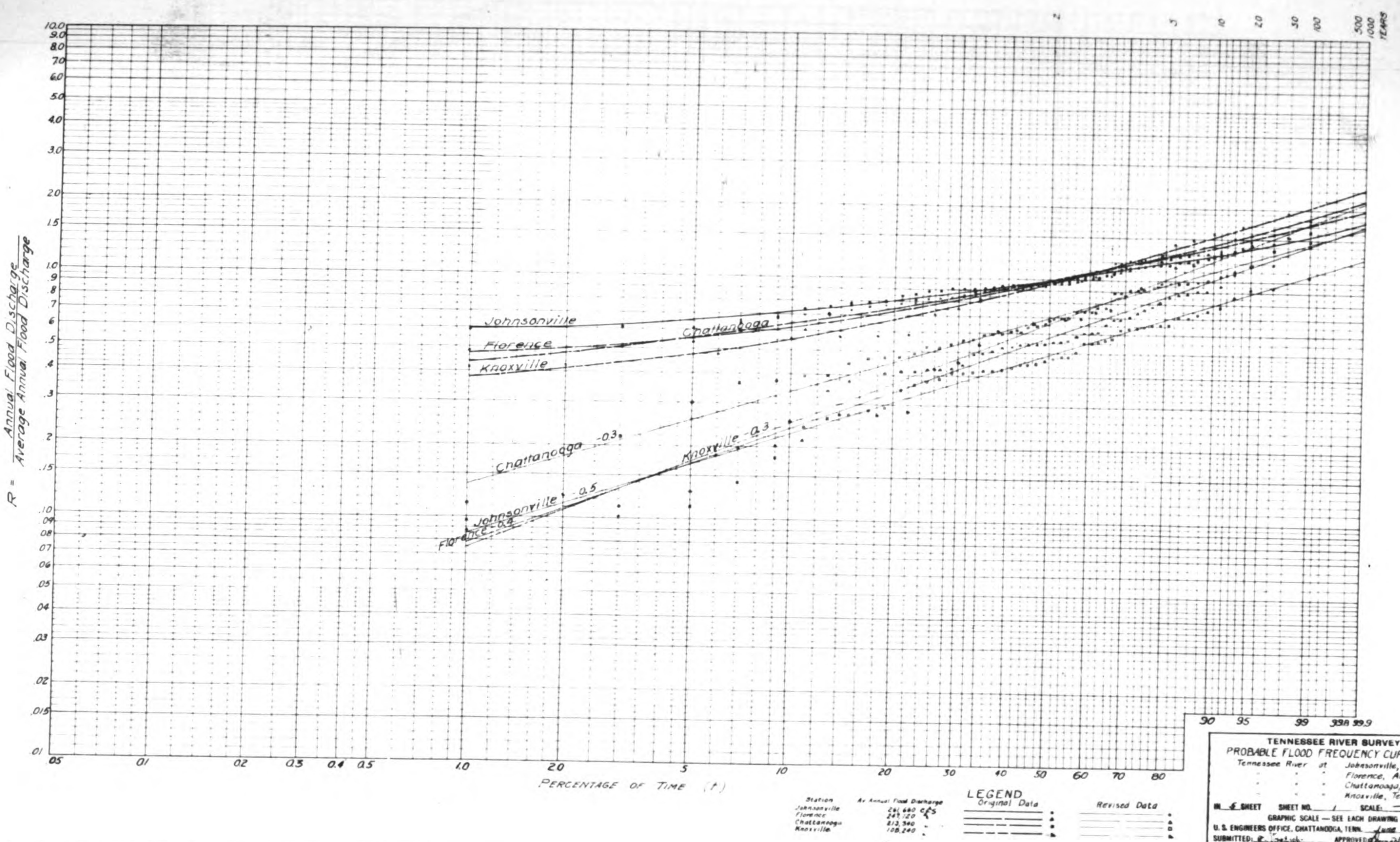
Water Supply Paper 771





From "Floods in the United States"

Water Supply Paper 771



TENNESSEE RIVER SURVEY

PROBABLE FLOOD FREQUENCY CURVES

Tennessee River at Johnsonville, Tenn.
" " " Florence, Ala.
" " " Chattanooga, Tenn.
" " " Knoxville, Tenn.

IN 6 SHEET SHEET NO. 1 SCALE: ——

GRAPHIC SCALE — SEE EACH DRAWING

U. S. ENGINEERS OFFICE, CHATTANOOGA, TENN. June 7 1929

SUBMITTED: R. Tatsch APPROVED:

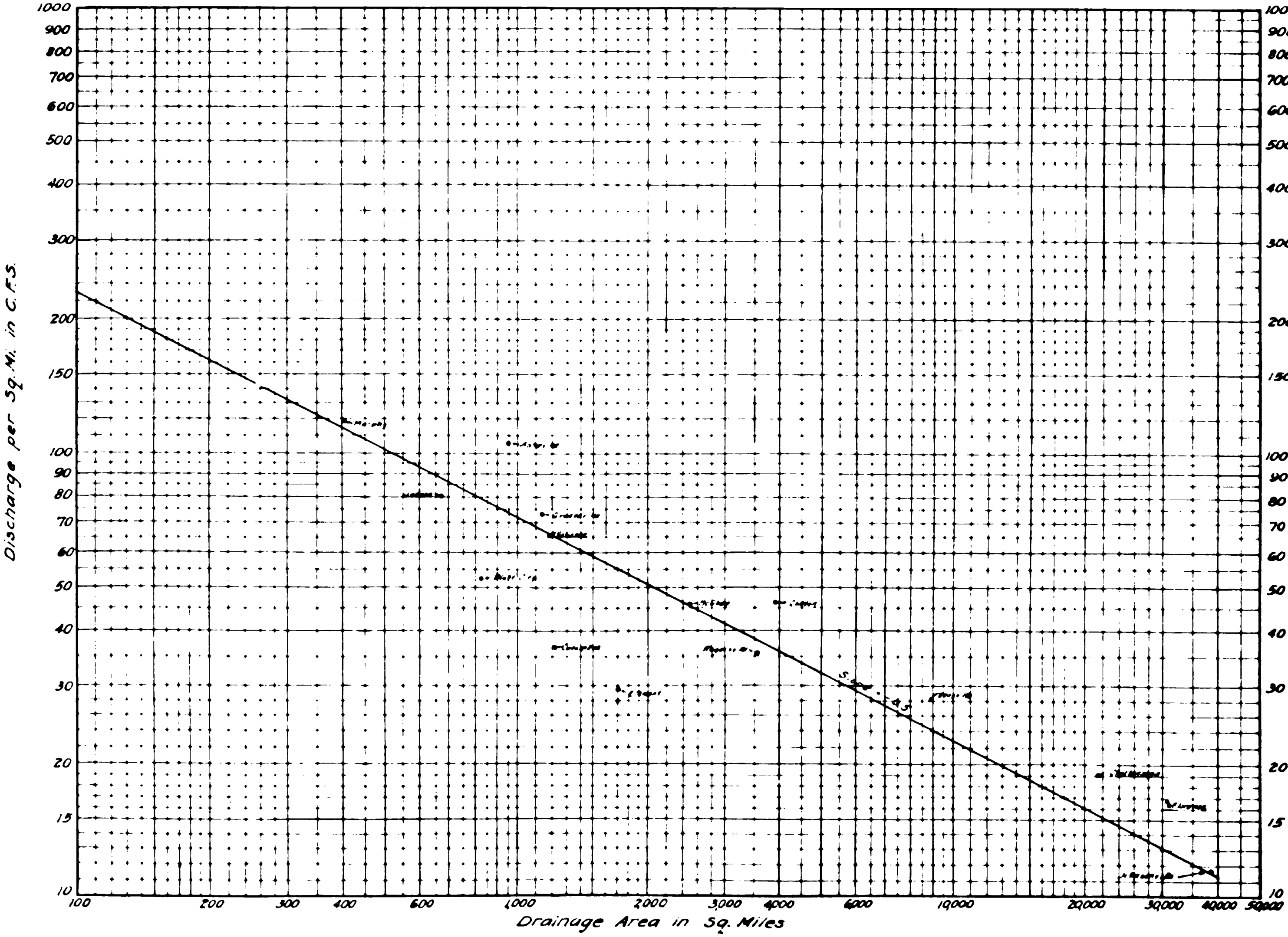
TRANSMITTED WITH REPORT ON THE TENNESSEE RIVER AND TRIBUTARIES

DRAWN BY JPW FILE NO. DATE Dec. 15 1929

Goodrich's Skew Probability Chart Trans. Am. Soc. C.E. Vol. 91, 1927, p. 13.

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



TENNESSEE RIVER SURVEY

RELATION BETWEEN 500-YEAR FLOOD AND DRAINAGE AREA

IN 5 SHEETS SHEET NO. 5 SCALE

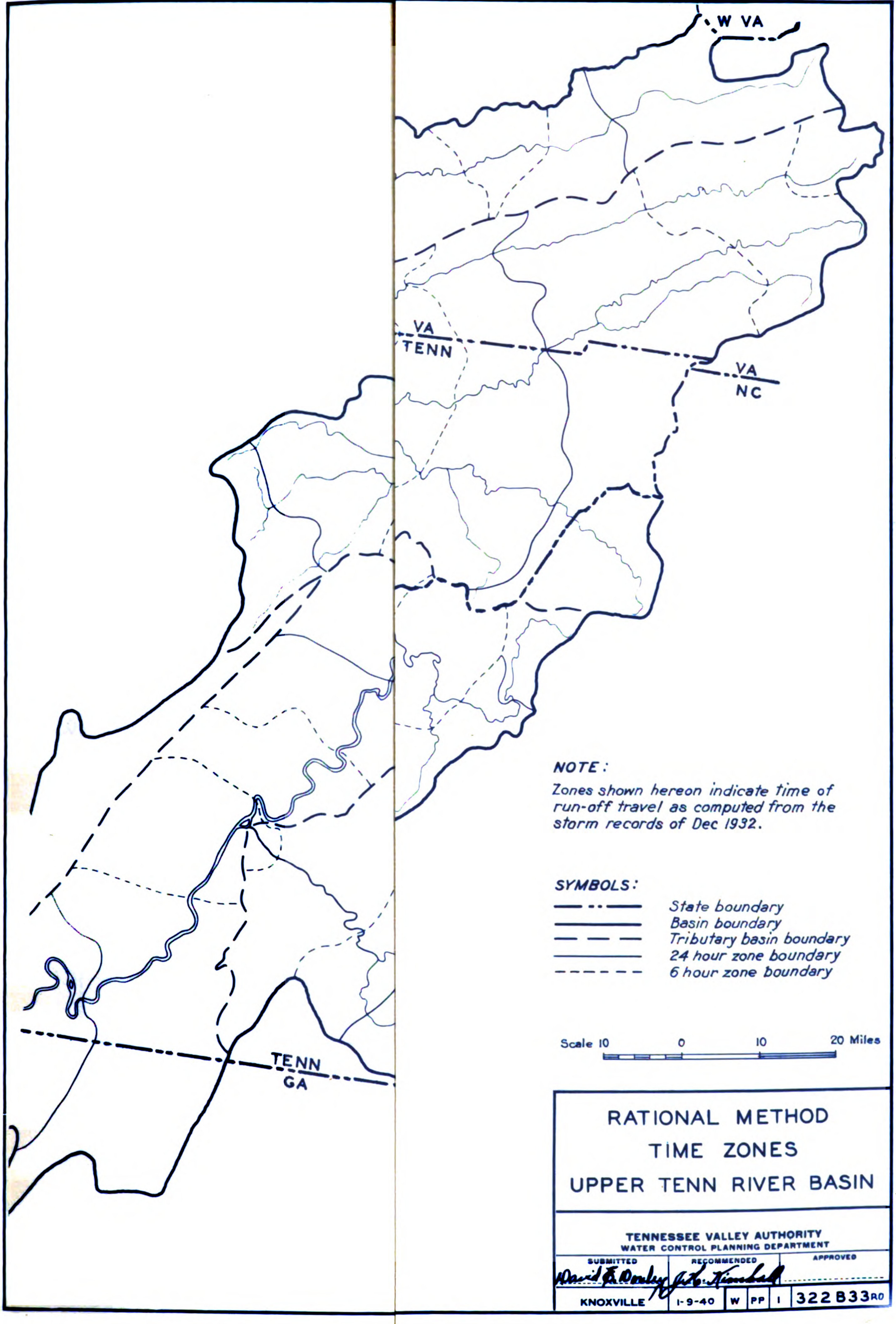
GRAPHIC SCALE — SEE EACH DRAWING

U. S. ENGINEER OFFICE, CHATTANOOGA, TENN. June 7

SUBMITTED

TRANSMITTED WITH REPORT ON THE TENNESSEE RIVER AND TRIBUTARIES

DRAWN BY FILE NO. DATE Dec. 18



NOTE:

Zones shown hereon indicate time of run-off travel as computed from the storm records of Dec 1932.

SYMBOLS:

- *State boundary*
- *Basin boundary*
- *Tributary basin boundary*
- *24 hour zone boundary*
- *6 hour zone boundary*

