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Hydraulics of river bed degradation, Willow Creek, Iowa

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Hydraulics of river bed degradation,

Willow Creek, Iowa

by

Heidargholi Massoudi

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CHAPTER I.

INTRODUCTION

Natural alluvial streams are seldom in a true state of equilibrium. The discharge, flow characteristics, sediment injection by tributaries, and the character of the sediment complex vary with time. For instance, if attention is focused on a specific reach of a natural stream, it will be noticed that the sediment seldom enters and leaves the reach at equal rates, nor does the stream bed remain exactly the same. This nonequilibrium condition is called "degradation" if the stream bed continues to lower as a result of more sediment leaving than entering, and "aggradation" if the stream bed rises in elevation as a result of less sediment leaving than entering.

A stream channel is considered stable if, over several controlling time periods, i.e., many years, the stream bed does not change its average bed elevation. This does not exclude the possibility of seasonal variations, or even slowly falling or rising short-term trends which are reversed only after several years. Under equilibrium (stable) conditions, the slope of the stream bed relates to its hydraulic, hydrological, and sedimentological characteristics. If one or more of the above components are disturbed naturally or by human interference, the stream will adjust, dynamically and geometrically, so that a new state of equilibrium is reached. Of great concern is the stream slope adjustment, which in turn will result in aggradation or degradation, depending on the steepening or flattening of the bed profile.

The construction of a large dam across an alluvial river causes vital changes in the regime of sediment transport all the way along the river. The reservoir will trap sediment and also change the flow regime. The sediment free water released from the reservoir has the ability to cause erosion of the bed downstream of the dam as bed load and suspended sediment are once again entrained, and degradation of the river bed occurs. Degradation continues as a function of discharge, time, and size distribution of the bed materials, until a new equilibrium is established.

Another typical example of human impact on the river regimen is channel improvement for land reclamation or flood control purposes. Until the early part of this century, the western Iowa streams were meandering natural rivers, which overflowed frequently and either prevented the conversion from prairie or pasture to crop land, or damaged agricultural crops planted in the flood plain. However, in the early part of this century, many of the streams were straightened and diked with the objective of achieving drainage and flood control on the adjacent riparian flood plain lands. Although the methods proved to be beneficial in terms of flood control and land reclamation, they often resulted in severe bed degradation. Due to the bed degradation, bridges crossing these streams have been endangered, both through scour around piers and piling and undermining of abutments. As a result, extensive repairs and even complete replacement have been required for many bridges in western Iowa. Channel widening results in a loss of farmland also.

Major factors causing degradation in western Iowa streams are often hydraulic or hydrologic in nature. Straightened channels resulted in steeper slopes, compared to the original stream slopes; consequently, the erosion of the bed and banks proceeded at a higher rate, due to the increase in velocity, boundary shear, and tractive force. Regardless of the increased slope effect, the dredged channels had much smoother perimeters than the meandering streams. This resulted in a lower surface friction factor (reduced roughness) and therefore added to the increased velocity component. As time progressed, the main channels deepened and they carried more and more of the flood discharge as overbank flows became less and less. This also resulted in more tractive force on the bed material. Another factor would be the land use change, from the prairie grasses originally covering the land surface, to the present agricultural crop pattern. However, compared to the other factors, the land use factor is considered to be a minor one, at least for the land use changes occurring during the present century.

In this present study, Willow Creek, a typical degrading channel in western Iowa, was selected for study. This stream has about 130 sq. mi. (337 km^2) of drainage area at the Missouri River bluff line. It was straightened during the period 1908-1920. About 41 miles (66 km) of the original stream length, above its mouth, was shortened to 29.6 miles (47.6 km). This represents a 28% reduction in length. The stream began degrading immediately after completion of construction, and it is still deepening. The critical degradation reach (most active and maximum channel depth below flood plain) has moved upstream with time. To control the degradation of this stream, and stabilize the channel grade, three major flume structures were built during the period 1968-1973, at a total cost of about one million dollars (according to the Iowa Department of Transportation; see Lohnes et al., 1980).

The objectives of this research study are:

1. Determine the extent of degradation of the Willow Creek channel.
2. Evaluate the physiographic characteristics of the basin, located in the loess-covered hills of western Iowa, which may contribute to the degradation problem.
3. Evaluate the hydrologic and hydraulic characteristics of the basin and its channel network, which may contribute to the degradation problem.
4. Determine cause and effect relationships, and estimate the future equilibrium bed profile, if no stabilization structures are introduced.

The estimated equilibrium bed profile of Willow Creek will be related to the hydrological, hydraulic, and physiographic characteristics of the stream. The dynamic processes studied for this stream might then be applied to other western Iowa streams, permitting additional evaluation of degradation problems and planning for positive grade stabilization facilities to control such problems. The financial picture is a necessary part of the total scene. A considerable expenditure of money would be required to solve all of the problems now being studied in 13 western Iowa counties (Lohnes et al., 1980), if grade stabilizing structures are used extensively. The results of this research may assist in determining the optimum number and location of these.

CHAPTER II.

LITERATURE REVIEW

Complexity of the Problem

The phenomenon of sediment transport, which involves the interaction of fluid with solid particles, presents one of the most complicated problems in hydraulic engineering. Raudkivi (1976) states, "even the motion of a single particle in a turbulent fluid can be described very inadequately and a complete analytical method of the sediment transport problem is a long way off yet." The problem involves the movement of a mixture of fluid and erodible materials within boundaries which deform through erosion and deposition. This subject has had to rely heavily on experimental studies. From these results, numerous sediment transport equations have been developed. Examples can be found in: Chang and Hill (1976), Einstein (1950), Meyer-Peter and Muller (1948), Toffaleti (1969) and Vanoni (1975). All are empirical in nature. Moreover, the experiments have been conducted with noncohesive materials such as sand, which fails to completely describe the composition of a natural stream's bed materials. The above mentioned problems become even more complex when erodible materials are partly or entirely cohesive.

Basic Mathematical Theory

Under equilibrium conditions, certain relationships exist among the primary variables, such as discharge, velocity, slope, sediment characteristics, and boundary geometry of the river. However, a temporal change of any of these variables will inevitably result in an adjustment of the other variables, particularly the slope adjustment (Gessler, 1971) factor, and degradation or aggradation occurs. By considering the equations for flow and sediment transport, the cause of degradation can be described and studied.

The Manning equation for open channel flow will be used in this study. Of course, other flow equations might be used (Chezy equation, for example; Chow, 1959). The Manning equation is given as

$$U = \frac{1.49}{n} R^{2/3} S^{1/2} \tag{1}$$

where

U = average flow velocity in the cross section of flow, fps

n = Manning roughness coefficient

A = cross sectional area, ft^2

R = hydraulic radius, ft

S = slope of energy line or streambed, for uniform flow, ft/ft

Many sediment transport equations have been developed. However, most such equations are written in the following general form

$$G_s = a\ (\tau_o - \tau_c)^b \tag{2}$$

where

G_s = rate of sediment flow per unit width

τ_o = γRS, bottom shear force, or unit tractive force (γ = specific weight) a = some dimensional constant

τ_c = critical shear force, also called critical unit tractive force

b = a power coefficient

For a given geometry and discharge, equation 1 can be used to determine the depth of flow and hydraulic radius. Then equation 2 can be used to calculate the value of sediment load. By the following algebraic manipulations, the effect of various parameters on sediment scour and transport can be described, and quantified. If the cross-section is rectangular and the flow depth approximates the hydraulic radius ($R \cong D$), then the Manning equation, combined with continuity, is given as

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2} \tag{3}$$

which, for the assumed conditions, becomes

$$Q = \frac{1.49}{n} (WD) (D)^{2/3} S^{1/2}$$

where

Q = discharge, cfs

W = channel width, ft

D = depth, ft

from which

$$D = \left[\frac{Qn}{1.49 W S^{1/2}}\right]^{3/5} \tag{4}$$

The bed shear stress is calculated by the following formula

$$\tau_o = \gamma RS \approx \gamma DS \tag{5}$$

which permits substituting D from equation 4 and obtaining

$$\tau_o = \gamma \left[\frac{Qn}{1.49\ W\ S^{1/2}} \right]^{3/5} S \propto S^{7/10} \tag{6}$$

Thus, an increase in slope, according to equation 6, leads to an increase in sediment transport capability. Further, if the sediment inflow into the stream reach remains constant, the overall result is an increase in scour potential, leading to additional degradation.

Another qualitative interpretation between discharge per unit of width, slope, sediment grain size, and sediment flow rate per unit width can be made by using the following general expression, as proposed by Lane (1955):

$$G_s\ d_{50} \sim q\ S \tag{7}$$

where

G_s = sediment flow rate per unit width

d_{50} = grain size diameter of which 50% by weight of the materials are finer

q = discharge flow rate per unit width

S = slope of energy line

Assuming a rectangular cross section, equation 7 can be written in the following form:

$$G_s\ d_{50} \sim U\ D\ S \tag{8}$$

Introduce the Manning equation, and change it stepwise, from

$$U = \frac{1.49}{n} R^{2/3} S^{1/2} \tag{1}$$

to

$$U = \frac{1.49}{n} \cdot \frac{\sqrt{g}}{\sqrt{g}} \cdot R^{1/2} \cdot R^{1/6} \cdot S^{1/2}$$

or

$$U = \frac{1.49}{n\sqrt{g}} R^{1/6} \sqrt{gRS}$$

But Chow (1959) states

$$\sqrt{gRS} = \sqrt{\frac{\tau_o}{\rho}} = \text{shear velocity}$$

therefore,

$$U = \frac{1.49}{n\sqrt{g}} R^{1/6} \sqrt{\frac{\tau_o}{\rho}} = \frac{1.49}{n\sqrt{g\rho}} R^{1/6} \tau_o^{1/2}$$

Substitute for U in equation 8, stepwise

$$G_s\, d_{50} \sim \frac{1.49}{n\sqrt{g\rho}} R^{1/6} \cdot \tau^{1/2} \cdot D \cdot S$$

$$G_s\, d_{50} \sim \frac{1.49}{n\sqrt{g\rho}} \cdot \frac{\gamma}{\gamma} \cdot R^{1/6} \cdot \tau^{1/2} \cdot D \cdot S$$

$$G_s\, d_{50} \sim \frac{1.49\, R^{1/6}}{n\, \gamma^{3/2}} (\gamma DS)\, \tau^{1/2}$$

Assuming R = constant, and $\tau = \gamma D S$, then

$$G_s\, d_{50} \sim \tau^{3/2} \tag{9}$$

Next, consider the Meyer-Peter and Muller (1948) sediment transport formula:

$$G_s = K\,(\tau_o - \tau_c)^{3/2} \qquad (10)$$

where, K is a dimensional constant and the other parameters are the same as defined earlier.

From equation 7, it is evident, for a given discharge, that an increase in slope results in an increase in the sediment transport variable, G_s. In natural streams, the change in slope might be due to channel straightening, or slow lowering of a fixed point like the erosion of an overfall or nick point. Another case would be the discharge of a stream into a lake or another river of larger size. If the water level in the lake or larger stream drops, it will result in over-steepening of the stream's energy gradient, and degradation of the inflow stream can occur. For example, the drop in water stage of the Missouri River might result in degradation of western Iowa streams at their confluence with the major border river.

The size of stream bed material determines its resistance to movement. According to equation 7, a reduction in grain size of the sediment will result in an increase in G_s, the sediment transport rate. The smaller the grain diameter, the smaller the critical tractive force, τ_c, and thus equation 2 indicates that a larger volume of sediment can be transported.

Equation 2 provides a measure of the sediment carrying capacity of the stream. If the sediment inflow rate into a given stream reach is reduced, the difference between capacity (as indicated by equation 2) and inflow would have to come from the river bed, resulting in degradation. A good example of this potential is the construction of a dam and reservoir, which can lead to degradation downstream of the dam as clear water is released.

The unit tractive force exerted on the stream bed, τ_o, is directly related to the flow depth ($\tau_o = \gamma DS$). An increase in discharge will increase the depth of flow, and consequently will result in an increase in the unit tractive force. Since the sediment transport capacity, G_s, is determined by the difference between τ_o and τ_c, an increase in discharge leads to an increase in sediment carrying capacity of the stream. An increase in discharge that can occur from a given amount of rainfall could be attributed to a change in land use patterns in a stream's watershed, such as in western Iowa, where the prairie and timber lands were converted to agricultural crops in the late nineteenth century. Another case might be the artificial diversion of water into an existing river, which will result in a sudden increase in discharge. These activities are typical of those which have occurred in Western Iowa.

Degradation Studies

Most efforts to investigate the degradation phenomenon have been related to the degradation occurring below dams. All laboratory experiments done to date are somehow related to this specific subject. Research about degradation in alluvial rivers is in general very limited.

However, there are a number of research papers regarding the actual degradation below dams (Gama, 1957; Komura and Masuta, 1963). These papers are either the result of flume studies or analytical methods, and compared to the field data.

Degradation below dams

When a dam and reservoir are constructed on a natural stream, and there is a high trap efficiency of the reservoir, the majority of the sediment being transported by the river will settle in the reservoir. The water released from the dam then becomes almost sediment free, especially during normal flow conditions. The clear water released has the ability to pick up sediment particles from the stream bed and banks downstream of the dam. As a result of this erosion and sediment transport, degradation of the river bed occurs. Degradation continues as a function of discharge from the dam and time, until a new equilibrium is established. The rate of river bed degradation is rather rapid at the beginning but becomes small as the new stable profile is reached (gradual reduction in the stream slope).

The composition of the bed material plays an important role in the rate of degradation occurring below the dam. The finer fraction of bed materials are removed from the bed surface by sorting, and are transported downstream. Consequently, the river bed in the upstream reaches gradually becomes "armored" with coarser bed material, which reduces or prevents further lowering of the bed. It has been observed (Komura and Simons, 1967) after closure of a dam for example, that the median grain size, d_{50}, becomes approximately equal to the range of

d_{80} through d_{95} sizes that existed before closure of the dam. Here, the d_{50}, d_{80}, and d_{95} variables are the grain sizes of the bed material of which 50%, 80%, and 95% by weight are finer respectively.

The degradation of a river below a dam will certainly have a great impact on hydraulic structures located downstream of the dam. The prediction of a possible change of the river bed has a great economic importance, since the erosion downstream from dams may undermine expensive structures. Costly measures such as preservation of existing bridge piers, bank protection works, hydroelectric facilities, etc., may become necessary. The effect of degradation, of course, does not have to be exclusively detrimental, but will under certain circumstances (hydraulic power generation, for example) lead to a significant increase in head across the dam, as has happened at numerous locations in the United States. To use this potential, the engineer should be able to predict the future rate of degradation and to design and locate the level of the hydraulic turbines and draft tubes accordingly (see Linsley and Franzini, 1979).

Bed degradation may extend a long distance downstream of a structure. For instance, degradation was traced about 185 miles (300 km) downstream of Sariyar Dam in Turkey (Simons and Senturk, 1977). Yassin (1979) evaluated the degradation of the Nile River, downstream of the Aswan High Dam. Degradation of the Missouri River, downstream of Gavin's Point dam, has extended over 200 miles (322 km); there the Platte River serves as a sediment input to stabilize the river (see Shen, 1971a and 1971b). This degradation occurred during the period 1955-1980.

Laboratory flume studies

One approach frequently used is to simplify the problem by studying selected variables in laboratory experiments. In such an investigation, the overall problem is evaluated in a piecemeal, sequential manner. The controlled variables include the rate of sediment injection, the size and character of sediment, the flow rate, or one of the several other factors which are found in alluvial streams.

There have been many flume studies made regarding degradation. In a degradation study in the laboratory, usually a rectangular flume with smooth walls is selected. To simulate the bed material, a fairly uniform sand is chosen, and discharge can be regulated and measured by some type of flow meter. Sand is supplied at the upstream end of the flume, usually by means of an elevator and, after being transported to the downstream end, is retained in a sediment trap. The difference between the quantity of sand fed upstream, and the quantity trapped in the bucket will determine the amount transported from the bed. The elevations of water surface and bed are checked during the experiment to detect the pattern and extent of degradation.

Perhaps one of the classical laboratory studies for degradation was conducted by Newton (1951). The experiment was conducted at the St. Anthony Falls Hydraulic Laboratory, University of Minnesota. Newton's work has been the basis for further degradation studies. Since then a number of researchers have tried to check their degradation models against Newton's data. The plan for the experimental program was to make a number of test runs under controlled conditions.

A test run consisted of first, the establishment of an equilibrium condition for a selected flow of water, Q, and a selected rate of sediment transportation, G; and second, the readjustment of the sand bed under the same flow, Q, but with no sediment being fed into the channel at the upstream end. This procedure was considered to simulate the occurrences in a natural stream, where the construction of a dam prevents the normal passage of sediment and results in readjustment of the stream bed downstream.

Newton conducted the experiment with uniform Ottawa sand with median grain size of 0.69 mm. The laboratory flume was 12 inches wide, 30 feet long, and 2 feet deep. The quantity of sand fed at the upstream end was controlled by a sand elevator. To investigate the pattern of degradation, the author used four different combinations of discharge, Q, and feed rate of sediment, G (test #1, Q = .2 cfs, G = 0.011 lbs of sand per second; test #2, Q = 0.4, G = 0.0226; test #3 Q = 0.2, G = 0.0226; and test #4, Q = 0.3, G = 0.0226). The degradation tests were preceded by a run of sufficient duration with constant rate of sediment feed to establish equilibrium conditions. The equilibrium was considered to have been met when the average rate of sediment discharged into the downstream traps became equal to the calculated rate of sediment input from the feed elevator. The sediment feed was then abruptly stopped and the bed allowed to degrade. The pattern of degradation was measured periodically by suddenly raising the tail water and temporarily arresting the sediment motion. As an indication of the degradation pattern, the results of test #3 are shown in Figure 1.

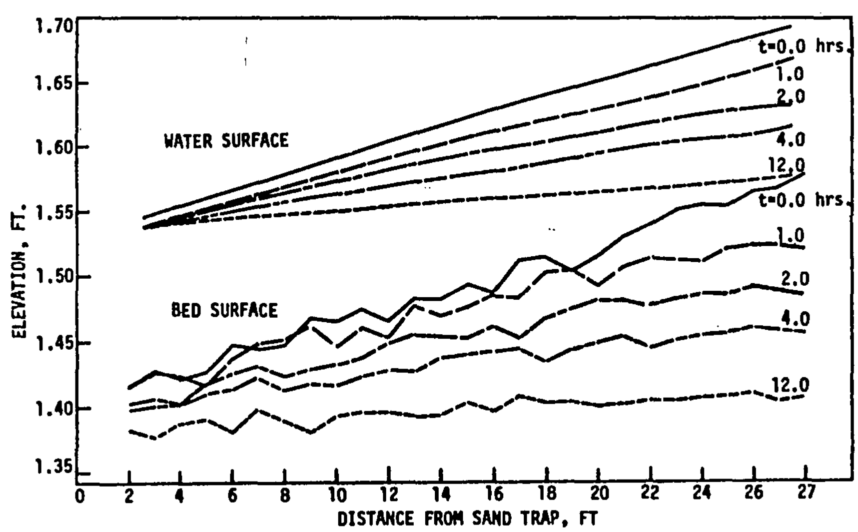


Figure 1. Typical plot of observed profile data of test No. 3 (after Newton, 1951)
(For metric conversion, multiply ft. by 0.305 for m.)

As Newton has pointed out, "the data may be considered a basis for a general qualitative relation, but should not be accepted as precise from a quantitative viewpoint." As it is observed from the figure, as the time increases, the rate of degradation decreases and equilibrium at the end of degradation tends to be reached asymptotically with time.

Similar experiments were conducted by Suryanarayana (1969) at Colorado State University. His experimental flume was 60 feet long, 2 feet wide, and 2.5 feet deep. The author conducted the experiments with three different sands; two of them Ottawa sand with median sizes of 0.33 and 0.45 mm, and the third one with median size of 0.90 mm. The procedures were similar to those of Newton. However, Suryanarayana showed that the observed values of sediment transport, G, and excess shear stress, $\tau - \tau_c$, followed the general form of equation 2. By plotting G and $\tau - \tau_c$ on log-log paper, the author fitted a straight line to the data and computed the values of "a" and "b" in equation 2 for three different sands used in the experiment. The values for the coefficients varied considerably from one sand to another. For example, the value of "a" varied from 1.6 to 145 . Such a wide range in the value of "a" reveals that the sediment transport equations are derived from and suited to a particular set of conditions. Thus, many of the equations developed experimentally and available today may not be applicable to another situation.

Although the experimental approach has been the most fruitful approach in establishing the mechanics of sediment transportation, it has not, however, been a wholly satisfactory procedure, principally because of the limitations of small laboratory flumes and because mutual interference effects have necessarily been ignored. For instance, the width change and meandering existing in natural streams both are constrained by the effect of the walls of laboratory flumes.

Analytical methods

Background A variety of analytical methods for the prediction of degradation downstream of dams are available today (for example, Aksoy, 1970; Hale et al., 1970; Hwang, 1975; Komura and Simons, 1967; and Tinney, 1962). Common to almost all methods is a set of equations describing the fluid flow, the sediment transport, and the continuity equation for sediment load. These equations have to be solved simultaneously.

Tinney (1962) presented a trial and error method for degradation. The author compared his analytical solution to Newton's data (1951), and obtained a good agreement between the observed data of Newton's experiment, and his computed bed profile.

Komura and Simons (1967) presented a mathematical procedure for predicting river bed degradation below dams, and incorporated the effect of armoring in their lengthy formulation. The authors paid much attention to final equilibrium but little to the transient phases.

Hwang (1975) presented a method for degradation prediction in sand bed channels. The author applied his method to the reaches of natural channel below the Milburn Dam on Middle Loup River in Nebraska, and indicated that the stream bed would reach equilibrium in 16 years. According to Hwang, the maximum predicted scour at the dam site would be about 16 feet (5 m).

Application of analytical methods The selection of appropriate transport formula in degradation studies relies in general on empirical reasoning and past experiences that a hydraulic engineer has about the river reach under study. For instance, Komura and Simons (1967) based their analysis on a sediment transport formula derived by Kalinske (1942). Hale et al. (1970) adopted Bagnold's formula (1966), Tinney (1962) evaluated the process of channel degradation by using the bed load transport equations of Meyer-Peter and Muller (1948), and Hwang (1975) incorporated the statistically based total sediment load equation presented by Shen (1971a, 1971b). In most cases, no reason was given why a specific formula was adopted.

Peters and Bowler (1967) in a review of the work by Komura and Simons (1967) showed that the selection of different transport formulas would result in different degradation prediction, all other variables held the same. The author simulated two of the experiments carried out by Newton (1951). Applying nine different transport formulas, the author indicated that each formula resulted in different degradation prediction, and compared to observed data, all formulas underestimated the degradation.

In spite of the variability in the procedures and the equation results, the end result of such analytical methods is a prediction equation which relates the rate of channel degradation to certain pertinent variables. The form of the equation and the variables involved differ from one case to the other. The purpose here is not to discuss various differential equations for degradation; however, the general form of such equations can be indicated in the following form.

$$\frac{\partial Z}{\partial t} = f\left[\text{sediment characteristics, flow conditions, stream geometry, etc.}\right] \qquad (11)$$

where

$\frac{\partial Z}{\partial t}$ is the rate of degradation, ft or m per unit of time

Method of computation The method of computation for degradation is rather similar in all methods. For example, Tinney (1962) has given the following procedure for his method.

The degradation rate is computed by starting with known bed and water surface configuration. The rate of degradation is first computed from the differential equation at regular positions along the stream, and the new position of the bed at the end of a time interval Δt is computed from:

$$Z_{t + \Delta t} = Z_t + \left(\frac{\partial Z}{\partial t}\right)_t \Delta t \qquad (12)$$

where Z_t and $Z_{t + \Delta t}$ are bed elevations at time t and $t + \Delta t$, and $\left(\frac{\partial Z}{\partial t}\right)_t$ is the rate of degradation of time t obtained from the related differential equation. A smooth curve is fitted to the newly computed elevations, and the new water surface corresponding to this smoothed bed configuration is then computed from conventional backwater equations. The process is repeated until $\tau = \tau_c$ everywhere along the bed.

Limitations of analytical methods Analytical methods therefore present an approach for predicting degradation which is possible after the introduction of a large number of more or less arbitrary assumptions. For instance, in analytical formulations, the stream cross-section is assumed to be rectangular; moreover, the widening of the stream, the variation of discharge, and the cohesion of the bed material are usually ignored. For example, in applying their method to a natural stream reach below a dam in Nebraska, Komura and Simons (1967) made the following assumptions.

"1. sediment transport is completely arrested by the dam.
2. the river banks are not erodible
3. seasonal variations in discharge and temperature of water do not occur
4. sediment injections by tributaries do not occur
5. meandering and growth of vegetation do not occur."

However, such assumptions are not applicable to the streams of western Iowa. For example, the width of Willow Creek at the present time is about four times the width when the ditch was first constructed. Also, the variation of discharge cannot be ignored.

Due to the serious limitations of the available laboratory and analytical methods, it is unlikely that a reliable result can be obtained when applying them to these degrading western Iowa streams. A specific procedure will be developed so that the equilibrium profile of the Willow Creek can be estimated. This procedure must take into account the variations in discharge, as well as stream straightening, widening and deepening.

Statistical Degradation Models

Geomorphic approach

Another approach to study the extent and rate of degradation could be based on the stream bed elevations for previous years. This procedure is usually favored by geomorphologists by which they establish empirical relations between the altitude of the stream bed and other controlling variables such as: discharge, drainage area, slope, etc. It is emphasized, however, that such expressions are empirical in nature, and their application beyond the conditions under which they have been developed is risky and might lead to erroneous results.

If sufficient data during different time periods are available, they can be plotted on log-log or semi-log paper to examine the progression of degradation. The extrapolation of the fitted curve might, in some instances, provide an approximate estimate of the future degradation. However, due to the lack of data for previous years, extrapolation based on the limited data available might be quite misleading.

A few geomorphic results will be reviewed for use in this study. J.T. Hack (1957), by using the data of several streams in Virginia and Maryland, demonstrated that the longitudinal profile of a stream in equilibrium is approximated by the general equation

$$B = C - k \ln (L) \tag{13}$$

where

B = the altitude of the reach

L = the distance along the stream measured from the head of of the stream

k = the slope of the line on semi-log plot

C = a constant

Hack's equation was used by Lohnes et al. (1980) in estimating the equilibrium profile of Willow Creek.

Ruhe and Daniels (1965) plotted the stream bed elevations versus time for three locations along the main channel of the Willow Creek, and derived a regression equation describing a relation between the channel depth and time in the following form.

$$D = 1.8 + 20.9 \log T \tag{14}$$

where

D = depth of channel below flood plain, feet

T = time in years, since T_o, where T_o = initial year

These empirical equations are discussed further in a subsequent chapter.

Degradation in cohesive soil

In most upland streams, a considerable portion of bed materials is in the clay and silt size range. The cohesion of the soil plays an important role regarding the erodibility of such materials. Unfortunately, however, the most that can be found in the literature is the rough estimation of the critical condition for soil erosion and for a specific soil sample tested in the laboratory or field. Efforts have been made (Dunn, 1959; Flaxman, 1963; Moore and Masch, 1962; Partheniades and Paaswell, 1970; Smerdon and Beasley, 1961) to correlate the critical tractive force, τ_c, of cohesive soil to different soil properties. The critical condition for erosion in alluvial streams with cohesive materials will be discussed in detail in Chapter VI, where an erosion model is developed and the equilibrium profile of the Willow Creek is estimated.

Summary

Sediment transport has been studied in the laboratory, in small streams and canals, and in large rivers. Therefore, the subject has been of interest to mathematicians, physicists, hydraulic engineers, civil engineers, and agricultural engineers. Publications pertaining to river hydraulics and large scale phenomena include those of Bagnold (1966), Chang (1979), Gama (1957), Gessler (1970, 1971), Graf (1971), Aksoy (1970), Leliavsky (1955), and Shen (1971a, 1971b, 1979). Special studies of major rivers below dams include those by Aksoy (1970), Chang et al. (1967), Gama (1957), Garde et al. (1977), Hale et al. (1970), Komura and Simons (1967), and Peters and Bowler (1967). These are of value in the current study because Willow Creek discharges into the Boyer River, then to the Missouri River. The latter is undergoing degradation due to the upstream dams and reservoirs. Small area studies, such as are of interest to agricultural engineers, include Iowa studies by Campbell et al. (1972), Dirks (1981), and Lohnes et al. (1980). Small channel erosion phenomena have, in a similar sense, been studied by Holland and Pickup (1976), Hughes (1980), Hey (1978), and Pickup and Moresby (1975). Canal scour, under more constant discharge conditions, was reported earliest by Fortier and Scobey (1926), Kramer (1935), then by Lane (1955) and Vanoni (1975). Newer concepts in river hydraulics have recently been published, such as stream power (Chang and Hill, 1977), additional morphology emphasis (Pickup and Moresby, 1975; Pickup and Riager, 1979), and new computer simulation techniques (Croley, 1977; Pickup, 1976; and Shen, 1979). All of these studies have assisted in advancing the level of knowledge concerning channel scour and sediment transport. However, in such a complicated subject, there remain many unknowns.

CHAPTER III.

CHARACTERISTICS OF THE ORIGINAL WILLOW CREEK AND WILLOW DRAINAGE DITCH

Location

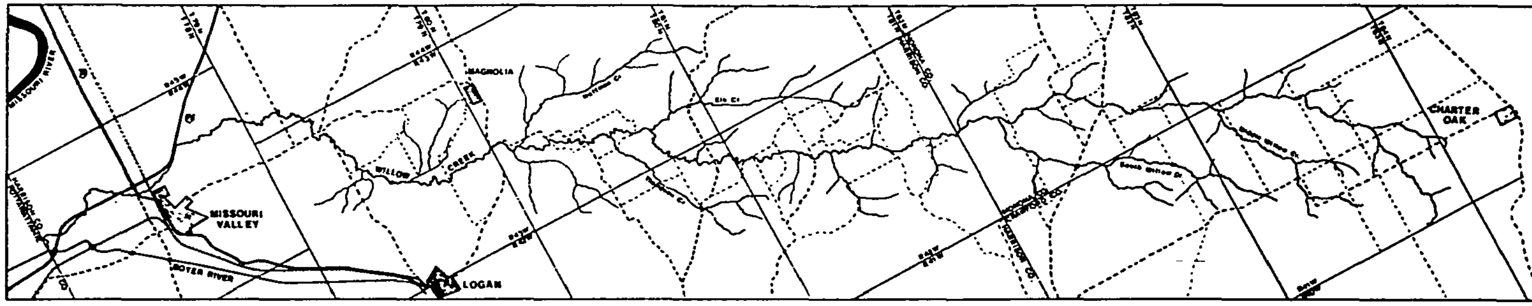
The Willow Creek originates two miles (3.2 km) south of Charter Oak, Crawford County, Iowa, and it flows through Monona and Harrison Counties until it reaches its confluence with the Boyer River. The stream has a long, narrow basin and its watershed mostly consists of loess soils. About 30 miles (48 km) of the downstream portion of the stream is a man-made ditch which was constructed in the early part of this century for flood control and land reclamation purposes.

Figure 2 illustrates the plan view of original Willow Creek and its drainage ditch.

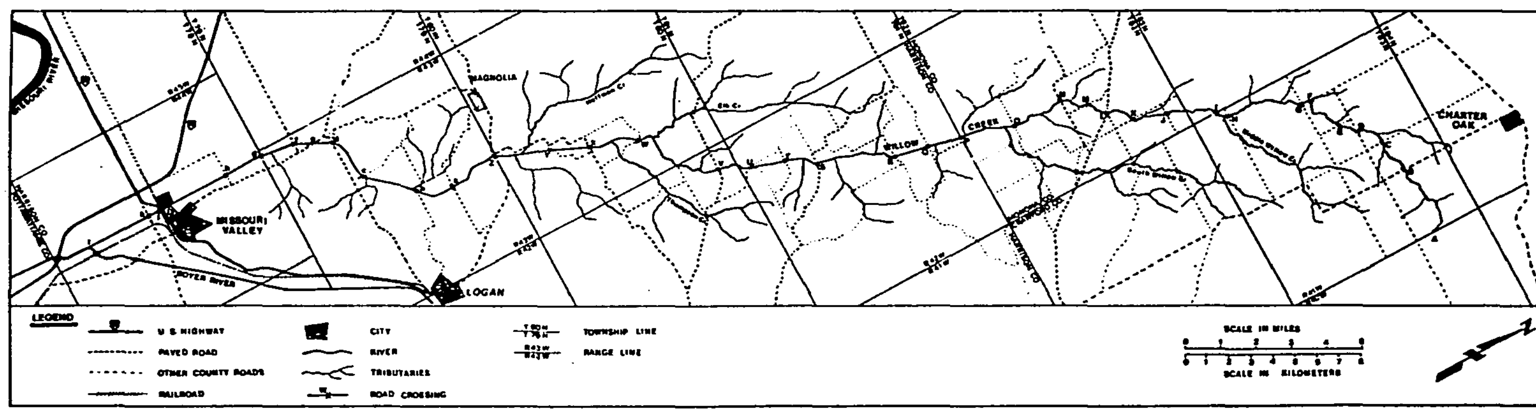
Since the time of construction, the ditch has widened and deepened extensively along its entire length. The causes and extent of these changes will be reported in subsequent sections.

Original Willow Creek

Prior to straightening, the Willow Creek was a meandering stream, and its total length from the drainage divide to its confluence with the Boyer River was 57.4 miles (92.4 km). The geometric cross-section of the creek was not hydraulically sufficient to pass the flood discharges. Consequently, the creek commonly flooded its valley to a depth of several feet, and the valley was considered unfit for cultivation. In the early



a. Original Willow Creek, prior to straightening



b. Willow drainage ditch, after straightening

Figure 2. Plan view of original Willow Creek and Willow drainage ditch

part of this century, stream dredges and dredging technology made extensive straightening possible, and Willow Creek became a candidate.

Cross section geometry of the stream

Little information is available on the cross section geometry of the Willow Creek prior to 1900. However, during the initial drainage district surveys, conducted from 1916 through 1919, some information on the cross section of the stream was obtained. During the detailed survey of the Willow Drainage Ditch (more about the ditch later), approximate cross sections were obtained for several locations, where the original Willow Creek and Willow Ditch crossed each other. The author obtained this information from the old surveying field books kept in the County Engineer's Office, Harrison County, Iowa.

The drainage district information includes: 1) the elevation of the right and left banks, and the centerline of the stream bed, 2) the width of the stream at the top and at the water surface level, and 3) the depth of water at the time of the survey. Using this information, 14 cross sections were constructed. Figure 3 shows the locations of the sections as they were surveyed in 1916 to 1919. Dimensions of the sections are given in Table 1.

Unfortunately, similar information for the downstream part of the stream could not be found. However, the examination of the longitudinal profile of the ditch revealed that the geometry of the creek in the downstream reach was very similar to the upstream reaches. For instance, the depth of the stream below the flood plain remained almost the same value

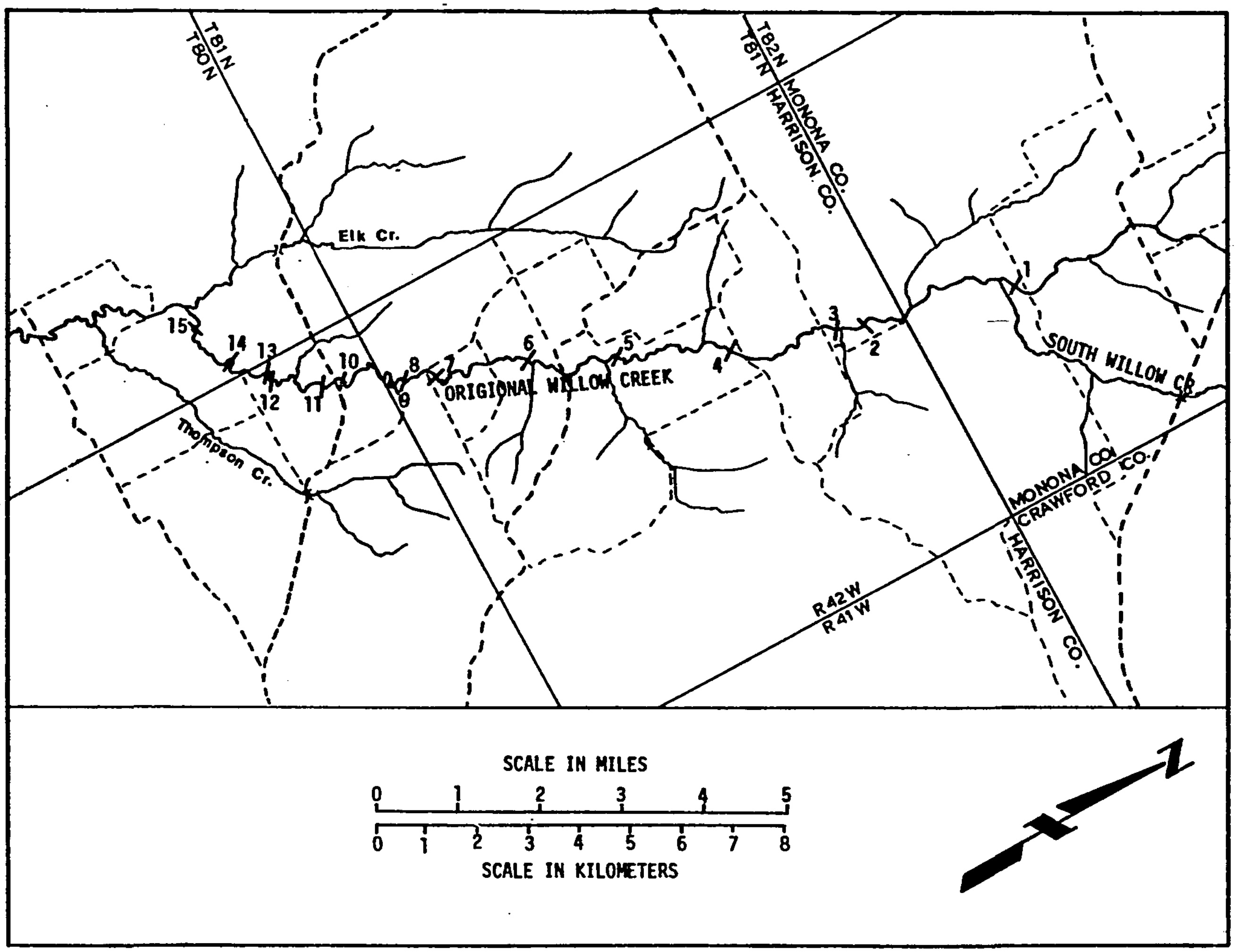


Figure 3. Location map of the cross sections in Table 1

as in the upper reaches. This will suggest that, for practical purposes, the data presented in Table 1 could be used as average values for the entire length of the creek.

Table 1. Cross sections of the original Willow Creek in 1916-1919

	Distance from the drainage divide, mi		Geometric characteristics of the stream (feet)				
Section No.	Along the original Willow Creek	Along the Willow Ditch	Top width	Depth	Width 2' above the stream bed[a]	Stream bed elevation (NGVD)	Stream bank elevation
1	17.11	16.95	50	13	-	1185.2	1198.2
2	19.47	18.86	53	12.5	-	1168.8	1181.3
3	19.19	19.22	61	12	-	1167	1179
4	21.69	20.63	53	11	19	1154.8	1165.5
5	23.37	22.02	49	12	-	1143.7	1155.7
6	24.90	23.30	62	12	14.5	1134.8	1146.8
7	26.24	24.31	69	11	-	1127.7	1138.7
8	26.87	24.68	48	12.5	17	1122.8	1135.3
9	28.15	25.47	74	11.5	18	1116.3	1127.8
10	28.53	25.73	49	11	19	1114.8	1125.8
11	29.52	26.45	58	13	10	1106.8	1119.8
12	30.00	26.77	40	13	-	1104.3	1117.3
13	30.43	27.01	58	11	-	1103.3	1114.3
14	31.25	27.63	56	14	-	1097.3	1111.3

[a]The depth of water was about 2 feet during the survey (0.6 m).

Longitudinal profile of the stream

The profile of the original Willow Creek was reconstructed by plotting the elevation of the channel bottom at the points the drainage ditch crossed the original channel of the old stream. Figure 4 illustrates the profile of the original stream as it appeared in the period 1916-1919. Although the gradient of the stream varied along the main channel, in

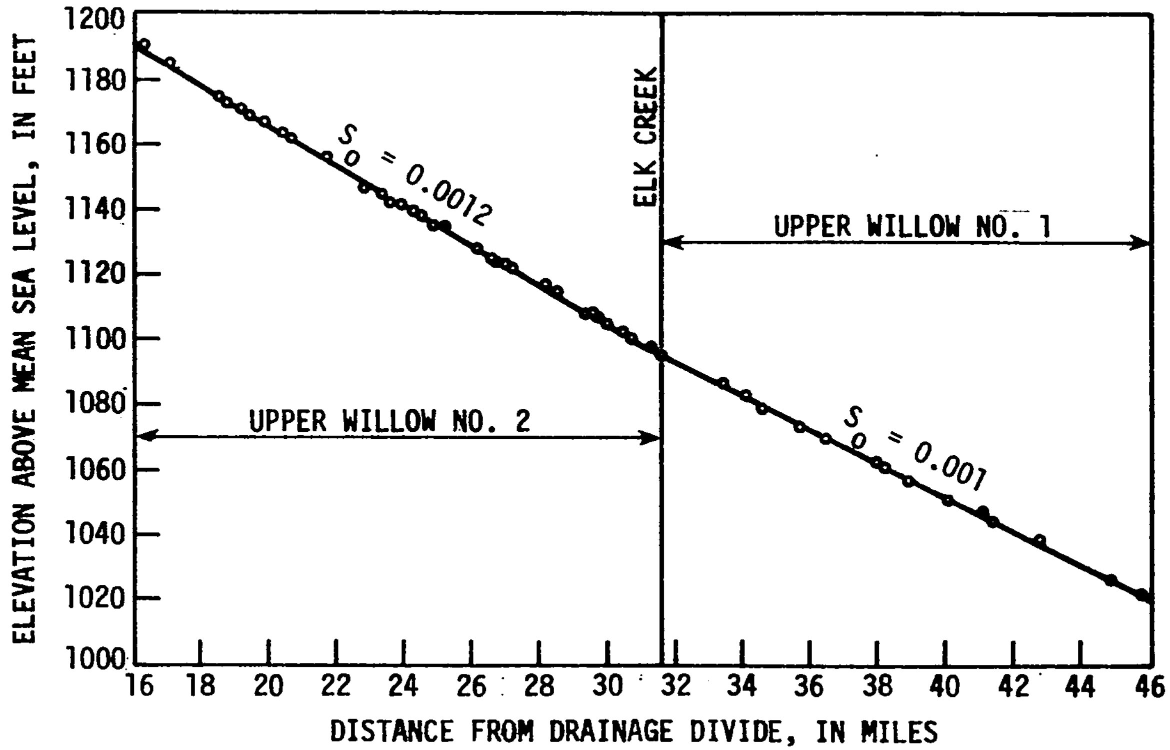


Figure 4. Longitudinal profile of the original Willow Creek in Upper Willow Nos. 1 and 2 drainage districts

terms of average values, two main segments could be identified. Between miles 16 and 31.5, the average gradient or bed slope was about 0.0012 (6.34 ft/mile or 1.2 meters/km). However, it decreased to 0.001 (5.28 ft/mile or 1 meter/km) for the reach between miles 31.5 and 46. The flattening of the gradient in the lower reach appears to be related to drainage area size, because of the entry of the larger tributaries, such as Elk Creek, and Thompson Creek, into the river system.

History of the Willow Ditch

To reduce the flooding hazard by Willow Creek, a drainage ditch was constructed from Monona County to the Boyer Drainage Ditch. The construction of the Willow Ditch, with a total length of 29.6 miles (47.6 km), was accomplished in three different stages.

Harrison and Pottawattamie Drainage Ditch

The first section of the work was started in June 1906 and completed late in 1908. This part started from the Boyer Drainage Ditch in Section 28, Township 78N, Range 44W; and extended 7.72 miles (12.42 km) toward the north to Section 19, Township 79N, Range 44W, Calhoun Township. This section of the improvement was entirely located in Harrison County, north of Pottawattamie County, Iowa. However, this section of the Ditch was included in the combined Harrison and Pottawattamie Drainage District, and was abbreviated as H & P Drainage District, so this part of the ditch will be referred as "H & P" in the text.

The geometric dimensions of the H & P section were: an 18 ft. (5.5 m) bottom, 1:1 side slopes, and a depth of 15 ft. (4.6m) from berm top to the bottom of the channel. The top of the berm was approximately 3 ft. (1m) above the natural ground level. The selected channel bed slopes were 0.03% (1.58 ft/mile or 0.3 meters/km) for the first 3.79 miles (6.10 km) downstream and 0.05% (2.65 ft/mile or 0.5 meters/km) for the next 3.93 miles (6.3 km) upstream (Missouri River flood plain).

Upper Willow Drainage Ditch No. 1

Petitioned in September 1915, construction in this reach started in October 1916, and was completed in October 1919. The total length of the ditch was 10.25 miles (16.50 km). For simplicity, this part of the ditch will be abbreviated as UW #1 hereafter.

The geometric dimensions of UW #1 were: a 12 ft.(3.7m) bottom width, 1:1 side slopes and 15 ft. (4.6m) berms. For 9.93 miles (15.98km), a uniform slope equal to 0.145% (7.66 ft/mile or 1.45 m/km) was adopted; however, a slope equal to 0.136% (7.18 ft/mile or 1.36 m/km) was selected for the rest of the ditch. The UW #1 was entirely constructed in cut, and the average cut was approximately 15 ft. (4.6m), varying between 11-19 ft.(3.4-5.8m). A strip of land 140 feet wide was allocated as right-of-way for UW #1 (43 meters).

Besides the main channel, 19 laterals were established ranging in length from 360-1,800 ft.(110-550 m), and with a base width from 4-10 ft. (1.2-3m); 1:1 side slopes, and 5-10 ft. (1.5-3 m) berms. The right-of-way required for laterals was from 30-90 ft.(9-27m). The total construction cost for UW #1 and its tributaries was about $84,000 (1919).

Upper Willow Drainage Ditch No. 2

This section was the continuation of UW #1 with the total length equal to 11.63 miles (18.7km). The UW #2 was petitioned in October 1917, and it was constructed between April 1919 and June 1920. The bottom width of the ditch was 12 ft. (3.7m) for the first 3.38 miles (5.44km), 10 ft. (3.1m) for the next 7.75 miles (12.47km), and 8 ft. (2.4m) for the last 0.68 miles (1.10km). The side slopes were 1:1 in all the above three segments with a berm width of 15 ft. (4.6m). The depth of the cut averaged 11 ft. (3.4m) throughout the UW #2. In contrast to the uniform slope in UW #1, the slope varied along the ditch in this district, and the design slopes for different reaches are listed in Table 2.

Table 2. Design channel slopes in Upper Willow No. 2 in 1920

	Section intervals		Slope		
	(miles)	(km)	percent	m/km	feet/mile
upstream direction	1.25	2.0	0.167	1.67	8.80
	1.38	2.2	0.2	2.0	10.56
	1.90	3.1	0.16	1.60	8.45
	3.18	5.1	0.138	1.38	7.28
	1.01	1.6	0.167	1.67	8.80
	1.69	2.7	0.133	1.33	7.04
	1.22	2.0	0.194	1.94	10.24
	11.63 mi	18.7 km			

Besides the main channel, 8 laterals were established as part of the improvement. Their average length was from 330 to 1,450 ft. (100-442m) with a 4-6 ft. (1.2-1.8m) base width, 6-15 ft. (1.8-4.6m) berms, 1:1 side slopes, and a 40-90 ft (12-27m) right-of-way. The total construction cost of the improvement was about $90,000 in 1920.

