ANALYSIS OF DROUGHT FLOWS AND OF METHODS FOR DETERMINING THE SELF-PURIFICATION CAPACITY OF THE ILLINOIS RIVER

By

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ANALYSIS OF DROUGHT FLOWS AND OF METHODS FOR
DETERMINING THE SELF-PURIFICATION CAPACITY
OF THE ILLINOIS RIVER

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF SYMBOLS

$p(x)$ - Probability of an occurrence being less than or equal to x; for drought flows, the probability of the flow being greater than or equal to x

T_x - Recurrence interval or return period; $T_x = \frac{1}{1-p(x)}$

e - Base of natural logarithms = 2.71828

ε - Minimum low flow approached in Gumbel's extremal value theory for droughts

n - Number of observed occurrences

$\bar{X}$ - The arithmetic mean, $\bar{X} = \frac{1}{n} \sum_{i=1}^{n} X_i$

s - Standard deviation, $s = \frac{1}{n-1} \sum_{i=1}^{n} (X_i-\bar{X})^2$

C_v - Coefficient of variability, $C_v = \frac{S}{\bar{X}}$

a - Skewness, $a = \frac{n}{(n-1)(n-2)} \sum_{i=1}^{n} (X_i-\bar{X})^3$

C_s - Coefficient of skewness, $\frac{a}{S^3}$

K_2 - Coefficient of reaeration, days^{-1}

K_L - Gas mass transfer coefficient, $K_L = K_2 x H$, days^{-1} feet

K_1 - Coefficient of deoxygenation, days^{-1}

D - Dissolved oxygen deficit, mg/l

D_a - Initial dissolved oxygen deficit, mg/l

D_c - Critical dissolved oxygen deficit, mg/l

D_{max} - Maximum allowable dissolved oxygen deficit, mg/l

L_a - Initial ultimate oxygen demand of organic matter, mg/l

t - Time in days

t_c - Time in days downstream to the critical dissolved oxygen deficit

A_i - Interfacial surface area of stream

L - Length of reach - miles or feet as designated

$\frac{dM}{dt}$ - Rate of gas transfer; pounds of oxygen per day

cfs - Cubic feet per second

BOD - Biochemical oxygen demand, mg/l

DO - Dissolved oxygen concentration, mg/l

H - Hydraulic depth in feet

W - Top width of flow in feet

Q - Flow rate in cfs

A - Cross-sectional area, ft^2

n - Manning's coefficient of roughness

WP - Wetted perimeter, feet

S - Slope of hydraulic grade line

CHAPTER I

INTRODUCTION

An increased public awareness of pollution of the waters of the United States culminated in the Federal Water Pollution Control Act Amendments of 1972 (1). Laws which gave the Federal government the power to seek an immediate court injunction against polluters endangering public health or livelihood, also formed a structured pollution abatement program. The United States Environmental Protection Agency was empowered to take direct action, or provide assistance to states retaining the primary responsibility for the quality of their waters.

The Illinois River in Oklahoma is a river of unspoiled natural beauty, having a drainage area of 1660 square miles in Oklahoma and Arkansas. Discharges into this river in Oklahoma are presently almost solely a function of precipitation. Pasture and woodlands occupy the majority of the basin area, with less than ten percent being harvested cropland. Tahlequah is the only municipality located in the Oklahoma part of the basin with a population of over one thousand. The beauty of this wilderness-type area has attracted a growing number of recreationists, with canoeing and floating down the river a favorite pastime.

The unspoiled quality of the area has also attracted the developer. Solitude is an entity that many people are beginning to seek in its growing scarcity, and a cabin along the banks of the free and easy flowing Illinois presents the perfect "get away for it all" home.

The quest for this retreat is suggested in a 3000-lot development by Frates Properties, Inc., near the junction of Flint Creek and the Illinois River. This development will be larger than any city in the Oklahoma part of the basin. Controversy has been stirred over the proposed septic tank sewerage system of this development.

Plans were presented in Arkansas for increased use of the organic waste assimilative capacity of the river for the disposal of municipal secondary effluent wastes. Two regional wastewater treatment plants were planned to discharge directly into the Illinois River. Although the effects of the discharges were not known, it was qualitatively decided that Lake Frances, on the Illinois River at the Oklahoma-Arkansas border, should no longer be used for the Siloam Springs water supply.

A challenge lies in the opportunity to prevent excessive pollution of the Illinois River, rather than to pollute and then spend dollars to clean up the damage. A practical design is thus required to assess the pollutional capacity of the river and its tributaries. Water quality planning can become feasible only if it is known to what extent wastes can be assimilated by the waters and not cause a detrimental effect to the life it supports. The pollutional capacity of a stream depends on many variables, including the hydrology and geology of the area. It is these factors that will determine streamflow and the initial quality of water in the stream. These characteristics must be known before any type of estimate can be made on pollutional discharge limitations into the river course. Some analyses on available data concerning these characteristics are presented in this study for the Illinois River basin. The first analysis will be a statistical determination of low flows using available streamflow data, for it is at low flows that a stream will have the least dilution capacity for assimilation of pollution materials. The second part will involve correlating the determined low flows with cross-sections and ratings of flow versus gage height in order to try to define the area, depth, and velocity of flow at these gaging stations, as these are affectors of reaeration rate.

The most common measurement of a stream's viability is the dissolved oxygen (DO) concentration. It is the dissolved oxygen in the stream which supports living organisms in the aquatic environment. Organic wastes are the pollutional material that have been found to cause the greatest drain on the oxygen supply of the stream. This is by biological degradation, or use of the carbon and oxygen for synthesis and respiration of microorganisms. This oxygen drain, when existing in a stream, is in constant competition with oxygen being replenished by the atmosphere in seeking an equilibrium concentration. Based on these concepts, mathematical models for predicting the dissolved oxygen levels in a stream were initiated by Streeter and Phelps in 1926 (2). Since this time, modifications of their methods and new concepts for predicting the DO profile in a stream have evolved. One new concept of planning for minimum DO levels, Busch's solution for stream assimilation capacity (3), will be studied in this thesis and compared to the traditional Streeter-Phelps equation.

Data concerning the Illinois River is presently insufficient for any developmental plans to be made with confidence that the existing pyramind of life would not be upset along the river. It does not seem practical to allow development and increased industry to expand until a pollution problem necessitates using more advanced technology to alleviate a situation that may have been prevented by better planning. Each state has a responsibility for basin planning and monitoring, as outlined in Public Law 92-500. The Environmental Protection Agency requires environmental impact statements in connection with construction grants for publicly owned waste treatment facilities, and when issuing permits for discharges of pollutants from new sources. No point source discharge is allowed without a permit. The Department of Health, Education and Welfare is required to make an environmental impact statement for the septic tank sewerage system of the Frates' Flint Ridge Development. Eventually, someone must start gathering further information and data on the Illinois River basin and initiate a planning process.

Even if a good data base is obtained and calculations made as to what discharges can be allowed with the minimum DO concentration maintained, a further problem of design still exists. The river translates special qualities, imitating a wilderness area, that public sentiment increasingly feels is deserving of a public park. Legislation specially designed for the Illinois River exists in the Oklahoma Scenic Rivers Act. Federal legislation is being sought to include the river in the National Wild and Scenic Rivers Act, which would further inhibit development along the shoreline. But even in this case, parks attract people, and too many people pursuing recreational activities on the river pose an additional pollution problem.

So, no matter what the type of development, the river must be understood if it is to remain a viable resource. This necessitates having a data base which will allow planning for future development.

A challenge exists to obtain and translate information on the Illinois River basin that will allow plans to be formulated that contain minimal conflict with existing natural plans. The purposes of this thesis are to present initial analyses of low flow data on the Illinois River, to compare Busch's proposed water quality model to the Streeter-Phelps equation, and to further define the water quality management problem of the Illinois River basin.

CHAPTER II

LITERATURE REVIEW

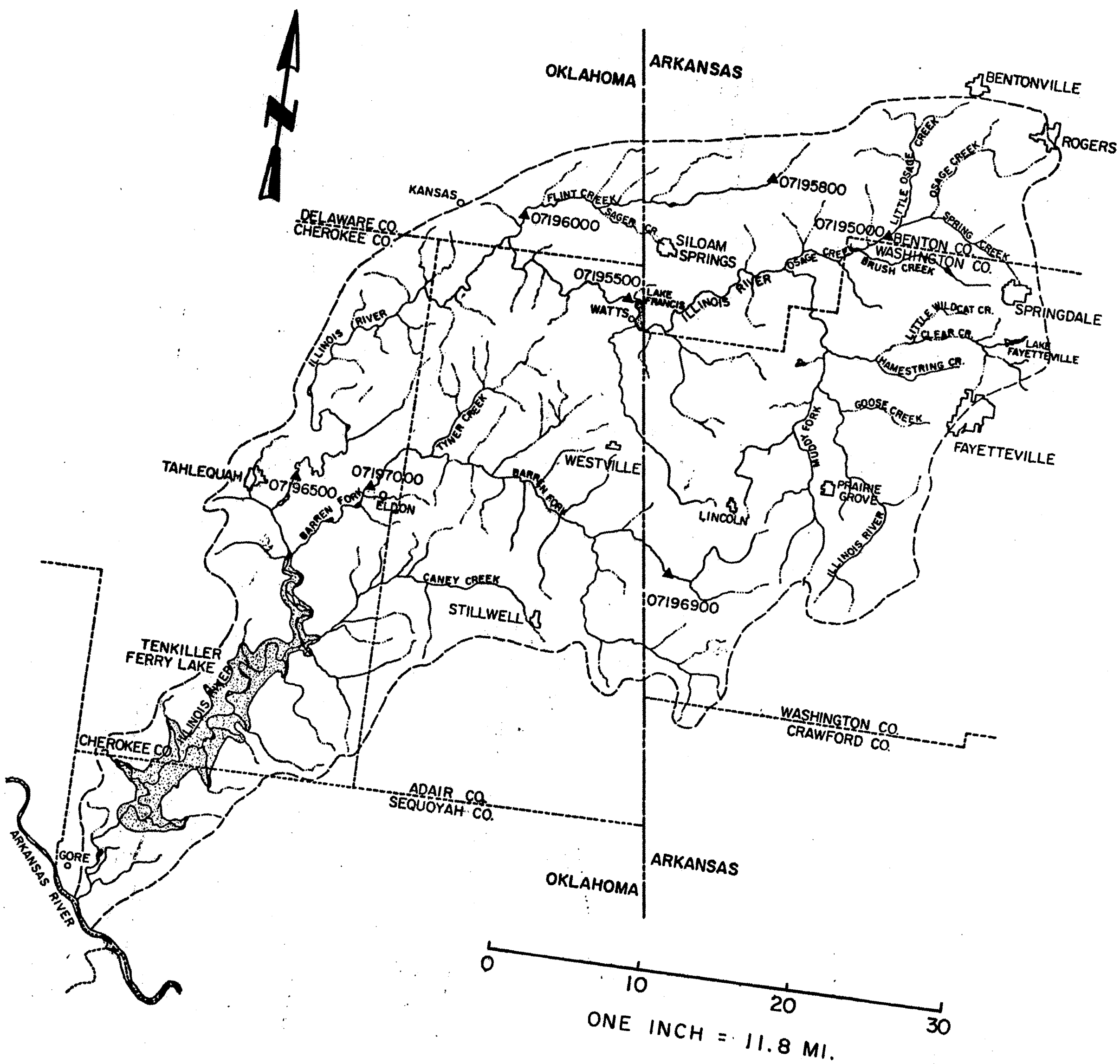
This chapter consists of five units. First, a summary of available historical data on the quantity and quality of waters in the Illinois River basin will be presented, followed by a survey of the methods for statistical analysis of these records in the treatment of drought flows. The third unit will describe mathematical models available for predicting the dissolved oxygen profile in a stream. Next will be a further defining of the problem confronting the Illinois River, by looking at the development which has taken place and development which is planned for the basin. Finally, regulations that concern future water quality planning for the basin will be presented.

A. Available Streamflow Data

The Illinois River basin is located in Benton and Washington Counties of Arkansas, and Cherokee and Adair Counties of Oklahoma, as shown in Figure 1. Its source is in the Boston Mountains near Fayetteville, Arkansas, from whence it flows westerly across the state, then turns southerly to its junction with the Arkansas River. Principal tributaries include Osage Creek, Ballard Creek, Flint Creek, Barren Fork, Caney Creek, Muddy Fork, and Evansville Creek.

The topography of the basin ranges from rolling hills to mountainous. The basin has a drainage area of 1660 square miles, of which 900

Figure 1. Map of the Illinois River Basin and Gaging Stations


OKLAHOMA
ARKANSAS
BENTONVILLE
ROGERS
KANSAS
07195800
LITTLE OSAGE CREEK
OSAGE CREEK
DELAWARE CO.
CHEROKEE CO.
FLINT CREEK
SAGER CR.
07196000
SILOAM SPRINGS
07195000
BENTON CO.
WASHINGTON CO.
SPRING CREEK
OSAGE CREEK
BRUSH CREEK
07195500
LAKE FRANCIS
WATTS
ILLINOIS RIVER
SPRINGDALE
LITTLE WILDCAT CR.
CLEAR CR.
LAKE FAYETTEVILLE
HAMESTRING CR.
ILLINOIS RIVER
GOOSE CREEK
TYNER CREEK
MUDDY FORK
FAYETTEVILLE
TAHLEQUAH
07196500
07197000
WESTVILLE
BARREN FORK
ELDON
BARREN FORK
PRAIRIE GROVE
LINCOLN
ILLINOIS RIVER
CANEY CREEK
STILLWELL
07196900
TENKILLER FERRY LAKE
ILLINOIS RIVER
WASHINGTON CO.
CRAWFORD CO.
CHEROKEE CO.
ADAIR CO.
SEQUOYAH CO.
GORE
ARKANSAS RIVER
ARKANSAS
OKLAHOMA
0
10
20
30
ONE INCH = 11.8 MI.

square miles lie in Oklahoma. The elevation of the river varies from 1600 feet mean sea level at the source, to 500 feet mean sea level at the mouth.

The climate of the area is typified by long, hot summers and short, mild winters. The average annual precipitation ranges from 44 to 46 inches. Average temperatures are from 50^{o} to 63^{o}F.

There are six United States Geological Survey streamflow gaging stations in the Illinois River basin, and are located as shown on a map of the basin on Figure 1. Pertinent data concerning these gaging stations is given in the following table:

TABLE I

STREAMFLOW GAGING STATIONS FOR THE ILLINOIS RIVER BASIN IN OKLAHOMA

USGS Gaging Station	Location	Drainage Area	Period of Record
07195500	Illinois River, near Watts, Oklahoma	635 $mi.^2$	8/55 to 9/73
07196000	Flint Creek, near Kansas, Oklahoma	110	8/55 to 9/73
07196500	Illinois River, at Tahlequah, Oklahoma	959	10/35 to 9/73
07197000	Barren Fork, near Eldon, Oklahoma	307	10/48 to 9/73
07198000	Illinois River, near Gore, Oklahoma	1626	4/39 to 9/73

The station near Gore is located below Lake Tenkiller and will not be of concern in this thesis.

Daily records of streamflow are published by the United States Geological Survey in the annual series "Surface Water Supply of the United States, Part 7" (4), with more recent daily data available in "Water Resources Data for Oklahoma, Part 1" (5).

Water quality data has been collected at two of the streamflow gaging stations in Oklahoma--at the Watts station and at the Gore station--with the following period of record:

TABLE II

WATER QUALITY GAGING STATIONS FOR THE ILLINOIS RIVER BASIN IN OKLAHOMA

USGS Gaging Station	Analysis	Period of Record
Watts 07195500	chemical quality	10/55 to 8/56 10/59 to 5/61 7/69 to present
	temperature and dissolved oxygen	7/69 to present
Gore 07198000	chemical quality	10/53 to present
	temperature	10/53 to 9/63

Water quality data is reported in the United States Geological water supply papers, "Quality of Surface Waters of the United States, Part 7" (6), with more recent data available in "Water Resources Data for Oklahoma, Part 2" (7).

The chemical water quality of the Illinois River is indicated to be excellent, and the waters are of good quality for use in municipal, agricultural, and most industrial uses. Concentrations of sulfates, chlorides, and nitrates are usually less than 10 mg/l, and hardness averages about 85 mg/l as calcium carbonate. The pH is slightly basic, ranging from 7.2 to 8.0. The sediment load carried by the river is usually small, and exhibits fast settling characteristics under quiescent conditions. Recent data taken on dissolved oxygen levels indicates that the stream also has excellent biochemical quality.

In 1959, the United States Geological Survey published two reports on the Illinois River basin. One was a hydrographic survey assessing the water use and the issuance of water rights on the Illinois River and its tributaries made at the request of the City of Tulsa (8). The other was a companion report on the magnitude, distribution, and quality of waters of the Illinois River (9). In this latter report, the occurrence of low flows was studied at Tahlequah, and at Eldon on the Barren Fork. This data will be discussed in the part of this chapter on low flows.

In 1969, a water resources planning study was completed by the Oklahoma State University and the University of Arkansas which summarized all available hydrologic data on the Arkansas River and its tributaries (10). A compact commission used this data for allocating reliable conservation storage to the states of Oklahoma and Arkansas. A follow-up report was completed in 1971 on water quality management (11).

B. Low Flow Analysis

The design flow for pollution control is usually based on statistical analysis of historical records of drought flow, for it is at low flow that the stream will have the least capacity to assimilate organic waste materials and maintain an acceptable DO concentration. A year is the basic time unit when dealing with streamflow, and the year is usually defined as from March to April for drought flow analysis in order to include the dry part of the year as a whole. The extreme low flow for the year is determined as the average daily flow for drought flows of various durations, with the low 1-day, 7-day, or 30-day average low flow sufficing for many practical applications.

For flood flows, the base time unit of one year yields extreme values which are independent events, but this condition may not hold for drought flows. Hydrological factors influencing drought flow may extend the period through which completely independent minima may occur to over two years for some drainage basins (12). Therefore, some drought flows determined from records of single years may not truly be drought flows, or completely independent minima from year to year.

Plotting data on probability paper is the most common engineering treatment of statistical data, and requires ranking the occurrences for determining plotting positions. Weibull's plotting position formula is used for many statistical distributions and has also been recommended for extreme value distributions (13). This formula to determine the probability of an occurrence being less than or equal to a given value is

$$p(x) = \frac{m}{n+1} \qquad (1)$$

where m is the rank of the occurrence, n is the total number of occurrences, and p(x) is the probability of an occurrence being less than or equal to x. By ranking occurrences in increasing order of magnitude, using this plotting position formula and plotting on probability paper, the magnitude of an occurrence for any given recurrence interval may be obtained by

$$T(x) = \frac{1}{1-p(x)} \tag{2}$$

where T(x) is the recurrence interval, or the average return period (in years) for an occurrence of a given magnitude.

Since drought flows are a set of extreme values, the applicability of extreme value theory to drought flows is in order. This theory assumes that the more extreme values deviate from the mean to a greater extent than the values below the mean-a skewed distribution exists for the data.

Gumbel (14) has modified the extreme value theory developed for floods for drought flows. Gumbel's extremal value theory for flood flows is based on the equation:

$$p(x) = e^{-e^{-y}} \tag{3}$$

where e is the base of natural logarithms, and y is a function of the streamflow.

For drought flows, the extreme value distribution developed by Gumbel becomes a three-parameter distribution of the form:

$$p(x) = \exp -\left[\left(\frac{x-\varepsilon}{u-\varepsilon}\right)^{\alpha}\right] \tag{4}$$

where ε is the minimum flow approached and is greater than or equal to zero, while u, x, and α are calculated from the mean ($\bar{X}$), standard deviation (s), and skewness (a) of the distribution.