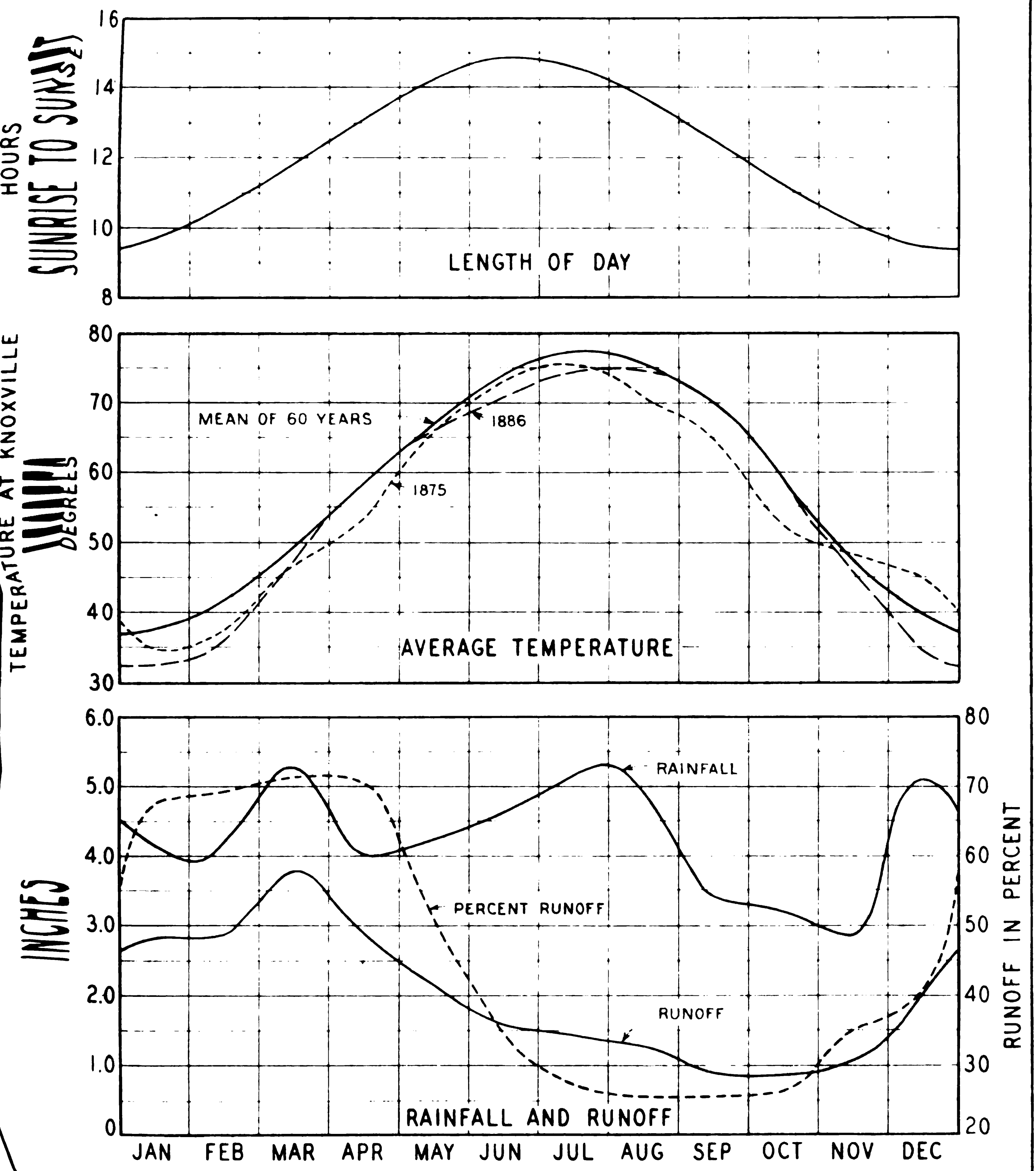
Scale 10 0 10 20 Miles

RATIONAL METHOD
TIME ZONES
UPPER TENN RIVER BASIN

TENNESSEE VALLEY AUTHORITY
WATER CONTROL PLANNING DEPARTMENT

SUBMITTED	RECOMMENDED	APPROVED
David E. Donley	J. L. Kimball	

KNOXVILLE	1-9-40	W	PP	1	322 B 33 RO



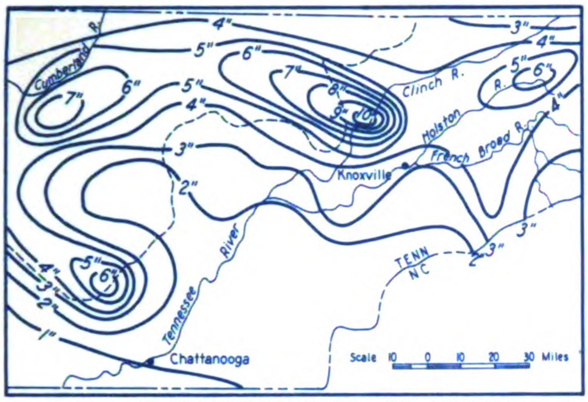
METEOROLOGY
TENNESSEE RIVER BASIN
ABOVE CHATTANOOGA

FLOOD CONTROL INVESTIGATIONS
TENNESSEE VALLEY AUTHORITY
WATER CONTROL PLANNING DEPARTMENT

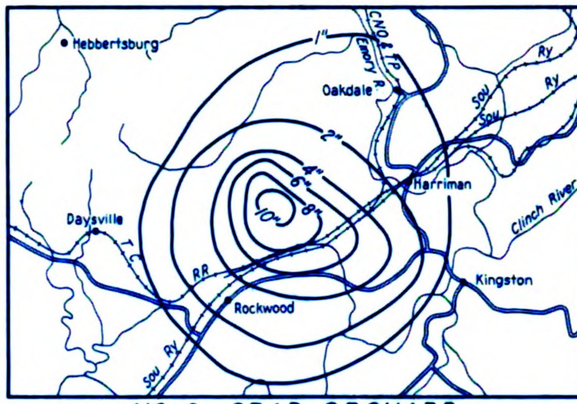
SUBMITTED David E. Donley | RECOMMENDED J. K. Kimball | APPROVED

KNOXVILLE | 11-18-38 | W | PP | O | 1A132R

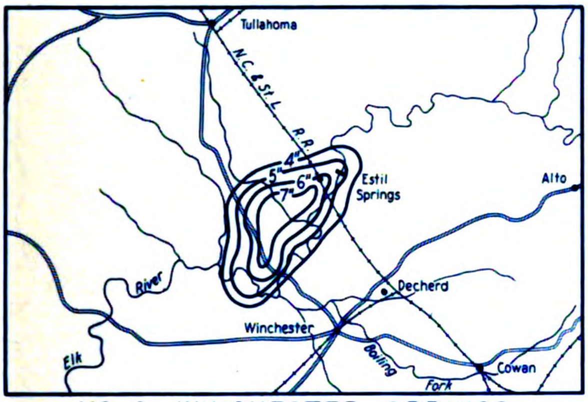
MADE CHKD. SUPV. INSP.
DRAWN
TRCD
CHKD
COMPUTED WAS
ENGINEER David E. Donley



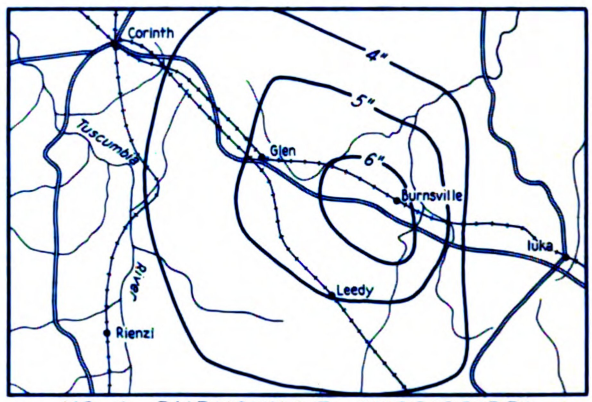
NO. 1 CUMBERLAND PLATEAU
JUNE 29, 1928



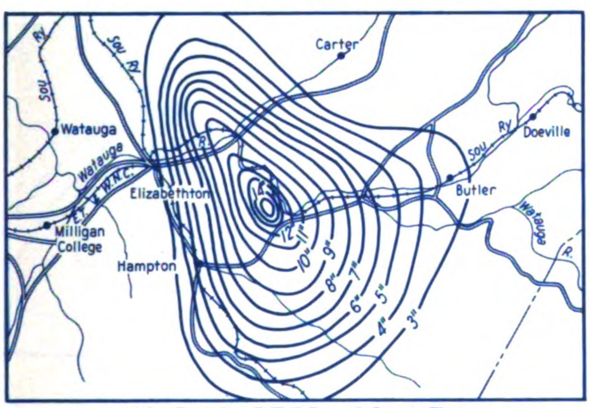
NO. 2 CRAB ORCHARD
MAY 22, 1938



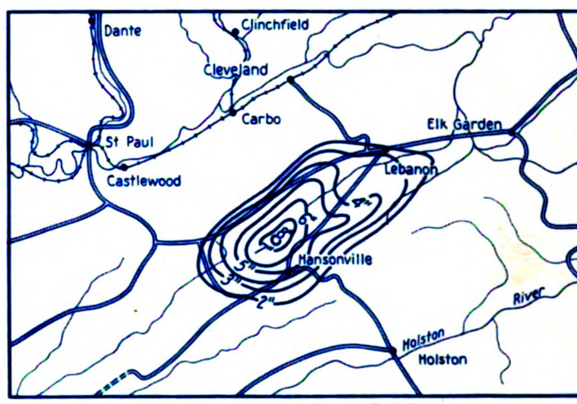
NO. 3 WINCHESTER SPRINGS
JULY 8, 1938



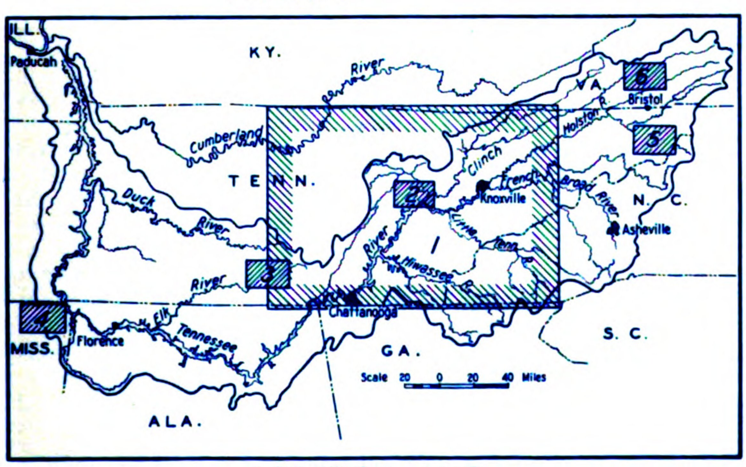
NO. 4 BURNSVILLE, MISSISSIPPI
AUGUST 29-30, 1938



NO. 5 CARTER COUNTY
JUNE 13, 1924



NO. 6 CEDAR CREEK
MAY 25, 1939



LOCATION MAP

NOTE:
The storms shown are believed to be typical of the summer thundershowers and cloudbursts that are likely to occur over almost any section of the Tennessee River Basin.

Scale 4 0 4 8 Miles
Except as noted

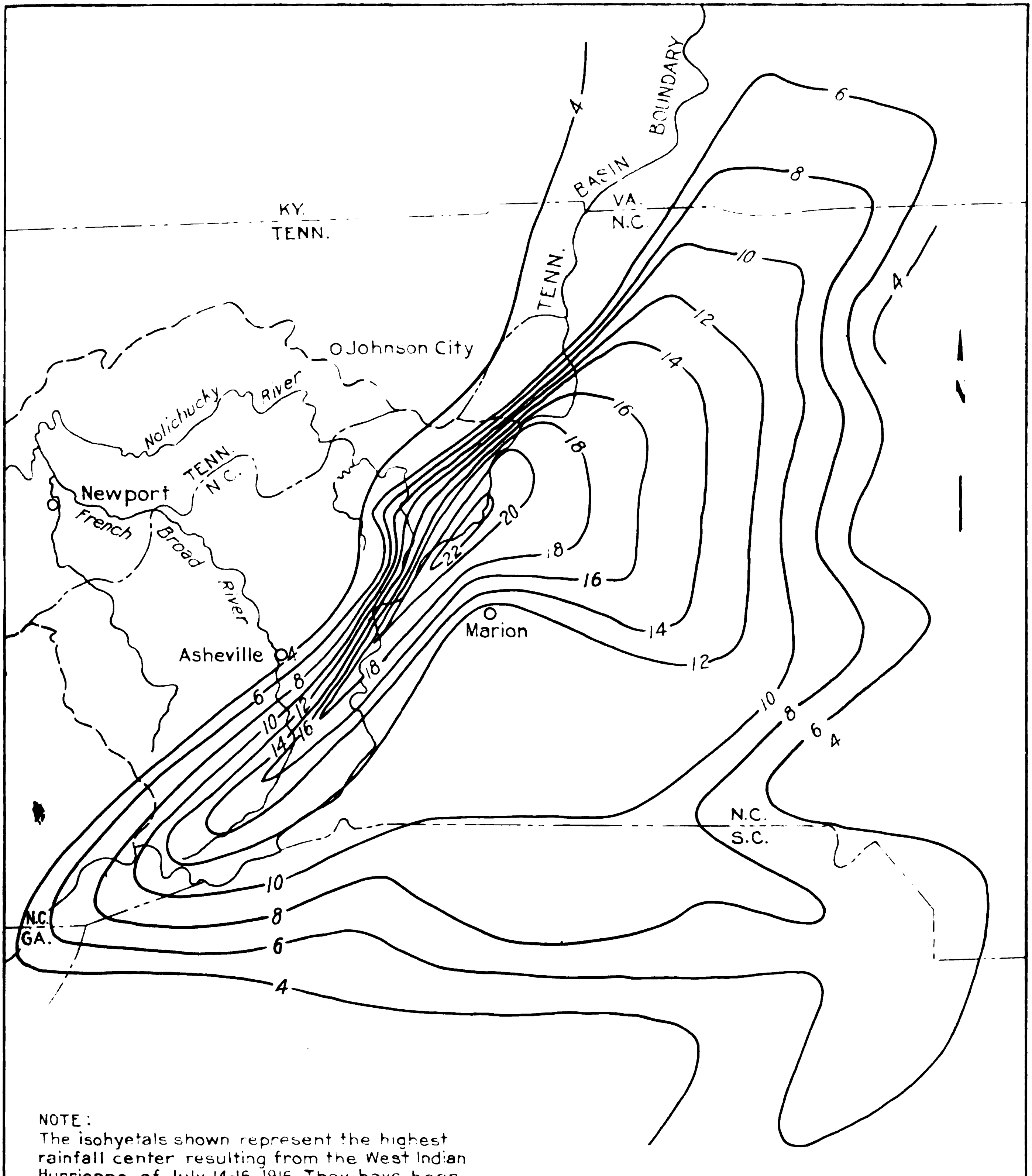
TYPICAL STORMS CLOUDBURST OR THUNDERSTORM TYPE TENNESSEE VALLEY REGION

FLOOD CONTROL INVESTIGATIONS
TENNESSEE VALLEY AUTHORITY
WATER CONTROL PLANNING DEPARTMENT

SUBMITTED	RECOMMENDED	APPROVED
David B. Donley	J. H. Kimball	

KNOXVILLE	8-8-39	W	PP	I	322G21R0

TRACED ... ENGINEER F.T. Mitchell CHKD. ...



NOTE:
The isohyetals shown represent the highest rainfall center resulting from the West Indian Hurricane of July 14-16, 1916. They have been plotted from data published by the U.S.W.B. The high elevations northeast of Asheville resulted in the intense rainfall being concentrated in this area.
Topography of the area was carefully considered in determining the shape of these lines.

DISTRIBUTION OF RAINFALL
WESTERN NORTH CAROLINA
STORM OF JULY 14-16, 1916

FLOOD CONTROL INVESTIGATIONS
TENNESSEE VALLEY AUTHORITY
WATER CONTROL PLANNING DEPARTMENT

KNOXVILLE 8-29-38 W PP O 1A30R

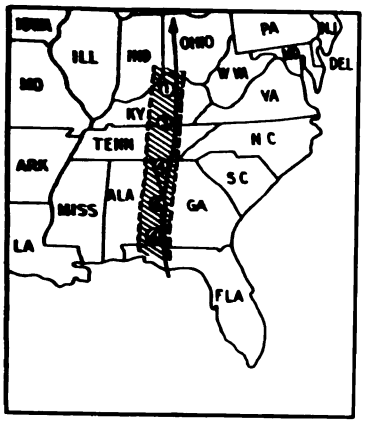
REV. NO. DATE MADE CHKD. SUPV. INSP.