Figure 5 is the location map for the above three mentioned drainage ditches. The design longitudinal profiles of UW #1 and 2 are illustrated in Figure 6.

Some of the materials presented in this chapter will be used in Chapters V and VI to discuss the cause of degradation, and to estimate the equilibrium profile of the stream.

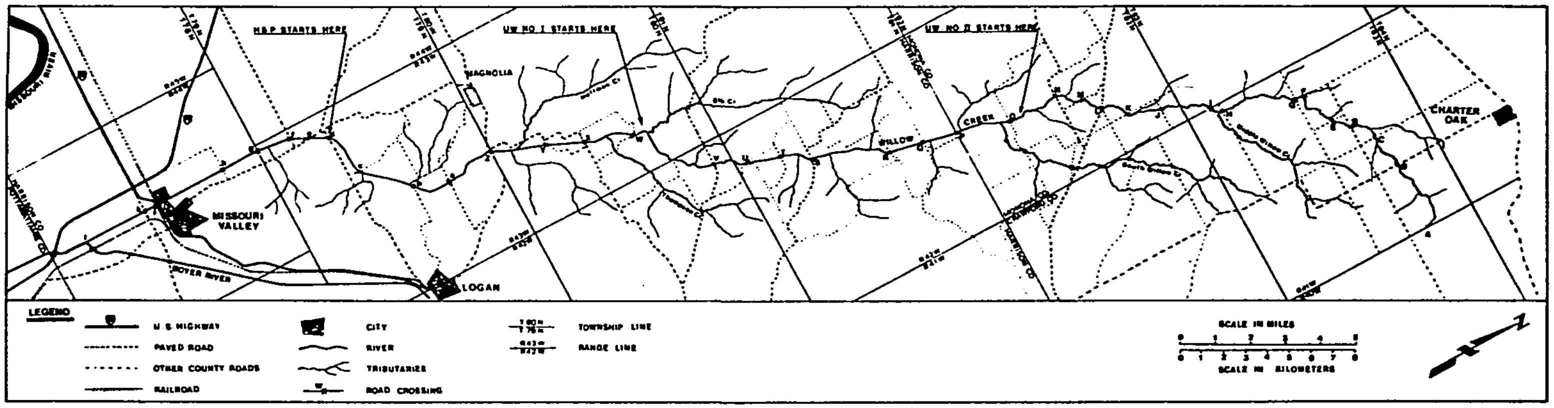


Figure 5. Location map of H&P, Upper Willow No. 1, and Upper Willow No. 2 drainage districts

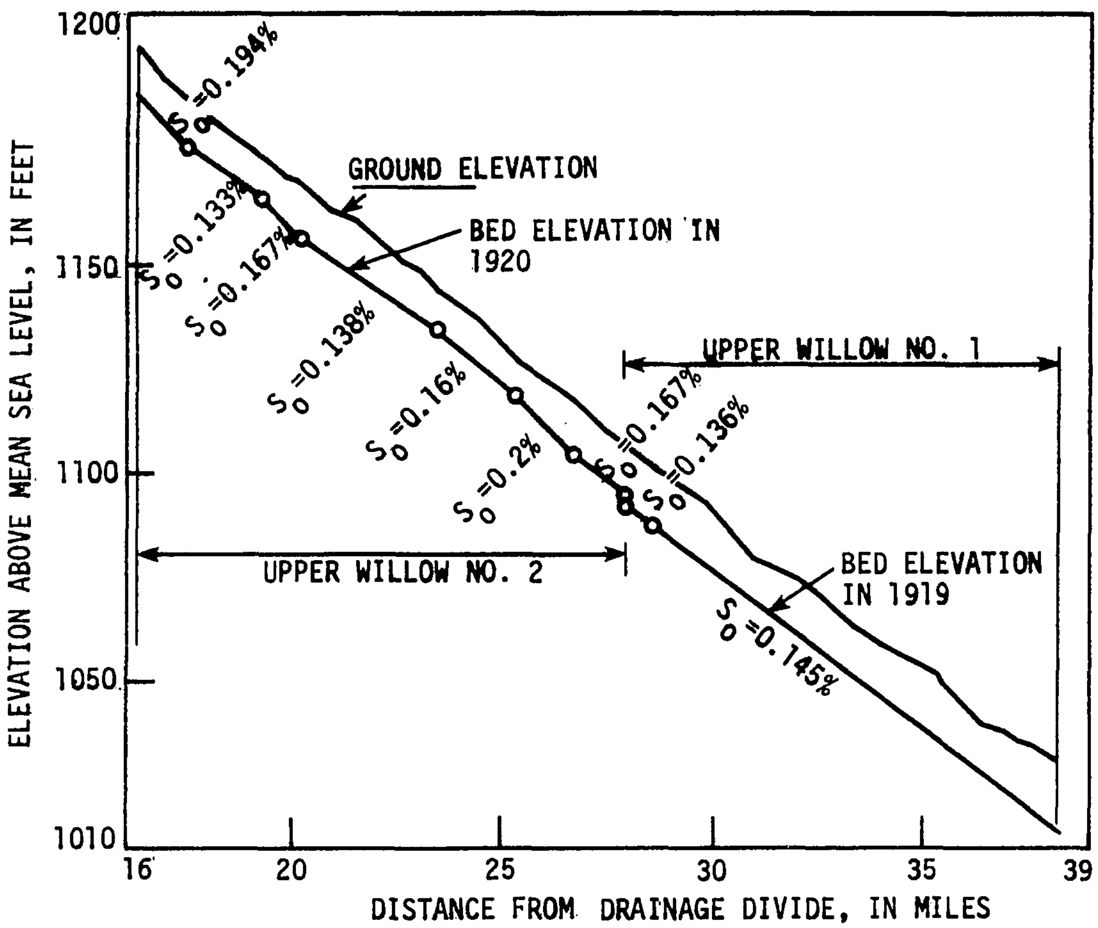


Figure 6. Design longitudinal profile of the Willow drainage ditch in upper Willow Nos. 1 and 2 drainage districts

CHAPTER IV.

HYDROLOGY

The Willow Creek drainage basin represents a typical watershed in western Iowa, having a long but narrow shape. The shape of the watershed of Willow Creek can very well be approximated by a rectangle whose length from the drainage divide to the Missouri River bluff line is 38.3 miles (61.6 km), and its average width is 3.40 miles (~5.5 km). The maximum and minimum width of the basin are 6 miles (9.6 km) and 2.65 miles (4.3 km), respectively. The main channel flow direction is from the northeast towards the southwest. Figure 7 illustrates the watershed boundaries of the Willow Creek and its tributaries. This figure was prepared by the author, and more detailed information is in Appendix A. Due to the above mentioned special configuration of the watershed, it is expected that the variation of discharge with distance follows a linear function in different reaches. As stated earlier, the drainage area is 130 square miles (337 sq. km).

Dominant Discharge

For a degrading channel, one of the important variables contributing to the bed and bank erosion is the discharge carried by the stream. In irrigation channels, the discharge is either constant or it varies in a rather narrow range. It hardly needs stating that all natural rivers are subjected to flow that is not steady, but varies from case to case because of wide differences in the hydrology of individual catchments.

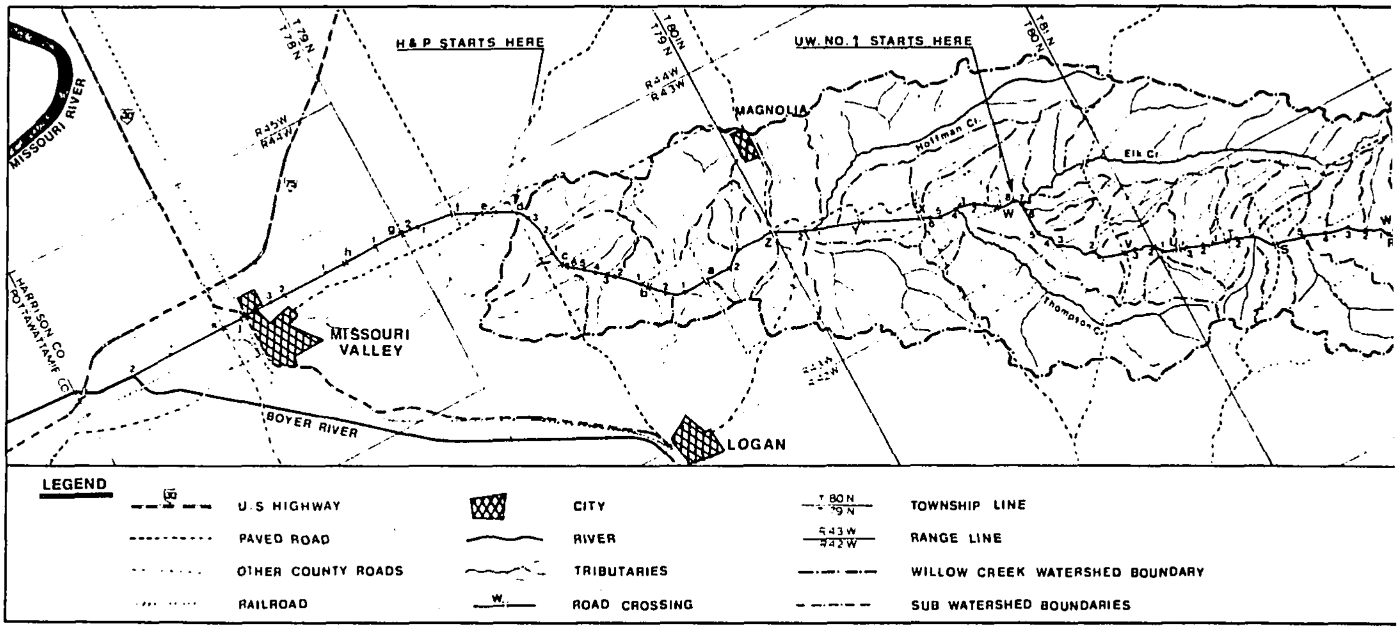
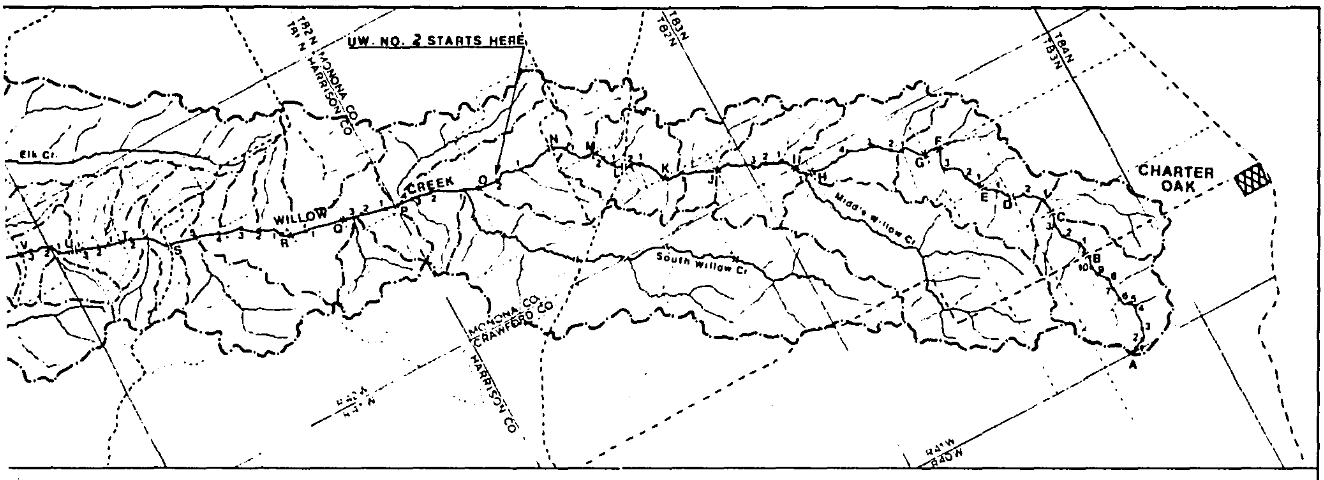


Figure 7. Drainage basin boundaries of Willow Creek and its tributaries



D BOUNDARY

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Not only does river flow vary with the season of the year, it varies from one year to the next, and superimposed on the broad seasonal changes are floods. Even in a given geographic region, a small tributary will have a different runoff pattern than the main river basin.

For a given set of conditions, for the bed and bank materials to be removed and transported out of the reach, a certain time period is required. Since the major flood peaks last for rather short periods (measured in hours), they should not be considered as the controlling discharge for degradation. Instead, more frequent discharges are considered much more important. Due to the wide range of variations of flow in natural streams, the analysis of individual flows with regard to degradation is a rather tedious and unnecessary job. The question for consideration is what discharge, within the wide range of flows that actually occurs, "dominates" the channel formation and associated degradation. This discharge which is called the "dominant discharge" has not been firmly defined, and the concept in geomorphic processes is somewhat tenuous (Benson and Thomas, 1966).

Leopold, Wolman and Miller (1964) say, "the most meaningful discharge for any discussion of channel morphology is that which forms or maintains the channel - the effective discharge can often be approximated by bankfull discharge. In many rivers, the bankfull discharge is one that has a recurrence interval of about 1.5 years."

Carlson (1965) has concluded that the dominant discharge which controls meander wave length is a range of flows, possibly falling-stage flows, between the mean of the monthly maximum discharges and mean annual discharge.

In an analysis of factors controlling bank erosion, Wolman and Miller (1960) summarized earlier studies that included observations on the discharges that were most responsible for shaping a channel: "In an analysis of factors controlling bank erosion, Wolman has shown that lateral cutting of the cohesive channel bank of a small stream in Maryland occurs mostly during the winter months, when flows of a size which occurs eight to ten times per year attack previously wetted banks."

Benson and Thomas (1966) state that, "the forming and maintenance of channel cross sections and the movement of meanders must be accompanied by the movement of sediment. It seems therefore, logical to suggest a definition of dominant discharge as the discharge that over a long time period transports the most sediment." The authors combined the sediment-rating and discharge-duration information to compute the dominant discharge.

Ackers (1972), in a review of the subject, indicated that some researchers have examined the problem purely on the basis of statistical correlation by asking the question, "Considering a given geometric feature, what basis of defining discharge minimizes the scatter?", or in other words, "What steady discharge would yield a channel configuration and geometry most closely approximating to that actually resulting from the naturally occurring flows?" The author, in a model experiment, confirmed that the bankfull discharge was the dominant condition for shaping the meanders.

Selection of the dominant discharge for Willow Creek is not easy to determine at this point. The stream is not in equilibrium, and due to the severe degradation, the water never overflows the banks in the downstream reaches. In other words, the bankfull discharge for this stream at many locations is even larger than the 50-year flood magnitude. The dominant discharge will be selected based on a flood discharge-frequency study, and will be presented in the following section.

Hydrologic Data of Willow Creek

Discharge records

A gaging station was established by the U.S. Army Corps of Engineers District, Omaha, Nebraska in 1946. However, no records are available prior to 1948. The station consisted of a chain weight gage with a graphic recorder. The location of the station was on the left bank of the Willow Creek at the old county bridge between section 19 and 20 of Calhoun Township, Harrison County, Iowa. The chain weight gage was replaced by a wire weight gage in 1951. The old bridge was raised in 1961 and the gage well was moved to the right bank in October 1961 (from the record in Willow Creek file at the Army Engineers District, Omaha, Nebraska). The new county bridge was completed in 1967 (point d in Figure 7 - this point will be referred as the Logan gaging station hereafter).

Since 1948 until September 1971, the station was operated by the U.S. Army Corps of Engineers. No recording was made from October 1971 until September 1972. The station was again put into operation in October 1972 by the U.S. Geological Survey, but recording was completely discontinued in October 1975.

Unfortunately, the daily discharges are available for 1967-1971, and 1972-1975 water years only. Prior to 1967, no daily discharge was ever published by the U.S. Army Corps of Engineers. The available data are in terms of gage readings only. Table 3 shows the mean, maximum, and minimum monthly discharge values at the gaging station for the above mentioned period. For the period indicated, the maximum value was 2230 cfs (63.1 cms), which occurred on May 18, 1974, and the minimum was zero, which occurred on February 17, 1967.

Flood peaks

The annual series of flood peaks as determined by the U.S. Army Corps of Engineers and U.S. Geological Survey, are illustrated in Table 4. The maximum flood occurred on June 15, 1957, and the peak as estimated by the U.S. Army was about 21,000 cfs (594 cms). The values given in this table will be used later in this chapter in a flood frequency analysis of the stream. The basic statistical equation for determining the magnitude and frequency of flood discharges at a gaging station (Chow, 1964; Lara, 1974) is:

$$X_t = M + KS \tag{15}$$

where X_t is the magnitude, M is the mean, S is the standard deviation and K is the slope of the regression line and a function of skew and the recurrence interval, t.

Table 3. Mean, maximum and minimum monthly discharge of Willow Creek at Logan gaging station (cfs)[a]

		Water Year								
		1967	1968	1969	1970	1971	1972	1973	1974	1975
	Mean	8.91	11.8	32.2	15.2	11.7	-	31.5	74.1	19.7
OCT	Max	12	18	521	29	61	-	155	314	32
	Min	6.7	7.8	5.9	12	3.4	-	23	49	15
	Mean	10.4	13.5	10.9	15.5	10.7	-	56	58.1	21.7
NOV	Max	13	22	13	23	21	-	121	94	27
	Min	8.6	9.3	7.2	13	6.3	-	42	50	20
	Mean	12.8	9.20	9.97	11.5	7.19	-	28.2	43.1	21.3
DEC	Max	18.0	17	13	14	25	-	150	58	31
	Min	2.0	7.6	8.1	8.3	3.2	-	12	33	13
	Mean	8.36	7.81	9.23	6.09	1.56	-	50.1	41.8	24.7
JAN	Max	10	10	9.4	8.1	3.1	-	150	50	35
	Min	6.8	6.3	9.0	4.6	.6	-	30	40	18
	Mean	8.41	6.89	12	23.1	154	-	64.9	50.8	22.4
FEB	Max	38	9.8	60	48	985	-	472	103	37
	Min	0	5.6	9	5.0	1.4	-	25	29	20
	Mean	14.3	13.3	160	49.5	202	-	140	47.4	73.8
MAR	Max	27	23	1450	298	1290	-	633	72	296
	Min	8.8	9.8	11	23	22	-	73	35	30
	Mean	11.8	12.5	53.4	22.2	14.7	-	108	47.2	73
APR	Max	30	24	80	34	25	-	311	114	393
	Min	8.3	9.1	38	16	12	-	77	33	48
	Mean	16.1	8.83	38.5	18.9	14.9	-	92.6	279	76.7
MAY	Max	107	13	53	101	35	-	183	2230	441
	Min	4.9	7.6	24	10	10	-	65	39	36

[a]For metric units, multiply all values by 0.0283 to give cubic meters per second (cms).

Table 3. Continued

		Water Year								
		1967	1968	1969	1970	1971	1972	1973	1974	1975
	Mean	108	24.9	38.1	19.1	51.6	-	78.7	61.3	57.1
JUN	Max	276	318	157	120	561	-	170	137	150
	Min	16	4.3	18	5.9	8.0	-	48	41	34
	Mean	23	26.1	54.6	5.36	36.2	-	80.4	30.2	31.7
JUL	Max	124	245	325	7.9	398	-	714	40	75
	Min	13	7.1	17	4.0	7.1	-	27	22	21
	Mean	8.6	30.1	23	4.63	3.93	-	33.4	52.7	52.4
AUG	Max	14	376	124	22	8.3	-	57	322	790
	Min	5.6	7.1	8.8	2.9	1.3	-	22	17	18
	Mean	9.81	16.3	21.2	6.59	2.65	-	67.9	18.7	19.9
SEP	Max	35	110	177	23	4.0	-	919	44	50
	Min	7.2	6.0	11	3	1.2	-	20	15	13

Table 4. Annual series of flood peaks of Willow Creek at Logan gaging station[a]

Year	Month	Stage Height (ft)	Discharge (cfs)
1948	March	10.65	1,870
1949	September	18.68	9,340
1950	August	19.45	10,200
1951	August	21.15	13,600
1952	June	14.10	4,780
1953	May	9.10	1,185
1954	June	14.12	4,150
1955	March	7.31	280
1956	July	10.63	1,090
1957	June	24.9	21,000
1958	July	9.32	660
1959	May	14.15	3,200
1960	May	15.76	4,800
1961	August	9.22	630
1962	March	17.60	7,200
1963	August	10.35	1,000
1964	April	8.20	370
1965	February	15.20	4,150
1966	February	7.98	310
1967	June	8.49	402
1968	August	10.52	1,060
1969	March	11.45	1,450
1970	March	9.2	620
1971	February	14.73	2,220
1972			
1973	July	13.11	2,100
1974	May	18.78	7,970
1975	August	14.46	3,220

[a]Data from files of the U.S. Geological Survey and U.S. Army Corps of Engineers; multiply stage values, in feet, by 0.305 for meters, and discharge, cfs, by 0.0283 for cms.

Flood Frequency Analysis of Willow Creek

As stated earlier, the extent of degradation of the stream bed is dependent on the flow characteristics of the stream as well as the sediment and bed characteristics. It was also pointed out that a representative value of discharge, called the "dominant discharge," is a representative flow which can be used to determine the final equilibrium geometry of the river. Direct determination of such a value is not possible; however, the floods of different recurrence intervals could be estimated, and then the dominant discharge could be selected on the basis of flood frequency analysis.

Not only is the magnitude of flow (the dominant discharge) at a given point desired, but also its variation along the channel length. Thus, it is necessary that the functional relationship between the flood discharge for a given recurrence interval and the distance along the channel be established.

Although the findings of this research are based on the Willow Creek drainage basin, the overall purpose is to expand the results to other similar watersheds in western Iowa. For this reason, the flood frequency analysis is carried out in two parts. First, the Willow Creek will be considered as an ungaged stream and a regional flood frequency analysis will be made so that the results can be used in similar situations. Secondly, the observed flood data in Table 4 will be analyzed, and finally, a comparison will be made between the observed and computed values.

Regional flood frequency methods

It is a time-consuming practice to collect and publish flood discharge data for any specified watershed. Only a sample of the total number of streams in a region can be gaged because of cost and budget limitations. So, it seems logical to evaluate the flow characteristics of Willow Creek as it were an ungaged stream, and then compare the results with observed data. In this way, the flow characteristics of similar but ungaged watersheds might be evaluated with a greater degree of confidence.

INRC method (Iowa Natural Resources Council) The hydrologists have regionalized the streams and have provided some guidelines for flood predictions in ungaged watersheds. The estimating equations are derived from regional relationships between floods of specific return periods (frequency of recurrence) and selected basin characteristics. Such a guideline for Iowa streams was presented by Lara (1973). The state of Iowa was divided into two main hydrologic regions, and different regression models were developed for each region. Figure 8, from the above reference, illustrates the geographic locations of these regions. As it is seen, Willow Creek is located in Hydrologic Region No. I. Therefore, the appropriate models for this region will be examined. The following models were suggested by Lara (1973).

Model I: $$Q_t = C_t\ (D_a)^{X_t} \tag{16}$$

Model II: $$Q_t = C_t\ (D_a)^{X_t}\ (S*)^{Y_t} \tag{17}$$

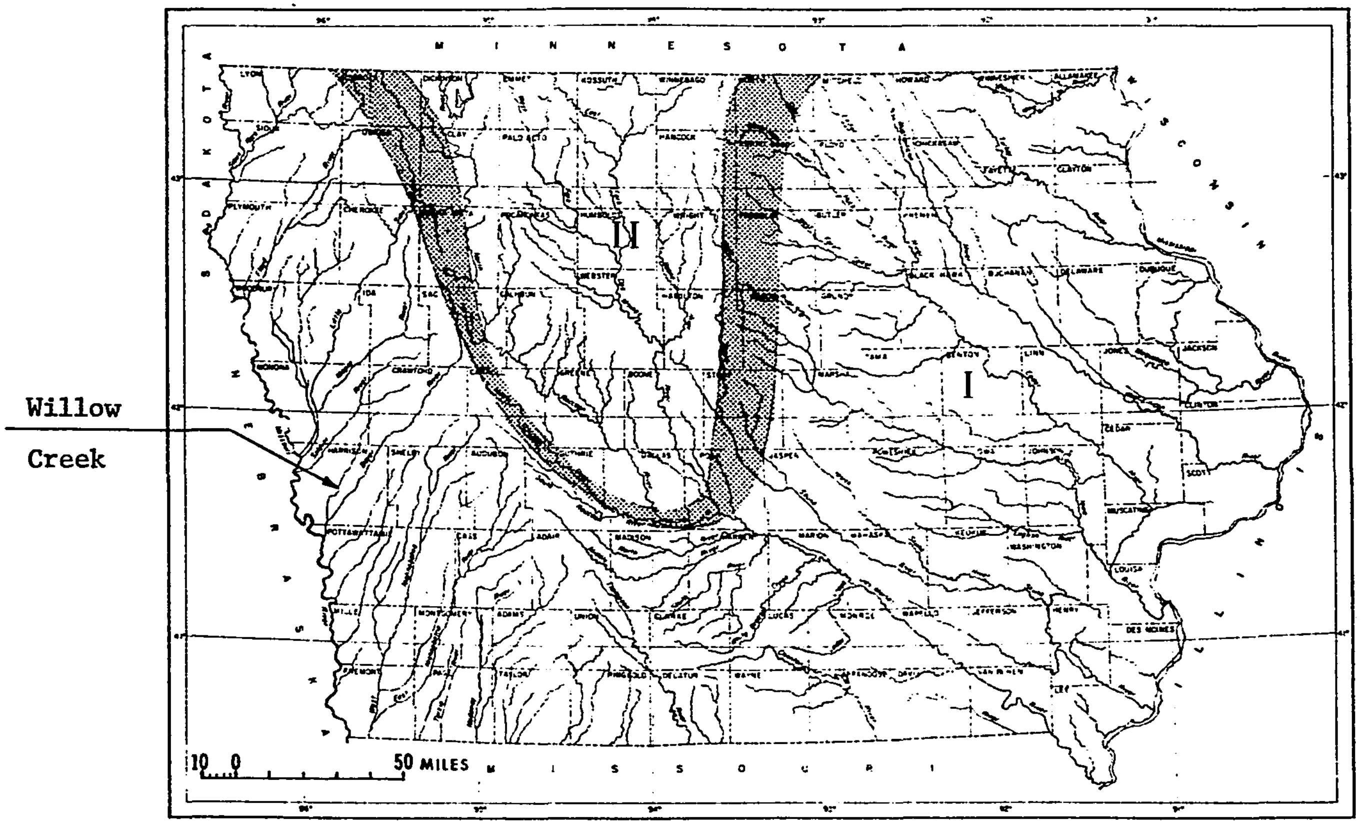


Figure 8. Hydrologic regions of Iowa (after Lara, 1973)

where

D_a = drainage area, in sq mi (D_a = 640 A; A in acres)

S* = main channel slope, in feet per mile, determined from elevations at points 10 percent and 85 percent of the distance along the channel from any selected point to the drainage divide, in square miles

Q_t = discharge for a t-year recurrence interval, in cubic feet per second, cfs

C_t, X_t, and Y_t are regression equation coefficients for given recurrence interval t.

These models relate the floods of the given recurrence intervals to the basin parameters.

Since the evaluation of S* along the Willow Creek is possible by using the USGS 1:24,000 topographic maps, the preferred model is Equation 17.

The values of C_t, X_t, and Y_t as presented in Bulletin No. 11 of Iowa Natural Resources Council (Lara, 1973) are reported in Table 5. Also, equation 17 indicates that the two variables, that is, D_a and S*, can be evaluated at different points along a stream.

Table 5. Regression coefficients for equation 17

t, years	C_t	X_t	Y_t
2	31.2	0.701	0.490
5	82.5	0.651	0.445
10	143	0.618	0.410
25	262	0.579	0.369
50	394	0.551	0.335
100	571	0.524	0.305

Drainage area D_a In order to establish a fairly accurate relationship between the drainage area and the distance from the drainage divide, the author calculated the drainage areas at many locations along the main channel of Willow Creek, as well as along its tributaries. Aerial photographs of 1965 and 1971 with the scale of 1:24,000 were used in this survey. A stereoscope was used to locate the drainage boundaries. These boundaries were then transferred to the 1:24,000 topographic maps. The areas enclosed by the boundaries were measured by planimeter and the results for several locations were checked against the values given by USGS (Larimer, 1974), and they agreed very well. The drainage boundaries of Willow Creek and its tributaries were shown in Figure 7.

More detailed information on the values of the drainage areas can be found in Appendix A.

Figure 9 illustrates the functional relationship between the drainage area and the distance from the drainage divide. The figure also shows the location of the main tributaries and the two drainage districts, that is, Upper Willow Nos. 1 and 2. As it is observed, the relationship between the drainage area and the distance can be represented by a step function whose components are straight lines in most of the intervals. However, for one interval, a parabolic function was fitted to the data. According to the figure, eight intervals could be identified. Mathematical expressions were fitted to the data and the results are summarized in Table 6.

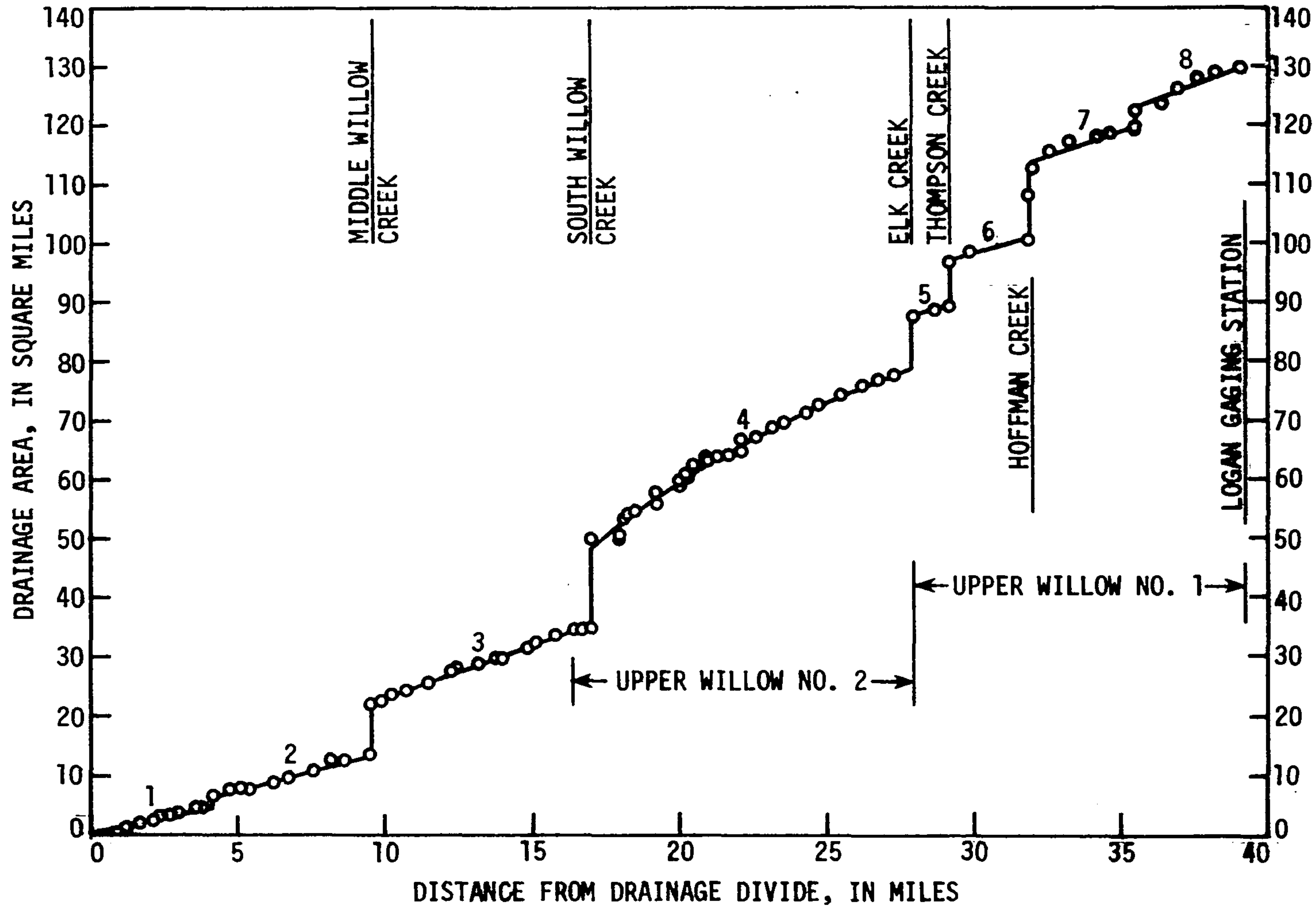


Figure 9. Relationship between drainage area and distance from drainage divide

Table 6. Mathematical expressions for Figure 9

Section number in downstream direction	Interval (miles)	Drainage area function, D_a sq miles	
1	$0 < X < 4.2$	$D_a = 1.056\ X$	(18)
2	$4.2 < X < 9.6$	$D_a = 1.34\ X + 0.82$	(19)
3	$9.6 < X < 17.1$	$D_a = 1.81\ X + 4.69$	(20)
4	$17.1 < X < 27.8$	$D_a = 24.23 + \sqrt{219.33\ X - 3173.67}$	(21)
5	$27.8 < X < 29.2$	$D_a = 1.08\ X + 57.4$	(22)
6	$29.2 < X < 31.9$	$D_a = 1.16\ X + 63.4$	(23)
7	$31.9 < X < 35.5$	$D_a = 1.67\ X + 60.83$	(24)
8	$35.5 < X < 38.25$	$D_a = 2.54\ X + 32.96$	(25)

The variable X in the above expressions is in miles. Should the distance be desired in km, the results must be multiplied by 1.609. The above mathematical models will be used later to establish the discharge-distance relationships.

Slope By definition, S* for any given point, X miles downstream of the drainage divide, is expressed as follows.

$$S^* = \frac{\text{Elevation at 15\% X - Elevation at 90\% X}}{\text{75\% X}} \tag{26}$$

where, X is to be expressed in miles.

Equation 26 was used to evaluate the S* at many selected points on the stream, with results shown in Figure 10. The elevations were determined by interpolation of 1:24,000 scale topographic maps. The distance X

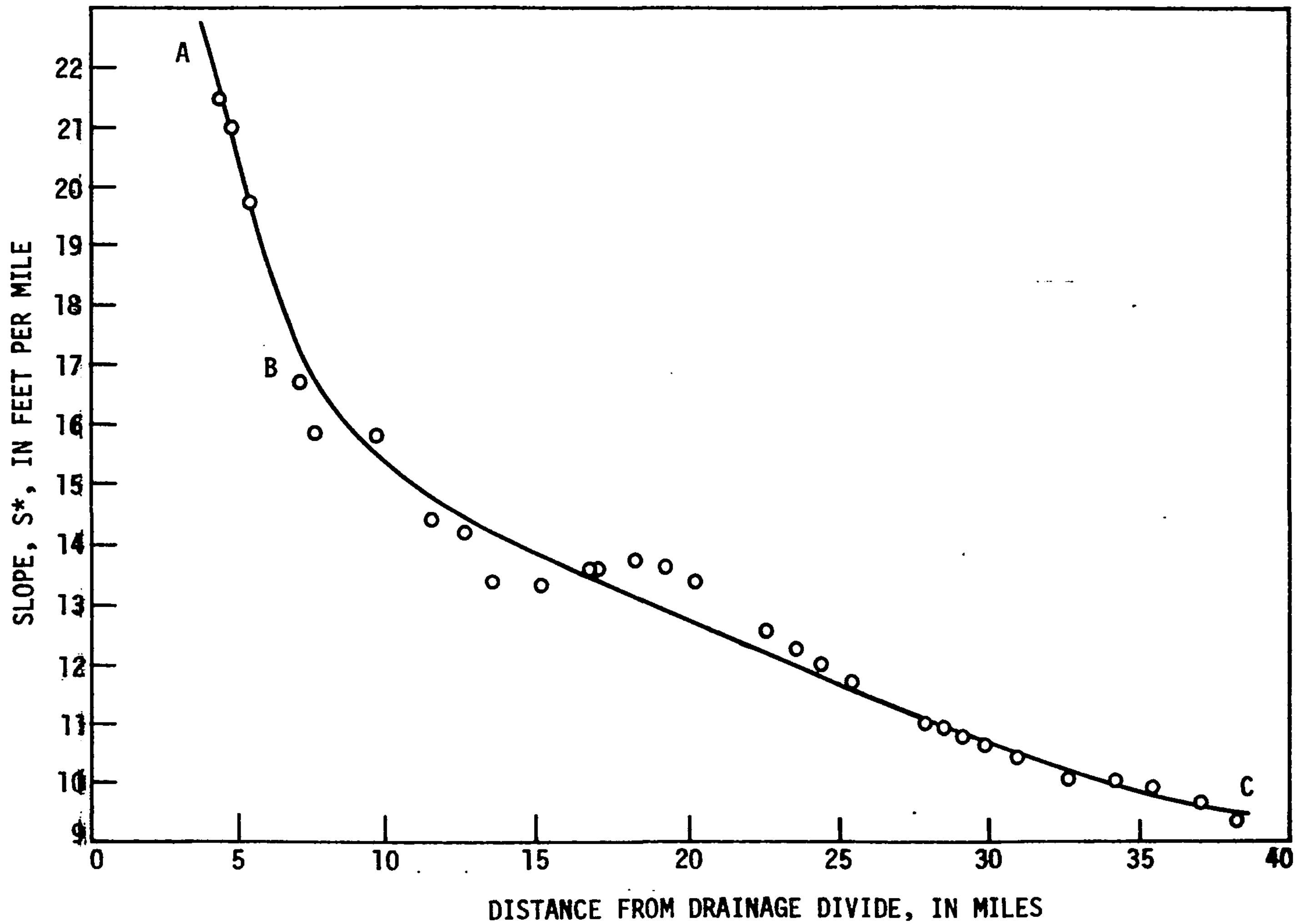


Figure 10. Relationship between S* and distance from drainage divide

was taken from Table A-1 of Appendix A. The solid line in Figure 10 is the average line through the scattered data. The segment AB of this curve can be represented by a straight line. The segment BC is plotted on semi-log paper which is shown in Figure 11. Notice that for segment BC, the data are scattered, with the maximum deviation of 0.75 units/in S*. However, in this range the overall effect would

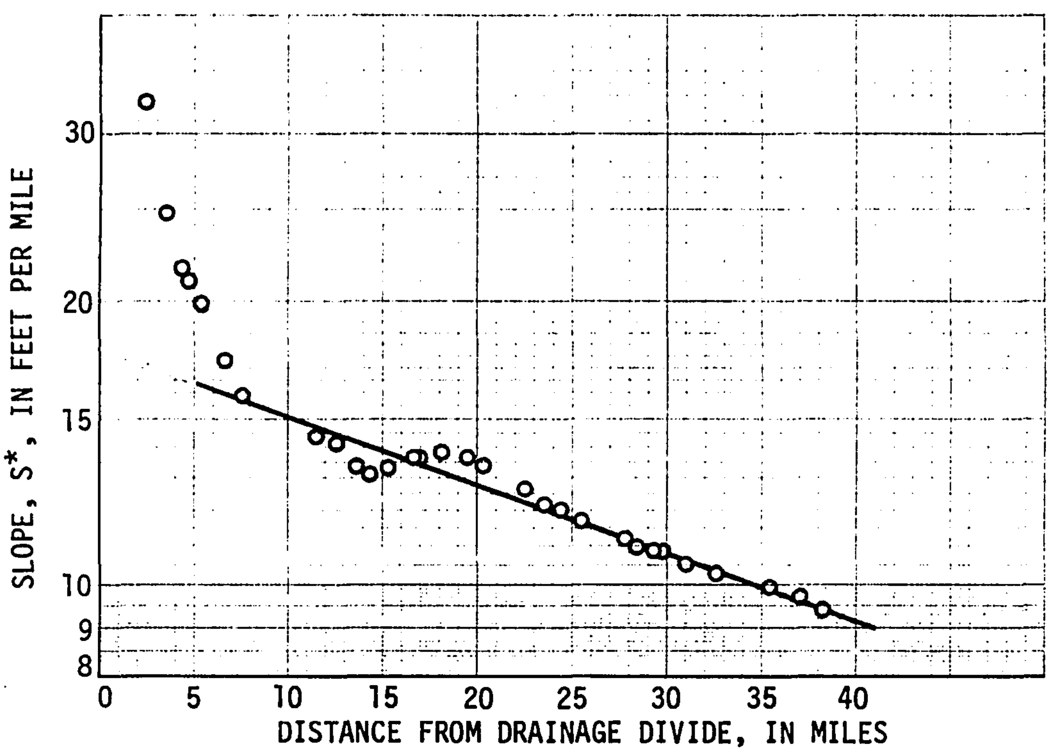


Figure 11. Relationship between S* and distance from drainage divide, semi-log plot

not be significant. To show this, let S* take on two values of 13 and 13.75 feet/mile. For instance, if equation 17 for recurrence interval of 25 years is used, the results would become:

$$S^* = 13, \quad Q_{25} = C_t \; (D_a)^{X_t} \; (13)^{0.367} = 2.56 \; C_t \; (D_a)^{X_t}$$

$$S^* = 13.75, \quad Q_{25} = C_t \; (D_a)^{X_t} \; (13.75)^{0.367} = 2.62 \; C_t \; (D_a)^{X_t}$$

The difference in Q_{25} is only 2 percent.

The equation of the average lines in Figures 10 and 11 can be expressed by the following functions:

$$S^* = -1.92 \, X + 30.18 \qquad 0 < X < 7.5 \tag{27}$$

$$S^* = 17.1 \, (10)^{-0.00712 \, X} \qquad 7.5 < X < 37.9 \tag{28}$$

where, S* is in feet per mile and X is in miles.

Expressions for D_a, drainage area,and S*, the slope, have been derived which can be substituted in equation 17 to solve for the flood discharge for any desired recurrence interval. The flood discharges at selected locations have been computed and are summarized in Table 7. The discharge versus distance for different recurrence intervals is plotted in Figure 12.

Figure 13 also illustrates the flood frequency curves at different reaches for the Willow Creek. It is observed that one might be able to determine the flood discharge at any given point and for any desired recurrence interval.

Table 7. Relationship between the flood discharge and distance for different recurrence intervals, INRC method, Model II[a]

Section No.	Distance from drainage divide (miles)	Drainage area sq. mi	Drainage area acres	Flood discharge, cfs Q_2	Q_5	Q_{10}	Q_{25}	Q_{50}	Q_{100}
1	2	2.11	1350	260	580	870	1340	1780	2290
	4.2	4.44	2842	400	860	1280	1930	2530	3200
2	4.2	6.45	4130	530	1100	1610	2400	3100	3900
	6	8.86	5670	600	1260	1830	2710	3490	4370
	8	11.54	7385	660	1370	2000	2950	3800	4750
	9.6	13.68	8755	740	1520	2190	3230	4140	5150
3	9.6	22.07	14125	1030	2070	2950	4260	5380	6620
	12	26.41	16900	1150	2280	3240	4660	5870	7180
	14	30.03	19220	1240	2450	3460	4960	6230	7600
	16	33.65	21540	1320	2600	3670	5230	6560	7990
	17.1	35.64	22810	1360	2680	3770	5380	6730	8190
4	17.1	48.25	30880	1680	3260	4550	6400	7950	9600
	20	59.06	37800	1900	3650	5050	7080	8750	10520
	22	64.87	41520	1990	3820	5280	7380	9110	10940
	24	69.95	44770	2070	3950	5460	7620	9400	11270
	26	74.52	47690	2130	4060	5600	7810	9620	11540
	27.8	78.30	50110	2170	4140	5710	7950	9790	11730
5	27.8	87.42	55950	2340	4450	6110	8470	10410	12430
	29.2	88.94	56920	2350	4450	6120	8490	10430	12460
6	29.2	97.27	62260	2500	4710	6460	8930	10950	13050
	31.9	100.40	64256	2500	4720	6470	8950	10980	13090
7	31.9	114.10	73020	2730	5130	7000	9640	11780	14000
	34	117.61	75270	2740	5150	7040	9690	11840	14070
	35.5	120.12	76880	2750	5170	7060	9720	11880	14120
8	35.5	123.13	78800	2800	5250	7170	9860	12040	14300
	38.25	130.12	83280	2850	5330	7280	10010	12230	14520

[a]For metric units, multiply miles by 1.609 to give km, sq. mi by 2.59 to give sq km, acres by 0.405 for hectares, and cfs by 0.0283 for cms.