A test for whether a set of low flow observations conforms to the extreme value theory for low flows is given by Gumbel:

$$\varepsilon \gtreqless 0 \text{ if } \bar{X}+s\left(A(\alpha) - B(\alpha)\right) \gtreqless 0 \tag{5}$$

Values of $A(\alpha)$ and $B(\alpha)$ are given in a table developed by Garabedian for observed coefficients of skewness (14). If from this test, ε assumes a negative value, the theory is not applicable for the data.

A result of application of this theory is that the logarithms of drought flows may be plotted versus the probability of the flow being less than or equal to a given severity of drought flow on extremal probability paper. The drought flows are ranked in decreasing order of magnitude for determining their plotting position. Gumbel's extreme value theory for low flows is thus referred to as Gumbel's logextremal distribution.

The data will plot a straight line if the minimum low flow for the distribution is zero ($\varepsilon=0$), and will be concave downward if the minimum low flow approached is greater than zero ($\varepsilon>0$). A concave upward curve indicates that for the distribution of $\varepsilon<0$ and hence the data does not conform to Gumbel's logextremal distribution for low flows.

Other types of theoretical distributions that have been used to fit low flow data are log-normal and Log Pearson Type III. These distributions are based on three parameters, and are applicable to drought flows in that they assume a minimum value at one end of the distribution. The log-normal distribution can be easily applied graphically by plotting the logarithms of the data on normal probability. A straight line will be formed if the logarithms of the data are a normal distribution. To fit the data by the Log Pearson Type III method requires finding $\bar{x}$, S, and a, for the distribution (13).

Fifty-five streams in the State of New York were statistically analyzed for drought flow distributions by log-normal, Log Pearson Type III, and Gumbel's logextremal value methods of theoretically fitting data in a study by O'Connor (15). The streams were selected on the criterion that the length of record must have been longer than twenty-five years, the stations were to be uniformly distributed throughout the state, and no significant diversions or controls on the streams could be present. The drought flows analyzed were based on the yearly minimum 7-day average low flows.

Conclusions of the study by O'Connor were that the parameters defining the log-normal distribution are more easily understandable and facilited better understanding of the distribution. Also, the length of record on most of the streams was shorter than thirty years and was too short to draw any definite conclusions concerning the reliability of a fit defined by any of the three methods. The log-normal method of fitting was recommended from the results of the study.

Matalas (16), in a study for the United States Geological Survey, analyzed the 7-day average low flow distribution of 40 gaging stations throughout the United States. This was done using theoretical statistical fits of the log-normal, Log Pearson Type III, and Gumbel's log-extremal value distributions. The Gumbel distribution and the Log Pearson Type III were found to best fit the data, and were almost synonomous in their fitting of the data.

Hardison and Martin (17) did a low flow frequency analysis for the United States Geological Survey on 85 stream gaging stations in twenty-two states south of the Great Lakes and east of the Mississippi, but also including Arkansas, Oklahoma, Louisiana, Missouri, and Texas. The study was done using logextremal probability paper to define all of the distributions graphically. The data was plotted for several durations of low flow, varying from the 7-day average to the 274-day average low flow. The variability of the resulting curves indicated that much difficulty would be encountered in attempting to fit a single theoretical statistical distribution to all of the data at every station. The Mountain Fork River near Eagletown, Oklahoma, exhibited a very steeply sloping, concave upward curve that was not duplicated by any of the other streams in the study.

Some of the questions raised in the report by Hardison and Martin were to what extent the slope of the curves depended on the rate of base flow recession, and to what extent the frequency distribution was influenced by the length of dry periods. Also, to what extent the spacing between curves of low flows at different duration periods depended upon the rainfall that falls during periods of low flow was asked by the report.

Velz (12)(18) recommends the use of Gumbel's logextremal probability paper and explains in detail the application of this method. His work was done principally on predicting low flows for various water courses in the State of Michigan. From the results of these studies, he presented a method for graphically analyzing the case of a concave downward curve on logextremal probability paper. This is for the case when the minimum flow approached is greater than zero from Gumbel's theory. This method can also be applied to certain cases of flow regulation.

Riggs (19) in a survey of the results from various theoretical statistical and logextremal graphical analyses of drought flows, including those cited in this literature review, concluded that:

1) A long streamflow record is best for determining low flow characteristics in a basin. In the absence of a long period of record, correlation of the data with that of neighboring basins to extend the period of record is desirable if a good correlation of observed data exists.

2) Particular basin characteristics define the shape of the frequency curve; no one shape is generally applicable.

3) The effects of basin characteristics and sampling errors are much greater than errors in fitting a curve to plotted points; thus, the use of a theoretical distribution has little if any advantage over a graphical fit.

Kincannon, Kao, and Stover (11)(20) studied the low flow distributions of three gaging stations located on Bird Creek and the Arkansas River in northeastern Oklahoma. The distributions were fitted using the Johnson S_B distribution and Gumbel's extreme value distribution for low flows. This was done for the 1, 3, 7, 14, 30, 60, and 90-day low flows for the streams. Nearly all of the flow distributions at the three stations had a lower flow limit of zero. Two cases, the 60 and 90-day low flows on the Arkansas River, had a lower flow limit greater than zero. The Johnson S_B distribution gave a better fit for the flows on the Arkansas River, while Gumbel's distribution gave the best fit for the small flow station on Bird Creek. It was concluded from this study that the Johnson S_B distribution, which assumes both an upper and lower limit for the drought flows, gave a better fit of the data.

A low flow analysis was done by the United States Geological Survey on the Barren Fork at Eldon, and the Illinois River at Tahlequah in 1959 (9). This study was done by plotting the data on logextremal probability paper. The data was correlated with the White River in Arkansas in order to extend the period of record. The graphs presented in the report by the USGS show straight line plots at Eldon for the 7- and 30-day low flow durations, and presents two intersecting straight lines at an obtuse angle at Tahlequah for the 7- and 30-day average low flows. The first line was moderately sloping for probabilities less than 80 percent, and very steeply sloping for probabilities greater than 80 percent. The flows at the 10-year recurrence interval for the 7- and 30-day average low flow distributions were determined to be 12 cfs and 20 cfs, respectively, at Tahlequah and 4.2 cfs and 5.7 cfs at Eldon on the Barren Fork. No discussion was given concerning these plots, and no data points were plotted to define the lines.

The consensus of the work done on low flows is that Gumbel's log-extremal distribution is the most generally applicable for low flow data. Plotting the data on logextremal probability paper facilitates defining this distribution. This was done for the gaging stations at Tahlequah and Eldon in the Illinois River basin by the United States Geological Survey, but was done using 16 years less data than is now available. An examination of the applicability of Gumbel's logextremal theory to stations in the Illinois River basin will be presented in this thesis.

C. Mathematically Modelling Dissolved Oxygen Concentrations

The understanding of the importance of dissolved oxygen in relation to the ability of a stream to oxidize organic matter owes much to the pioneer work in sewage biochemistry done in England near the end of the 19th Century, and continued in America after the turn of the century. Studies on the nature of organic stream self-purification were empirical in that recording stream conditions in analytical terms was made with no development made toward a set of general principles.

The first attempt at mathematically defining stream self-purification was made by Streeter and Phelps (2). The concepts and mathematical formulations which they presented are still being used with little modification in many instances even today, although with much reservation.

Streeter and Phelps viewed the deoxygenation characteristics of a stream as the liabilities on a balance sheet, and reaeration as assets which must be related to time, temperature, and other physical characteristics of the stream. The governing law they presented for deoxygenation was "the rate of biochemical oxidation of organic matter is proportional to the remaining concentration of unoxidized substance, measured in terms of oxidizability" (2), or

$$- \frac{dL}{dT} = K_1 L \tag{6}$$

where L is the oxygen demand of the organic substance remaining, t is the time elapsed, and K_1 is a constant defining the rate at which the reaction proceeds. On the reaeration side of the balance, the rate of oxygen replenishment was found to be proportional to the oxygen deficit remaining at any time, or

$$- \frac{dD}{dt} = K_2D \tag{7}$$

where D is the oxygen saturation deficit, t is the time elapsed, and K_2 is a constant affecting the rate of oxygen transfer across the interface. Taking these two factors, deoxygenation and reaeration, and adding them as on a balance sheet, yields the differential equation:

$$\frac{dD}{dT} = K_1L - K_2D \tag{8}$$

which, when solved, yields the classical Streeter-Phelps equation:

$$D = \frac{K_1L_a}{K_2-K_1} \left(e^{-K_1t} - e^{-K_2t}\right) + D_a e^{-K_2t} \tag{9}$$

where D_a is the initial dissolved oxygen saturation deficit, D is the saturation deficit at time t, and L_a is the initial ultimate oxygen demand, all in mg/l; t is the time elapsed in days, and K_1 and K_2 are the coefficients of deoxygenation and reaeration in days^{-1}.

Though it is used in many practical problems, there are many problems associated with the concepts and usage of the Streeter-Phelps sag equation. The equation is based on the assumptions that

1) the flow is steady and uniform throughout the reach,

2) there is only one source of pollutant discharge per reach, a point discharge which upon mixing becomes constant in concentration throughout the cross-sectional area of flow,

3) there is only one type of oxygen demand in the reach, and that

is caused by the point discharge,

4) oxygen transfer takes place only from the atmosphere to the stream,

5) reaeration and deoxygenation can be defined by first-order decreasing rate equations, and

6) the coefficients of deoxygenation and reaeration are constant for a given reach.

In the second assumption, the value of the initial oxygen demand will not properly define the amount of organic material in the reach if there are other inflows, channel scouring, or sedimentation adding or subtracting organic material. Also, there will be a time and distance involved in the complete mixing of the pollutant which will depend on stream characteristics such as cross-section and velocity.

The third assumption eliminates the existence of abnormal oxygen demands in the stream such as nitrification, benthnic and sludge deposits, immediate oxygen demands such as the oxidation of hydrogen sulfide, and biological extraction and accumulation on rocks and shorelines (12). Photosynthetic organisms can also cause an abnormal depletion of oxygen through respiration at night, then produce oxygen by photosynthesis, serving as a source of oxygen other than the atmosphere.

In the fifth assumption, experimentation verifies the expressing of reaeration as a first-order decreasing rate reaction, but serious doubts are present in the assumption that deoxygenation, or oxygen uptake, proceeds by first-order decreasing rate kinetics. Studies by Bhatla and Gaudy (21) show that the kinetics of oxygen uptake in a BOD bottle (often used for determination of kinetics of oxygen uptake in the stream) are many times characterized by an early exponentially increasing phase similar to a microbial growth curve, then followed by a plateau and another autocatalytic-type curve. These kinetics for oxygen uptake which vary for different situations, many times defy approximation by a first-order decreasing rate equation.

Another inconsistency in this assumption is that oxygen uptake has been found to vary with the concentration of waste. Jennelle and Gaudy (22) have shown that a Monod-type relationship exists between waste concentration and the rate of oxygen uptake in the exponential phase of uptake. Thus, the bottle dilution technique of BOD determination should not be used to define the rate of oxygen uptake in the stream except at that concentration.

Another shortcoming in the BOD bottle technique is that deoxygenation may be affected by mixing. Work by Ali and Bewtra (23) shows a definite indication that oxygen uptake was affected by mixing, while Jennelle and Gaudy (22) noted that mixing played no significant effect in their studies. Thus, the coefficient of deoxygenation may not truly describe kinetics of oxygen uptake, and this, coupled with the shortcomings of determination in a BOD bottle, allows that only a compensating error in the determination of K_2 may describe the true dissolved oxygen profile in a stream.

The sixth assumption is weakened by the fact that in the dynamic and varying conditions of the environment, the constants will change with different hydrological conditions. In truth, K_1 and K_2 are not really constants, but vary with temperature, turbulence, waste loading, streamflow, weather, and other factors (24).

Due to these limitations in the Streeter-Phelps equation, there have been innumerable modifications of this sag equation for determining the dissolved oxygen depletion curve in a stream. Thomas (25) modified the equation by adding the constant K_3, which is indicative of the type of waste loading and the quiescence or turbulence of the receiving water. Camp (26) modified the sag equation to include the effects of photosynthesis by adding another constant. Hull (27) derived a modification intent on eliminating the need for K_1, K_2, and the time of travel by incorporating these constants into a single constant. This modification cannot be used without field data and the realization that the Streeter-Phelps equation is more of an empirical than rational formulation for natural stream purification. These and other modifications of the sag equation attempt to take into account the effects of photosynthesis, sedimentation, local runoff, plant respiration, and benthal demands.

In addition to modifications of the sag equation, many new methods have been proposed for modelling the DO concentration in a stream. Churchill and Buckingham (28) developed a statistical method for determining BOD and dissolved oxygen values in a stream. This method assumes that the DO level in a stream depends only upon BOD, temperature, and flow. By solving regression equations correlating data from other streams with similar basin characteristics, a dissolved oxygen profile can be obtained for a given organic loading. Second-order rate equations have been reported to model BOD consumption data more closely in some instances by Young and Clark (29), but this has been shown only for singular cases.

Other models based on the Streeter-Phelps equation have been introduced which consider variable conditions in a stream. A stoichastic probability model has been developed by Thayer and Krutchkoff (30) in an attempt to bracket possible stream conditions, while an unsteady state model has been developed by Di Toro and O'Connor (31) for showing time variance of conditions at a given point subject to unsteady flow. A more complete survey and assessment of mathematical models has been made by Harper (32) at Washington State University.

The mathematical models mentioned require evaluations of the coefficient of reaeration, K_2, and the coefficient of deoxygenation, K_1. The modern methods for evaluating K_2 for stream conditions are empirical formulations expressing K_2 as a function of stream depth, temperature, and velocity, of the general form

$$K_2 = C\frac{V^a}{H^n} \tag{10}$$

where V is the velocity, H is the depth, a, n, and C are constants, and C depends upon the temperature. These equations have been developed by Streeter (2), O'Connor and Dobbins (33), Churchill (34), and Isaacs and Gaudy (35). Other methods and equations for determining K_2 are available.

The constant K_1 is evaluated either by the dilution BOD bottle technique and solving the equation

$$y_T = L\left(1 - e^{-K_1 t}\right) \tag{11}$$

where y_T is the oxygen demand remaining at time t, and L is the ultimate BOD, or is evaluated by determining the ultimate BODs at two points on the stream and determining the time of travel between these points, then solving the integrated equation for first-order decreasing rate deoxygenation

$$L_b = L_a\, e^{-K_1 t} \tag{12}$$

where L_b is the downstream ultimate BOD, and L_a is the initial ultimate BOD and t is the travel time between the points.

Another usable method for predicting the DO sag is that developed by Velz (12), which employs a rational method of accounting for the expense and replenishmant of dissolved oxygen. Forms are set up representing deoxygenation and reaeration in a reach of a stream. By performing an oxygen balance for each successive reach, a dissolved oxygen profile can be obtained. This method is most flexible in taking full advantage of detailed waste loading information and other measurements such as stream depths, widths, and volume, but uses first-order decreasing rate relationships for reaeration and deoxygenation.

A different approach has been taken by Busch (3) in his method for stream assimilation capacity. His idea is that a stream's maximum assimilative capacity does not depend upon variable conditions, but depends only upon the minimum reaeration capacity of the stream. Thus, there is no reason to even consider the interrelationship of deoxygenation and reaeration in a water quality management program. His solution uses the general expression for gas transfer to a liquid

$$\frac{dM}{dt} = K_{L\ min}\, D_{max}\, A_i \tag{13}$$

where M is the mass of oxygen transferred, K_L is the mass transfer coefficient, A_i is the interfacial surface area, and D_{max} is the maximum allowable dissolved oxygen deficit.

The solution to Busch's equation yields the maximum uniform loading rate of an oxygen demand that can be applied and not cause a drop in the dissolved oxygen below that which is allowed, subject to the worst stream conditions of reaeration.

Because of the probable fallacy of using the dilute, quiescent conditions of a BOD bottle in evaluating K_1, which may not truly describe oxygen uptake kinetics in the stream, Peil and Gaudy (36) have proposed a more truly rational approach in predicting a stream's dissolved oxygen profile. An open jug reactor is used to determine the oxygen uptake curve for various waste to stream concentrations, and is stirred at reaeration rates known to not differ significantly from those of the stream. Then for given stream reaches, when the coefficient of reaeration is determined from stream cross-sections, and critical flows and waste loadings are known, the dissolved oxygen profile may be determined by numerically integrating reaeration and deoxygenation by replacing the deoxygenation term in the Streeter-Phelps equation with the determined oxygen uptake profile. Results obtained from the open jug technique as compared to a simulated stream in which good controls could be exercised in regard to temperature, velocity distribution, and subsequently K_2, proved the workability of this method.

This section has presented some of the main efforts made in estimating the effects of discharging organic pollutants into a watercourse. The amount of material which has amassed since Streeter and Phelps' work in 1926 is a testimony to the difficulties inherent in modeling the complex natural phenomena of stream self-purification. Many of the models are too complex to be usable, or have no proven advantage in fitting actual stream data. For this reason, a simple concept for planning such as that presented by Busch would seem to have much merit if it is a usable and practical concept. Comparing this model to a mathematical model based on the kinetics of oxygen uptake and oxygen transfer in a stream should be the first step in understanding how this model differs from the others that have been presented in the past fifty years.

D. Development in the Illinois River Basin

The main waters of the Illinois River are blocked by only two dams in their travels through Oklahoma and Arkansas. Tenkiller Ferry Lake is located about thirteen miles from the confluence of the Illinois River with the Arkansas River, and was completed by the United States Army Corps of Engineers in July, 1952. The lake has a storage capacity of 1,230,000 acre-feet, and was developed for water supply, flood control, and hydropower. Lake Frances is a small lake located on the Oklahoma-Arkansas border and serves as the water supply for Siloam Springs. It has a total storage capacity of 1,930 acre-feet. Several small lakes have been developed on tributaries of the Illinois River in Arkansas (37). In January, 1969, in a report to the Arkansas-Oklahoma Arkansas River Compact Committee, an assessment of the present water use was made in Oklahoma, based on the issued valid water rights (37). The determined uses are shown in Table III.

Tahlequah, population 9,254 (1970 Census), and Siloam Springs, Arkansas, population 6,009 (1970 Census) are the only two "major" cities located in the Illinois River basin in the Oklahoma area. Tahlequah uses the Illinois River for a source of water supply, and discharges secondary municipal waste back to the waters. Siloam Springs draws its water from Lake Frances, while disposing of its secondary sewage into

Sager Creek, a tributary of Flint Creek, 0.5 mile from the Oklahoma border. There are five other cities in Arkansas and one industry, Allen Cannery, presently discharging into tributaries of the Illinois River.

TABLE III

WATER USAGE IN THE ILLINOIS RIVER BASIN IN OKLAHOMA

	Municipal and Industrial	Irrigation
Illinois River	5,000 acre-feet	10,000 acre-feet
Flint Creek	-	6,500 acre-feet
Barren Fork	-	8,700 acre-feet
Tenkiller Ferry Lake	33,000 acre-feet	700 acre-feet

Farming is a principal land usage in the Oklahoma part of the basin consisting of 57 percent of the land area, but only eleven percent of this is broken for cropland. Much of the area is in pastures and woodlands. Permits to further develop water rights have been issued on 8,300 acres of land as of January, 1969, with 26,700 acres suitable for irrigation if the water were available (37).

A sizable poultry industry has been developed in northwestern Arkansas and northeastern Oklahoma along the course of the Illinois River. The pollution potential associated with runoff from these poultry farms was reported in a study done in Arkansas by Hileman (38).

This study was a characterization of the waste from a typical poultry farm, with the runoff potential of the waste qualitatively assessed from observations made on water quality in the area.

