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UNITED STATES
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GUIDE FOR COMPUTING WATER SURFACE PROFILES

Sedimentation Section Report

HYDROLOGY BRANCH

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ACKNOWLEDGMENT

This technical guide was prepared by Joe M. Lara under the immediate supervision of Kenneth B. Schroeder and Whitney M. Borland, Assistant Head, and Head, Sedimentation Section, respectively. The Sedimentation Section is a subunit of the Hydrology Branch, Herbert S. Riesbol, Chief, which is under the Division of Project Investigations, John R. Riter, Chief Development Engineer. This Division is responsible to the Assistant Commissioner and Chief Engineer, L. N. McClellan, Bureau of Reclamation, U. S. Department of the Interior.

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INTRODUCTION

Water surface profiles are required for two principal uses by the Bureau of Reclamation; one--to determine tail water rating curves, and two--to derive the backwater curves above a dam. Project development is facilitated by including in the investigational work program, the routine collection of the required field data for the derivation of these curves as recommended in this guide. Generally, this objective will not require any special effort, since it can be fitted into the scheme of routine field observations made during the course of preliminary surveys. Usually these data are sufficiently accurate for deriving tail water and back water curves to be used either in the preparation of feasibility grade designs and estimates or in the preparation of final designs and specifications. However, accuracy is dependent on a realistic analysis of the field data collected and a selection of the proper method to make the office computations. Thus, inferior office computational procedures applied to a good set of field data, and conversely, elaborate computations applied to an inferior set of data can destroy or reduce the accuracy of the derived water surface profiles.

The information presented in the forepart of this guide is particularly suited to the development of water surface profiles to be used in the preparation of final designs and specifications or in the analysis of reconnaissance and feasibility grade studies where a high degree of accuracy is required. Information on the necessary field data and applicable methods for the development of backwater and tail water curves having a degree of accuracy suitable for the usual reconnaissance and feasibility studies or for final designs and specifications where a high degree of accuracy is not required are contained in the

appendix to this guide. Selection of the proper procedure to be followed requires the exercise of sound engineering judgment on behalf of the planning engineer. The exercise of such judgment is a necessity if we are to plan projects adequately with a minimum of cost.