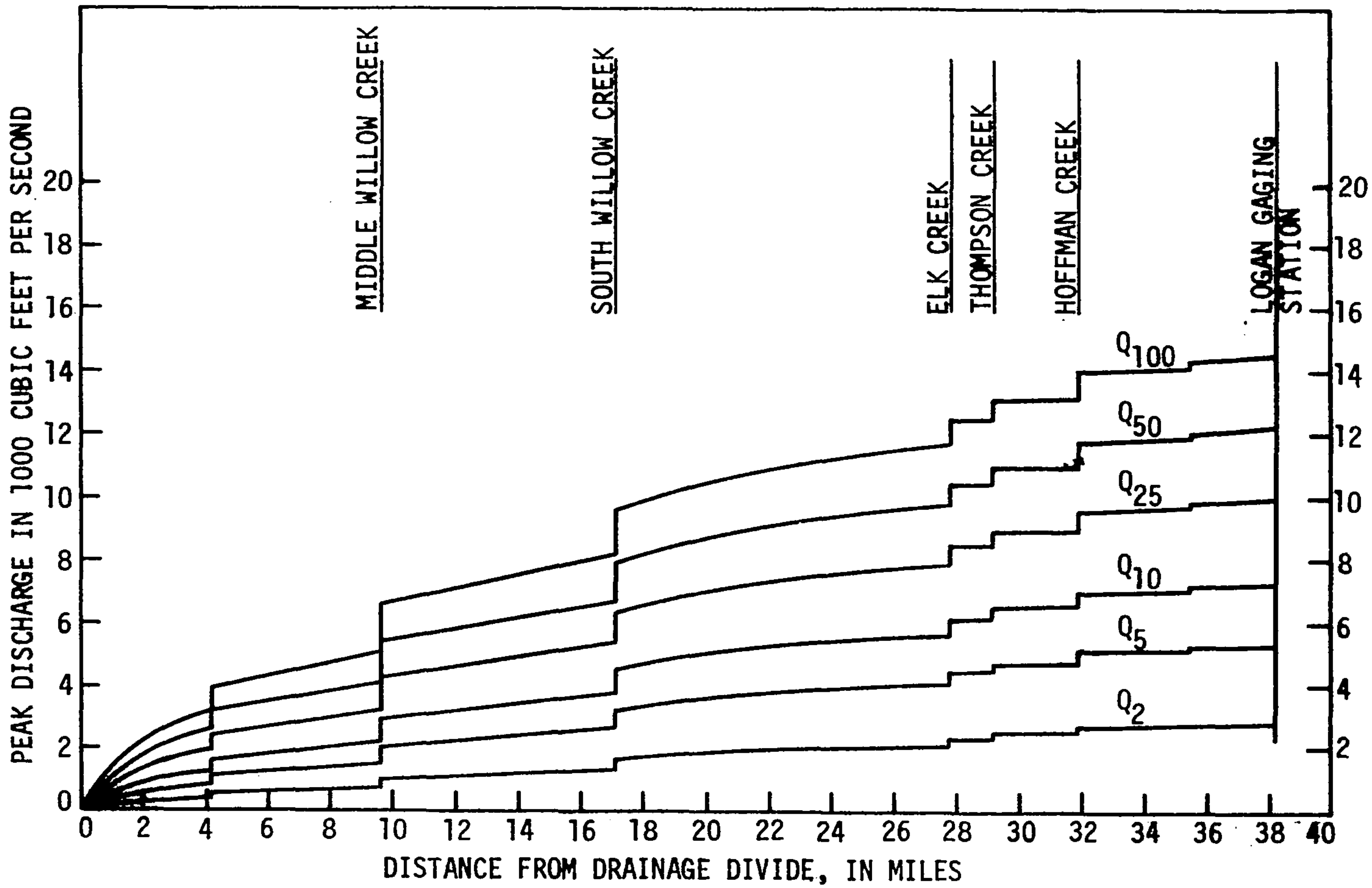


Figure 12. Relationship between the peak rate of runoff and distance in Willow Creek for different recurrence intervals (INRC method, Model II)

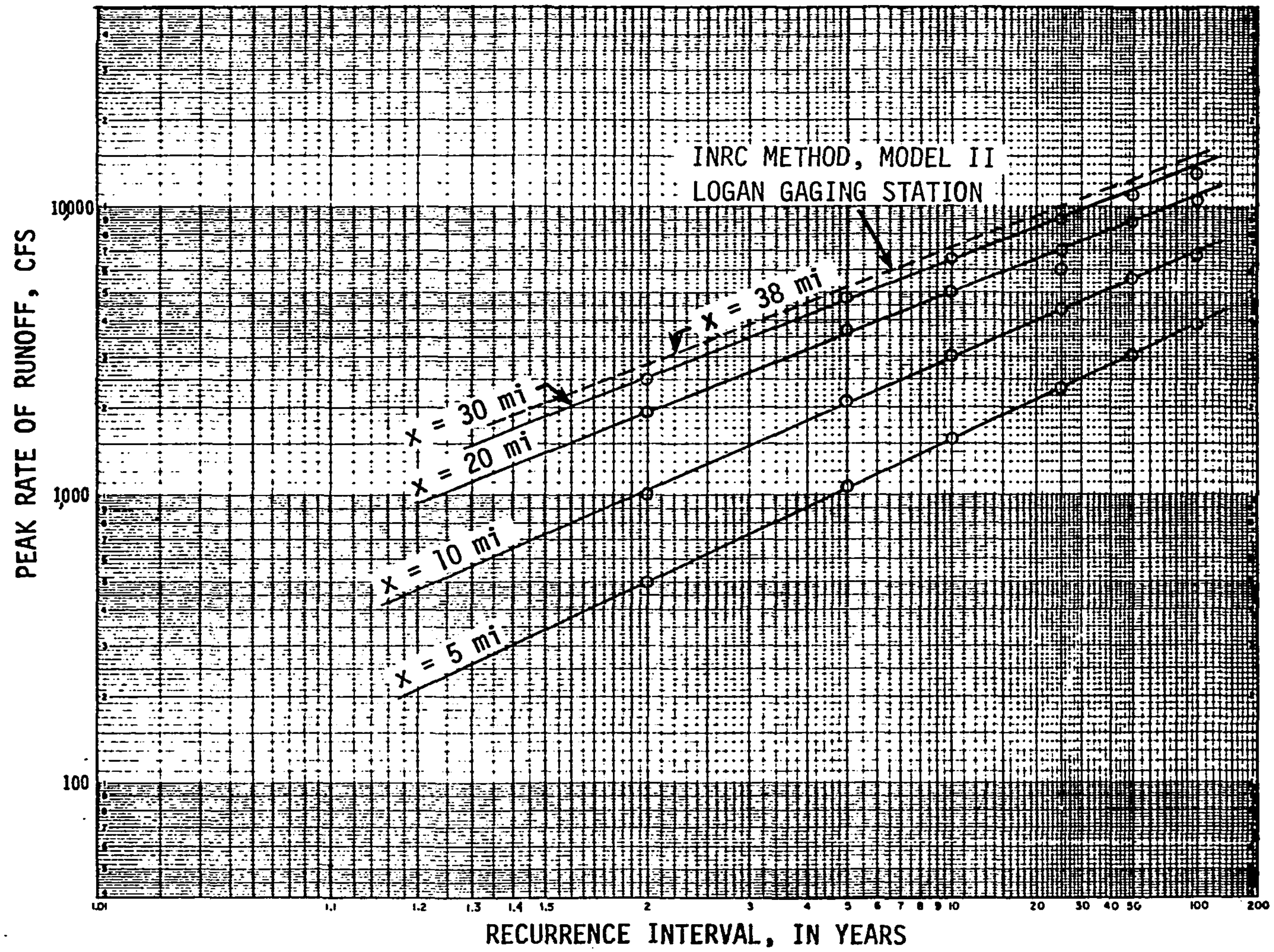


Figure 13. Flood discharge-frequency curves for Willow Creek at stations indicated

Besides the above computations (based on equation 17), model I (equation 16) was also used to compute the differences in discharge prediction. The model underestimated the flood peaks. For example, at Logan gaging station the estimated flood peaks were: Q_2 = 2,660, Q_5 = 5,030, Q_{10} = 6,800, Q_{25} = 9,530, Q_{50} = 11,680, and Q_{100} = 13,980 cubic feet per second, while model II resulted in 2,850, 5,330, 7,280, 10,010, 12,230, and 14,520 cubic feet per second respectively. However, as mentioned earlier, only model II will be considered for this study.

IDOT method (Iowa Department of Transportation)

The method presently used by the Iowa State Department of Transportation (IDOT, formerly the Iowa State Highway Commission) for small ungaged watersheds is shown in Figure 14. To be able to determine the peak rate of runoff for a given watershed, for a given recurrence interval, three variables are required: a frequency factor (FF), a land use and slope factor (LF), and the drainage area. To express the chart mathematically, Rossmiller (1974) derived the following complex equation which reflects the chart curve results up to 20,000 acres (31.25 square miles).

$$Q = 6.499\, A^{\left(\frac{0.858}{A^{0.0155}}\right)} - \frac{\ln (0.11A)^{1.88}\, A^{\left(\frac{1.21}{A^{0.05}}\right)}}{75} \qquad (29)$$

where, Q is in cubic feet per second and A is in acres.
The complexity of this equation was due to the inclusion of the curved portion of the chart for drainage areas less than 100 acres.

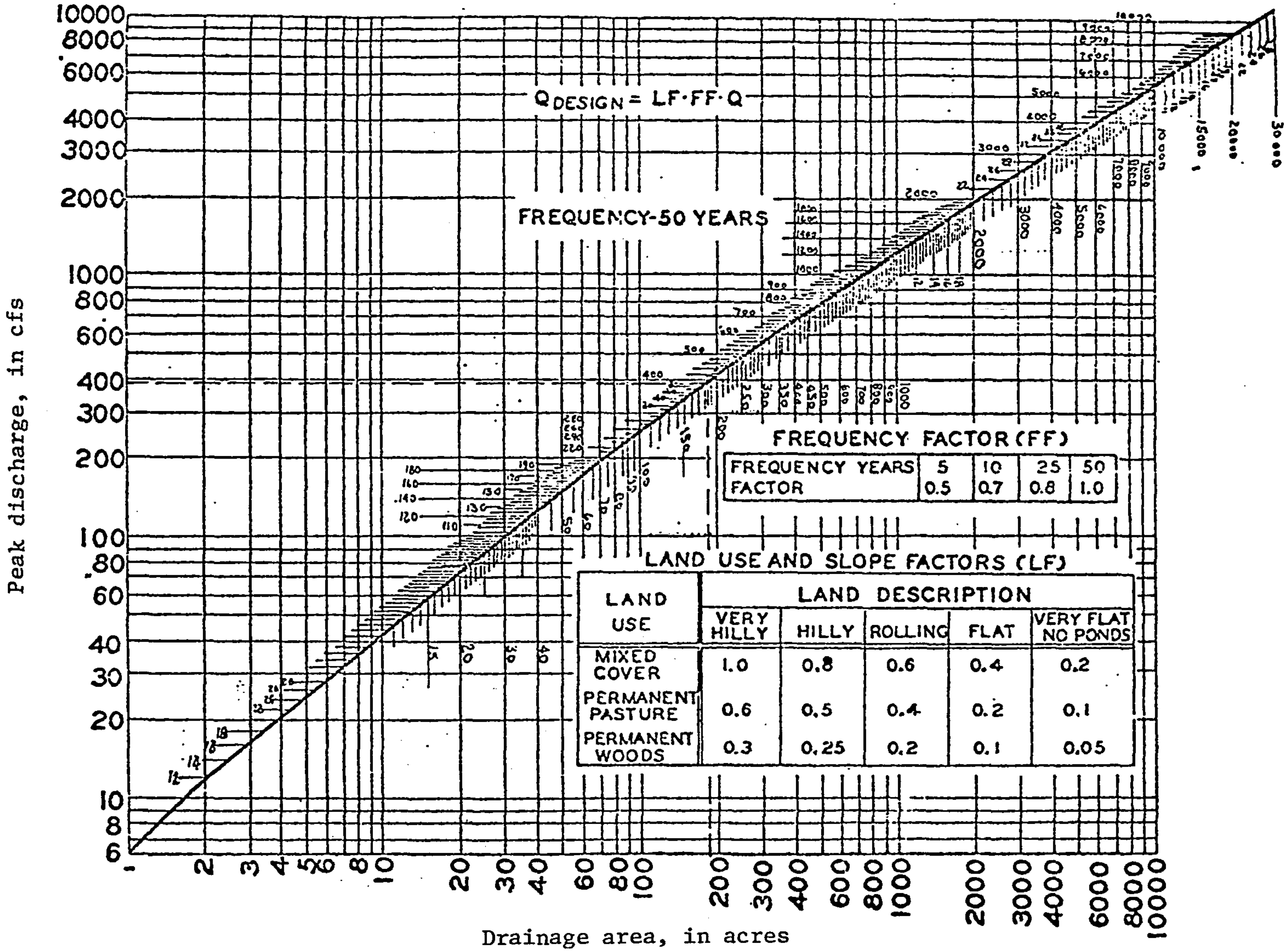


Figure 14. Relationship between the peak rate of runoff and drainage area

Since in the present study this small size of drainage area is less important, the author derived a much simpler mathematical relationship which very well describes the chart above 100 acres, but less than 10,000 acres. The equation could be written in the following form:

$$Q = 13.10\ A^{0.656} \tag{30}$$

where, Q is in cubic feet per second and A is expresses in acres. Should the drainage area be expressed in square miles, the equation takes the following form:

$$Q = 908\ D_a^{\ 0.656} \tag{31}$$

Thus, instead of using the chart in Figure 14, equation 30 or 31 is recommended to be used, which will essentially give the same results for $100 < A < 10{,}000$ ac ($0.16 < D_a < 15.6$ sq mi). The comparison among the three methods; IDOT chart, Rossmiller's equation, and author's equation is made in Table 8. Note that equation 30 is good for drainage areas less than 10,000 acres (15.6 sq. mi), however, it was used up to 20,000 acres (31.25 sq. mi) for comparison purposes only. The drainage area in Upper Willow Nos. 1 and 2 varies from 16.38 to 130 square miles (42.4-337 sq. km). Therefore, equations 30 and 31 should not be applied in those reaches. In a subsequent section, an equation will be developed which can be used in Upper Willow drainage districts for flood discharge prediction.

Table 8. Comparison of discharge using the IDOT chart and equations 29 and 30[a]

Drainage area (A), acres	Peak rate of runoff (Q), cfs		
	IDOT chart (Figure 8)	Equation 29	Equation 30
100	250	252	268
200	415	413	423
400	675	669	667
600	885	882	870
800	1,060	1,071	1,051
1,000	1,240	1,244	1,217
2,000	1,960	1,969	1,917
4,000	3,080	3,092	3,021
6,000	3,970	4,013	3,942
8,000	4,800	4,822	4,760
10,000	5,520	5,555	5,511
11,000	5,900	5,900	5,867
12,000	6,200	6,234	6,211
13,000	6,550	6,556	6,546
14,000	6,900	6,869	6,872
15,000	7,200	7,173	7,190
16,000	7,450	7,469	7,501
17,000	7,750	7,759	7,806
18,000	8,050	8,041	8,104
19,000	8,300	8,317	8,396
20,000	8,600	8,588	8,684

[a]For metric conversion, multiply acres by 0.405 for hectares and cfs by 0.0283 for cms.

According to the IDOT method, the peak rate of runoff for any given recurrence interval is estimated from the following formula.

$$Q_t = LF \times FF \times Q \tag{32}$$

where

Q_t = flood peak for recurrence interval t, cfs

LF = land use and slope factor

FF = frequency factor

Q = the rate of discharge, in cfs, obtained from Figure 14 or from equation 30 as was suggested earlier

The land use and slope factor depends on the topography of the watershed and vegetation cover. The topography of the Willow Creek drainage basin can be classified as "hilly." The vegetation cover consists mainly of row crops with limited separate spots of wood and pasture, so, it is expected that the land use factor, at the present time, is some value in the range of 0.8 to 0.9. More is said about the land use change later.

The frequency factor, FF, depends on the design recurrence interval, and the corresponding values as suggested by IDOT are shown in Figure 14. To check the trend of the given values, the author plotted the frequency factors versus recurrence intervals on log-log paper, as indicated in Figure 15. To the author, the value of 0.7 which is given to a recurrence interval of 10 years, appears to be high, and it is suggested that 0.62 can be considered instead. To make the corresponding computations easier, the equation of the fitted line in the figure

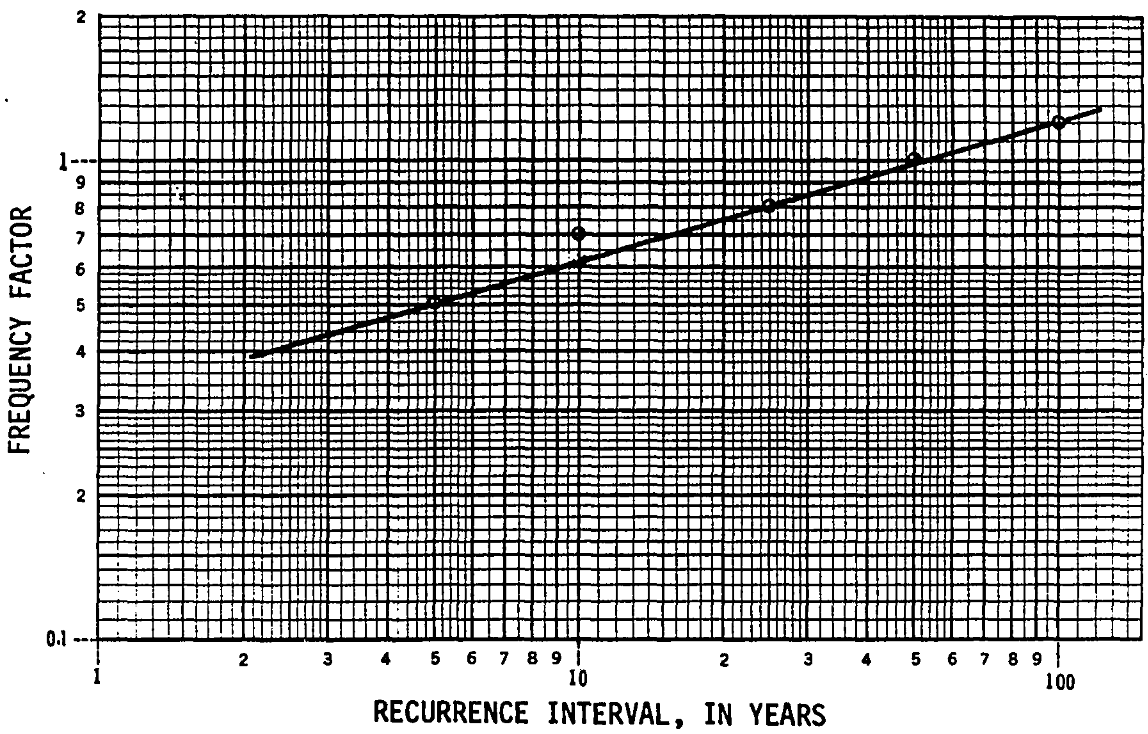


Figure 15. Relationship between frequency factor and recurrence interval

will be used later on to estimate the peak rate of runoff. The equation of the line in Figure 15 was derived and is expressed in the following form:

$$FF = 0.308\ (RI)^{0.301} \tag{33}$$

where, FF and RI are frequency factor and recurrence intervals respectively.

For drainage areas above 10,000 acres, extrapolation of the chart in Figure 14 tends to overestimate the peak discharge. To use the chart beyond 10,000 acres of drainage area, the author has made the following modification,which is based on the idea that the estimation by this method must agree with the INRC method. To force fit the curve to reproduce approximately the same results as obtained by the INRC method, the following steps were carried out.

From Table 7, the values of discharge and drainage areas for a recurrence interval of 50 years were plotted on log-log paper, and a straight line was fitted to the data. The equation of the line is:

$$Q = 42.29\ A^{0.504} \tag{34}$$

An approximate value of 0.8 was assumed for the land use and slope factor for Willow Creek drainage basin. The design Qs of the modified chart multiplied by 0.8 should reproduce the same values as obtained by equation 34. Therefore, if the ordinates of the curve of equation 34 are divided by 0.8, the modified IDOT chart will be obtained, and its equation is:

$$Q = 52.85\ A^{0.504} \tag{35}$$

Of course, it is understood that the shape of such a curve should be more concave downward; however, equation 35 is an approximation which will be adequate for our purposes, for drainage areas above 10,000 acres.

The comparison of the above mentioned methods is made in Figure 16.

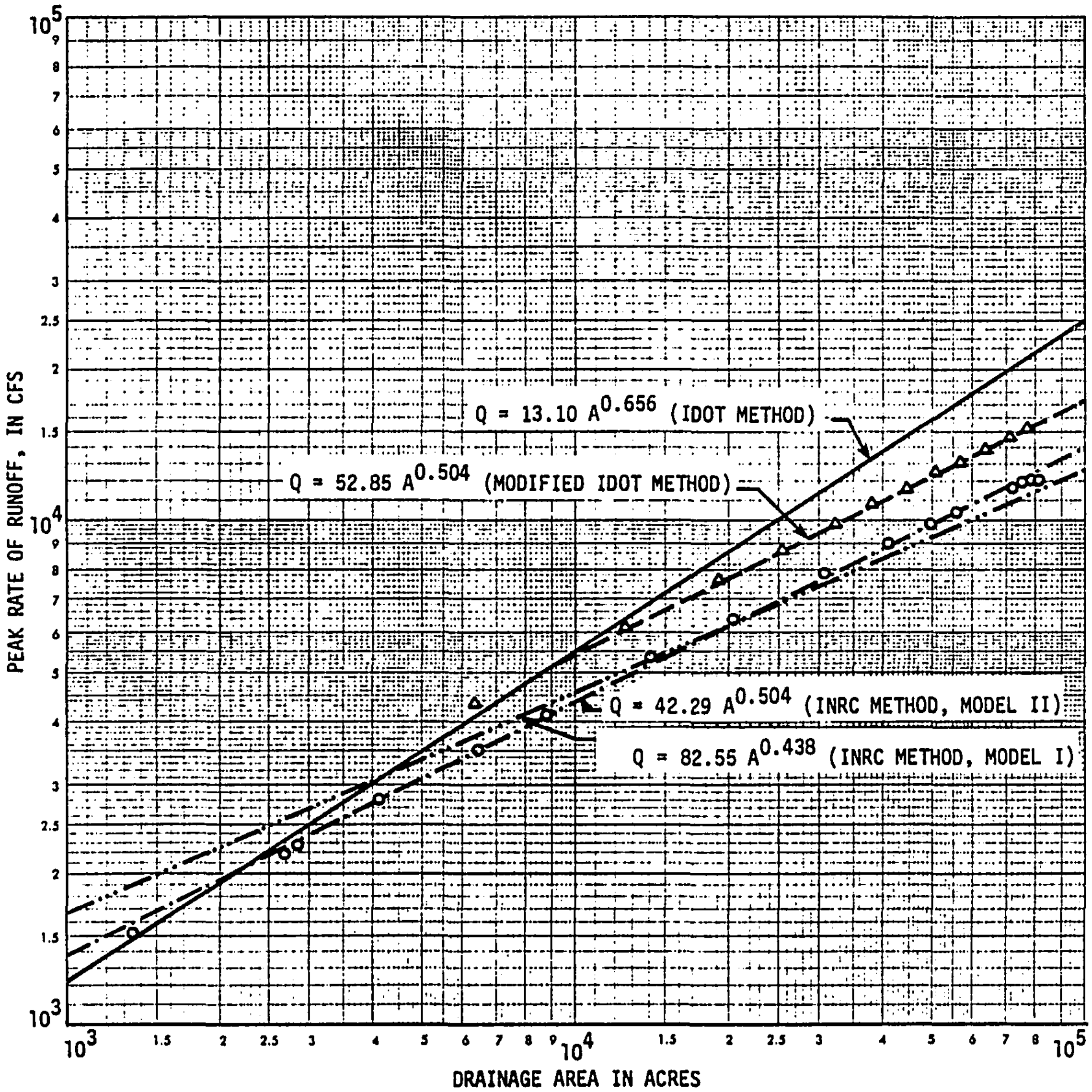


Figure 16. Adjustment of the extrapolated IDOT method, using the INRC Model II method, to obtain a "modified" IDOT method, for the 50-year flood discharge, Willow Creek

Also on the plot, the curve for model No. I ($Q = C_t D_a^{X_t}$) of INRC is shown to indicate that, compared to model II ($Q = C_t D_a^{X_t} S*^{Y_t}$), the discharge is overestimated for drainage areas less than 20,000 acres, and underestimated for drainage areas above this value.

Now to summarize, substitute equations 30, 33, and 35 into equation 32, and express the drainage area in terms of square miles, the following equations are obtained.

$$Q_t = 279.66 \text{ LF } (RI)^{0.301} D_a^{0.656} \tag{36}$$

for drainage areas $0.16 < D_a < 15.6$ square miles ($100 < A < 10{,}000$ acres) and

$$Q_t = 422.58 \text{ LF } (RI)^{0.301} D_a^{0.504} \tag{37}$$

for drainage areas $15.6 < D_a < 130$ square miles ($10{,}000 < A < 83{,}000$ acres).

At any selected point on the stream, and for a given recurrence interval, if the land use and slope factor is known, equations 36 and 37 will provide the peak discharge, in cfs (multiply by 0.0283 for cms).

To illustrate, the variation of discharge along the Willow Creek for recurrence intervals 10, 25, 50 and 100 years, and for different land use and shape factors ranging from 0.5 - 1.0, is calculated and the results are plotted on Figures 17, 18, 19, and 20. Also, for each recurrence interval, the curves in Figure 12 are superimposed on the above figures to indicate the effect of land use and slope factor on the variation of peak runoff.

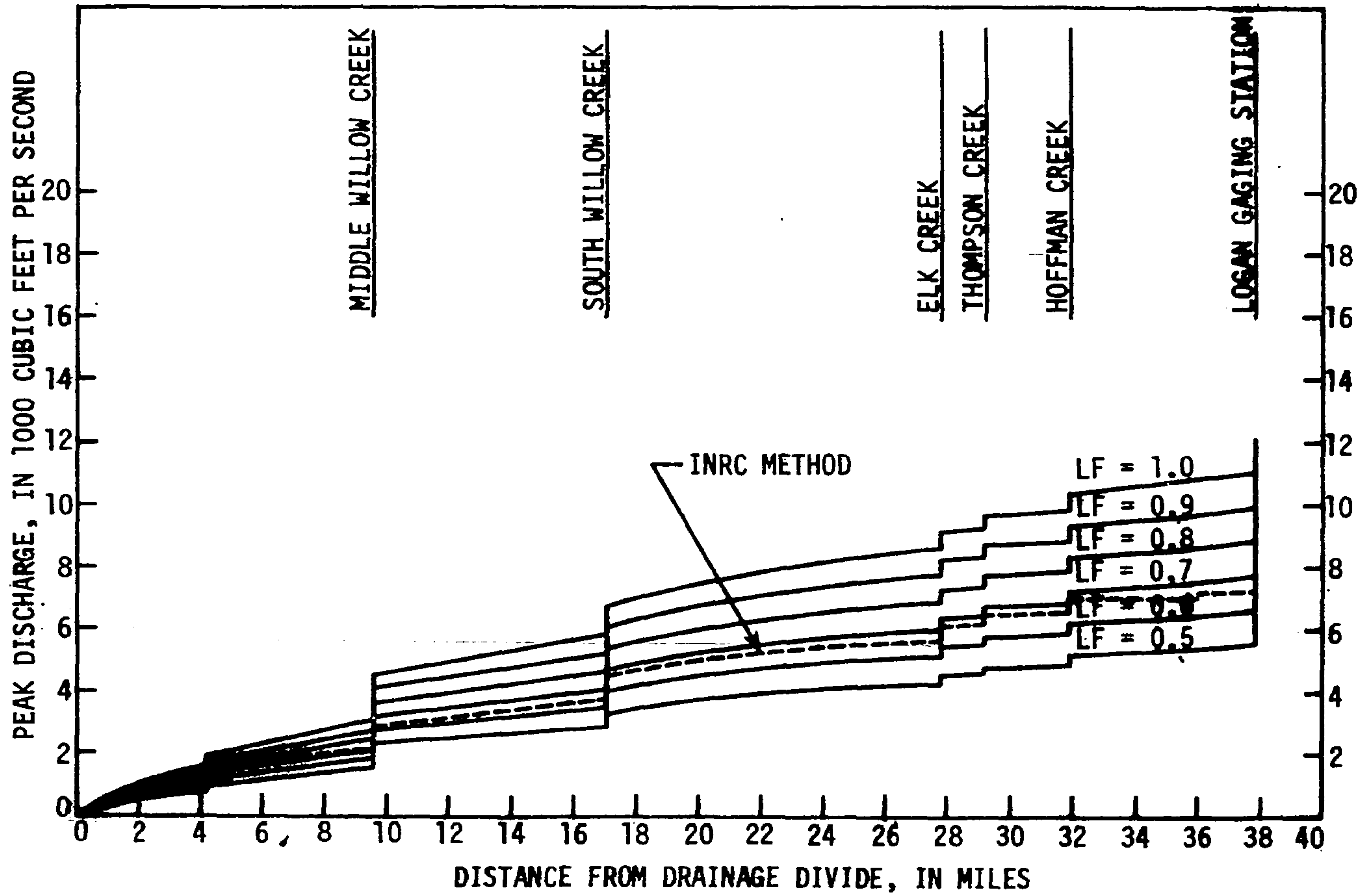


Figure 17. Relationship between the peak rate of runoff and distance (modified IDOT method) for different land use and slope factors in Willow Creek (recurrence interval = 10 years)

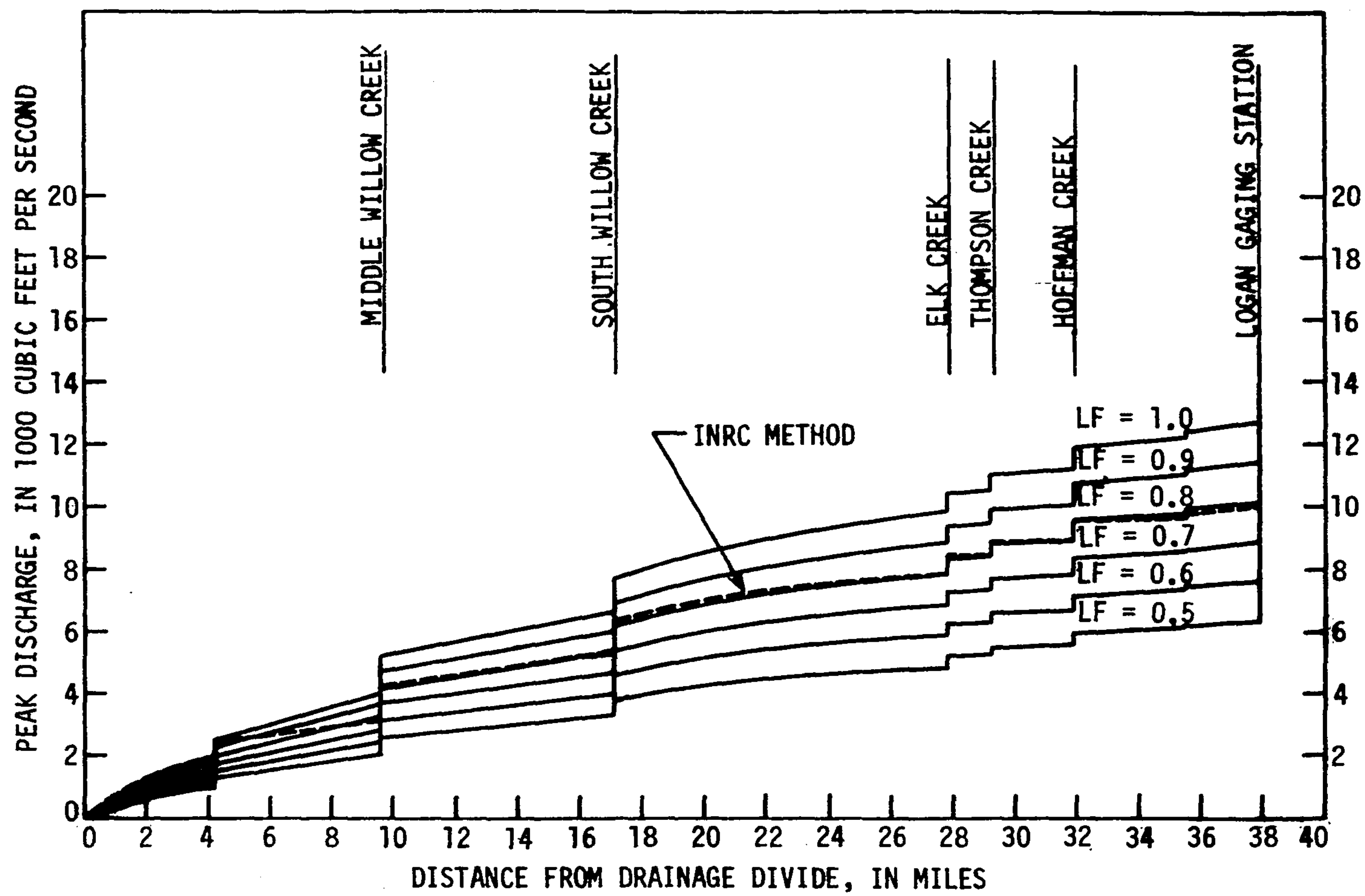


Figure 18. Relationship between the peak rate of runoff and distance (modified IDOT method) for different land use and slope factors in Willow Creek (recurrence interval = 25 years)

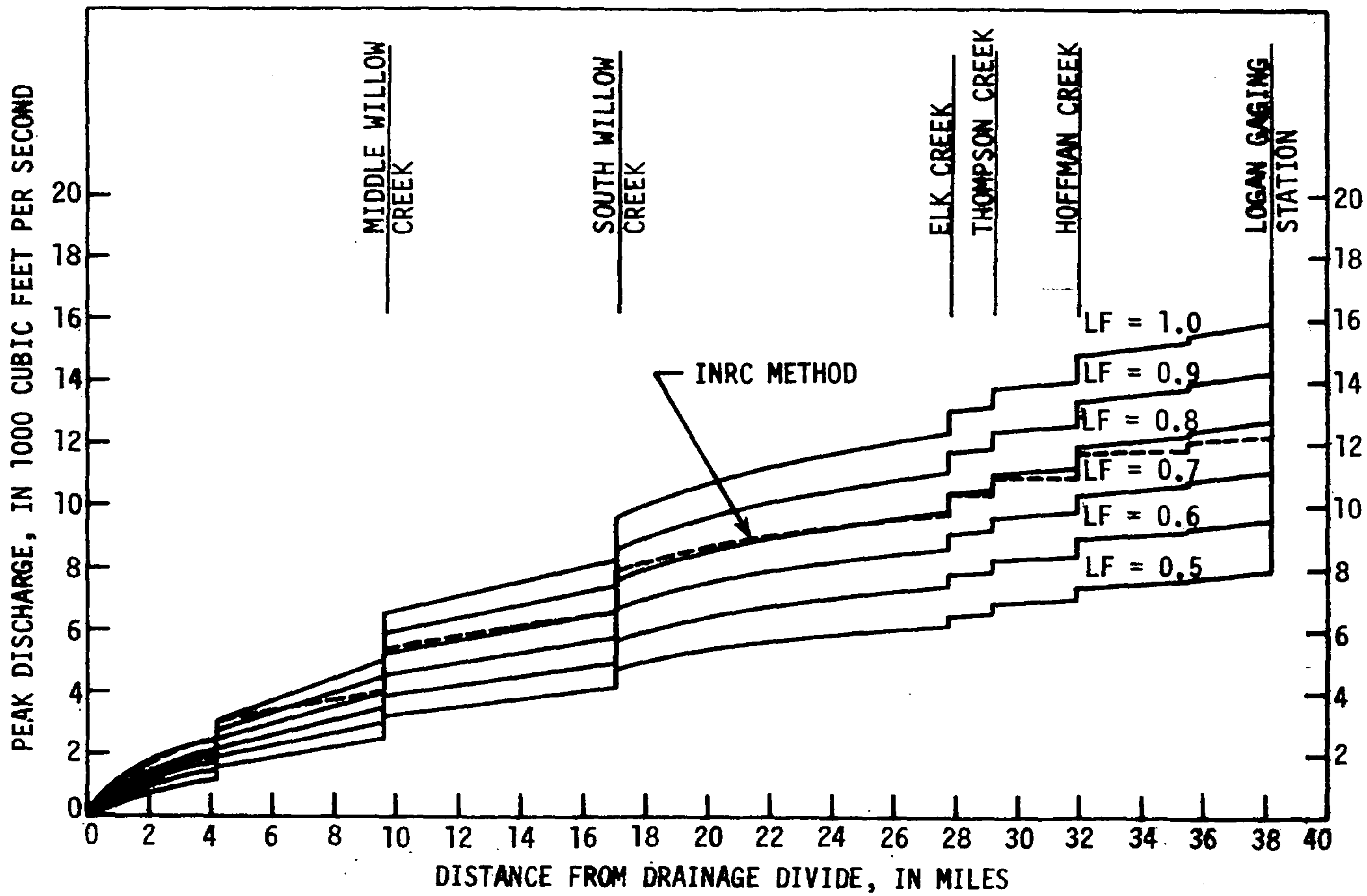


Figure 19. Relationship between the peak rate of runoff and distance (modified IDOT method) for different land use and slope factors in Willow Creek (recurrence interval = 50 years)

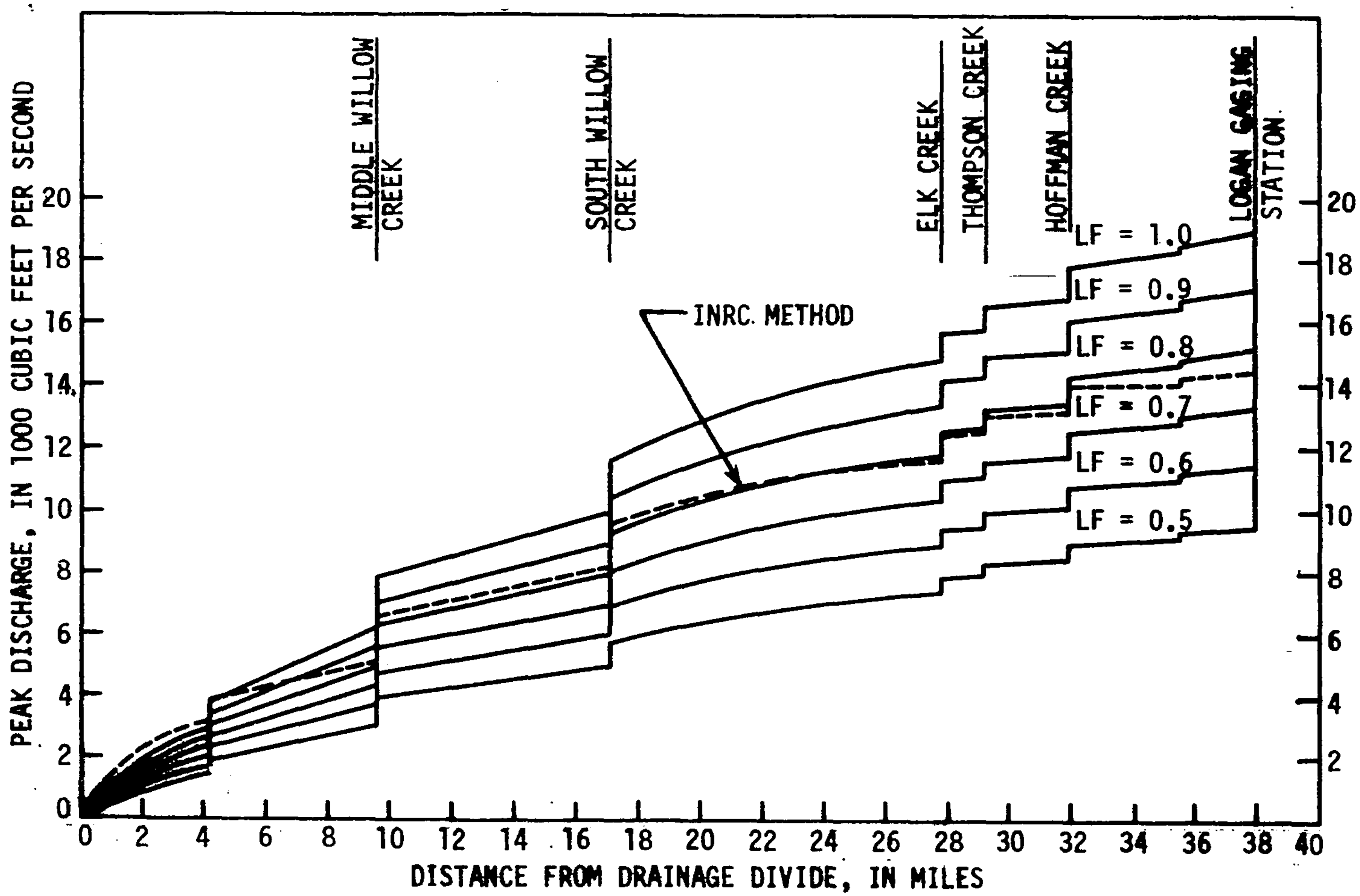


Figure 20. Relationship between the peak rate of runoff and distance (modified IDOT method) for different land use and slope factors in Willow Creek (recurrence interval = 100 years)

<u>Flood frequency analysis of the observed data</u>

As it was mentioned earlier, the annual series of flood peaks of the Willow Creek were listed in Table 4. These data were obtained by the Omaha District, U.S. Army Corps of Engineers, and are not in the data record or flood analysis of the USGS. Different flood frequency methods are available (Chow, 1964; Haan, 1977; Linsley et al., 1975; Yevjevich, 1972). For the data of Willow Creek, the log-Pearson type III distribution was used as it is recommended by several state and federal agencies (Lara, 1974; U.S. Water Resources Council, 1977). A log-Pearson type III distribution was fitted to the data in Table 4. The standard deviation and the skew coefficient of logarithms of the peaks were 0.5328 and 0.05 respectively. However, a generalized skew coefficient of -0.4 is suggested for hydrologic region I (Lara, 1974). Also, a value of -0.3 can be estimated for the Willow Creek using the map of the generalized skew coefficient in Bulletin No. 17A of U.S. Water Resources Council (1977). For the present analysis, a value of -0.4 was selected, to better correspond to state studies. Equation 15 was then used to compute Q_ts.

The computed flood frequency curve and the observed peak data are illustrated in Figure 21. The figure also illustrates the upper and lower confidence limit curves for the level of confidence of 0.05 and 0.95 respectively. The magnitudes of peak discharge for selected frequencies are listed in Table 9.

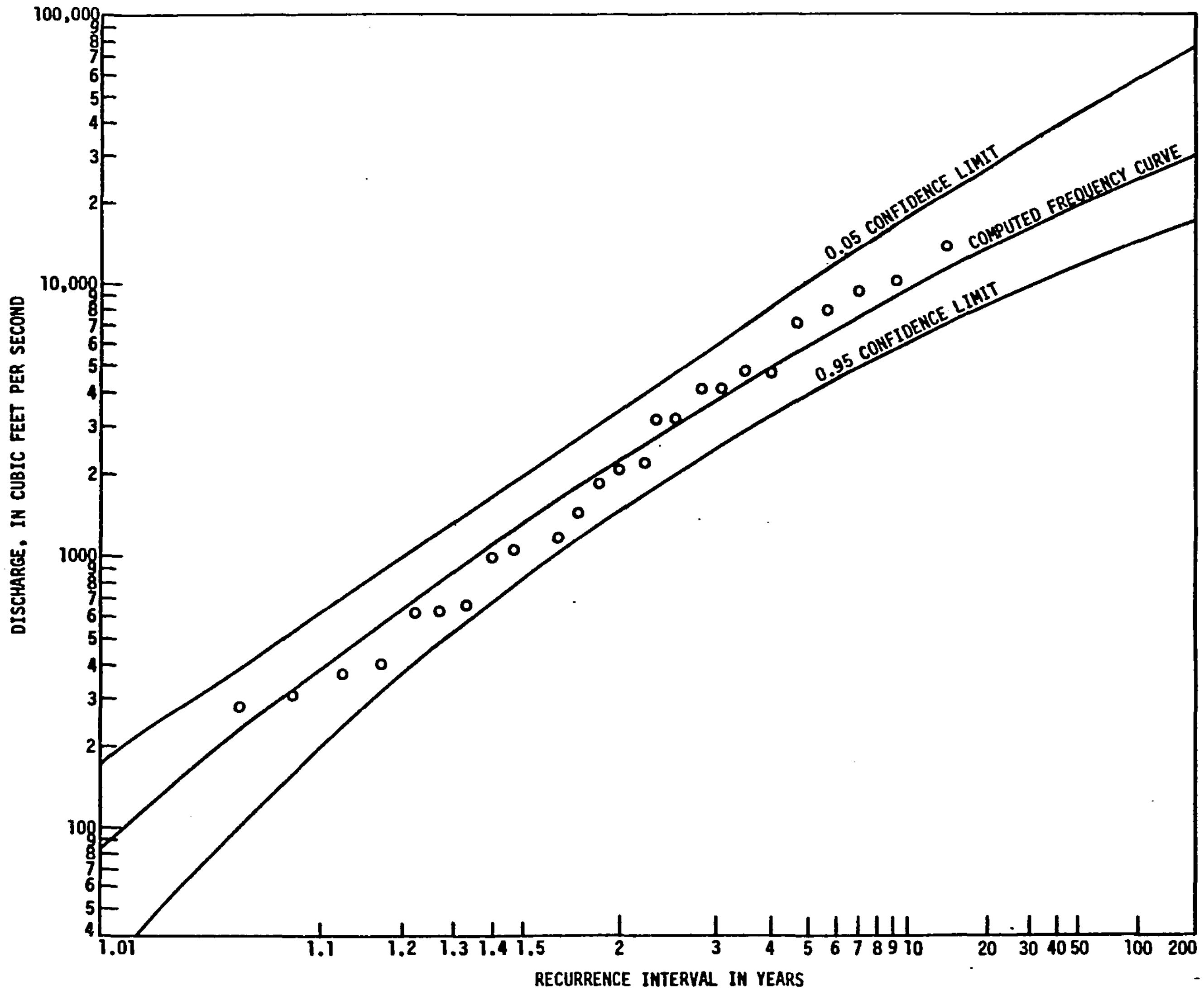


Figure 21. Flood frequency curve for Willow Creek, Iowa

Figure 21 illustrates that at the Logan gaging station, the estimated peak rate of runoff for a given frequency is much higher than those computed using the IDOT or INRC methods. The values obtained using the IDOT and INRC methods are in the lower range of the limits shown in Figure 21. This author believes that the magnitude of flood peaks given in Table 9 may be greater than expected. The reason for this is explained below.

The H & P drainage ditch, which starts just below the Logan gaging station, has experienced frequent aggradation in the past. Daniel (1960) stated that the flood waters in April, 1952 had backed up the Boyer and Willow drainage ditch, causing considerable silting in the channel. Therefore, due to the backwater effect from either the Missouri River or the Boyer River, overestimation of the stage in Logan gaging station may have occurred.

The author's personal discussions with the U.S. Army Corps of Engineers and the U.S. Geological Survey revealed that the maximum discharge which has been measured at the station is 3,260 cfs. The other peak values listed in Table 4 have been estimated using the slope-area method, or by extrapolation of the discharge rating curves beyond the range of the measured flow data. If overestimation of the stage during floods occurred, then the slope-area method will result in a higher than actual value. The extrapolation of the discharge rating curves has the same limitations.

Four regional flood frequency methods have been applied to 131 streams in Iowa, and a comparison was made in Bulletin No. 12 of Iowa Natural Resources Council (Lara, 1974). From this reference, the Q_{50} for 13 streams with drainage areas between 30 and 880 square miles in western Iowa are plotted in Figure 22. If the Willow Creek station data, at a drainage area of 130 square miles (at Logan gaging station), is superimposed on this plot, a relative comparison is possible. Figure 22 indicates, for a drainage area of 130 square miles, that none of the methods provide such high discharge values compared to those from Figure 21. Due to the lack of certainty about the measured data, the IDOT method, which was modified in terms of eqs. 36 and 37, will be used in a subsequent chapter to represent the peak flood discharges along the stream. The equation also has a computational advantage for the purpose of this study (see Croley, 1977).