DRAWN COMPUTED

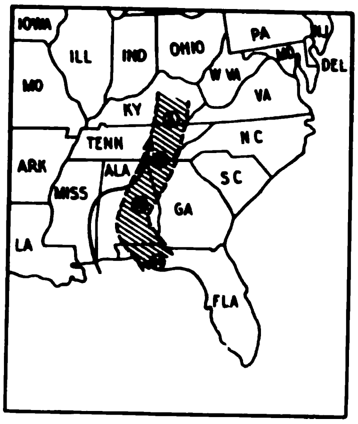
TRCD

CHKD. R.S. Mitchell

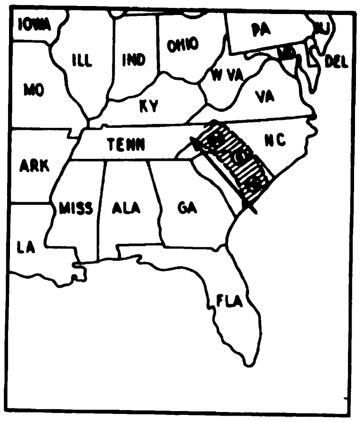
FU 33-91 63



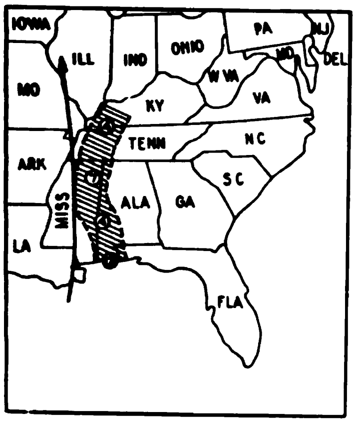
SEP 4-5, 1915



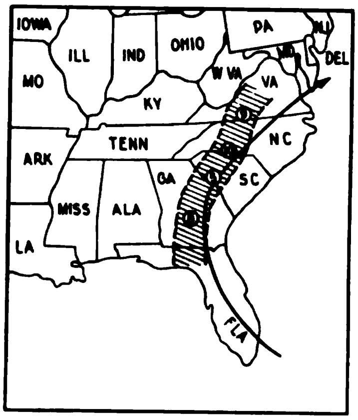
JUL 5-10, 1916



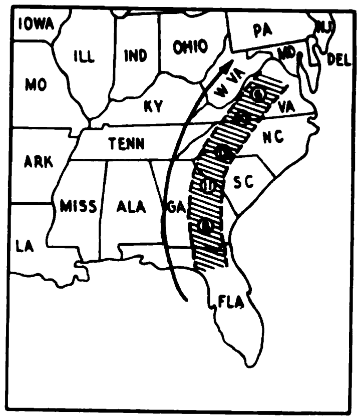
JUL 14-17, 1916



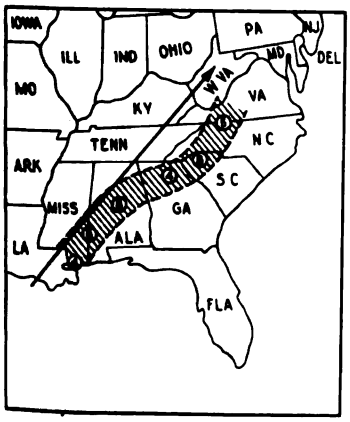
OCT 16-19, 1923



AUG 10-11, 1928



AUG 14-16, 1928



OCT 15-18, 1932

NOTE:
The hurricane paths shown represent the movements of the low pressure areas which accompanied the disturbances.
The maximum rainfall areas shown represent the 100-mile wide zone of maximum rainfall which resulted from the passage of these storms.
Basic data for determining these paths and areas were taken from published records of the U S Weather Bureau.

Scale 200 0 400 800 Miles

LEGEND

Maximum rainfall area

Hurricane path

④ Rainfall-inches

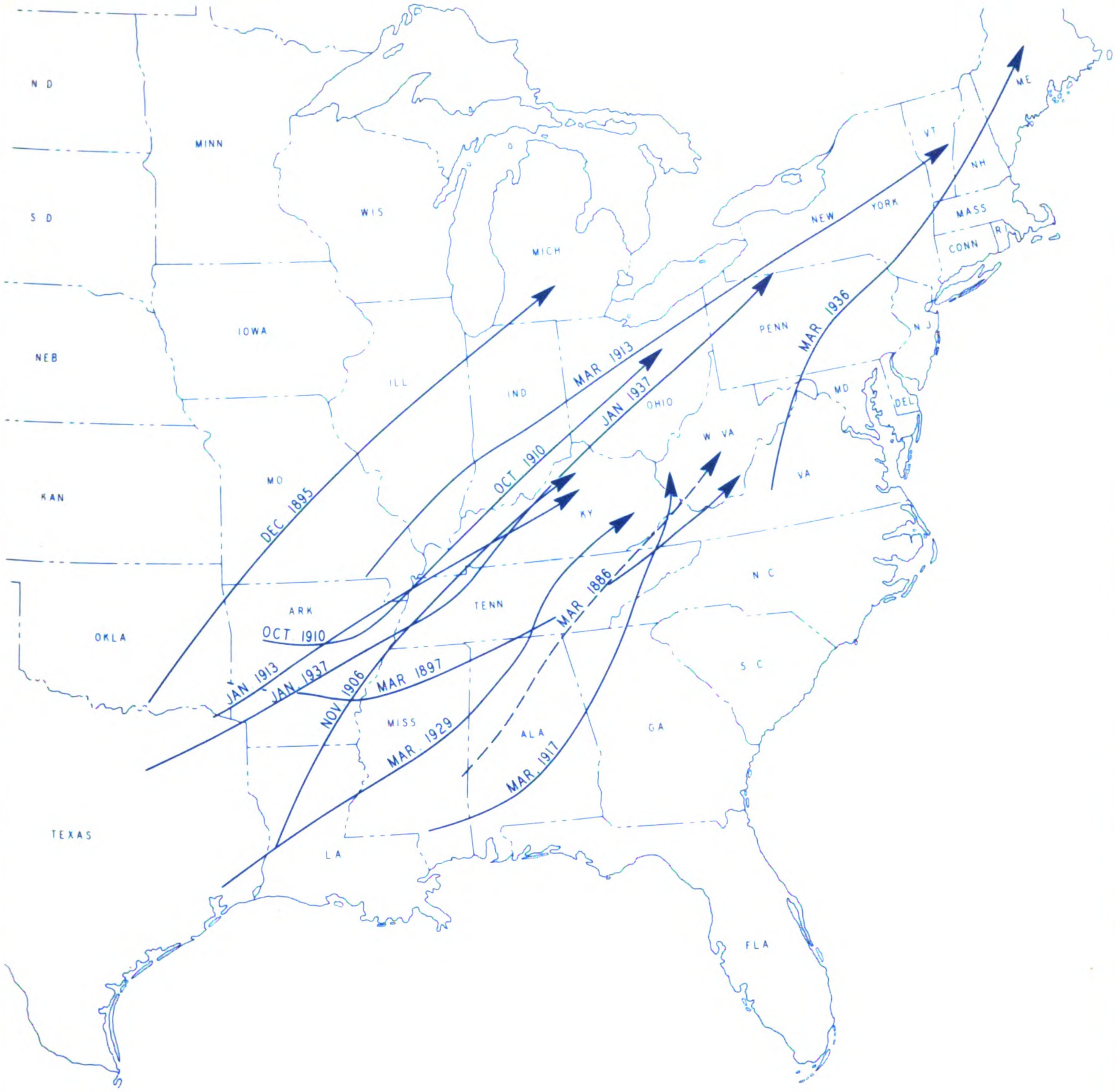
WEST INDIAN HURRICANE LOW PRESSURE MOVEMENTS AND THE CORRESPONDING RAINFALL AREAS

FLOOD CONTROL INVESTIGATIONS
TENNESSEE VALLEY AUTHORITY
WATER CONTROL PLANNING DEPARTMENT

SUBMITTED	RECOMMENDED	APPROVED
David E. Donley	J. B. Kimball	[illegible]

KNOXVILLE	10-4-38	W	PP	O	1A68RC

DRAWN R.L.P.	COMPUTED
TRCD.	ENGINEER R. L. Mitchell
CHKD.	



NOTE:
The arrows shown indicate the location of the center and the length of the rainfall zones of the outstanding storms which have occurred, or which it is believed may reoccur, in the vicinity of the Tennessee River basin. Data for this study were taken from the publications of the Miami Conservancy District and of the U.S. Weather Bureau.

Scale 50 0 100 200 Miles

REV. NO.	DATE	MADE	CHKD.	SUPV.	INSP.
1	8-4-39	T.B.			
Graphic scale corrected.					

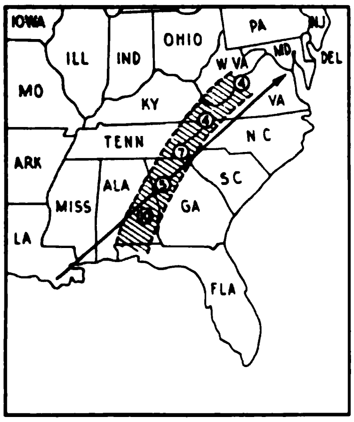
DRWN.
TRCD. L.H.P.
CHKD.
COMPUTED
ENGINEER David E. Donley

OUTSTANDING STORMS OCCURRENCE AND PATHS OF GREAT RAINFALL

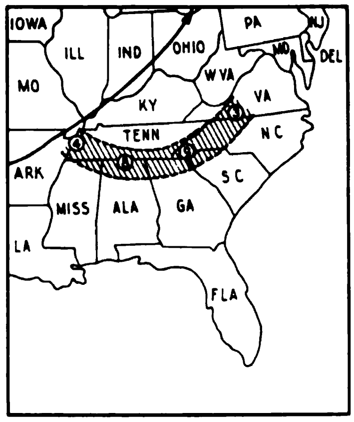
FLOOD CONTROL INVESTIGATIONS
TENNESSEE VALLEY AUTHORITY
WATER CONTROL PLANNING DEPARTMENT

SUBMITTED	RECOMMENDED	APPROVED
David E. Donley	J. H. Kimball	T. B. Parker

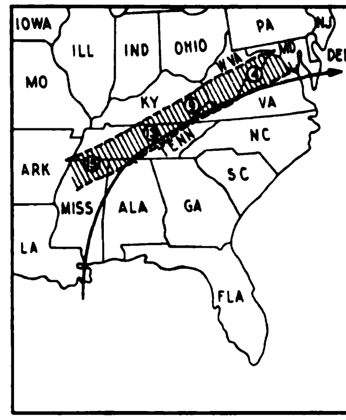
KNOXVILLE	11-12-37	G	GP	0	9A20 R1



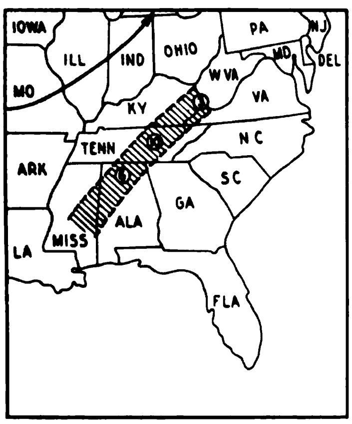
MAR 1-5, 1917



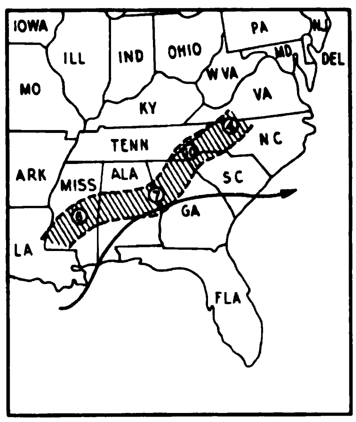
APR 1-5, 1920



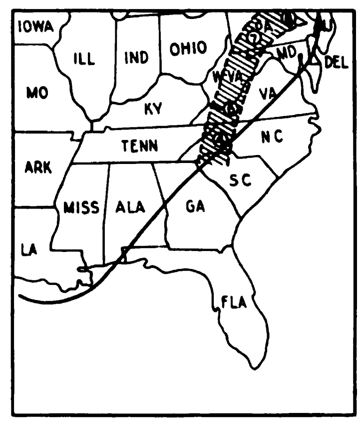
DEC 26-29, 1926



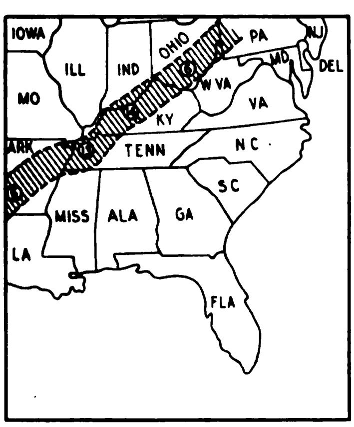
MAR 21-24, 1929



DEC 24-31, 1932



MAR 16-19, 1936



JAN 20-25, 1937

NOTE:
The paths of low pressure areas represent those that accompanied the cyclonic disturbances shown. The maximum rainfall areas shown represent the 100-mile wide zone of maximum rainfall which resulted from the passage of these storms. Basic data for determining these paths and areas were taken from published records of the U S Weather Bureau.

Scale 200 0 400 800 Miles

LEGEND

- Maximum rainfall area
- Path of low pressure area
- ④ Rainfall - inches

CYCLONIC STORMS LOW PRESSURE MOVEMENTS AND THE CORRESPONDING RAINFALL AREAS

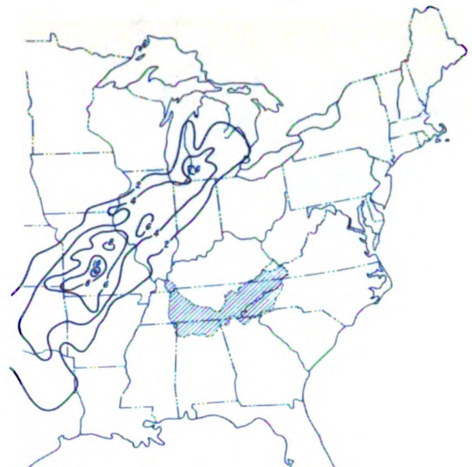
FLOOD CONTROL INVESTIGATIONS
TENNESSEE VALLEY AUTHORITY
WATER CONTROL PLANNING DEPARTMENT

SUBMITTED	RECOMMENDED	APPROVED
KNOXVILLE	10-4-38	W PP 0 1A69RO

DRWN ELM
TRCD
CHKD
COMPUTED
ENGINEER

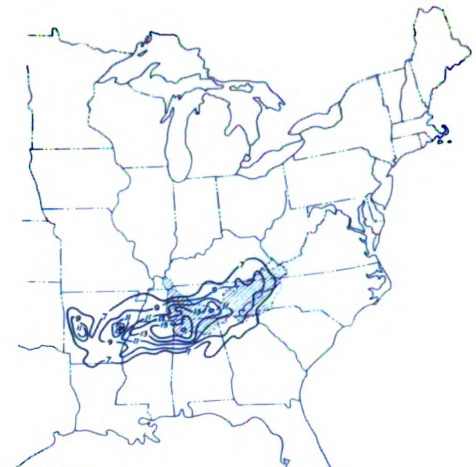
NOTE:
Data shown hereon were taken from "Storm Rainfall of Eastern United States," Part V Technical Reports, Miami Conservancy District.

2-DAY PERIOD MAY 31-JUN 1, 1889



NOTE:
Data shown hereon were taken from "Storm Rainfall of Eastern United States," Part V Technical Reports, Miami Conservancy District.