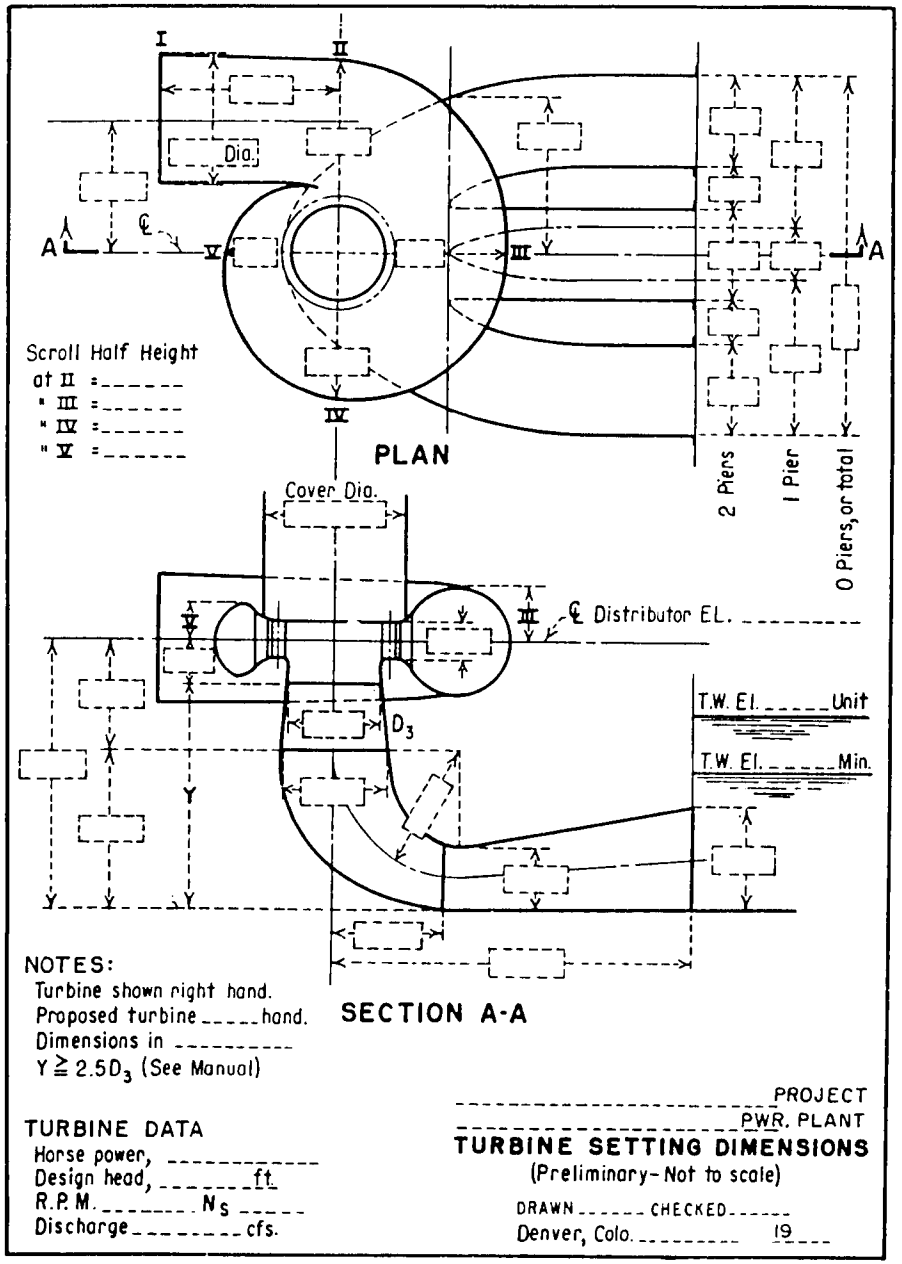
The determination of the degree of accuracy required in the preparation of reconnaissance and feasibility studies for any specific structure within a project plan is dependent upon the degree of effect which the water surface profile will have upon the cost of the particular structure and, in turn, upon the proportion of cost of the particular structure in terms of the total plan. For example, the cost of a small diversion dam may be greatly influenced by the tail water condition and yet the cost of that diversion dam in terms of proportion of the entire plan may be so insignificant that it would have no effect on the feasibility of the plan. In other cases such as the back water profile where a reservoir may encroach upon a town and other improvements, a relatively high degree of accuracy may be required in order to establish limits for reservoir capacity regardless of the cost differentials involved. Again, where a tail water curve is to be used for the design of a stilling basin or the evaluation of a low head powerplant, only a fair degree of accuracy is required for reconnaissance and feasibility reports. As another example, in the case of emergency spillways discharging into an adjacent draw, or in the case of ski jump and tunnel outlet wpillways, very little tail water information is required.

The tail water curve plays a very important part in the design of power and pumping plants. Figures 1 and 2 are preliminary

design data sheets showing the tail water elevations of immediate importance for power and pumping plants, respectively. Minimum tail water elevations should be determined with sufficient accuracy to maintain adequate submergence of the draft tubes as noted in the figures. Tail water elevations are necessary in determining the limiting heads for turbine operation and maximum efficiency. For example, hydraulic turbines may have objectionable vibration when operating under maximum tail water elevations due primarily to the inability of the unit to admit atmospheric air below the turbine runner when operating at the lower gate positions. Upper floor levels of pumping and powerplants are located at planes higher than maximum tail water elevations for the protection of electrical and other damageable hydraulic equipment. An appreciable error in determining the maximum tail water elevation will result in a higher and, consequently, costlier structure. Minimum and normal tail water elevations should be determined within the practical limit of $\pm$ 1 foot, although construction costs will usually tolerate a somewhat larger error.

The tail water rating curve furnishes the basic data needed in the design of energy dissipators. The sketch on the following page illustrates an application of the tail water curve data to the design of a stilling basin, one of the most common forms of energy dissipators of spillways and outlet works. The objective is to obtain the best practical match of the selected tail water elevation with the conjugate depth (D_2) of the hydraulic jump throughout the range in discharge.

