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SIZING OF HIGHWAY CULVERTS AND BRIDGES: A HISTORICAL REVIEW OF METHODS AND CRITERIA

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16 Abstract

This report examines the methods and criteria that highway engineers have used to size culverts and bridges in Kansas and throughout the U. S. from the start of highway construction to the present. The focus is directed to methods actually employed by highway engineers rather than methods described in the research literature. Hydrologic methods are categorized as early or modern, according to whether a specific recurrence interval is associated with the design. Modern hydrologic methods are based on statistical analyses of systematic records of streamflow or rainfall data; the desired recurrence interval is an input to the design.

In the early days, culverts and bridges were sized by empirical methods developed from experiences with existing structures during floods. Most of these methods were developed by or for the railroads. No particular recurrence intervals were associated with the resulting designs. Early highway engineers were aware of the shortcomings of these design methods, but they were hampered by a shortage of reliable streamflow data and rainfall data. The transition to modern frequency-based design methods generally occurred during the 1950s.

The highway-building era in Kansas began in 1917 with the creation of the Kansas State Highway Commission (KSHC), the predecessor of the current Kansas Department of Transportation (KDOT, since 1975). Prior to the mid-1950s, most culverts and bridges on Kansas highways were sized with the Talbot formula, Dun's table and other empirical methods. KSHC and KDOT have employed frequency-based design methods such as the Rational method and USGS regression equations since the 1960s. Highway culverts and bridges have been designed for recurrence intervals of 25 years or greater over this period. The hydrologic methods and design guidelines employed by KSHC and KDOT have been within the mainstream of highway engineering practice nationwide. Hydrologic methods have been improved as more streamflow data have become available. However, flood frequency estimates for small watersheds still have large standard errors.

The federal government has specified a minimum recurrence interval of 50 years for culverts and bridges on Interstate highways since 1956. However, FHWA and its predecessors have never specified hydrologic design criteria for structures on non-Interstate highways.

The engineering professions understanding of culvert and bridge hydraulics has advanced greatly over the last century. The Talbot formula, Dun's table and similar empirical design methods did not explicitly consider the hydraulic characteristics of the structure. Modern design methods require hydraulic analyses of proposed designs. A series of technical reports published by U.S. Bureau of Public Roads in the 1960s provided highway engineers with practical guidance on the hydraulic aspects of culverts and bridges.

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Final Report

Prepared by

Bruce M. McEnroe

A Report on Research Sponsored By

THE KANSAS DEPARTMENT OF TRANSPORTATION
TOPEKA, KANSAS

and

UNIVERSITY OF KANSAS CENTER FOR RESEARCH, INC.
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PREFACE

The Kansas Department of Transportation's (KDOT) Kansas Transportation Research and New-Developments (K-TRAN) Research Program funded this research project. It is an ongoing, cooperative and comprehensive research program addressing transportation needs of the state of Kansas utilizing academic and research resources from KDOT, Kansas State University and the University of Kansas. Transportation professionals in KDOT and the universities jointly develop the projects included in the research program.

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Chapter 1

Introduction

This report examines the methods and criteria that highway engineers have used to size culverts and bridges in Kansas and throughout the U. S. from the early days of highway construction to the present. The focus is directed to methods actually employed by highway engineers rather than methods described in the research literature. Hydrologic methods are categorized as early or modern, according to whether a specific recurrence interval is associated with the design. Modern hydrologic methods are based on statistical analyses of systematic records of streamflow or rainfall data; the desired recurrence interval is an input to the design. Early methods, developed before systematic data records were available, yielded designs with no associated recurrence interval. Chapters 2 and 3 survey the early and modern methods from a nationwide perspective. Chapter 4 provides an overview of developments in culvert and bridge hydraulics. Chapter 5 reviews past and current federal criteria relevant to the sizing of culverts and bridges. Chapter 6 examines the design practices employed by the Kansas State Highway Commission (KSHC) and the Kansas Department of Transportation (KDOT) on Kansas highways.

Chapter 2

Early Methods

2.1 Overview

In the earliest days of road-building in the United States, drainage structures were necessarily sized by judgment. Over time, engineers developed design aids based on the observed performance of existing structures and, in some cases, limited hydrologic data. Railroad engineers, in particular, developed and published numerous tables and formulas for waterway sizing. A comprehensive report on waterway sizing published by the American Railroad Engineering and Maintenance of Way Association (AREMWA) in 1911 presents six formulas for waterway area and 21 formulas for design discharge. A historical review of waterway sizing methods by V. T. Chow (1962) lists 12 formulas for waterway area and 62 formulas for design discharge. However, only a few of these formulas were ever widely employed by American highway engineers. The most popular early methods for hydrologic design of drainage structures included Dun’s table, the Myers and Talbot formulas for waterway area, and the Burkli-Ziegler formula for discharge.

Despite the shortage of reliable hydrologic data, the hydrologic and hydraulic factors that affect the sizing of drainage structures appear to have been well understood. One early textbook on highway design listed these factors as follows:

The area of the waterway required depends (1) upon the rate of rainfall; (2) the kind and condition of the soil; (3) the character and inclination of the surface; (4) the condition and inclination of the bed of the stream; (5) the shape of the area to be drained, and the position of the branches of the stream; (6) the form of the mouth and the inclination of the bed of the culvert; and (7) whether it is permissible to back water up above the culvert, thereby causing it to discharge under a head. (Byrne, *A Treatise on Highway Construction*, 4th ed., 1902)

An explanation of the roles played by these seven factors is followed by a sensible recommendation for the sizing new drainage structures:

Valuable data on the proper size of any particular culvert may be obtained (1) by observing the existing openings on the same stream; (2) by measuring, preferably at time of high water, a cross-section of the stream at some narrow place; and (3) by determining the height of high water as indicated by drift and the evidence of the inhabitants of the neighborhood. With these data and a careful consideration of the various matters referred to [in the previous quotation], it is possible to determine the proper area of the water-way with a reasonable degree of accuracy. (Byrne, 1902)

2.2 Flood Magnitude and Economics

Early highway engineers differed in their opinions on the acceptable frequency of roadway flooding. One of the earliest American textbooks on road-building stated that culverts should be sized for the worst-case scenario.

Their size must be proportioned to the greatest quantity of water which they can ever be required to pass, and should be at least 18 inches square, or large enough to admit a boy to enter to clean them out. (Gillespie, *A Manual of the Principles and Practices of Roadmaking*, 6th edition, 1853)

However, most early textbooks on highway engineering advocated consideration of the economic trade-offs in the sizing of culverts and bridges. The following statements are representative.

Especial care is required to provide an ample way for the water to be passed. If the culvert is too small, it is liable to cause a washout, entailing interruption of traffic and cost of repairs, and possibly may cause accidents that will require the payment of large sums for damages. On the other hand, if the culvert is made unnecessarily large, the cost of construction is needlessly increased. Any one can make a culvert large enough, but it is the province of the engineer to design one of sufficient but not extravagant size. (Byrne, *A Treatise on Highway Construction*, 4th edition, 1902)

The economy of designing a bridge or culvert to take the maximum discharge from an area should be determined. . . . It would be economical to build the structure to meet maximum conditions if the interest on the first cost was less than the cost to repair whatever damage was incurred by the use of a structure furnishing a smaller waterway. Where a loss of life would be involved, however, the structure should be designed to meet maximum conditions. (Blanchard and Drowne, Text-Book on Highway Engineering, 1913)

The first step to take is to decide on what magnitude of flood should be provided for. Extreme floods may occur but once or twice in a century; and the cost of caring adequately for such a contingency is excessive and unwarranted in many instances. Here the best judgment of the engineer will be needed; for the temptation will be to use too rigidly the principle that the loss due to the extreme flood is justified if it does not exceed the capitalized cost of the additional waterway necessary to prevent it. The difficulty in applying this principle is to foresee all the items that will enter into some future loss, and thus arrive at a true aggregate. The engineer should be liberal in assuming the magnitude of the discharge to be provided for as well as in forecasting the probable amount of damage that in the future might be caused by an abnormally great flood. (Waddell, Bridge Engineering, 1916)

It may not always be wise to provide for extreme cases of high water that have occurred only once in a generation; it may be cheaper to risk the washing out of a road or culvert." (Chatburn, Highway Engineering, 1921).

2.3 Dun's Table

The best-known table for sizing of waterway openings was developed by James Dun, the chief engineer of the Atchison, Topeka and Santa Fe Railway. Dun created the first version of his table in the early 1890s and revised it several times after floods on the AT&SF railway system. Dun and others published the final version, shown as Table 1, in the *Journal of the Western Society of Civil Engineers* in 1906 (Bremner et al., 1906). Dun noted that the recommended waterway areas for the region that includes all of Kansas "were prepared from observations of streams in Southwest Missouri, Eastern Kansas, Western Arkansas and the southeastern portions of the Indian Territory [Oklahoma]." He also noted that the table "indicates larger waterways than are required in Western Kansas and level portions of Missouri, Colorado, New Mexico and Western Texas."

Dun's table was widely adopted by highway engineers in the plains states, where it remained popular through the start of the modern era. A 1953 survey of design practices by the University of Illinois (Chow, 1962) found that the highway departments of five plains states, including Kansas, listed Dun's table as an acceptable design method.

2.4 Myers Formula

American railroad engineer E. T. C. Myers developed the first formula for waterway area. The Myers formula was first published in the Proceedings of the Engineers Club of Philadelphia in 1879 (Cleeman, 1879). The Myers formula is:

$$A = C\sqrt{D}$$

in which

A = area of waterway (ft2)

D = drainage area (acres)

C = a coefficient recommended to be 1.0 as a minimum for flat country, 1.6 for hilly compact ground, 4.0 as a minimum for mountainous and rocky country, and higher values in exceptional cases.

As Chief Engineer of the Richmond, Fredricksburg and Potomac Railroad in Virginia, Meyers developed his formula from observations of structures in the general vicinity of the railroad line. Following its publication, the Myers formula was "used to a great extent by railroad engineers in the eastern part of the United States" (Blanchard and Drowne, 1913), and was included in several early texts on highway engineering. However, the Myers formula does not appear to have been widely adopted by highway engineers.