Another major industry that has developed in the Illinois River basin is that of recreation. Canoeing the gently to swiftly flowing, clear waters of the Illinois River has become popular from near the Arkansas border to the City of Tahlequah to the south. Many canoe rental shops and stopovers are now along the watercourse, and the number of canoers has increased greatly over the past several years. These reaches above Tenkiller are also fishing grounds, being designated small-mouth bass fisheries containing large and small-mouth bass, catfish, sunfish, and white bass, among others. Canoeing, fishing, and camping are pursuits enjoyed by many along this river.

In 1955, the City of Tulsa made application to the Oklahoma Water Resources Board for water rights to dam a portion of the Illinois River near Tahlequah in order to supplement the city's water supply (8). These rights were not granted, due to two circumstances:

1) no hydrographic or hydrologic data was available from which to assess optimumly the storage capacity available, and

2) the Illinois River is an interstate river system and no compact was in existence between the States of Oklahoma and Arkansas for determining how much water was to be allocated to each state (7). Thus, the hydrographic and hydrologic surveys were made available by the United States Geological Survey in 1959 (8)(9), and a report to the Oklahoma and Arkansas Compact Committee was submitted in 1969 (37).

In the report to the Compact Committee, the storage capabilities of five potential reservoir sites were given, of which three were on the main stem of the Illinois--a Siloam Springs dam site, Chewey dam site, and a Tahlequah dam site. The United States Army Corps of Engineers recommended in 1969 that the City of Tulsa coordinate its plans with those of the Corps to develop the Chewey dam site, approximately four miles south of the junction of Flint Creek with the Illinois River (39). No further plans have been made since 1970 when the passage of the Oklahoma State Scenic Rivers Act gave the Governor of the State of Oklahoma the right to veto any further projects on the Illinois River.

Concern for the quality of water in the Illinois River has had its emergence from proposed development both in Oklahoma and Arkansas. In Oklahoma, Frates Properties, Inc., and Context Development Co., Miami, Florida, have a joint venture to sell 3,000 tracts of land along a 5-mile, 7000-acre area at the junction of Flint Creek and the Illinois River in Adair and Delaware Counties (40). Frates also has an option to buy 14,000 acres on the opposite bank of the river. Presently proposed development had approval of the State Water Resources Board to use river water for consumptive uses, which was planned to be returned to the river by way of septic tanks as the sewage disposal system. Controversy over the effectiveness of septic tanks in treating domestic wastes from considerations of the geology of the area, and the potentially deleterious impact of clearing land along the riverside, spurred the Illinois River Conservation Council, Inc., and The Scenic Rivers Association to pursue a lawsuit naming the Department of Housing and Urban Development (HUD) as the defendant. Flint Ridge developers were later included in the suit by the U. S. District Judge Luther Bohannon. Judge Bohannon's decision on August 2, 1974, delineated that

1) all interstate land sales must be suspended on the Flint Ridge

development, and

2) HUD must conduct an environmental impact of the development's effects on the river (41). The decision is currently being appealed.

The controversy over development in Arkansas revolves around two regional waste treatment plants proposed above Lake Frances, and a power plant proposed along Little Flint Creek. The plan for regional waste treatment calls for Fayetteville, Springdale, Rogers, Bentonville, and several other small towns to be served by a single waste treatment plant which would discharge into the main waters of the Illinois, approximately twenty miles above another regional plant serving Siloam Springs, Gentry, Decatur, Gravette, and Sulphur Springs, which would discharge into the Illinois River just above Lake Frances. The plan also calls for Siloam Springs to obtain its drinking water from Beaver Lake, Arkansas, instead of from Lake Frances. The plan recommends that the Illinois River in Arkansas be re-classified so that the dissolved oxygen standard would be reduced from 5 mg/l to 4 mg/l. An alternative plan calls for several multiple city and single city plants of which none would discharge into the main stem of the Illinois River--the case which is presently observed (42). The Environmental Protection Agency in unofficial statements has shown little inclination toward approving the concepts in Arkansas' Northwest Regional Plan (43).

Southwest Electric Power Company has proposed a power plant to be placed two miles southwest of Gentry along Little Flint Creek. Controversy has arisen over the strong possibility that Little Flint Creek would become dry in the summer months by the creation of a 530-acre lake at the power generation site. Possible thermal pollution of this lake and its effect on surrounding wells, as well as the emission of sulfur dioxide into the air are other questions that need to be answered (44).

How much should the river be used? The waters have been called vital to Tulsa's water supply, private owners of lands adjacent to the river intend to build and live there, recreationists keep increasing in numbers, and the unknown assimilative capacity of the river is intended to be put to use. The river is being planned for many uses, and in order to facilitate decision-making, the existing conditions and resilience of the river need to be understood.

E. Water Quality Regulations

The regulations which govern the quality of the Illinois River and the present proposed discharges into the waters can be grouped into state and federal regulations.

The state standards are in-stream standards and may be found in "Water Quality Standards for the State of Oklahoma," published by the Oklahoma Water Resources Board (45). Some of the specific criteria outlined in the standards are

1) historical records shall serve as a guide for mineral quality, and waste discharges should not lower the quality described by these records,

2) bacterial concentrations of other than natural origin will be maintained below levels detrimental to beneficial use, and numbers are given for specific cases in the standards,

3) solids and turbidity should not be present due to sources of other than natural origin,

4) differential temperature changes from other than natural sources

should not exceed 5^{o}F, nor cause the temperature to exceed 70^{o}F in trout streams, 75^{o}F in small-mouth bass streams, or 93^{o} in warm-water streams,

5) the dissolved oxygen concentration should not be less than 4 mg/l except in the vicinity of a point discharge when the stream flow is less than 200 percent of the waste flow, and

6) pH values below 6.5 or above 8.5 must not be due to waste discharges.

The criteria also include that no waste discharges may be made below the Tenkiller Dam.

Another state regulation previously mentioned is the inclusion of the Illinois River in the Oklahoma State Scenic Rivers Act. The essential element of this act is that the Governor of the State of Oklahoma has the right to veto any further reservoir projects along the Illinois River and tributaries.

Federal regulation was implemented in 1972 with the passage of the Federal Water Pollution Control Act Amendments of 1972, Public Law 92-500 (1). This law delineates that water quality standards previously in effect for interstate waters will remain in effect, subject to EPA approval, and intra-state standards must be set by the states or EPA. States are also required to submit reports to EPA on the quality of waters within their borders. The first such report was due on January 1, 1975.

Water quality planning for each river basin in each state is to be accomplished within a specific period of time. This planning has been designed to mesh with the national goal of all waters being suitable for fishing and swimming by mid-1983. This is to be accomplished by applying "best practicable" treatment to municipal waste discharges by 1983, and "best available" technology by 1983 for industrial waste discharges. These regulations may be superimposed by more stringent controls in order to meet water quality standards. The Federal legislation also requires environmental impact statements in connection with construction grants for publicly owned waste treatment facilities and when issuing permits for discharges of pollutants from new sources.

Monitoring of waters is included under Titles I, II, III, and IV of the Act, with intent to determine compliance with required permits for all point source discharges into national waters (1). Proposed regulations concerning this monitoring may be found in the Federal Register, Volume 39, No. 168 (46).

The point source control must be integrated into specific basin plans in each state, as described in Section 303(e) of the Federal Amendments (1). Each basin plan will provide for orderly water quality management by four steps:

1) outlining the plan,

2) determining priorities in implementing the plan,

3) scheduling action, and

4) coordinating planning with the agencies and organizations involved.

This is a 5-year plan which is to be continuously updated to account for changes in the basin situation. The plan should provide

1) program design,

2) initial analysis of available data, and

3) priority listing of water pollution problems needed to be

resolved in developing the basin plan.

Another type of federal control that would be specific for the Illinois River in Oklahoma would be inclusion of the river in the National Wild and Scenic Rivers Act which would serve to protect the river from shoreline over-development and pollution (41). Senators Bellmon and Bartlett of Oklahoma are working for the inclusion of the Illinois River in this Act. The Bureau of Outdoor Recreation is authorized to determine whether the river meets the standards required to qualify as a national scenic river. The question is whether the Illinois River (also Flint Creek and Barren Fork) has already been developed to an extent that might exlude it. A time factor is also involved in that the workings of the federal bureaucracy seem to be slower than expanding recreational and land development.

Regulations are available at both the state and federal levels that would allow water quality planning. Applicability of these controls is the question that remains to be answered.

CHAPTER III

METHODS OF STUDY

The methods of study for three separate analyses are presented in this chapter. The analyses are

1) evaluation of design low flows by fitting a statistical distribution to historical records of low flow conditions,

2) studies of cross-sections available at the gaging stations for discerning streamflow characteristics, and

3) a study of Busch's method for stream assimilation capacity as compared to the Streeter-Phelps equation.

A. Low Flow Analysis

The historical records for discharge measurements at gaging stations on the Illinois River near Watts (07195500) and near Tahlequah (07196500), Flint Creek near Kansas (07196000), and the Barren Fork at Eldon (07197000) were the stations analyzed for low flows. For each water year, rather than the period from March to April recommended for drought flows, the lowest 1-day, 7-day, and 30-day average flow was recorded. These values were then ranked in decreasing order of magnitude as recommended by Gumbel (13) for his logextremal distribution. The plotting positions for these ranked flows were obtained from the formula: plotting position = m/(n+1), as defined in Chapter II. The values of flow and the logarithms of the values were then plotted versus their plotting positions on normal probability paper and extremal probability for the 7-day average low flows. This was done to find the best graphical definition of the drought flow distributions at the gaging stations. Parameters to further statistically define each distribution, such as the mean ($\bar{x}$), standard deviation (s), coefficient of variability (C_v), and coefficient of skewness (C_s) were then calculated from the data. The distributions defined by these parameters were compared with those defined by the plots on the probability paper. This type of procedure was also carried out for the 1- and 30-day low flows, with the step of plotting on different types of probability paper eliminated since these distributions paralleled the 7-day average low flow distributions. The value of drought flow for a 10-year recurrence was then obtained from the 90 percent probability level on the graphical distributions of low flows.

B. Correlating Low Flows With Cross-sections

Cross-sections were obtained from the Oklahoma City office of the United States Geological Survey for the gaging stations at Watts, Kansas, Tahlequah, and Eldon. Also, ratings of discharge versus gage height in tabular form were obtained. These data are included in Appendix A.

The cross-sections and rating tables were used to find the hydraulic depth (H), top width of flow (W), area of flow (A), and the velocity (V) at the 1-, 7-, and 30-day low flows for a recurrence interval of ten years for each cross section. This was done by first finding the gage height corresponding to the value of flow (Q) in question. The top width was found on the cross-section at the gage height representing this flow. The area was calculated as the region between the gage height at the 10-year recurrence flow, and the gage height at zero flow. The hydraulic depth was then found from the equation, H = A/W. The velocity was calculated from the continuity equation, V = Q/A.

United States Geological Survey quadrangle maps of Watts, Kansas, Welling, and Stillwell, Oklahoma, were used to calculate approximate slopes of the river at the gaging stations. This was done by measuring the distance along the river between contour intervals with a map-measure, then dividing the 20-ft contour interval on the maps by the distance to obtain the slope. The scale of the maps was 1:24,000. These values for the slope were then used along with the velocity, area, and wetted perimeter in Manning's equation

$$V = \frac{1.49}{n} \left(\frac{A}{WP}\right)^{2/3} S^{\frac{1}{2}} \tag{14}$$

where n is Manning's coefficient of roughness, A is the cross-sectional area of flow, WP is the wetted perimeter in feet, and S is the slope of the channel. This was done to compare the coefficient of roughness computed for each of the different gaging stations.

C. Assumptions for Comparison of Busch's Method to the Streeter-Phelps Equation

In the comparison of Busch's method for stream assimilation capacity with the Streeter-Phelps Sag Equation, the 12.8-mile reach of Flint Creek from the state line to its junction with the Illinois River was used for the computations. The cross-section of Flint Creek at the gaging station near Kansas, and the 1-, 7-, and 30-day, 10-year recurrence interval low flows were assumed to be valid throughout the length of the reach. The data used was thus the cross-sectional results for H, V, W, and A determined at these flows. Two more flows were picked at random and evaluated for these parameters to aid in illustrative purposes.

As defined in Chapter II, the Streeter-Phelps equation is

$$D = \frac{K_1 La}{K_2 - K_1}\left(e^{-K_1 t} - e^{-K_2 t}\right) + D_a e^{-K_2 t} \tag{9}$$

and Busch's equation for stream assimilation capacity is

$$\frac{dM}{dt} = K_{L(min)}\, D_{max}\, A_i \tag{13}$$

where $K_L = K_2 x H$.

The coefficient of reaeration (K_2) was calculated by a formula developed by Isaacs, Chulavachana, and Bogart (47) for a simulated stream apparatus at New Mexico State University

$$k_2 = 2.833 \frac{V}{H^{1.5}} \quad \text{(base 10 logarithms)} \tag{15}$$

This was done for all values of flow in order to determine the minimum K_L which was then used in all calculations using Busch's formulation, and the K_2 corresponding to this $K_L \left(K_2 = \frac{K_L}{H}\right)$ for a given flow was used in all calculations involving the Streeter-Phelps equation. The calculated values for k_2 from the formula developed by Isaacs, et al. were converted to a natural logarithm base by multiplying by 2.303, and then corrected for temperature by use of the formula

$$K_{2(T)} = 1.0241\ K_2^{(T-20^{\circ}C)} \tag{16}$$

Analysis of the water quality data available indicated that the highest weekly average temperature condition that might be expected was approximately 29°C, so this value was used in the correction formula. The value of 29°C was also used in finding the saturation value of dissolved oxygen for the reach. The effect of chlorides and suspended solids on this saturation concentration was considered to be negligible as discerned from available water quality data, so this saturation value was determined directly from solubility tables of oxygen in distilled water. This value was assumed to be 7.77 mg/l for all calculations for both equations. The maximum allowable deficit was thus 3.77 mg/l (7.77 - 4.00) for Busch's equation.

The initial dissolved oxygen concentration for the Streeter-Phelps equation was assumed to be 5.0 mg/l in all calculations since this is the minimum value allowed for Flint Creek in the State of Arkansas. This value made D_a in the Streeter-Phelps equation 2.77 mg/l (7.77 - 5.00).

Since no data is available on the rate of deoxygenation as might be predicted for possible waste loadings in the future, a constant of deoxygenation was assumed to be 0.23 days^{-1} for all calculations involving the Streeter-Phelps equation. Using other values for K_1 in the calculations would change the values in the results, but not the ideas presented.

The time corresponding to any distance downstream in the Streeter-Phelps equation was found by the relation, distance = V x t, where velocity is as found from the cross-section for a given flow. The computation of the Streeter-Phelps equation was facilitated by a computer program previously developed by this author, and is included in Appendix B.

CHAPTER IV

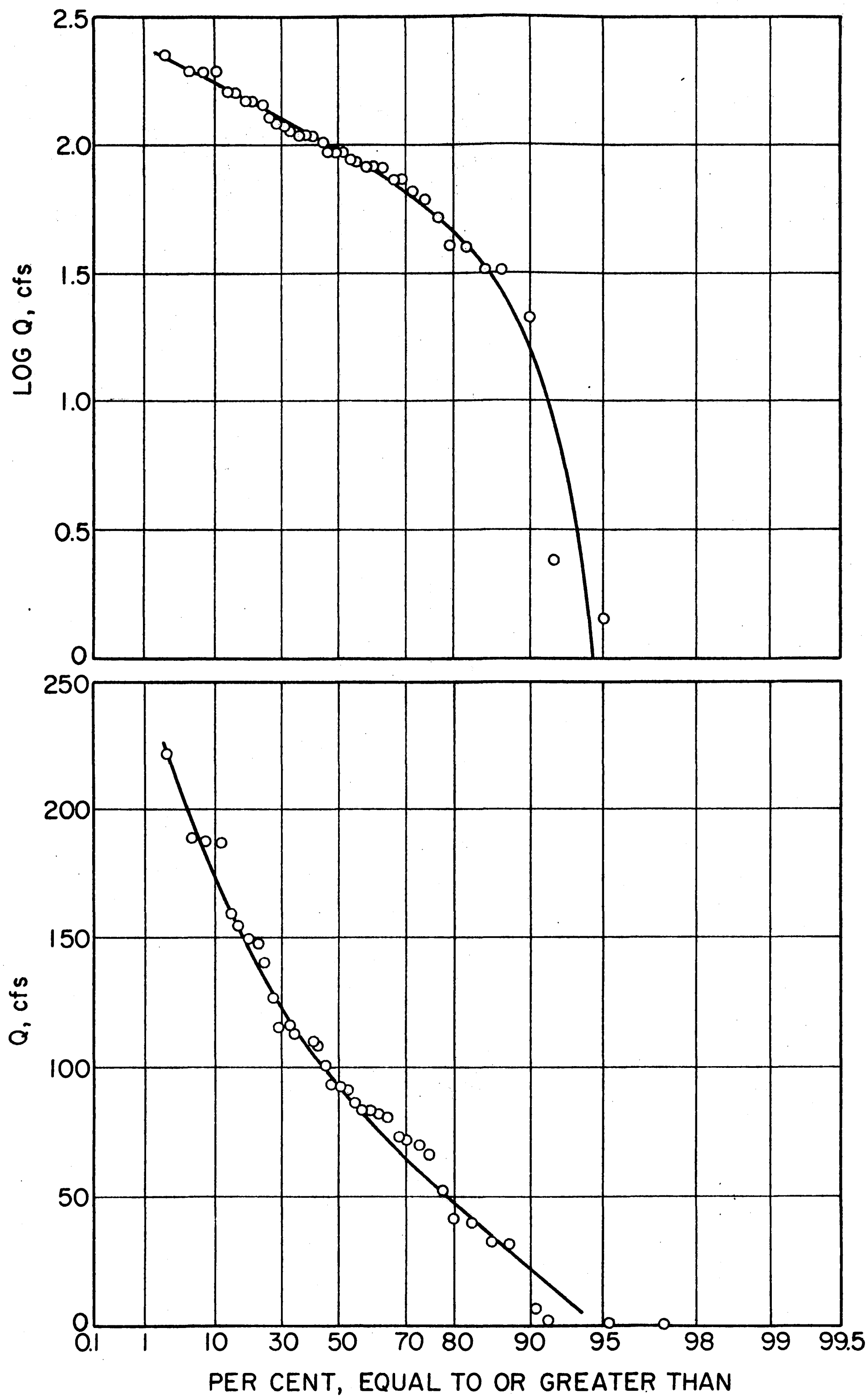
RESULTS AND DISCUSSION

The results and discussion will be divided into three units. The first unit presents the results from the statistical analysis of the low flow data. The second presents the correlation of the 10-year recurrence interval low flows to the cross-sections. Finally, the comparison of Busch's method for stream assimilation capacity to the Streeter-Phelps equation will be presented.

A. Low Flow Analysis

Since Tahlequah has the longest period of record (38 years), the plots made on this gaging station for the logextremal, extremal, lognormal, and normal probability graphs have been shown in Figures 2 and 3 to exemplify the types of distributions encountered in the Illinois River basin. These graphs are for the 7-day average low flows. In Figure 2, the logextremal plot (Gumbel's extremal value theory for low flows) shows a concave downward curve that breaks into a very steeply sloping portion between the 80 and 90 percent frequency levels. If assymptotes were to be drawn to this curve, the lines would be similar to the graphs made in the low flow analysis for the 1959 study on the hydrology of the Illinois River basin by the United States Geological Survey (9). This curve is also similar to the plots made by Hardison and Martin (17) for the Mountain Fork River near Eagletown, Oklahoma.