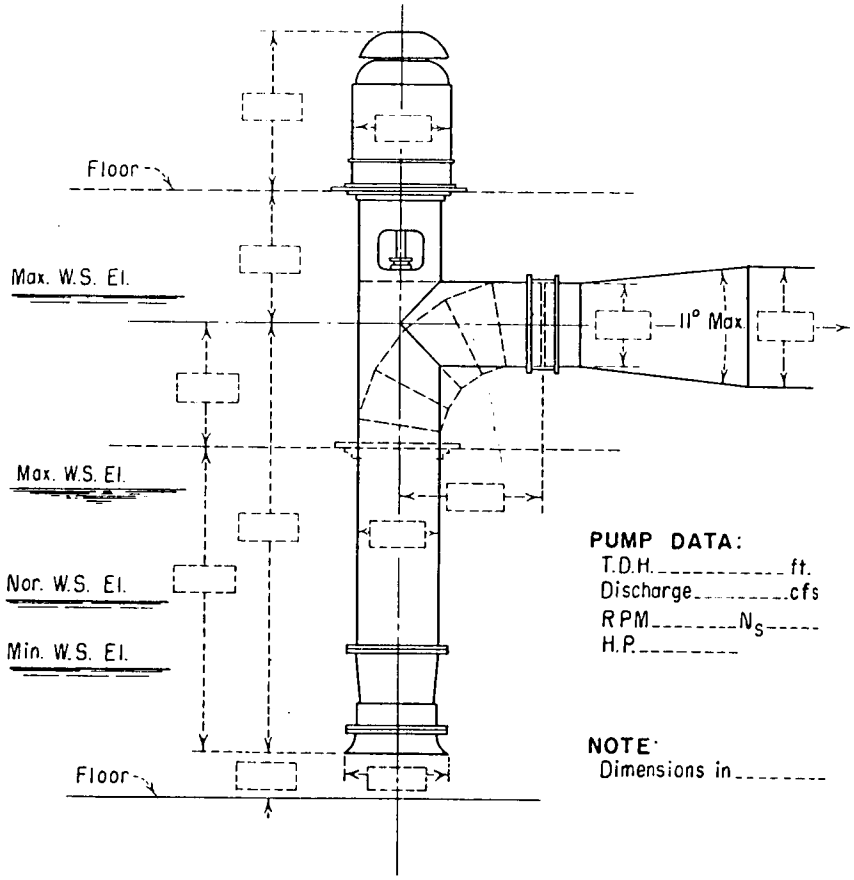
Figure 1



TURBINE DATA
Horse power,
Design head, ft.
R.P.M. N_s
Discharge cfs.

PROJECT
PWR. PLANT
TURBINE SETTING DIMENSIONS
(Preliminary-Not to scale)
DRAWN CHECKED
Denver, Colo. 19

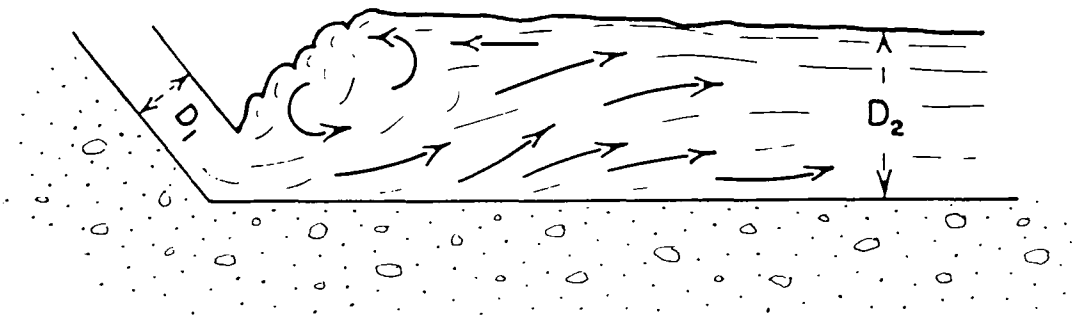
Figure 2



PUMP DATA:
T.D.H. ft.
Discharge cfs
RPM N_s
H.P.

NOTE:
Dimensions in

PROJECT
Pmp. Plant
PUMP SETTING DIMENSIONS
(Preliminary-Not to scale)
Denver, Colo. 19



The determination of the length, width, and depth of the stilling basin is directly related to the tail water curve. The designer selects the most economic arrangement of the stilling basin which fulfills the governing hydraulic requirements. It is desirable to determine the tail water elevations throughout the operating range. The jump will not form if the floor of the stilling basin is too high and, therefore, the pool will be ineffective *[1]. If the floor is too low, the pool efficiency is reduced with a concomitant increase in costs.