2.5 Talbot Formula

In 1887, Professor A. N. Talbot of the University of Illinois formula proposed a new formula for waterway area:

$$A = C\,D^{3/4}$$

in which

A = area of waterway (ft^2)

D = drainage area (acres)

C = a coefficient

Professor Talbot offered the following guidance for selection of the coefficient C:

> I conclude that for rolling agricultural country, subject to floods at the time of melting snow, and with the length of valley three or four times the width, one-

> third is the proper value for C. In districts not affected by snow and where the length of the valley is several times the width, one-fifth or one-sixth or even less may be used. C should be increased for steep side slopes, especially if the upper part of the valley has a much greater fall than the channel at the culvert.
>
> In any case, judgment must be the main dependence, the formula being a guide to it. On a road already constructed the C may be determined for the character of surface along that line by comparing the formula with the high-water mark of a known drainage area. Experience and observation on similar water-courses is the most valuable guide. A knowledge of the action of streams of similar situations in floods and of the effects of peculiar formations is of far more value than any extended formula. (Talbot, 1887-88)

In a subsequent discussion, Talbot added, “For steep and rocky ground C varies from two-thirds to unity.”

The Talbot formula gained widespread popularity among railroad and highway engineers. The 1911 AREMWA report on waterway sizing stated that the Talbot formula had “been very generally adopted, particularly in the West and in the southwestern portion of the country.” The highway departments of 25 states, including Kansas, listed the Talbot formula as an acceptable design method in the University of Illinois’s 1953 survey of design practices (Chow, 1962).

2.6 Burkli-Ziegler Formula

Burkli-Ziegler, a Swiss engineer, published his formula for design discharge in 1880. Hering introduced it to U. S. practice in a paper published in 1891. The Burkli-Ziegler formula is:

$$q = C\,r\sqrt{\frac{S}{A}}$$

in which

q = unit discharge (cfs/acre)

C = a coefficient ranging from 0.31 to 0.75, depending on the nature of the surface; 0.62 is recommended for general use

r = rainfall intensity (in./hr)

S = general grade of the area (ft/1000 ft)

A = drainage area (acres)

The Burkli-Ziegler formula was the most popular of the early formulas for design discharge. The highway department of eight states listed the Burkli-Ziegler formula as an acceptable design method in the University of Illinois's 1953 survey of design practices (Chow, 1962). However, there is no indication that it was ever widely used in Kansas. Originally developed for urban drainage applications, the Burkli-Ziegler formula was not well suited for highway engineering in the rural Midwestern states. Highway engineering texts that presented the Burkli-Ziegler formula generally provided little or no guidance on how the rainfall and slope inputs were to be determined.

2.7 Rational Method

According to Dooge (1957), the Rational method for calculation of design discharges was first described by Irish engineer Thomas Mulvany in 1851 (Mulvany, 1851). The method was introduced to the United States by Kuichling in 1889 (Kuichling, 1889), but it did not become popular with highway engineers until much later. Neither the term "Rational method" nor the famous formula Q = C i A appear in the papers of Mulvany and Kuichling. However, Kuichling does described the essence of the method:

> The safer method, in the writer's opinion, will be to estimate the probable future amount of impervious surface on the given area, either with reference to the density of population or in any other more reliable manner that may be devised, and to assume that all of the water that falls upon such surface will run off without loss; further, since the topography of the area is supposed to be known, the grades and length of the longest tributaries to the outlet sewer can readily be determined, as well as their approximate diameters, and thence also the velocities of flow therein; from these elements, the time required for the flood-waters to reach the

> outlet sewers from the most distant points in the area can next be found, and when the relation between the probable maximum intensity of the rain and its corresponding duration are known, as exhibited in the preceding, the maximum rate of rainfall belonging to the time so found can then be deduced. By proceeding in this manner, it is believed that the least error will accrue in the results, and that the dimensions of a sewer so computed will be found adequate until the assumed amount of impervious surface or density of population has been exceeded.

Kuichling explained the advantages of the Rational method, compared to other formulas for design discharge, as follows:

> It may be urged that the process indicated is nothing more than a crude approximation, and that some one of the various empirical formulas might as well have been applied; but to this it may be answered that the method is at all events intelligible and rational, besides being founded upon a somewhat better array of ascertained facts than is the case with the empirical formulas mentioned; it also has the merit of compelling the exercise of an engineer's judgment and discretion with respect to the future of particular localities of a city, or even of different portions of the same large drainage area, instead of dealing alike with all. Moreover, it rarely happens that the history and composition of such formulas become known to the majority of those who may be called upon to apply them, and hence a process in which every single component can be thoroughly scrutinized and amended to suit different circumstances will generally prove to be safer than the application of indefinite rules. (Kuichling, 1889)

The lack of reliable guidance for the estimation of the runoff coefficient, time of concentration and rainfall intensity probably explains why the Rational method was not embraced by early highway engineers.

Blanchard and Drowne's *Text-Book on Highway Engineering* (1913), the only early textbook on highway engineering to mention the Rational method, refers to it as the "formula for run-off." The authors provide a graph of rainfall intensity versus duration with no mention of geographic limits on its applicability and no indication of recurrence interval. The rainfall intensities in the graph would have recurrence intervals in the one-to-two-year range in eastern Kansas. The authors define the runoff coefficient incorrectly as "the percentage of run-off", which is a common mistake even today, and state that its value "depends entirely upon the judgment of the engineer."   Their recommended ranges of runoff coefficients for different land covers are consistent with modern practice.

2.8 Observations on Reliability of Early Methods

Early highway engineers were well aware of the shortcomings and uncertainties of their hydrologic design methods. A. M. Wellington's critique of the Myers formula, published in the *Railroad Gazette* in 1886, is a gem.

> It is natural for fallible man to wish to reduce everything to a rule, even if it be only a rule of thumb. The responsibility of the individual is much diminished if he has something of that kind to lean on, and in so doubtful a matter as the proper size of culverts, this is especially natural. It is well, however, to be certain that we are not simply making a rule where there is no rule, and so laying the foundation of future trouble, and we must confess doubts as to whether this is not the case with the various formulas for proportioning waterways for culverts … when in addition to the probable variations in maximum rainfall and possible future changes in the conditions of the surfaces are considered, we cannot but regard the proportioning of culverts by a formula as entirely futile…
>
> For culverts, if we were called upon to suggest a formula, we could do no better than this: Estimate the necessary area as carefully as possible by the existing evidences of maximum flow, which let equal to A. Then will $\sqrt[3]{8}$ A equal the proper area for the culvert. In more popular language: 'Guess at the proper size and double it.' We apprehend that this formula will give far more satisfactory and trustworthy results than that which our correspondent quotes [the Myers formula] or any other which purports to be of general application to a problem subject to such extremely diverse conditions. (Wellington, 1886)

A. T. Byrne explained the fundamental problem in a nutshell:

> Numerous empirical formulas have been proposed for this and similar problems; but at best they are all only approximate, since no formula can give accurate results with inaccurate data. (Byrne, *A Treatise on Highway Construction*, 4th ed., 1902)

Byrne offered the following observation on the precision required in the sizing of drainage structures:

> The determination of the values of the different factors entering into the problem is almost wholly a matter of judgment. An estimate for any one of the above factors is liable to be in error from 100 to 200 percent, or even more, and of

> course any result deduced from such data must be very uncertain. Fortunately, mathematical exactness is not required by the problem nor warranted by the data. The question is not one of 10 or 20 percent of increase; for if a 2-foot pipe is insufficient, a 3-foot pipe will probably be the next size, an increase of 225 per cent; and if a 6-foot arch-culvert is too small, an 8-foot will be used, an increase of 180 per cent. The real question is whether a 2-foot pipe or an 8-foot arch-culvert is needed.

Blanchard and Drowne (1913) advocated the use of empirical methods or formulas developed over time from local observations. They correctly observed that methods that consider drainage area alone cannot provide universally satisfactory results.

The first step in designing a bridge or a culvert is to determine the size of opening necessary to take the water. The practice of basing this determination on a mere guess should be condemned, since it will usually result in an uneconomical design or a wash-out, which, in the case of structures of any size or importance, may involve loss of life and property. The proper size of the opening for a culvert or bridge cannot be determined by measuring the cross-section of the water at the point where the bridge or culvert is located unless the water at the time happens to be at its maximum stage. There are various ways, however, in which the amount of water and the size of opening can be determined. The variables which enter into the problem make the results more or less approximate, and different formulas may give widely varying results for the same conditions. In any event the use of such results is better than a mere guess, and if some method or formula can be applied to conditions in any one locality for a sufficient length of time, and proper study be given to the factors which tend to make the results approximate, in time the method or formula can be depended upon to give results which, for that particular locality at least, are reasonably accurate.

Empirical formulas are many in number and give results which are extremely variable. This may be accounted for in some instances by the fact that the formulas were calculated for conditions in some one locality that do not agree with those in another. Again, some of these formulas have only one variable in them, namely, the drainage area, and it cannot be expected that the results by such formulas will agree with those obtained by formulas which have coefficients that are to be applied for different soil conditions, steepness of slope, etc. (Blanchard and Drowne, *Text-Book on Highway Engineering*, 1913).

Eminent bridge engineer J. A. Waddell considered Dun's table to be more reliable than any of the empirical formulas.

As a rule, calculations for waterway areas are restricted to small openings, such as culverts, for determining which various formulae from time to time have been proposed and adopted more or less generally. Unfortunately, many of these are widely divergent, mainly because of variations in the governing conditions, such as the area of drainage basin, amount of annual rainfall, intensity, extent, and duration of rain storms, slope of stream and its tributaries, character of soil and quality and extent of vegetation. These factors certainly constitute a valid excuse for considerable divergence in the resulting values of stream areas and discharges as calculated by the various formulae that have received more or less endorsement by the engineering profession; but they are by no means a legitimate reason for the ridiculously large variations that one notes when applying such formulae for some particular case.

The author's judgment in respect to choice of formulae for sectional areas of stream would be to discard them all and use Dun's table, which gives data based on actual records up to areas of 6,500 square miles. (Waddell, *Bridge Engineering*, 1916)

Chapter 3

Modern Methods

3.1 Overview

Modern methods for sizing of waterway openings are based on frequency analysis of streamflow and/or rainfall data; i.e., structures are sized for a flood with a specific recurrence interval. Research by the Bureau of Public Roads and others in the 1940s laid the groundwork for frequency-based sizing of culverts and bridges. The transition to frequency-based design in highway engineering practice occurred mainly in the 1950s.