Figure 2. Logextremal and Arithmetic-extremal Plots of 7-day Average Low Flows at Tahlequah (07196500), n = 38 years

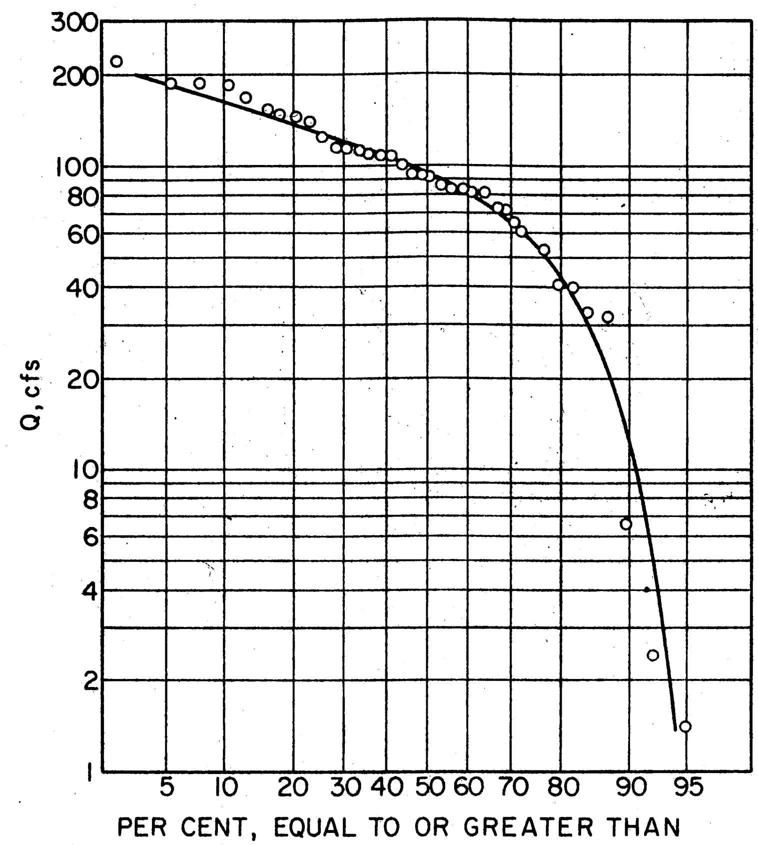

2.5
2.0
1.5
1.0
0.5
0
LOG Q, cfs
250
200
150
100
50
0
Q, cfs
0.1
1
10
30
50
70
80
90
95
98
99
99.5
PER CENT, EQUAL TO OR GREATER THAN

To fit this data with accuracy using a logextremal plot would not be possible, since the points are scattered in a steeply sloping manner for frequencies greater than 80 percent. Also shown in Figure 2 is the plot of the flows versus frequency on extremal probability paper. This plot yields a curve which is slightly concave downward for the Tahlequah gaging station.

The log-normal plot in Figure 3 yields a distribution resembling the logextremal plot in Figure 2. The curve is concave upward, and the slope becomes very steep for probabilities greater than 80 percent. The plot of the data on normal probability paper in Figure 3 shows a distribution that can be fit readily with a straight line. The same conclusion was reached for the 1-, 7-, and 30-day low flows at the gaging stations near Watts, Kansas, and Tahlequah. This fit did not hold true for the Barren Fork near Eldon. The straight line fits on normal probability paper, and the fact that the Barren Fork distribution was different will be further discussed after presenting the results from analyzing the low flows as normal distributions at the gaging stations.

The graphs on normal probability paper for the 1-, 7-, and 30-day low flows at the four gaging stations studied are presented in Figures 4, 5, 6, and 7. Since the data fit a straight line for the stations at Watts, Kansas, and Tahlequah on normal probability paper, the mean and standard deviation were calculated, then compared to the mean and standard deviation that were observed from the best graphical fit of a straight line. The coefficient of variability was then calculated for both of these cases, and the coefficient of skewness was computed for the data sets of the 7-day average flows since the 7-day average is the most often used in decreeing a design low flow. These statistical

Figure 3. Log-normal and Normal Plots of 7-day Average Low Flows at Tahlequah (07196500), n = 38 years


300
200
100
80
60
40
20
10
8
6
4
2
1
Q, cfs
5 10 20 30 40 50 60 70 80 90 95
PER CENT, EQUAL TO OR GREATER THAN

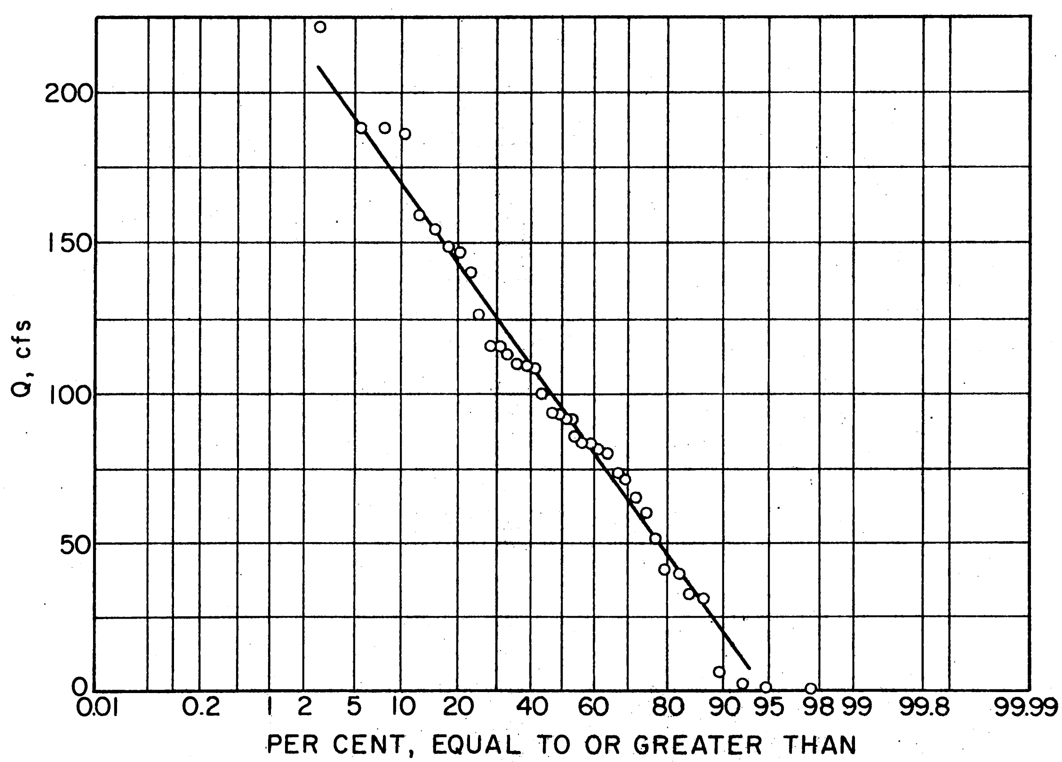

200
150
100
50
0
Q, cfs
0.01 0.2 1 2 5 10 20 40 60 80 90 95 98 99 99.8 99.99
PER CENT, EQUAL TO OR GREATER THAN

Figure 4. The 1-, 7-, and 30-day Average Low Flow Distributions for Watts (07195500), n = 18 years

1-day Average Low Flows

	Computed	Graphically Determined
$\bar{X}$	62.5	60.8
S	36.0	38.0
C_v	.58	.62

7-day Average Low Flows

	Computed	Graphically Determined
$\bar{X}$	73.4	70.2
S	39.4	42.5
C_v	.54	.60
C_s	.35	-

30-day Average Low Flows

	Computed	Graphically Determined
$\bar{X}$	109.2	115.0
S	62.2	75.0
C_v	.57	.65

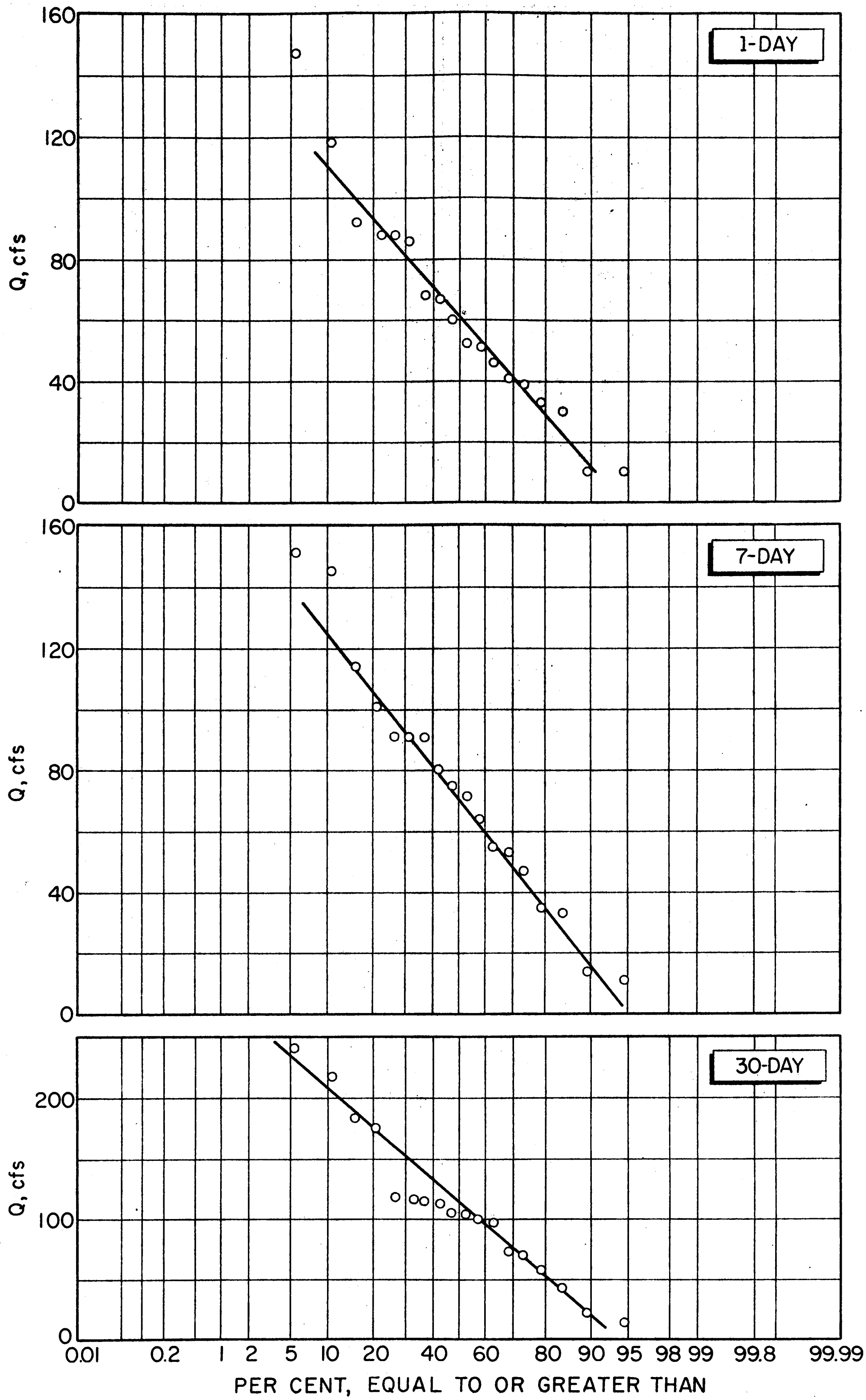

1-DAY
7-DAY
30-DAY
Q, cfs
160
120
80
40
0
200
100
0.01 0.2 1 2 5 10 20 40 60 80 90 95 98 99 99.8 99.99
PER CENT, EQUAL TO OR GREATER THAN

Figure 5. The 1-, 7-, and 30-day Average Low Flow Distributions for Kansas (07196000), n = 18 years

1-day Average Low Flows

	Computed	Graphically Determined
$\bar{X}$	12.6	12.5
S	7.1	8.4
C_v	.56	.67

7-day Average Low Flows

	Computed	Graphically Determined
$\bar{X}$	13.6	13.6
S	7.3	8.1
C_v	.54	.60
C_s	-.21	-

30-day Average Low Flows

	Computed	Graphically Determined
$\bar{X}$	20.8	19.5
S	12.8	10.6
C_v	.62	.54

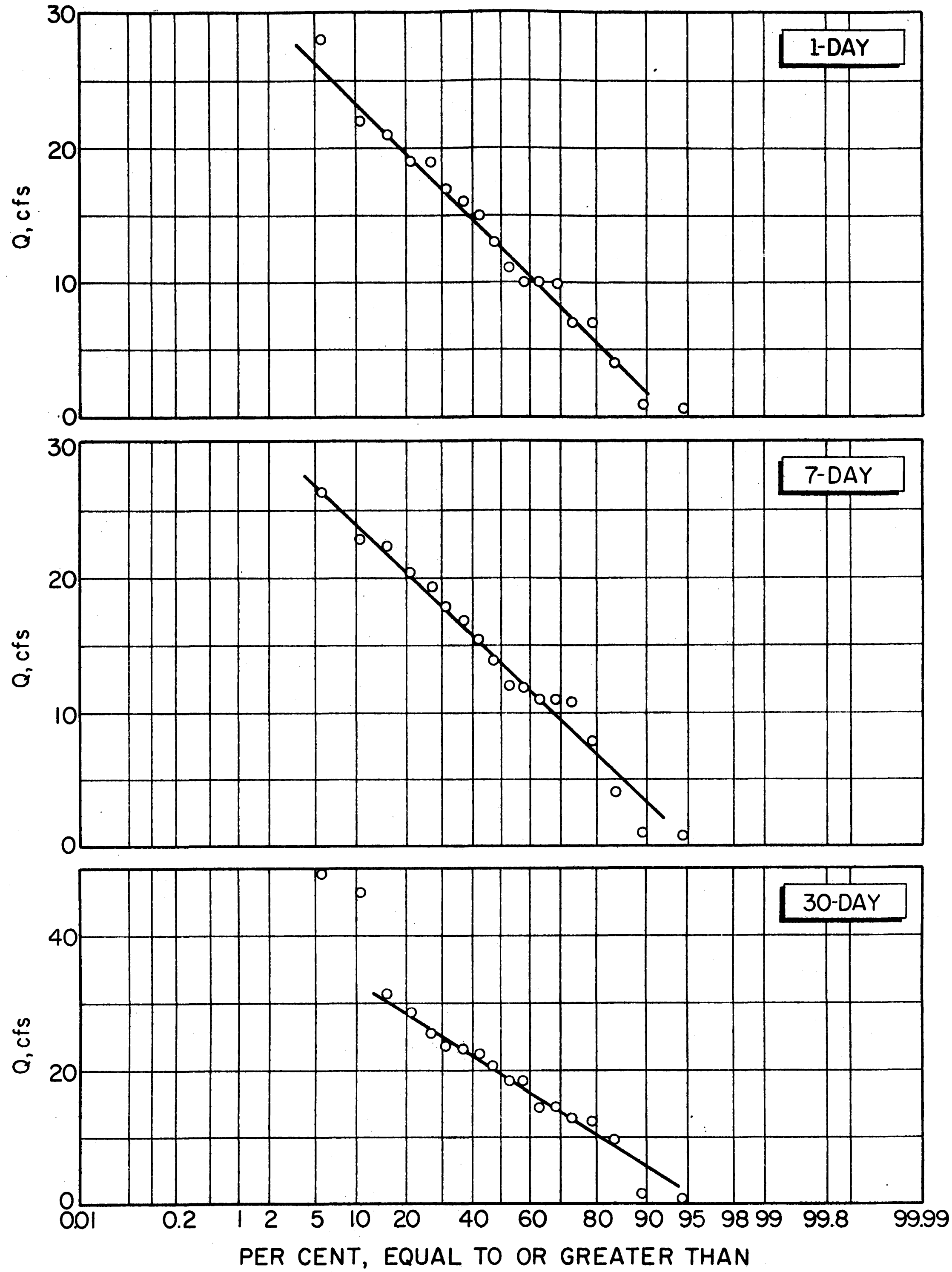

1-DAY
7-DAY
30-DAY
Q, cfs
30
20
10
0
40
0.01
0.2
1
2
5
10
20
40
60
80
90
95
98
99
99.8
99.99
PER CENT, EQUAL TO OR GREATER THAN

Figure 6. The 1-, 7-, and 30-day Average Low Flow Distributions for Tahlequah (07196500), n = 38 years

	Computed	Graphically Determined
	1-day Average Low Flows	
$\bar{X}$	87.9	88.0
S	52.9	58.0
C_v	.60	.66
	7-day Average Low Flows	
$\bar{X}$	95.0	94.0
S	55.3	57.0
C_v	.58	.60
C_s	.19	-
	30-day Average Low Flows	
$\bar{X}$	133.8	133.0
S	82.3	82.0
C_v	.62	.62