In the design of large dams, tail water elevations are needed in making stability and stress analyses. The analysis of forces acting on the dam due to earthquakes also utilizes tail water data. Tail water curves are also required for stress and stability studies of diversion dams. Data from the curves are necessary in order to allow proper provision in the design for energy dissipation. Data of downstream water surface elevations may also be necessary to determine the flood discharge in cases of submerged overpour. Again, tail water elevations should be determined within the overall limit of ± 1 foot.

---

*Numbers in brackets refer to items in the Bibliography, page

The backwater profile above a dam and reservoir is required for reservoir area land acquisitions, to determine the effects of the reservoir on existing improvements or developments such as pumping plants, powerplants, towns, buildings, railroad and highway locations, to control the location of alterations and replacements due to inundation of present facilities, and to assist the design of new facilities along or above the reservoir.

The accuracies quoted above are those desired by the designers. However, water surface profiles developed by theoretical computations may often be considered accurate only within a practical range of 3 to 5 feet. This is due to the limitations of available data and to the various assumptions that must be made in analyzing the prevailing hydraulic conditions of channels in their natural state. Observed profiles furnish the most accurate information.

The hydraulic engineer should include in his studies a notation concerning the practical limit of accuracy of the tail water curve. The design engineer, in turn, can make the necessary adjustments in his designs to account for the difference between desired and derived accuracies. Additional factors affecting the accuracy are the disruptions of the channel hydraulics caused by construction activities and downstream degradation due to clear water releases from the reservoir. No attempt is made in this guide, however, to expand on these factors since each must be treated separately. The effects of construction activities are

determined directly by field observations and inspections on the site. The preparation of a special study is often necessary to determine the anticipated degradation.

The symbols used in this report are defined as they appear in the text. For additional convenience, the definitions have been included in the Glossary at the end of the report. Numbers in brackets refer to the Bibliography, which has also been placed at the end of the report.

THEORY

Six theoretical methods of computing water surface profiles are outlined in this guide. These are: (1) the Standard Step Method by Woodward and Posey, (2) Leach's Method, (3) Methods I and II, Corps of Engineers, U. S. Army, and (4) Methods A and B developed by the Bureau of Reclamation.

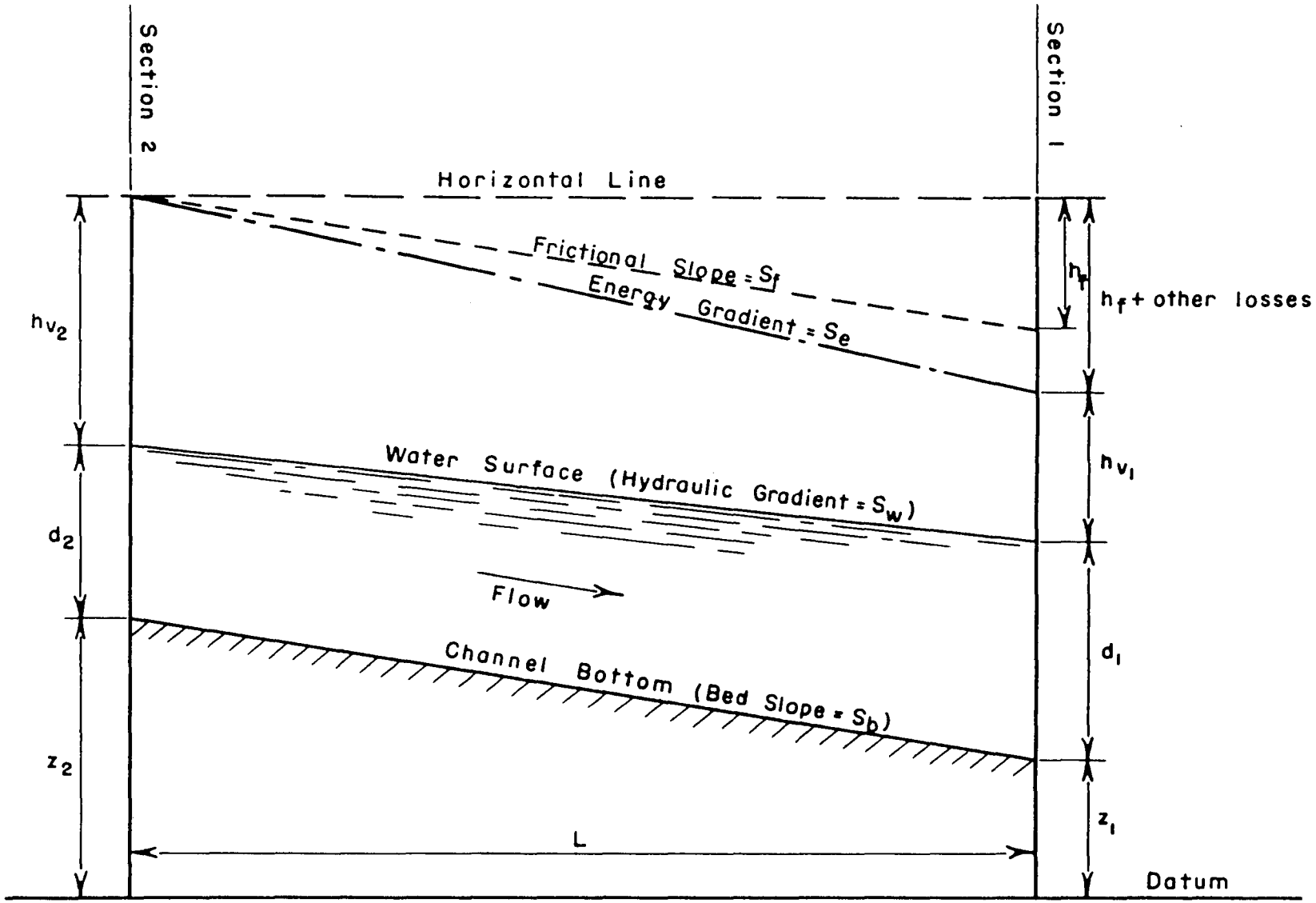
The basic underlying theory of all these methods is the Bernoulli Energy Equation. To understand the basic principle of water surface profile computations, a discussion of Bernoulli's equation is presented.