The first highway engineering textbook to advocate the modern approach to waterway sizing was *Highway Engineering* by Hewes and Oglesby, published in 1954. The following paragraph from the Foreword signaled the new approach:

> In the past, hydrologic data of value to highway engineers have been fragmentary, and hydraulic designs based on them often seemed pointless. For this and other reasons, drainage design before about 1940 was largely by rule-of-thumb methods, many of them of doubtful validity. Since that time, and particularly since World War II, highway engineers have devoted increasing attention to drainage problems. Research already completed or underway will greatly extend present knowledge. The results of these efforts are already and will be increasingly apparent in better and more economical highway drainage.

This textbook offered the following lucid explanation of the role of probability in hydrologic design:

> It should be understood at the outset that predictions regarding future rainfall or runoff from accumulated records rest on the laws of probability: in other words, the chance that a given event will or will not take place. To illustrate, consider the statement that a culvert is designed to carry a "50-year" flood. This means that, if past experience is repeated, the chances are 1 in 50 that the structure will flow full or be overtaxed once in a particular year. It does not mean that the design flood or a larger one will occur exactly one time in 50 years; in fact, the chances are only 64 in 100 that a flood of this magnitude will occur in a given 50-year period. On the other hand, several floods of this or greater magnitude could

> occur in successive years or in a single year, but the chance for either combination is extremely small.

Hewes and Oglesby's 1954 textbook presented two methods for estimation of design flows with specified recurrence intervals: the Bureau of Public Roads method (Izzard, 1953) and the Rational method. It also included a brief discussion of unit hydrographs and their uses in highway engineering. A brief section on "empirical formulas," which focused on their shortcomings, presented the Talbot formula with the following disclaimer:

> The Talbot formula was first proposed before the turn of the century, when practically nothing was known regarding hydrology or hydraulic design. . . . Its widespread adoption in the highway field probably can be attributed to its simplicity and the lack of something better.

It is worth noting that the Talbot formula, albeit with the disclaimer, was retained in 2nd (1963), 3rd (1975) and 4th (1982) editions of this textbook.

3.2 Recurrence Intervals for Design

The University of Illinois's 1953 survey of the design practices of state highway departments (Chow, 1962) showed little agreement on recurrence intervals for bridges and culverts at the start of the modern era. The recurrence intervals reported for "culverts, small bridges and the drainage structures in the secondary highway system" ranged from 5 to 100 years. The most common recurrence interval for these structures was 25 years. The recurrence intervals reported for "bridges, large culverts, and the drainage structures in the primary highway system" ranged from 5 to 100 years. The most common recurrence interval for these larger or more important structures was 50 years.

The Bureau of Public Roads first implemented frequency-based design criteria for drainage structures on interstate highways in 1956. These design criteria specified a minimum recurrence interval of 50 years for drainage structures on interstate highways. The federal government has never specified minimum recurrence intervals for drainage structures on non-interstate highways.

Federal regulations implemented in 1979 (23 CFR Part 650; see Section 5.5) require consideration of "capital costs and risks, and to other economic, engineering, social and environmental concerns" in selection of recurrence intervals for bridges and culverts. FHWA's Hydraulic Engineering Circular No. 17, "Design of Encroachments on Flood Plains Using Risk Analysis" (1984) provides guidance for selection of recurrence intervals by the method of "least total expected cost."

3.3 Rainfall Frequency

The first reliable rainfall-frequency maps for daily and longer-duration rainfalls were published in 1917 by the Miami (Ohio) Conservancy District in a report titled *Storm Rainfall of Eastern United States* (MCD, 1917). These maps provide rainfall estimates for durations of one to six days and recurrence intervals of 15 to 100 years for the United States east of the 103rd meridian, which includes all of Kansas.

The first reliable rainfall-frequency estimates for shorter durations were published in 1935 by the U.S. Department of Agriculture in a report titled *Rainfall Intensity-Frequency Data* (Yarnell, 1935). This report provides nationwide rainfall maps for durations from 5 minutes to 24 hours and recurrence intervals from 2 to 100 years.

The next significant report on rainfall frequencies for the eastern and central United States was the U. S. Weather Bureau's *Technical Paper No. 40,* "Rainfall Frequency Atlas of the United States," published in 1961 (Hershfield, 1961). *Technical Paper No. 40* covers durations from 30 minutes to 24 hours and recurrence intervals from 1 to 100 years. It remains the most widely accepted source of rainfall estimates for durations over one hour.

The National Weather Service (NWS) issued revised rainfall frequency estimates for durations of 5 to 60 minutes for the eastern and central United States in 1977. *NOAA Technical Memorandum NWS HYDRO-35* (Frederick, et al., 1977) contains maps of rainfall depths for durations of 5, 15 and 60 minutes and recurrence intervals of 2 and 100 years, and interpolation formulas for intermediate durations and recurrence intervals. *HYDRO-35* remains the most widely accepted source of rainfall estimates for durations of 60 minutes and shorter.

3.4 Flood Frequency Analysis

The modern era of flood frequency analysis began in the early 1940s with a series of groundbreaking papers by E. J. Gumbel (e.g., Gumbel, 1945). Before Gumbel, flood data were analyzed by ad-hoc graphical methods. Gumbel developed a theoretically sound method based on fitting an extreme-value Type I probability distribution to the record of annual peak flows.

In the 1970s the U. S. Water Resources Council (USWRC) developed recommended procedures for flood frequency analysis to be applied by all federal agencies. These procedures were published initially in USWRC's *Bulletin 17*, "Guidelines for Determining Flood Flow Frequency" (1976). Revised guidelines were published as *Bulletin 17A* (Interagency Advisory Committee on Water Data, 1977) and *Bulletin 17B* (Interagency Advisory Committee on Water Data, 1981). The *Bulletin 17B* procedures have been applied by federal agencies since 1981. In the USWRC method of flood frequency analysis, the record of annual flood peaks are fitted with a log-Pearson Type III probability distribution rather than Gumbel's extreme-value probability distribution.

3.5 Rational Method

The modern Rational method is a simple frequency-based design method. The method requires rainfall frequency data for short durations, which first became available nationwide in 1935 with publication of the U. S. Department of Agriculture's *Rainfall Intensity-Frequency Data* (Yarnell, 1935). Johns Hopkins University's Storm Drainage Research Project, initiated in 1949, demonstrated the solid theoretical foundation of the frequency-based Rational method and produced relationships for urban runoff coefficients and lag times (Schaake, et al., 1967). Nearly every highway engineering textbook published since 1950 has included the frequency-based Rational method. However, guidelines for determination of runoff coefficients and times of concentration vary widely. Stated limits on the applicability of the method also differ. Its applicability to small urban watersheds is generally accepted. The Rational method is also applicable to larger urban watersheds and rural watersheds, but more research is needed to guide the selection of runoff coefficients for these conditions.

3.6 SCS Methods

The Soil Conservation Service (SCS) of the U. S. Department of Agriculture developed its own set of hydrologic design methods, centered around the curve-number runoff model, in the early 1950s. First published in a 1954 handbook titled *Hydrology Guide for Use in Watershed Planning*, these methods were incorporated into the SCS *National Engineering Handbook* as Section 4, Hydrology, in 1956. The SCS hydrologic methods soon gained widespread acceptance among highway engineers.

The SCS hydrologic design procedures were expanded in 1975 with publication of *Technical Release 55* (TR-55), "Urban Hydrology for Small Watersheds." TR-55 provided tabular and graphical methods for estimation of design discharges and development of design hydrographs, based on a 24-hour design storm with a particular temporal pattern. The SCS released an updated version of TR-55 as a computer program in 1986. The TR-55 methods are accepted by FHWA and many state highway agencies.

3.7 BPR Methods

In the 1950s the Bureau of Public Roads developed a widely used method for estimating design flows for drainage structures on small ungaged streams (Izzard, 1953). The BPR method provided discharge estimates for recurrence intervals from 5, 10, 25 and 50 years for rural watersheds smaller than 1000 acres in the eastern and central U.S. (Izzard, 1953). This method, based loosely on the index-flood method of regional flood frequency analysis (Dalrymple, 1950) was developed from analyses of SCS data for experimental watersheds in Maryland, Ohio, Wisconsin and Nebraska (Potter, 1950). The inputs to the 1953 BPR method are drainage area, a rainfall factor obtained from a map, and a land-use-and-slope factor obtained from a table.

In 1961, BPR published an updated method applicable to watersheds with areas up to 25 mi^2 (Potter, 1961). The report states optimistically that the standard errors of the estimates "may be assumed to be less than ±20 percent of the estimated values," but the true standard errors were much larger. The standard error of design-discharge estimate is a measure of the uncertainty of the estimate. The probability that the estimated discharge is within one standard error of the true discharge is approximately 68%. The BPR methods, which remained popular through the 1960s, were gradually supplanted by regional flood-frequency equations developed by the USGS.

3.8 Regression Equations for Flood Frequency

In the 1960s, the USGS and other agencies began to develop regional flood-frequency relations by regression analysis rather than graphical methods. These regional regression equations were widely adopted for estimation of design flows on unregulated rural streams. New regression equations are developed periodically to incorporate new data. Section 6.4 covers flood-frequency regression equations for Kansas.

In the late 1960s, a National Cooperative Highway Research Program project attempted to develop nationwide regression equations for flood discharges on small rural streams (Bock, et al., 1972). However, the resulting nationwide equations were found to be inferior to existing regional methods (Woo, 1974).

Chapter 4

Culvert and Bridge Hydraulics

4.1 Culvert Hydraulics

Guidance on culvert hydraulics in early highway engineering textbooks was incomplete at best. The fundamental differences in culvert performance under inlet control and outlet control were generally ignored. Several early textbooks advocate the sizing of culverts by a “uniform flow” method that considers barrel friction but not inlet or outlet conditions. However, certain authors did provide some sound qualitative guidance; for example:

> The efficiency of the culvert may be materially increased by so arranging the upper end that the water may enter it without being retarded. The discharging capacity of a culvert can also be increased by increasing the inclination of its bed, provided that the channel below will allow the water to flow away freely after having passed the culvert.
>
> The discharging capacity of the culvert can be greatly increased by allowing the water to dam up above it. A culvert will discharge twice as much under a head of four feet than under a head of one foot. This can be done safely only with a well-constructed culvert. (Byrne, *A Treatise on Highway Construction*, 4th ed., 1902)

In 1922-23, engineers from the Bureau of Public Roads and the University of Iowa conducted ground-breaking research on culvert hydraulics in the University of Iowa hydraulics laboratory. Articles in Public Roads in 1924 and 1926 summarized the findings from this research program for highway engineers. The introduction to the 1924 article provided the following overview of the key findings:

Three facts stand out from the results of the tests as worthy of the most serious consideration of highway engineers.