Table 9. Results of station frequency analysis for Willow Creek, using observed data of COE and USGS

Recurrence interval, years	Magnitude of flood discharge cfs	cms
2	2250	64
5	5900	167
10	9400	265
25	14900	420
50	19700	560
100	25000	710

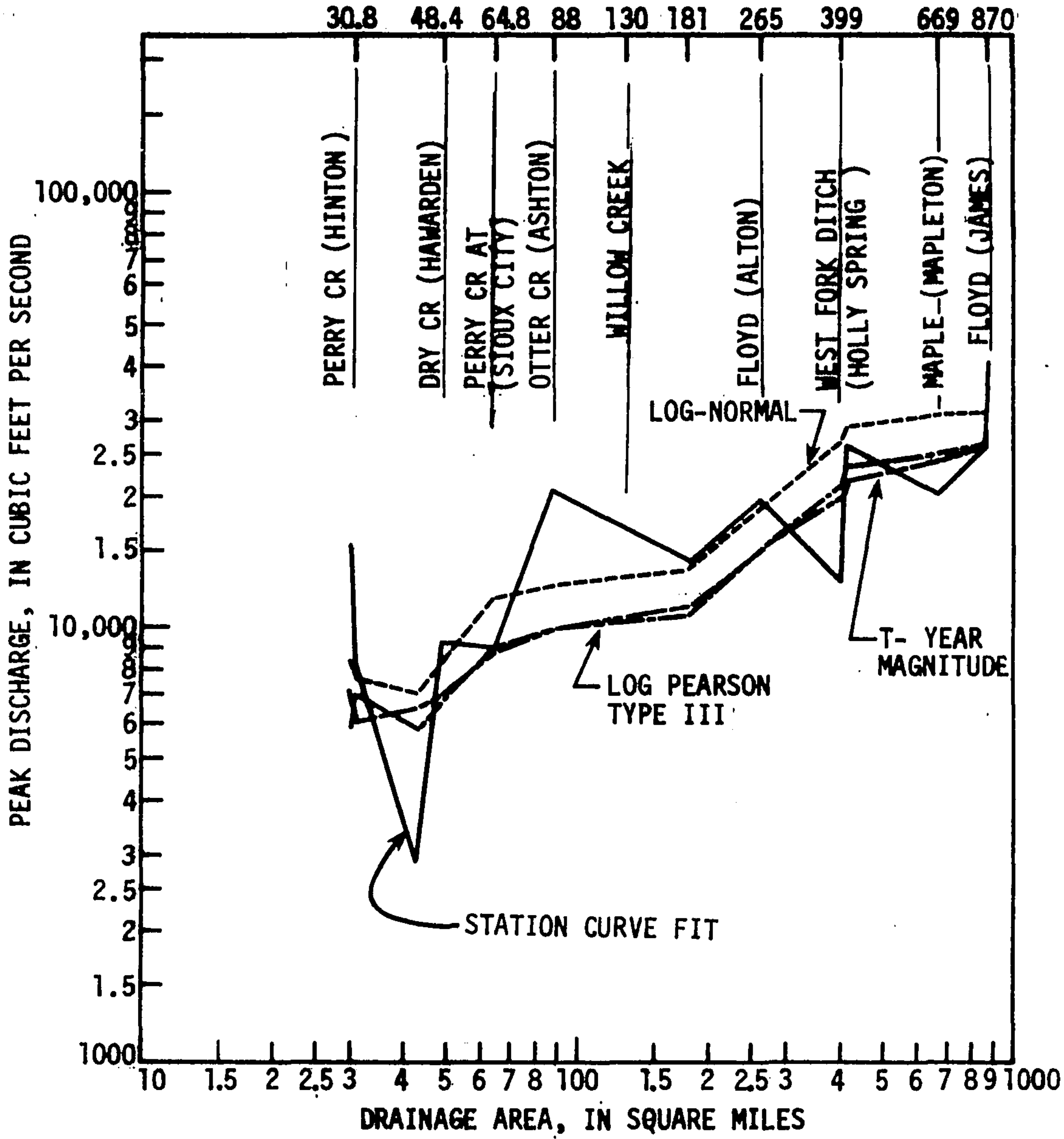


Figure 22. Comparison of the 50-year flood discharge by applying four flood frequency methods

CHAPTER V.

CAUSES AND EXTENT OF DEGRADATION

Background

As was explained in Chapter III, the Willow drainage ditch was constructed in three different stages (e.g. H & P, Upper Willow Nos. 1 and 2). The lower part of the drainage ditch (H & P) has been subject to filling since construction, and it has been cleaned several times. This first part actually is located on the flood plain of the Missouri River, as the outlet of Willow Creek proceeds from the bluff line southward to the Boyer River channel, just before it joins the Missouri River.

In contrast to the silting problem in the H & P segment, the Upper Willow Nos. 1 and 2 reaches have experienced severe degradation, beginning immediately after construction. The historical data indicate that the junction point of H & P and Upper Willow No. 1 (see Figure 36a) has not been subject to degradation. This location, near the bluff line, can be considered as the division point between the degradation upstream and aggradation downstream. This point will be used as the starting point for the computation of degradation which will be presented in Chapter VI.

Those factors considered most important in contributing to the degradation of the ditch in Upper Willow drainage districts are discussed in the following sections.

Stream Slope

One of the main reasons for the occurrence of severe degradation in the Upper Willow drainage districts is the increased gradient due to straightening. The original Willow Creek, in this part, had a gradient ranging from 0.0010 - 0.0012 (Figure 4). By using the data in Table 1, and assuming an average cross section, it is easy to show that the original maximum capacity of the creek at bankfull discharge was about 2,500 cfs (71 cms). Consequently, any flow rate exceeding this value flooded the adjacent flood plain.

The degree of straightening, in the three different drainage ditch reaches, can be observed in Figures 23, 24, and 25.

Compared to the original stream alignment, the H & P drainage ditch was constructed on a much different relocated alignment. However, the Upper Willow drainage districts were located almost on the same path of the original stream, but eliminating the meanders.

The increase in stream bed slope can be determined from Figures 26 and 27, in which the distance of the drainage ditch has been plotted versus the length of the original stream. The vertical distance between the curves in these figures and the 45^{o} line represents the length by which the channel has been shortened at that point.

From Figure 26, it is concluded that, for example, nine miles (14.5 km) of the drainage ditch is equivalent to 13.2 miles (21.2 km) of the original stream. This represents a 32% reduction in length. In terms of slope, this resulted in a 47% increase in slope, compared

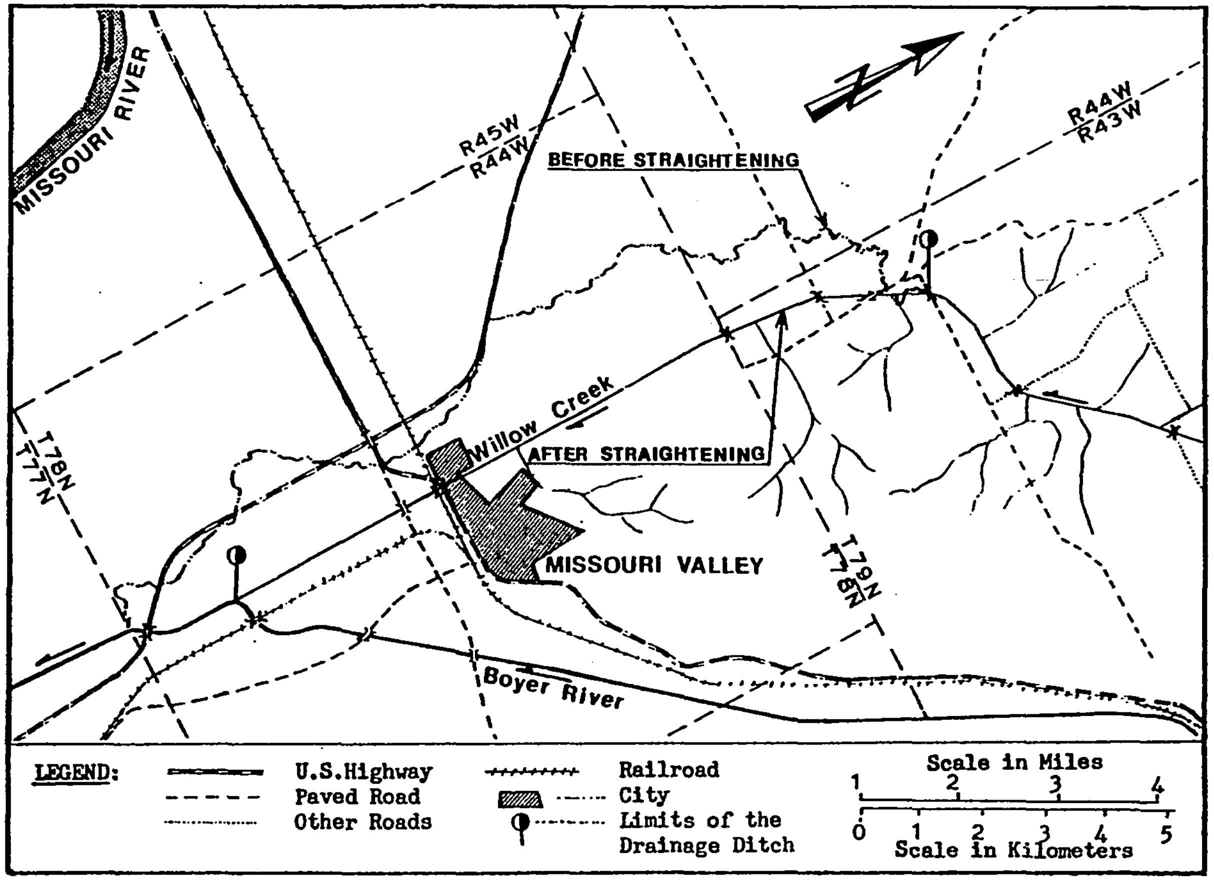


Figure 23. Plan view of the original Willow Creek and H & P drainage ditch

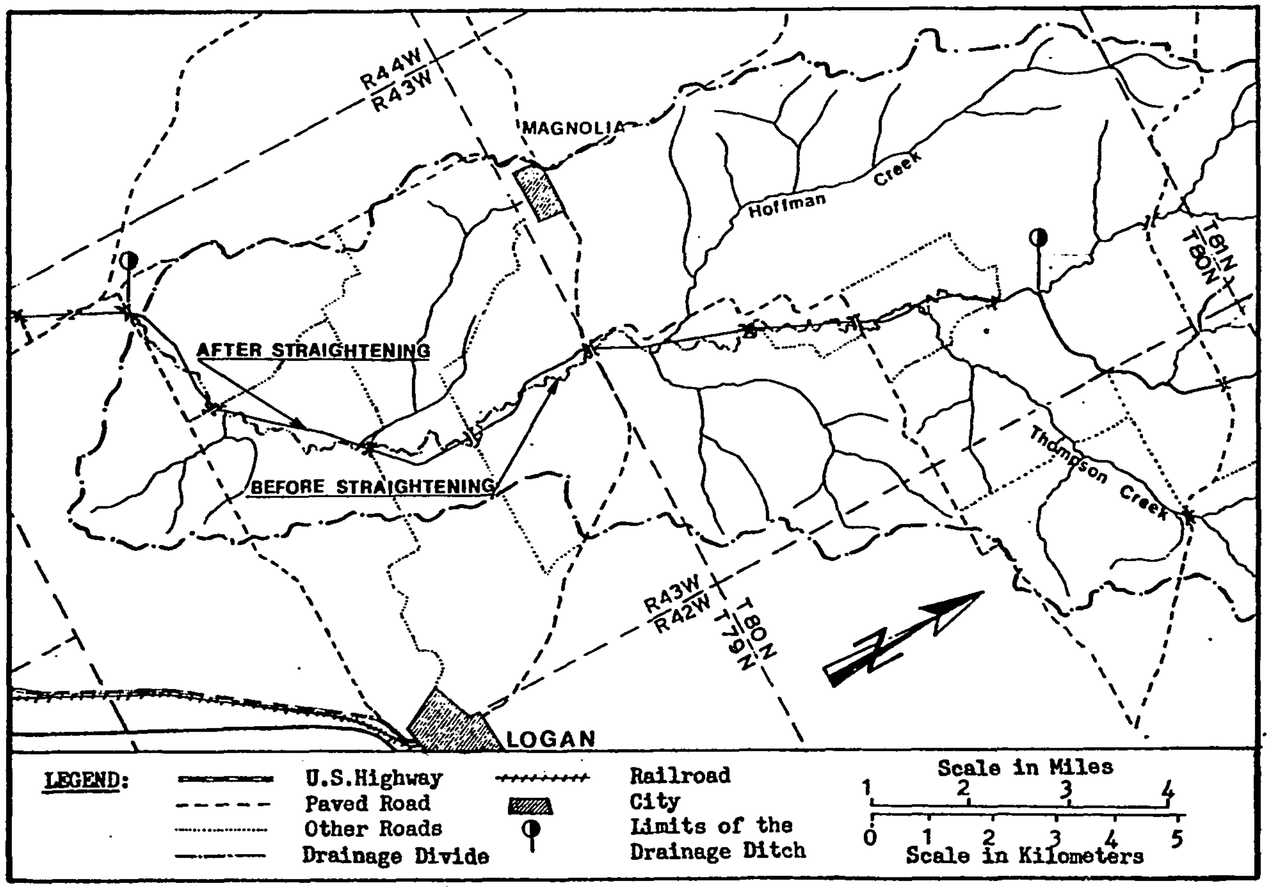


Figure 24. Plan view of the original Willow Creek and Upper Willow No. 1

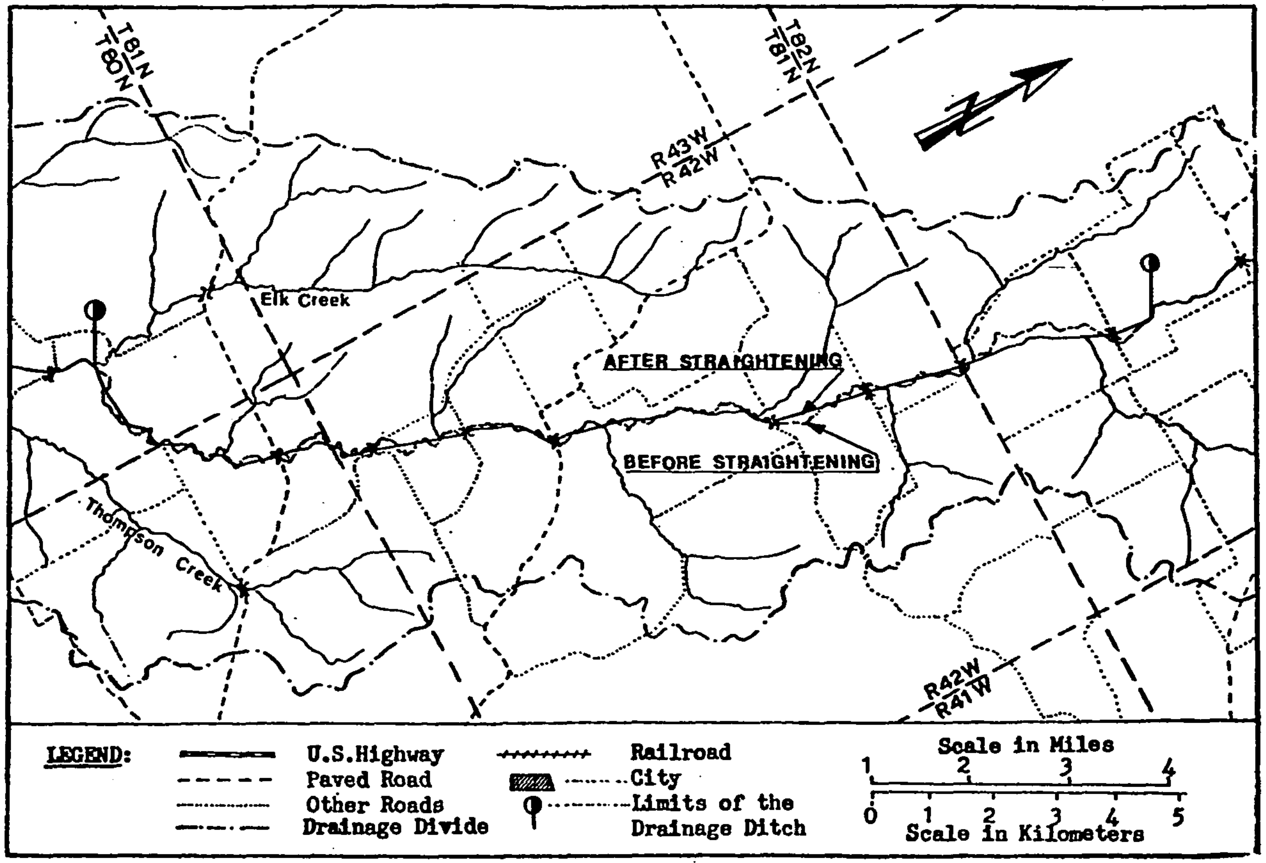


Figure 25. Plan view of the original Willow Creek and Upper Willow No. 2

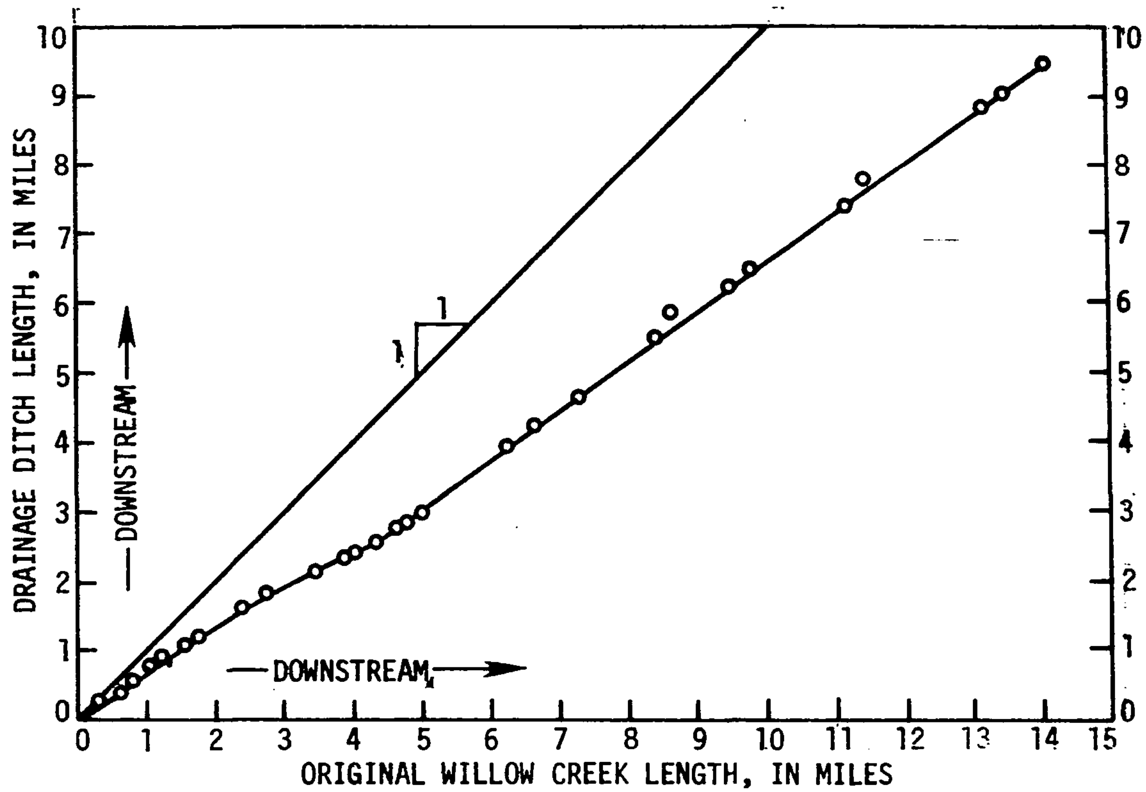


Figure 26. Comparison between the lengths of the original Willow Creek and Upper Willow drainage ditch No. 1

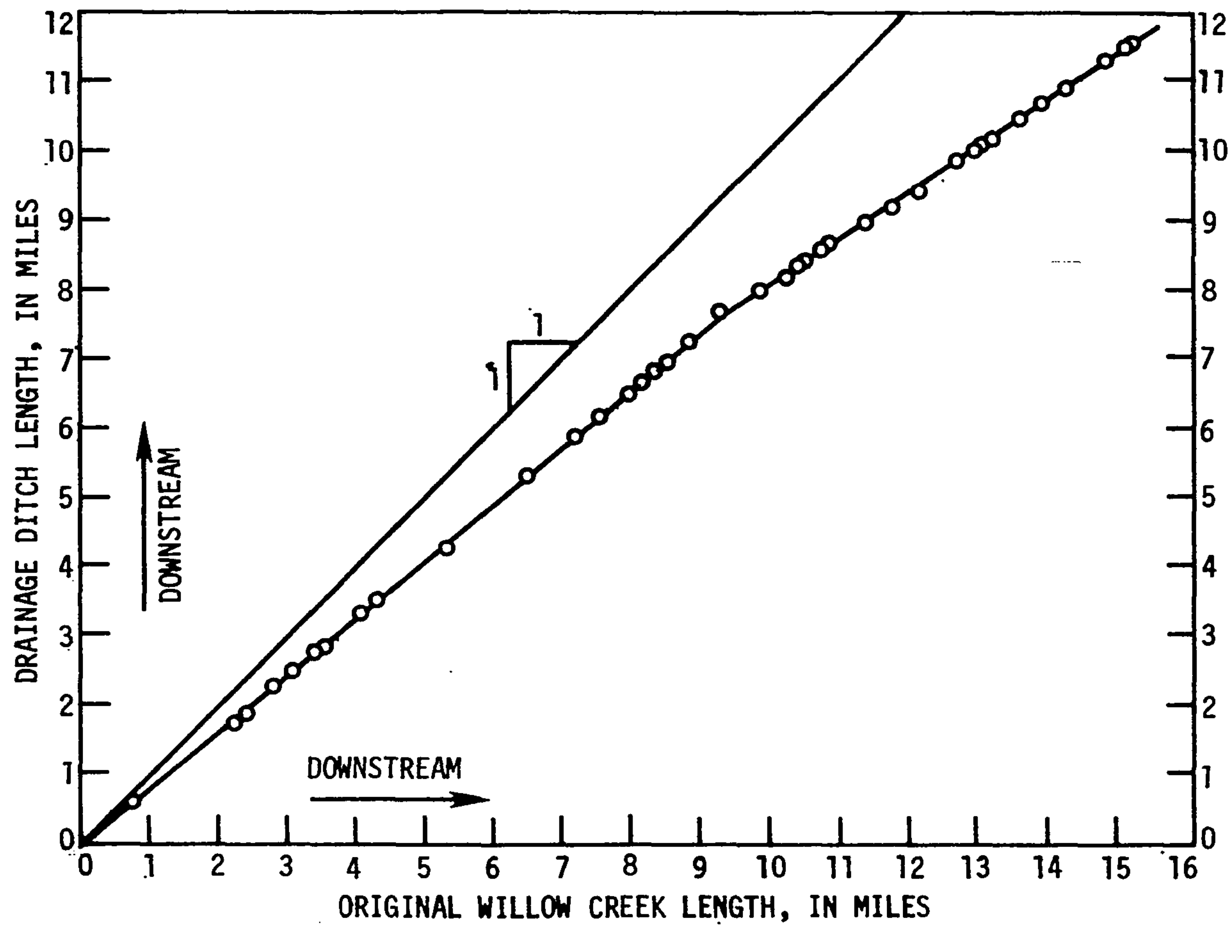


Figure 27. Comparison between the lengths of the original Willow Creek and Upper Willow drainage ditch No. 2

to the original slope. Similar conclusions can be made from Figure 27, in which the decrease in the total length of the stream is 25%, and corresponding increase in average slope is 33%. Figure 28 is the result of the combination of data in Figures 26 and 27.

In addition to the increased gradient at the completion of construction, continued changes in the stream slope, especially in Upper Willow No. 2, have resulted from severe scour. From Figure 6 (page 36) it can be observed that the slope of the ditch varied somewhat along the ditch. Slope variations, especially from mild to steep, and variations in bed materials, have been responsible for the formation and development of the knickpoints (overfalls). It is interesting to note that the junction point of Upper Willow No. 1 and 2 had about a 2 ft. (0.6m) overfall at the time of construction, because of arbitrary changes in the design bed elevation. This author believes that the introduction of this overfall was made to reduce the amount of earth work in the construction of Upper Willow No. 2.

Increase in Discharge

The variation in the discharge in the channel can be related to two components, the land use factor and increase in discharge capacity.

Land use factor

It has been demonstrated that the conversion of the natural grass prairie land into cultivation has increased the surface runoff significantly. Piest et al. (1977) estimated that the surface runoff volume

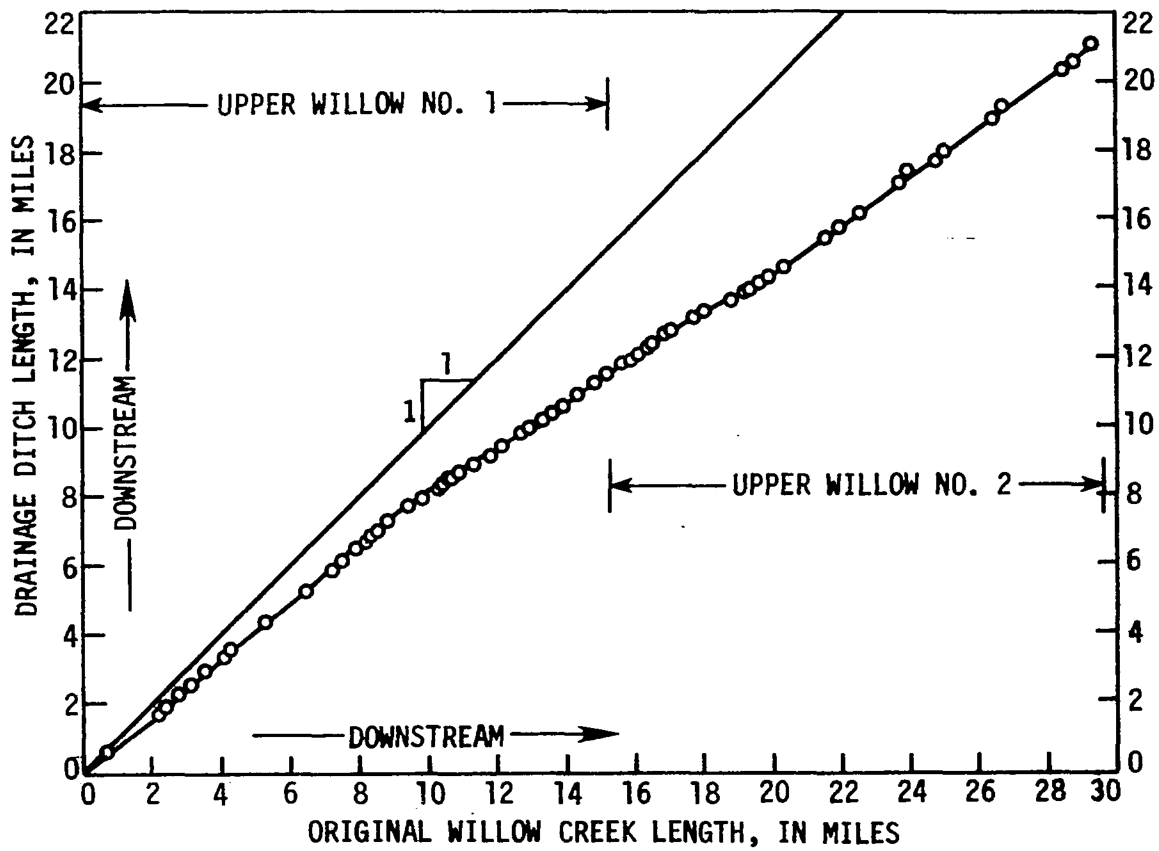


Figure 28. Comparison between the lengths of the original Willow Creek and Upper Willow drainage ditches Nos. 1 and 2 (combination of Figures 26 & 27)

was increased up to 2 to 3 times by row cropping, compared to prairie grass cover. Estimates as high as 80 times for the peak discharge have been reported (Leopold et al., 1964). Figure 29 illustrates the rate of land conversion in Harrison County, since the early 1800s. After 1920-25, no significant increase in the area under cultivation has occurred. A review of Figure 14 (p. 60) illustrates the historic impact of land use changes. Assuming today's LF = 0.8 (mixed cover, hilly), and the original prairie and timber LF to be between 0.4 and 0.5, then the increase in all flood discharges (since 1840) would be in the range of 60 to 100 percent (1.6 to 2.0 times). Most of this increase occurred before 1920, but illustrates the "carry over" for degradation stress of the straightened channel.

Increase in carrying capacity of the channel

As was shown previously, the water carrying capacity of the original Willow Creek at bankfull discharge was about 2,500 cfs (71 cms). In contrast, the designed Willow ditch, with more smooth cross sections, had a much higher capacity. To illustrate, the Upper Willow No. 1 with the original slope (0.145%), the given cross section (i.e. 12 feet bottom width, 1:1 side slope, and 15 feet depth), and a roughness coefficient of about 0.02 (assumed) could carry about 5,000 cfs (142 cms). That is twice the capacity of the original stream. As the degradation continued the capacity increased, resulting in more and more scour. The increased carrying capacity of the ditch is supported by the fact there is no evidence that the Willow Creek has ever experienced overbank flooding in these two Upper Willow drainage districts after 1920.

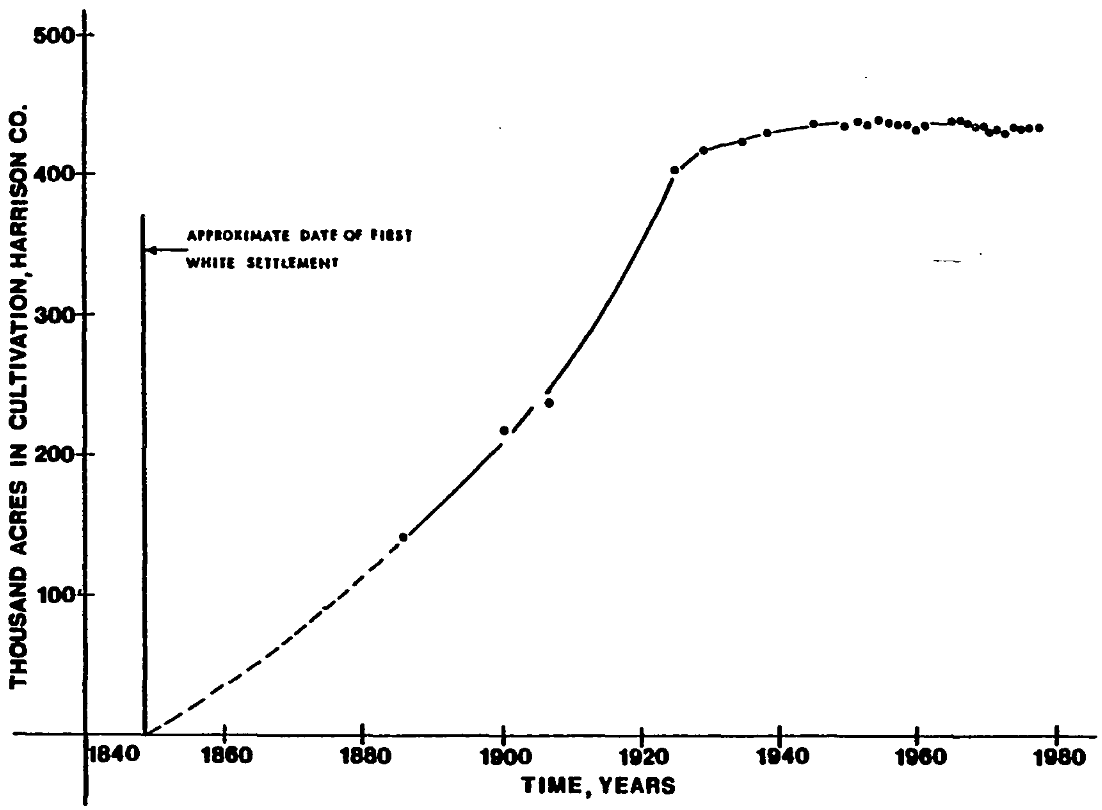


Figure 29. Number of acres put into cultivation with time, Harrison County, Iowa
Source: U.S. Department of Agriculture (after Dirks, 1981)

Also, there is no evidence that any degradation in the Missouri and Boyer Rivers, if any, could have had any effect on the degradation of the Willow Creek, the reason being that the H & P outlet ditch (about 7 miles long) for Willow Creek has been subject to frequent aggradation in the past 70 years. Therefore, there was no way that the degradation of the Upper Willow Creek could have been affected by changes in the regimen of the major rivers.

Observed Mechanics of Degradation

Observations on Willow Creek prior to 1960 (Daniel, 1960; and oral communication during field investigation with Mr. Thomas, retired Harrison County Engineer) have illustrated that the formation and development of the knickpoints (overfalls) along the drainage ditch has been one of the lead factors in the degradation process in the region. A knickpoint is an overfall which is formed on the stream bed and migrates upstream. Its advancement depends on the discharge, location in the stream, and upstream conditions. It can be considered a discontinuity in the sediment transport process. Downstream of the knickpoint, a plunge pool is created, and the wave action of the water in the pool helps remove bed and bank material from the base and sides of the knickpoint. Knickpoints up to 10 feet (3 m) high and plunge pools up to 10 feet (3 m) deep, have been sited by the residents of the Willow Creek valley in the early 1930s (Daniel, 1960; Piest et al., 1977).

No quantitative record on the number and characteristics of the knickpoints is available. As cited by Daniel (1960), the knickpoints were most numerous in the lower part of the ditch in the 1930s (oral communication of Daniel with the residents of the Willow Creek valley during his studies).

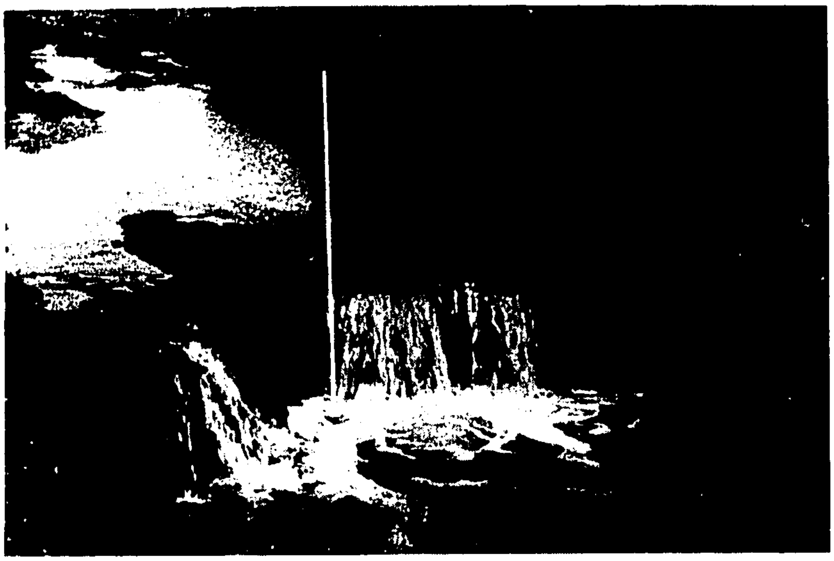
The ultimate destination of the knickpoint cannot be described quantitatively. Usually, in its final stage, the vertical face of the knickpoint becomes flatter, until it converts to a series of riffles and dunes before disappearing.

Figure 30 shows a vertical knickpoint on Willow Creek in 1957. The location of the knickpoint was about 0.5 miles (0.8 km) north of the Monona-Harrison County line (i.e. 0.5 miles north of bridge P; see Figure 36a for location) 1.5 feet (0.46 m) high (Daniel, 1960). The action of water at the plunge pool of the knickpoint can be seen in Figure 31.

After the passage of the knickpoint, the channel bed downstream of the knickpoint starts adjusting to the new conditions. The deepening of the channel is not entirely by the action of the knickpoint; in contrast the major part of the bed scour occurs after the passage of the knickpoint. The downstream adjustment may be distorted if the stream reach is struck by another migrating knickpoint approaching from downstream.

Figure 30. Knickpoint in Willow Creek, September 1957. Location: 0.5 miles north of the Monona-Harrison County line (after Daniel, 1960).

Figure 31. Action of water at the plunge pool of a knickpoint (after Daniel, 1960).



Extent of Degradation in Willow Creek

Recorded changes in the Willow drainage ditch during the past years permit an approximate analysis of the degradation of the stream. The analysis is based on the following data:

1. The county and state bridge records
2. A reconnaissance survey of part of the ditch in 1942, by SCS, U.S. Department of Agriculture
3. Survey of 1958, by Daniel (1960)
4. Survey of the creek by this author, in 1980

In regard to the above data, the following two remarks are made.

First, in the above surveys, the bridge crossings have been the only references for the measurement of the bed elevations. It means that any variations of the stream bed, between the bridges, have been ignored. For instance, the existence or location of knickpoints was not detected in the 1980 field survey.

Second, starting in 1968, the degradation pattern of the stream was distorted by the construction of three flumes (grade stabilization structures) in Willow Creek. The first flume structure was built in 1968 in Monona County, 3 miles (4.8 km) north of the county line. The second and third flume structures were built in Harrison County (see points S and X in Figure 36a). The first flume (S) was built in 1971, and the other one (X) was built in 1973. Due to a serious piping problem, the Monona County flume failed in March 1978, and was completely washed away. The flume was rebuilt in the following year (about a 37 ft,[11.3 m], net drop from headwater to tail water).

Figure 32 and 33 illustrate the flume S as pictured in spring, 1980. After the construction of the flume, the stream channel below the structure has deepened about 2 feet (0.6 m) compared to the stream elevation prior to the construction.

Figure 34 illustrates the Willow Creek channel at a point (bridge Q) 3.3 miles (5.3 km) upstream of flume S. The upstream channel has filled to the crest elevation, and the sediment deposits can be seen in the picture. At this location, about 16 feet (4.9 m) of sediment has been deposited.

Figure 35 shows the stream channel downstream of flume X. At this location, the depth of downstream degradation is about 2 feet (0.6 m) compared to the bed elevation 15 years ago.

The longitudinal profile of the stream at several time periods is illustrated in Figure 36b. The plan view of the stream is also provided in Figure 36a so that the points can be located more easily.

From Figure 36b, it is clearly found that the stream adjustment started from the downstream and advanced upstream with time. Comparison of the stream profiles of 1958 and 1966 illustrates that the stream was approaching equilibrium in its lower reaches. The rate of degradation near the bluff line has been little compared to the upstream reaches.

Investigation of Figure 36 reveals that, at any given time, a rather sharp increase in the local stream bed slope can be recognized. The location of this high gradient reach will shift upstream with time.

The rate of degradation in the reach is higher than the other parts of the stream. This steep front can be considered as a long, flat knickpoint. As time progresses, the erosional front proceeds upstream, and additional vertical channel degradation occurs in the upstream direction.

Figure 32. Upstream view of the flume S as pictured in Spring 1980. Since the time of construction (1971), about 30 feet of sediment has been deposited at this location.

Figure 33. Downstream view of the flume S as pictured in Spring 1980. Note the bank erosion on the left side of the picture. The estimated amount of the additional bed degradation downstream of the flume is about 2 feet during the last 10 years.

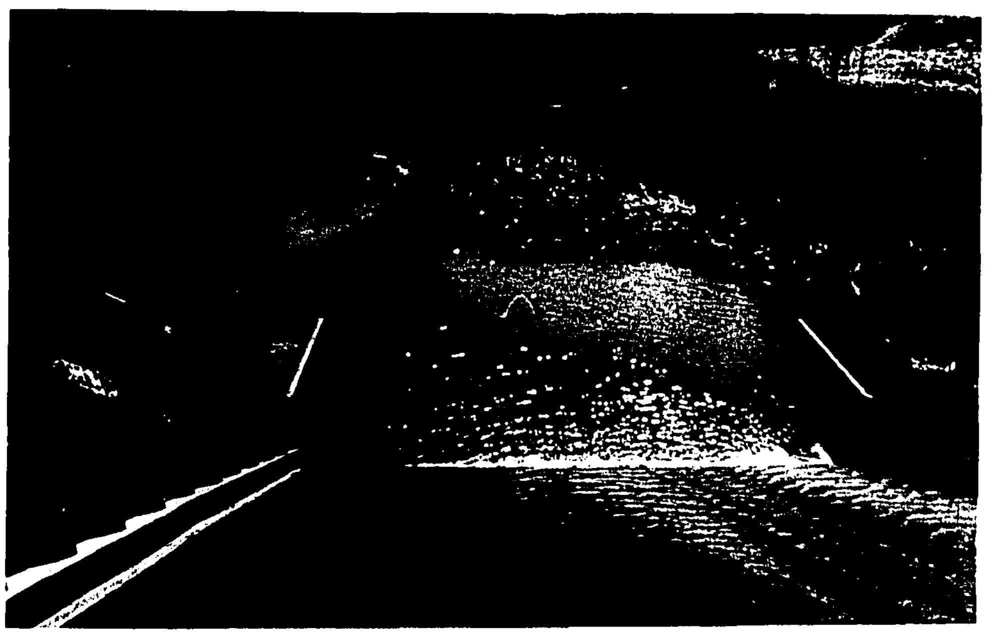


Figure 34. View of the Willow Creek from bridge deck (bridge Q) looking upstream. This location is about 3.3 miles upstream from the flume. The water is backed up, and about 16 feet of sediment has been deposited in the last 10 years.

Figure 35. View of the stilling basin of flume X from the bridge deck (Spring 1980).

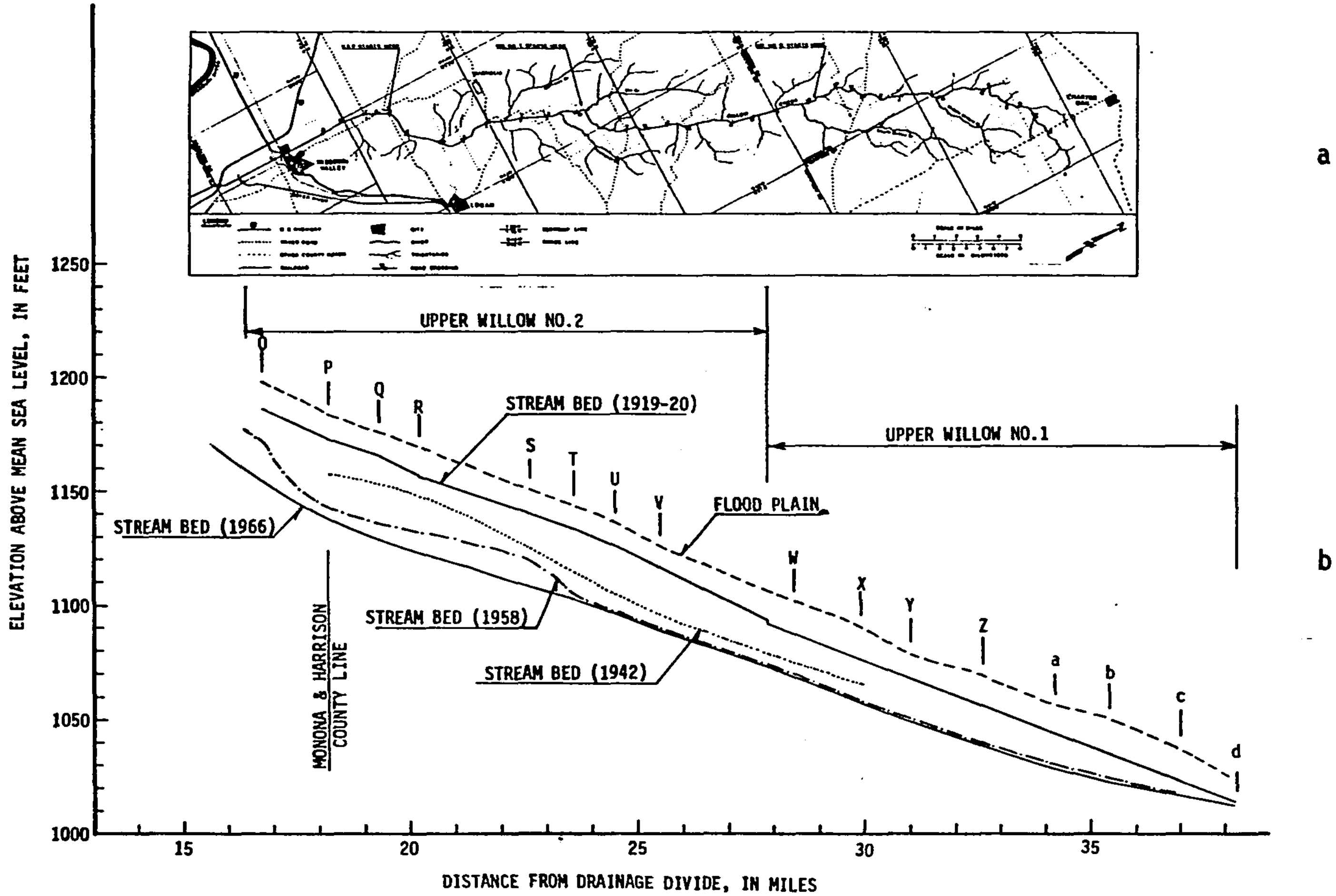


Figure 36. Comparison of the Willow Creek stream bed in different time periods

CHAPTER VI.

PREDICTION OF DEGRADATION

An ideal solution in the evaluation of a degradation problem, in a degrading stream, would be a model which can predict the extent of bed and bank scour at any given time, and for any desired reach of the stream. However, as indicated earlier, it is very difficult to reach this objective because of the uncertainty involved in the formulation of the problem and the vagaries of Mother Nature. In particular, due to the lack of data in the past, the behavior of the stream during transient phases can not be estimated with confidence. For instance, two major variables in degradation studies are the flow and the cross-sectional area characteristics of the stream along its length, which frequently are not available.

For practical applications, the past history of the stream is less important compared to its present and future conditions. The road engineer is concerned with the possible scour around the bridge piers which might eventually undermine the structure. The nearby farmer is concerned with his crop land which is washed away through the stream bank erosion. To control stream degradation, the prediction of maximum potential scour is essential. If the stable longitudinal profile of the stream can be estimated, then remedial works can be planned with a greater confidence, and provision of grade stabilization structures can be made more economically.

Ruhe and Daniels (1965), by analyzing the data of Willow Creek, derived an empirical equation (equation 14 in Chapter II) which relates the depth of degradation to time. The equation is:

$$D = 1.8 + 20.9 \log T \tag{14}$$

where

D = depth of degradation in feet

T = time in years, measured from 1920

One should recognize the limitations of this formula, however, because it is wrong to extrapolate such an empirical expression beyond the range of the original data. In such phenomena, the boundary conditions should be specified. According to equation 14, the degradation will extend indefinitely, which is theoretically impossible, since once the land has eroded to sea level, for example, there will be no more vertical degradation. Practically speaking, there should be an ultimate depth which remains essentially the same, regardless of time.

In another attempt to solve the ultimate degree of degradation, Daniel (1960) and others used Hack's equation (equation 13 in Chapter II) to predict the maximum depth of entrenchment in Willow Creek. Hack (1957), by observations on seven streams in Virginia and Maryland, developed the following formula:

$$B = c - k \ln (L) \tag{13}$$

where

B = the altitude of the reach, ft, mean sea level datum

L = the distance along the stream measured from the head of the stream, miles

c and k are constants

However, the streams used in the above investigation were all in equilibrium and the bed material was completely different from that in Western Iowa. The median size of the bed material for the streams in Virginia and Maryland, reported by Hack, was in the range of gravel and boulders (7 to 600 mm), whereas the streams in Western Iowa are flowing over fine textured material, that is, silt and clay. It is unlikely that this expression developed in the Eastern United States would be suitable for application to Western Iowa streams.

Lohnes et al. (1980) proposed a rational method for the rate of degradation in the following form:

$$\frac{dh}{dt} = -k'h \tag{38}$$

where

dh/dt = rate of degradation

h = elevation of the reach of stream above base level

k' = a constant describing the rate of degradation

However, as the authors explained, this equation also has the limitation that theoretically the channel would never reach equilibrium. However, with time, the rate of degradation decreases markedly.

In the present study, the equilibrium (stable) profile of the Willow Creek is predicted on the basis of hydrological and hydraulic characteristics of the stream. The method uses the stream bed elevation surveyed in 1966, as well as the flood distribution along the stream channel (using a "dominant" discharge), as was discussed in Chapter IV. The methodology can be applied to similar streams in the region.

Method of Prediction

Analysis of the historical degradation data in Chapter V indicated that the entrenchment of the Willow Drainage Ditch has been progressive, and characterized by a process of headward encroachment of an erosional front called a "knickpoint." In places where the stream has little or no flow, the knickpoint becomes almost stationary. On the other hand, at higher flows, the migration rate would increase. Daniel (1960), on July 9, 1957, observed a knickpoint about 0.5 mi (0.8 km) north of the Monona-Harrison County line which was about 1.5 ft (0.4 m) high. The rate of upstream migration of the knickpoint as observed by Daniel was as follows:

> "Between July 9, 1957 and April 27, 1958, the knickpoint moved upstream 85-90 feet and the vertical height increased to three feet. From April 27 to May 1, 1958, the knickpoint moved upstream about 600 feet; it was then essentially stationary until July 1, 1958, but between July 1 and August 15, 1958, moved upstream 1,400 feet before disappearing into a series of riffles."