4-DAY PERIOD DEC 17-20, 1895

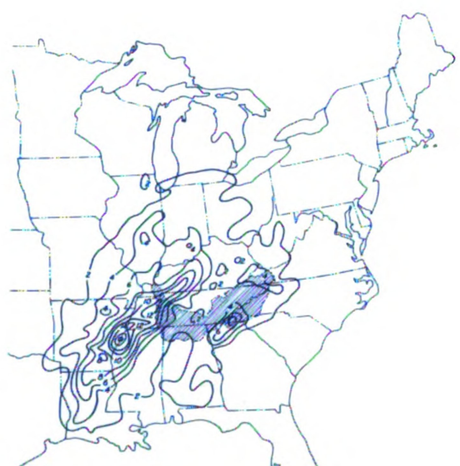


NOTE:
Data shown hereon were derived from published records of U.S. Weather Bureau.

17-DAY PERIOD MAR 3-19, 1897

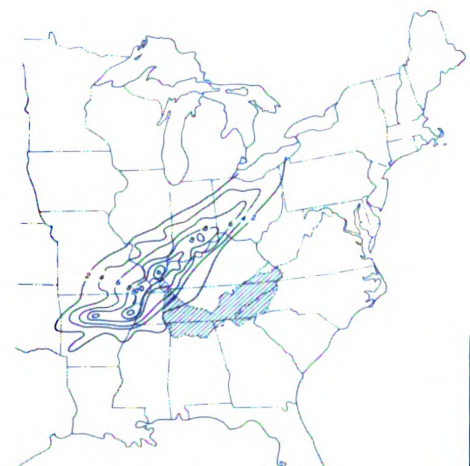
NOTE:
Data shown hereon were derived from published records of U.S. Weather Bureau.

11-DAY PERIOD MAR 9-19, 1897



NOTE:
Data shown hereon were taken from "Storm Rainfall of Eastern United States," Part V Technical Reports, Miami Conservancy District.

5-DAY PERIOD NOV 17-21, 1906



NOTE:
Data shown hereon were taken from "Storm Rainfall of Eastern United States," Part V Technical Reports, Miami Conservancy District.

3-DAY PERIOD OCT 4-6, 1910

NOTE:
These storms were selected as being typical examples of the storms which may occur over the Tennessee Valley area.

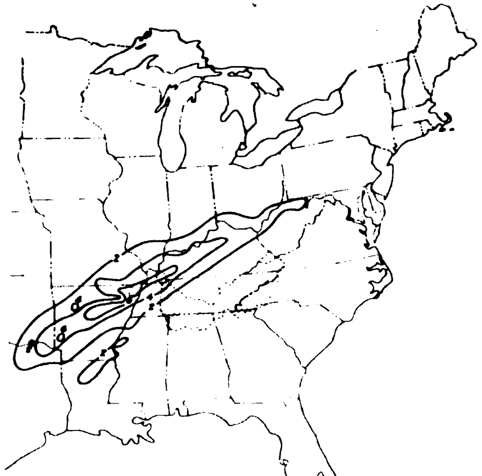
Scale 200 0 200 400 600 Miles

EXCESSIVE STORM RAINFALL

1889, 1895, 1897, 1906 AND 1910

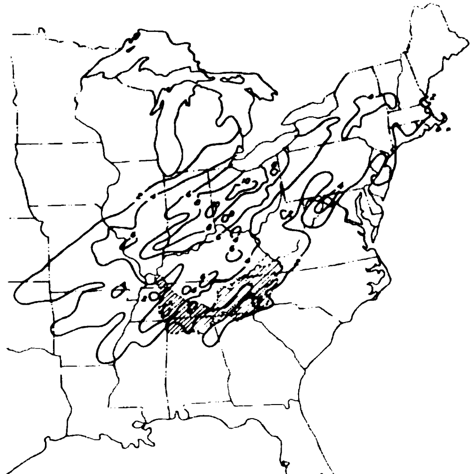
FLOOD CONTROL INVESTIGATIONS
TENNESSEE VALLEY AUTHORITY
WATER CONTROL PLANNING DEPARTMENT

SUBMITTED	RECOMMENDED	APPROVED
David B. Donley	J.H. Kimball	T.B. Parker
KNOXVILLE	11-16-38 W PP O	1A125 R0



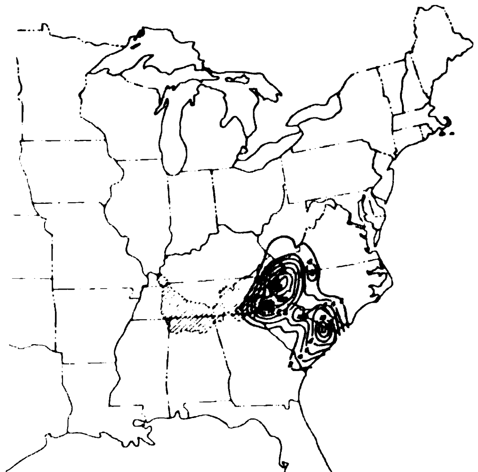
NOTE:
Data shown hereon were taken from "Storm Rainfall of Eastern United States," Part V Technical Reports, Miami Conservancy District.

3-DAY PERIOD JAN 10-12, 1913



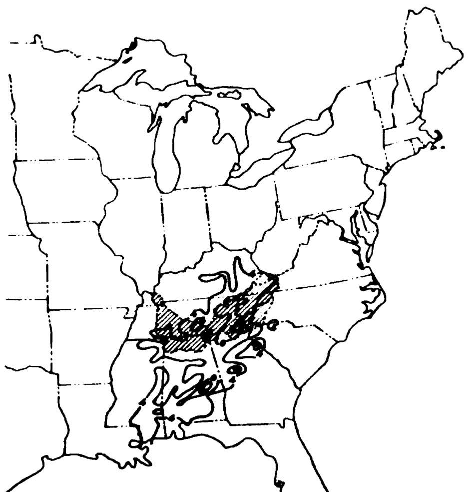
NOTE:
Data shown hereon were taken from "Storm Rainfall of Eastern United States," Part V Technical Reports, Miami Conservancy District.

5-DAY PERIOD MAR 23-27, 1913



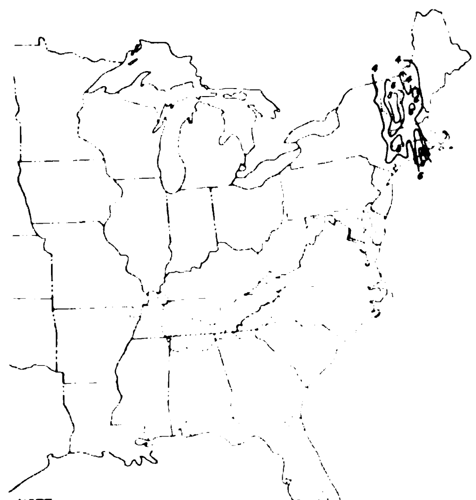
NOTE:
Data shown hereon were taken from "Storm Rainfall of Eastern United States," Part V Technical Reports, Miami Conservancy District.

3-DAY PERIOD JUL 14-16, 1916



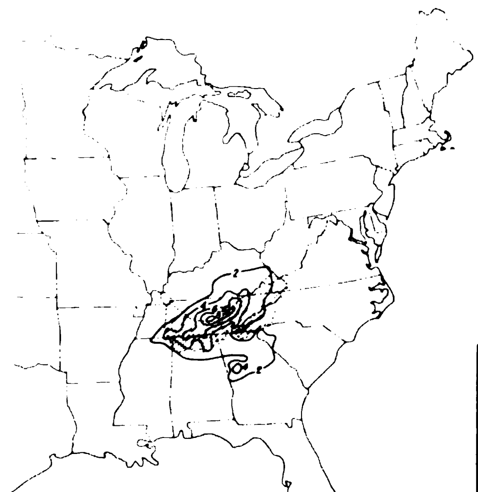
NOTE:
Data shown hereon were derived from published records of the U.S. Weather Bureau.

7-DAY PERIOD FEB 27-MAR 5, 1917



NOTE:
Data shown hereon were taken from "Journal of the Boston Society of Civil Engineers September, 1930."

4-DAY PERIOD NOV 2-5, 1927



NOTE:
Data shown hereon were taken from the "Report on the Economic Height of Norris Dam" by E.B. Debler, November 1, 1933.

2-DAY PERIOD MAR 22-23, 1929

NOTE:
These storms were selected as being typical examples of the storms which may occur over the Tennessee Valley area.

Scale 200 0 200 400 600 Miles

EXCESSIVE STORM RAINFALL

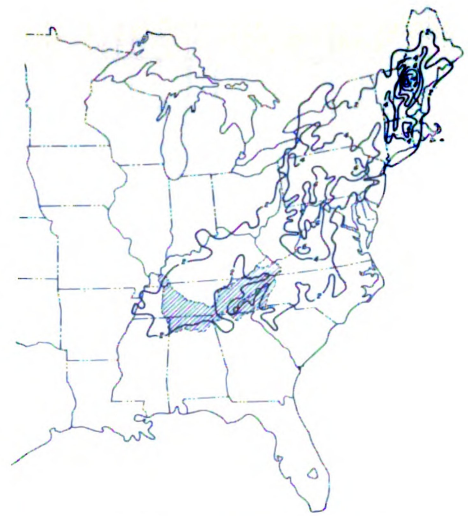
1913, 1916, 1917, 1927 AND 1929

FLOOD CONTROL INVESTIGATIONS
TENNESSEE VALLEY AUTHORITY
WATER CONTROL PLANNING DEPARTMENT

SUBMITTED	RECOMMENDED	APPROVED
David B. Donley	J. H. Kimball	T. B. Parker

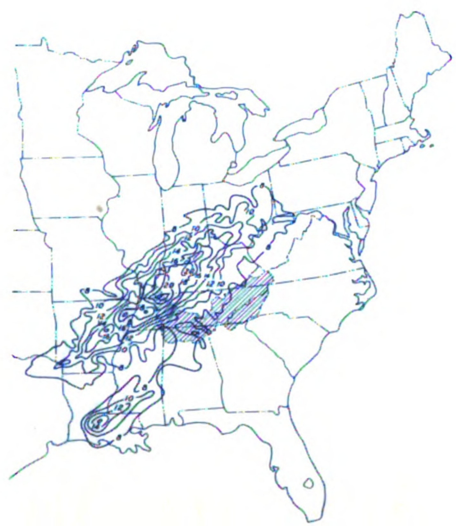
KNOXVILLE	11-16-38	W	PP	O	1A126 R0

14-DAY PERIOD MAR 9-22, 1936

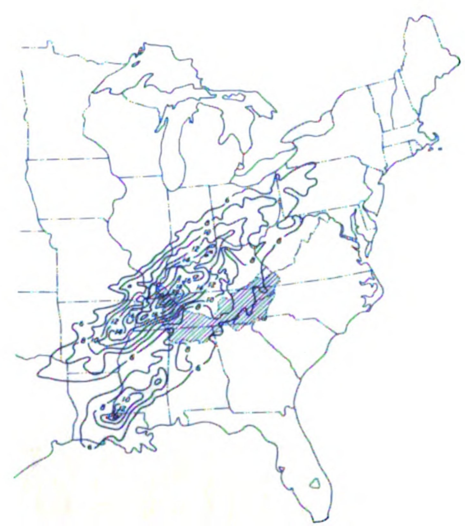


7-DAY PERIOD MAR 16-22, 1936

5-DAY PERIOD MAR 16-20, 1936



20-DAY PERIOD JAN 6-25, 1937



14-DAY PERIOD JAN 12-25, 1937

NOTE:
These storms were selected as being typical examples of the storms which may occur over the Tennessee Valley area.
Data shown hereon were derived from published records of the U.S. Weather Bureau.

Scale 200 0 200 400 600 Miles

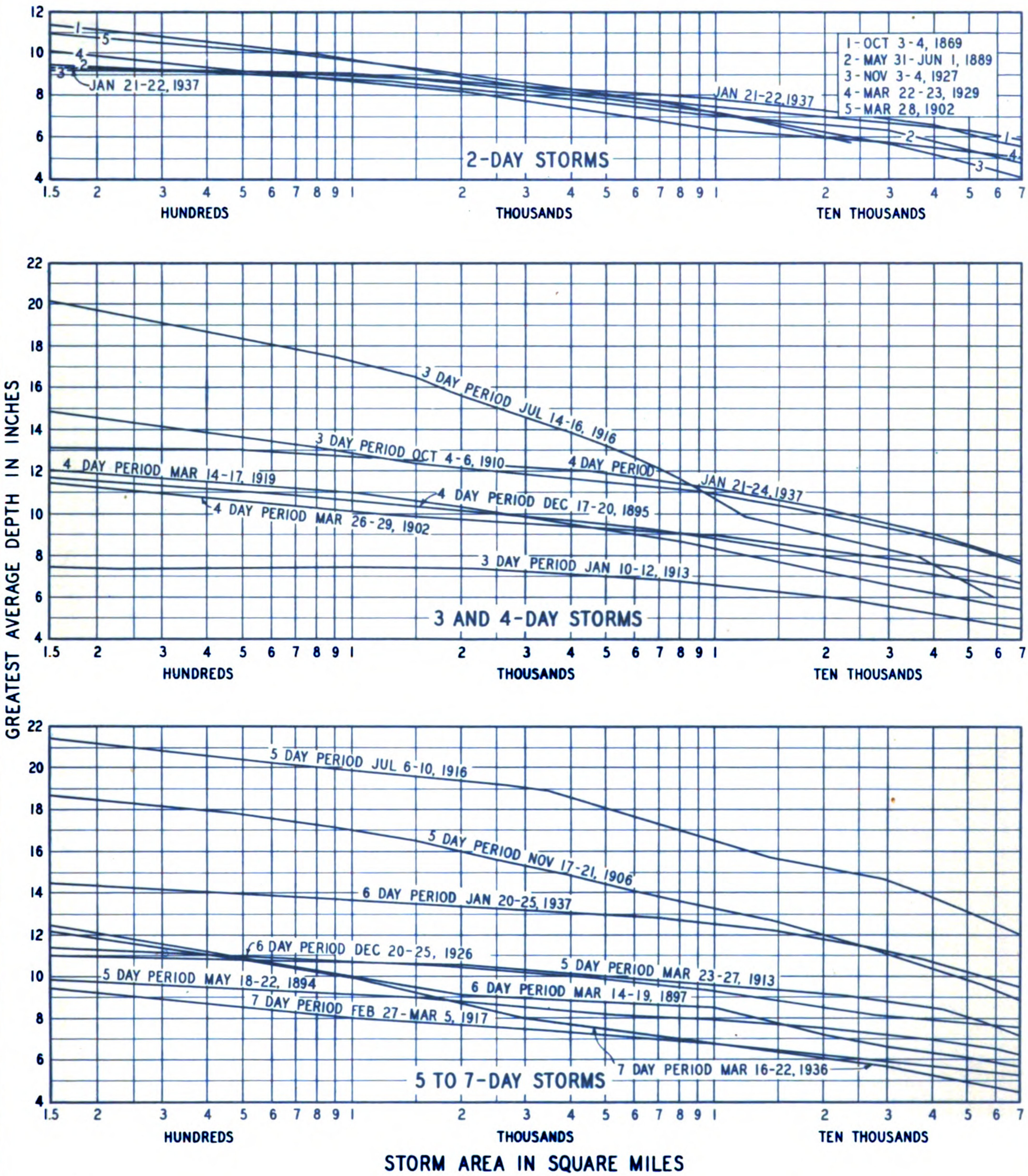
EXCESSIVE STORM RAINFALL

1936 AND 1937

FLOOD CONTROL INVESTIGATIONS
TENNESSEE VALLEY AUTHORITY
WATER CONTROL PLANNING DEPARTMENT

SUBMITTED	RECOMMENDED	APPROVED
David E. Donley	J. L. Kimball	[illegible]

KNOXVILLE 11-16-38 W PP O 1A127R0



NOTE:
The Time-Area-Depth curves shown represent storms which have occurred over areas in eastern and northeastern United States, and which have run-off characteristics similar to those of the Upper Tennessee River basin. These curves therefore give an indication of the rainfall which may reasonably be expected to occur over this area. Storms occurring along the South Atlantic and the Gulf of Mexico coasts have been excluded.
Data for these curves have been taken from published data of the Miami Conservancy District, the US Weather Bureau and the TVA.