The first is that highway engineers must pay more attention to the coefficient of roughness of the material forming the culvert. So long as the different materials used for culvert pipe did not differ greatly in roughness and hence in their frictional resistance to moving water, engineers were perhaps justified in not giving this factor much consideration. But in recent years a new material, corrugated metal, has been extensively manufactured into culvert pipe. Pipes made of this material are shown by these tests to offer much greater frictional resistance to the flow of water than other materials used, such as vitrified clay, cast iron, concrete, and timber. While for pipes of each material the coefficient of roughness in the Kutter formula is shown to increase with increase in the size of pipe, the tests show that, for all sizes it is nearly twice as great for corrugated metal as for concrete and vitrified clay pipe.

The second fact brought out by the tests is that the quantity of water a culvert will discharge is directly proportional to the square root of the head and bears no relation to the grade at which the pipe is laid, if the pipe flows full, as it should at maximum capacity. The water in a pipe culvert under these conditions does not act as does that flowing in an open ditch where the quantity of discharge is dependent upon the slope or grade of the water surface in the ditch, but, as is the case in any pipe flowing full, the discharge depends upon the water pressure available to force the water through the opening and the pipe. In the case of a culvert the water pressure which causes discharge is furnished by the difference between the water level at the entrance and the outlet. The depth of submergence has no effect on this discharge, so long as the difference of the water levels at the two ends of the culvert remains the same.

The third observation is that the head loss at the culvert entrance is an important factor in determining the discharge and varies greatly with the type of entrance used. The data on the effect of different types of entrance on the entrance loss are among the most interesting of the findings from the tests.

The next, and most recent, major advances in culvert hydraulics resulted from a research program at the National Bureau of Standards (NBS) hydraulics laboratory in 1950s and 1960s. The NBS research program focused mainly on entrance conditions and their effects on culvert performance. This research produced dimensionless head-discharge relationships for inlet control and entrance-loss coefficients for outlet control for a large variety of entrance types. It also produced recommended designs for side-tapered and slope-tapered entrances. The NBS research findings formed the basis for a series of practical reports on culvert hydraulics by BPR and FHWA:

> *Hydraulic Engineering Circular No. 5*, "Hydraulic Charts for the Selection of Highway Culverts," Bureau of Public Roads, 1961 (revised 1965).
>
> *Hydraulic Engineering Circular No. 10*, "Capacity Charts for the Hydraulic Design of Highway Culverts," Federal Highway Administration, 1972.
>
> *Hydraulic Engineering Circular No. 13*, "Hydraulic Design of Improved Inlets for Culverts," Federal Highway Administration, 1972.
>
> *Hydraulic Design Series No. 5*, "Hydraulic Design of Highway Culverts," Federal Highway Administration, 1985.

Hydraulic design practice for highway culverts has not changed significantly since the publication of these reports.

4.2 Bridge Hydraulics

The two main issues in bridge hydraulics are the backwater caused by bridges during floods and scour around bridge piers and abutments during floods.

The two classic laboratory studies of backwater from bridge piers were those of Nagler (1918) and Yarnell (1934). In 1960, the Bureau of Public Roads published *Hydraulic Design Series No. 1* (HDS-1), "Hydraulics of Bridge Waterways" (Bradley, 1960), based in large part on laboratory studies conducted at Colorado State University. The second edition of HDS-1, published in 1978 (Bradley, 1978), incorporated new findings from USGS field measurements at bridges during floods. The 1978 edition of HDS-1 remains a widely accepted reference on backwater effects at bridges.

The pioneering research on scour at bridge piers and abutments was conducted at the University of Iowa in the 1950s under the sponsorship of the Iowa State Highway Commission and the Bureau of Public Roads. The results from this research were broadly disseminated in 1960 (Laursen, 1960). FHWA issued interim procedures for evaluation of bridge scour in 1988. FHWA's *Hydraulic Engineering Circular No. 18 (HEC-18)*, "Evaluating Scour at Bridges," was first published in 1991 and has undergone several revisions (FHWA, 2001).

Chapter 5

Federal Design Criteria

5.1 Overview

From the start of the Federal-Aid Highway Program in 1916 until the start of the Interstate System, the Bureau of Public Roads and its predecessor agencies had no criteria for the sizing of bridges and culverts. BPR issued the first such guidance on August 10, 1956, in a revision of Policy and Procedure Memorandum (PPM) 20-4, "Policy on Interstate System Projects." FHWA made slight revisions to the criteria for Interstate System projects and added new criteria for non-Interstate federal-aid projects on April 26, 1967, in Instructional Memorandum (IM) 20-1-67, "Evaluation of Flood Hazards – Federally-Financed Highways." FHWA incorporated these criteria into PPM 40-2, "Design Standards for Federal-Aid Projects," on May 12, 1969. In 1979, revised hydrologic design criteria were adopted in the Code of Federal Regulations (CFR) as Title 23: Highways; Part 650—Bridges, Structures and Hydraulics; Subpart A—Location and Hydraulic Design of Encroachments on Floodplains. These regulations, which are also published in FHWA's Federal-Aid Policy Guide, have been applicable to federal-aid projects from November 15, 1979, to the present.

5.2 PPM 20-4

BPR's Policy and Procedure Memorandum (PPM) 20-4, "Policy on Interstate System Projects," issued on August 10, 1956, established the first hydrologic design criteria for Interstate System projects.

> Section 4(d). "Designs for all culverts and bridges over streams shall be in accord with the Standard Specifications for Highway Bridges of the American Association of State Highway Officials to accommodate floods as least at great as

> that for a 50-year frequency or the greatest flood of record, whichever is greater, with runoff based on land development expected in the watershed 20 years hence and with backwater limited to an amount which will not result in damage to upstream property or to the highway. All other drainage facilities are to be designed to keep the traveled ways usable during storms at least as great as that for a 10-year frequency, except that a 50-year frequency shall be used for underpasses and other depressed roadways where ponded water can be removed only through the storm drainage system.

The AASHO Standard Specifications cited in this policy do not specify any additional hydrologic design criteria.

Certain requirements of the policy were problematic. The requirement to size the structure for the 50-year flood or "the greatest flood of record" required excessively large and costly structures in locations where the flood of record greatly exceeded the 50-year flood. The requirement to base the design discharge on "land development expected in the watershed 20 years hence" introduced additional uncertainty. . . . Reliable predictions of the extent and timing of future development over a 20-year horizon are generally not possible. State highway agencies have no control over development outside the highway right-of-way. These two problematic requirements were omitted from subsequent revisions of the policy.

5.3 IM 20-1-67

FHWA's Instructional Memorandum (IM) 20-1-67, "Evaluation of Flood Hazards – Federally-Financed Highways," dated April 26, 1967, relaxed some of the hydrologic design requirements for Interstate System projects and established the first federal design criteria for sizing of waterways on non-Interstate federal-aid projects.

> The following general criteria, revised from PPM 20-4, dated August 10, 1956, shall be used for the design of drainage structures.
>
> (a) Interstate System Projects

Designs for all culverts and bridges over streams shall be in accord with the Standard Specifications for Highway Bridges of the American Association of State Highway Officials, 9th Edition, 1965, to accommodate floods as least as great as that for a 50-year frequency or the greatest flood or record, whichever is greater, with runoff based on land development expected in the watershed 20 years hence and with backwater limited to an amount which will not result in damage to upstream property or to the highway. Where the greatest flood of record is considerably larger than the 50-year flood and the cost to provide for such an exceptional flood without damage or flooding to the roadway or adjacent property is shown by analysis to be excessive for the protection given, a lesser flood, but not less than the flood of 50-year frequency, may be used for design. The effect of flood-control structures on reducing floods should be considered in determining the design flood. Roadway inlets for pavement drainage should be spaced so that not more than half of a through traffic lane would be flooded during a 10-year frequency storm, except that a 50-year frequency shall be used for underpasses and other depressed roadways where ponded water can be removed only through the storm drainage system.

(b) Other Federal-Aid Projects

Designs for culverts, bridges and other drainage facilities on highways other than the Interstate System shall be in accordance with the requirements of paragraph (a), except that the design floods may be reduced if conditions warrant lower standards. The flood frequency selected for design should be consistent with the magnitude of damage to adjacent property and the importance of the highway.

Unlike PPM 20-4, IM 20-1-67 did not require waterways on Interstate highways to be designed for the flood of record. The new policy for drainage structures on non-Interstate federal-aid highways did not specify a minimum recurrence interval. The statement concerning flood-control structures clarified an issue not addressed previously in PPM 20-4.

IM 20-1-67 also included the following recommendations regarding hydrologic and hydraulic studies for bridges and culverts.

The attached 'Guidelines for Preparation of Hydraulic Report on Bridge Waterways or Flood Plain Encroachments' can be used as a checklist of the data which should be considered for inclusion in hydraulic reports that are used to evaluate the effects of highway crossings of major waterways or encroachment upon (along) the flood plain of such major waterways. Less comprehensive reports would be proper for culverts and bridges that cross or encroach upon the waterway of small or minor streams. The costs of preparing such reports are reimbursable as preliminary engineering. Such engineering costs should be commensurate with the importance and cost of the highway or drainage structure and the difficulty in collecting and analyzing the flood data.

5.4 PPM 40-2

FHWA incorporated the hydrologic design criteria from IM 20-1-67 into a revision of PPM 40-2, "Design Standards for Federal-Aid Projects" dated May 12, 1969. PPM 40-2 was first issued in 1954 and was amended several times prior to the revision of May 12, 1969. The original memorandum and earlier revisions did not include any hydrologic design criteria. The wording of the hydrologic design criteria in the May 12, 1969, revision of PPM 40-2 differ slightly from those in IM 20-1-67, but the substance is identical:

> Section 5(a). Design of Drainage Structures
>
> (1) All culverts and bridges over streams shall be designed in accordance with AASHO Standard Specifications for Highway Bridges to accommodate floods as least as great as that for a 50-year frequency or the greatest flood of record, whichever is greater, with the runoff based on the land development expected in the watershed 20 years hence and with backwater limited to an amount which will not result in damage to upstream property or to the highway. Where the greatest flood of record is considerably larger than the 50-year flood and the cost to provide for such an exceptional flood without damage to the roadway or adjacent property is shown by analysis for the protection given, a lesser flood may be used for design. For highways other than those on the Interstate System the design flood may be less than a 50-year frequency where conditions warrant lower standards. The flood frequency selected for design should be consistent with the magnitude of the damage to adjacent property and the importance of the highway. The effect of flood-control structures on reducing floods should be considered in determining the design flood.
>
> (2) Roadway inlets for pavement drainage should be spaced so that not more than half of a through traffic lane will be flooded during a 10-year frequency storm, except that a 50-year frequency shall be used for underpasses and other depressed roadways where ponded water can be removed only through the storm drain system.