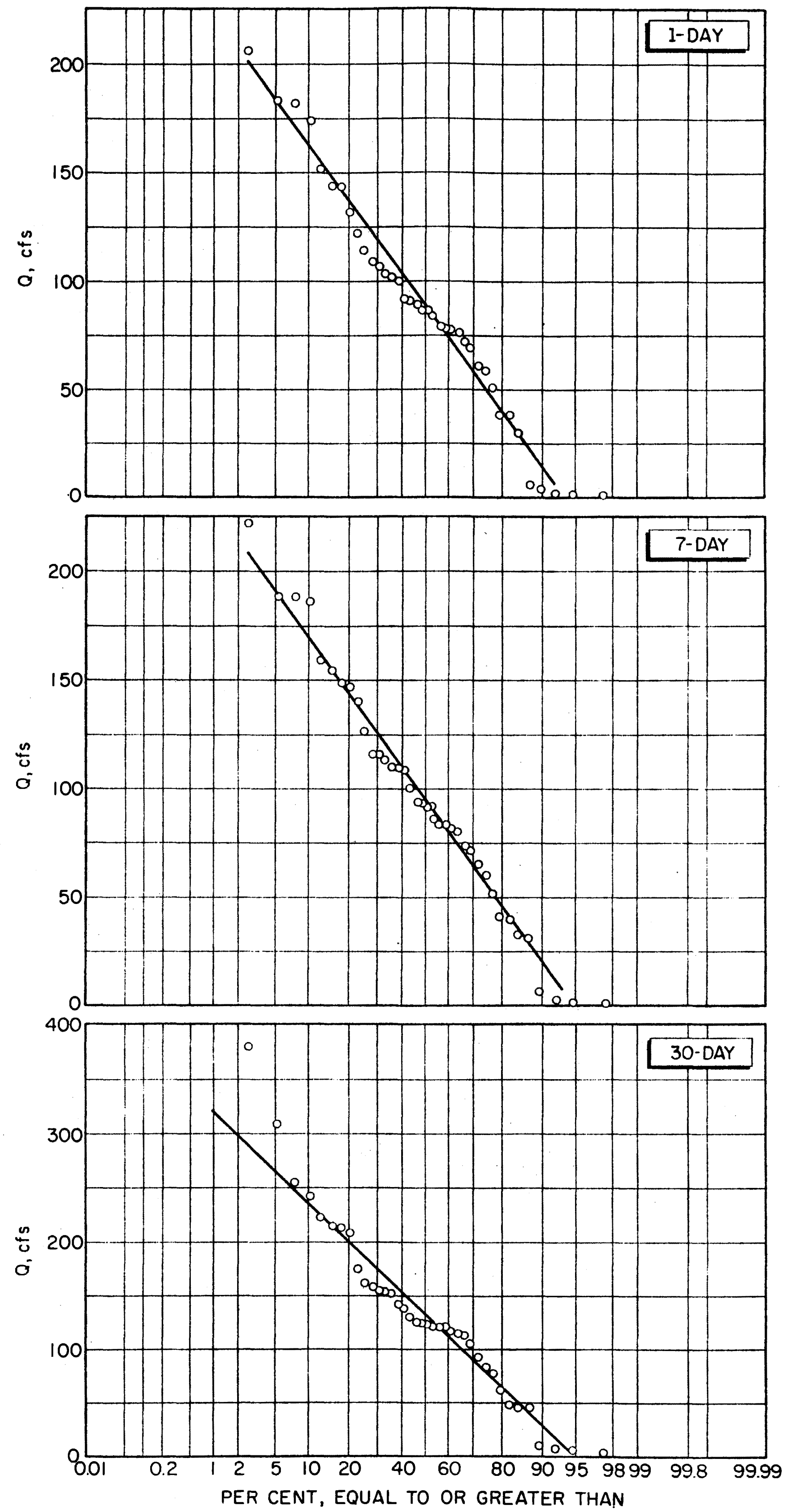
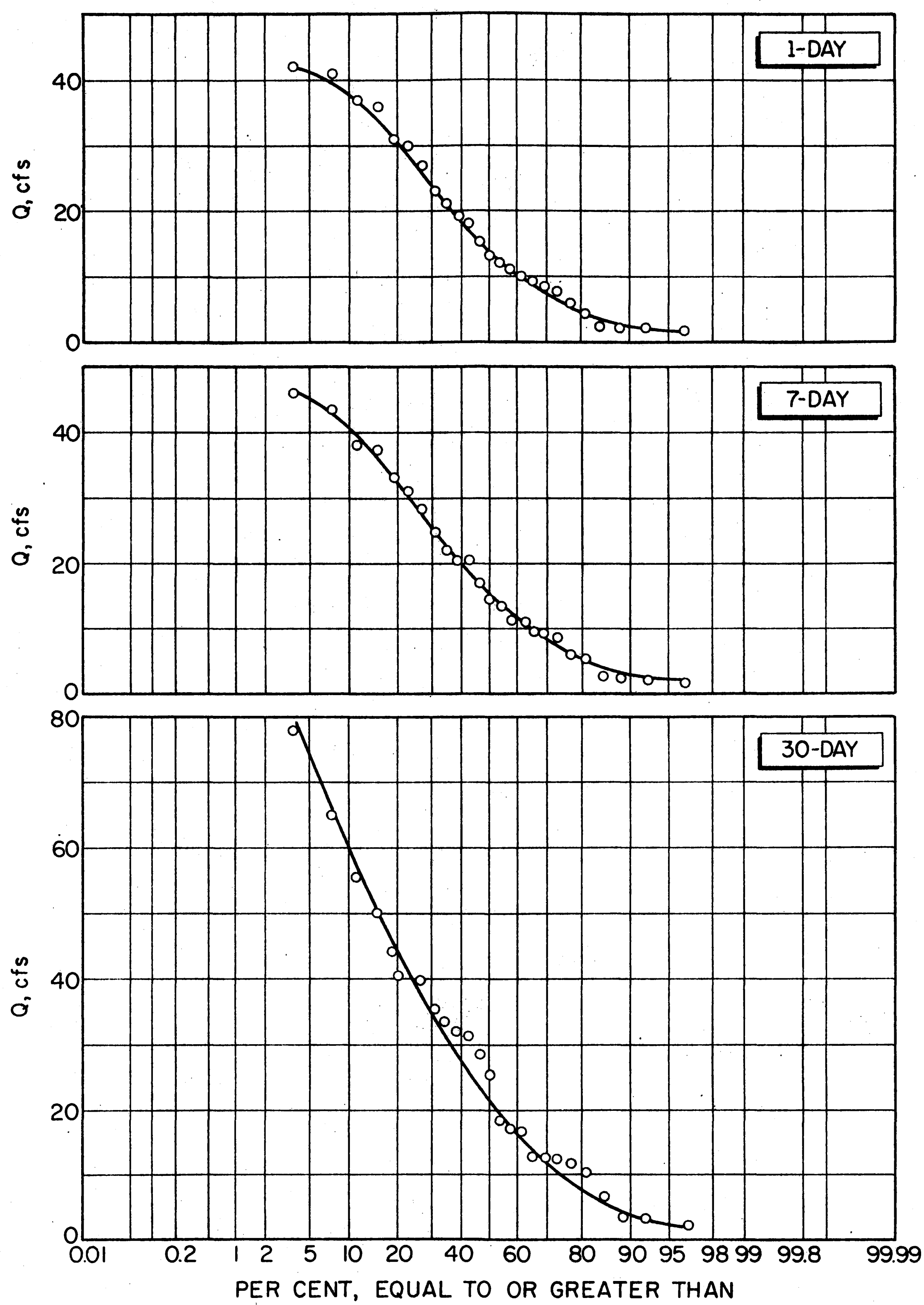

1-DAY
7-DAY
30-DAY
Q, cfs
0
50
100
150
200
300
400
0.01
0.2
1
2
5
10
20
40
60
80
90
95
98
99
99.8
99.99
PER CENT, EQUAL TO OR GREATER THAN

Figure 7. The 1-, 7-, and 30-day Average Low Flow Distributions for Eldon (07197000), n = 25 years

Computed from Data

	1-day Average Low Flows	7-day Average Low Flows	30-day Average Low Flows
$\bar{X}$	17.2	18.4	27.4
S	12.9	13.6	20.1
C_v	.75	.74	.73
C_s	-	5.22	


1-DAY
7-DAY
30-DAY
Q, cfs
Q, cfs
Q, cfs
0
20
40
0
20
40
0
20
40
60
80
0.01
0.2
1
2
5
10
20
40
60
80
90
95
98
99
99.8
99.99
PER CENT, EQUAL TO OR GREATER THAN

parameters are given with the respective graphs.

For the gaging stations at Watts (Figure 4), the theoretical best fit from the mean and standard deviation did not seem to be as good as a line drawn to fit the points graphically. The graphical mean was determined to be lower than the computed mean for the 1- and 7-day low flows, while a good straight line fit of the data could not be observed with the 30-day average low flows. A best straight line fit on the graph yielded a mean which was higher by six cfs than the computed mean of the data. The standard deviations determined from these best straight line fits were higher for the three flow distributions than a theoretical fit of the data would have determined. The coefficient of variability was thus higher from the graphical determination than that computed from the data. A high coefficient of variability of 0.60 for the 7-day average low flow was determined from the graphs, with coefficients of variability of 0.62 and 0.65 for the 1- and 30-day average low flows, respectively. The coefficient of skewness was found to be 0.347 for the 7-day average low flow data. This skewness toward the high flow side is probably attributable to the two high flows at the lowest probabilities that do not fit on the line of the drought flow trend. As discussed by Velz (12), these may not truly be drought flows.

The means calculated from the data for the gaging station at Kansas (Figure 5) closely define the observed fits on the graphs. The standard deviations, however, are observed to be greater by the lines formed from plotting the data than those calculated from the data for the 1- and 7-day average low flows--8.4 versus 7.1, and 8.1 versus 7.3, respectively. Two flows at the lowest frequencies for the 30-day average low flows are much greater than would seem in line with the rest of the data, and thus the standard deviation from the graph is much less than that computed from the data, 10.6 versus 12.8. Approximately the same variability of the flows at this gaging station is observed as was noted for the Watts gaging station, C_v being 0.67, 0.60, and 0.54 for the 1-, 7-, and 30-day average low flows, respectively. The coefficient of skewness was calculated to be -0.206, or the distribution was skewed to the low flow side for the 7-day average low flows.

Thirty-eight years of data have been collected at the Tahlequah gaging station (Figure 6), and the means and standard deviations calculated for the 1-, 7-, and 30-day average low flows are very close to those graphically observed to be more representative of the trend of the major population of the data. The standard deviations again seemed to be slightly greater from a plot of the data than those calculated, so the greater slopes were used since this yields a slightly more conservative estimate of low flows for frequencies greater than 50 percent. The coefficients of variability were much like those for Watts and Kansas, being 0.66, 0.60, and 0.62 for the 1-, 7-, and 30-day average low flows. The data for the 7-day average low flows was skewed slightly right, C_s calculated to be 0.193. Like the stations at Watts and Kansas, the 1- and 7-day average low flows better defined a straight line than the 30-day average low flows. For all three gaging stations, the data points were most tightly knit about a line for the 7-day average low flows.

The Barren Fork at Eldon (Figure 7) exhibited a different type of plot in that the flows were noticeably skewed from the graphical analyses. The coefficient of skewness was calculated to be 5.22 from the 7-day average low flows. This data could have been fit by Gumbel's logextremal theory for drought flows as a limiting flow is approached at the lower magnitudes of flow for the distribution. Plotting these data on logextremal probability paper yielded approximately straight lines, unlike for the other three gaging stations. The graphs included are on normal probability paper. The coefficient of variability was calculated to be approximately 0.74 for all three flow distributions, but the meaning of this value is not the same as for the other three gaging stations, since the distribution is highly skewed.

Taking the coefficients of skewness calculated at the gaging stations for the 7-day low flows and testing them for fitting a log-extremal distribution by using the test equation proposed by Gumbel (13) and defined in Chapter II of this thesis

$$\varepsilon \gtrless 0 \text{ if } \bar{x} + S\left(A(\alpha) - B(\alpha)\right) \gtrless 0 \qquad (5)$$

yields the results shown in Table IV.

These calculations verify the graphical conclusion that the Barren Fork at Eldon is the only gaging station that can be fit by Gumbel's logextremal distribution or any distribution for which a lower limit is assumed. Since $\varepsilon<0$, the data does not show a tendency to approach a minimum flow at Watts, Kansas, and Tahlequah. The concave upward curves observed for the plots on log-extremal probability are the indicators that the minimum low flow is less than zero for these flows.

Analyzing the low flows graphically on normal probability paper seems to be the best way to achieve a graphical interpretation of the low flow distributions observed at Watts, Tahlequah, and Kansas. A possible reason that these distributions are nearly normal is the high degree of flow variability of these streams in the Oklahoma hills--extreme drought flows in some years being very much below the mean drought flow, and a variable base flow that depends upon the severity of drought as well as the physical basin characteristics (19). These distributions do not conform to Gumbel's logextremal theory, and cannot accurately be treated graphically as distributions approaching a minimum flow greater than or equal to zero.

TABLE IV

TESTING FOR A LOGEXTREMAL DISTRIBUTION

Gaging Station	C_s	$\bar{X} + S\left[A(\alpha) - B(\alpha)\right]$
Watts 07195500	0.347	-20.0
Kansas 0719600	-0.206	-16.2
Tahlequah 07196500	0.193	-51.9
Eldon 07197000	5.22	11.5

Decisions related to drought flow usually center around a 10-year recurrence interval, and the Environmental Protection Agency has designated that a 7-day average 10-year recurrence low flow should be used for design purposes. The 1-, 7-, and 30-day averages at a 10-year recurrence interval were taken from the graphs at the four gaging stations. These low flows are listed in Table V.

TABLE V

10-YEAR RECURRENCE INTERVAL LOW FLOWS (cfs)
(fraction of mean low flow in parentheses)

Station	1-day Average		7-day Average		30-day Average	
Watts 07195500	12	(0.20)	16	(0.23)	20	(0.17)
Kansas 07196000	1.8	(0.14)	3.3	(0.24)	5.8	(0.30)
Tahlequah 07196500	14	(0.16)	20	(0.21)	28	(0.21)
Eldon 07197000	2.3	(0.13)	2.8	(0.15)	3.5	(0.13)

The 10-year recurrence low flows at Tahlequah for the 7- and 30-day average low flows were determined to be 20 and 28 cfs, compared to 12 and 20 cfs determined in the 1959 United States Geological Survey study (8). The 1959 study was made using logextremal probability paper, and with sixteen fewer years of record, which explains the differences in the values obtained. The 10-year recurrence interval low flows computed in this thesis were very low compared to usual flows in these streams, being even less than one-quarter of the mean yearly low flow for all except the 30-day average low flows on Flint Creek. This characteristic was also indicated by the high coefficient of variability calculated for these low flow distributions.

A further quantitative analysis of the 10-year recurrence interval low flows is made possible by comparing the yields listed in Table VI. The yield is defined as the ratio of the flow to the drainage area above the station and can be used to compare the different gaging stations. It can be seen that at the 7-day average low flows, the flow on Flint Creek shows the highest yield, 0.030. A higher yield is recorded at Watts (0.025) than at Tahlequah (0.021) for the 7-day average flow, with the Barren Fork showing a very poor yield of 0.009. The gaging stations at Watts and Tahlequah show approximately the same variability in yield for the different drought flow durations. The Flint Creek shows a wider variability in yield between the 1-, 7-, and 30-day low flows than Watts and Tahlequah, while the Barren Fork near Eldon shows little variation in yield between the different low flow durations. This is probably due to a reliable base flow being the 10-year recurrence flow for all three low flow durations on the Barren Fork. In studies done in Michigan, Velz (18) considered 7-day average 10-year drought flow yields of 0.1 to be very low. The very low yields calculated at the four gaging stations in the Illinois River basin are indicative of the severe dry summer periods for which Oklahoma is noted.

B. Correlating Low Flows With Cross-sections

The results of using cross-sections and ratings of flow versus gage height at the four gaging stations to determine channel and flow characteristics at low flows are given in Table VII.

TABLE VI

YIELD FOR 10-YEAR RECURRENCE INTERVAL LOW FLOWS
cfs/mi^2

Station	1-day Average	7-day Average	30-day Average
Watts 07195500	0.019	0.025	0.032
Kansas 07196000	0.016	0.030	0.053
Tahlequah 07196500	0.015	0.021	0.029
Eldon 07197000	0.007	0.009	0.011

TABLE VII

CORRELATION OF CROSS-SECTIONS AND 10-YEAR RECURRENCE INTERVAL LOW FLOWS
(1-, 7-, and 30-day Average Low Flows)

Low Flow (cfs)	A (ft^2)	W (ft)	H (ft)	V (ft/sec)
	(Watts)(07195500)			
12	24.7	190	0.13	0.48
16	32.3	190	0.17	0.50
20	39.9	190	0.21	0.50
	(Kansas)(07196000)			
1.8	9.4	157	0.06	0.19
3.3	17.3	157	0.11	0.19
5.8	26.7	157	0.17	0.22
	(Tahlequah)(07196500)			
14	73.0	152	0.48	0.19
20	91.2	152	0.60	0.22
28	111.0	152	0.73	0.25
	(Eldon)(07197000)			
2.3	26.0	186	0.14	0.09
2.8	29.8	186	0.16	0.09
3.5	31.6	186	0.17	0.10

These studies show that a wide channel is present at all four locations, permitting only a very small effective depth of flow. This depth varies from 0.06 ft for the 1-day average low flow near Kansas, to 0.73 ft for the 30-day average low flow near Tahlequah. The hydraulic depth was equal to the difference in gage height since the cross-section approximated a rectangular section due to this very small effective hydraulic depth.

The velocities of flow on Flint Creek near Kansas were calculated to be nearly the same as those of the Illinois River near Tahlequah, being approximately 0.20 ft/sec. The velocities at Watts on the Illinois River were computed to be more than double this, averaging 0.5 ft/sec. The velocities on the Barren Fork near Eldon were much lower, averaging approximately 0.1 ft/sec. The average slope for the reach containing the gaging station, and the velocity and area from the cross-section allowed comparison of the values calculated in Table VII by computation of the coefficient of roughness (n) from Manning's equation

$$n = \frac{1.49}{V}\left(\frac{A}{WP}\right)^{2/3} S^{\frac{1}{2}} \tag{14}$$

Results of these computations for the 7-day average low flows showed

Gaging Station	n
Watts (07195500)	0.027
Kansas (07196000)	0.057
Tahlequah (07196500)	0.129
Eldon (07197000)	0.180

Since all of these cross-sections have approximately the same characteristics in nearly the same type of terrain, it would be expected that the n values for the four stations would be about the same. The value of n at the Watts gaging station is in the range that would be expected for these watercourses, but the variability of the computed values allows little comparison of the channel and flow characteristics at the cross-sections. These results question the reliability of using the cross-sections to calculate the area and velocity of flow. Another possible reason for the discrepancy between the n values is that averaging the slope between 20 ft contour intervals is too gross, and does not truly give a representative slope at the cross-sections.

The K_2 values calculated by the use of the formula proposed by Isaacs, et al. (47) for the comparison of Busch's method for stream assimilation to the Streeter-Phelps equation are presented in this section. These calculated coefficients of reaeration are shown in Table VIII for the various low flows on the Flint Creek near Kansas. The K_2 values were calculated to be very high using the very shallow depths of less than one foot. The validity of using these very small depths in calculating K_2 values is questionable, since this depth may not truly represent the flow characteristics of the stream, and the empirical reaeration formula was not developed using very shallow depths varying from 0.06 to 0.17 ft. The use of these values should probably be limited to the comparison that will be made of the models for stream assimilative capacity.

TABLE VIII

CALCULATION OF K_2 AND K_L FOR THE CROSS-SECTION AT FLINT CREEK

$$K_2 = 2.833 \frac{V}{H^{1.5}}$$

Flow (cfs)	H (ft)	V (ft/sec)	K_2	K_L
1.8	0.06	0.19	110.0	6.6
3.3	0.11	0.19	44.1	4.8
5.8	0.17	0.22	26.7	4.5
67.0	0.88	0.47	4.6	4.0
133.0	1.27	0.64	3.6	4.6

C. Comparison of Busch's Method to the Streeter-Phelps Equation

Busch's solution for stream assimilative capacity uses the general expression for gas transfer to a liquid, as defined in Chapter II

$$\frac{dM}{dt} = K_{L(min)} \, D_{(max)} \, A_i \tag{13}$$

This equation defines the maximum rate at which oxygen can be uniformly transferred to a reach subject to the worst conditions of reaeration, at the maximum allowable dissolved oxygen deficit. The only kinetics involved in this equation are those used in defining the worst conditions of reaeration for the reach. The kinetics of deoxygenation are not involved, since the rate at which an ultimate biological oxygen demand can be satisfied must be constant and equal to the maximum rate at which oxygen can be transferred. This is if the dissolved oxygen deficit is to remain constant at its maximum allowable value in the reach.

A point discharge of an organic loading into a stream must be defined by some type of kinetics of oxygen uptake if the location and magnitude of the maximum dissolved oxygen deficit is to be found. The Streeter-Phelps equation was developed for such a calculation, and considers that the ultimate biological oxygen demand is not constant, but is decreasing with passage through the reach as biological oxidation occurs. A series of point discharges can approximate a uniform loading, but for a given length of reach, a certain magnitude of organic loading will cause a greater maximum dissolved oxygen deficit if it is applied at a point than if it is divided and distributed throughout the reach.

This was shown by the Streeter-Phelps equation for the Flint Creek reach by first assuming an initial BOD of 8840 lbs/day at the 7-day low flow of 3.3 cfs for a 20-mile reach. The critical DO value was then calculated. An initial BOD of 4420 lbs/day was next assumed for a 10-mile reach and an artificial waste flow was added containing a biological oxygen demand of 4420 lbs/day at the 10-mile point or half-way through the original 20-mile reach. This artificial waste flow was added at a very high concentration of 15,400 mg/l at 0.1 cfs so as not to alter the flow characteristics of the stream significantly. The maximum DO sag for this case was calculated. In a similar manner, the reach was then divided into four 5-mile increments with 2210 lbs/day of BOD added at the beginning of each increment, then eight 2.5-mile increments with 1105 lbs/day of BOD added at the beginning of each increment. These same calculations were then repeated with the lengths of all reaches halved. The results are summarized in Table IX. These results, although subject to many assumptions, show that when a design waste loading is calculated for a given length of reach, the lower DO concentration will be produced by a single point discharge into the reach.