REV. NO.	DATE	MADE	CHKD	SUPV.	INSP.
2	1-15-40	T.B.	R.L.M.	D.E.D.	
2 & 4 day 1937 storm added.					
1	6-29-39	R.L.M.		D.E.D.	
Curves added.					

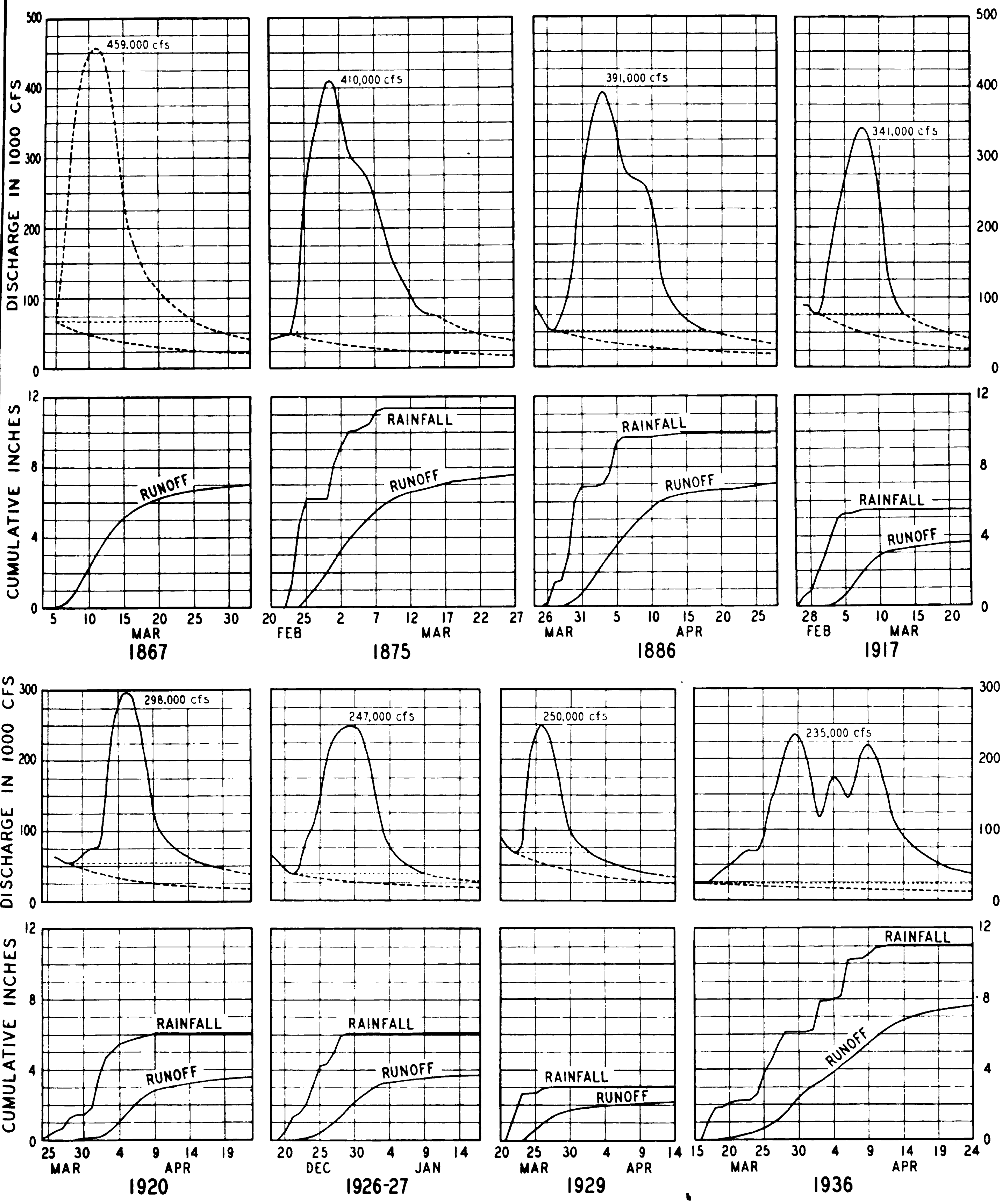
DRWN. W.A.B. COMPUTED
TRCD. J.L. ENGINEER M. Bowmaster
CHKD.

TIME-AREA-DEPTH CURVES
EASTERN UNITED STATES

FLOOD CONTROL INVESTIGATIONS
TENNESSEE VALLEY AUTHORITY
WATER CONTROL PLANNING DEPARTMENT

SUBMITTED	RECOMMENDED	APPROVED
David E. Donley	J. L. Kimball	T. B. Parker

KNOXVILLE | 8-31-38 | W | PP | 0 | 1A33 R2



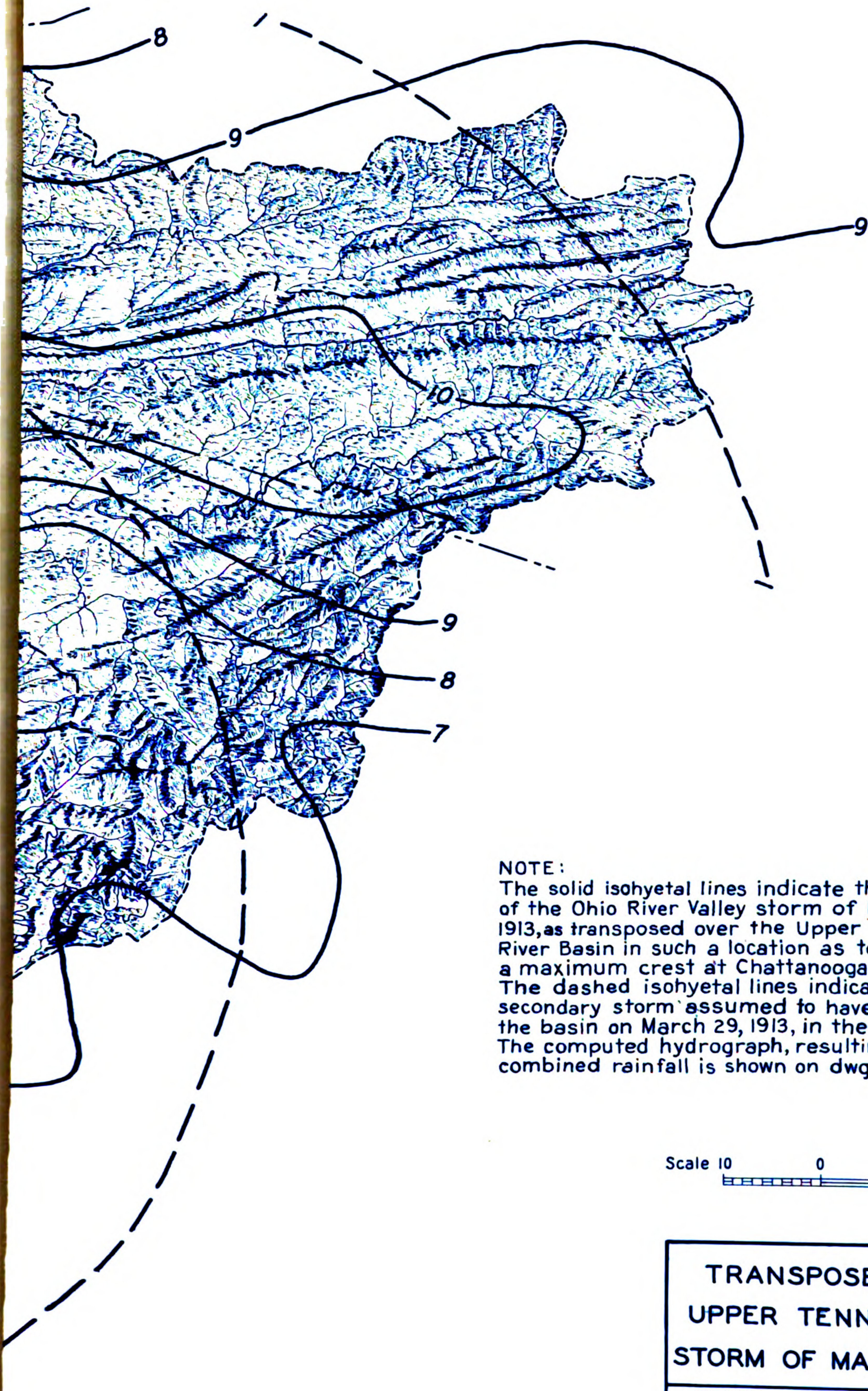
RAINFALL AND RUNOFF GRAPHS PRINCIPAL FLOODS AT CHATTANOOGA, TENN

FLOOD CONTROL INVESTIGATIONS
TENNESSEE VALLEY AUTHORITY
WATER CONTROL PLANNING DEPARTMENT

SUBMITTED	RECOMMENDED	APPROVED
David E. Donley	J. [illegible]	[illegible]

KNOXVILLE	11-10-38	W	PP	O	1A124RO

DRWN	COMPUTED
TRCD.	ENGINEER
CHKD.	[illegible]



NOTE:
The solid isohyetal lines indicate the position of the Ohio River Valley storm of March 22-27, 1913, as transposed over the Upper Tennessee River Basin in such a location as to produce a maximum crest at Chattanooga, Tenn.
The dashed isohyetal lines indicate a hypothetial secondary storm assumed to have occurred over the basin on March 29, 1913, in the position shown.
The computed hydrograph, resulting from the combined rainfall is shown on dwg. W-PP-0-1A63

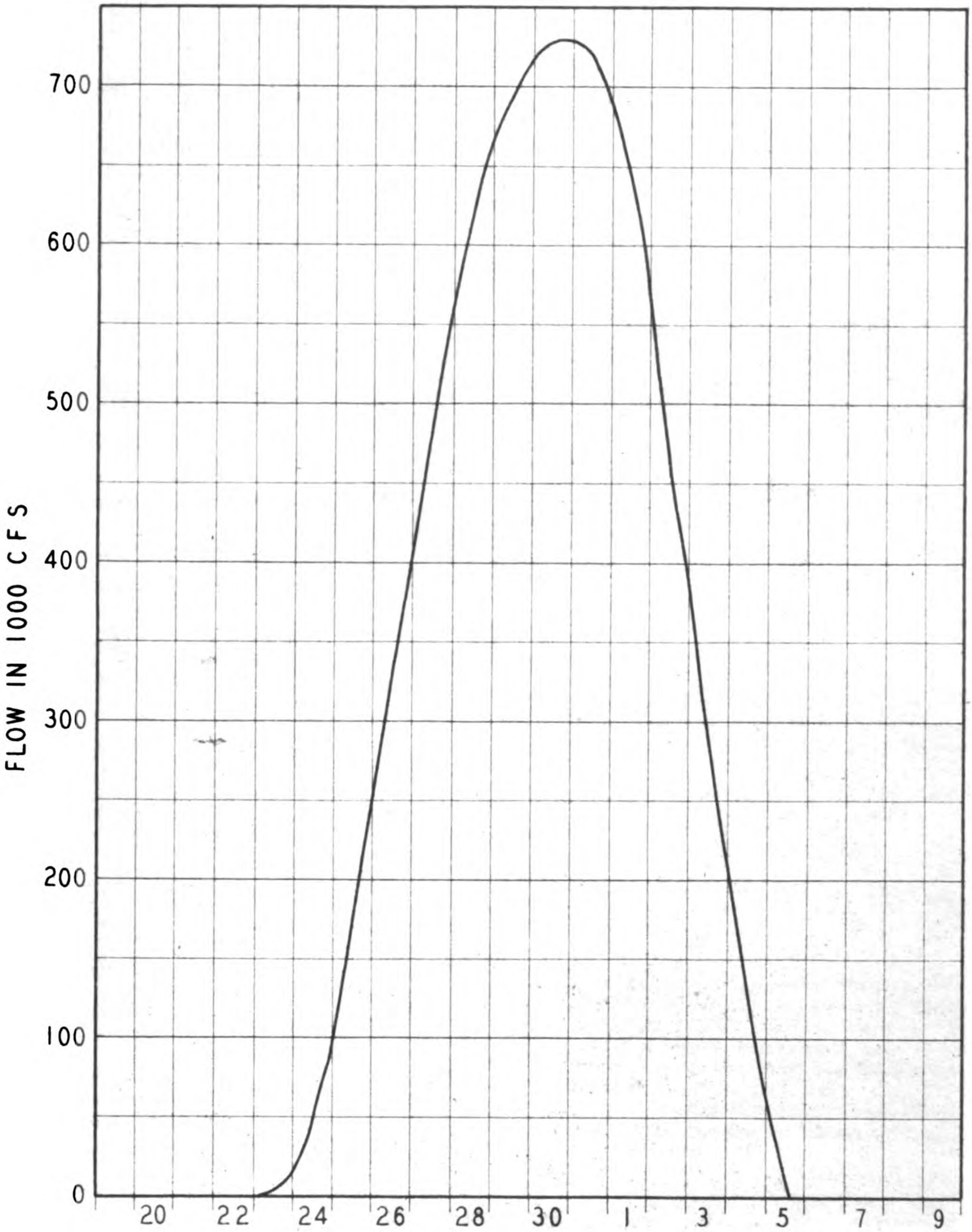
Scale 10 0 10 20 Miles

TRANSPOSED POSITION
UPPER TENN. RIVER BASIN
STORM OF MARCH 22-27, 1913

FLOOD CONTROL INVESTIGATIONS
TENNESSEE VALLEY AUTHORITY
WATER CONTROL PLANNING DEPARTMENT

SUBMITTED	RECOMMENDED	APPROVED
David E. Donley	J. H. Kimball	

KNOXVILLE	9-20-38	W	PP	0	1C54R



NOTE:
Ohio River Valley storm of March 22-27, 1913 assumed applied to the Upper Tennessee River Basin in position shown on dwg No. W-PP-0-1C54 together with a one-day hypothetical secondary storm assumed to occur two days after the above storm and in the position shown on the same drawing.
A 90% runoff factor was assumed and the crest reduced to a discharge in cfs equal to 5000 √Drainage Area (sq mile).
The drainage area above Chattanooga is 21,400 sq miles.

REV. NO.	DATE	MADE	CHKD	SUPV.	INSP.