5.5 3 CFR Part 650

The current federal criteria for hydrologic design of bridges and culverts are incorporated in the Code of Federal Regulations as Title 23: Highways; Part 650 -- Bridges, Structures and Hydraulics, and also published in FHWA's Federal-Aid Policy Guide. Effective November 15, 1979, these regulations superseded the hydrologic design criteria issued previously in PPM 20-4, IM 20-1-67 and PPM 40-2. The relevant sections are 650.115 and 650.117:

> 650.115 Design Standards
> (a) The design selected for an encroachment shall be supported by analyses of hydraulic design alternatives with consideration given to capital costs and risks, and to other economic, engineering, social and environmental concerns.
> (1) Consideration of capital costs and risks shall include, as appropriate, a risk analysis or assessment which includes:
> (i) The overtopping flood or the base flood, whichever is greater, or
> (ii) The greatest flood which must flow through the highway drainage structure(s), where overtopping is not practicable. The greatest flood used in the analysis is subject to state-of-the-art capability to estimate the exceedance probability.
> (2) The design flood for encroachments by through lanes of Interstate highways shall not be less than the flood with a 2-percent chance of being exceeded in any given year. No minimum design flood is specified for Interstate highway ramps and frontage roads or for other highways
> (3) Freeboard shall be provided, where practicable, to protect bridge structures from debris- and scour-related failures.
> (4) The effect of existing flood control channels, levees and reservoirs shall be considered in estimating the peak discharge and stage for all floods considered in the design.
> (5) The design of encroachments shall be consistent with standards established by the FEMA, State, and local governmental agencies for the administration of the National Flood Insurance Program for:
> (i) All direct Federal highway actions, unless the standards are demonstrably inappropriate, and
> (ii) Federal-aid highway actions where a regulatory floodplain has been designated or where studies are underway to establish a regulatory floodway.
>
> 650.117 Content of Design Studies
> (a) The detail of design studies shall be commensurate with the risk associated with the encroachment and with other economic, environmental and social concerns.
> (b) Studies by highway agencies shall contain:
> > (1) The hydrologic and hydraulic data and design computations,
> > (2) The analysis required by 650.115(a), and
>
> (3) For proposed direct Federal highway actions, the reasons, when applicable, why FEMA criteria (44 CFR 60.3) are demonstrably inappropriate.
> (c) For encroachment locations, project plans shall show:
> (1) The magnitude, approximate probability of exceedance and, at appropriate locations, the water surface elevations associated with the overtopping flood or the flood of 650.115(a)(1)(ii), and

> (2) The magnitude and water surface elevation of the base flood, if larger than the overtopping flood.

The hydrologic design criteria in 23 CFR, Part 650 are largely consistent with the previous criteria. The requirements for design frequency are identical to those in IM 20-1-67. Drainage structures on Interstate highways must be designed for "the flood with a 2-percent chance of being exceeded in any given year," which is the 50-year flood. No minimum design flood is specified for non-Interstate highways. The regulations further state that no minimum design flood is required for "Interstate highway ramps and frontage roads." Interstate ramps and frontage roads were not addressed in the previous design criteria. The most significant change in the design criteria in 23 CFR Part 650 is the omission of the previous requirement to consider "land development expected in the watershed 20 years hence" in calculation of design flows.

23 CFR Part 650 includes some new instructions for design studies. The design "shall be supported by analyses of hydraulic design alternatives with consideration given to capital costs and risks, and to other economic, engineering, social and environmental concerns." Design studies should consider the "overtopping flood," the smallest flood that would overtop the roadway. If the recurrence interval of the overtopping flood is estimated to be less than 100 years, the design studies should also consider the "base flood," which is defined as the 100-year flood. However, the "Applicability" section of the supplementary information published with the final rule (Federal Register, Vol. 44, No. 228) includes the following statement:

> The FHWA intends that all encroachments be assessed. However, the level of review should be consistent with the risk and impact. Little or no risk or impact would only require discussion and hydraulic design studies which are commensurate with that risk or impact.

This statement is consistent with IM 20-1-67, which states “Less comprehensive reports would be proper for culverts and bridges that cross or encroach upon the waterway of small or minor streams.”

5.6 Summary of Federal Design Criteria

Prior to 1956, the federal government had no hydrologic design criteria for highway bridges and culverts. FHWA (previously BPR) has required a minimum design frequency of 50 years for bridges and culverts on Interstate highways from 1956 to the present. FHWA previously imposed two additional requirements on Interstate projects. From 1956 to 1967, FHWA required that the structure be designed for the flood of record if its frequency exceeded 50 years. From 1956 to 1979, FHWA required that the design flood be based on land development projected 20 years into the future. FHWA has never specified a minimum design frequency for bridges and culverts on non-Interstate highways.

Chapter 6

Design Practices in Kansas

6.1 Hydrologic Methods and Criteria

The University of Illinois's 1953 survey (Chow, 1962) provides a snapshot of hydrologic design practices in Kansas and elsewhere at the start of the modern era of highway construction. The respondent from the Kansas State Highway Commission listed the Talbot formula, Dun's table and "USGS data," but not the Rational method, as hydrologic design methods used by KSHC in 1953. A recurrence interval of 25 years was reported for "culverts, small bridges and drainage structures in the secondary highway system," and recurrence intervals of "25 years and up" were reported for "bridges, large culverts and drainage structures in the primary highway system." The hydrologic methods and recurrence intervals reported by KSHC in the 1953 survey were consistent with the practices reported by the highway departments of nearby states. The transition to frequency-based design was underway, as evidenced by the reported recurrence intervals, but the older non-frequency-based design methods were also still in use. Although earlier design practices are not well documented, the responses to the 1953 survey indicate that most Kansas highway culverts and bridges constructed prior to this date on ungaged streams were probably sized with the Talbot formula or Dun's table, which was based on the experience of the Atchison, Topeka and Santa Fe Railroad in Kansas and elsewhere.

Design tables and graphs from the archives of KDOT's Bureau of Design confirm that the Kansas State Highway Commission made use of both the Talbot formula and Dun's table for waterway sizing. Appendix B shows a version of Dun's table specifically for Kansas, dated 1927. This table is consistent with the more general one in Appendix A. Other tables and graphs provided waterway areas computed by the Talbot equation using coefficients ranging from 0.2 to 1.0. The column headings and curve labels indicated the recommended values of Talbot's coefficient, C, for different types of terrain.

Mountainous	C = 1.0
Hilly	C = 0.6 to 0.8
Rolling	C = 0.5
Slightly rolling	C = 0.4
Flat	C = 0.3
Very flat	C = 0.2

The transition to frequency-based hydrologic design in Kansas appears to have been completed by 1966. A KSHC document titled "Reinforced Concrete Box Culvert Design" dated January 3, 1966, specifies the frequency-based BPR method (Izzard, 1953) as the preferred hydrologic method for watersheds under 1000 acres. This document includes tables of 25-year discharge versus drainage area for Kansas counties, grouped by BPR rainfall factor, based on mixed cover.

The 1966 KSCH document also includes a table of "capacities" for concrete and metal pipe culverts, metal arch culverts and concrete box culverts of many sizes. The capacities listed in this table are discharges that the culverts would convey under inlet control at a headwater depth equal to the diameter of the pipe or the rise of the box culvert (HW = D). This table indicates that, as of 1966, culverts on state highways in Kansas were sized for inlet control with HW = D, which is a very conservative design criterion in most situations. Most culverts designed for this criterion could actually pass a much higher discharge without flooding the highway or adjacent structures. A possible exception would be a culvert with high tailwater due to a downstream constriction or a confluence with a larger stream.

In 1975, KDOT issued the KDOT Design Manual, Volume III, "Elements of Drainage and Culvert Design." This document, with a 1990 addendum, provides comprehensive guidance for hydrologic and hydraulic design. Volume III of the Design Manual includes the following hydrologic methods:

- Rational method -- design discharges for rural watersheds up to 640 acres and urban watersheds up to 1000 acres
- USGS regression equations -- design discharges for unregulated streams with rural watersheds over 640 acres
- Modified Rational method – design hydrographs for watersheds up to 1000 acres
- FENL-H method – design discharges and hydrographs for rural and urban watersheds from 400 acres to 500 mi2

The USGS regression equations included in the Volume III of the Design Manual were published in 1975. These equations have been superseded by revised regression equations published in 1987 and 2000.

Although not included in Volume III, the SCS methods have long been considered acceptable hydrologic design methods by KDOT. The 1990 addendum to Volume III states that "SCS methods are appropriate for either rural or urban areas of any size up to about 20 mi^2."

Volume III of the Design Manual also provides guidance for selection of recurrence intervals and allowable water surface (AWS) levels for design. The guidelines for recurrence interval include the following provisions:

- 100-year protection for most buildings
- 50-year protection for interstate highways
- 25-year protection for primary routes, secondary routes and major sideroads
- 10-year protection for minor sideroads

The AWS guideline for roads is the top of the subgrade at the outside edge of the shoulder. The headwater level upstream of culverts is limited only by the AWS levels for the road and any structures that could be affected by backwater. This guideline is a significant change from the previous guideline of design for HW = D in KSHC's 1966 culvert-design document.

6.2 Rainfall Frequency

The Rational method, the SCS methods and others require rainfall frequency information. Reasonably accurate rainfall frequency information for Kansas, covering the durations and recurrence intervals needed for highway applications, has been available since the U. S. Department of Agriculture published its nationwide report on rainfall frequency in 1935. (Yarnell, 1935).

KDOT's *Rainfall Tables for Counties in Kansas* was originally developed from the rainfall frequency maps in *Technical Paper 40* (Hershfield, 1961). These rainfall tables were revised in 1992 to incorporate the improved estimates for short-durations rainfalls from *HYDRO-35*, and re-issued with minor corrections in 1997.