The above changes in the rate of knickpoint migration should had been related to the discharge variation in the stream. It can be realized that the transient phase of degradation which is accompanied by the presence and migration of the knickpoint is almost impossible to predict theoretically. However, it is believed the stable profile which will be established after the passage of the knickpoint can be predicted.

The stabilization of the reach downstream of a knickpoint starts by the mass movement of the saturated bank material at the sides of the stream. The blocks are moved downward towards the stream bottom and eventually are removed by subsequent discharges. Slump blocks up to 30 feet have been observed in Willow Creek (Daniel, 1960).

Taking into consideration the appropriate hydraulic and hydrologic variables, a method is developed which will generate the stream bed profile under given flow and channel bed conditions. By predicting the final stable profile of the stream, the deviation from the existing profile can be determined. Consequently, provisions for channel stabilization works can subsequently be made to control future potential problems.

In this research, attention is focused on the final equilibrium profile, and a simple computer program developed which can be applied to Willow Creek and to other streams in the same region.

Factors which are considered to be the primary degradation variables, used in developing a computer model, are:

1. Distribution of the magnitude of flood discharge along the length of the stream, for given recurrence intervals
2. Stream geometry

3. Original stream bed elevations

4. Width to depth ratio

5. Channel roughness

6. Bed resistance to scour and critical shear forces and stresses

These will be discussed in the following sections.

Flow distribution along the stream

The magnitude of discharge (flow rate) will increase in the downstream direction for usual flood or dry weather events. The rate of increase for Willow Creek can be predicted either by using the INRC method (Figures 12 and 13), the IDOT method (Figures 17-20), or by eqs. 36 and 37. However, the application of eqs. 36 and 37 has advantage that flow data will be generated for any specified recurrence inter- r-val, and takes into account the effect of land use in the watershed. By inserting different drainage areas into the equation, the discharge distribution along the stream channel can be calculated. Moreover, the equations are computationally preferable, because they easily fit into a digital computer program. Drainage areas for different reaches of Willow Creek are determined by equations given in Table 6.

In the computer program, the discharge is computed at locations one mile apart, and also at points where a major tributary joins the main stream. The variation of discharge inside the interval (within one mile) is ignored, however. The dominant discharge in any interval is assumed to be the flow rate computed at the downstream end of the reach, for the desired recurrence interval.

Stream geometry

At any given station along the stream, the cross-sectional area and the stream bed slope are the two most important geometric characteristics. The response of the stream is usually reflected by changes in these two factors. For degrading channels of substantial width, the deepening of the bed is the major part of the scour; however, for narrow streams in alluvium, deepening and widening can occur simultaneously. To predict the equilibrium profile, the existing longitudinal profile of the stream bed, as well as typical normal cross sections, must be known.

Stream bed elevations The first survey of the stream bed elevations in Willow Creek was performed by Daniel in 1958 (Daniel, 1960). The author used a barometer for the survey, and therefore, due to potential instrumental error, the results are approximate (see Appendix A for detail).

In 1966, the County Engineering Office of Harrison County, Iowa performed a leveling survey, and the stream bed profile was prepared for Upper Willow Nos. 1 and 2. Unfortunately, the elevations were determined at bridge locations only. The data are reported in Table A.1 in Appendix A. For the intervals used in this study, the corresponding elevations were computed by interpolation. By using other miscellaneous bridge records, obtained from IDOT (Iowa Department of Transportation), the 1966 bed profile was extended about four miles farther upstream.

The 1966 longitudinal profile of the stream will be used in a subsequent section as the basis for determining the equilibrium profile.

Cross-sectional area of the stream During the 1966 surveys, transverse cross sections were also obtained at several bridge sites crossing Willow Creek. The corresponding data were made available for this study by the Harrison County Engineer's Office. In the County Engineer's Office, the waterway opening area under the bridge deck is usually reported as the stream cross section. However, in some instances, the bridge deck might be several feet above the flood plain elevation. The elevation difference was determined and subtracted from the original data. Moreover, since several bridges were skewed, the skew angle was estimated and the normal cross sections were computed. Figure 37 shows the results after these modifications were made. Table 10 also lists the top width, depth, and base width of the channel in 1966.

Width to depth ratio The width of the stream channel at the average flood plain elevation, divided by the depth of the stream, is called the "width to depth ratio, $\frac{W}{D}$." The term frequently has been used in the study of alluvial streams; although the ratio will differ from one stream to another, or from one reach to the other. Nevertheless, the relationship between $\frac{W}{D}$ and the distance along the stream channel can be described by a regression equation. The value of $\frac{W}{D}$ at bridge locations was computed and listed in Table 10. The relationship between the $\frac{W}{D}$ ratio and distance from drainage divide is

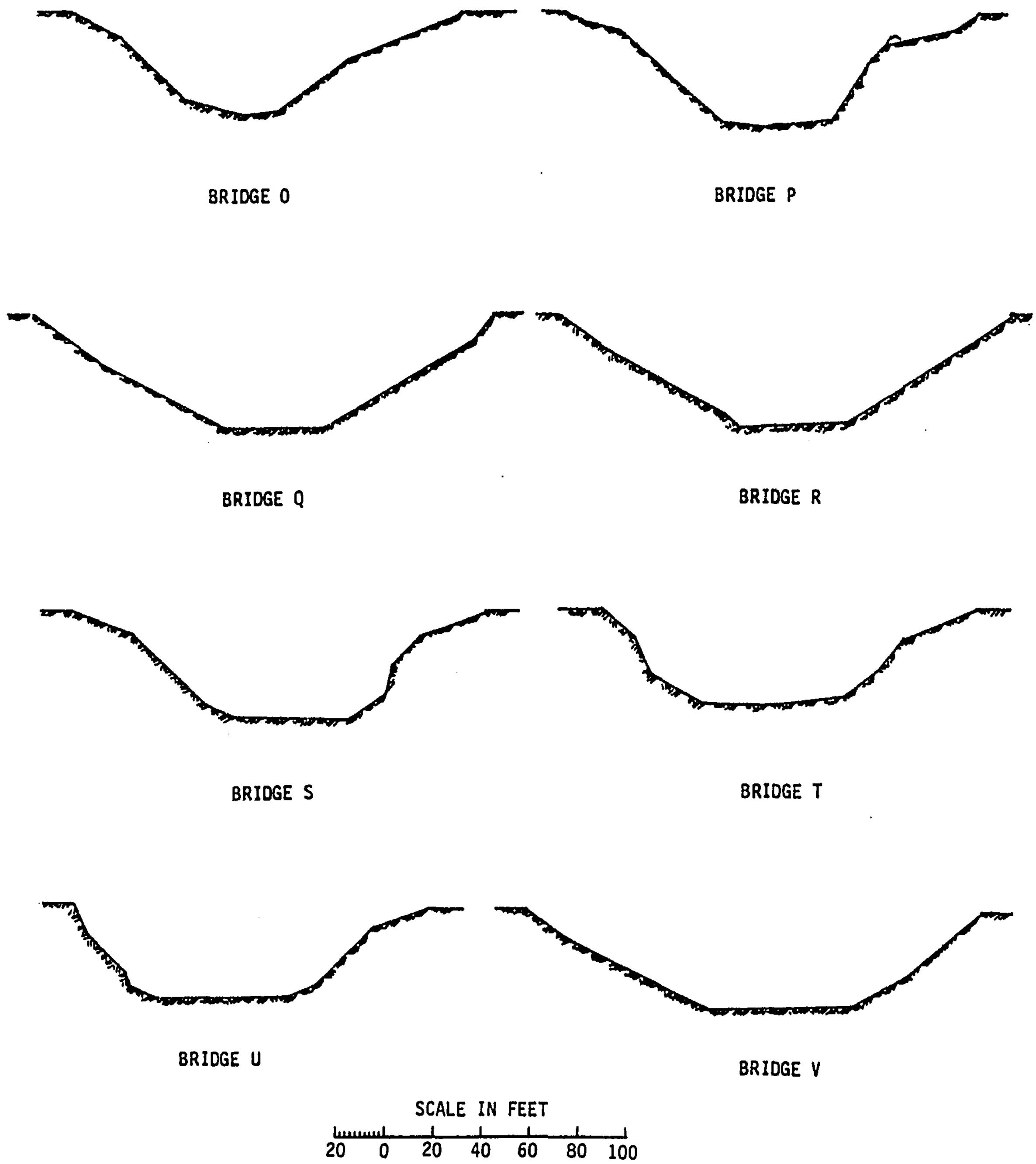


Figure 37a. Cross section of the Willow Creek at bridge crossings in 1966 for bridges O, P, Q, R, S, T, U, and V

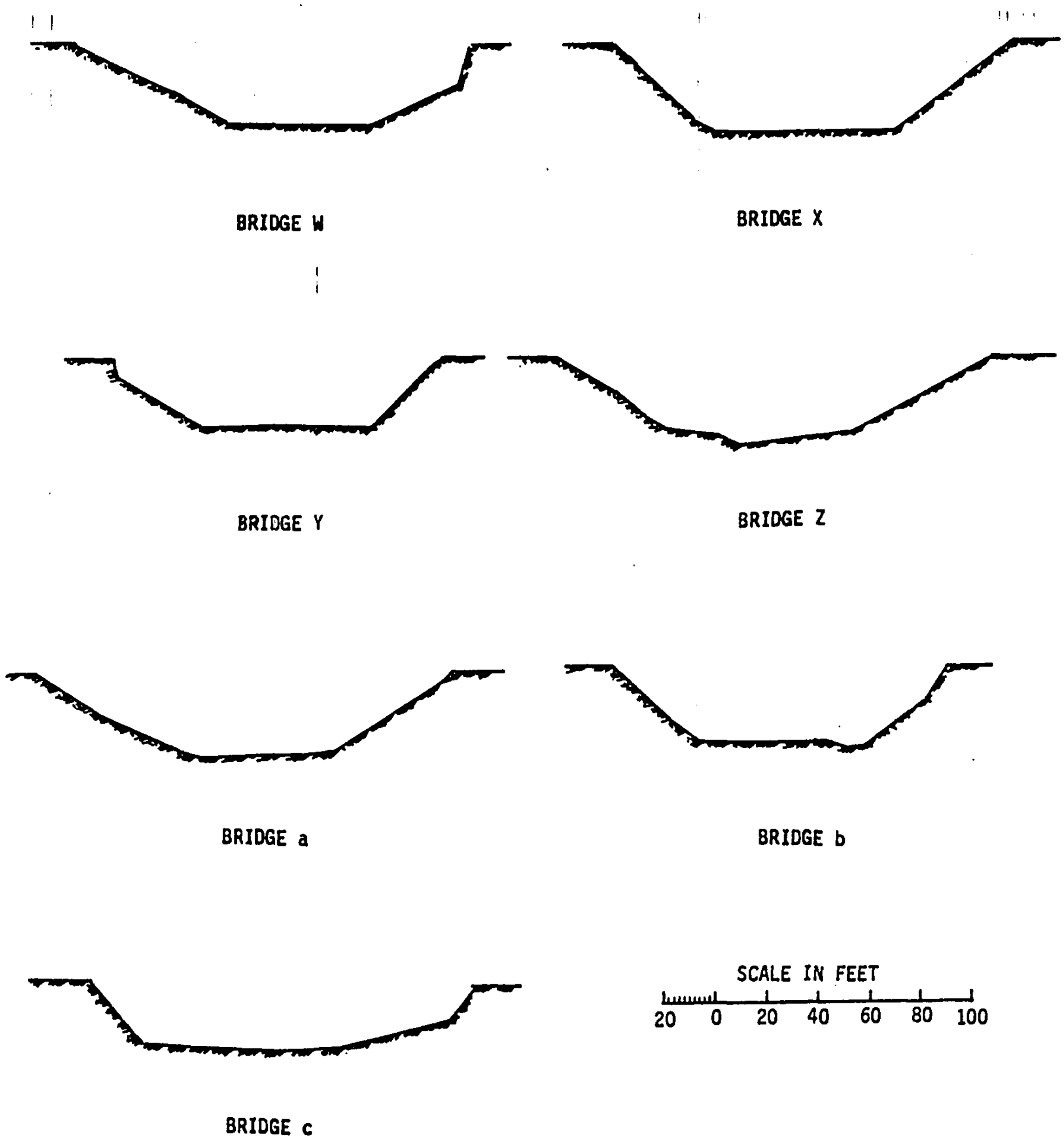


Figure 37b. Cross section of the Willow Creek at bridge crossings in 1966 for bridges W, X, Y, Z, a, b, and c

illustrated in Figure 38. The equation of the best fit line is:

$$\frac{W}{D} = 0.077\ X + 5.23 \tag{39}$$

where X is in miles

Equation 39 provides the value of $\frac{W}{D}$ ratio at any desired location along the stream channel. From the equation, it can be seen that the value of $\frac{W}{D}$ ratio changes along the main channel, however, its value remains the same at any fixed point. The assumption of constant width to depth ratio requires that the bank side slopes remain unchanged as well. This is a simplified assumption, because it seems logical to assume that the stream banks side slopes become steeper at higher depths. If the latter argument can be described quantitatively, an expression for $\frac{W}{D}$ can be established so that the variation of the ratio with depth can be considered in the model formulation.

<u>Stream bottom width</u> Table 10 illustrates also that the stream bottom width, B, has increased in the downstream direction. The plot of B versus the distance is illustrated in Figure 39. The equation of the best fit line is:

$$B = 1.67\ X + 12.79 \tag{40}$$

where B is in feet and X is in miles

This expression will also be used later.

Table 10. Cross section of the Willow Creek at bridge crossing in 1966[a]

Location of bridge	Distance from drainage divide (miles)	Channel top width at flood plain level (feet)	Depth of stream bed below flood plain (ft)	Approx. base width (feet)	Width to depth ratio (W/D)
O	16.74	160	42	39	3.81
P	18.22	171	47	44	3.64
Q	19.33	188	48	44	3.92
R	20.20	190	48	48	3.96
S	22.63	176	46	52	3.83
T	23.56	156	39	50	4.00
U	24.48	148	38	54	3.89
V	25.52	190	42	58	4.52
W	28.44	154	34	58	4.53
X	29.90	156	36	66	4.33
Y	31.01	130	28	66	4.64
Z	32.61	170	36	68	4.72
a	34.20	160	32	66	5.00
b	35.38	130	27	70	4.81
c	36.95	140	26	72	5.38

[a]For metric units, multiply miles by 1.609 to obtain km and feet by 0.305 to obtain meters.

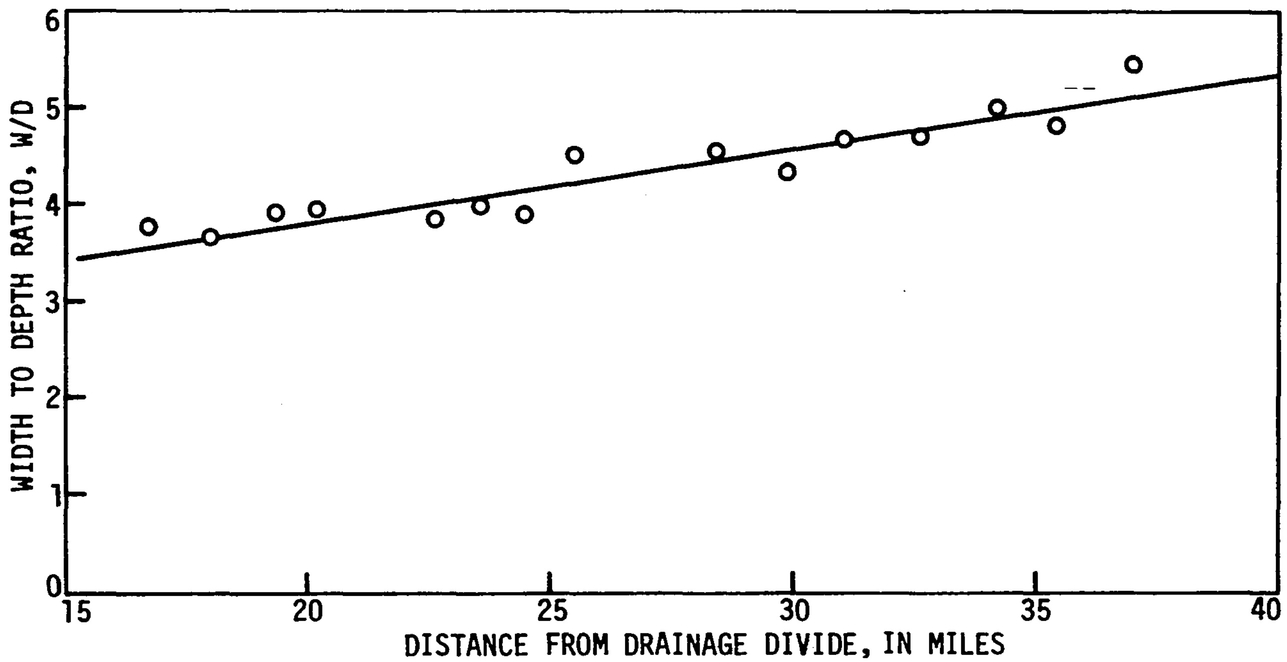


Fig. 38. Relationship between the width to depth ratio and distance in Willow Creek.

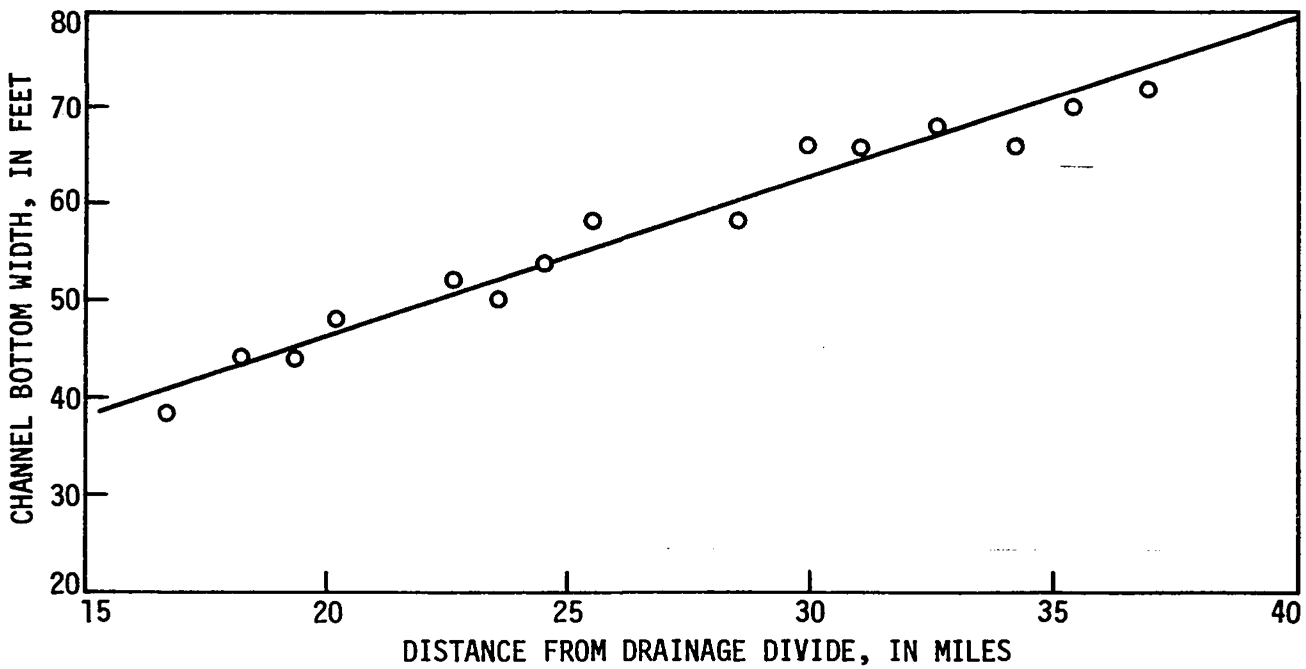


Fig. 39. Relationship between the bottom width and distance

<u>Stream roughness coefficient</u>

All hydraulic computations involving flow in open channels require an evaluation of the roughness characteristics of the channel. There are no resistance diagrams or quantitative expressions available similar to those used in pipe flow problems. Therefore, the selection of the roughness coefficient for natural streams relies fairly well on experience, unless actual flow measurements at characteristic channel locations are obtained, permitting the roughness coefficient to be computed directly (see Brater and King, 1976; Chow, 1959; Graf, 1971; Morris and Wiggert, 1972; and Venard and Street, 1975).

Manning roughness coefficients for 50 stream channels located at different points in the United States have been determined by the U.S. Geological Survey (Barnes, 1967). Combined with color photographs, this reference is a useful tool for selecting the roughness coefficients for natural streams. The comparison of the color photographs from Willow Creek, with those reported in the above reference, indicated that the Manning coefficient should be in the range of 0.030 to 0.040, with a median value of 0.035. Though the roughness coefficient might vary slightly along the stream, however, it is assumed constant in this study. Should the variation be considered, its value should be expressed as a function of distance and incorporated into the model.

<u>Resistance of the bed material to erosion</u>

Water flowing in a stream exerts a shear force on the bed of the stream. The resistance forces which tend to hold the bed particles in place differ from one material to the other. When hydrodynamic forces, acting on a grain of sediment, have reached a critical value, the particle will initiate motion. This state is usually termed the "critical or threshold condition." The initiation of motion by bed material is involved in many hydraulic problems, such as local scour, slope stability, design of stable channels, degradation, and aggradation studies. These problems can be solved only if the threshold condition of sediment motion is estimated.

In Chapter II, it was indicated that the shear stress on the channel bed was expressed by the formula:

$$\tau = \gamma D S \tag{5}$$

where

τ = shear stress (lbs/ft^2, N/m^2)

γ = specific weight of water (lbs/ ft^3, N/m^3)

D = depth of water (ft, m)

S = slope of energy grade line (ft per ft, m/m)

By using this equation, the shear stress exerted on the bed can be evaluated under any desired flow conditions. It should be mentioned, however, that the shear stress value obtained by the above formula predicts the maximum value on the channel periphery. The distribution of the shear stress on the periphery of the channel is not uniform. Lane (1952) demonstrated that in a trapezoidal section, the maximum shear stress is applied on the bed and its value depends on the B/D ratio (where, B = base width, and D = depth of water). Figure 40, which is taken from Raudkivi (1976), illustrates the shear stress distribution (as determined by Lane) on the periphery of a trapezoidal channel: τ_{max} equals 0.89, 0.97, and 0.99 times $\tau = \gamma\ D\ S$ for $\frac{B}{D}$ = 2, 4, and 8, respectively. However, the maximum value on the sides equals 0.735, 0.75, and 0.76 times $\tau = \gamma\ D\ S$, respectively. In the present study, the value of $\tau = \gamma\ D\ S$ will be considered as the maximum value for beginning of motion as a simplifying assumption.

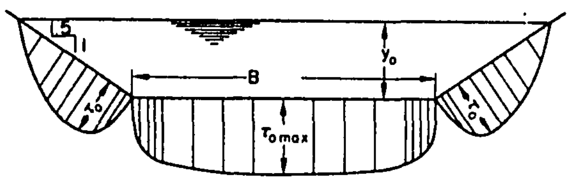


Figure 40. Shear stress distribution over the periphery of a trapezoidal channel (after Lane, 1952)

Critical shear stress, τ_c

Estimation of applied shear force on the stream bed, obtained from equation 2, is a direct computation. Under any flow condition, the equation provides the value of shear stress on the stream bed. However, the major problem is to evaluate the critical value of the shear stress. Efforts have been made to relate the capacity of soil to withstand the drag force to the type of soil, the purpose being to prepare tables or charts which would specify the permissible shear force for any kind of material. The trend of research has been different for cohesive and for noncohesive soils (see also The Asphalt Institute, 1978).

Critical shear stress for noncohesive materials

For this type of material, the main resistance to erosion is provided by the submerged weight of the sediment derived from gravity forces. For this reason, researchers have tried to relate the critical shear stress to the particle size. However, the available information is not quite conclusive or adequate. For example, there is no agreement when dealing with nonuniform materials. To illustrate this deficiency, Figure 41 (which is taken from Graf, 1971) clearly indicates the uncertainty in the selection of the critical shear stress. Moreover, Figure 41 and other similar charts will not provide the critical tractive force for fine bed materials (<0.1 mm).

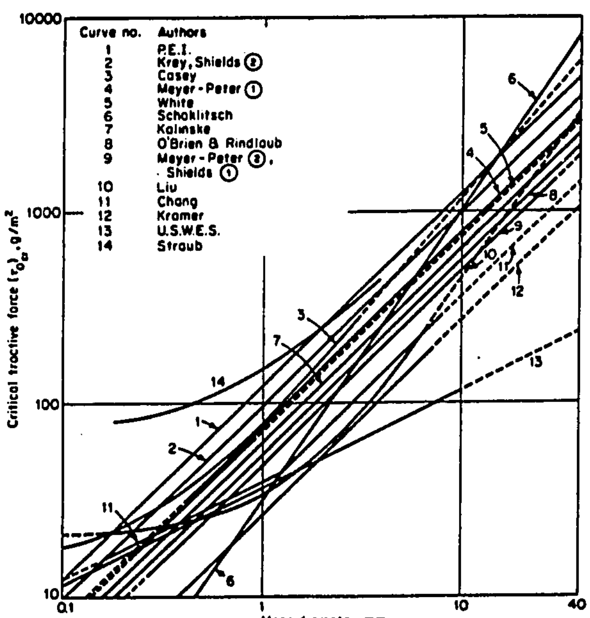


Figure 41. Critical shear stress as function of grain diameter; a comparison (after Graf, 1971)

Critical shear stress for cohesive materials

In cohesive soils, a portion of the soil mass consists of fine particles in the range of silt and clay size. In noncohesive soils, the resistance to erosion is basically dependent on the grain size. However, for cohesive soils, the mineralogy of clay, the chemistry of eroding water, and electro-chemical complexities involved can all alter the resistance of the soil to erosion. Little progress has been made in the definition and evaluation of the erosion resistance of cohesive soils. Even the laboratory experiments on cohesive soils followed, by many years, the first work on noncohesive materials. As Raudkivi (1976) stated: "it is not surprising because the problem is outside the field of interest of the physical chemist and clay mineralogist, and too complex for the civil engineer." For cohesive soils, the value of critical tractive force would be increased, but there is no functional relationship which related cohesion and grain size (Raudkivi, 1976).

A few studies have been made to correlate the critical shear stress of cohesive soils to different soil properties. These properties include: plasticity index, void ratio, percent clay, etc. (Dunn, 1959; Smerdon and Beasley, 1961; Flaxman, 1963; Moore and Masch, 1962). Still, the knowledge of this phase of sedimentation is in a primitive state. Experimental results have been reported in terms of regression equations or charts which, at best, would provide critical shear stress value under special flow boundary conditions. In most experiments on cohesive soils, the following soil properties have been used. They are defined here to provide more clarification:

1. Liquid limit, L_ℓ, is the moisture content, in percentage by weight, of oven-dried soil at which the soil will just start to flow when jarred slightly.
2. Plastic limit, P_ℓ, is the lowest moisture content, in percentage by weight, of oven-dried soil at which the soil can be rolled into 3 mm diameter threads without breaking into pieces.
3. Plasticity index, PI, is the difference, $L_\ell - P_\ell$. It is the range of moisture content in which the soil is plastic. When $P_\ell > L_\ell$, the plasticity index is taken as zero.
4. Vane shear strength, S_v, is the ultimate strength of the soil specimen while tested by a vane shear device.
5. Void ratio, e, is the ratio of void volume to solid volume.

Dunn (1959) conducted a laboratory study on soil samples taken from channels in Nebraska, Wyoming, and Colorado. The author subjected the samples to erosion by a submerged vertical water jet impinging on the specimen. The critical condition was defined by the author as the flow at which the water became cloudy and no subsequent clearing occurred. A plot of the critical shear stress versus the vane shear strength, S_v, gave a straight line. The shear stress was also related to the plasticity index. The value of the PI was in the range of 6 to 16, resulting in τ_c being in the range of 0.35 to 0.50 lb/ft^2. The effect of the plasticity index on the critical tractive shear force has also been investigated by other researchers.

For example, Smerdon and Beasley (1961) correlated the shear stress to PI and determined τ_c for eleven cohesive farm soils from Missouri ranging from silty loam to clay, by placing them in a tilting hydraulic flume. The following regression equation was obtained:

$$\tau_c = 0.0034\ (PI)^{0.84} \tag{41}$$

where

τ_c = critical shear stress in lb/ft^2

PI = plasticity index

This equation, for the range of PI used in the experiments (10-20), will give the critical shear stress. For their experiments, τ_c ranged from 0.02 - 0.047 lb/ft^2.

It is surprising that Dunn, for almost the same range of PI, obtained much higher values. The two investigations led to critical shear stress differing by a factor of 10 to 15.

Flaxman (1963) examined a number of natural streams, studying the plasticity index, and found that some soils of low plasticity (or in one or two cases, no plasticity) were exhibiting considerable resistance to erosion.

Partheniades and Paaswell (1970) reported that for ephemeral streams, the critical shear stress was in the range of 0.4 to 0.6 lb/ft^2. The authors indicated that the value could be as high as 1.2, with good vegetation cover and an intense root system.

The clay content will have a pronounced effect on the soil erodibility. Smerdon and Beasley (1961) showed that the critical shear stress increased with increasing percent clay content.

Since the void ratio of bed material is an indication of the bed compaction, it is reasonable to believe that the critical shear stress would increase with decreasing void ratio. This has been verified by Lyle and Smerdon (1965) in their experiments. A summary on the subject has been presented by the Task Committee on Erosion of Cohesive Materials, ASCE (1968).

Summary comments

Research through the literature dealing with critical shear stress of cohesive soil has revealed that the critical shear stress will not depend merely on PI or any other single variable. The interaction of several important variables must be considered simultaneously. Erodibility of the soil in-situ can be greatly different from the same, but disturbed sample in the laboratory. The research performed on soil specimens, with specially designed test equipment, has provided to date no general conclusion or solution. The available charts and regression equations are useful tools, at the most, for qualitative interpretation.

On the other hand, the value of critical shear stress is an important criterion which is to be evaluated as accurately as possible The value for Willow Creek was estimated on the basis of hydraulic and geomorphic characteristics of the original Willow Creek. Partial dependency on τ_c for noncohesive bed material was necessary, but supported by the fact that the bed material usually is wet (saturated) and cohesiveness may be reduced substantially in such a reworked loess (silt) situation.

Estimation of the Critical Shear Stress

Original channel

The original Willow Creek was a meandering stream, and was not degrading prior to straightening. Price, in a survey of the Willow Creek in 1916, indicated that the river was aggrading in the lower part of Upper Willow No. 1 (Harrison County, Iowa, Drainage Record No. 7, P. 604; on file at the Drainage Clerk's Office, Court House, Harrison County, Iowa). Thus, it will be reasonable to assume that the original stream was, at least, in vertical equilibrium. However, this does not exclude the possibility of lateral migration or meandering. Based on this assumption, the critical condition for sediment movement in Willow Creek can be evaluated.

Investigation of data shown in Figure 4, and listed in Table 1 in Chapter III, revealed that usable information was available. From Figure 4, it is observed that the stream had almost a uniform slope in Upper Willow No. 2 ($S = 0.12\%$). The data in Table 1 will also indicate that the stream had almost a uniform cross section. If we approximate the cross section of the old stream by a trapezoid, the following figure will represent the average cross-sectional area of the stream prior to straightening.

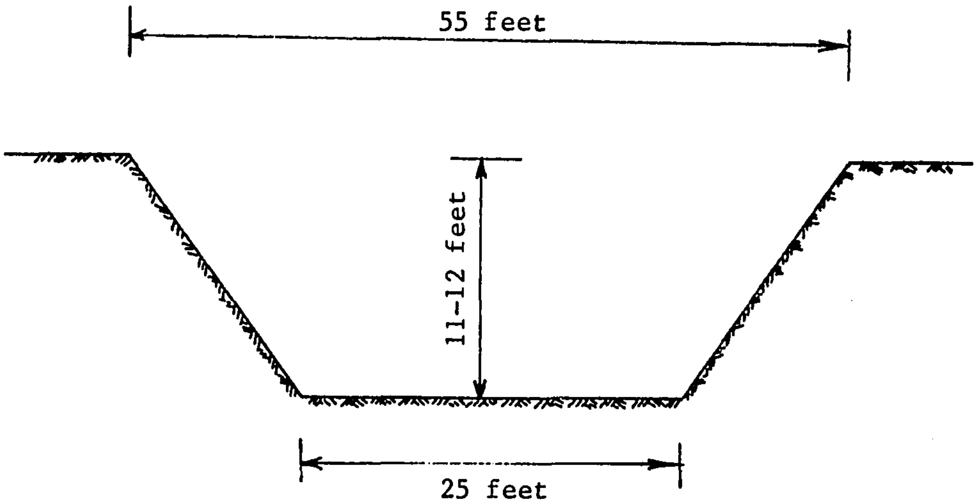


Figure 42. Approximate average cross section of the original Willow Creek in Upper Willow No. 2

Determining critical shear stress

Assuming that the roughness characteristics of the old stream were similar to that of the existing creek (i.e. about 0.035), the bankfull capacity of the old stream can be estimated, using the Manning equation.

$$Q = \frac{1.49}{n} \; A R^{2/3} S^{1/2} \tag{3}$$

By substituting the data in the formula, the bankfull discharge is calculated to be in the range of 2,200 to 2,700 cfs (62-76 cms). Due to the meandering nature of the stream, and corresponding energy losses, it is believed that the actual bankfull discharge might had been even less than this discharge. This illustrates why the Willow Creek flood plain was frequently innundated by flood waters, prior to straightening.

Considering the flow conditions at bankfull stage, the value of critical shear stress can be evaluated, approximately, using equation 5 in Chapter II.

$$\tau = \gamma \; D \; S \tag{5}$$

If the substitution is made, then

$$\tau = (62.4 \; \frac{lb}{ft^3}) \; (11.5 \; ft) \; (0.0012 \; \frac{ft}{ft})$$

$$\tau = 0.85 \; \frac{lb}{ft^2} \quad \text{or} \quad 40.70 \; N/m^2$$

In subsequent computations, it will be assumed that the value obtained here is equivalent to the critical value.

Description of the Degradation Model

Background

As it was indicated earlier, Willow Creek has widened and deepened simultaneously. This makes the problem more difficult, compared to wide streams, in which the deepening is the major or, in most cases, the only component of degradation. Therefore, in the prediction method which is developed here, the lateral changes in the channel section have been incorporated implicitly into the model (applicable to straightened streams).

In the development of the prediction model, the following assumptions have been made:

First, the value of the width to depth ratio at a given location is constant, regardless of the depth of degradation. Although this assumption might not be satisfied completely, it is expected that it changes around a mean value, as obtained by equation 39. The assumption of constant $\frac{W}{D}$ requires the side slopes to remain unchanged.

In natural streams, it is very difficult to find uniform cross sections in a selected reach. Even in a long and straight reach, two cross sections will never be exactly the same. Moreover, the configuration of the stream cross section would seldom if ever match a regular geometric shape, such as: trapezoidal, rectangular, etc. A "representative" cross section is usually selected. Moreover, in engineering practice, the form of the stream cross section frequently is approximated by a regular geometric shape. This assumption greatly simplifies the problem.

For Willow Creek, the observations in the field have revealed that a trapezoid is an adequate representation of the stream cross section. Although there are some changes in the side slopes along the stream length, a side slope of 1:1 is believed to be most representative. Figures 43 and 44 illustrate the shape of the stream

Figure 43. Cross section of the Willow Creek at bridge O. The direction of flow is from top to bottom.

Figure 44. Cross section of the Willow Creek at bridge U. The direction of flow is from bottom to top.

cross section at two locations. In some locations, the banks near the top become almost vertical, however, the choice of a trapezoid for the lower part of the section, which is important in hydraulic calculations, is still a valid assumption.

Second, the bottom width of the stream in 1966 at any desired location can be estimated by equation 40. This appears to be a reasonably valid assumption, using the available field data. The reason for selecting the 1966 data is the fact that the regimen of the stream distorted after the construction of the flume structures during 1968-1978. Therefore, the latest survey (1980) can not be used directly for degradation computations.

Third, the critical shear stress, τ_c, of the stream bed material is assumed to be equal to $0.85 \frac{lb}{ft^2}$ as was estimated from the original Willow Creek data. It is assumed that the critical shear stress value is the same for the entire length of the stream, even in the degraded position.

To determine the kind of existing bed material in terms of sediment size, 14 soil samples were collected from different locations along the stream, and were analyzed for particle size distribution. The complete results are reported in Appendix B. The uniformity among the particle size distribution curves is an indication that the resistance to erosion is reasonably uniform along the stream. Figure B.16, which is considered to be representative of the size distribution, indicates that the bed material consists mainly of fine materials in the silt and clay range.

It is understood that the value of critical shear stress might change with depth. The soil resistance to erosion might become higher in deep layers. However, no quantitative relationship can be established and, for the present study, the value of critical shear stress is assumed to be constant for all depths.

Fourth, the starting point in the stream, for evaluating the future equilibrium profile, is the matching point between degradation in the Upper Willow and aggradation in the H & P drainage ditch. A 26 mile reach (41.8 km) was studied. This length was selected because the data for stream bed elevation were not available beyond this length. In the computational procedure to be carried out with a digital computer, the stream length was divided into 27 stations. The sections were in general, one mile (1.61 km) apart, except when a major tributary joined the main channel. Then the section was placed at the confluence point.

Fifth, it is also assumed that natural channel stabilization will be established by the change in the slope and the enlargement of the cross section only. In other words, a coarsening of the bed material will not occur. This is supported by the investigation of the particle size distribution curves in Appendix B. The results indicate that, in all cases, the amount of coarse material is quite small. Only a small percentage of fine and medium sand is indicated, which is not coarse enough to achieve an armoring of the bed. In contrast, the size of sand mixture can be eroded more easily than the parent cohesive material.

Sixth, the Manning roughness coefficient is assumed to be equal to 0.035 and remains unchanged during the course of degradation.

Seventh, the average land use and slope factor for the drainage basin is assumed to be equal to 0.80, which is believed to be a representative value.

Computational steps

The computational steps for the given input data, are as follows:

Initial cross section   The cross section of the stream at each location is estimated using equation 40. This equation will generate the base width of the channel by which the trapezoidal shape with the assumed 1:1 side slopes can be constructed.

Initial stream bed elevation   The initial stream bed elevation at each section is calculated from the 1966 longitudinal profile by interpolation. The distance, drainage area, and stream bed elevation at selected sections are listed in Table 11.

Table 11. Distance, drainage area, and elevation at selected sections[a]

Section No.	Stream bed elevation (1966, feet)	Distance from drainage divide (miles)	Drainage area (square miles)
1	1229.0	12.00	26.41
2	1214.0	13.00	28.22
3	1197.0	14.00	30.03
4	1178.0	15.00	31.84
5	1164.5	16.00	33.65

[a]To convert to metric units, multiply feet by 0.305 for meters, miles by 1.609 for km, and square miles by 2.59 for square kilometers.

Table 11. continued

Section No.	Stream bed elevation (1966 feet)	Distance from drainage divide (miles)	Drainage area (square miles)
6	1150.5	17.10	48.25
7	1139.5	18.00	52.06
8	1130.5	19.00	55.75
9	1124.0	20.00	59.06
10	1118.0	21.00	62.08
11	1112.0	22.00	64.87
12	1106.5	23.00	67.48
13	1099.5	24.00	69.95
14	1093.0	25.00	72.29
15	1086.5	26.00	74.52
16	1079.5	27.00	76.65
17	1074.0	27.80	87.42
18	1063.0	29.20	97.27
19	1057.0	30.00	98.20
20	1049.5	31.00	99.36
21	1043.0	31.90	114.01
22	1035.5	33.00	116.78
23	1029.5	34.00	119.30
24	1024.5	35.00	121.82
25	1020.5	36.00	124.34
26	1016.5	37.00	126.86
27	1012.5	38.25	130.01

Discharge For any selected recurrence interval, the distribution of discharge along the length of the channel is computed using equation 37. The variation of flow inside the length increment (one mile, 1.6 km) is ignored, and the computed discharge at each location will be used to compute the normal depth in the upstream reach. That is, the maximum discharge in each interval will be utilized in the computations that follow. Note that $A > 10,000$ acres for all sections.

In the discussion about the dominant discharge in Chapter IV, it was realized that there was some agreement, for alluvial streams in regimen, the bankfull flow could be considered as dominant. The bankfull discharge for the original Willow Creek was estimated to be approximately equal to 2,500 cfs (71 cms). The calculation was based on the geometric characteristics of part of the stream which is now Upper Willow No. 2. The drainage area in Upper Willow No. 2 changes from a minimum value of 34.7 square miles (89.7 sq. km) to a maximum value of 87.4 square miles (226 sq. km). By using equation 37, the recurrence interval for the bankfull discharge of the original stream can be estimated.

Assume a value of 0.80 for land use and slope factor. Then, for the high and low values of drainage areas (i.e. 87.4 and 34.7 square miles), the result would be (for the 1915 prestraightening condition):

$$Q = 422.58\ (LF)\ (RI)^{0.301}\ (D_a)^{0.504} \tag{37}$$

Substitute the correct numerical values, then

$$2{,}500\ \text{cfs} = 422.58\ (0.80)\ (RI)^{0.301}\ (34.7)^{.504}$$

or

$$RI = 2\ \text{years}$$

Similarly, for the maximum value of drainage area,

$$2{,}500\ \text{cfs} = 422.58\ (0.80)\ (RI)^{.301}\ (87.4)^{.504}$$

or

$$RI = 0.43\ \text{years}$$

where

RI = 0.43, and 2 years are the recurrence intervals of the bankfull discharge at the lower and upper ends respectively.

From the above simple calculation, it can be seen that for the lower parts of the stream (Upper Willow No. 1), the recurrence interval for 2,500 cfs would be even less than 0.43 years.

From the above discussion, it is clear that the dominant discharge for the Willow Creek should be greater than Q_2. But what discharge dominates the stream regime, Q_5, Q_{10} . . .? Unfortunately, no direct answer can be given. However, based on the following argument, a reasonable value of dominant discharge can be selected for Willow Creek.

From the stream bed changes recorded in the past years (1958, 1966, 1980, and other miscellaneous bridge records), it has been observed that the longitudinal profile of the stream in the most downstream reach has been almost stable during the last 35 years. The length of this reach is about 5-6 miles (8-9.7 km), located upstream of the Logan gaging station. Keeping this in mind, it can be argued that the dominant discharge would be the one that generates a stable bed that is most similar to the existing stable profile in this reach. Thus, by trying several discharges in the computer model (Q_2, Q_5, Q_{10}, etc.), the appropriate value can be selected by comparison. This frequency and magnitude of peak discharge can then be used to estimate the future stabilized bed profile, if no additional grade control structures are used.

Method of computation The computation of degradation starts at the downstream end of a reach, and is computed in the upstream direction. The initial starting point is called the "fixed point," and its elevation will not change. As it was discussed in Chapter V, the lower end of Upper Willow No. 1 has such a character because no considerable change has been observed during the last 60 years. This point is bridge "d" which is shown in Figures 5 and 32a, and is a point near the bluff line.

Iterative routine The channel intervals illustrated in Table 11 are checked next against scour criteria, starting from section 26 (miles 37-38.25). This section is examined to determine if it is stable under the given flow and cross section. The applied shear stress is computed using equation 5, ($\tau = \gamma$ D S). If the scour criteria are not satisfied, (i.e. $\tau > \tau_c$), the bed elevation at the section is lowered by a depth increment ΔD. Lowering the stream bed at the section will result in a decrease in the channel slope in the reach being evaluated. Due to the drop in bed elevation, the channel cross section will increase for a stated water surface elevation. The combination of new slope and cross section will result in a lower value of applied shear stress. If the new value of shear stress, τ, is greater than τ_c, the iteration will continue until $\tau < \tau_c$. Then the computation will shift to the next reach or stream section.

Change in slope If the slope in trial i is S_i, the channel slope in the next trial would be

$$S_{i+1} = S_i - \frac{\Delta D}{\Delta L} \qquad (42)$$

where ΔD is the amount of degradation and ΔL is the distance increment. By using this equation, the change in the bed slope can be calculated in the computer program.

<u>Change in cross section</u> The section enlargement in each iteration is illustrated in Figure 45.

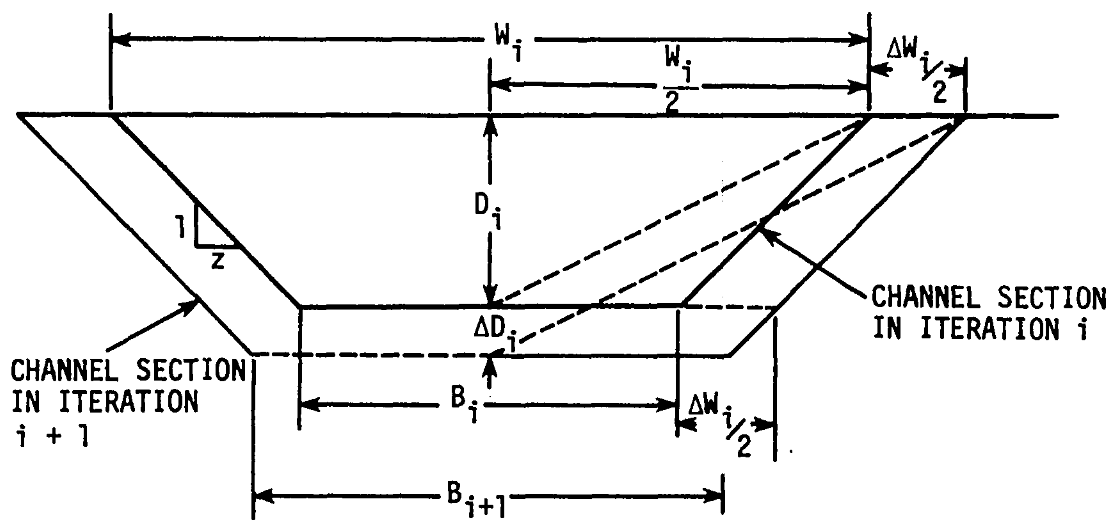


Figure 45. Widening of the channel cross section during degradation

Assuming that width to depth ratio is constant, if the stream bed is scoured by ΔD feet (see Figure 45), it can be shown that the increase in bottom width would be

$$\Delta B_i = \Delta D \left(\frac{W}{D} - 2Z\right) \tag{43}$$

where

ΔB_i = the increase in the bottom width in ith trial, feet

ΔD = the depth of scour in each iteration, assumed equal to 0.25 feet

$\frac{W}{D}$ = width to depth ratio

Z = side slope (Z:1, horizontal to vertical)

We have already assumed Z = 1, therefore

$$\Delta B_i = \Delta D \left(\frac{W}{D} - 2\right) \tag{44}$$

and the new bottom width would become

$$B_{i+1} = B_i + \Delta D \left(\frac{W}{D} - 2\right) \tag{45}$$

Equation 45 will provide a new and larger cross section, which will be used in the next trial iteration.