DRWN. RLM COMPUTED
TRCD. DJF
CHKD. ENGINEER R L Mitchell

ESTIMATED HYDROGRAPH
TENNESSEE RIVER AT
CHATTANOOGA, TENN
TRANSPOSED 1913 STORM

FLOOD CONTROL INVESTIGATIONS
TENNESSEE VALLEY AUTHORITY
WATER CONTROL PLANNING DEPARTMENT

SUBMITTED David E. Donley RECOMMENDED J. L. Kimball APPROVED

KNOXVILLE 9-30-38 W PP 0 1A63 R

NOTE:
The isohyetal lines shown indicate the position of the Ohio River Valley storm of January 12-25, 1937 as transposed over the Upper Tennessee Basin in such a location as to produce a maximum crest at Chattanooga, Tenn.
The computed hydrograph resulting from this rainfall is shown on dwg. W-PP-0-1A64.

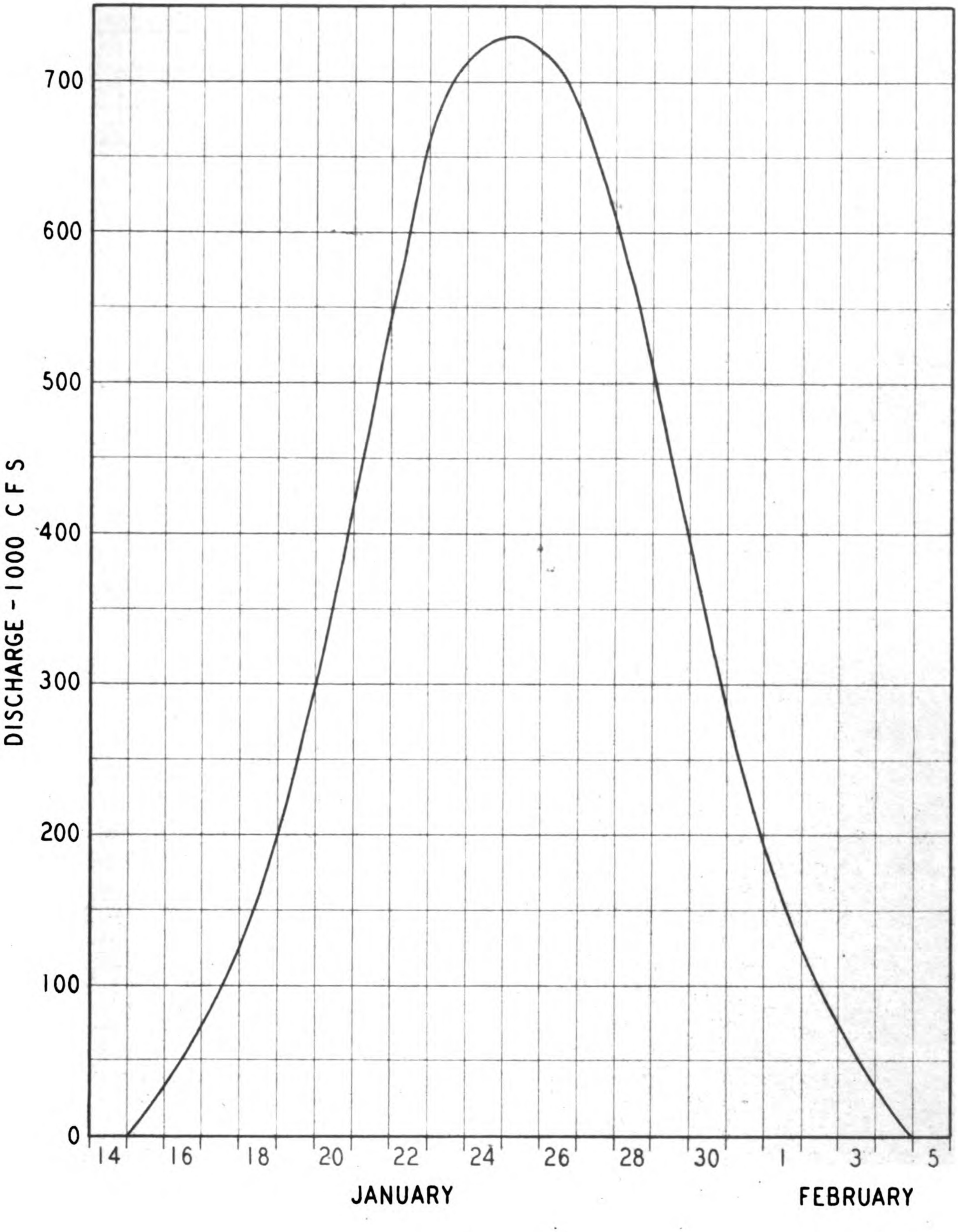
Scale 10 0 10 20 Miles

TRANSPOSED POSITION
UPPER TENN. RIVER BASIN
STORM OF JANUARY 12-25, 1937

FLOOD CONTROL INVESTIGATIONS
TENNESSEE VALLEY AUTHORITY
WATER CONTROL PLANNING DEPARTMENT

SUBMITTED	RECOMMENDED	APPROVED
KNOXVILLE	9-20-38 W PP 0	IC56R

REV. NO. DATE
DRWN
TRCD L.M.
CHKD



NOTE:
Ohio River Valley storm of January 12-25, 1937 assumed applied to the Upper Tennessee River Basin in position shown on dwg No. W-PP-0-1C56.
A 90% runoff factor was assumed and the crest reduced to a discharge in cfs equal to 5000 $\sqrt{\text{Drainage Area (sq mile)}}$.
The drainage area above Chattanooga is 21,400 sq miles.

REV. NO.	DATE	MADE	CHKD.	SUPV.	INSP.

DRWN. RLM COMPUTED
TRCD. DJF ENGINEER R L Mitchell
CHKD.

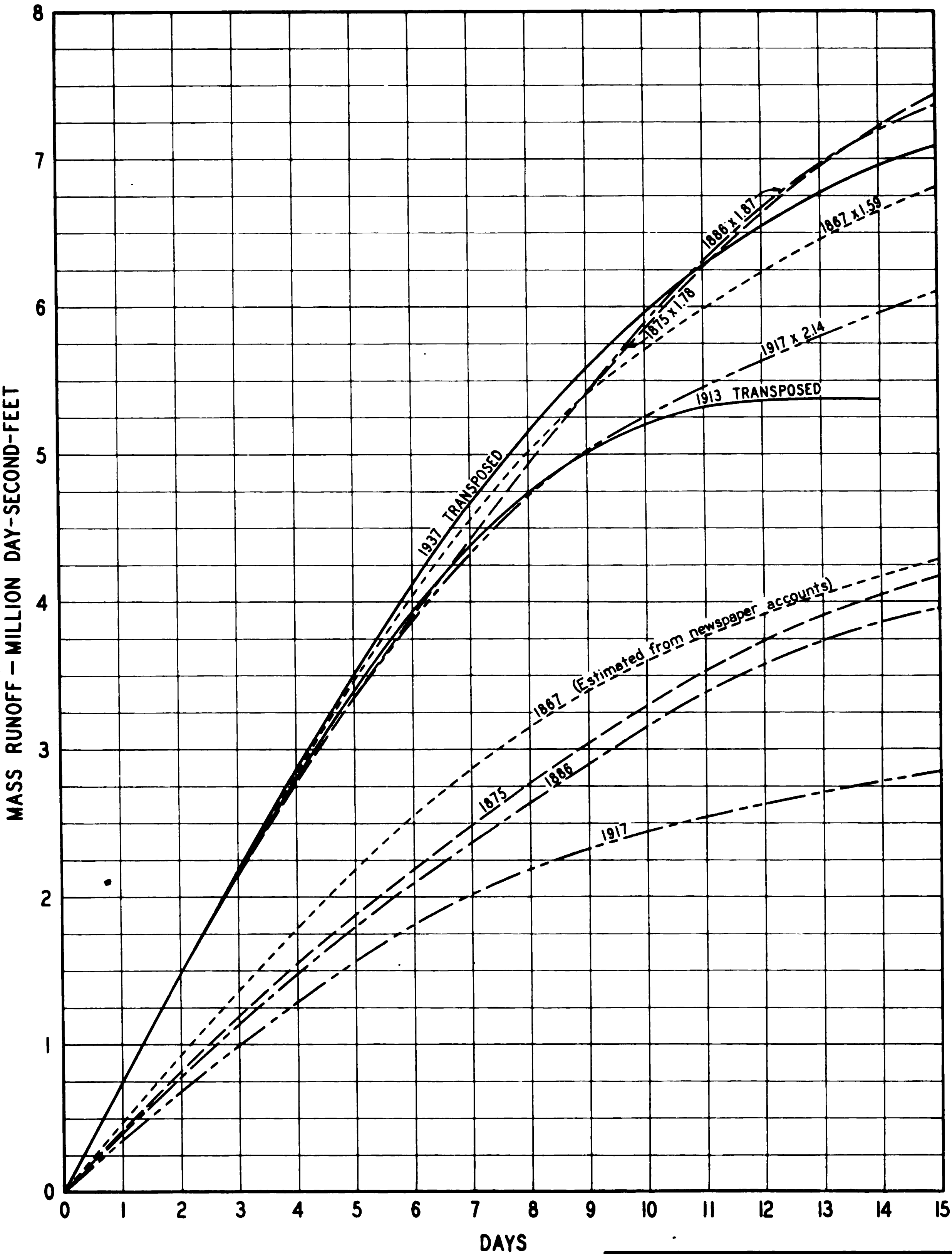
ESTIMATED HYDROGRAPH TENNESSEE RIVER AT CHATTANOOGA, TENN TRANSPOSED 1937 STORM

FLOOD CONTROL INVESTIGATIONS
TENNESSEE VALLEY AUTHORITY
WATER CONTROL PLANNING DEPARTMENT

SUBMITTED	RECOMMENDED	APPROVED
David E. Donley	J. H. Kimball	

KNOXVILLE	9-29-38	W	PP	0	1A64 R

FU 1-20.32



NOTE:
The mass-duration curve for each flood was obtained by summing, in order of magnitude, the daily flows downward from the crest of the flood. Mass-duration curves for the floods of 1867, 1875, 1886 and 1917 are shown as determined from actual flows, and from the actual daily flows in each flood increased by the ratio of the maximum assumed flood crest of 730,000 cfs to the actual crest.

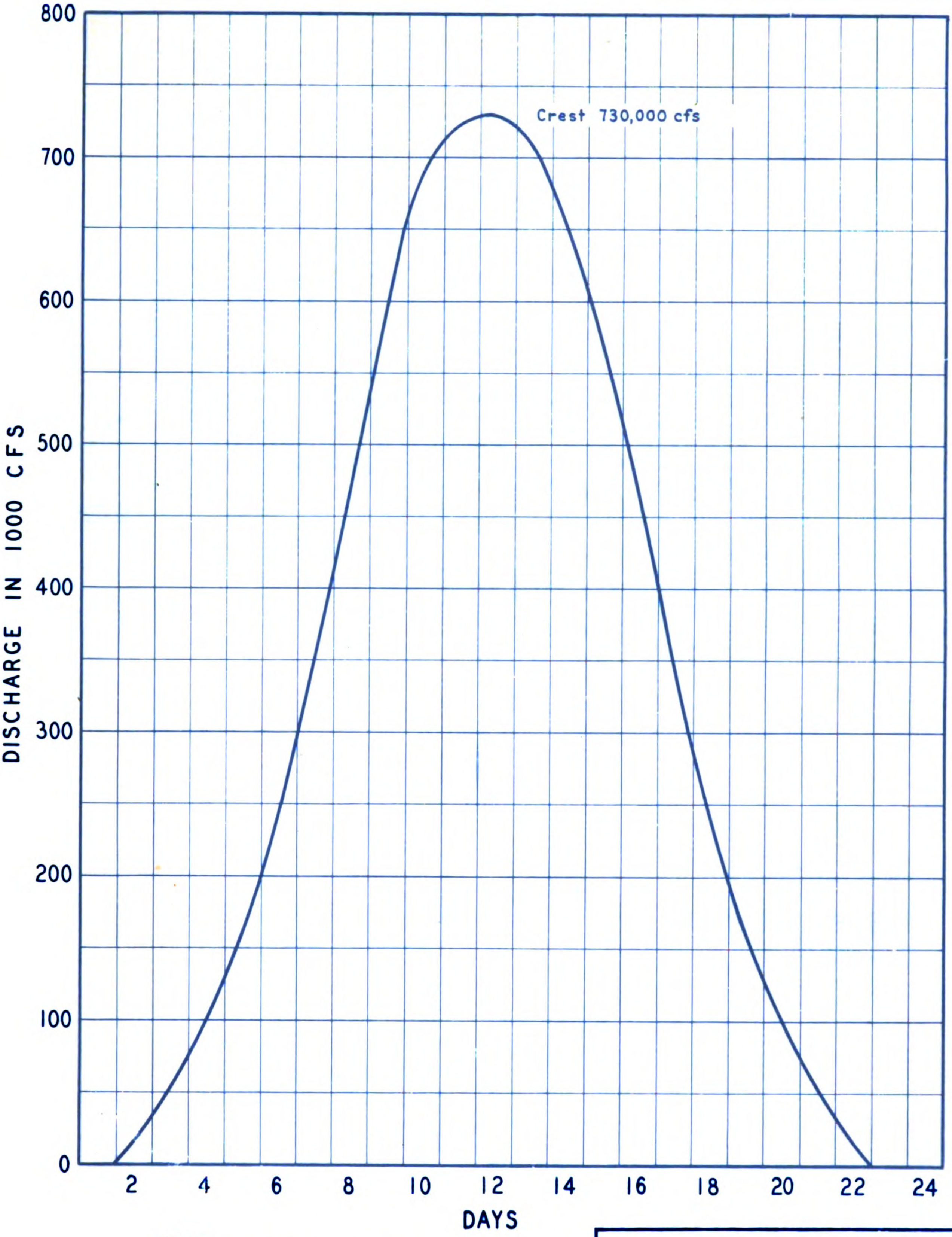
MASS-DURATION CURVES HISTORICAL AND HYPOTHETICAL FLOODS – TENN. RIVER AT CHATTANOOGA, TENN.

FLOOD CONTROL INVESTIGATIONS
TENNESSEE VALLEY AUTHORITY
WATER CONTROL PLANNING DEPARTMENT

SUBMITTED	RECOMMENDED	APPROVED
David B. Donley	J.K. Kimball	T.B. Parker
KNOXVILLE	9-28-38 W PP 0	1A51 R

DRWN. WAB
TRCD. L.H.D.
CHKD.
COMPUTED 10.E.O.
ENGINEER McAlister

32



NOTE:
It is assumed that a flood of this magnitude may occur throughout the flood season with its crest not later than March 15. Substantial floods of somewhat less magnitude may occur as late as April 15.

REV. NO.	DATE	MADE	CHKD	SUPV.	INSP.