The most up-to-date rainfall frequency estimates for the Kansas City metropolitan area are those published by the Kansas City Metro Chapter of the American Public Works Association in 2002 (Young and McEnroe, 2002). The rainfalls depths in this report are larger than the values in the NWS atlases for short durations and long recurrence intervals (up to 14% higher for the 10-minute, 100-year event) and lower for short durations and short return periods (13% lower for the 5-minute, 2-year event). These differences are due primarily to differences in statistical methods. This study found no statistically significant trends in the magnitude or frequency of extreme daily rainfalls in the Kansas City area.

6.3 Streamflow Data

Streamflow gaging in Kansas began in 1895 with a cooperative agreement between the U. S. Geological Survey and the Kansas Board of Irrigation Survey, Experiment and

Demonstration. Seven sites, all on major rivers, were gaged initially, and more sites were added later. The cooperative streamflow-gaging program was discontinued in mid-1906. In 1917, the newly formed Kansas Water Commission (KWC) and the USGS initiated a new cooperative streamflow-gaging program. Streamflow data reports published by the KWC in 1920 and 1925 show 27 active gages in 1919 and 42 active gages in 1924. In 1927, the responsibilities of the KWC were transferred to the Division of Water Resources (DWR) of the Kansas State Board of Agriculture. DWR published streamflow data reports in 1929, 1936 and 1939. During this period the number of active gages remained between 40 and 50. In 1957, when the newly created Kansas Water Resources Board assumed direction of the streamflow-gaging program, the number of active gages exceeded 90, although many of the records were brief and discontinuous. The USGS currently maintains approximately 170 continuous-record streamflow gages in Kansas. The USGS stream-gaging program in Kansas is funded jointly by several federal agencies, state agencies and local governments. KDOT provides funding for USGS stream gaging and hydrologic studies through a cooperative agreement initiated in 1956.

6.4 Flood Frequency

The first major report on flood frequency for Kansas streams was prepared by the USGS and published by the Kansas Water Resources Board in 1960 (Ellis and Edelen, 1960). This report presented the first regional flood-frequency relationships for Kansas. These relationships provided estimates of flood discharges with recurrence intervals up to 50 years, based on drainage area and geographic location, for unregulated streams with drainage areas over 150 mi^2. No regional relationships were provided for drainage areas under 150 mi^2.

The first regional flood-frequency relationships for small streams in Kansas were published by the USGS in 1966 (Irza, 1966). These relationships for drainage areas under 70 mi^2

were labeled “preliminary” because they were based on only 8 years of peak-flow data (1957-1964) for 95 stations. The report presented state-wide regression equations for discharges with recurrence intervals of 1.2 years, 2.33 years (the mean annual flood for a Gumbel probability distribution), 5 years and 10 years. The three inputs to these equations are drainage area, average channel slope, and average number of wet days per year, which is obtained from a map. The standard error of estimate for the 10-year equation, expressed in percent, is +100%/-49%.

A more comprehensive USGS study of Kansas flood frequency, published by the Kansas Water Resources Board in 1975 (Jordan and Irza, 1975), provided statewide equations for flood discharges with recurrence intervals from 2 to 100 years for unregulated rural streams with drainage areas from 0.4 to 10,000 mi^2. The two inputs to these equations are the drainage area and the 2-year, 24-hour rainfall, which is obtained from a map. The standard errors range from +50%/-31% for the 5-year equation to +74%/-42% for the 100-year equation. The 1975 USGS equations were the first regional flood-frequency equations to be widely used to compute design flows for highway culverts and bridges in Kansas.

The USGS published updated flood-frequency equations for Kansas in 1987 (Clement, 1987) and 2000 (Rasmussen and Perry, 2000). The 1987 equations have four inputs: drainage area, 2-year 24-hour rainfall, average channel slope and generalized soil permeability. The 2-year 24-hour rainfall and the generalized soil permeability are obtained from maps. Standard errors range from +35%/-26% for the 10-year equation to +46%/-31% for the 100-year equation. The latest update, published in 2000, provides two sets of statewide flood-frequency equations: one for drainage areas over 30 mi^2 and another set for drainage area under 30 mi^2. The equations for drainage areas over 30 mi^2 have four inputs: drainage area, mean annual precipitation, channel slope and generalized soil permeability. Standard errors range from +35%/-26% for the

10-year equation to +42%/-30% for the 100-year equation. The equations for drainage areas under 30 square miles have only two inputs: drainage area and mean annual precipitation. These equations for small watersheds have large standard errors: +52%/-34% for the 10-year equation and +71%/-41% for the 100-year equation.

The Frequency-Equivalent Nonlinear Hydrograph (FENL-H) method was developed for KDOT by Professor Robert L. Smith of the University of Kansas in 1982 (Smith, 1982). It is applicable to drainage areas from 400 acres to 500 mi^2. The FENL-H equations provide estimates of flood discharges with recurrence intervals of 2 through 100 years based on drainage area and mean annual runoff, which is obtained from a map. The FENL-H method also yields a triangular flood hydrograph that is useful for storage routing. The method includes adjustments for urbanization and channel modifications.

6.5 Culvert Hydraulics

KDOT has generally relied on culvert hydraulics guidance published by FHWA and BPR, supplemented by design charts from hydraulic model studies at the state universities. The culvert capacities in the 1966 KSHC culvert design guidelines were obtained from BPR nomographs for inlet control dated 1956 and 1963. Volume III of the KDOT Design Manual, issued in the mid-1970s, includes a more comprehensive treatment of culvert hydraulics, with instructions and charts for analysis of outlet control as well as inlet control. The culvert hydraulics guidance in the KDOT design manual is generally consistent with the FHWA's HEC-10 (1972) and HEC-13 (1972). The KDOT Design Manual also includes inlet-control rating curves for box culverts with 45° wingwalls from hydraulic model studies at Kansas State University (Kubitza, 1955). Hydraulic model studies at the University of Kansas in early 1990s produced inlet-control rating curves for pipe culverts with KDOT's standard end treatments (McEnroe and Bartley, 1993; McEnroe and Johnson, 1994).

Chapter 7

Conclusions

In the early days of highway construction, culverts and bridges were sized by empirical methods developed from experiences with existing structures during floods. Most of the these methods were developed by or for the railroads. No particular recurrence intervals were associated with the resulting designs. Early highway engineers were aware of the shortcomings of these design methods, but they were hampered by a shortage of reliable streamflow data and rainfall data. The transition to modern frequency-based design methods generally occurred during the 1950s.

The highway-building era in Kansas began in 1917 with the creation of the Kansas State Highway Commission. Prior to the mid-1950s, most culverts and bridges on Kansas highways were sized with the Talbot formula, Dun's table and other empirical methods. KSHC and KDOT have employed frequency-based design methods such as the Rational method and USGS regression equations since the 1960s. Highway culverts and bridges have been designed for recurrence intervals of 25 years or greater over this period. The hydrologic methods and design guidelines employed by KSHC and KDOT have been within the mainstream of highway engineering practice nationwide. Hydrologic methods have been improved as more streamflow data have become available. However, flood frequency estimates for small watersheds still have large standard errors.

The federal government has specified a minimum recurrence interval of 50 years for culverts and bridges on Interstate highways since 1956. However, FHWA and its predecessors have never specified hydrologic design criteria for structures on non-Interstate highways.

The engineering professions understanding of culvert and bridge hydraulics has advanced greatly over the last century. The Talbot formula, Dun’s table and similar empirical design methods did not explicitly consider the hydraulic characteristics of the structure. Modern design methods require hydraulic analyses of proposed designs. A series of technical reports published by BPR in the 1960s provided highway engineers with practical guidance on the hydraulic aspects of culverts and bridges.

REFERENCES

AREMWA (1911). "Report of the Sub-Committee of Roadway Committee No. 1, Bulletin 131," Proceedings, American Railway Engineering and Maintenance of Way Association, Vol. 12, Part 3, pp. 481-528.

Blanchard, A. H. and H. B. Drowne (1913). *Text-Book on Highway Engineering*, John Wiley & Sons, New York.

Bock, P., I. Enger, G. P. Malhotra and D. A. Chisholm (1972). "Estimating Peak Runoff Rates from Ungaged Small Rural Watersheds," *National Cooperative Highway Research Program Report No. 136*, Highway Research Board, National Research Council.

Bradley, J. N. (1960). "Hydraulics of Bridge Waterways," *Hydraulic Engineering Circular No. 1*, Bureau of Public Roads.

Bremner, G. H., J. Dun, J. W. Alvord and A. N. Talbot (1906). "Symposium on Methods of Determining the Size of Waterways," *Journal, Western Society of Civil Engineers*, Vol. 11, pp. 137-190.

Bureau of Public Roads (1956). Policy and Procedure Memorandum (PPM) 20-4, "Policy on Interstate System Projects," August 10, 1956.

Bureau of Public Road (1961, revised 1965). *Hydraulic Engineering Circular No. 5*, "Hydraulic Charts for the Selection of Highway Culverts."

Bureau of Public Roads (1964). "Peak Rates of Runoff from Small Watersheds in Portions of Kansas," *Hydraulic Engineering Circular No. 4 (Kansas)*.

Byrne, A. T. (1902). *A Treatise on Highway Construction*, 4^{th} ed., John Wiley & Sons, New York.

Chatburn, G. R. (1921). *Highway Engineering: Rural Roads and Pavements*," John Wiley and Sons, New York.

Chow, V. T. (1962). "Hydrologic Determination of Waterway Areas for Design of Drainage Structures in Small Drainage Basins, *Engineering Experiment Station Bulletin No. 462*, University of Illinois.

Cleeman, T. M. (1879). "Proper Amount of Water-Way for Culverts," *Proceedings, Engineers' Club of Philadelphia*, Vol. 1, pp. 146-149.

Clement, R.W. (1987). "Floods in Kansas and Techniques for Estimating Their Magnitude and Frequency on Unregulated Streams," *Water-Resources Investigations Report 87-4008*, U. S. Geological Survey.

Dalrymple, T. (1950). "Regional Flood Frequency," *Research Report 11-B*, Highway Research Board, pp. 4-20.

Dooge, J. C. I. (1957). "The Rational Method for Estimating Flood Peaks," *Engineering*, Vol. 184, pp. 311-313 & 374-377.

Ellis, D. W. and G. W. Edelen, Jr. (1960). "Kansas Stream flow Characteristics. Part 3: Flood Frequency." *Technical Report No. 3*, Kansas Water Resources Board.