TABLE IX

MAXIMUM DO DEFICITS FROM THE STREETER-PHELPS EQUATION FOR PROPORTIONAL REACH INCREMENTS AND WASTE LOADINGS

BOD Loadings at Each Reach Increment (lbs/day)	Number of Increments	Length of Increment (miles)	Maximum Critical DO Deficit (mg/l)	Distance Downstream to Max. Critical Deficit (miles)
8840	1	20	5.06	.33
4420	2	10	3.67	10.36
2210	4	5	2.96	15.32
1105	8	2.5	2.46	17.76
		(reach lengths halved)		
8840	1	10	5.06	.33
4420	2	5	4.18	5.34
2210	4	2.5	3.64	7.80
1105	8	1.25	3.15	8.99

Busch's solution for stream assimilation capacity calculates the waste loading that can be uniformly applied to a reach of given length. In his article (3) he states that a shorter length of reach must be used in the calculations for a point discharge, but no quantitative definitions of this shorter length of reach are given. Thus, when a given length of reach is used in calculating the amount of oxygen that can uniformly be transferred over the length, the worst conditions--those of a point source in which the maximum dissolved oxygen deficit is defined by the competing kinetics of reaeration and oxygen uptake--are not considered. This can be illustrated by comparing Busch's method for stream assimilation capacity with the Streeter-Phelps equation for the 12.8-mile Flint Creek reach in Oklahoma. The comparison is accomplished by determining the length of reach necessary for use in Busch's equation that will yield a BOD loading sufficient to cause the Streeter-Phelps equation to predict a critical dissolved oxygen concentration equal to 4.0 mg/l when the loading is applied at a single point. This analysis was accomplished for the five different flows listed for the Flint Creek cross-section near Kansas. These are listed in Table VIII, Chapter III. The mass transfer coefficient, K_L, was assumed to be 4.0 days^{-1} for Busch's equation, since this is the minimum K_L calculated for the five different flows. The K_2 used in the Streeter-Phelps equation for any of the given flows was determined by dividing this minimum K_L (4.0) by the depth at that flow. Other assumptions were given in Chaper III, Methods of Study.

Busch's equation becomes, considering units

$$\frac{dM}{dt} = K_{L(min)} \times D_{max} \times W \times L \times 6.236 \times 10^{-5} \tag{17}$$

where $\frac{dM}{dt}$ is in lbs/day, W is in feet, and L is the length of the reach in feet. This can be converted to a waste concentration in the river for a given flow by

$$L_a = \frac{dM}{dt} \Big/ Q = \frac{(1.157 \times 10^{-5})}{Q} W\ L\ D_{max} K_L (min) \qquad (18)$$

where Q is in cfs, and L_a is in mg/l.

The Streeter-Phelps equation for the critical deficit is

$$D_c = \frac{1}{f} L_a \times e^{-K_1 T_c}$$

where $f = \frac{K_2}{K_1}$. Inserting the L_a calculated from equation (18), and solving for the case when $D_c = D_{max}$, yields

$$e^{K_1 T_c} = \frac{1}{f} \left(1.157 \times 10^{-5}\right) W \frac{L}{Q} K_{L(min)}$$

$$t_c = \frac{1}{K_1} \ \ell n \frac{1}{f} \left(1.157 \times 10^{-5}\right) W \frac{L}{Q} K_{L(min)} \qquad (19)$$

but also from Streeter-Phelps equation, at the critical deficit

$$t_c = \frac{1}{K_1(t-1)} \ \ell n \left[F \left(1-(F-1) \frac{D_a}{L_a} \right) \right]$$

but again substituting the L_a calculated from equation (18) yields

$$t_c = \frac{1}{K_1(f-1)} \ \ell n \left[f \left(1-(f-1) \frac{D_a Q}{1.157 \times 10^{-5}\ W L D_{max} K_{L(min)}} \right) \right] \qquad (20)$$

Setting equation (19) equal to equation (20) and solving, gives

$$L = \frac{fQ}{\left(1.157\text{x}10^{-5}\right)WK_{L(min)}}\left[f\left(1-(f-1)\frac{D_aQ}{\left(1.157\text{x}10^{-5}\right)WLD_{max}K_{L(min)}}\right)\right]^{\frac{1}{f-1}} \quad (21)$$

Since all of the variables can be determined for a given flow in equation (21) except L, this equation can be solved by iteration for the value of L in Busch's equation for which the predicted waste loading will cause the critical dissolved oxygen level to equal the minimum allowable value from computation in the Streeter-Phelps equation. A summary of the calculations for the Flint Creek cross-section are given in Table X.

From the equation

$$\frac{dM}{dt} = K_{L(min)}\ D_{max}\ WL \quad (22)$$

For a given flow and cross-section, the waste loading predicted is directly proportional to the length of the reach. Thus, the design loading calculated by Busch's equation at the 7-day low flow (3.3 cfs) for a 14.6-mile reach proportionately exceeds that predicted for a 14.5-mile reach, and is proportionately less than that predicted for a 17.0-mile reach. But as shown on Table X for a point discharge at the beginning of these different reach lengths, the loading predicted for a 14.6-mile reach of 11,365 lbs/day will cause a minimum dissolved oxygen concentration of 4.0 mg/l by the Streeter-Phelps equation, no matter what the length of the reach may actually be. Thus, if the 14.5-mile reach is used in Busch's formulation for stream assimilation capacity, the dissolved oxygen concentration will stay above 4.0 mg/l as predicted by the Streeter-Phelps equation, but if a 17.0-mile reach is used for

TABLE X

LOADINGS FROM BUSCH'S EQUATION USED AS A POINT DISCHARGE IN THE STREETER-PHELPS EQUATION

	Reach length, miles									
	14.5 mi		14.6 mi		17.0 mi		37.2 mi		52.0 mi	
Flow (cfs)	lbs BOD/day	DO (mg/l)	lbs BOD/day	DO (mg/l)	lbs BOD/day	DO (mg/l)	lbs BOD/day	DO (mg/l)	lbs BOD/day	DO (mg/l)
1.8	11,300	4.00	11,380	3.95	13,250	3.02	29,000	0	40,535	0
3.3	11,300	4.04	11,380	4.00	13,250	3.12	29,000	0	40,535	0
5.8	11,300	4.19	11,380	4.16	13,250	4.0	29,000	0	40,535	0
67	11,660	5.00	11,740	5.00	13,670	5.0	29,925	4.0	41,825	1.59
133	11,730	5.00	11,810	5.00	13,754	5.0	30,110	4.62	42,085	4.0

lbs BOD/day = calculated from Busch's equation for stream assimilation capacity using length of reach given

DO (mg/l) = calculated from the Streeter-Phelps equation for a point discharge using the loading calculated from Busch's equation, and is the minimum or critical DO concentration for the reach the calculation in Busch's equation, the Streeter-Phelps equation predicts that the dissolved oxygen concentration will be lowered to 3.12 mg/l at this flow of 3.3 cfs. There is only one reach length which will cause the DO to sag to the minimum allowable value, and this length is independent of the actual length of the reach.

From the calculations, at each flow there is a certain length of reach associated with this flow for which the design loading calculated from Busch's equation will cause the Streeter-Phelps equation to predict a DO concentration of 4.0 mg/l. Since the length of this reach increases with increasing flow, it can be ascertained that the design loadings predicted from Busch's equation become more conservative for increasing values of a design low flow for the given cross-section. For example, if a 14.5-mile reach was actually the length of the reach and the design flow was 67 cfs, Busch's equation calculates a design loading of 11,660 lbs of oxygen demand per day, which the Streeter-Phelps equation predicts would cause no sag in the dissolved oxygen concentration. A BOD loading of 29,925 lbs per day is necessary to cause the dissolved oxygen concentration to drop to 4.0 mg/l, according to the Streeter-Phelps equation. This corresponds to a reach length of 37.2 miles for the flow of 67 cfs. But if the design flow for this 14.5-mile reach was 1.8 cfs, the loading predicted from Busch's equation would be 11,300 lbs of oxygen demand/day, just 360 lbs less than that predicted for the flow of 67 cfs. For this case, however, the Streeter-Phelps equation predicts that the dissolved oxygen concentration would sag to 4.0 mg/l, the minimum allowable value. The reason for this observation is that for a given length of reach in Busch's equation, the

the design waste loading is directly proportional to the width of the water surface on the cross-section, since this defines the minimum interfacial surface area available for oxygen transfer. Thus, for the steep bank slopes on Flint Creek, the width varies little with depth, and thus the assimilative capacity as predicted by Busch's formulation varies little for increasing flows. The Streeter-Phelps equation, however, predicts that the assimilative capacity for a point source discharge is much less for lower flows than for increasing flows. The reason for this can be shown from the differential form of the equation

$$\frac{dD}{dt} = K_1 L - K_2 D \tag{8}$$

From this equation, K_1 and the initial value for D (D_a) are constant at the instant of discharge of flow in Table X, while K_2 is inversely proportional to the depth

$$K_2 = \frac{K_{L(min)}}{H}$$

If it is assumed that at the instant after discharge $\frac{dD}{dt} = 0$, or the DO profile will not sag, the Streeter-Phelps equation becomes

$$0 = K_1 L_a - \frac{K_{L(min)}}{H} D_a$$

but since $L_a = \frac{\text{loading}}{Q}$ where the loading is the rate of BOD application

$$K_1 \frac{\text{loading}}{Q} = \frac{K_{L(min)} D_a}{H}$$

$$\text{loading} = \frac{K_{L(min)} D_a}{K_1} \frac{Q}{H}$$

or the loading rate is proportional to $\frac{Q}{H}$.

Since the flow rate increases much faster than at a linear rate with increasing depths (by the rating tables in Appendix A), the loading can be increased with increasing depths and still not cause the DO concentration to sag. This dilution of pollutants with increased flow rate in the Streeter-Phelps equation has been shown for the specific case $\frac{dD}{dt} = 0$ in order to aid in the explanation of the calculations shown in Table X; or that the total BOD loading can be greatly increased with increasing flow and still maintain a minimum DO concentration greater than 4.0 mg/l by the Streeter-Phelps equation.

Busch's equation does not take into account the fact that dilution is going to have an influence on the rate of change of the dissolved oxygen deficit for different flow regimes, and thus on the maximum dissolved oxygen deficit. The amount of water passing a point per unit time, or the discharge (Q) associated with a given depth, is not considered in Busch's formulation except in the determination of $K_{L(min)}$.

Busch's method gives no information concerning the location of the critical DO deficit, so no comparison can be made with the Streeter-Phelps equation's computation of this location. The critical DO deficit was calculated to occur within a half-mile for most of the data used in the Streeter-Phelps equation. For example, at the 7-day average low flow for a BOD loading at 11,380 lbs/day, the critical DO deficit occurred .30 mi downstream in 2.2 hours. The very shallow depths and high K_2 values used in the equation were the affectors of this calculated quick sag and recovery.

Thus, these results show two points:

1) For a given flow there is a single length of reach which, when

used in Busch's equation for stream assimilation capacity, the loading predicted will cause the dissolved oxygen concentration to sag to the minimum allowable value, as predicted for a point source by the Streeter-Phelps equation. This length depends only upon $K_{L(min)}$ and W, so there is no relationship between this reach length and the actual length of the reach.

2) For a given reach length, the BOD loadings predicted by Busch's equation, when applied to a point source, are the least conservative for lower values of flow as compared to the Streeter-Phelps equation, because the velocity of flow or the relation of discharge to depth is not taken into account except for determining $K_{L(min)}$.

The Streeter Phelps equation was not meant to be used as the best possible prediction of the DO profile in these comparisons; its limitations have been previously discussed. It was used for the purpose of

1) showing that for a given length of reach, a point discharge will place the greatest burden upon the oxygen resources of the stream, and

2) showing the anomolies that exist between using a kinetic model to predict the dissolved oxygen profile caused by a point waste discharge and calculating allowable waste loadings for a reach by using a uniform rate of oxygen transfer over the entire reach.

The concept of using only the minimum reaeration capacity of a stream to predict its assimilative capacity may be a valid line of reasoning. Still, incorporating this idea into the kinetics of competing reaeration and deoxygenation initiated by a point discharge seems to be a more rational way to proceed than in designing for uniform loadings. Uniform loadings usually come under the heading of natural pollution and are nearly impossible to predict. Point discharges are the vandals that have frequently been known to upset natural balances in a stream when a town or industry discharges its wastes into the stream. These wastes are predictable and capable of being controlled. Further defining the applicability of the general aeration formula as presented in Busch's "Five-Minute Solution for Stream Assimilation Capacity," (3) seems necessary if it is to be used in stream pollution problems.

CHAPTER V

SUMMARY AND CONCLUSIONS

The distributions of yearly drought flows at the Watts, Tahlequah, and Kansas gaging stations on the Illinois River and Flint Creek were found not to conform to Gumbel's logextremal theory for droughts. These distributions were only slightly skewed and thus were best fit by a straight line on normal probability paper. Logextremal probability paper should not be used to define this type of distribution, as the steeply sloping, concave up curve indicates that the minimum low flow approached is less than zero.

The high degree of variability that was observed in the drought flows at Watts, Tahlequah, and Kansas--the coefficient of variability being greater than 0.60 for the 1-, 7-, and 30-day low flows--and the physical basin characteristics are the reason for the nearly normal distribution. A different distribution was exhibited by the Barren Fork at Eldon in that the low flows observed at this station approached a minimum low flow.

The 10-year recurrence low flows at these gaging stations are very low, being less than one-fourth of the mean drought flow because of the high variability from year to year of the low flows. The yields of these 10-year recurrence interval drought flows per square mile of drainage area are also very poor, being less than 0.06 cfs/mi^2 at all gaging stations.

The correlation of the 10-year recurrence low flows with the cross-sections to determine the width, depth, area, and velocity of flow yielded results that, from the method of determination and by testing in Manning's equation, were thought to be unreliable. Thus, the only use of these results was in using the Flint Creek cross-section for the comparison of Busch's method for stream assimilation capacity to the Streeter-Phelps equation.

To apply a given quantity of organic waste material at a point will cause the dissolved oxygen profile of the stream to sag to a greater extent than if the waste is divided and spread more uniformly throughout the reach. Busch's method for stream assimilative capacity involves calculating the maximum uniform loading rate that a biochemical oxygen demand can be applied to a reach of stream. Applying the loading calculated by Busch's formula as a point discharge to be treated by the Streeter-Phelps equation shows that

1) the actual length of a reach of stream is independent of the length of reach which when used to calculate a BOD loading by Busch's equation, this loading applied at a point will cause the dissolved oxygen concentration to sag to the minimum allowable value, and

2) the dilution capacity of a river or the discharge associated with a given depth is not totally accounted for in Busch's formulation.

There is thus no correlation between the loading that would be predicted by Busch's equation to maintain the dissolved oxygen concentration at acceptable levels, and that predicted by a kinetic model such as the Streeter-Phelps equation when a point discharge is being considered.

CHAPTER VI

SUGGESTIONS FOR FUTURE WORK

The present quality of the waters in the Illinois River basin needs to be better defined than available water quality data allows. Routine sampling, such as monthly samples taken at five or six locations along Flint Creek and the Illinois River, could provide a background in water quality variability subject to seasonal changes. Emphasis should probably be given to the months of August, September, and October, when nearly all of the historical drought flows have occurred, with possibly a more intensive sampling program undertaken during these periods when the quality of water with respect to a stable hydrograph would allow better correlation of water quality to the hydrology of the basin. Such a sampling program now undertaken at the Oklahoma State University could allow correlation of the data with the National Weather Service River Forecast Hydrology Model which has been calibrated by Ron Martin at the Oklahoma State University to better fit low flows in the Illinois River basin. Historical water quality records for the basin could then be defined as the function of hydrological conditions.

Cross-sections at various intervals along the river, and the velocity of flow, determined by either tracer studies or by measurement at a single point, need to be determined so that the hydraulics and thus the reaeration capacity of the stream can be better defined. Then, by predicting the types of loadings, the organic waste assimilative capacity of the stream could be predicted by models such as the Streeter-Phelps equation or a truly more rational approach to determining stream DO levels such as that proposed by Peil and Gaudy (36). Determination of this capacity could play a beneficial role in formulating intelligent plans for the basin.

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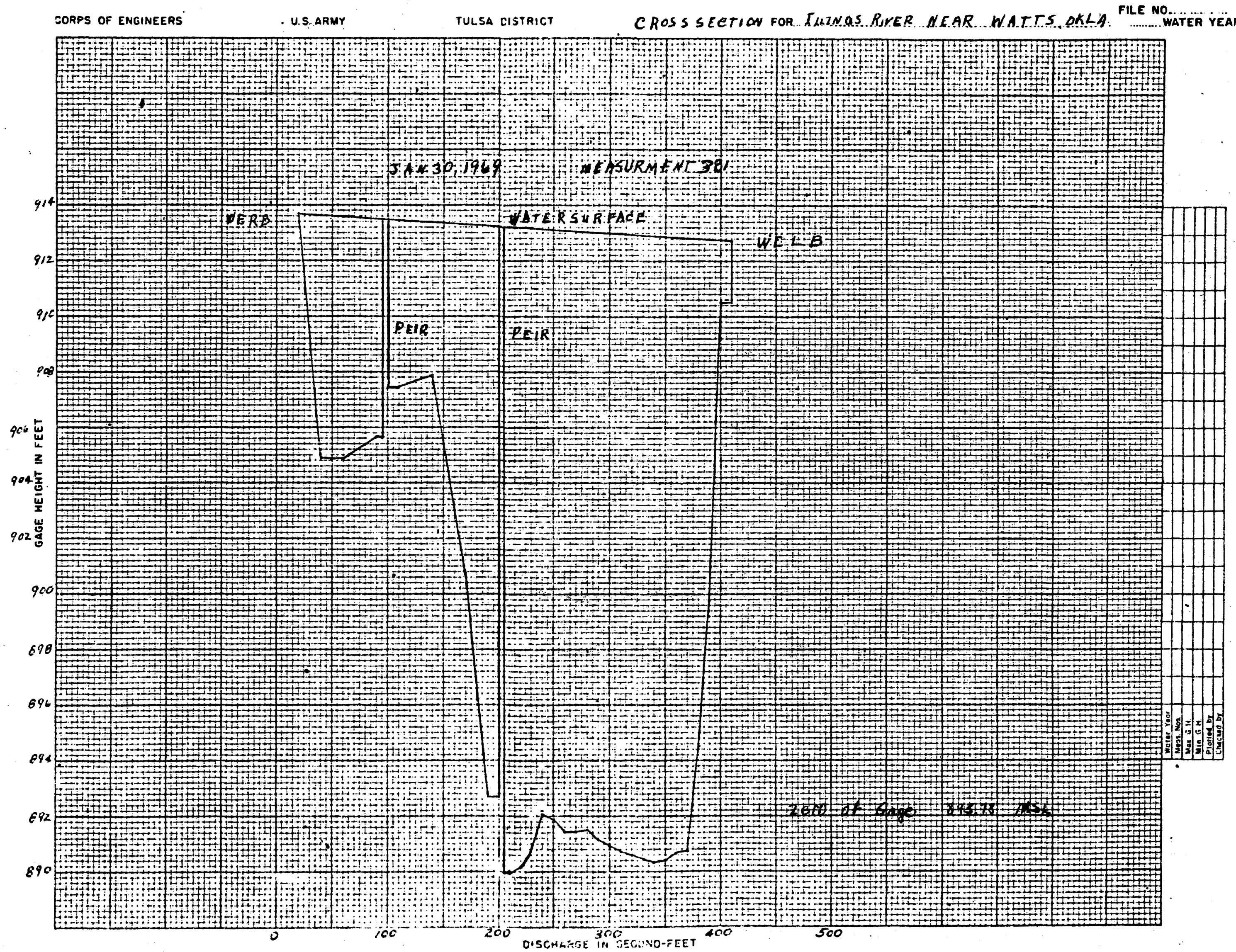
APPENDIX A