DRWN W.A.B. | COMPUTED
TRCD C.L.R. | ENGINEER R.L. Mitchell
CHKD | W.A. Baumeister

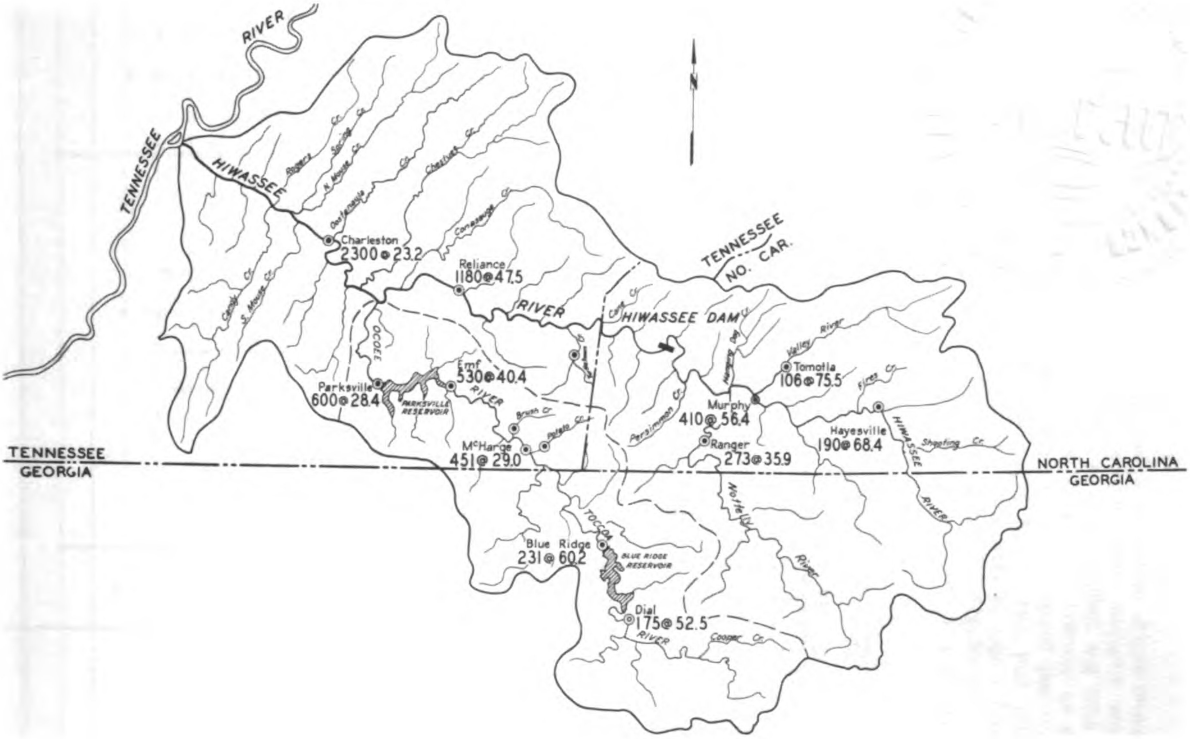
HYDROGRAPH
MAXIMUM ASSUMED FLOOD
TENNESSEE RIVER
AT CHATTANOOGA TENN

FLOOD CONTROL INVESTIGATIONS
TENNESSEE VALLEY AUTHORITY
WATER CONTROL PLANNING DEPARTMENT

SUBMITTED	RECOMMENDED	APPROVED
David E. Donley	J.H. Kimball	A.O. Monroe

KNOXVILLE	9-20-38	W	PP	0	1A53R0

FU 25-50.03



MAXIMUM RECORDED RUN-OFF RATES			
STATION	DRAINAGE AREA SQ. MILES	PEAK FLOW C.F.S. PER SQ. MILE	DATE OF OCCURRENCE
Charleston	2,300	23.2	4 - 3 - 20
Reliance	1,180	47.5	11 - 19 - 06
Murphy	410	56.4	3 - 19 - 99
Hayesville	190	68.4	12 - 7 - 22
Tomotla	106	75.5	2 - 3 - 36
Ranger	273	35.9	2 - 22 - 02
Parksville	600	28.4	7 - 10 - 16
Emf	530	40.4	7 - 10 - 16
McHarge	451	29.0	1 - 21 - 22
Dial	175	52.5	7 - 9 - 16
Blue Ridge	231	60.2	7 - 9 - 16

Maximum length of record = 39 years at Murphy
Average length of record = 18 years
Low rate at McHarge is due to station being established in May 1917 and therefore no record is available for 1916 flood.

LEGEND

● Stream Gaging Stations

Note:
Maximum recorded flood rates shown were taken from Bulletin No. 34 "Water Resources of Tennessee" and published records of the U.S. Geological Survey.
The drainage area and maximum recorded run-off per square mile at each gaging station are shown thus: 2300 @ 23.2

Scale 5 0 5 10 Miles

MAXIMUM RECORDED RUN-OFF

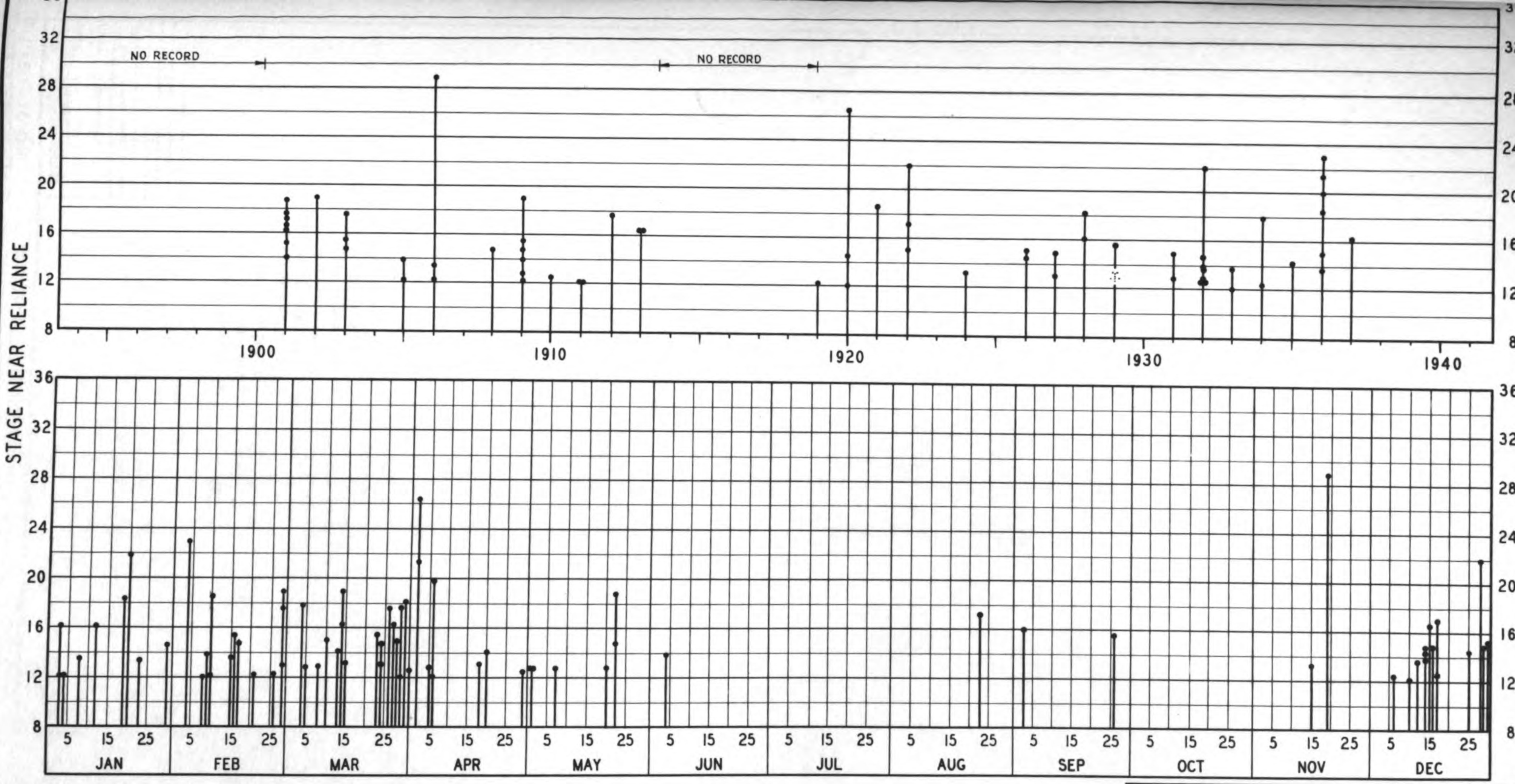
HIWASSEE-OCOEE RIVER BASINS
TENNESSEE VALLEY AUTHORITY
WATER CONTROL PLANNING DEPARTMENT

SUBMITTED	RECOMMENDED	APPROVED
J. A. Kimball	Jas. S. Bowman	R. A. Monroe

KNOXVILLE	8-16-37	5	GP	9	5A6 RO

REV. NO.	DATE	MADE	CHKD.
DRWN. NED			
TRCD. L.L.R.			
CHKD. DED			

PREPARED BY David E. Wooley



Gage Zero :
Elevation 718.34 (1936 Supp. Adj.)

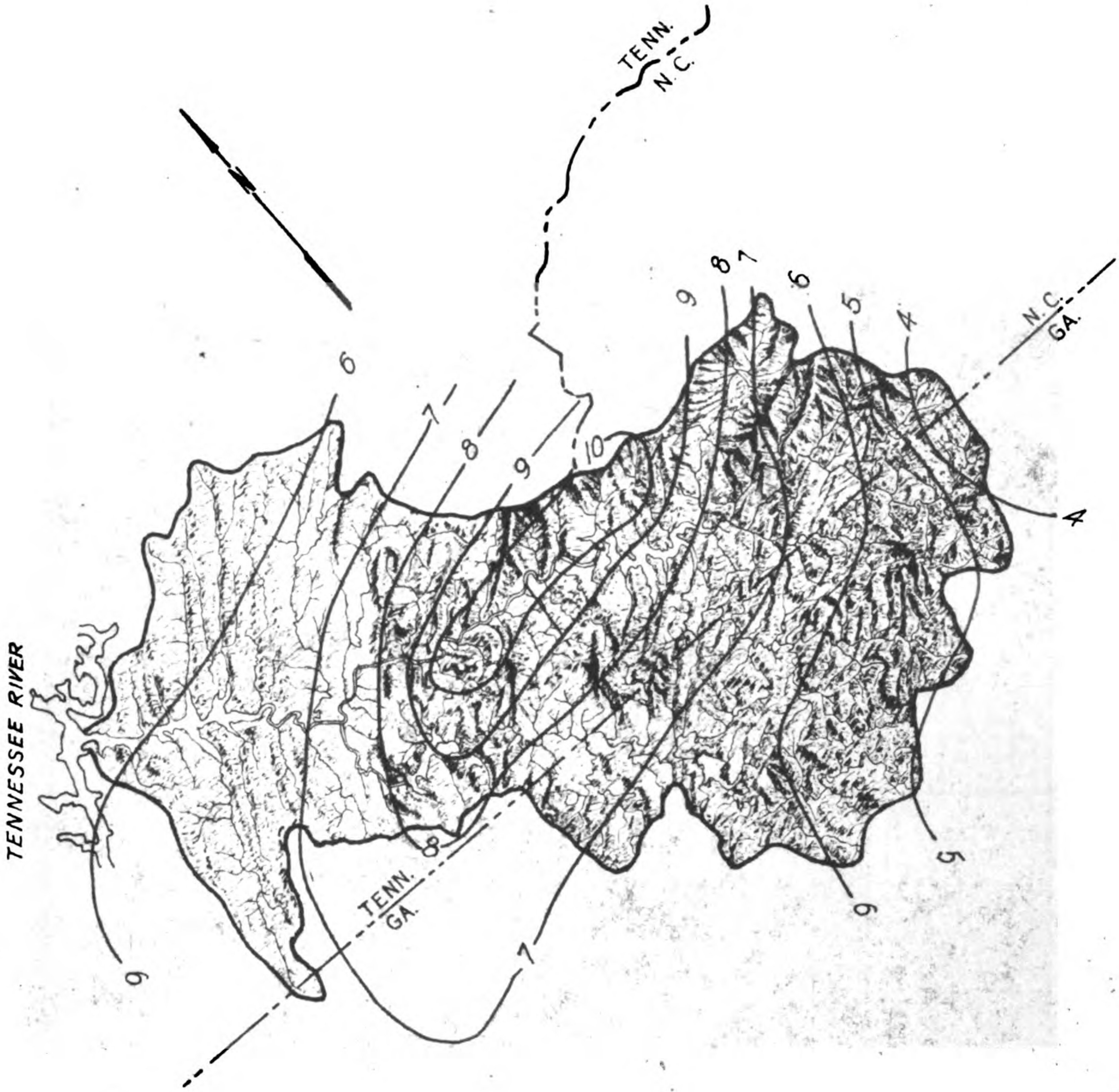
NOTE :
The stages shown apply to the present gage of the U.S. Geological Survey near Reliance, Tennessee, which has been operated since October 1, 1926. From August 19, 1900 to December 31, 1913 and from February 1, 1919 to September 30, 1926, stages were read on a gage three miles upstream and transferred to the present gage by comparison of the rating curves at the two points. Stages at Apalachia, 16 miles upstream, are available for the period of January 1, 1914 to December 31, 1922, but this information has been excluded from the diagram because of the uncertain relationship between stages at the two stations. Crest stages from recorder charts and published records are shown as far as available, all other stages being daily reading of a staff gage.

DISTRIBUTION OF FLOODS
HIWASSEE RIVER
NEAR RELIANCE, TENN

FLOOD CONTROL INVESTIGATIONS
TENNESSEE VALLEY AUTHORITY
WATER CONTROL PLANNING DEPARTMENT

SUBMITTED	RECOMMENDED	APPROVED
David E. Donley	J. H. Kimball	R.A. Monroe
KNOXVILLE	6-13-38 W GP O	5A9 RO

DRWN. KWK	COMPUTED KWK & WHS
RCD. R.C.G.	ENGINEER WOB
CHKD. WHS	



NOTE :
The isohyetals shown indicate the position of the East Tennessee storm of March 22-23, 1929 as transposed over the Hiwassee River Basin in such a location as to produce maximum crests at Hiwassee Dam and at mouth of Hiwassee River.
The computed hydrographs resulting from this rainfall are shown on dwg 5-GP-0-5A12. & W-PP-1-311A20.

Due to lack of satisfactory Topographic Maps relief shown was prepared from Aerial Mosaics with control from Planimetric Sheets prepared by TVA.

Scale 10 0 10 20 Miles

REV. NO.	DATE	MADE	CHKD.	SUPV.	INSP.