Federal Highway Administration (1967). Instructional Memorandum (IM) 20-1-67 (32-44), "Evaluation of Flood Hazards – Federally-Financed Highways," April 26, 1967.

Federal Highway Administration (1969). Policy and Procedure Memorandum 40-2, "Design Standards for Federal-Aid Projects," revision dated May 12, 1969.

Federal Highway Administration (1972). *Hydraulic Engineering Circular No. 10*, "Capacity Charts for the Hydraulic Design of Highway Culverts."

Federal Highway Administration (1972). *Hydraulic Engineering Circular No. 13*, "Hydraulic Design of Improved Inlets for Culverts."

Federal Highway Administration (1979). Code of Federal Regulations, Title 23: Highways; Part 650 – Bridges, Structures and Hydraulics, *Federal Register*, Vol. 44, No. 228, November 26, 1979.

Federal Highway Administration (1985). *Hydraulic Design Series No. 5*, "Hydraulic Design of Highway Culverts."

Federal Highway Administration (2001). *Hydraulic Engineering Circular No. 18*, "Evaluating Scour at Bridges," 4th edition.

Frederick, R. H., V.A. Meyers and E. P. Auciello (1977). "Five- to 60-Minute Precipitation Frequency for the Eastern and Central United States," *NOAA Technical Memorandum NWS HYDRO-35*, National Weather Service.

Furness, L.W. (1957). "Development of a Balanced Stream-gaging Program for Kansas." *Bulletin 1*, Kansas Water Resources Board.

Gillespie, W. M. (1853). *A Manual of the Principles and Practices of Roadmaking: Comprising the Location, Construction, and Improvement of Roads and Railroads, 6th ed.*, A. S. Barnes & Co., New York.

Grover, O. L. (1919). "Determining Sizes of Culverts," *Public Roads*, Bureau of Public Roads, U. S. Department of Agriculture, pp. 39-44.

Gumbel, E. J. (1945). “Floods Estimated by Probability Method,” *Engineering News-Record*, June 14, 1945.

Harrison, L. J., J. L. Morris, J. M. Normann and F. L. Johnson (1972). “Hydraulic Design of Improved Inlets for Culverts,” *Hydraulic Engineering Circular No. 13*, Federal Highway Administration.

Hering, R. (1881). “Sewerage Systems,” *Transactions, American Society of Civil Engineers*, Vol. 10, pp. 361-384.

Herr, L. A. and H. G. Bossy (1965). “Hydraulic Charts for the Selection of Highway Culverts,” *Hydraulic Engineering Circular No. 5*, Bureau of Public Roads.

Herr, L. A. and H. G. Bossy (1972). “Capacity Charts for the Hydraulic Design of Highway Culverts,” *Hydraulic Engineering Circular No. 10*, Federal Highway Administration.

Hershfield, D. M. (1961). “Rainfall Frequency Atlas of the United States for Durations from 30 Minutes to 24 Hours and Return Periods of 1 to 100 Years,” *Technical Paper No. 40*, U. S. Weather Bureau.

Hewes, L. I. And Oglesby, C. H. (1954). *Highway Engineering*, John Wiley and Sons.

Interagency Advisory Committee on Water Data (1981). “Guidelines for Determining Flood Flow Frequency,” *Bulletin 17B*, U. S. Geological Survey.

Irza, T.J. (1966). “Preliminary Flood-Frequency Relations for Small Streams in Kansas.” *Open-File Report*, U. S. Geological Survey.

Izzard, C. F. (1953). “Peak Discharge for Highway Drainage Design,” *Proceedings, American Society of Civil Engineers*, Separate 320.

Jordan, P.R. and T. J. Irza (1975). “Kansas Streamflow Characteristics: Magnitude and Frequency of Floods in Kansas, Unregulated Streams,” *Technical Report No. 11*, Kansas Water Resources Board.

Kansas Water Commission (1920). *Surface Waters of Kansas, 1895-1919*.

Kansas Water Commission (1928). *Surface Waters of Kansas, 1919-1924*.

Kubitza, W. K. (circa 1955). “Hydraulic Entrance Characteristics of Culverts,” research report on hydraulic model studies at Kansas State University, prepared for the Kansas State Highway Commission.

Kuichling, E. (1889). “The Relation Between Rainfall and the Discharge of Sewers in Populous Districts,” *Transactions, American Society of Civil Engineers*, Vol. 20, pp. 1-56.

Laursen, E. M. (1960). "Scour at Bridge Crossings," Journal of the Hydraulics Division, American Society of Civil Engineers, Vol. 86, No. HY2, pp.39-54.

McEnroe, B. M., and P. Gonzalez-Quesada (2003). "Storm Durations and Antecedent Conditions for Flood Discharge Estimation," *Report No. K-TRAN KU-02-04*, Kansas Department of Transportation.

McEnroe, B. M and H. Zhao (2001). "Lag Times of Urban and Developing Watersheds in Johnson County, Kansas," *Report No. K-TRAN: KU-99-5*, Kansas Department of Transportation.

McEnroe, B. M., and H. Zhao (2000). "Lag Times and Peak Coefficients for Small Rural Watersheds in Kansas," *Report No. K-TRAN: KU-98-1*, Kansas Department of Transportation.

McEnroe, B. M., and L. M. Johnson (1994). "Development of Hydraulic Design Charts for Type I and Type III Metal and Concrete End Sections for Pipe Culverts," *Report No. K-TRAN: KU-94-4*, Kansas Department of Transportation.

McEnroe, B. M. and J. A. Bartley (1993). "Development of Hydraulic Design Charts for Type IV End Sections for Pipe Culverts," *Report No. K-TRAN: KU-93-5*, Kansas Department of Transportation, 1993.

McEnroe, B. M., and J. P. Jones (1992). "Rainfall Intensity Tables for Kansas Counties," *Report No. K-TRAN: KU-92-1, Vol. II*, Kansas Department of Transportation (revised 1997).

Miami Conservancy District (1917). "Storm Rainfall of the Eastern United States," *Technical Reports, Part V*, State of Ohio.

Mulvany, T. J. (1851). "On the Use of Self-Registering Rain and Flood Gauges in Making Observations of the Relations of Rainfall and Flood Discharges in a Given Catchment," *Transactions, Institution of Civil Engineers of Ireland*, Vol. 4, Part 2, p. 18.

Mohler, J.C. (1936). *Stream-flow Data 1928-1935*, Kansas State Board of Agriculture, Division of Water Resources.

Mohler, J.C. (1940). *Stream-flow data 1935-1939*, Kansas State Board of Agriculture, Division of Water Resources.

Nagler, F. A. (1918). "Obstruction of Bridge Piers to the Flow of Water," *Transactions, American Society of Civil Engineers*, Vol. 82, p. 334.

Normann, J. M., R. H. Houghtalen and W. J. Johnston (1985). "Hydraulic Design of Highway Culverts," *Hydraulic Design Series No. 5*, Federal Highway Administration.

Oglesby, C. H. and R. G. Hicks (1982). *Highway Engineering*, 4th ed., John Wiley and Sons.

Potter, W. H. (1950). "Surface Runoff from Agricultural Watersheds," *Research Report 11-B*, Highway Research Board, pp. 21-35.

Potter, W. H. (1961). "Peak Rates of Runoff from Small Watersheds," *Hydraulic Design Series No. 2*, Bureau of Public Roads.

Public Roads Administration (1949). *Highway Practice in the United States of America*.

Rasmussen, P. P. and C. A. Perry (2000). "Estimation of Peak Streamflows for Unregulated Rural Streams in Kansas," *Water-Resources Investigations Report 00-4079*, U. S. Geological Survey.

Schaake, J. C., J. C. Geyer and J. W. Knapp (1967). "Experimental Investigation of the Rational Method," *Journal of the Hydraulics Division*, American Society of Civil Engineers, Vol. 93, No. HY6, pp. 353-370.

Smith, R. L. (1982). "Hydrologic Design Utilizing Frequency-Equivalent Hydrographs" (report to Kansas Department of Transportation), Department of Civil Engineering, University of Kansas.

Soil Conservation Service (1954). *Hydrology Guide for Use in Watershed Planning*.

Soil Conservation Service (1956). *National Engineering Handbook*, Section 4, Hydrology.

Soil Conservation Service (1975). "Urban Hydrology for Small Watersheds," Technical Release 55.

Soil Conservation Service (1986). "Urban Hydrology for Small Watersheds," Technical Release 55.

Talbot, A. N. (1887-88). "The Determination of Water-Way for Bridges and Culverts," Selected Papers of the Engineers' Club, *Technograph No. 2*, University of Illinois, pp. 14-22.

Waddell, J. A. L. (1916). "Determination of Waterways," *Bridge Engineering*, Vol. 2, John Wiley & Sons, New York, pp. 1109-1136.

Wellington, A. M. (1886). "Culvert Proportions," Editorial, *Railroad Gazette*, Vol. 18, pp. 629-630.

Yarnell, D. L., S. H. Woodward and F. A. Nagler (1924). "The Flow of Water Through Pipe Culverts," *Public Roads*, Vol. 5, No. 1, Bureau of Public Roads, pp. 19-33.

Yarnell, D. L., F. A. Nagler and S. M. Woodward (1926). "The Flow of Water Through Culverts," *Studies in Engineering, Bulletin 1*, University of Iowa.

Yarnell (1934.) “Bridge Piers as Channel Obstructions,” *Technical Bulletin No. 442,* U. S. Department of Agriculture.

Yarnell, D. L. (1935). “Rainfall Intensity-Frequency Data,” *Miscellaneous Publication No. 204,* U. S. Department of Agriculture.

Young, C. B. and B. M. McEnroe (2002). *Precipitation Frequency Estimates for the Kansas City Metropolitan Area*, Kansas City Metropolitan Chapter, American Public Works Association.