After the first channel interval has reached equilibrium, the next upstream reach increment is examined. Even if the second reach had been in equilibrium before the degradation occurred in the first interval, due to the lowering of the downstream bed elevation, the increased slope will create a larger shear stress. The channel will then lose its initial stability. The same procedure will be followed for the entire length of the stream channel until the equilibrium is reached. At the equilibrium, the stream would have a larger cross section and a flatter slope at different reaches. As the computations proceed upstream, the selected dominant discharge is decreasing in magnitude (eq. 37).

Sensitivity of the model

Under the given stream geometry, the stream bed elevations for the final equilibrium profile depend on the discharge selected for the "dominant" discharge, the Manning's roughness coefficient, land use, and the critical shear stress. To observe the response of the stream degradation model under different conditions, several combinations of these controlling variables were selected, and are listed in Table 12.

Table 12. Variation of controlling variables in the input data for the channel degradation model

Discharge, frequency	Manning roughness coefficient	Critical shear stress, lbs/ft^2	Land use and slope factor
Q_2	0.035	0.85	0.80
Q_5	0.025	0.85	0.80
	0.035	0.85	0.80
	0.045	0.85	0.80
Q_{10}	0.035	0.85	0.80

The computer output and final elevations of the future stable profile for each of the above combinations are reported in Appendix C.

As it is seen from the table, the Manning roughness coefficient is the only variable which was changed in the model. However, it does not exclude the possibility of changing other variables, such as, critical shear stress and land use factors. If the bed material changes along the stream channel, or even with the depth, these variations can be incorporated into the computer model quite easily.

Results and Discussion

A complete listing of the computer program as well as the definition of variables are reported in Appendix C. The computer results for the selected combinations are also listed in the Appendix. For each set of data, three pages of computer print out are included. The first page shows selected values for: the total number of selected sections, Manning roughness coefficient, land use factor, recurrence interval, scour increment (depth) used in the iteration, and critical shear stress. Except for the recurrence interval and roughness coefficient, the other variables are held the same in all cases. The second page of the computer output is a list of initial stream bed elevations, distance from drainage divide, drainage area, and the computed discharge at each section. In the third page, the predicted equilibrium profile is tabulated.

The results are summarized in Tables 13 and 14. One must read up from the bottom of the table, if one wishes to picture the stream bed degradation as progressing in the upstream direction. Using the data in these tables, three equilibrium profiles for Q_2, Q_5, and Q_{10} were plotted, and are shown in Figure 46. Except for the discharge, the other variables are the same in the three cases, making the comparison much easier. The stream bed profiles for 1966 and 1980 conditions are also illustrated in the figure. The alphabetical symbols in the figure designate the location of the bridge crossings, as explained in Appendix

Table 13. Final equilibrium profile of Willow Creek for Q_2 and Q_{10}[a]

Manning roughness coefficient, n = 0.035
Critical shear stress, τ_c = 0.85 lb/ft^2 (40.7 N/m^2)
Land use and slope factor = 0.80

Distance from drainage divide (miles)	Stream bed elevation (1966, feet)	Q_2 Discharge (cfs)	Q_2 Stable slope	Q_2 Final bed elevation (feet)	Q_{10} Discharge (cfs)	Q_{10} Stable slope	Q_{10} Final bed elevation (feet)
12.00	1229.0	2169	0.00246	1204.3	3520	0.00213	1184.5
13.00	1214.0	2242	0.00241	1191.3	3640	0.00213	1173.3
14.00	1197.0	2314	0.00232	1178.5	3756	0.00199	1162.0
15.00	1178.0	2383	0.00204	1166.3	3868	0.00180	1151.5
16.00	1164.5	2450	0.00172	1155.5	3977	0.00146	1142.0
17.10	1150.5	2938	0.00158	1145.5	4770	0.00132	1133.5
18.00	1139.5	3053	0.00142	1138.0	4956	0.00123	1127.3
19.00	1130.5	3160	0.00123	1130.5	5130	0.00114	1120.8
20.00	1124.0	3253	0.00114	1124.0	5281	0.00114	1114.8
21.00	1118.0	3336	0.00114	1118.0	5415	0.00118	1108.8
22.00	1112.0	3411	0.00104	1112.0	5537	0.00118	1102.5
23.00	1106.5	3480	0.00133	1106.5	5648	0.00123	1096.3
24.00	1099.5	3543	0.00123	1099.5	5751	0.00123	1089.8
25.00	1093.0	3602	0.00123	1093.0	5847	0.00123	1083.3
26.00	1086.5	3658	0.00133	1086.5	5938	0.00128	1076.8
27.00	1079.5	3710	0.00142	1079.5	6023	0.00124	1070.0
27.80	1074.0	3965	0.00142	1073.5	6435	0.00118	1064.8
29.20	1063.0	4184	0.00142	1063.0	6791	0.00112	1056.0
30.00	1057.0	4204	0.00142	1057.0	6824	0.00114	1051.3

[a]To convert to metric units, multiply feet by 0.305 for meters, miles by 1.609 for km, and cfs by 0.0283 for cms.

Table 13. continued

Manning roughness coefficient, n = 0.035
Critical shear stress, τ_c = 0.85 lb/ft^2 (40.7 N/m^2)
Land use and slope factor = 0.80

Distance from drainage divide (miles)	Stream bed elevation (1966, feet)	Q_2 Discharge (cfs)	Q_2 Stable slope	Q_2 Final bed elevation (feet)	Q_{10} Discharge (cfs)	Q_{10} Stable slope	Q_{10} Final bed elevation (feet)
31.00	1049.5	4229	0.00137	1049.5	6864	0.00105	1045.3
31.90	1043.0	4532	0.00129	1043.0	7357	0.00099	1040.3
33.00	1035.5	4587	0.00114	1035.5	7447	0.00095	1034.5
34.00	1029.5	4637	0.00095	1029.5	7527	0.00095	1029.5
35.00	1024.5	4686	0.00076	1024.5	7607	0.00076	1024.5
36.00	1020.5	4735	0.00076	1020.5	7686	0.00076	1020.5
37.00	1016.5	4783	0.00061	1016.5	7764	0.00061	1016.5
38.25	1012.5	4842			7860		

Table 14. Final equilibrium profile of Willow Creek, for Q_5 [a]

Distance from drainage divide (miles)	Stream bed elevation (1966, ft)	Discharge (cfs)	Critical shear stress, $\tau_c = 0.85$ lb/ft^2 (40.7 N/m^2) Land use and slope factor = 0.80					
			Roughness coefficient n = 0.025		Roughness coefficient n = 0.035		Roughness coefficient n = 0.045	
			Stable slope	Final bed elevation (feet)	Stable slope	Final bed elevation (feet)	Stable slope	Final bed elevation
12.00	1229.0	2857	0.00256	1206.5	0.00227	1193.8	0.00213	1182.5
13.00	1214.0	2954	0.00251	1193.0	0.00223	1181.8	0.00208	1171.3
14.00	1197.0	3048	0.00237	1179.8	0.00208	1170.0	0.00194	1160.3
15.00	1178.0	3140	0.00213	1167.3	0.00185	1159.0	0.00175	1150.0
16.00	1164.5	3228	0.00176	1156.0	0.00155	1149.3	0.00146	1140.8
17.10	1150.5	3871	0.00158	1145.8	0.00137	1140.3	0.00132	1132.3
18.00	1139.5	4022	0.00147	1138.3	0.00123	1133.8	0.00118	1126.0
19.00	1130.5	4164	0.00123	1130.5	0.00118	1127.3	0.00114	1119.8
20.00	1124.0	4287	0.00114	1124.0	0.00118	1121.0	0.00114	1113.8
21.00	1118.0	4396	0.00114	1118.0	0.00118	1114.8	0.00118	1107.8
22.00	1112.0	4494	0.00104	1112.0	0.00123	1108.5	0.00118	1101.5
23.00	1106.5	4585	0.00133	1106.5	0.00128	1102.0	0.00123	1095.3
24.00	1099.5	4668	0.00123	1099.5	0.00128	1095.3	0.00123	1088.8
25.00	1093.0	4746	0.00123	1093.0	0.00133	1088.5	0.00123	1082.3
26.00	1086.5	4820	0.00133	1086.5	0.00137	1081.5	0.00128	1075.8
27.00	1079.5	4889	0.00136	1079.5	0.00130	1074.3	0.00118	1069.0
27.80	1074.0	5224	0.00145	1073.8	0.00129	1068.8	0.00118	1064.0
29.20	1063.0	5512	0.00142	1063.0	0.00124	1059.3	0.00112	1055.3
30.00	1057.0	5539	0.00142	1057.0	0.00123	1054.0	0.00109	1050.5

[a] To convert to metric units, multiply feet by 0.305 for meters, miles by 1.609 for km, and cfs by 0.0283 for cms.

Table 14. continued

Distance from drainage divide (miles)	Stream bed elevation (1966, ft)	Discharge (cfs)	Critical shear stress, $\tau_c = 0.85$ lb/ft^2 (40.7 N/m^2) Land use and slope factor = 0.80					
			Roughness coefficient n = 0.025		Roughness coefficient n = 0.035		Roughness coefficient n = 0.045	
			Stable slope	Final bed elevation (feet)	Stable slope	Final bed elevation (feet)	Stable slope	Final bed elevation (feet)
31.00	1049.5	5572	0.00137	1049.5	0.00116	1047.5	0.00100	1044.8
31.90	1043.0	5972	0.00129	1043.0	0.00112	1042.0	0.00099	1040.0
33.00	1035.5	6044	0.00114	1035.5	0.00114	1035.5	0.00095	1034.3
34.00	1029.5	6110	0.00095	1029.5	0.00095	1029.5	0.00090	1029.3
35.00	1024.5	6174	0.00076	1024.5	0.00076	1024.5	0.00076	1024.5
36.00	1020.5	6238	0.00076	1020.5	0.00076	1020.5	0.00076	1020.5
37.00	1016.5	6302	0.00061	1016.5	0.00061	1016.5	0.00061	1016.5
38.25	1012.5	6380						

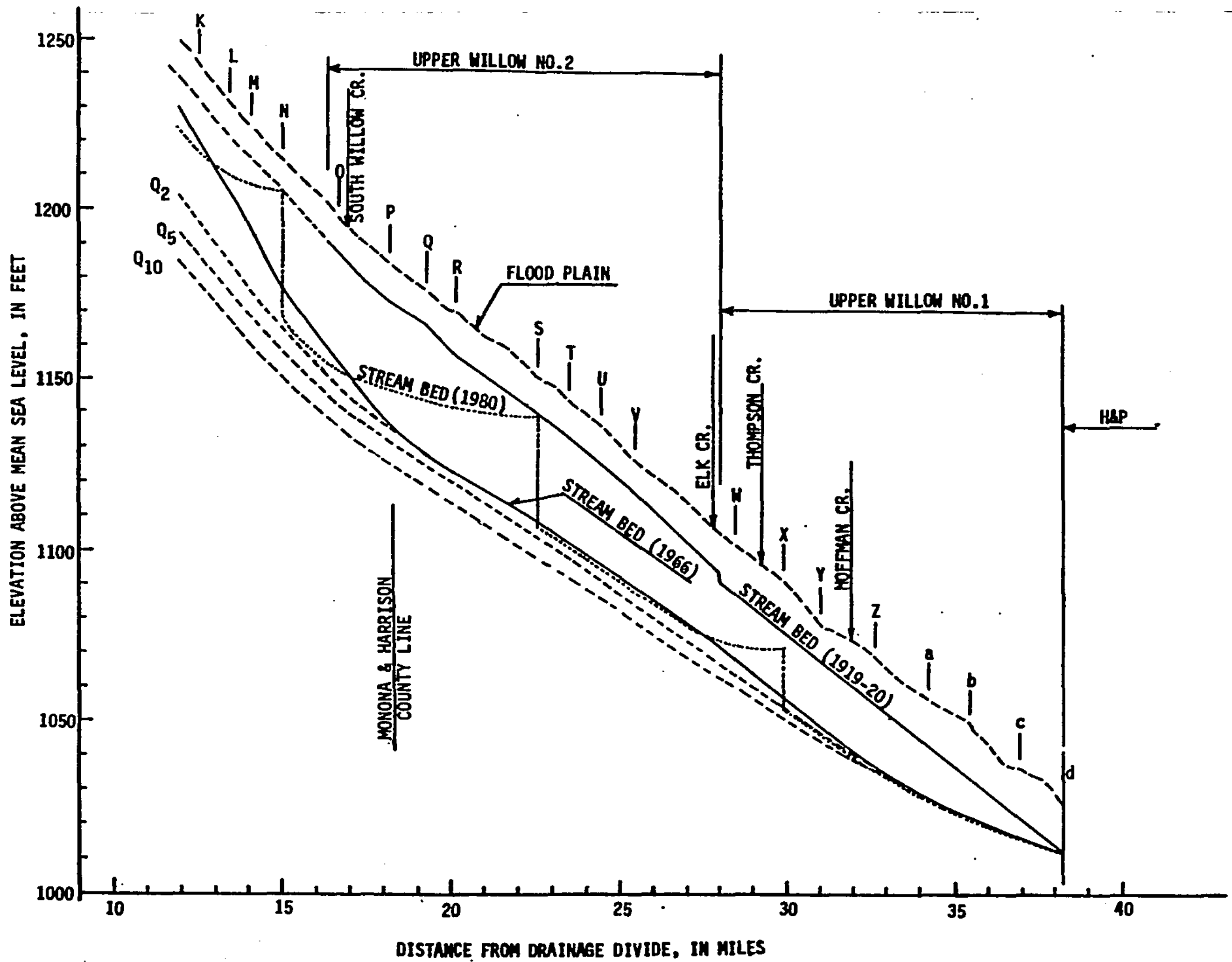


Figure 46. Predicted equilibrium profiles for Q_2, Q_5, and Q_{10}
($n = 0.035$; $\tau_c = 0.85$ lb/ft^2)

A. Points N, S, and X are the locations of three grade stabilization structures.

In Figure 46, the coordinate axes have been selected such that the horizontal axis (which designates the distance) coincides with the mean sea level datum. The vertical axis (which shows the elevation) is assumed to pass through the origin of the stream (i.e. the drainage divide).

Final equilibrium profile for Q_2

From Figure 46, it is observed that the equilibrium profile for Q_2 coincides with the 1966 stream bed profile, for about 75 percent of the study reach. This does not agree with the 1980 field observations, however. The more recent stream bed data indicate that degradation has occurred in some parts of the stream, after 1966. This implies that the dominant discharge, which was discussed earlier in this chapter, is greater than Q_2. It is interesting however, to see that even under this condition (Q_2), considerable degradation was predicted in the upstream reaches, if the flume structure at point N (see Figure 46) did not exist. Using this figure, it is estimated that about 20 feet (6 m) of additional degradation would have occurred in the future at mile 12 (19.3 km). Even more degradation would have been expected beyond the study reach, until the discharge approaches zero at the drainage divide (implying zero degradation at the divide, since D = 0).

<u>Final equilibrium profile for Q_{10}</u>

The predicted profile, using Q_{10} as the dominant discharge, seems to overestimate the ultimate degradation. It starts to diverge from the 1966 stream bed at mile 35 (56.3 km). Between miles 18 and 28 (30 to 45 km), an additional 10 feet (3 m) of degradation is simulated for the future equilibrium profile. It is interesting to see that the predicted profile and the original profile (1966) have almost the same gradient within this 10 mile (16.1 km) reach. This is a reasonable result, because the stream in 1966 had a rather uniform slope in this reach. The predicted depth of degradation at mile 12 (19.3 km), is as great as 48 feet (14.6 m). This is an extreme case, which is not likely to happen. The results of the 10 year recurrence interval flood as the dominant discharge is presented for comparative purposes only.

<u>Final equilibrium profile for Q_5</u>

This profile follows the same trend as Q_{10} for the lower reaches of the stream. The predicted degradation is slightly less in this case. The equilibrium profile starts to diverge at mile 33 (53 km).

The predicted stream bed downstream of flume X is slightly above the 1980 bed profile, suggesting that the 5 year peak discharge might tend to underestimate the ultimate degradation depth. However, the difference between two profiles in this reach is only in the order of one foot (30 cm). As has been declared before, the accuracy of the original data might be no better than $\pm$ 1-2 feet (0.3 - 0.6 m). For instance, in the 1980 survey, the stream bed elevations were obtained by measuring the depth of the stream bed below the bridge deck. Further, the elevations of the bridge decks were determined from U.S. Geological Survey topographic maps which are $\pm$ 2 feet in accuracy. A portion of additional degradation occurs below the flume structure, and also is attributed to the deposition of sediment upstream. During the early years of operation (1971-1975), much sediment was deposited upstream of flumes S and X, resulting in some periods of clearer water to scour the downstream channel.

Investigation of the final equilibrium profile, corresponding to the 5 year recurrence interval, indicates that downstream of mile 18 (30 km), which coincides with the position of the erosional front, the degradation depth of 3-5 feet (0.9 - 1.5 m) would have occurred without the construction of flume structures. Upstream of this reach (i.e. mile 18), in which the original profile is steeper, the depth of degradation increases and reaches a value of about 35 feet (10.7 m) at mile 12 (19.3 km).

Investigation of these equilibrium profiles has revealed that downstream of mile 18 (29 km), for either Q_5 or Q_{10}, the additional depth of degradation would be uniform in a rather long reach. For instance, for Q_5, in a 12 mile (19.3 km) reach, an additional depth of about 2-3 feet (0.6 - 0.9 m) would be degraded. Similarly for Q_{10}, the channel in the same reach, a deepening of 8-10 feet (2.4 - 3 m) would occur. However, common to all flow conditions upstream of mile 18 (29 km), the equilibrium profiles diverge from the initial (1966) stream bed. The amount of additional degradation is extensive, and the channel in mile 12, for example, would deepen an additional amount of 25-30 feet (7.6 - 15.2 m), depending on the selected dominant discharge. The stable stream slope (mile 12) is about 10-12 ft. per mile.

Impact of the equilibrium profile on the design of grade stabilization structures

A more meaningful interpretation can be made concerning the question of degradation control. Due to the high engineering and construction costs of such control measures, the optimum design of grade stabilization structures is of great importance. The size and location should be selected based on the extent of future degradation. If the stream in a certain reach has stabilized naturally (such as the existing lower reach), then no structure is needed. Construction of a grade stabilizing structure in a stabilized reach is not money well spent. On the other hand, in another reach of the stream which is subject to severe degradation, the provision of such a structure may be justified, physically and economically. Therefore, it seems appropriate to review the status of the three existing flume structures, in regard to the equilibrium profile of the stream.

In Chapter V, a brief discussion was presented regarding the flume structures located on Willow Creek. The locations of these flumes are illustrated in Figure 46 (i.e. points N, S, and X). From the figure, it is observed that the reservoirs above flumes S and X (in Harrison County) have completely been filled with sediment, during the period

1971-1980. Although no recorded data are available, the author's personal visits with the Harrison County Engineering Office personnel revealed that the reservoirs filled in the first four years of operation. This is an indication of the serious erosion problem existing in the upstream reaches of the basin. Although a portion of the sediment has originated from upland areas, nevertheless, the majority of deposition is attributed to channel degradation.

By using the stream data of 1966 and 1980, the volume of deposited sediment was estimated to be 850 acre-feet (1.05×10^6 m^3) upstream of flume S, and 200 acre-feet (2.47×10^5 m^3) upstream of flume X.

Investigation of the 1980 data illustrates that the stream slope in the reach of newly deposited sediments is much less than the original slope (see Figure 46). The average slope upstream of flume S, for example, has decreased to 0.0005, which is about 55 percent less than the 1966 stream bed slope. The effect of the structure is limited to the reach having the lower slope. Gradually, the slope steepens, once again matching the upstream slope. Therefore, in engineering evaluations, it is very important not to overestimate the effectiveness of these types of structures (staircase concept).

The decrease in the stream slope upstream of the flumes has not been due to the change of the bed material. The results of particle size analysis of the soil specimens collected from sediment deposits have indicated that the soil texture in the deposits is similar to the parent bed material (compare Figure B.8 with the other particle size distribution curves in Appendix B). It is interesting to see that the cohesive materials forming the stream bed, when detached and deposited again, will exhibit a much lower resistance to erosion. There has been no overburden to add pressure and weight, and to develop a more cohesive structure. The more cohesive sediment particles are normally eroded as an aggradation. However, these newly deposited sediments can be detached in a much smaller size (even in individual particle sizes).

Besides the stabilization achieved upstream of a structure, the flume structures will stop the migration of the degradation just downstream of the structure. The relative importance of the two functions is different, depending on the structure. In the following section, the conditions of the three flume structures on the Willow Creek are examined.

Flume N From Figure 46, it is seen that due to the steep slope of the stream channel, the effective length above the flume is much less than those for flumes S and X. However, the greater share of the flume function is in stopping the migration of the erosional front upstream. The location of the final equilibrium profile (even for Q_2) illustrates that, without flume S, the flume structure N could not stay in place. This author believes that an extensive depth of degradation could eventually undermine this structure, if flume S is not maintained in place.

Flume S   The 1980 stream data above this flume indicate that about 5.5 miles (8.9 km) of the channel above the structure has been stabilized. As was mentioned above, the placement of this structure has preserved the integrity of flume N also. Without flume S, about 2-3 feet (0.6 - 0.9 m) of additional degradation could have occurred in the vicinity of the flume. Since the maintenance cost of four bridges (P, Q, R, and S) would have been more without the flume, the choice to or not to construct such a structure depends on engineering economics. However, one of the advantages of this flume is its effect on the safety of flume N.

Flume X   From Figure 46, it is observed that the effective length upstream of the flume is less than 2 miles (3.2 km). Only two bridges are protected by this flume. However, the flume has stopped the upstream movement of degradation. As previously mentioned, the expected depth of degradation in the vicinity of this flume is not very great. Therefore, to prevent the upstream migration of the degradation, much simpler structures (such as vertical cut off walls) might have been used. To this author, the stream is stabilizing in this reach with or without Flume X; it may not have been needed.

Effect of Manning roughness coefficient

As it was explained before, the flood discharge for a 5-year recurrence interval (as the dominant discharge) was used with three different values of the roughness coefficient (i.e. n = 0.025, 0.035, and 0.045). The associated elevations of the final equilibrium profiles are illustrated in Table 14, and they are plotted in Figure 47.

As the roughness coefficient increases, the degradation depth increases. Increasing the Manning coefficient from 0.030 to 0.045 has resulted in an increase of 11 feet (3.4 m) in the degradation depth at the end of the study reach. In contrast, a decrease in the coefficient (from 0.035 to 0.025) has resulted in a decrease of about 14 feet (4.3 m) in the ultimate degradation depth at the end of the study reach (mile 12).

In degrading streams, the vegetative cover on the slopes changes along the stream length. Due to the relative stability in the lower reaches, the plants have a greater chance to grow and remain in position. On the other hand, upstream reaches are subjected to severe erosion, and the state of unstability (continued bed and bank erosion) makes it more difficult for the plants to grow. Field observations from Willow Creek have revealed that large trees have grown on the banks of the Upper Willow No. 1 (relatively stable) while the vegetation cover of the uppermost part of the Upper Willow No. 2 mostly consists of small bushes and brush. If the effect of these changes can be quantified (by direct flow measurement in the desired reach), then the Manning roughness coefficient might be described as a function of distance (i.e. $n = f(x)$) along the stream channel, and/or as a function of active versus less active degradation. If such an expression is developed, the function could be incorporated into the computer model.

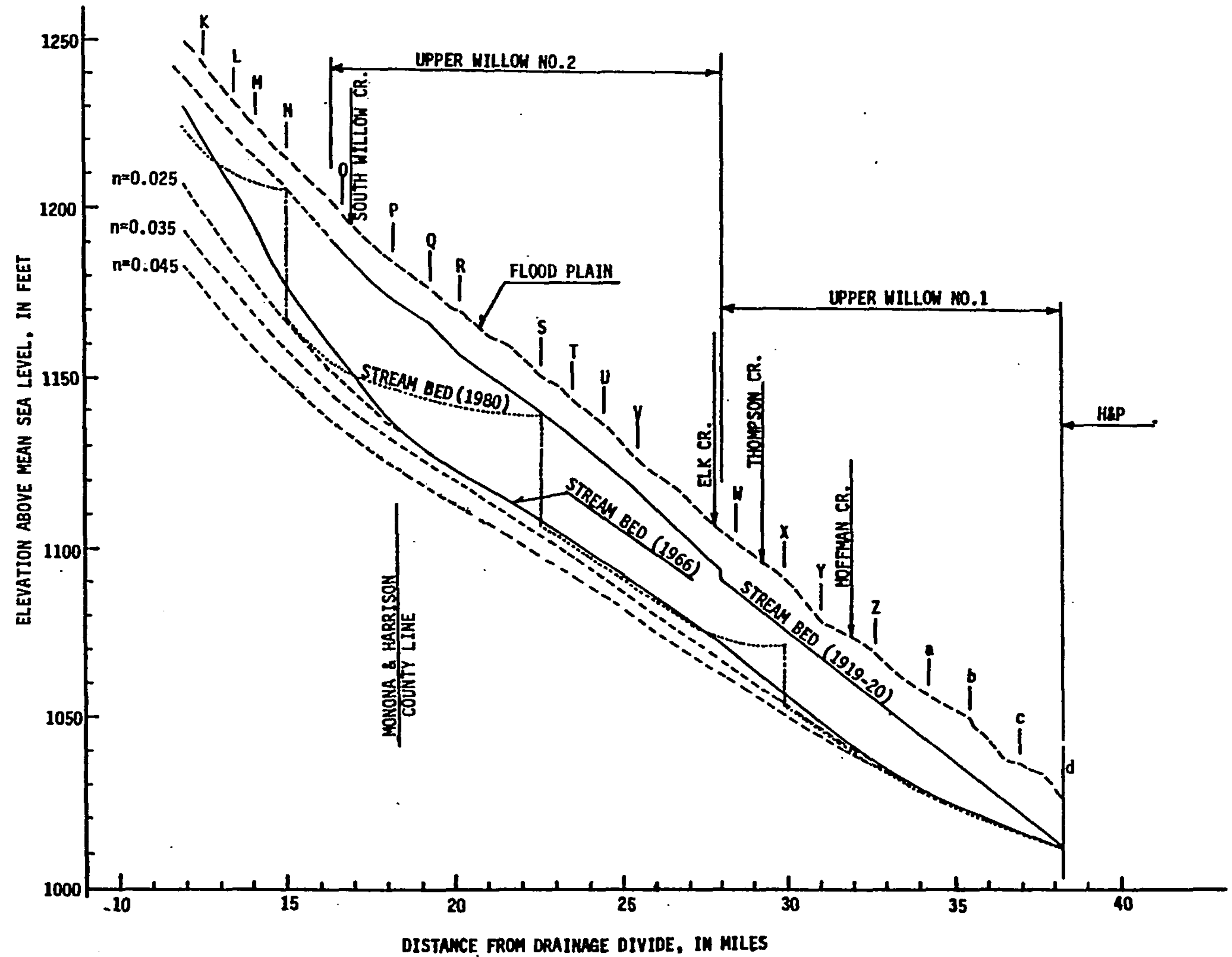


Figure 47. Predicted equilibrium profiles for Q_5 with different Manning roughness coefficients

The roughness coefficient will change with water depth (Chow, 1959). If the relationship can be established, the computer model can be modified accordingly. It also is reasonable to state that the predicted equilibrium depth of degradation in Upper Willow No. 2 is overestimated, particularly for greater "n" values (see τ_c effect, below). More quantitative interpretation requires more field data.

Effect of critical shear stress, τ_c

A constant value of 0.85 lb/ft^2 (40.70 N/m^2) was selected for this study; however the shear resistance might vary along the channel, with depth, etc. For instance, if a stream flows through stratified soil layers with varying properties, the resistance to scour would be different in each layer. If the stream bed reaches coarse material, the channel may be stabilized by armoring. The inclusion of these variations into the degradation calculations is possible if more quantitative information of critical shear stress, τ_c, becomes available.

If vegetation continues to grow on the channel banks and encroaches gradually into the bed, the value of Manning's "n" would tend to increase. However, the value of τ_c also should increase, to reflect the erosion resistance of the roots of shrubs and small trees, along the perimeter of the cross section. Therefore, an increase in Manning's "n" would be offset by an increase in the value of τ_c, and one should consider these to be interdependent and compensating variables.

CHAPTER VII.

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

Model development

Many alluvial streams in western Iowa were straightened in the early part of this century. The straightening was done for flood control and land reclamation purposes. Since then, severe degradation of the bed and scour and sloughing of the stream banks have occurred. The deepening and widening in these streams have endangered the safety of the bridges crossing the streams. The problem is of major concern to state and county officials who are responsible for the bridges and other facilities. The degradation can be controlled by the provision of grade stabilization structures. However, the optimum design of such structures requires an estimation of the final stable profile of the stream.

Available analytical methods for degradation prediction are based primarily on laboratory experiments on noncohesive bed materials, and do not provide a reliable solution in alluvial streams having cohesive materials in their beds. Moreover, these analytical methods are complicated to use, require the evaluation of a large number of variables, and still require many simplifying assumptions. Statistical methods exist, but require a large amount of basic field data, including pre-straightening channel information. Unfortunately, the recorded data regarding the stream changes are seldom available.

An equilibrium stream profile model was developed, for Willow Creek in western Iowa, a typical degrading stream. The model can be applied to streams with essentially no pre-degradation record, but for which existing data can be obtained. A "dominant discharge" was selected to represent the variation of flow in the stream channel. Most of the available methods for degradation are one dimensional, that is, only the vertical deepening of the stream is considered. However, the widening of stream channels is considered to be as important in the western Iowa streams as vertical degradation, and cannot be ignored. The widening of the channel section was incorporated into the equilibrium profile model.

The developed model can be used to predict the final equilibrium profile of other streams, which then can provide sufficient information for the design of grade stabilization structures.

Application of the model to other streams

The equilibrium profile model for channel degradation was developed and tested using Willow Creek data. However, the similarity of soils, climate, and hydrological characteristics in western Iowa streams makes it possible to extend the results of this study beyond the Willow Creek watershed. The inclusion of land use and slope factors into the discharge formula makes the method more flexible, and it can be applied to streams with essentially no bed elevation data for the past years. The present geometry of the river (i.e. transverse cross sections and longitudinal profile) must be surveyed, however, and channel bed characteristics should be compared to that of Willow Creek.

In applying the method to ungaged streams, the following procedure is recommended:

1. Make a detailed survey of the stream, including the traverse cross sections and the longitudinal profile of the stream.

Once the detailed information on the cross section geometry of the stream is available, the value of the width-to-depth ratio, side slopes, bottom width, etc. can be determined. The variations of these variables with distance and depth can provide the necessary information for degradation computations.

2. Determine the variation of discharge along the stream channel, using the available regional flood frequency methods, or eqs. 36 and 37 which were developed in Chapter IV. Drainage areas at selected points on the stream must be determined. More accurate discharge-distance relationships can be established by using 1:24000 U.S. Geological Survey topographic maps. These maps are available for most drainage basins.

3. Evaluate the Manning roughness coefficient for the stream in question. Due to the impact of the value of this coefficient on the final results, it is recommended that a field study be done to

determine the n value (i.e. direct measurement of the flow, cross-sectional area, and slope of a study reach).

4. Evaluate the critical shear stress as an important variable, which should be determined as accurately as possible. Study of the stable reaches in a stream will permit one to make a reasonable estimate of this factor. If there is a lack of data, a value in the range of 0.7 - 1.0 lb/ft^2 (33.6 - 47.9 N/m^2) can be used as an initial approximation (for western Iowa streams).

5. Although the method was applied to the main channel of the Willow Creek only, the prediction of degradation in the tributaries should follow the same procedure. Once the final profile of the main stream is known, the equilibrium profile of the tributaries can be estimated. The final elevation of the main channel at the confluence point should be considered as the initial elevation for making a degradation prediction in a tributary.

Recommendations

This study and other similar investigations have shown that stream bed degradation starts from the downstream and progresses upstream. This means that the stability of the stream in any reach will depend highly on the stability of the downstream reaches. In other words, in the investigation for provision of possible grade stabilization structures, the entire drainage basin should be considered as a planning unit. Close cooperation among the county and state agencies in the same watershed is necessary. Any given section of the stream cannot gain stability (other factors remaining equal), unless the downstream reaches have already reached equilibrium.

By comparison of the final equilibrium profile with the initial stream bed profile, it can be demonstrated also that the expected depth of degradation in certain reaches might not be so great. Then one can avoid using costly flume structures at such locations. The progress of degradation at these locations can be stopped by providing much simpler structures. The structural problems involved should be studied, of course. This author believes that in areas where the expected additional degradation is in the range of 1 - 5 feet (0.3 - 1.5 m), provision of simpler structures than flume structures could be recommended.

As it has been mentioned repeatedly, the estimation of the critical conditions for the initiation of sediment motion in cohesive alluvial streams is a necessary, but difficult task. More research, especially combined with field investigation, is required, so that the evaluation of sediment transport of such materials can be made with more confidence. Both laboratory and field research will be required to yield results that will be of value to the practicing professions.

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APPENDIX A:

DISTANCE, DRAINAGE AREA, AND ELEVATION OF SELECTED POINTS ALONG THE MAIN CHANNEL OF WILLOW CREEK

Table A contains the historical data of the stream bed elevations, the lengths, and the drainage areas at different points along Willow Creek. The locations of selected data points are shown in Figure A.1. As it is indicated in the figure, point A is the beginning of the main channel at the drainage divide. The road crossings are designated by alphabetical letters, in a downstream direction, the upper case letters (A-Z) have been used to identify the first 26 points, while the lower case letters for the rest of the road crossings (a-k). Other points of interest, such as the junctions of the tributaries and elevation points located between any two adjacent bridges, are identified by the name of the upstream bridge and a numerical subscript. For instance, there are four points between bridges P and Q, therefore they are named P_1, P_2, P_3, and P_4, respectively. However, for the simplicity reason, only numerical subscripts are shown in Table A.1 and in Figure A.1. Namely, the numbers 1, 2, 3, and 4 in Table A.1 and in Figure A.1 and downstream of bridge P should be considered (P_1, P_2, P_3, and P_4, respectively).

Distances along the main channel (columns 2 and 3 in Table A.1) were determined either by using the original survey of the ditch, made in 1919 and 1920, or by using the USGS (United States Geological Survey) 1:24,000 scale maps.

Drainage areas (column 3) were determined by using the aerial photos of the 1965 and 1971 flights. Drainage boundaries were located on the photographs by using a stereoscope. The boundaries were then transferred to the 1:24,000 scale topographic maps. A planimeter was used to determine the drainage areas. For some points where a major tributary was connected to the main channel, two values were indicated for the drainage area. The lower and higher values were the drainage areas just before and after the junction point, respectively (proceeding downstream).

Ground elevations (column 5) were determined from the original survey of the creek (1916 and 1918).

Bed elevations in 1919 (column 6) were reported from the original engineering designs. The data were obtained from the file at the Drainage Clerk's Office, Court House, Harrison County, Iowa.

Bed elevations in 1958 (column 7) were reported by Daniel (1960). A telephone conversation with Mr. Daniel revealed that he had used a barometer to determine the bridge deck elevations in 1958. By subtracting the vertical distance between the bridge deck and the stream bed, he then calculated the stream bed elevations. Daniel used the U.S. Coast and Geodetic Survey bench mark at the Woodbine Railroad Station to convert the measured data to the mean sea level datum (MSL; now the National Geodetic Vertical Datum, NGVD). As a result of potential instrumental error, it is quite possible that the reported figures differ by plus or minus a few feet from the real stream bed elevations in 1958.

Stream bed elevations in 1966 (column 8) were obtained from the file at the Drainage Clerk's Office, Court House, Harrison County, Iowa. This set of data is the most complete set since the construction of the ditch. The survey was performed by the County Engineering Office by leveling the bridges decks along the stream. The original survey, unfortunately, was not based on the mean sea level datum. However, the author transformed the data to the mean sea level datum by using the elevations of the USGS bench marks at nearby locations.

Bed elevations in 1965 and 1971 were reported from the 1:24,000 scale topographic maps. As it is indicated in column 9 and 10 of the table, the contour intervals are 20 feet in the upper reaches and it decreases to 10 and 5 feet in the downstream direction.

Bed elevations in 1980 were measured by the author at selected locations. The author measured the distance between the bridge decks and stream bed for most of the road crossings on the Willow Creek. The data was then transformed to mean sea level datum.

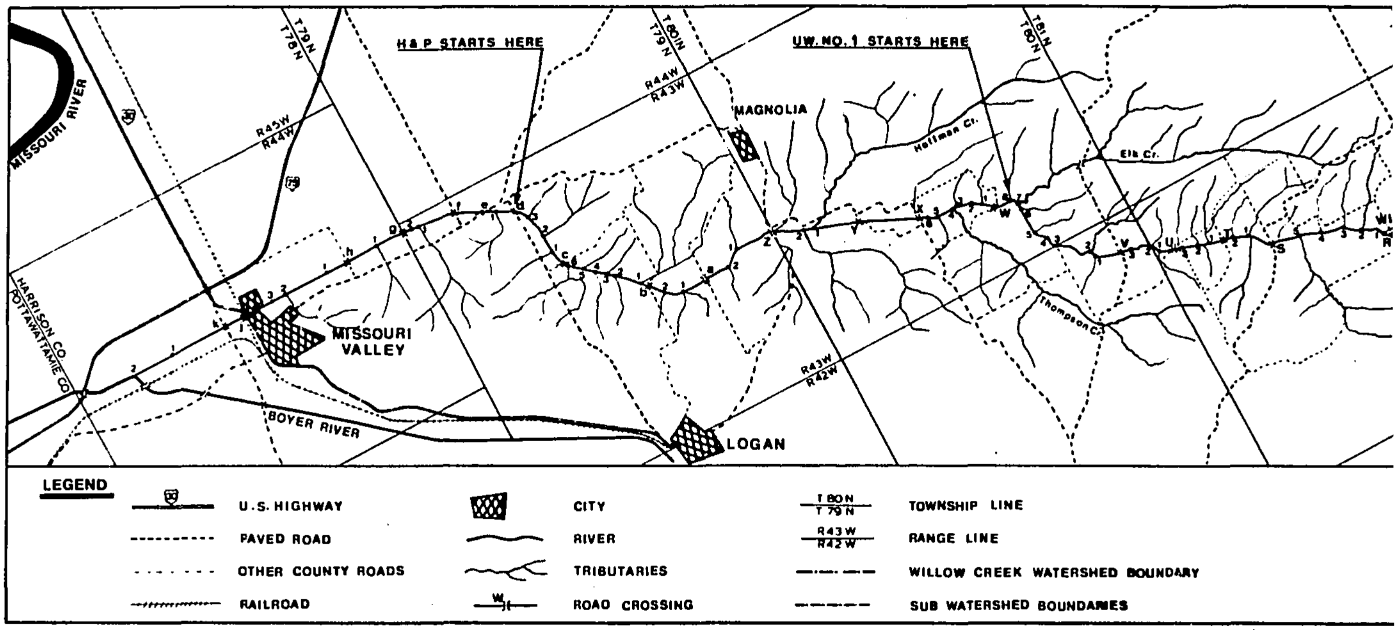
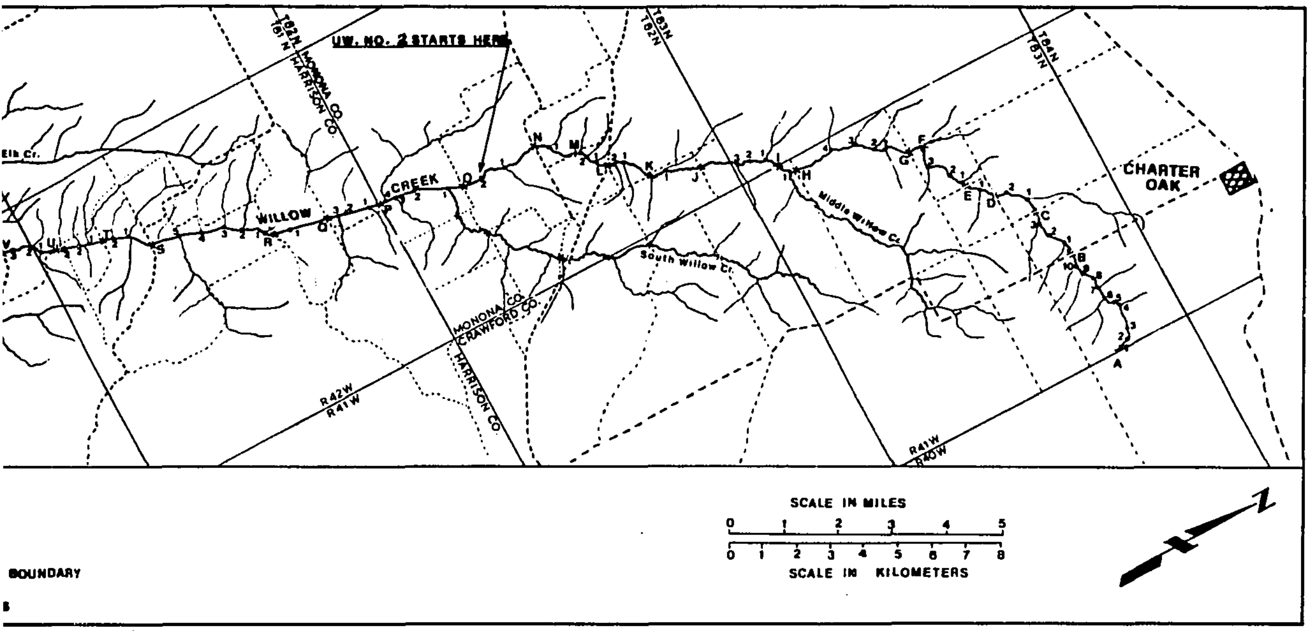


Figure A.1. Location map of the bridge crossings on the Willow Creek



UW. NO. 2 STARTS HERE
T62N
T61N
MONONA CO.
HARRISON CO.
T63N
T62N
T64N
T63N
CHARTER OAK
Elk Cr.
WILLOW
CREEK
Middle Willow Cr.
South Willow Cr.
MONONA CO.
CRAWFORD CO.
HARRISON CO.
R42W
R41W
R41W
R40W
SCALE IN MILES
0 1 2 3 4 5
0 1 2 3 4 5 6 7 8
SCALE IN KILOMETERS
N
BOUNDARY

Table A-1. Distance, drainage area, and elevation of selected points along the main channel of Willow Creek

1	2	3	4	5	6	7	8	9	10	11
Point #	Distance from drainage divide		Drainage area in square miles	Ground elevation in feet	Bed elevation above mean sea level in feet					
								Based on aerial survey in the year of:		
	Feet	Miles			1919	1958	1966	1965	1971	1980
A –	0	0							1480	
1	650	0.12							1440	
2	2000	0.38							1420	
3	3650	0.69							1400	
4	5800	1.10							1380	
5	6300	1.19	0.89							
6	7550	1.43								
7	9100	1.72	1.83							
8	10400	1.97							1360	
9	11500	2.18	2.36							
10	12500	2.37	2.76							
B –	14100	2.67	3.03							
1	15900	3.01	3.53							
2	17000	3.22							1340	
3	19150	3.63	4.08							
C –	20400	3.86	4.15							
1	22000	4.17	6.26							
2	23900	4.53							1320	
D –	25300	4.79	7.11							
1	27250	5.16	7.49							
E –	28400	5.38	7.58							
1	29200	5.53	7.93							
2	30100	5.70							1300	
3	33300	6.31	8.93							
F –	35800	6.78	9.08							
G –	37300	7.06								
1	40150	7.60	10.73						1280	
2	41000	7.77	11.48							
3	43200	8.18	12.33							
4	45700	8.66	12.48						1260	

Table A-1. Continued

1	2	3	4	5	6	7	8	9	10	11
Point #	Distance from drainage divide		Drainage area in square miles	Ground elevation in feet	Bed elevation above mean sea level in feet					
								Based on aerial survey in the year of:		
	Feet	Miles			1919	1958	1966	1965	1971	1980
H -	50100	9.49	13.44							
1	50900	9.64	22.10							
I -	52250	9.90	22.27							
1	53900	10.21	23.39							
2	55250	10.46							1240	
3	56900	10.78	23.97							
J -	60800	11.52	25.44							
1	64700	12.25	27.48							
K -	66550	12.60					1220		1220	
1	69550	13.17	28.67							
2	70650	13.38								
L -	71350	13.51	29.10				1206.5			
1	72650	13.76	29.55							
2	73850	13.99	29.89							
M -	74650	14.14					1194.0			
1	78250	14.82	31.68							
N -	79800	15.11	32.17				1176.5		1200 1180	
1	83550	15.82	33.77							
2	86500	16.38	34.17	1202	1191.3	UW No.2 starts here			1160	
O -	88400	16.74	34.59	1198	1186.9	1172	1155.0			1152.0
1	89900	17.03	34.68 49.58	1194						
2	92900	17.60			1178.3					
3	95050	18.00	50.29 50.89							
4	95500	18.09	53.44						1140	
P -	96200	18.22	53.90	1184	1173.1	1141	1137.0			1147.5
1	97750	18.52	54.65							
2	100050	18.95	55.62							
3	101150	19.16	55.76 57.76							

Table A-1. Continued

1	2	3	4	5	6	7	8	9	10	11
Point #	Distance from drainage divide		Drainage area in square miles	Ground elevation in feet	Bed elevation above mean sea level in feet					
								Based on aerial survey in the year of:		
	Feet	Miles			1919	1958	1966	1965	1971	1980
Q -	102050	19.33		1176	1165.9	1135	1128.5			1143.5
1	105800	20.04	58.86	1170						
			59.78							
R -	106700	20.20	59.95	1170	1157.1	1132	1123.0			1142.0
			60.70							
1	107300	20.32	62.58		1156.3					
2	10650	20.96	62.97	1163					1120	
			63.38							
3	112450	21.30	63.88							
4	114450	21.67	63.13	1160						
5	116700	22.10	64.57							
			66.78							
S -	119500	22.63	67.06	1151	1140.6	1111	1108.5			1139.5
										1107.0
1	122300	23.17	68.88	1148						
2	124000	23.48			1134.3					
T -	124400	23.56	69.30	1144	1133.7	1104	1103.0			1101.0
1	125800	23.83		1142						
2	127400	24.13							1100	
3	128550	24.34	71.19	1137						
U -	129250	24.48		1137	1126.1	1098	1096.5			1095.0
1	130700	24.76	72.92							
2	132300	25.06								
3	134000	25.38			1118.3					
V -	134750	25.52	73.93	1126	1116.8	1090	1089.5			1088.5
1	137350	26.01								
2	138200	26.18	75.71							
3	141600	27.81	76.53	1117	1104.3					
4	142600	27.01							1080	
5	144200	27.31	77.14	1112						
6	146400	27.73		1108						

Table A-1. Continued

1	2	3	4	5	6	7	8	9	10	11
Point #	Distance from drainage divide		Drainage area in square miles	Ground elevation in feet	Bed elevation above mean sea level in feet					
								Based on aerial survey in the year of:		
	Feet	Miles			1919	1958	1966	1965	1971	1980
7	147150	27.87	77.93 / 87.2							
8	147900	28.01			1094.3 / 1091.5	UW No. 1 starts here				
W –	150150	28.44		1102	1088.3	1070	1069.0			1073.0
1	151550	28.70	88.57							
2	152350	28.84			1085.3					
3	153700	29.11								
4	154300	29.22	89.09 / 96.47							
5	155900	29.53								
6	156800	29.70						1060		
X –	157850	29.90	98.69	1092	1077.1	1058	1057.5			1072.0 / 1047.0
Y –	163700	31.01		1079	1068.6	1051	1049.5			
1	168450	31.91	100.52 / 108.00							
2	169300	32.06	112.60	1074				1040		
Z –	172200	32.61	115.34	1070	1056.3	1040	1038.0			1035.5
1	176000	33.33	117.13	1063						
2	177950	33.70								
a –	180550	34.20	118.00	1057	1044.2	1031	1028.5			1027.0
1	183250	34.71	118.62							
2	184050	34.86								
b –	186800	35.38		1051	1035.1	1025	1023.0			1022.5
1	187450	35.50	118.97 / 122.65	1049						
2	190150	36.01		1044						
3	191300	36.23		1041				1020		
4	192250	36.41	123.83	1039						
5	193350	36.62		1038						
6	194750	36.88								

Table A-1. Continued

1	2	3	4	5	6	7	8	9	10	11
Point #	Distance from drainage divide		Drainage area in square miles	Ground elevation in feet	Bed elevation above mean sea level in feet					
								Based on aerial survey in the year of:		
	Feet	Miles			1919	1958	1966	1965	1971	1980
c –	195100	36.95	126.23	1037	1023.1	1018	1016.5			1016.0
1	196800	37.27		1035						
2	198450	37.59	128.10	1034						
3	200750	38.02								
d –	202000	38.26	129.00		1012.8	1009	1012.0	H&P starts here		1012.0
1	203450	38.53						1010		
e –	204950	38.82								
f –	207800	39.36								
1	211500	40.06								
2	212050	40.16						1005		
g –	213150	40.37								
1	215900	40.89								
h –	219150	41.51								
1	221250	41.90						1000		
2	225900	42.78								
3	228600	43.30						995		
i –	230200	43.60								
j –	230600	43.67								
k –	232550	44.04								
1	238250	45.12						990		
2	242800	45.98								

APPENDIX B:

PARTICLE SIZE DISTRIBUTION CURVES

OF STREAM BED MATERIAL

Of the various sediment properties, size has the greatest significance to the hydraulic engineer. In fact, size has been found to represent a sufficiently complete description of the sediment particle for many practical purposes, in alluvial stream studies. Grain size has a direct effect on the mobility of the particle; the smaller the size, the more readily it can be transported by flowing water.