DRWN. COMPUTED
TRCD.
CHKD. ENGINEER R. A. Mitchell

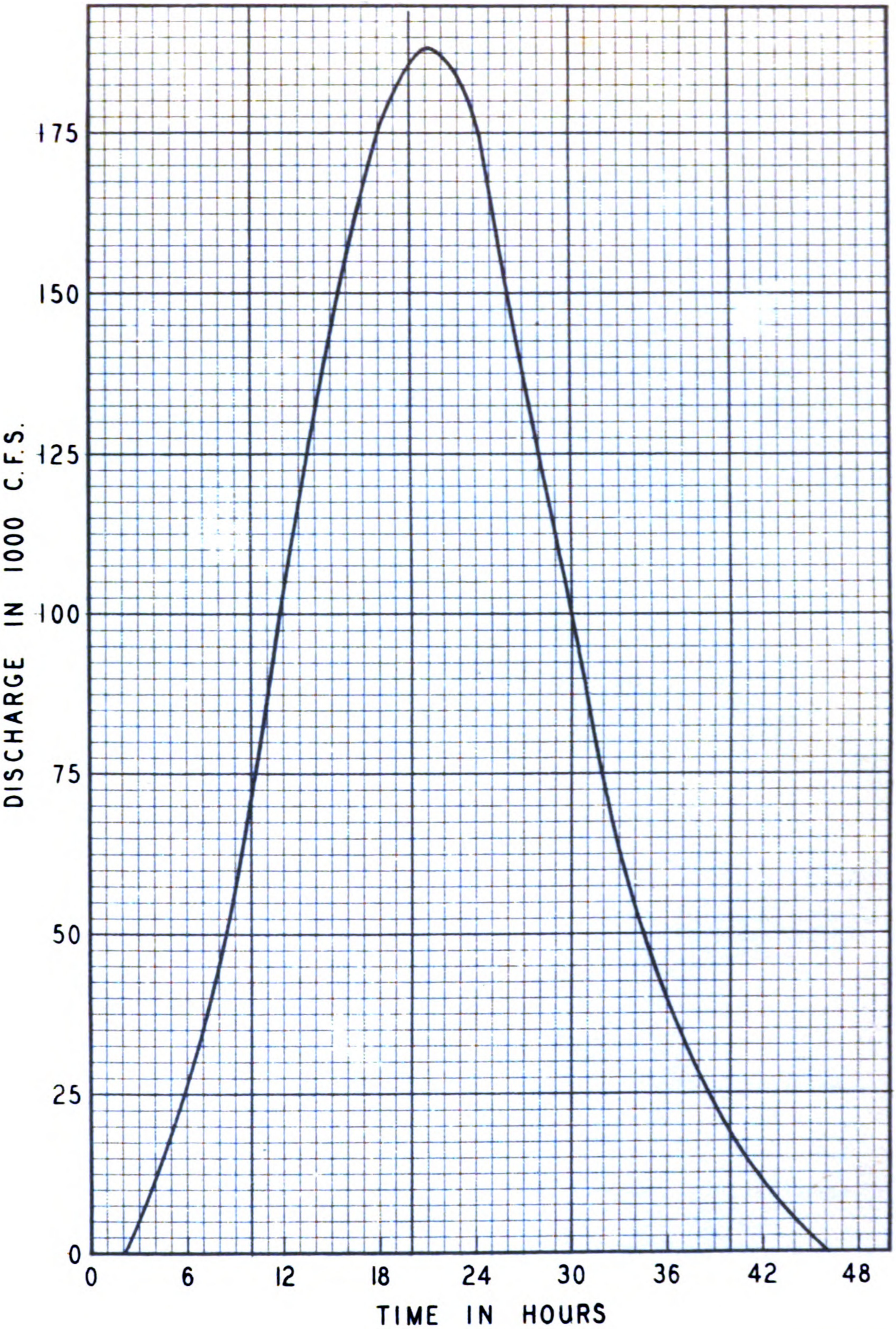
TRANSPOSED POSITION
STORM OF MARCH 22-23 1929
HIWASSEE RIVER BASIN

FLOOD CONTROL INVESTIGATIONS
TENNESSEE VALLEY AUTHORITY
WATER CONTROL PLANNING DEPARTMENT

SUBMITTED	RECOMMENDED	APPROVED
David E. Donley	J. H. Kimball	

KNOXVILLE 7-5-39 W PP 1 322A8 RO

FU 150-10.02
FU 153-10.01



NOTE:
Eastern Tennessee storm of March 22-23, 1929, assumed applied to the Hiwassee River basin in position on dwg W-PP-1-322A8
A 90% runoff factor was assumed and the crest reduced to discharge in cfs equal to $6000\sqrt{\text{Drainage Area (sq miles)}}$
Drainage area at this point is 977 sq miles.

Supersedes 5-GP-9-5A7 RO

REV NO.	DATE	MADE	CHKD	SUPV.	INSP.

DRWN. RLM COMPUTED R.L.M.
TRCD. LHD ENGINEER R.L. Mitchell
CHKD.

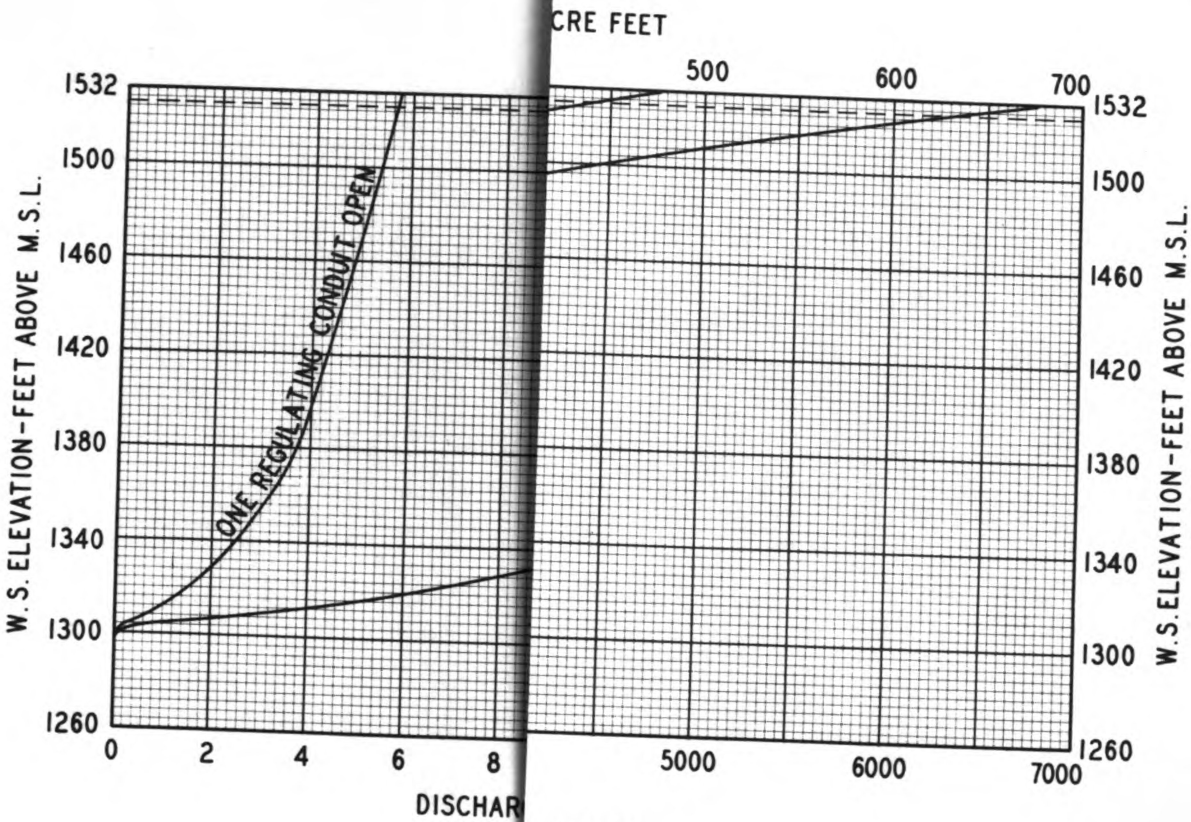
MAXIMUM ASSUMED FLOOD
HIWASSEE RIVER
HIWASSEE DAM N. C.

FLOOD CONTROL INVESTIGATIONS
TENNESSEE VALLEY AUTHORITY
WATER CONTROL PLANNING DEPARTMENT

SUBMITTED	RECOMMENDED	APPROVED
David E. Donley	J.A. Kimball	

KNOXVILLE	10-10-38	5	GP	0	5A12R

FU 153-10.01



EQUATIONS AND ASSUMED LOSSES:

SPILLWAY

$$Q = \frac{CLH^{1.62}}{D^{0.12}} = 569.36H^{1.62}$$ (R.N. Brudenell, Univ. of Iowa)

Where

C = 3.80 This value determined and the computed discharge checked by model tests in the T.V.A. Hydraulic Laboratory.

L = Length of Crest = 224'

D = Design Head on Crest = 28.5'

H = Actual Head on Crest

REGULATING CONDUIT

Entrance Loss = $0.05\frac{V^2}{2g}$

Friction Loss = $\frac{2.87n^2 L V^2}{D^{4/3}}$ (Manning's Formula)

Nozzle Loss = $\left(\frac{1}{C_v^2}-1\right)\frac{V_1^2}{2g}$

Where

V = Velocity in Pipe

V_1 = Velocity at Small End of Nozzle (Dia. = 7'-10")

n = 0.010

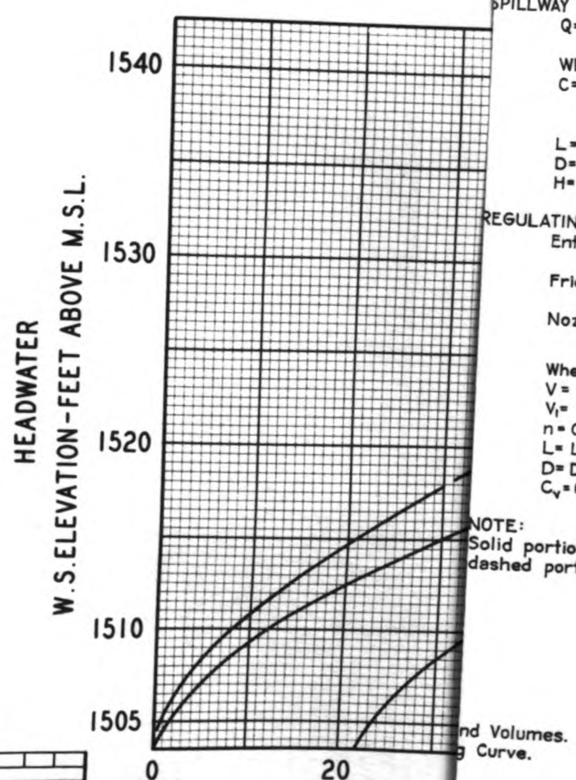
L = Length of Straight Pipe = 180.5'

D = Diameter of Pipe = 8'-6"

C_v = 0.98 (King's Handbook of Hydraulics, Pg. 58)

NOTE:

Solid portion of tailwater rating curve is based on actual gage readings; dashed portion is estimated by area-mean depth method.



nd Volumes.

g Curve.

REV. NO.	DATE	MADE	CHKD.	SUPV.	INSP.	SUBM.	RECM.

DSGN. ---- 11-19-37 | SUPV. H.W.G.

DRWN. A.J.R. 11-9-37 | INSP. W.G.H. 12-13-37

CHKD. H.V.J. 12-4-37

TRCD. J.Y.C. & J.B.

COMP. H.V.J. 1-12-38 | ENGINEER W.G. Huber

DAM

HYDRAULICS

AREA, CAPACITY & DISCHARGE CURVES

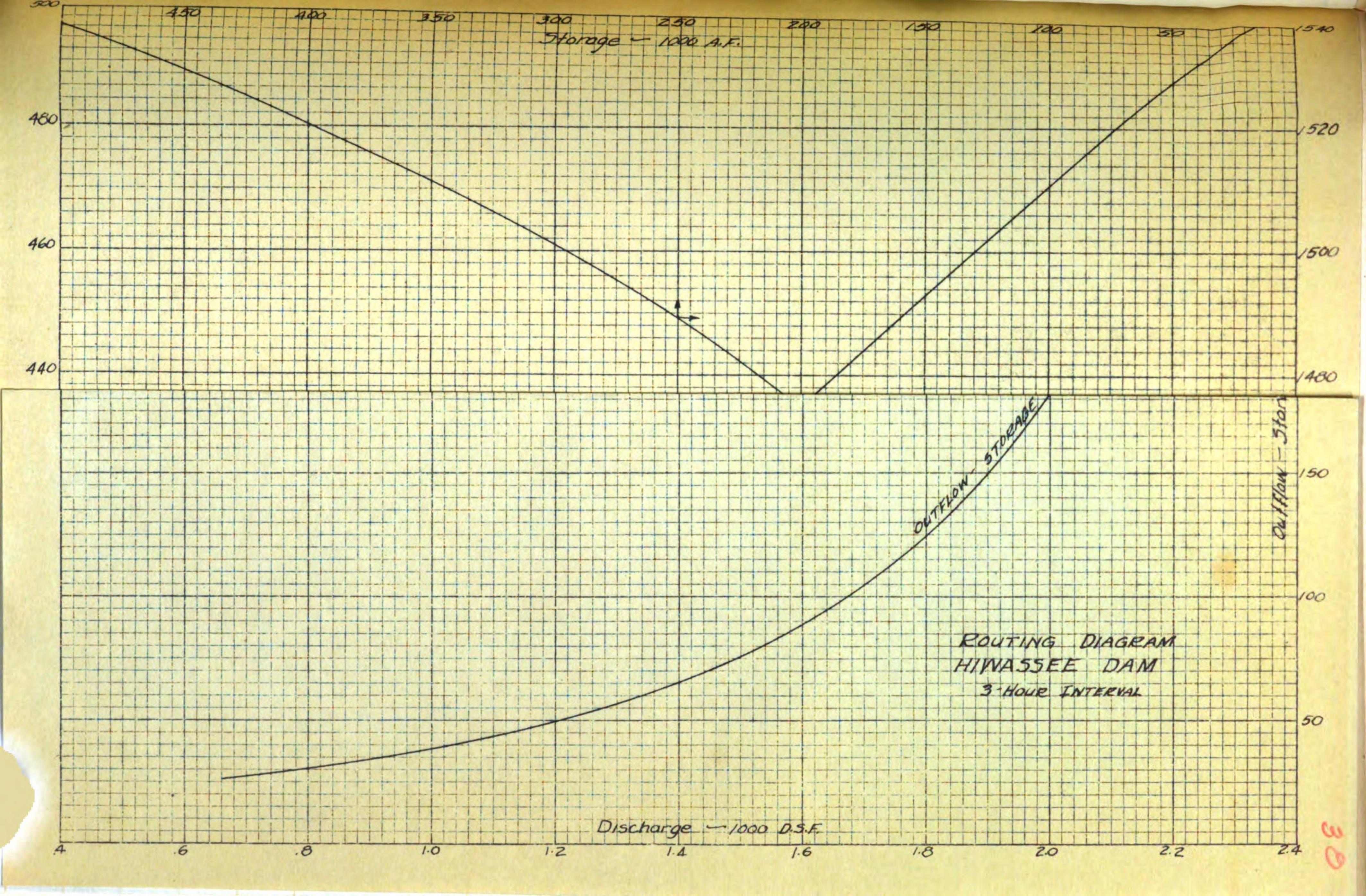
HIWASSEE PROJECT

TENNESSEE VALLEY AUTHORITY

ENGINEERING DESIGN DEPARTMENT

SUBMITTED	RECOMMENDED	APPROVED
Ross McDegal		

KNOXVILLE	12-19-37	5	C	4	10K214R


ROUTING DIAGRAM
HIWASSEE DAM
3-HOUR INTERVAL
Storage ~ 1000 A.F.
Discharge ~ 1000 D.S.F.
Outflow - Stor
OUTFLOW - STORAGE
500
450
400
350
300
250
200
150
100
50
480
460
440
1540
1520
1500
1480
150
100
50
.4
.6
.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
30

ROOM USE ONLY