Appendix A

Dun's Drainage Table

Atchison, Topeka and Santa Fe Railway System (1906)

Areas Drained in Square Miles	Areas of Waterway						
	Missouri and Kansas	Cast Pipe. Banks over 15 Ft. Use 80 Per Cent	Box and Arch Culverts. 1st Fig. Diam. 2d. Fig. Bench	Percentage of Column 2			
				Illinois	Indian Territory	Texas	New Mexico
1	2	3	4	5	6	7	8
.01	2.0	1-24 in.	2 x1 B				
.02	4.0	1-24 "	2 x2 "				
.03	6.0	1-30 "	2 x3 "				
.04	7.5	1-36 "	2½x3 "				
.05	9.0	1-42 "	3 x3 "	East of	South of	Use	Use
.06	10.5	1-42 "	3½x3 "	Streator	Purcell	Column	Column
.07	12.0	1-48 "	3 x4 "	use 60	use Texas	2	2
.08	13.5	2-36 "	2½x3 "	per cent	Column		
.09	15	2-36 "	2½x3 "				
.10	16	2-36 "	3 x3 "				
.15	25	2-48 "	3 x4 "				
.20	32	3-42 "	6 x4 A				
.25	38	3-48 "	6 x5 "				
.30	44	......	6 x5½ "				
.35	51	......	8 x4½ "				
.40	56	......	8 x5 "				
.45	62	......	8 x6 "				
.50	66	......	8 x6 "				
.55	70	......	8 x6½ "				
.60	74	......	10 x4½ "				
.65	78	......	10 x5 "				
.70	81	......	10 x5½ "				
.75	85	......	10 x6 "				
.80	88	......	10 x6½ "				
.85	91	......	10 x6½ "				
.90	94	......	10 x6½ "				
.95	97	......	12 x5 "				
1.0	100	......	12 x5 "				98½
1.1	110	......	12 x6 "			105	98½
1.2	120	......	12 x7 "			105	98½
1.3	130	......	12 x8 "			105	98½
1.4	140	......	14 x6½ "	West of	North of	105	98½
1.5	150	......	14 x7 "	Streator	Purcell	105	98½
1.6	160	......	16 x6½ "	use 80	use Col-	105	98½
1.7	170	......	16 x7 "	per cent	umn 2	105	98½
1.8	180	......	16 x7½ "			105	98½
1.9	190	......	16 x8 "			105	98½
2.0	200	......	18 x7 "			105	98½
2.2	220	......	18 x8 "			105	98½
2.4	240	......	18 x9½ "			105	98½
2.6	260	......	20 x8 "			105	98½
2.8	280	......	20 x9 "			105	98½
3.0	300	......	20 x9½ "			105	98½
3.2	321	......	22 x8½ "			105	98½
3.4	340	......	22 x9 "			105	98½
3.6	357	......	24 x8½ "			105	98½
3.8	373	......	24 x9 "			105	98½
4.0	388	......	28 x7 "			105	97
4.2	403	......	28 x7½ "			105	97
4.4	417	......	28 x8 "			105	97
4.6	430	......	28 x8½ "			105	97
4.8	443	......	28 x9 "			105	97
5.0	455	......	28 x9½ "	West of	North of	105	97
5.5	483	......	28 x10 "	Streator	Purcell	105	97
6.0	509	......	32 x7½ "	use 80	use Col-	105	97
6.5	533	......	32 x8 "	per cent	umn 2	105	97
7.0	556	......	32 x9 "			105	97
7.5	579	......	32 x10 "	East of	South of	105	97
8.0	601	......	32 x11 "	Streator	Purcell	105	97
8.5	622	......	32 x11½ "	use 60	use Texas	105	97
9.0	641	......	32 x12 "	per cent	Column	105	93½
9.5	660	......	32 x12½ "			105	93½
10	679	......	32 x13 "			105	93½
11	710	......				105	93½
12	740	......				105	93½
13	775	......	Bridges de-			105	93½
14	805	......	signed to			105	93½
15	835	......	provide area			105	93½
16	865	......	according to			105	94
17	890	......	circumstances			105	94
18	920	......				105	94
19	945	......				105	94
20	970	......				105	94
22	1,015	......				105	94

Areas Drained in Square Miles	Areas of Waterway				
	Missouri and Kansas	Percentage of Column 2			
		Illinois	Indian Territory	Texas	New Mexico
1	2	5	6	7	8
24	1,060			110	94
26	1,100			110	92
28	1,140			110	92
30	1,180			110	92
32	1,220	East of	South of	110	92
34	1,255	Streator	Purcell	110	92
36	1,290	use 60	use Texas	110	91
38	1,320	per cent	Column	110	91
40	1,350			110	91
45	1,435			110	91
50	1,510			110	89½
55	1,580			115	89½
60	1,650			115	89½
65	1,720			115	88
70	1,780			115	88
75	1,840			115	88
80	1,900			115	86½
85	1,960			115	86½
90	2,015			115	86½
95	2,065			115	86½
100	2,120			120	85
110	2,220			120	85
120	2,315			120	85
130	2,405			125	83½
140	2,500			125	83½
150	2,580			130	82
160	2,665			130	82
170	2,745			130	80½
180	2,820			130	80½
190	2,900			130	79
200	2,970			130	79
220	3,115	West of	North of	130	77½
240	3,245	Streator	Purcell	130	77½
260	3,370	use 80	use Col-	130	76
280	3,495	per cent	umn 2	130	76
300	3,615			130	74½
325	3,770			130	74½
350	3,900			130	73
375	4,035			130	73
400	4,165			130	71½
450	4,385			130	70
500	4,610			130	68½
550	4,825			130	67
600	5,030			130	65½
650	5,230			130	64
700	5,420			130	62½
750	5,610			130	61
800	5,800			130	59½
850	5,890			130	58
900	6,080			130	56½
950	6,230			130	....
1,000	6,380			130	....
1,100	6,705	West of	North of	130	....
1,200	6,960	Streator	Purcell	130	....
1,300	7,230	use 80	use Col-	130	....
1,400	7,480	per cent	umn 2	130	....
1,500	7,725			130	....
1,600	7,960	East of	South of	130	....
1,700	8,195	Streator	Purcell	130	....
1,800	8,390	use 60	use Texas	130	....
1,900	8,625	per cent	Column	130	....
2,000	8,820			130	....
2,200	9,240			130	....
2,400	9,605			130	....
2,600	9,970			130	....
2,800	10,320			130	....
3,000	10,640			130	....
3,500	11,445			130	....
4,000	12,160			130	....
4,500	12,825			130	....
5,000	13,500			130	....
5,500	14,080			130	....
6,000	14,520			130	....
6,500	15,140			130	....

The above classification by states is for convenience only, and merely denotes the general characteristics of topography and rainfall.

Column 2 in this table is prepared from observations of streams in Southwest Missouri, Eastern Kansas, Western Arkansas and the southeastern portions of the Indian Territory. In all this region steep, rocky slopes prevail and the soil absorbs but a small percentage of the rainfalls. It indicates larger waterways than are required in Western Kansas and level portions of Missouri, Colorado, New Mexico and Western Texas.

Source: Chow (1962)

Appendix B

Dun's Drainage Table for Kansas (1927)

THE DUN DRAINAGE TABLE
FOR KANSAS

Drainage Area Sq. Miles	Drainage Area Acres	Area of Waterway	Drainage Area Sq. Miles	Drainage Area Acres	Area of Waterway	Drainage Area Sq. Miles	Drainage Area Acres	Area of Waterway
.01	6.4	2.0	4.4	2816	417	140	89600	2500
.02	12.8	4.0	4.6	2944	430	150	96000	2580
.03	19.2	6.0	4.8	3072	443	160	102400	2665
.04	25.6	7.5	5.0	3200	455	170	108800	2745
.05	32.0	9.0	5.5	3520	483	180	115200	2820
.06	38.4	10.5	6.0	3840	509	190	121600	2900
.07	44.8	12.0	6.5	4160	533	200	128000	2970
.08	51.2	13.5	7.0	4480	556	220	140800	3115
.09	57.6	15	7.5	4800	579	240	153600	3245
.10	64	16	8.0	5120	601	260	166400	3370
.15	96	25	8.5	5440	622	280	179200	3495
.20	128	32	9.0	5760	641	300	192000	3615
.25	160	38	9.5	6080	660	325	208000	3770
.30	192	44	10	6400	679	350	224000	3900
.35	224	51	11	7040	710	375	240000	4035
.40	256	56	12	7680	740	400	256000	4165
.45	288	62	13	8320	775	450	288000	4385
.50	320	66	14	8960	805	500	320000	4610
.55	352	70	15	9600	835	550	352000	4825
.60	384	74	16	10240	865	600	384000	5030
.65	416	78	17	10880	890	650	416000	5230
.70	448	81	18	11520	920	700	448000	5420
.75	480	85	19	12160	945	750	480000	5610
.80	512	88	20	12800	970	800	512000	5800
.85	544	91	22	14080	1015	850	544000	5990
.90	576	94	24	15360	1060	900	576000	6080
.95	608	97	26	16640	1100	950	608000	6230
1.0	640	100	28	17920	1140	1000	640000	6380
1.1	704	110	30	19200	1180	1100	704000	6705
1.2	768	120	32	20480	1220	1200	768000	6960
1.3	832	130	34	21760	1255	1300	832000	7230
1.4	896	140	36	23040	1290	1400	896000	7480
1.5	960	150	38	24320	1320	1500	960000	7725
1.6	1024	160	40	25600	1350	1600	1024000	7960
1.7	1088	170	45	28800	1435	1700	1088000	8195
1.8	1152	180	50	32000	1510	1800	1152000	8390
1.9	1216	190	55	35200	1580	1900	1216000	8625
2.0	1280	200	60	38400	1650	2000	1280000	8820
2.2	1408	220	65	41600	1720	2200	1408000	9240
2.4	1536	240	70	44800	1780	2400	1536000	9605
2.6	1664	260	75	48000	1840	2600	1664000	9970
2.8	1792	280	80	51200	1900	2800	1792000	10320
3.0	1920	300	85	54400	1960	3000	1920000	10640
3.2	2048	321	90	57600	2015	3500	2240000	11445
3.4	2176	340	95	60800	2065	4000	2560000	12160
3.6	2304	357	100	64000	2120	4500	2880000	12825
3.8	2432	373	110	70400	2220	5000	3200000	13500
4.0	2560	388	120	76800	2315	5500	3520000	14080
4.2	2688	403	130	83200	2405	6000	3840000	14520

July 1927.

Source: KDOT Bureau of Design (archives)

K - TRAN

KANSAS TRANSPORTATION RESEARCH
AND
NEW - DEVELOPMENTS PROGRAM

A COOPERATIVE TRANSPORTATION RESEARCH PROGRAM BETWEEN:

KANSAS DEPARTMENT OF TRANSPORTATION

THE UNIVERSITY OF KANSAS

KANSAS STATE UNIVERSITY