In order to determine the size distribution of the materials forming the bed and the banks of Willow Creek, 14 soil samples were collected and analyzed for this purpose. The particle size analysis of the samples was performed by the method listed in the ASTM Soils Manual (The Asphalt Institute, 1978). According to this method, the distribution of particle sizes larger than 75 μm is determined by the sedimentation process, using a hydrometer. For each sample, the diameters of particles so determined were plotted on a logarithmic scale as the abscissa while the percentages smaller than the corresponding diameters were plotted on the arithmetic scale as the ordinates, using semi-log graph paper. A smooth curve was fitted to each set of data. The soil sampling locations and corresponding figures are tabulated in Table B.1.

It should be understood that for sediment transport problems, the behavior of a cohesionless soil can often be related to particle size distribution. However, the behavior of a cohesive soil usually depends much more on geological history and structure than on particle size.

Table B.1. Soil sampling locations

Sample #	Location Point	Particle Size Distribution Curve #	Description of the Soil Sample
1	O	B-1	from the stream bed
2	Q	B-2	from the right bank 9 ft above the stream bed
3	R	B-3	from the right bank 2 ft above the stream bed
4	T	B-4	from the left bank 7 ft above the stream bed
5	V	B-5	from the right bank 12 ft above the stream bed
6	B	B-6	from the right bank 1 ft above the stream bed
7	V-8	B-7	from the right bank 1 ft above the stream bed
8	X	B-8	from the sediment deposits upstream of flume
9	X	B-9	from the left bank 6 ft above the stream bed
10	Y	B-10	from the right bank 2 ft above the stream bed
11	Z	B-11	from the left bank 3 ft above the stream bed
12	a	B-12	from the stream bed
13	a	B-13	from the left bank 2 ft above the stream bed
14	d	B-14	from the right bank 5 ft above the stream bed

[a]For locating the points across the Willow Creek, Figure A.1 of Appendix A should be consulted

In spite of this fact, the particle size distribution curve is frequently used as an engineering tool to represent the size of the bed material.

Table B.2 illustrates the make-up of the above mentioned soil samples as obtained from the particle size distribution curves.

Table B.2. Percent size distribution of the bed and bank materials of the Willow Creek

Sample #	Clay	Silt			Sand			% Total
		Fine	Medium	Coarse	Fine	Medium	Coarse	
1	17	5	14	48	14	5	-	100
2	12.5	3.5	12.5	49.5	19	3	-	100
3	10	4	13	47	13	13	-	100
4	19	4.5	12.5	39	13	11.5	0.5	100
5	19	7.5	16.5	41	15.5	0.5	-	100
6	11	3	11	55	20	-	-	100
7	16.5	5.5	15	48	15	-	-	100
8	20	6	18	50	16	-	-	100
9	14	5	10	53	17	1	-	100
10	16	4.5	12	42.5	24.5	0.5	-	100
11	17	7	16	47	13	-	-	100
12	16	3.5	12.5	50	15	2	1	100
13	21.5	4.5	12	44	13	5	-	100
14	15	4	12	51	16	2	-	100

As it is noted from the table, the variations among the 14 different locations are not large. To illustrate this graphically, Figures B.1 through B.14 were superimposed so that the variations could be pictured properly. From Figure B.15 it is observed that all the particle size distribution curves fall inside a rather narrow band. For instance, d_{50} (particle size at which 50% of the soil weight is finer), which is frequantly used in sediment transport equations, ranges between 0.025 to 0.045 mm. For this reason and for all practical purposes, it is logical to use an average particle size distribution curve. Figure B.16 is such an average curve which was obtained by fitting an "average" curve through the data points in Figure B.15. In the calculations and discussions regarding the size of the bed material, Figures B.15 and B.16

have been used as a representative of the particle size distribution of the Willow Creek.

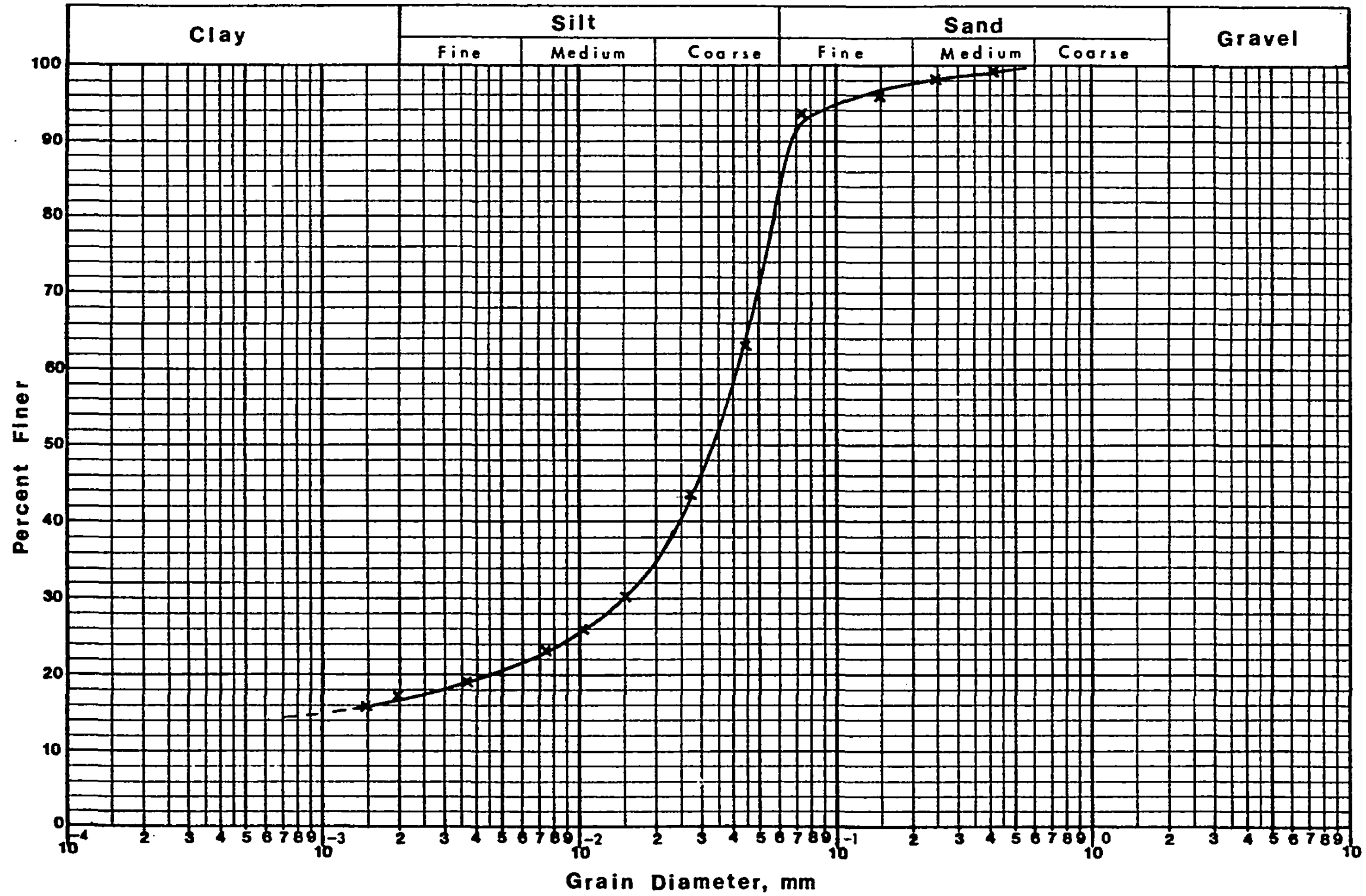


Figure B.1. Particle size distribution of the bed material at bridge "O" sample taken from the stream bed

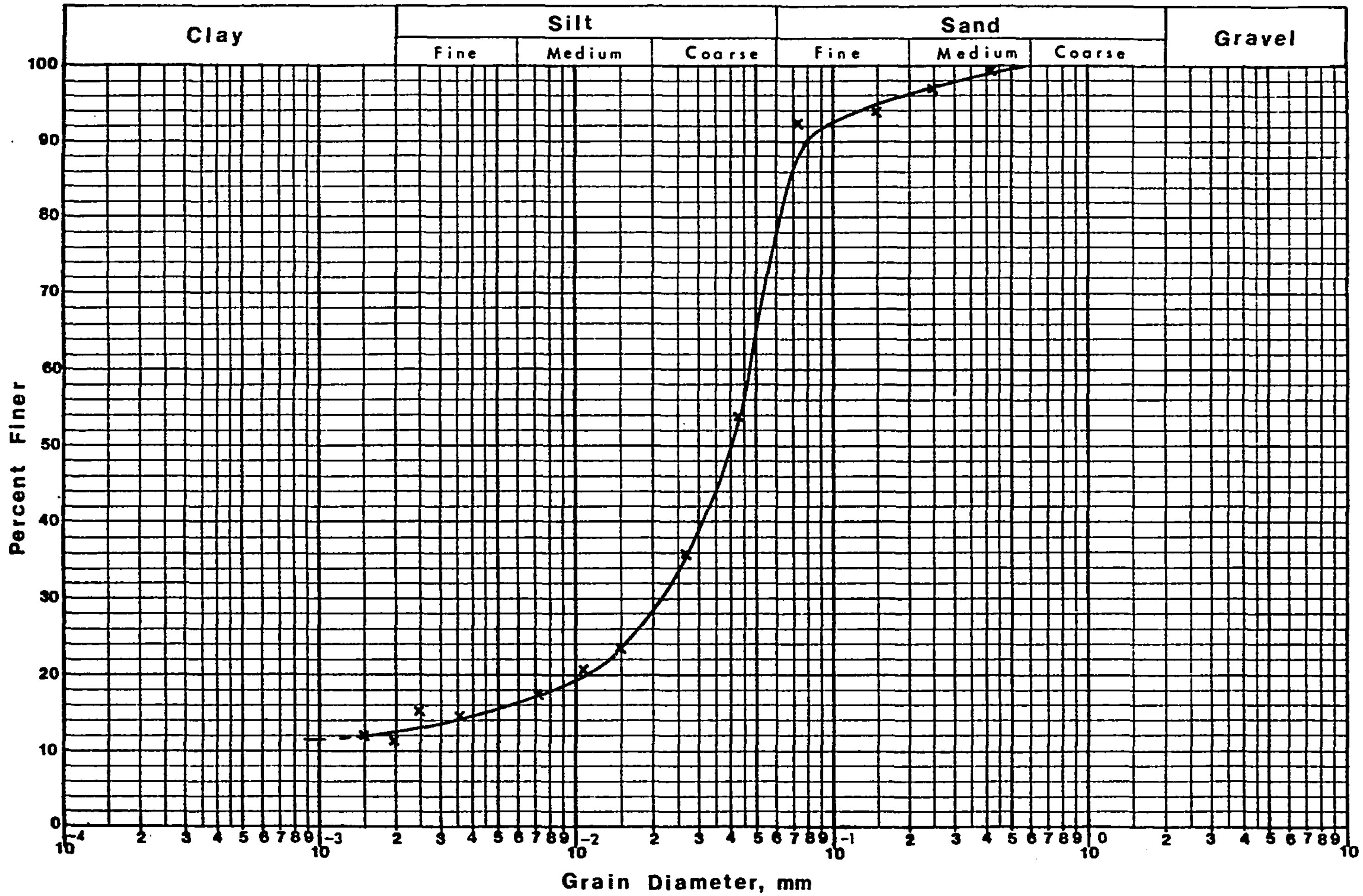


Figure B.2. Particle size distribution of the bed material at bridge "Q" sample taken from the right bank 9 feet above the stream bed

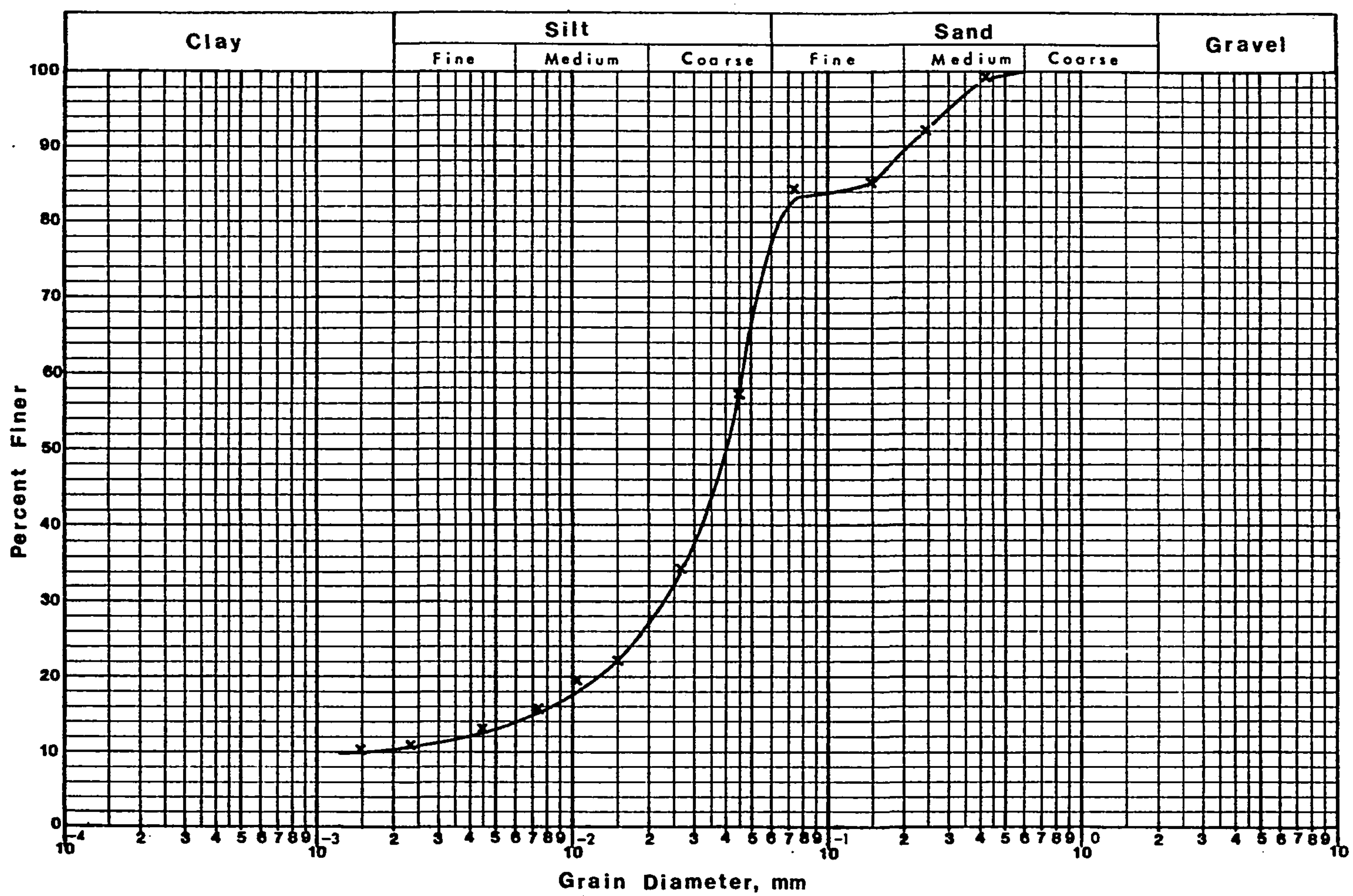


Figure B.3. Particle size distribution of the bed material at bridge "R" sample taken from the right bank 2 feet above the stream bed

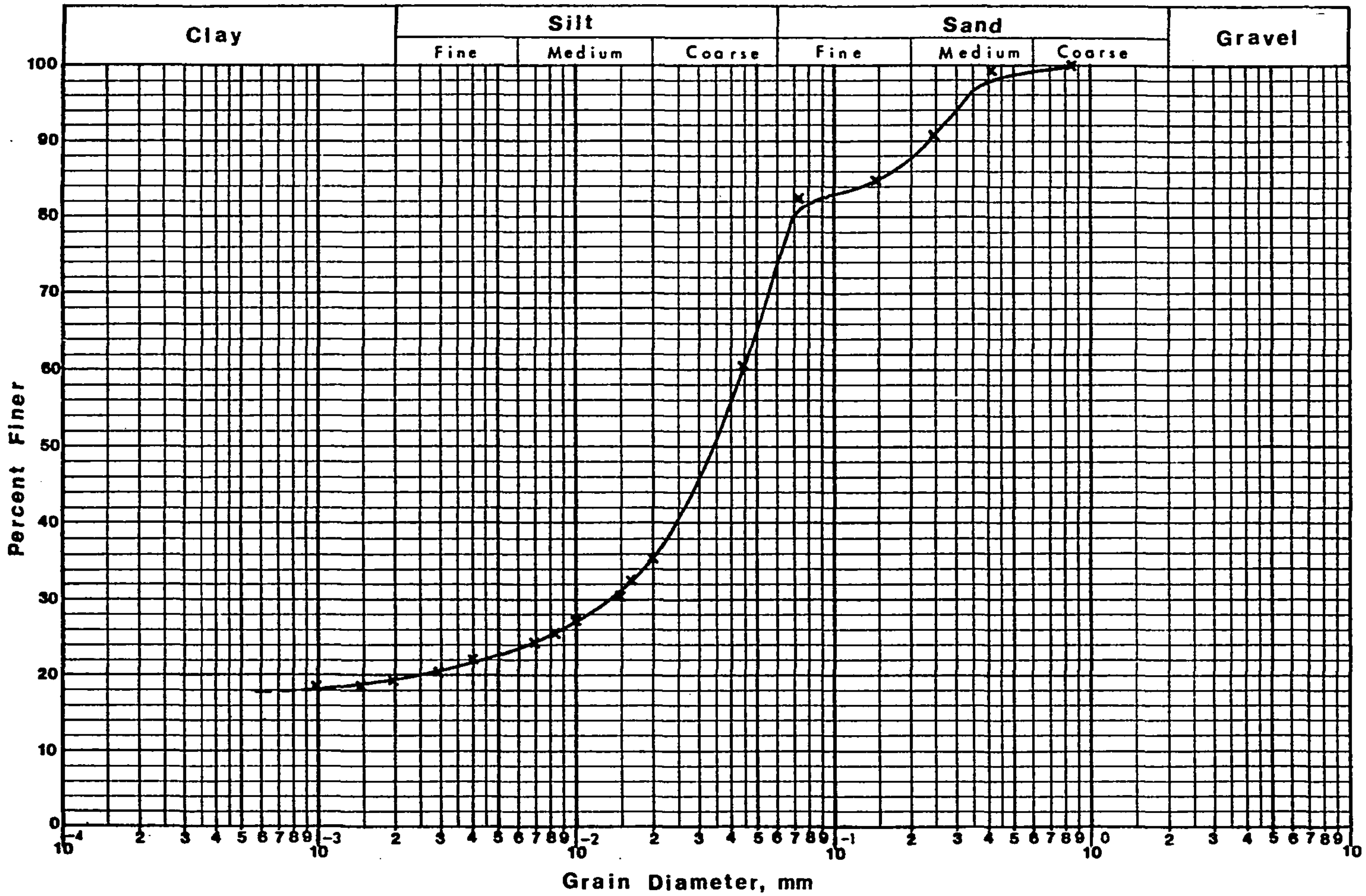


Figure B.4. Particle size distribution of the bed material at bridge "T" sample taken from the left bank 7 feet above the stream bed

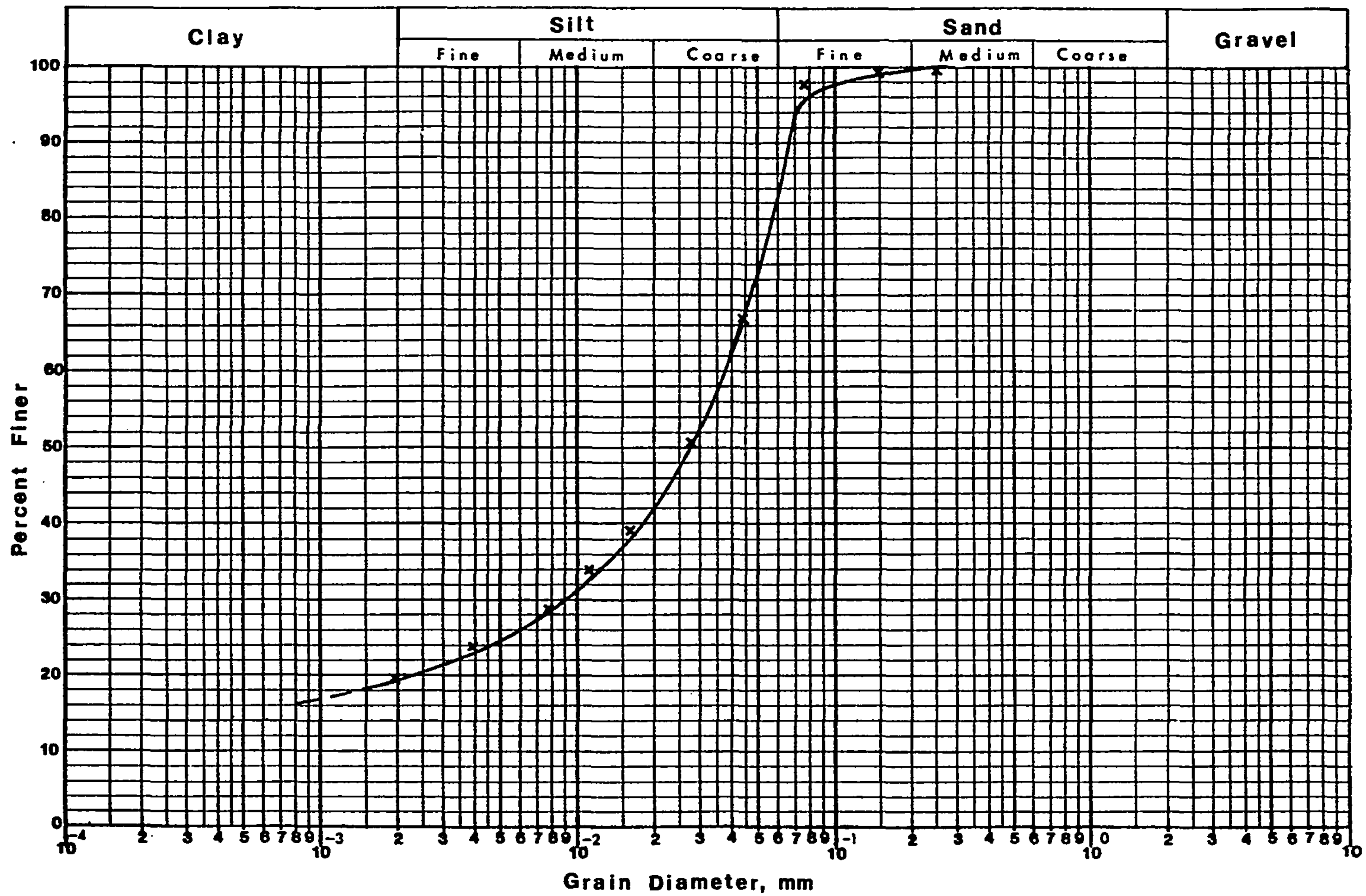


Figure B.5. Particle size distribution of the bed material at bridge "V" sample taken from the right bank 12 feet above the stream bed

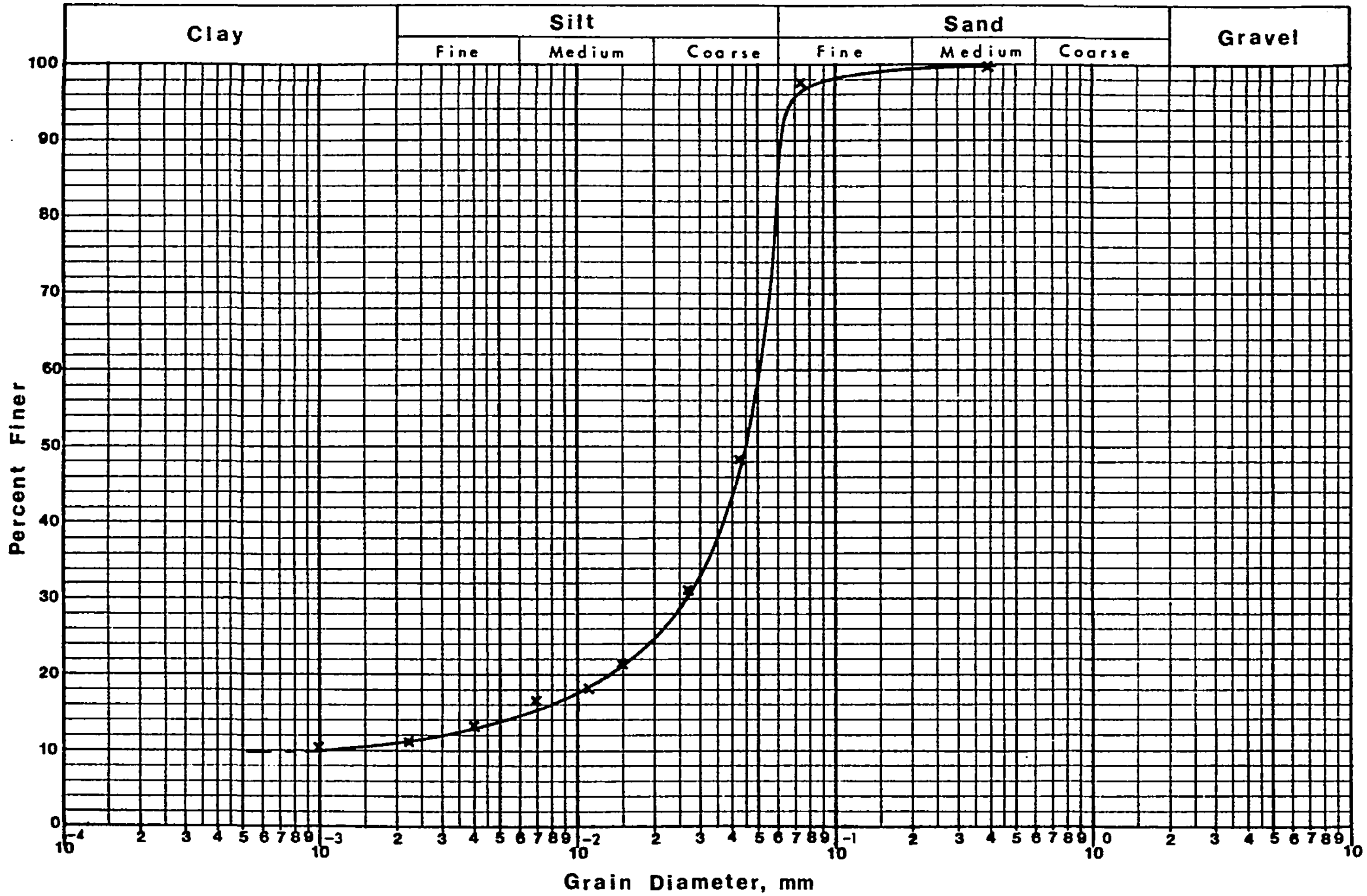


Figure B.6. Particle size distribution of the bed material at bridge "V" sample taken from the right bank 1 foot above the stream bed

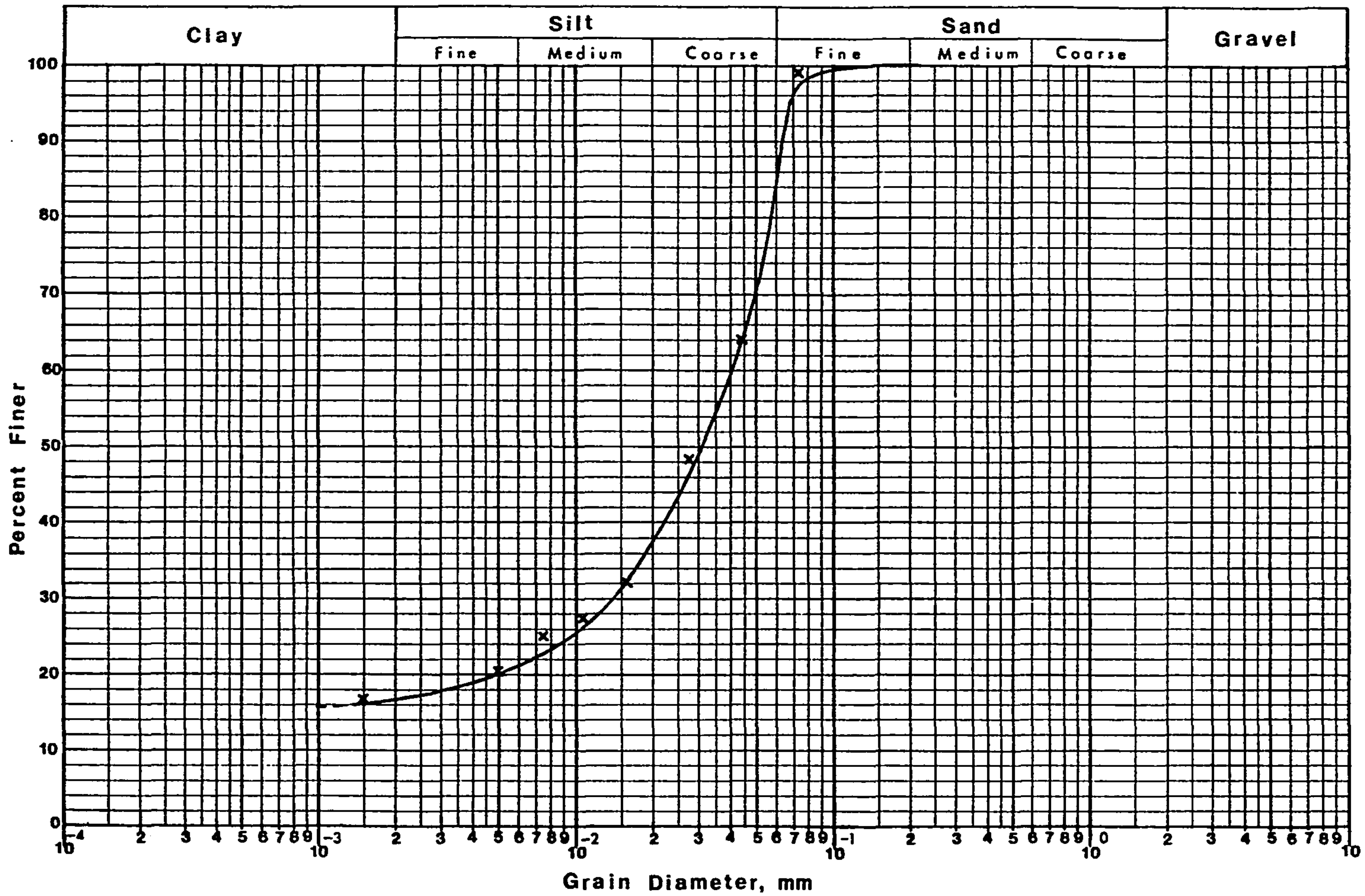


Figure B.7. Particle size distribution of the bed material at point "V7" sample taken from the right bank 1 foot above the stream bed

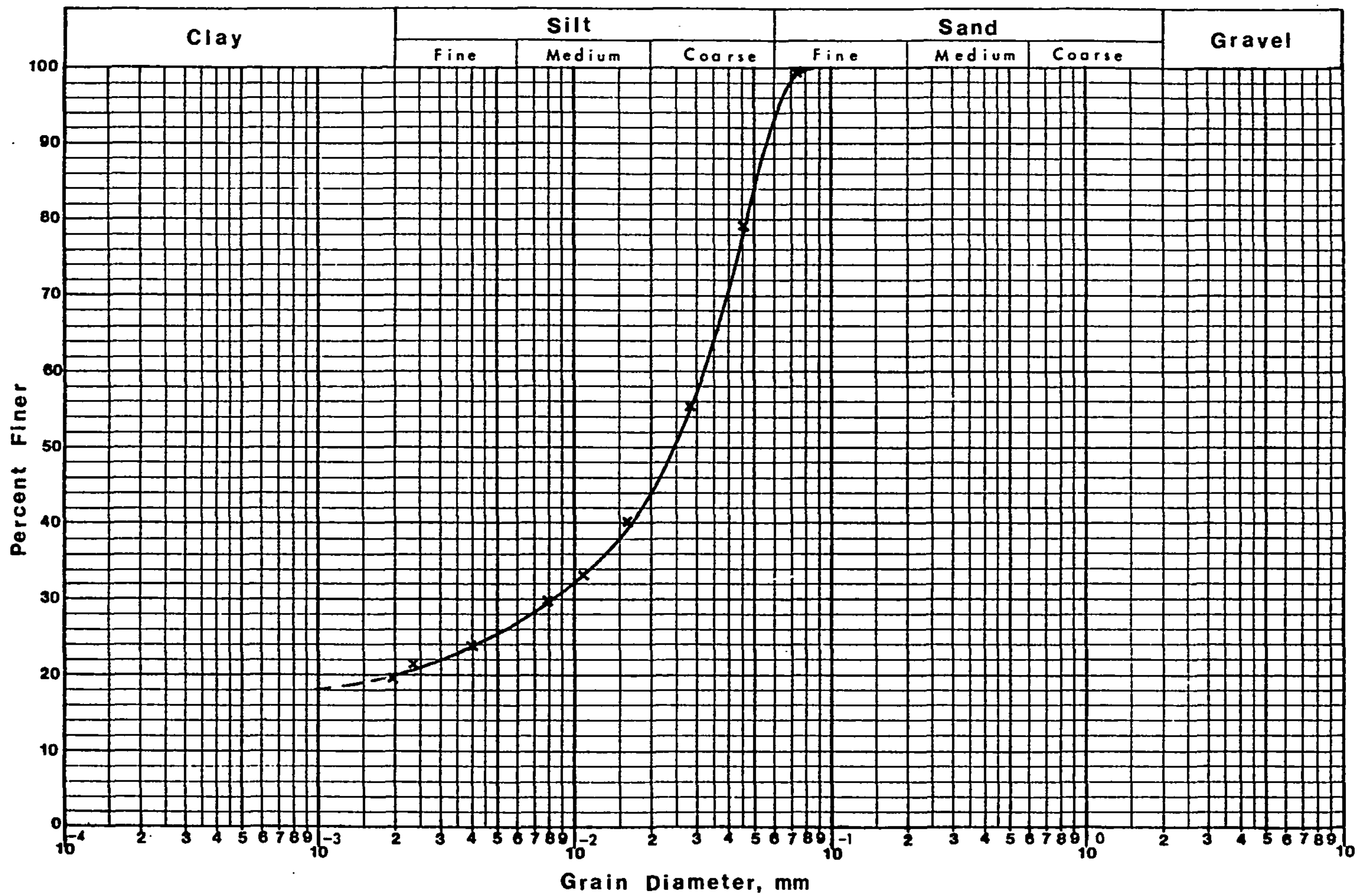


Figure B.8. Particle size distribution of the bed material at bridge "X" sample taken from the sediment deposits upstream of the flume

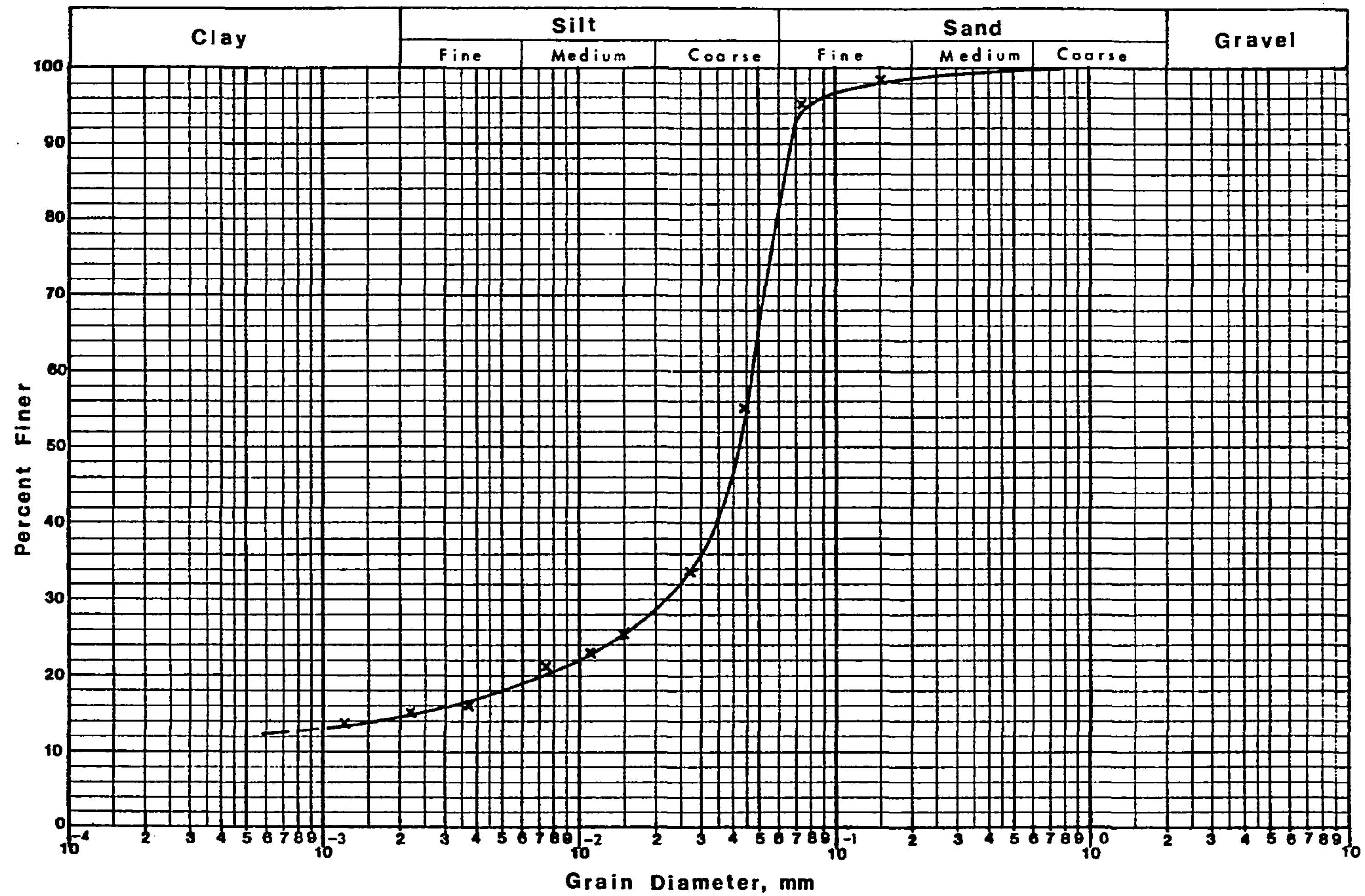


Figure B.9. Particle size distribution of the bed material at bridge "X" sample taken from the left bank 6 feet above the stream bed

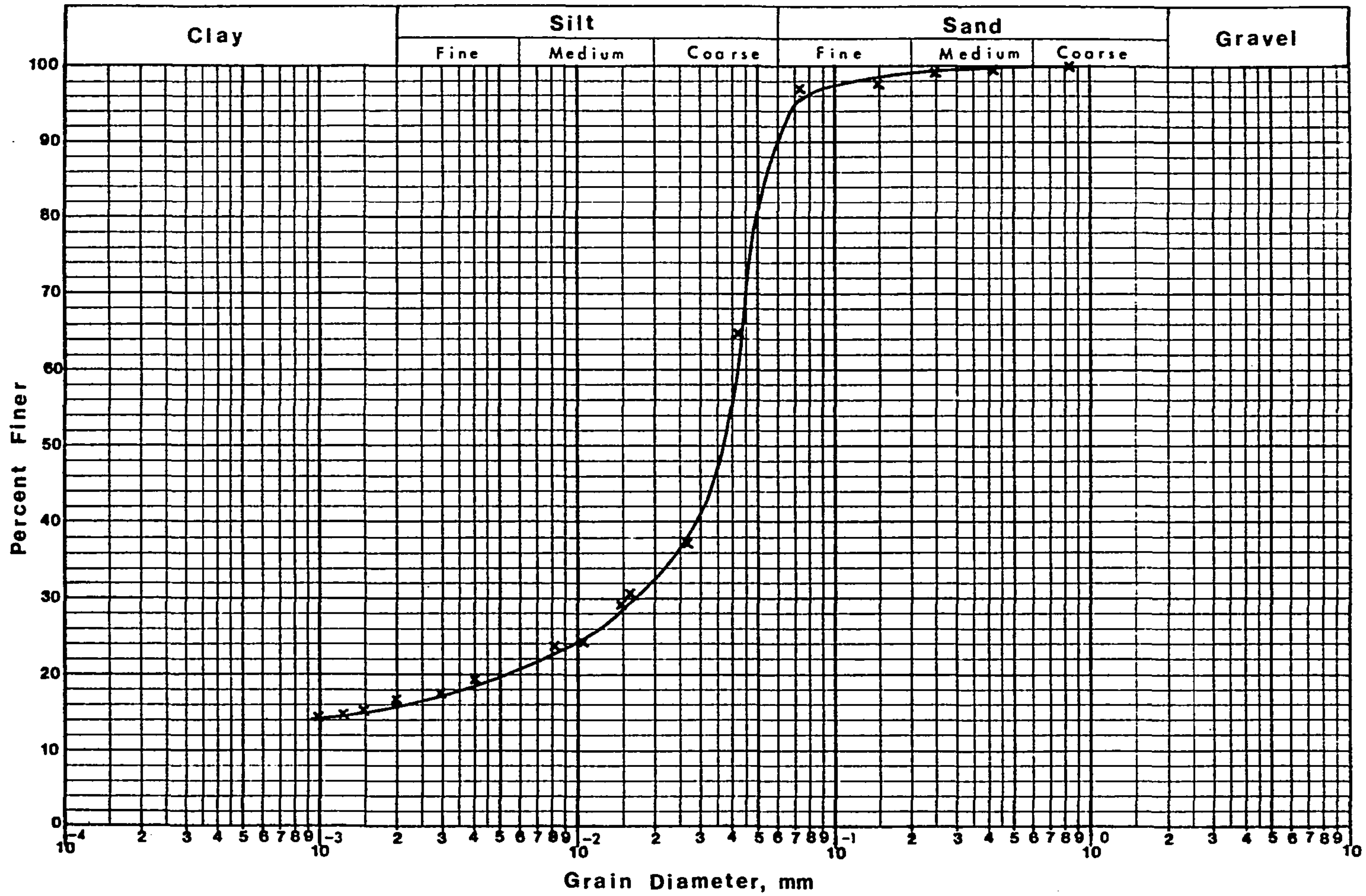


Figure B.10. Particle size distribution of the bed material at bridge "Y" sample taken from the right bank 2 feet above the stream bed

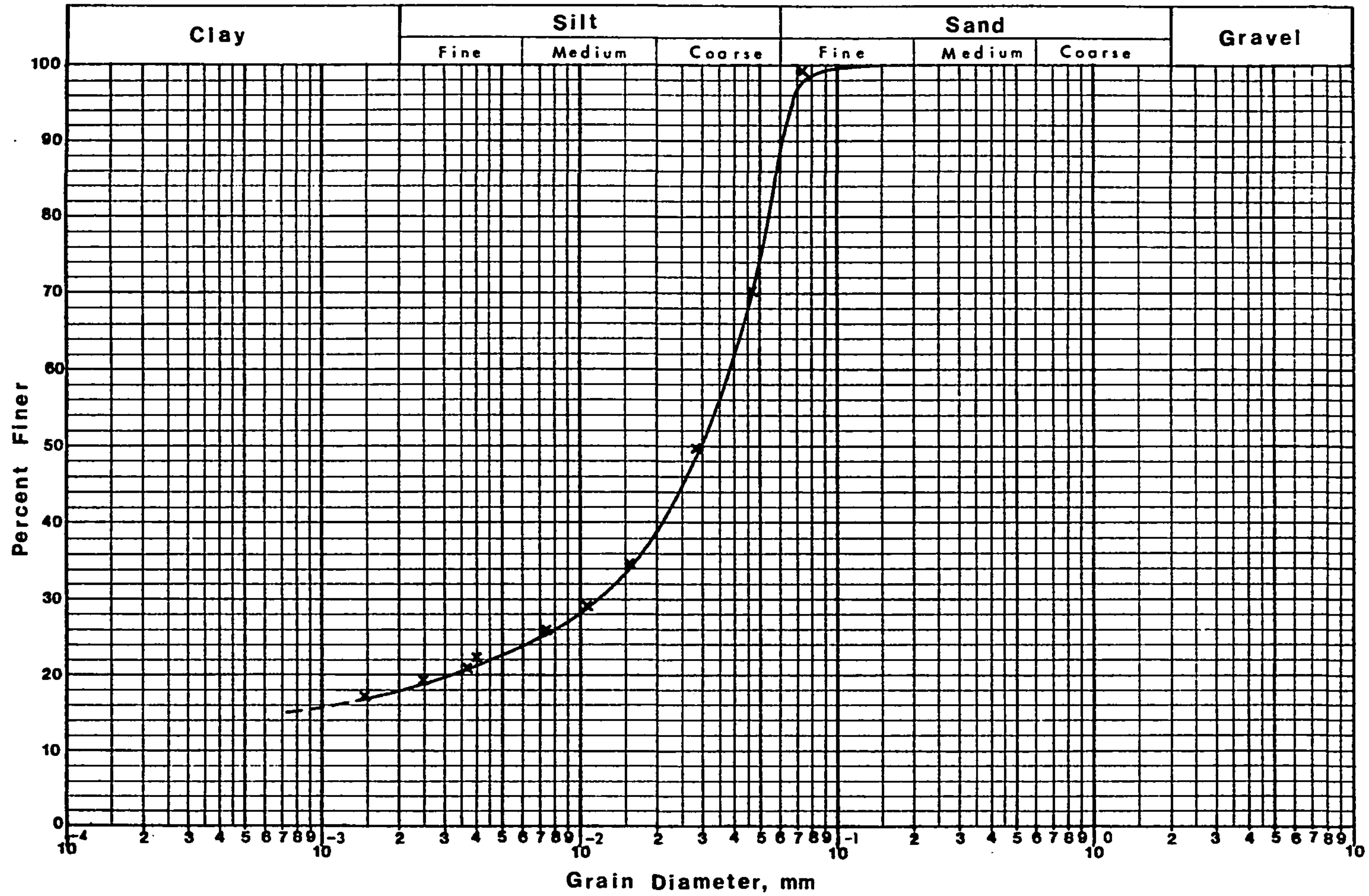


Figure B.11. Particle size distribution of the bed material at bridge "Z" sample taken from the left bank 3 feet above the stream bed