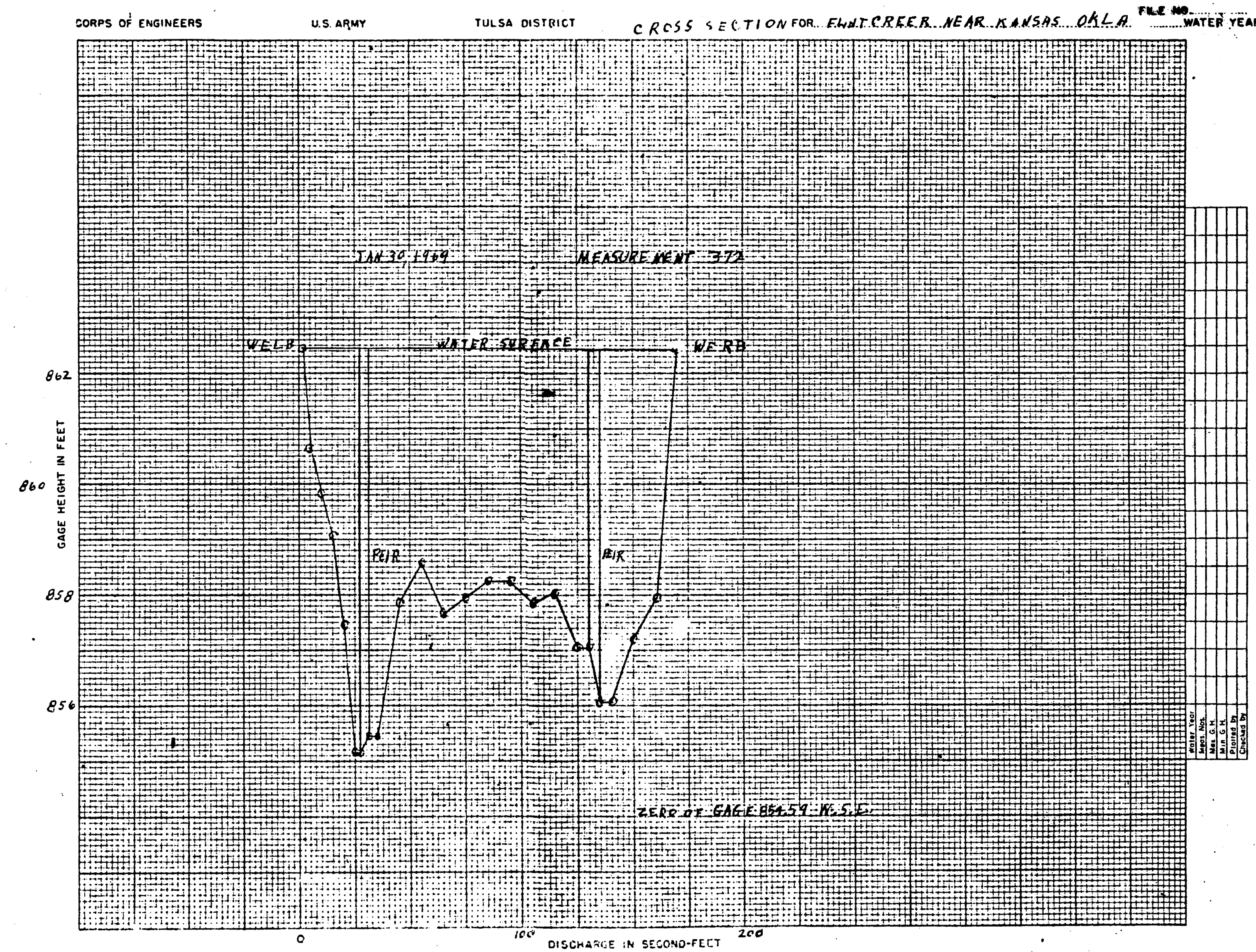
CROSS-SECTIONS AND RATING TABLES FOR THE GAGING STATIONS NEAR WATTS (07195500), KANSAS (07196000), TAHLEQUAH (07196500), AND ELDON (07197000)


CORPS OF ENGINEERS
U.S. ARMY
TULSA DISTRICT
CROSS SECTION FOR ILLINOIS RIVER NEAR WATTS, OKLA.
FILE NO.
WATER YEAR
JAN 30, 1969
MEASURMENT 381
WERB
WATER SURFACE
WELB
PEIR
PEIR
Zero of Gage 893.78 MSL
GAGE HEIGHT IN FEET
914
912
910
908
906
904
902
900
898
896
894
892
890
DISCHARGE IN SECOND-FEET
0
100
200
300
400
500
Water Year
Meas. Nos.
Max. G.H.
Min. G.H.
Plotted by
Checked by

EXPANDED RATING TABLE

07195500 ILLINOIS RIVER NEAR WATTS, OKLAHOMA TYPE LOG(SCALE OFFSET = 0.90) RATING NO 13

GAGE HEIGHT IN FEET	DISCHARGE IN CUBIC FEET PER SECOND .0	.1	.2	.3	.4	.5	.6	.7	.8	.9	DIFF IN Q PER FOOT GH
0.00											
1.00	9.0	19	30	41	53	65	78	92	106	123	131.0
2.00	140	159	178	199	222	245	269	295	321	349	243
3.00	378	408	439	470	502	534	567	601	636	675	337
4.00	715	756	798	841	886	932	980	1030	1080	1130	465
5.00	1180	1230	1290	1340	1400	1460	1520	1580	1640	1700	590
6.00	1770	1830	1900	1970	2040	2100	2180	2250	2320	2390	702
7.00	2470	2550	2620	2700	2770	2850	2920	3000	3080	3160	771
8.00	3240	3320	3400	3480	3570	3650	3740	3830	3920	4010	861
9.00	4100	4190	4280	4380	4470	4570	4670	4760	4860	4960	962
10.00	5060	5170	5270	5370	5480	5590	5700	5810	5920	6030	1076
11.00	6140	6250	6370	6480	6600	6720	6840	6950	7070	7200	1180
12.00	7320	7440	7570	7690	7830	7960	8100	8240	8380	8520	1342
13.00	8660	8800	8950	9090	9240	9390	9540	9690	9840	9990	1440
14.00	10100	10300	10500	10600	10800	10900	11100	11300	11400	11600	1656
15.00	11800	11900	12100	12300	12500	12600	12800	13000	13200	13400	1742
16.00	13500	13700	13900	14100	14300	14500	14700	14900	15100	15300	1959
17.00	15500	15700	15900	16100	16300	16500	16700	16900	17100	17400	2145
18.00	17600	17900	18200	18500	18800	19100	19400	19700	20000	20300	3061
19.00	20700	21000	21300	21600	22000	22300	22600	23000	23300	23600	3342
20.00	24000	24300	24700	25100	25400	25800	26100	26500	26900	27300	3611
21.00	27600	28000	28400	28800	29200	29600	30200	30800	31500	32100	5054
22.00	32700	33400	34100	34700	35400	36100	36800	37600	38300	39000	7056
23.00	39800	40600	41300	42100	42900	43700	44500	45400	46200	47100	8108
24.00	47900	48800	49700	50600	51500	52500	53400	54400	55300	56300	9360
25.00	57300	58300	59300	60400	61400	62500	63600	64600	65700	66900	10697
26.00	68000										


CORPS OF ENGINEERS
U.S. ARMY
TULSA DISTRICT
CROSS SECTION FOR FLINT CREEK NEAR KANSAS OKLA
FILE NO.
WATER YEAR
JAN 30, 1969
MEASUREMENT 372
WELB
WATER SURFACE
WERB
REIR
REIR
ZERO OF GAGE 854.59 W.S.E.
862
860
858
856
GAGE HEIGHT IN FEET
0
100
200
DISCHARGE IN SECOND-FEET
Water Year
Msmts. Nos.
Max G. H.
Min G. H.
Plotted by
Checked by

UNITED STATES DEPARTMENT OF INTERIOR - GEOLOGICAL SURVEY - WATER RESOURCES DIVISION

EXPANDED RATING TABLE

07196000 FLINT CREEK NEAR KANSAS, OKLAHOMA TYPE LOG(SCALE OFFSET = 5.50) RATING NO 08

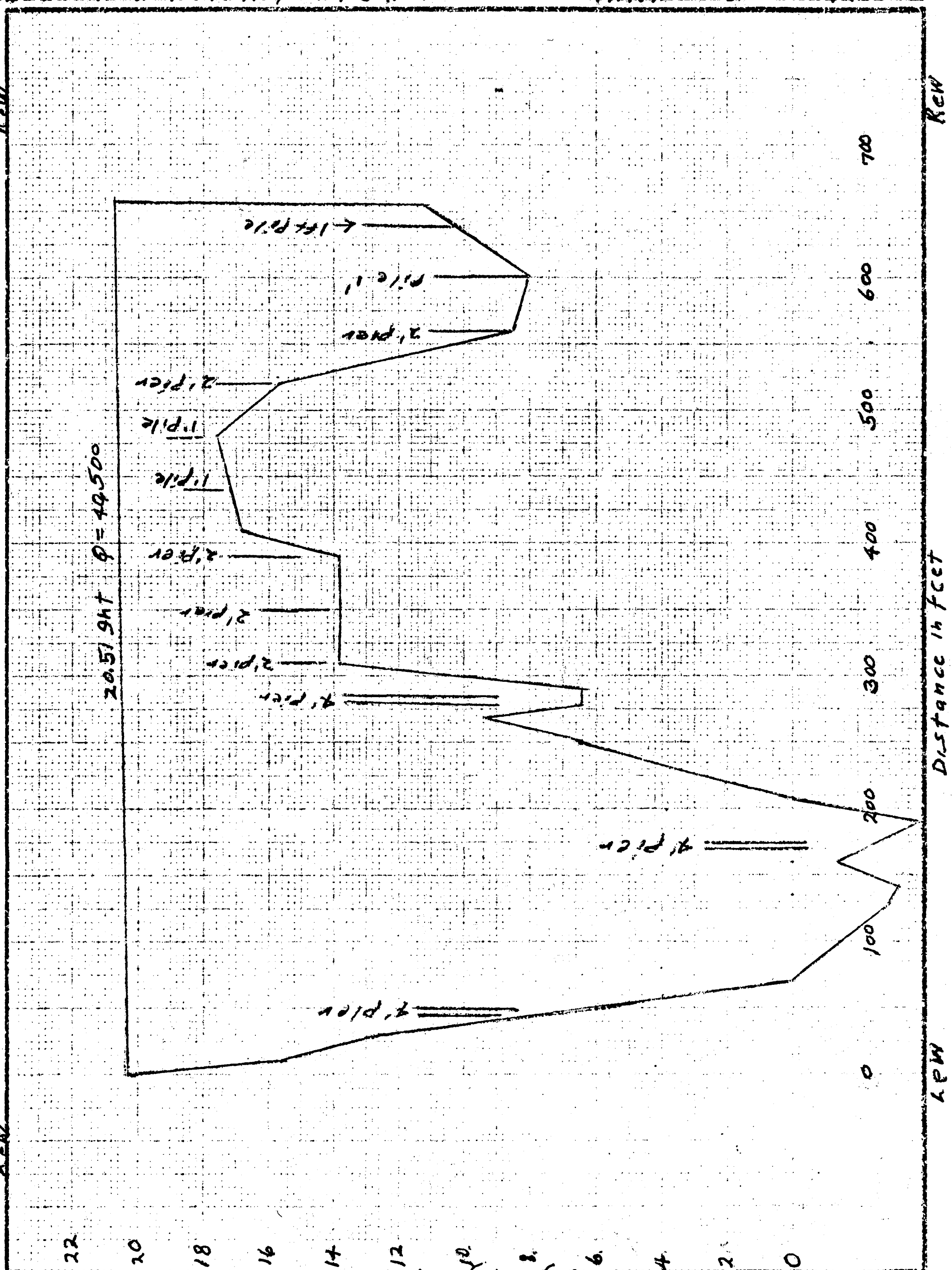
GAGE HEIGHT IN FEET	DISCHARGE IN CUBIC FEET PER SECOND .0	.1	.2	.3	.4	.5	.6	.7	.8	.9	DIFF IN Q PER FOOT GH
5.00						.00	3.0	7.1	12	18	
6.00	25	34	43	54	67	80	96	113	133	156	159
7.00	190	231	283	354	438	529	620	722	835	948	912
8.00	1070	1200	1340	1500	1680	1870	2080	2300	2490	2690	1821
9.00	2890	3110	3330	3560	3760	3970	4180	4390	4620	4850	2189
10.00	5080	5300	5520	5750	5980	6220	6470	6710	6960	7210	2450
11.00	7460	7720	7980	8240	8510	8790	9060	9340	9620	9900	2736
12.00	10200	10500	10800	11100	11400	11700	12000	12300	12600	12900	3112
13.00	13300	13600	13900	14300	14600	15000	15300	15700	16000	16400	3445
14.00	16700	17100	17500	17900	18300	18600	19000	19400	19800	20200	3856
15.00	20600	21100	21500	21900	22300	22700	23200	23600			

9-218-E
(March 1949)
Cross-Section

UNITED STATES DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY
WATER RESOURCES DIVISION

File ....

Illinois River nr. Tahlequah Okla. (Meas 725) 1973



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GPO 859-736

UNITED STATES DEPARTMENT OF INTERIOR - GEOLOGICAL SURVEY - WATER RESOURCES DIVISION

EXPANDED RATING TABLE

07196500 ILLINOIS RIVER NEAR TAHLEQUAH, OKLA. TYPE LOG(SCALE OFFSET = 1.50) RATING NO 12

GAGE HEIGHT IN FEET	DISCHARGE IN CUBIC FEET PER SECOND										DIFF IN Q PER FOOT GH
	.0	.1	.2	.3	.4	.5	.6	.7	.8	.9	
1.00						.00	1.0	3.1	6.1	10	
2.00	15	20	26	33	40	49	58	68	78	91	91
3.00	105	120	136	153	171	190	212	235	259	285	207
4.00	313	343	375	408	442	480	520	562	606	652	407
5.00	700	750	803	857	914	974	1040	1100	1160	1230	600
6.00	1300	1370	1440	1520	1590	1670	1750	1840	1920	2010	860
7.00	2100	2190	2280	2370	2470	2570	2670	2780	2880	2990	1004
8.00	3100	3210	3320	3430	3540	3660	3770	3890	4010	4130	1149
9.00	4250	4380	4510	4640	4770	4900	5040	5180	5330	5470	1366
10.00	5620	5770	5920	6070	6230	6390	6550	6710	6880	7040	1592
11.00	7210	7390	7560	7740	7910	8090	8270	8450	8640	8830	1807
12.00	9020	9210	9400	9600	9800	10000	10200	10400	10600	10800	1983
13.00	11000	11300	11500	11700	11900	12100	12400	12600	12800	13100	2261
14.00	13300	13500	13800	14000	14300	14500	14800	15000	15300	15600	2498
15.00	15800	16100	16300	16600	16900	17200	17400	17700	18000	18300	2786
16.00	18600	18900	19200	19600	20000	20300	20700	21100	21500	21900	3731
17.00	22300	22700	23100	23500	23900	24300	24800	25200	25600	26100	4212
18.00	26500	27000	27400	27900	28300	28800	29300	29800	30300	30700	4678
19.00	31200	31700	32200	32800	33500	34200	34900	35700	36400	37200	6761
20.00	38000	38800	39600	40400	41200	42000	42900	43700	44600	45500	8435
21.00	46400	47300	48300	49200	50100	51100	52100	53100	54100	55100	9791
22.00	56200	57200	58300	59400	60500	61600	62700	63800	65000	66200	11235
23.00	67400	68600	69800	71000	72300	73600	74900	76200	77500	78900	12845
24.00	80200	81600	83000	84500	85900	87400	88800	90300	91800	93400	14650
25.00	94900	96500	98100	99700	101000	103000	105000	106000	108000	110000	17070
26.00	112000	113000	115000	117000	119000	121000	122000	124000	126000	128000	18489
27.00	130000	132000	134000	136000	138000	140000	142000	145000	147000	149000	20852
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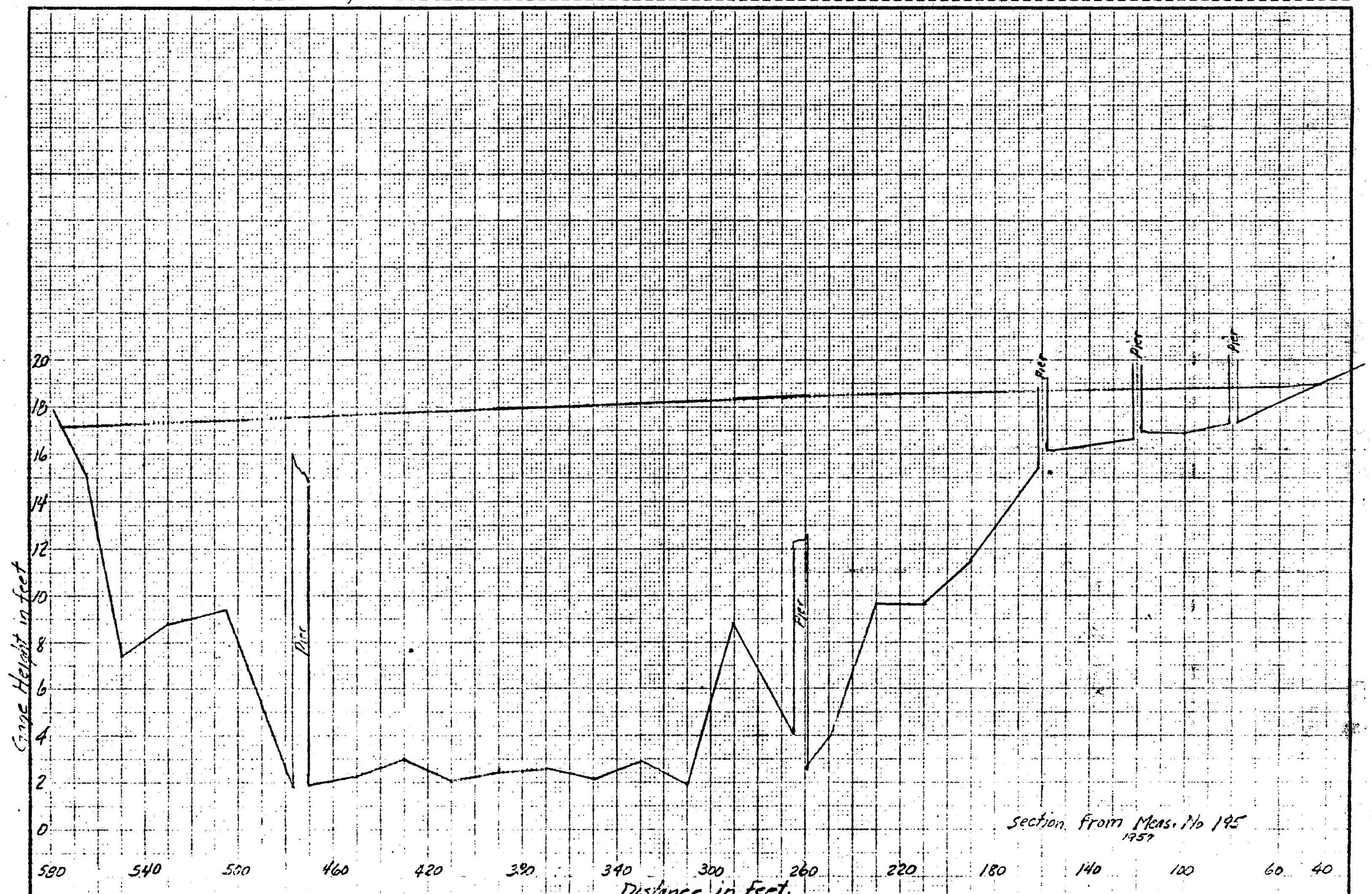
Cross-Section—Double

GEOLOGICAL SURVEY
WATER RESOURCES DIVISION

File 7-1770

Baron Fork near Eldon, Okla.