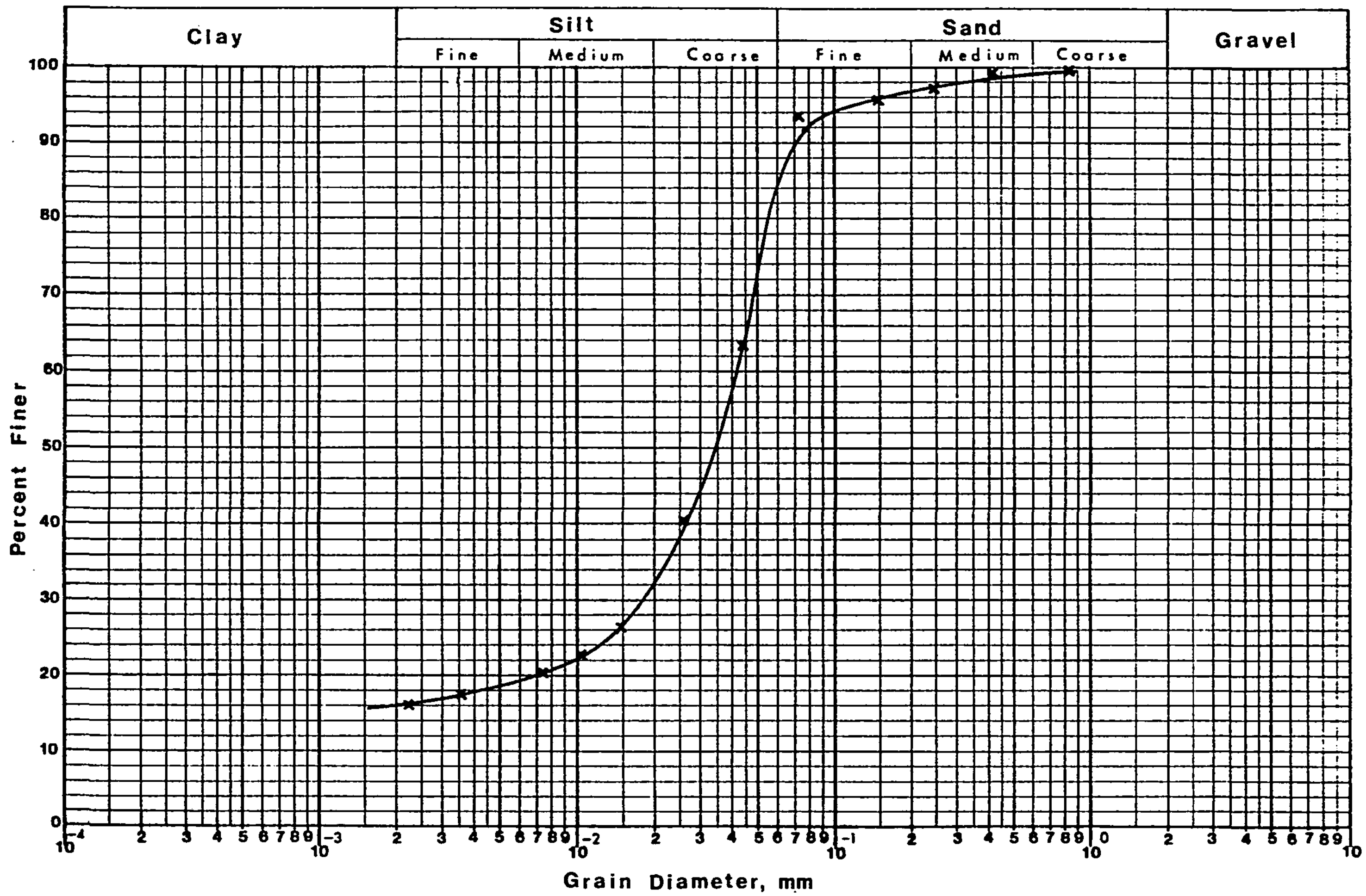


Figure B.12. Particle size distribution of the bed material at bridge "a" sample taken from the stream bed

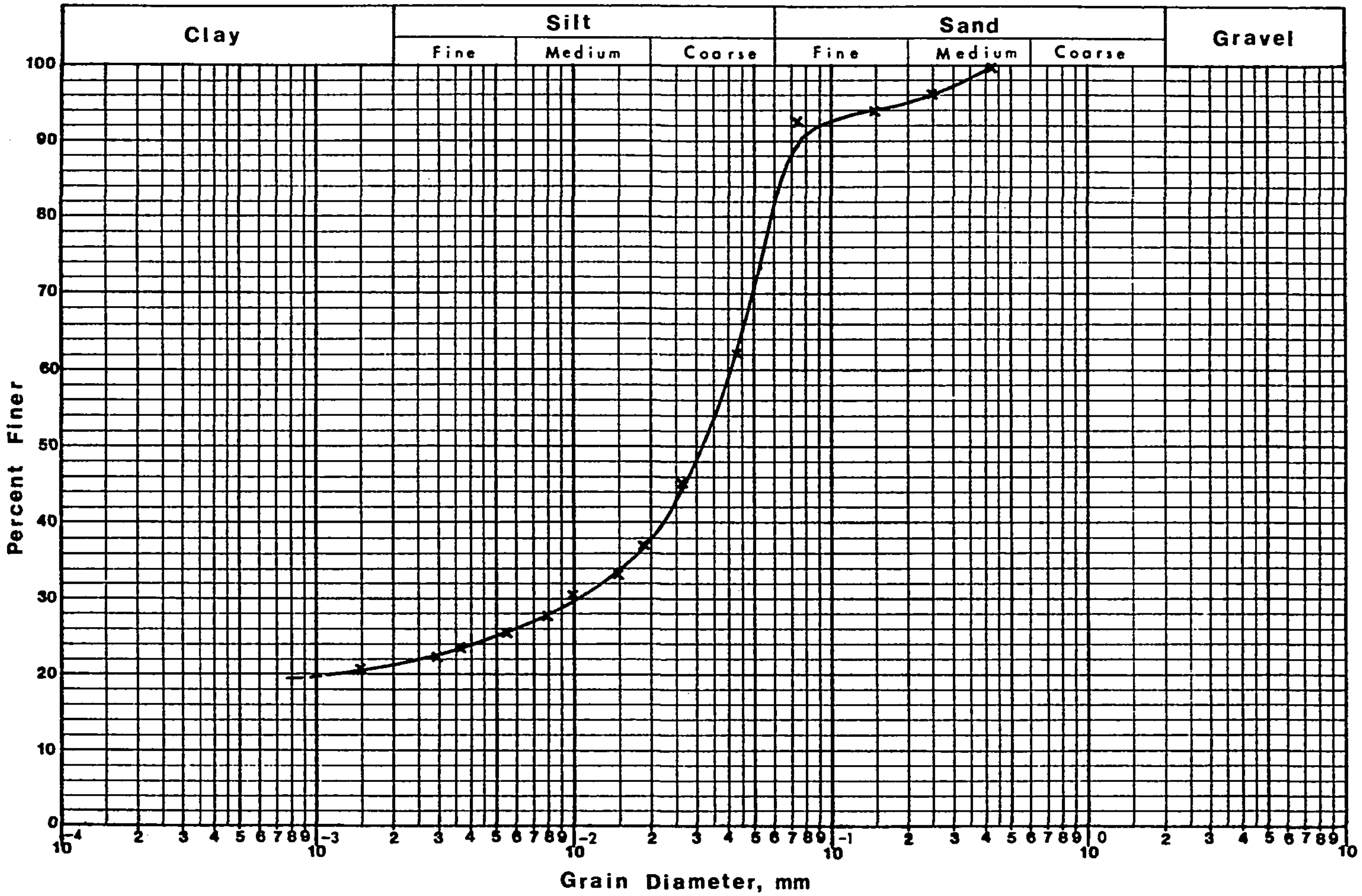


Figure B.13. Particle size distribution of the bed material at bridge "a" sample taken from the left bank 2 feet above the stream bed

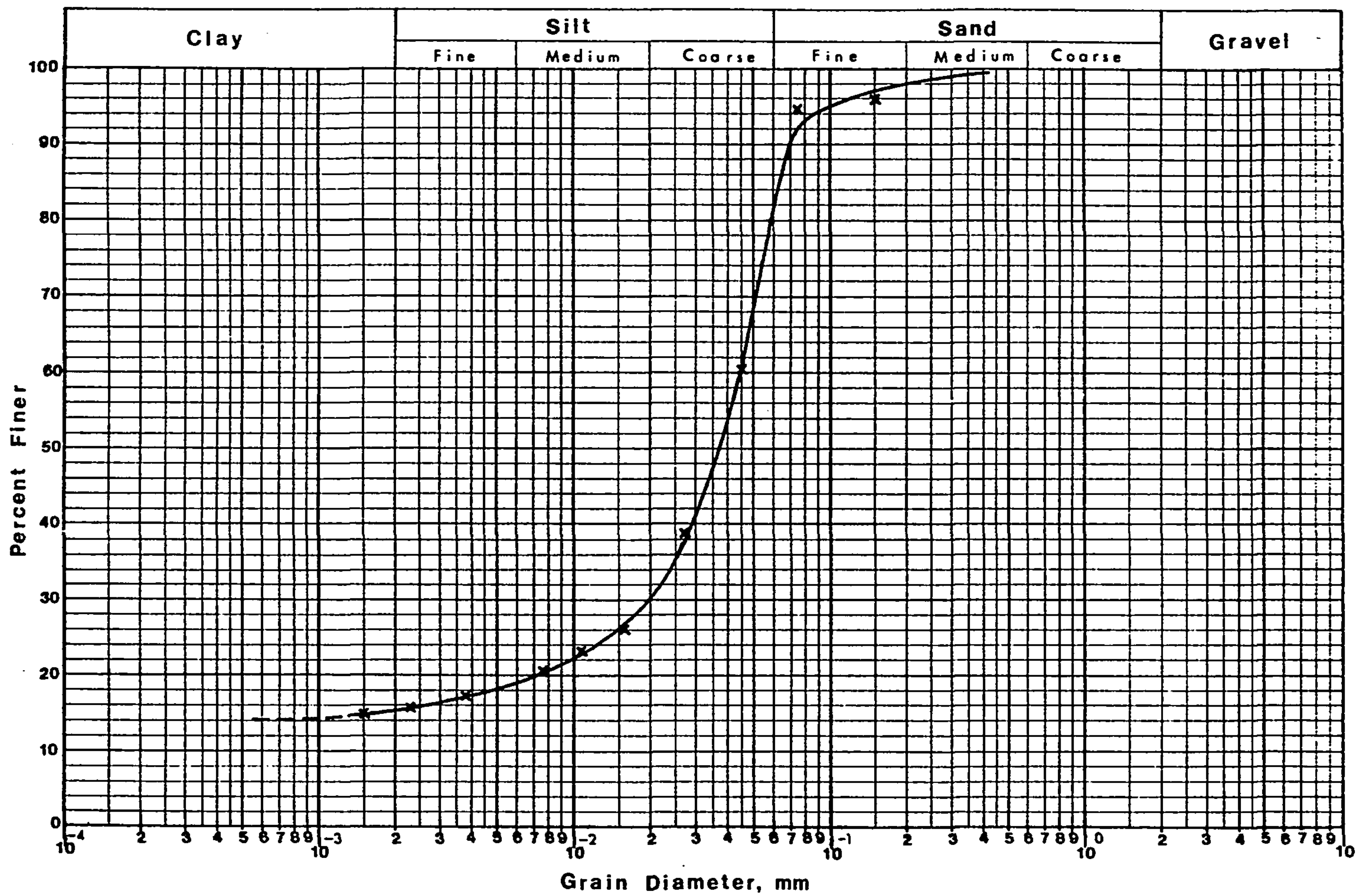


Figure B.14. Particle size distribution of the bed material at bridge "d" sample taken from the right bank 5 feet above the stream bed

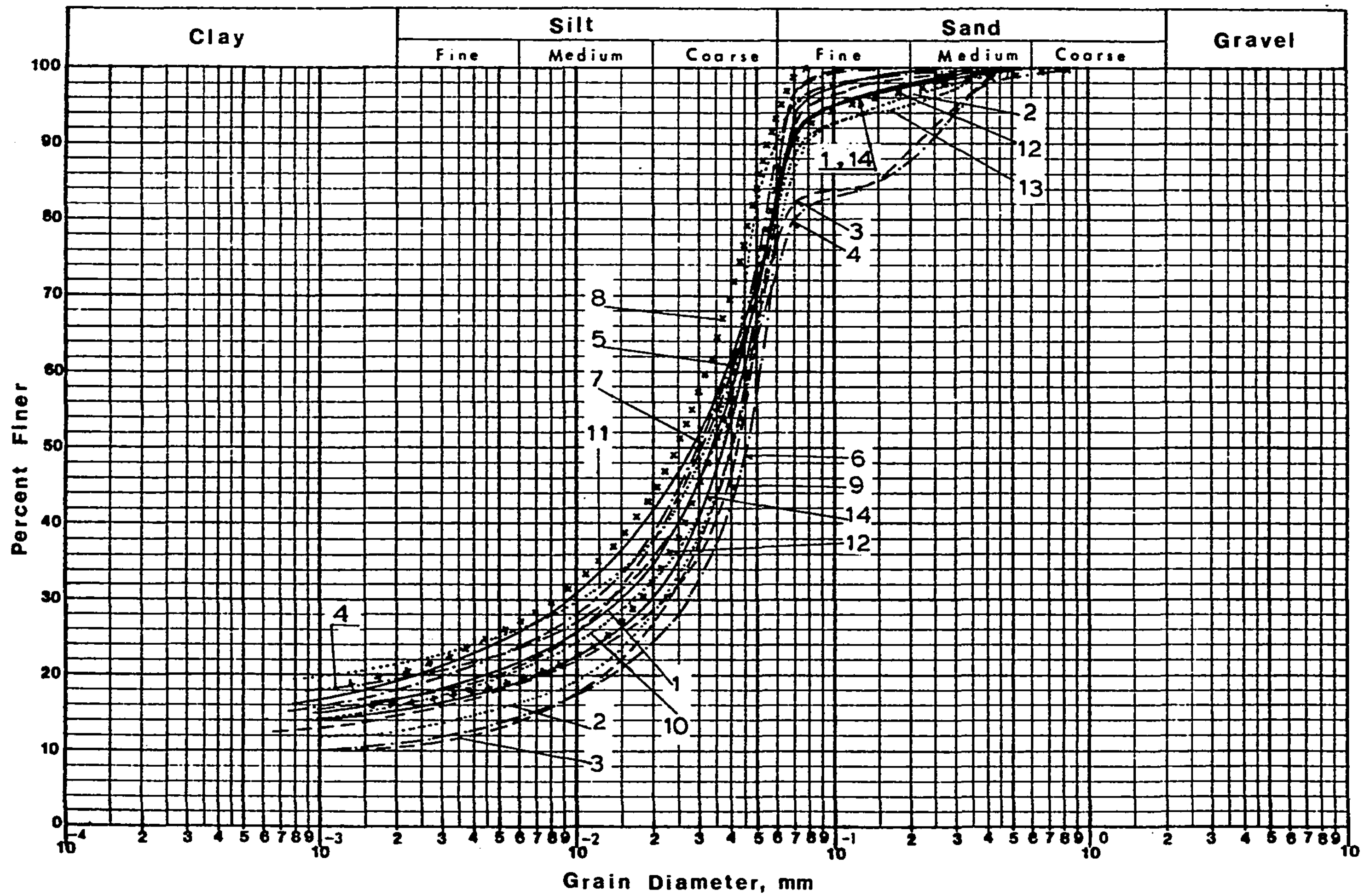


Figure B.15. Superposition of Figures B.1 through B.14

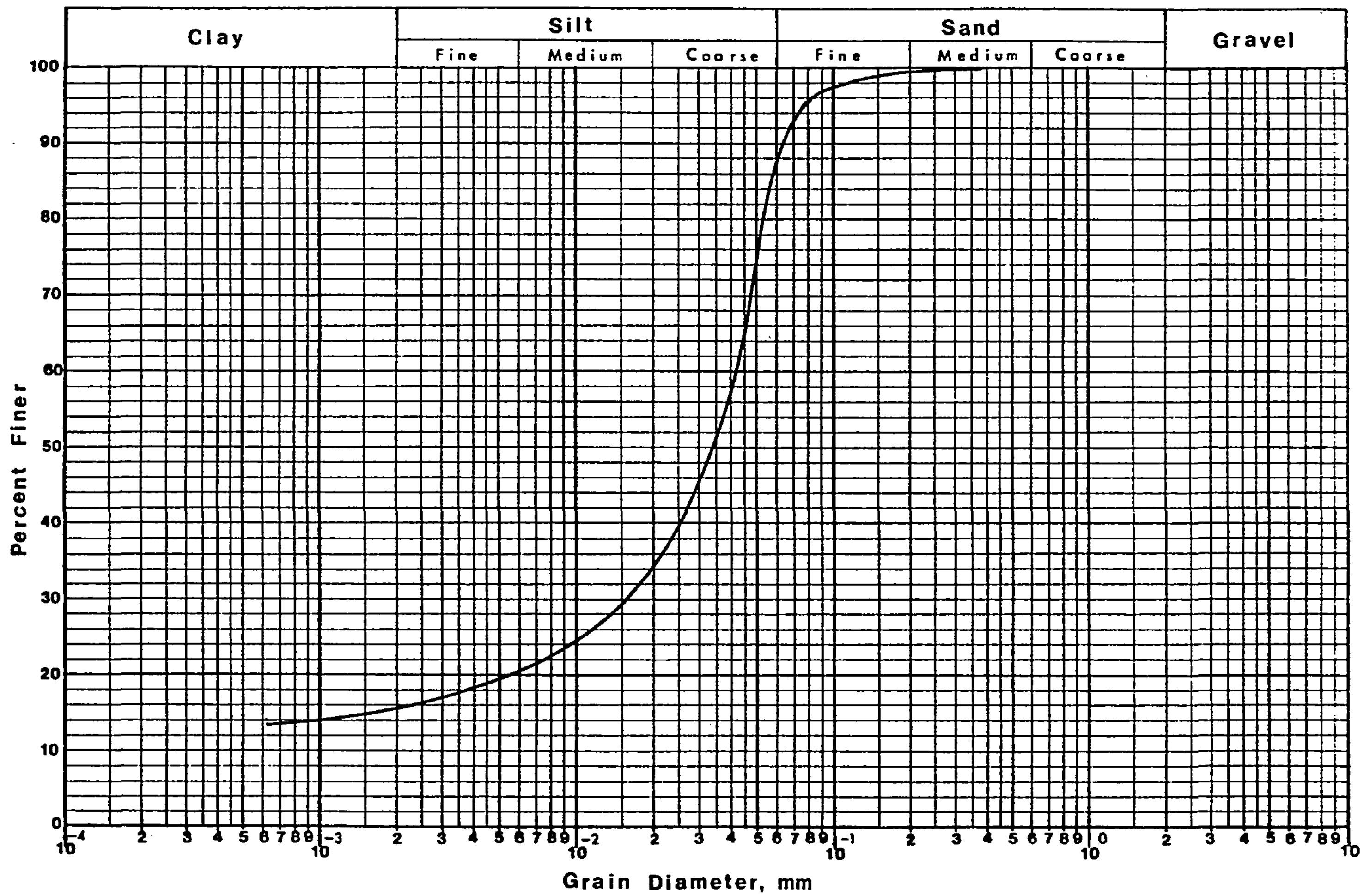


Figure B.16. Average curve for particle size distribution of the bed material of Willow Creek

APPENDIX C:

PRINT OUT OF COMPUTER PROGRAM

AND SAMPLE OUTPUT

```
C
C
C      PREDICTION OF EQUILIBRIUM LONGITUDINAL PROFILE OF WILLOW CREEK
C   ----------------------------------------------------------------
C
C       THE FOLLOWING COMPUTER PROGRAM PREDICTS THE STABLE PROFILE
C
C   OF WILLOW CREEK.THE PROGRAM HAS THE FLEXIBILITY THAT THE
C
C   RESPONCE OF THE STREAM CAN BE EVALUATED UNDER DIFFERENT LAND
C
C   USE AND FLOW CONDITIONS.THE FLEXIBILITY OF THE PROGRAM IS
C
C   REFLECTED IN 'MANNING ROUGHNESS COEFFICIENT','RECURRENCE INTERVAL',
C   'LAND USE AND SLOPE FACTOR',AND THE CRITICAL SHEAR STRESS.
C
C                PARAMETER DEFINITIONS
C               ------------------------
C
C     AUX2 = NAME OF THE FUNCTION USED IN 'ISLM' ROUTINE (ZSYSTM) TO
C            DETERMINE NORMAL DEPTH FOR A GIVEN DISCHARGE.
C
C      BWT = BOTTOM WIDTH,IN FEET.
C
C       DA = DRAINAGE AREA,IN SQUARE MILES.
C
C   DELELV = ELEVATION INCREMENT,FEET.
C
C     DINC = DISTANCE INCREMENT,FEET.
C
C      EPS = MAXIMUM ACCEPTABLE ERROR IN DISCHARGE IN EACH ITERATION.
C
C     FELV = FINAL BED ELEVATION.
C
C      FSL = FINAL SLOPE.
```

```
C
C       FWT = FINAL BOTTOM WIDTH .
C
C        HD = HORIZONTAL DISTANCE FROM DRAINAGE DIVIDE.
C
C       IER = ERROR TERM IN (ZSYSTM) .
C
C     ITMAX = MAXIMUM ALLOWABLE NUMBER OF ITERATIONS IN NORMAL
C             DEPTH CALCULATION IN EACH TRIAL.
C
C        LF = LAND USE AND SLOPE FACTOR.
C
C       MRC = MANNING ROUGHNESS COEFFICIENTS.
C
C        ND = NORMAL DEPTH,IN FEET.
C
C        NS = NUMBER OF SECTIONS.
C
C      NSIG = NUMBER OF SIGNIFICANT DIGITS USED IN (ZSYSTM).
C
C      OELV = ORIGINAL ELEVATION OF EACH SECTION.
C
C       PAR = PARAMETERS OF MANNING EQUATION USED TO CALCULATE
C             NORMAL DEPTH .
C
C        QP = PEAK DISCHARGE IN CUBIC FEET PER SECONDS.
C
C        RI = RECURRENCE INTERVAL,IN YEARS.
C
C         S = SLOPE OF EACH SECTION IN VARIOUS TRIALS.
C
C        TA = APPLIED SHEAR STRESS,IN POUND PER SQUARE FEET.
C
C       TCR = CRITICAL SHEAR STRESS,IN POUND PER SQUARE FEET.
C
```

```
C        WA = A PARAMETER USED IN 'ISLM' ROUTINE (ZSYSTM).
C
C       WDR = WIDTH TO DEPTH RATIO.
C
C    ZSYSTM = THE SELECTED 'ISLM' ROUTINE USED TO CALCULATE NORMAL
C             DEPTH FOR A GIVEN DISCHARGE.
```

```
C
C
C                THE MAIN COMPUTER PROGRAM
C             ------------------------------
C
      INTEGER  NSIG,N,ITMAX,IER
      REAL MRC,QP(50),LF,ND(300),X(1),WA(3),PAR(4),EPS,AUX2
      EXTERNAL  AUX2
      DIMENSION FELV(300),S(300,30),
     1          OELV(50),HD(50),WDR(50),BWT(300,50),
     2          FWT(50),FSL(50),DA(50),DINC(50)
C
C   READ THE REQUIRED INPUT DATA
C
      READ(5,1)NS,MRC,LF,RI,DELELV,TCR
      NK=NS+1
      READ(5,2)(OELV(I),I=1,NK)
      READ(5,2)(HD(I),I=1,NK)
    1 FORMAT(I5,5F10.5)
    2 FORMAT(7F10.3/7F10.3/7F10.3/7F10.3)
C
C   PRINT OUT THE INPUT DATA
C
      WRITE(6,40)NS,MRC,LF,RI,DELELV,TCR
   40 FORMAT('1',9X,'NUMBER OF SECTIONS,NS = ',I5///10X,
     1 'MANNING ROUGHNESS COEFFICIENT,MRC = ',F7.3,///10X,
     1 'LAND USE FACTOR,LF = ',F6.2///10X,
     2 'RECURRENCE INTERVAL,RI =',F4.0///10X,
     3 'SCOUR INCREMENT,SIN = ',F6.2///10X,
     3 'CRITICAL SHEAR STRESS,TCR = ',F6.2/5X,
     4 '----------------------------------------------------'///)
```

```
C
C     CALCULATE WIDTH TO DEPTH RATIO,BOTTOM WIDTH,AND HORIZONTAL
C     DISTANCE INCREMENT IN EACH SECTION AS A FUNCTION OF HORIZONTAL
C     DISTANCE FROM DRAINAGE DIVIDE.
C
      DO 4 I=2,NK
      JJ=I-1
      WDR(I)=-0.077*(38.25-HD(I))+5.23
C
C     THIS EQUATION IS APPLICABLE IN THE FIRST TRIAL ONLY,I.E.
C     BEFORE SCOUR INITIATES.
C
      BWT(1,I)=-1.67*(38.25-HD(I))+76.67
      DINC(I)=(HD(JJ)-HD(I))*5280.0
    4 CONTINUE
      S(1,2)=(OELV(2)-OELV(1))/(DINC(2))
C
C     COMPUTE DRAINAGE AREA AS A FUNCTION OF HORIZONTAL DISTANCE
C     FROM DRAINAGE CIVIDE.
C
      DO 7 I=1,NS
      IF(HD(I).LE.38.25) DA(I)=2.52*HD(I)+33.62
      IF(HD(I).LT.31.90) DA(I)=1.16*HD(I)+63.40
      IF(HD(I).LT.29.20) DA(I)=1.08*HD(I)+57.40
       IF(HD(I).LT.27.80.AND.HD(I).GE.17.1) DA(I)=24.23+
     1  (219.33*HD(I)-3173.67)**(1./2.)
      IF(HD(I).LT.17.10) DA(I)=1.81*HD(I)+4.69
      IF(HD(I).LT.9.60 ) DA(I)=1.34*HD(I)+0.82
      IF(HD(I).LT.4.20 ) DA(I)=1.06*HD(I)
C
C     CALCULATE THE PEAK DISCHARGE FROM EQUATION (37).
C
      QP(I)=422.58*LF*(RI)**0.301*(DA(I))**0.504
    7 CONTINUE
```

```
C
C    PRINT OUT COORDINATES,DRAINAGE AREA,AND PEAK DISCHARGE IN
C    EACH SECTION.
C
         WRITE(6,16)
   16    FORMAT(10X,'DRAINAGE AREA AND DISCHRGE IN EACH SECTION'/
      +  10X,'-------------------------------------------'/
      1  6X,'SECTION',3X,'STREAM BED',6X,'DISTANCE FROM',6X,
      2  'DRAINAGE ',7X,'DISCHARGE'/8X,'NO.',5X,'ELEVATICN',
      3  6X,'DRAINAGE DIVIDE',7X,'AREA'/15X,'(1966,FEET)',9X,
      4  '(MILES)',7X,'(SQUARE MILES)',5X,'(CFS)'/6X,'-----------------------------
      5--------------------------------------------'/)
         DO 80 I=1,NS
          KI=NS-I+1
         WRITE(6,50)I,OELV(KI),HD(KI),DA(KI),QP(KI)
   80    CONTINUE
   50    FORMAT(I11,F13.1,F16.2,F18.2,F15.0)
         WRITE(6,18)
   18    FORMAT('1',10X,'IGNORE TERMINAL ERRCRS')
C
C    INITIALIZE PARAMETERS FOR USE OF THE SELECTED 'ISLM' ROUTINE
C    (ZSYSTM) TO DETERMINE THE NORMAL DEPTH FOR CALCULATED DISCHARGE
C    IN EACH SECTICN FOR VARIOUS TRIALS.
C
          X(1)=10.0E0
          EPS=50.0E0
          NSIG=1
          N   =1
          ITMAX=5000
         DO 5 I=2,NS
         M=I+1
         MN=I-1
         J=1
         FELV(I)=OELV(I)
```

```
      PAR(1)=QP(MN)
      PAR(2)=MRC
      PAR(3)=BWT(J,I)
      PAR(4)=S(J,I)
    6 CALL ZSYSTM  (AUX2,EPS,NSIG,N,X,ITMAX,WA,PAR,IER)
C
C   COMPUTE APPLIED SHEAR STRESS IN EACH TRIAL AND COMPARE IT TO
C   THE CRITICAL SHEAR STRESS TO CHECK CHANNEL STABILITY.
C
      TA=X(1)*PAR(4)*62.4
      IF(TA.LE.TCR)GO TO 8
      L=J+1
C
C   COMPUTE NEW BOTTOM WIDTH,SLOPE,AND STREAM BED ELEVATION.
C
      BWT(L,I)=BWT(J,I)+DELELV*(WDR(I)-2.0)
      S(L,I)=S(J,I)-DELELV/DINC(I)
      FELV(I)=FELV(I)-DELELV
      J=J+1
C
C   USE NEW BOTTOM WIDTH AND SLOPE TO CALCULATE NORMAL DEPTH.
C
      PAR(3)=BWT(L,I)
      PAR(4)=S(L,I)
      GO TO 6
    8 S(1,M)=(OELV(M)-FELV(I))/DINC(M)
C
C   COMPUTE THE BOTTOM WIDTH AND THE SLOPE OF THE STABLE CHANNEL.
C
      FWT(I)=BWT(J,I)
      FSL(I)=S(J,I)
      ND(I)=X(1)
    5 CONTINUE
```

```
C
C    PRINT OUT THE FINAL RESULTS,I.E.THE CHARACTERISTICS OF THE
C    STABLE CHANNEL INCLUDING STABLE WIDTH,SLOPE,AND STREAM BED
C    ELEVATION.
C
        WRITE(6,17)
   17   FORMAT('1',19X,'COMPUTATION OF EQUILIBRIUM PROFILE'/
     + 19X,'-------------------------------------'/
     1 14X,'SECTION',9X,'DISTANCE FROM',8X,'STABLE'
     2 ,7X,'FINAL STREAM'/17X,'NO.',10X,'DRAINAGE DIVIDE',6X,
     3 'SLOPE', 7X,'BED ELEVATION'/33X,'(MILES)',
     4 27X,'(FEET)'/6X,'-----------------------------------------
     5----------------------------------')
        NK=NS-1
        DO 90 I=1,NK
         JK=NS-I+1
   90   WRITE(6,10)I,HD(JK),FSL(JK),FELV(JK)
   10   FORMAT(I20,F18.1,F19.5,F16.1)
        STOP
        END
C
C
C    THE ASSOCIATED FUNCTION TO CALCULATE NORMAL DEPTH IN THE 'ISLM'
C    ROUTINE (ZSYSTM).
C
         REAL FUNCTICN AUX2  (X,K,PAR)
         INTEGER        K
         REAL           X(1),PAR(4)
         AUX2=PAR(1)-((1.49/PAR(2))*((PAR(3)+X(1))*X(1))**(5.0/3.0)*
     1   PAR(4)**(1.0/2.0))/(PAR(3)+2.828*X(1))**(2.0/3.0)
         RETURN
         END
$ENTRY
```

```
NUMBER OF SECTIONS,NS =    27

MANNING ROUGHNESS COEFFICIENT,MRC =   0.035

LAND USE FACTOR,LF =   0.80

RECURRENCE INTERVAL,RI =  2.

SCOUR INCREMENT,SIN =   0.25

CRITICAL SHEAR STRESS,TCR =   0.85
```

DRAINAGE AREA AND DISCHRGE IN EACH SECTION

SECTION NO.	STREAM BED ELEVATION (1966,FEET)	DISTANCE FROM DRAINAGE DIVIDE (MILES)	DRAINAGE AREA (SQUARE MILES)	DISCHARGE (CFS)
1	1229.0	12.00	26.41	2169.
2	1214.0	13.00	28.22	2242.
3	1197.0	14.00	30.03	2314.
4	1178.0	15.00	31.84	2383.
5	1164.5	16.00	33.65	2450.
6	1150.5	17.10	48.25	2938.
7	1139.5	18.00	52.06	3053.
8	1130.5	19.00	55.75	3160.
9	1124.0	20.00	59.06	3253.
10	1118.0	21.00	62.08	3336.
11	1112.0	22.00	64.87	3411.
12	1106.5	23.00	67.48	3480.
13	1099.5	24.00	69.95	3543.
14	1093.0	25.00	72.29	3602.
15	1086.5	26.00	74.52	3658.
16	1079.5	27.00	76.65	3710.
17	1074.0	27.80	87.42	3965.
18	1063.0	29.20	97.27	4184.
19	1057.0	30.00	98.20	4204.
20	1049.5	31.00	99.36	4229.
21	1043.0	31.90	114.01	4532.
22	1035.5	33.00	116.78	4587.
23	1029.5	34.00	119.30	4637.
24	1024.5	35.00	121.82	4686.
25	1020.5	36.00	124.34	4735.
26	1016.5	37.00	126.86	4783.
27	1012.5	38.25	130.01	4842.

COMPUTATION OF EQUILIBRIUM PROFILE

SECTION NO.	DISTANCE FROM DRAINAGE DIVIDE (MILES)	STABLE SLOPE	FINAL STREAM BED ELEVATION (FEET)
1	12.0	0.00246	1204.3
2	13.0	0.00241	1191.3
3	14.0	0.00232	1178.5
4	15.0	0.00204	1166.3
5	16.0	0.00172	1155.5
6	17.1	0.00158	1145.5
7	18.0	0.00142	1138.0
8	19.0	0.00123	1130.5
9	20.0	0.00114	1124.0
10	21.0	0.00114	1118.0
11	22.0	0.00104	1112.0
12	23.0	0.00133	1106.5
13	24.0	0.00123	1099.5
14	25.0	0.00123	1093.0
15	26.0	0.00133	1086.5
16	27.0	0.00142	1079.5
17	27.8	0.00142	1073.5
18	29.2	0.00142	1063.0
19	30.0	0.00142	1057.0
20	31.0	0.00137	1049.5
21	31.9	0.00129	1043.0
22	33.0	0.00114	1035.5
23	34.0	0.00095	1029.5
24	35.0	0.00076	1024.5
25	36.0	0.00076	1020.5
26	37.0	0.00061	1016.5

NUMBER OF SECTIONS,NS = 27

MANNING ROUGHNESS COEFFICIENT,MRC = 0.025

LAND USE FACTOR,LF = 0.80

RECURRENCE INTERVAL,RI = 5.

SCOUR INCREMENT,SIN = 0.25

CRITICAL SHEAR STRESS,TCR = 0.85

DRAINAGE AREA AND DISCHRGE IN EACH SECTION

SECTION NO.	STREAM BED ELEVATION (1966,FEET)	DISTANCE FROM DRAINAGE DIVIDE (MILES)	DRAINAGE AREA (SQUARE MILES)	DISCHARGE (CFS)
1	1229.0	12.00	26.41	2857.
2	1214.0	13.00	28.22	2954.
3	1197.0	14.00	30.03	3048.
4	1178.0	15.00	31.84	3140.
5	1164.5	16.00	33.65	3228.
6	1150.5	17.10	48.25	3871.
7	1139.5	18.00	52.06	4022.
8	1130.5	19.00	55.75	4164.
9	1124.0	20.00	59.06	4287.
10	1118.0	21.00	62.08	4396.
11	1112.0	22.00	64.87	4494.
12	1106.5	23.00	67.48	4585.
13	1099.5	24.00	69.95	4668.
14	1093.0	25.00	72.29	4746.
15	1086.5	26.00	74.52	4820.
16	1079.5	27.00	76.65	4889.
17	1074.0	27.80	87.42	5224.
18	1063.0	29.20	97.27	5512.
19	1057.0	30.00	98.20	5539.
20	1049.5	31.00	99.36	5572.
21	1043.0	31.90	114.01	5972.
22	1035.5	33.00	116.78	6044.
23	1029.5	34.00	119.30	6110.
24	1024.5	35.00	121.82	6174.
25	1020.5	36.00	124.34	6238.
26	1016.5	37.00	126.86	6302.
27	1012.5	38.25	130.01	6380.

COMPUTATION OF EQUILIBRIUM PROFILE

SECTION NO.	DISTANCE FROM DRAINAGE DIVIDE (MILES)	STABLE SLOPE	FINAL STREAM BED ELEVATION (FEET)
1	12.0	0.00256	1206.5
2	13.0	0.00251	1193.0
3	14.0	0.00237	1179.8
4	15.0	0.00213	1167.3
5	16.0	0.00176	1156.0
6	17.1	0.00158	1145.8
7	18.0	0.00147	1138.3
8	19.0	0.00123	1130.5
9	20.0	0.00114	1124.0
10	21.0	0.00114	1118.0
11	22.0	0.00104	1112.0
12	23.0	0.00133	1106.5
13	24.0	0.00123	1099.5
14	25.0	0.00123	1093.0
15	26.0	0.00133	1086.5
16	27.0	0.00136	1079.5
17	27.8	0.00145	1073.8
18	29.2	0.00142	1063.0
19	30.0	0.00142	1057.0
20	31.0	0.00137	1049.5
21	31.9	0.00129	1043.0
22	33.0	0.00114	1035.5
23	34.0	0.00095	1029.5
24	35.0	0.00076	1024.5
25	36.0	0.00076	1020.5
26	37.0	0.00061	1016.5

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NUMBER OF SECTIONS,NS =    27

MANNING ROUGHNESS COEFFICIENT,MRC =   0.035

LAND USE FACTOR,LF =   0.80

RECURRENCE INTERVAL,RI =  5.

SCOUR INCREMENT,SIN =   0.25

CRITICAL SHEAR STRESS,TCR =   0.85
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DRAINAGE AREA AND DISCHRGE IN EACH SECTION

SECTION NO.	STREAM BED ELEVATION (1966,FEET)	DISTANCE FROM DRAINAGE DIVIDE (MILES)	DRAINAGE AREA (SQUARE MILES)	DISCHARGE (CFS)
1	1229.0	12.00	26.41	2857.
2	1214.0	13.00	28.22	2954.
3	1197.0	14.00	30.03	3048.
4	1178.0	15.00	31.84	3140.
5	1164.5	16.00	33.65	3228.
6	1150.5	17.10	48.25	3871.
7	1139.5	18.00	52.06	4022.
8	1130.5	19.00	55.75	4164.
9	1124.0	20.00	59.06	4287.
10	1118.0	21.00	62.08	4396.
11	1112.0	22.00	64.87	4494.
12	1106.5	23.00	67.48	4585.
13	1099.5	24.00	69.95	4668.
14	1093.0	25.00	72.29	4746.
15	1086.5	26.00	74.52	4820.
16	1079.5	27.00	76.65	4889.
17	1074.0	27.80	87.42	5224.
18	1063.0	29.20	97.27	5512.
19	1057.0	30.00	98.20	5539.
20	1049.5	31.00	99.36	5572.
21	1043.0	31.90	114.01	5972.
22	1035.5	33.00	116.78	6044.
23	1029.5	34.00	119.30	6110.
24	1024.5	35.00	121.82	6174.
25	1020.5	36.00	124.34	6238.
26	1016.5	37.00	126.86	6302.
27	1012.5	38.25	130.01	6380.

COMPUTATION OF EQUILIBRIUM PROFILE

SECTION NO.	DISTANCE FROM DRAINAGE DIVIDE (MILES)	STABLE SLOPE	FINAL STREAM BED ELEVATION (FEET)
1	12.0	0.00227	1193.8
2	13.0	0.00223	1181.8
3	14.0	0.00208	1170.0
4	15.0	0.00185	1159.0
5	16.0	0.00155	1149.3
6	17.1	0.00137	1140.3
7	18.0	0.00123	1133.8
8	19.0	0.00118	1127.3
9	20.0	0.00118	1121.0
10	21.0	0.00118	1114.8
11	22.0	0.00123	1108.5
12	23.0	0.00128	1102.0
13	24.0	0.00128	1095.3
14	25.0	0.00133	1088.5
15	26.0	0.00137	1081.5
16	27.0	0.00130	1074.3
17	27.8	0.00129	1068.8
18	29.2	0.00124	1059.3
19	30.0	0.00123	1054.0
20	31.0	0.00116	1047.5
21	31.9	0.00112	1042.0
22	33.0	0.00114	1035.5
23	34.0	0.00095	1029.5
24	35.0	0.00076	1024.5
25	36.0	0.00076	1020.5
26	37.0	0.00061	1016.5

NUMBER OF SECTIONS,NS =   27

MANNING ROUGHNESS COEFFICIENT,MRC =   0.045

LAND USE FACTOR,LF =   0.80

RECURRENCE INTERVAL,RI = 5.

SCOUR INCREMENT,SIN =   0.25

CRITICAL SHEAR STRESS,TCR =   0.85

DRAINAGE AREA AND DISCHRGE IN EACH SECTION

SECTION NO.	STREAM BED ELEVATION (1966,FEET)	DISTANCE FROM DRAINAGE DIVIDE (MILES)	DRAINAGE AREA (SQUARE MILES)	DISCHARGE (CFS)
1	1229.0	12.00	26.41	2857.
2	1214.0	13.00	28.22	2954.
3	1197.0	14.00	30.03	3048.
4	1178.0	15.00	31.84	3140.
5	1164.5	16.00	33.65	3228.
6	1150.5	17.10	48.25	3871.
7	1139.5	18.00	52.06	4022.
8	1130.5	19.00	55.75	4164.
9	1124.0	20.00	59.06	4287.
10	1118.0	21.00	62.08	4396.
11	1112.0	22.00	64.87	4494.
12	1106.5	23.00	67.48	4585.
13	1099.5	24.00	69.95	4668.
14	1093.0	25.00	72.29	4746.
15	1086.5	26.00	74.52	4820.
16	1079.5	27.00	76.65	4889.
17	1074.0	27.80	87.42	5224.
18	1063.0	29.20	97.27	5512.
19	1057.0	30.00	98.20	5539.
20	1049.5	31.00	99.36	5572.
21	1043.0	31.90	114.01	5972.
22	1035.5	33.00	116.78	6044.
23	1029.5	34.00	119.30	6110.
24	1024.5	35.00	121.82	6174.
25	1020.5	36.00	124.34	6238.
26	1016.5	37.00	126.86	6302.
27	1012.5	38.25	130.01	6380.

COMPUTATION OF EQUILIBRIUM PROFILE

SECTION NO.	DISTANCE FROM DRAINAGE DIVIDE (MILES)	STABLE SLOPE	FINAL STREAM BED ELEVATION (FEET)
1	12.0	0.00213	1182.5
2	13.0	0.00208	1171.3
3	14.0	0.00194	1160.3
4	15.0	0.00175	1150.0
5	16.0	0.00146	1140.8
6	17.1	0.00132	1132.3
7	18.0	0.00118	1126.0
8	19.0	0.00114	1119.8
9	20.0	0.00114	1113.8
10	21.0	0.00118	1107.8
11	22.0	0.00118	1101.5
12	23.0	0.00123	1095.3
13	24.0	0.00123	1088.8
14	25.0	0.00123	1082.3
15	26.0	0.00128	1075.8
16	27.0	0.00118	1069.0
17	27.8	0.00118	1064.0
18	29.2	0.00112	1055.3
19	30.0	0.00109	1050.5
20	31.0	0.00100	1044.8
21	31.9	0.00099	1040.0
22	33.0	0.00095	1034.3
23	34.0	0.00090	1029.3
24	35.0	0.00076	1024.5
25	36.0	0.00076	1020.5
26	37.0	0.00061	1016.5

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NUMBER OF SECTIONS,NS =    27

MANNING ROUGHNESS COEFFICIENT,MRC =   0.035

LAND USE FACTOR,LF =   0.80

RECURRENCE INTERVAL,RI = 10.

SCOUR INCREMENT,SIN =   0.25

CRITICAL SHEAR STRESS,TCR =   0.85
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DRAINAGE AREA AND DISCHRGE IN EACH SECTION

SECTION NO.	STREAM BED ELEVATION (1966,FEET)	DISTANCE FROM DRAINAGE DIVIDE (MILES)	DRAINAGE AREA (SQUARE MILES)	DISCHARGE (CFS)
1	1229.0	12.00	26.41	3520.
2	1214.0	13.00	28.22	3640.
3	1197.0	14.00	30.03	3756.
4	1178.0	15.00	31.84	3868.
5	1164.5	16.00	33.65	3977.
6	1150.5	17.10	48.25	4770.
7	1139.5	18.00	52.06	4956.
8	1130.5	19.00	55.75	5130.
9	1124.0	20.00	59.06	5281.
10	1118.0	21.00	62.08	5415.
11	1112.0	22.00	64.87	5537.
12	1106.5	23.00	67.48	5648.
13	1099.5	24.00	69.95	5751.
14	1093.0	25.00	72.29	5847.
15	1086.5	26.00	74.52	5938.
16	1079.5	27.00	76.65	6023.
17	1074.0	27.80	87.42	6435.
18	1063.0	29.20	97.27	6791.
19	1057.0	30.00	98.20	6824.
20	1049.5	31.00	99.36	6864.
21	1043.0	31.90	114.01	7357.
22	1035.5	33.00	116.78	7447.
23	1029.5	34.00	119.30	7527.
24	1024.5	35.00	121.82	7607.
25	1020.5	36.00	124.34	7686.
26	1016.5	37.00	126.86	7764.
27	1012.5	38.25	130.01	7860.

COMPUTATION OF EQUILIBRIUM PROFILE

SECTION NO.	DISTANCE FROM DRAINAGE DIVIDE (MILES)	STABLE SLOPE	FINAL STREAM BED ELEVATION (FEET)
1	12.0	0.00213	1184.5
2	13.0	0.00213	1173.3
3	14.0	0.00199	1162.0
4	15.0	0.00180	1151.5
5	16.0	0.00146	1142.0
6	17.1	0.00132	1133.5
7	18.0	0.00123	1127.3
8	19.0	0.00114	1120.8
9	20.0	0.00114	1114.8
10	21.0	0.00118	1108.8
11	22.0	0.00118	1102.5
12	23.0	0.00123	1096.3
13	24.0	0.00123	1089.8
14	25.0	0.00123	1083.3
15	26.0	0.00128	1076.8
16	27.0	0.00124	1070.0
17	27.8	0.00118	1064.8
18	29.2	0.00112	1056.0
19	30.0	0.00114	1051.3
20	31.0	0.00105	1045.3
21	31.9	0.00099	1040.3
22	33.0	0.00095	1034.5
23	34.0	0.00095	1029.5
24	35.0	0.00076	1024.5
25	36.0	0.00076	1020.5
26	37.0	0.00061	1016.5