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UNITED STATES DEPARTMENT OF INTERIOR - GEOLOGICAL SURVEY - WATER RESOURCES DIVISION

EXPANDED RATING TABLE

07197000 BARON FORK AT ELDON, OKLA TYPE LOG(SCALE OFFSET = 4.00) RATING NO 14

GAGE HEIGHT IN FEET	DISCHARGE IN CUBIC FEET PER SECOND .0	.1	.2	.3	.4	.5	.6	.7	.8	.9	DIFF IN Q PER FOOT GH
4.00	.00	1.0	4.1	8.0	13	19	26	35	46	59	73.00
5.00	72	87	103	120	139	161	185	211	239	269	238
6.00	300	330	362	395	431	468	507	548	590	637	403
7.00	685	736	789	843	899	955	1010	1070	1130	1190	575
8.00	1260	1320	1390	1450	1520	1590	1660	1730	1800	1870	692
9.00	1940	2020	2090	2160	2240	2320	2390	2470	2550	2630	840
10.00	2720	2810	2900	2990	3080	3180	3270	3370	3470	3570	1020
11.00	3670	3770	3880	4000	4120	4240	4360	4490	4610	4740	1203
12.00	4870	5000	5140	5270	5410	5550	5700	5850	6010	6180	1469
13.00	6340	6510	6680	6850	7020	7200	7380	7560	7750	7930	1780
14.00	8120	8320	8510	8710	8930	9150	9370	9600	9840	10100	2175
15.00	10300	10600	10800	11100	11300	11600	11800	12100	12400	12600	2587
16.00	12900	13200	13500	13800	14000	14400	14700	15100	15400	15800	3296
17.00	16200	16500	16900	17300	17700	18100	18500	18900	19300	19800	4030
18.00	20200	20600	21100	21500	22000	22400	22900	23400	23900	24400	4601
19.00	24800	25300	25700	26200	26700	27200	27600	28100	28600	29100	5000
20.00	29600	30100	30600	31200	31700	32200	32800	33300	33900	34400	5375
21.00	35000	35500	36100	36700	37300	37900	38500	39100	39700	40300	5926
22.00	40900										

APPENDIX B

COMPUTER PROGRAM FOR THE STREETER-PHELPS EQUATION

```
C     THE  "STREETER-PHELPS   EQUATION
C       THE UNITS ON THE VARIABLES
C       BOD=MG/L,  DO=MG/L ,  Q=CFS,  TEMP=CENT,  VEL=FT/SEC,  DEPTH=FT.
C       DISTANCE=MILES,
C
C       BL=BOD,     DO=DISSOLVED OXYGEN,     Q=FLOW RATE,     TEM=TEMPERATJRE
C       V=VELOCITY,    H=DEPTH,    DIS=DISTANCE     DOM=DOSSOLVED OXYGEN IN
C       THE INTERVAL,   SDO=SATURATED DISSOLVED OXYGEN,  PROB=THE PROBLEM
      DIMENSION BL(20),DO(20),Q(20),TEM(20),V(20),H(20),DIS(20),DOM(60)
     X ,SDO(20),PROB(20),TBL2(10),TD2(10),TA(10)
C
C
      READ(5,100)(PROB(I),I=1,20)
  100 FORMAT(20A4)
C
C
      WRITE(6,101)PROB
  101 FORMAT(1H1,20A4////)
      WRITE(6,30)
   30 FORMAT('     D.O. DEFICIT IS PLOTTED EVERY 2 MILES'/)
      READ(5,1)N2
          DO 600 L4=1,N2
C
      READ(5,1)M
C
C       THIS DO LOOP INCLUDES CALCULATIONS FOR EACH MINOR STREAM SYSTEM
C       L5=THE NUMBER OF THE SIMPLE STREAM SYSTEM
          DO 500 L5=1,M
C
C       N=THE NUMBER OF INFLUENTS TO THE STREAM + 1
      READ(5,1) N
    1  FORMAT(I5)
      READ(5,2) (BL(I),DO(I),Q(I),TEM(I),V(I),H(I),DIS(I),SDO(I),I=1,N)
    2 FORMAT(8F10.2)
C
          IF(L5.EQ.1) GO TO 300
          DO 301 J=1,N
C
          IF(BL(J).GT.0) GO TO 301
C
              BL(J)=TBL2(J)
              DO(J)=TD2(J)
              Q(J)=TA(J)
  301 CONTINUE
  300         NM1=N-1
C      WRITING OUT IMPORTANT ORIGINAL DATA ABOUT THE STAFTING RIVERCOURSE
      WRITE(6,102)DO(1),BL(1),Q(1),NM1,(DIS(I),I=1,NM1)
  102 FORMAT(' OF THE ORIGINAL STREAM '/' D.O.=',F5.2,' MG/L,  B.O.D.=',
     XF8.2,' MG/L,   Q=',
     XF8.2,' CFS '///,I5,' INTERVALS D.O. DEFICIT PLOTTED FROM EFFLUENT
     XDISCHARGE'/' IN MILES',/(F6.2),///)
              A=Q(1)
C
C   THIS DO LOOP INCLUDES THE CALCULATIONS FOR EACH INTERVAL OF THE STREAM
          DO 80 I=1,NM1
              IP1=I+1
C       SUMMING THE FLOWS OF ALL EFFLUENTS PLUS THE ORIGINAL SOURCE
              A=A+Q(IP1)
C       FINDING THE AVERAGE VELOCITY AND AVERAGE DEPTH FOR AN INCREMENT OF
C       THE STREAM OF RIVER
              VAV=(V(I)+V(IP1))/2.
              HAV=(H(I)+H(IP1))/2.
C       THE TEMPERATURE IS CALCULATED AT THE POINT WHERE AN EFFLJENT OR ANOTHER
C       STREAM MEETS THE ORIGINAL STREAM, AND THE TEMPERATURF IS PROPORTIONAL TO
C       THE FLOW RATES
              TEMP=(TEM(I)*(A-Q(IP1))+TEM(IP1)*Q(IP1))/A
C USING  THE AVERAGE VELOCITY,DEPTH , AND TEMPERATUREC WE CALCJLATE THE
C  COEFFICIENTS OF DEOXYGENATION AND REARATION (Y1 AND Y2)
      CALL VAL(Y1,Y2,VAV,HAV,TEMP)
              TIMF=(DIS(I)*5280.)/(VAV*86400.)
C  THE DISSOLVED OXYGEN AND BOD ARE CALCJLATED AT THE POINT OF DISCHARGE OF
C  A TRIBUTARY OR  A DISCHARGE INTO THE STREAM BY A COMMUNITY
          IF(I.GT.1) GO TO 15
              D1=(DO(1)*Q(1)+DO(2)*Q(2))/A
              BL1=(BL(1)*Q(1)+BL(2)*Q(2))/A
          GO TO 16
C       THE BOD AND THE DISSCLVED OXYGEN ARE IN PROPORTION TO THE FLOW RATES OF
C       THE RESPECTIVE STREAM OR EFFLUENT HAVING THAT BOD OR DISSOLVED OXYGEN
C       LEVEL
   15         D1=(D2*(A-Q(IP1))+DO(IP1)*Q(IP1))/A
              BL1=(BL2*(A-Q(IP1))+BL(IP1)*Q(IP1))/A
   16         C=DIS(I)*.5
              L=C
          IF(L.EQ.0) GO TO 19
          GO TO 21
   19         L=1
   21         LP1=L+1
              DOM(1)=SDO(I)-D1
C  THE BOD AND THE DISSOLVED ARE NOW CALCJLATED AT THE END OF THE INTERVAL
              BL2=BL1*EXP(-Y1*TIME)
              F=((Y1*BL1)/(Y2-Y1))*(EXP(-Y1*TIME)-EXP(-Y2*TIME))
              G=DOM(1)*EXP(-Y2*TIME)
              DOM(LP1)=F+G
              D2=SDO(IP1)-DOM(LP1)
          IF(D2.GE.0) GO TO 22
              DOM(LP1)=SDO(IP1)
              D2=0
C    IN THIS DO LOOP THE DISSOLVED OXYGEN IS CALCULATED AT EQUAL INCREMENTS
C     ALONG THE STREAM
   22     DO 81 J=2,L
              Y=J
              TIM2=Y*(TIME/C)
              F=((Y1*BL1)/(Y2-Y1))*(EXP(-Y1*TIM2)-EXP(-Y2*TIM2))
              G=DOM(1)*EXP(-Y2*TIM2)
              DOM(J)=F+G
              TES=SDO(I)-DOM(J)
          IF(TES.GE.0) GO TO 81
              DOM(J)=SDO(I)
```

```
   81 CONTINUE
C   THE CRITICAL DISSOLVED OXYGEN DEFFICIT AND THE CRITICAL TIME ARE
C     CALCULATED FOR THE INTERVAL
                TEST=DOM(1)*(Y2-Y1)/(Y1*BL1)
            IF(TEST.GT.1)GO TO 201
               F=ALOG((Y2/Y1)*(1.-DOM(1)*(Y2-Y1)/(Y1*BL1)))
               G=1./(Y2-Y1)
               TIMCR=G*F
               F=((Y1*BL1)/(Y2-Y1))*(EXP(-Y1*TIMCR)-EXP(-Y2*TIMCR))
               G=DOM(1)*EXP(-Y2*TIMCR)
               DOCR=F+G
C  THE CRITICAL DISTANCE FROM THE POINT OF DISCHARGE IS NOW CALCULATED WHERE
C      THE DISSOLVED OXYGEN DEFICIT IS A MAXIMUM
               DISTCR=VAV*TIMCR*86400.
  201 WRITE(6,104)DO(IP1),BL(IP1),Q(IP1)
  104 FORMAT(' THE EFFLUENT OR TRIBUTARY'//' D.O.=',F5.2,' MG/L,  B.O.D.
     X=',F8.2,' MG/L     Q=',F8.2,' CFS',//)
      WRITE(6,106)SDO(I)
  106 FORMAT(' THE SATURATED DISSOLVED OXYGEN VALUE AT THE POINT OF DISC
     XHARGE =',F5.3,' MG/L'//)
      WRITE(6,105)Y1,Y2
  105 FORMAT(' COEFFICIENT OF DEOXYGENATION, Y1=',F6.3,/'COEFFICIENT OF
     XREAERATION,Y2=',F7.3,//)
C     THE RESULTS ARE WRITTEN OUT FOR TWO DIFFERENT CASES--1) WHEN THE EXISTS
C       A DISSOLVED OXYGEN DEFICIT MAXIMUM WITHIN THE INTERVAL  AND 2) WHEN
C      THE MAXIMUM IS AT THE ENDD OF THE INTERVAL
            IF(TEST.GT.1)GO TO 17
           IF(TIMCR.GT.TIME) GO TO 17
           IF(TIMCR.LT.0) GO TO 17
      WRITE(6,50)DOM(1),DOM(LP1),BL1,BL2,DOCR,DISTCR,TIMCR
   50 FORMAT(' D.O. DEFICIT AT POINT OF DISCHARGE=',F5.2,' MG/L'/
     X ' D.O. DEFICIT AT END OF INTERVAL=',F5.2,' MG/L'/
     X ' B.O.D. AT POINT OF DISCHARGE=',F8.2,' MG/L'/
     X ' B.O.D. AT END OF INTERVAL=',F8.2,' MG/L'/
     X ' CRITICAL D.O. DEFICIT=',F5.2,' MG/L AT',F12.2,' FEET BELOW POIN
     XT OF DISCHARGE',F5.2,' DAYS DOWNSTREAM')
          GO TO 18
   17 WRITE(6,51) DOM(1),DOM(LP1),BL1,BL2
   51 FORMAT(' D.O. DEFICIT AT POINT OF DISCHARGE=',F5.2,' MG/L'/
     X ' D.O. DEFICIT AT END OF INTERVAL=',F5.2,' MG/L'/
     X ' B.O.D. AT POINT OF DISCHARGE=',F8.2,' MG/L'/
     X ' B.O.D. AT END OF INTERVAL=',F8.2,' MG/L')
   18 CALL GRAPH(DOM,L)
      WRITE(6,70)
   70 FORMAT(1H1)
   80 CONTINUE
      READ(5,1)L2
C
              TD2(L2)=D2
              TBL2(L2)=BL2
              TA(L2)=A
      WRITE(6,303)L5
  303 FORMAT(/////,' END OF SYSTEM NO.',I2,/////)
  500 CONTINUE
  60C CONTINUE
      STOP
      END
C
C
C
C
C
      SUBROUTINE VAL(Y1,Y2,VAV,HAV,TEMP)
C
C
      READ(5,1)ISW,ISQ,Y1,Y2
    1 FORMAT(2I5,2F10.2)
C      THE FIRST TEST VALUE IS A SWITCH WHICH WILL EQUAL 1 IF WE DO NOT NEED
C      TO CORRECT THE COEFFICIENTS VALUE TO OTHER THAN 20 DEGREES C
          IF(ISQ.EQ.1) GO TO 40
C     IF ISW EQUALS 1 , K2 HAS TO BE SUPPLIED ON A DATA CARD
          IF(ISW.EQ.1) GO TO 20
              Y2=(SQRT(.000081*VAV*3600.)/(HAV**1.5))*24.
   20         Y1=Y1*1.047**(TEMP-20)
              Y2=Y2*1.024**(TEMP-20)
   40 RETURN
      END
C
C
C
C
C
      SUBROUTINE GRAPH(DOM,L)
C      THIS SUBROUTINE IS USED ONLY FOR AIDING IN READING AND INTERPRETING
C      THE DATA
C
C     THIS SUBROUTINE WILL TAKE THE ARRAY OF DISSOLVED OXYGEN DEFICITS IN AN
C     INTERVAL AND WRITE THEM OUT  ALONG WITH A GRAPH REPRESENTING THESE VALUES
C
      DIMENSION DOM(L),G(60,110),E(60)
      DATA AST/1H*/, BLANK/1H /
          DO 2 I=1,L
C      MAGNIFYING THE SCALE
              E(I)=10.*DOM(I)
              K=E(I)
          IF(K.EQ.0) GO TO 5
          DO 3 N=1,K
              G(I,N)=AST
    3 CONTINUE
    5         KP1=K+1
          DO 4 J=KP1,110
              G(I,J)=BLANK
    4 CONTINUE
C      WRITING OUT THE DISSOLVED OXYGEN LEVEL FOLLOWED BY A REPRESENTATIVE
C      NUMBER OF ASTERICKS
      WRITE(6,6) DOM(I),(G(I,J),J=1,110)
    6 FORMAT(1H0,F5.2,' MG/L',110A1)
    2 CONTINUE
      RETURN
      END
```

APPENDIX C

ANNUAL MINIMUM FLOWS AT WATTS, KANSAS,

TAHLEQUAH, AND ELDON

ANNUAL MINIMUM FLOWS AT WATTS (07195500)

m	m/n+1	1-Day	7-Day	30-Day
1	.0527	147	151.2	241
2	.1053	118	145.1	217
3	.1579	92	113.6	184
4	.2106	88	101.3	175
5	.2632	88	91.3	118
6	.3158	86	91.3	116
7	.3683	68	90.7	115
8	.4211	67	80.0	113
9	.4737	60	75.0	105
10	.5264	52	71.8	104
11	.5790	51	64.0	100
12	.6316	46	55.0	97.4
13	.6843	41	53.0	73.5
14	.7369	39	46.7	69.9
15	.7895	33	34.4	56.5
16	.8421	30	32.7	44.2
17	.8948	10	13.8	20.9
18	.9474	10	11.1	14.9

ANNUAL MINIMUM FLOWS AT KANSAS (07196000)

m	m/n+1	1-Day	7-Day	30-Day
1	.0527	24	26.3	49.1
2	.1053	22	22.8	46.3
3	.1579	21	22.4	31.5
4	.2106	19	20.4	28.7
5	.2632	19	19.4	25.2
6	.3158	17	17.7	23.9
7	.3685	16	16.7	23.3
8	.4211	15	15.4	22.6
9	.4737	13	13.9	20.7
10	.5264	11	12.0	18.5
11	.5790	10	11.7	18.3
12	.6316	10	11.0	14.7
13	.6843	9.8	11.0	14.7
14	.7369	7	10.8	13.0
15	.7895	7	7.8	12.5
16	.8421	4	4.0	9.9
17	.8948	0.8	0.9	1.3
18	.9474	0.6	0.7	0.73

ANNUAL MINIMUM FLOWS AT TAHLEQUAH (07196500)

m	m/n+1	1-Day	7-Day	30-Day
1	.0257	206	221.0	380
2	.0513	183	188.1	309
3	.0770	182	187.7	255
4	.1026	174	186.6	242
5	.1282	152	159.4	224
6	.1539	144	154.3	215
7	.1795	141	149.0	214
8	.2052	132	147.1	207
9	.2308	122	140.0	175
10	.2565	113	126.0	162
11	.2821	109	116.2	158
12	.3077	107	115.0	155
13	.3333	103	113.2	154
14	.3590	102	110.0	152
15	.3847	100	109.8	142
16	.4103	92	108.1	138
17	.4359	91	100.8	130
18	.4616	89	94.1	125
19	.4872	87	93.5	124
20	.5129	87	92.4	124
21	.5385	83	86.4	121
22	.5641	79	84.4	121
23	.5898	78	83.6	121
24	.6154	78	82.3	117
25	.6411	77	81.0	115
26	.6667	72	73.1	113
27	.6923	69	72.1	105
28	.7180	61	65.1	93.5
29	.7436	58	60.6	84.5
30	.7693	51	51.7	78
31	.7949	38	40.7	62.5
32	.8206	38	39.7	49.6
33	.8462	30	33.0	45.7
34	.8718	6	32.0	45.6
35	.8975	3.6	6.6	10.5
36	.9231	1.1	2.4	7.1
37	.9488	1.0	1.4	5.4
38	.9744	0.1	0.1	3.2

ANNUAL MINIMUM FLOWS AT ELDON (07197000)

m	m/n+1	1-Day	7-Day	30-Day
1	.0385	42	46	77.8
2	.0770	41	43.3	64.9
3	.1154	37	38	55.5
4	.1539	36	37.4	50
5	.1923	31	33	44
6	.2308	30	31.1	40.3
7	.2693	27	28.4	39.7
8	.3077	23	24.8	35.3
9	.3462	21	22	33.4
10	.3847	19	20.4	32
11	.4231	18	20.4	31.4
12	.4616	15	17	28.4
13	.5000	13	14.4	25.3
14	.5385	12	13.6	18.1
15	.5770	11	11.1	17
16	.6154	10	11	16.9
17	.6539	9.3	9.6	12.8
18	.6923	8.5	9.3	12.6
19	.7308	7.8	8.7	12.4
20	.7693	6	6	11.7
21	.8077	4.4	5.1	10.2
22	.8462	2.6	2.7	6.6
23	.8847	2.2	2.4	3.2
24	.9231	2.2	2.4	3.1
25	.9616	1.8	1.8	2.0

2

VITA

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