The conditions of an open channel reach in steady non-uniform flow are diagrammatically illustrated in Figure 3, which appears on the following page.

The energy balance equation may be written

$$Z_2 + d_2 + h_{v_2} = Z_1 + d_1 + h_{v_1} + h_f + \text{other losses}$$

Figure 3



ENERGY OF OPEN-CHANNEL FLOW

where

Z_2 = streambed elevation referenced to a given datum (upstream section)

d_2 = depth of flow at upstream section

h_{v_2} = velocity head of upstream section

Z_1 = streambed elevation referenced to a given datum (downstream section)

d_1 = depth of flow at downstream section

h_{v_1} = velocity head of downstream section

h_f = friction head loss

Other losses = eddy, bend, and bridge pier losses.

All of the methods that will be outlined in this guide use basically the following hydraulic relationships to evaluate the various components of the energy balance equation

$$Q = AV, \quad V = Q/A, \quad h_v = \frac{V^2}{2g}$$

$$h_f = L\left(\frac{Qn}{1.486\ AR^{2/3}}\right)^2$$

where

Q = discharge in cubic feet per second

A = cross-sectional area in square feet

V = average velocity of flow in feet per second

g = gravitational acceleration in feet per second squared

L = reach length between sections in feet

n = roughness coefficient representing the reach

R = hydraulic radius which is computed by dividing the cross-sectional area by the wetted perimeter.

It is noted from above that Manning's equation is used to compute h_f. The normal expression of Manning's discharge equation is as follows:

$$Q = \frac{1.486}{n} AR^{2/3} \quad S_e^{1/2}$$

where S_e is the slope of the energy gradient.

Losses

The total losses in the energy balance equation consist of the friction head h_f, eddy loss, bend loss, and bridge pier loss.

The friction head can be determined by applying the Manning equation to express the friction slope S_f:

$$S_f = \left(\frac{Qn}{1.486\ AR^{2/3}} \right)^2$$

Expressing S_f as h_f/L, the total friction head for a reach can then be resolved by the following equation:

$$h_f = L \left(\frac{S_{f_1} + S_{f_2}}{2} \right)$$

where S_{f_1} and S_{f_2} are the frictional slopes derived by using the section characteristics of n, A, and R for Sections 1 and 2, respectively.

A simplified and practical concept of eddy loss can be understood better from a study of the energy losses incident to the conversion of kinetic head to static head and vice versa in open channel flow. While there are, at present, no methods available for quantitatively determining these losses, it is known they do occur and that they manifest themselves usually in the form of turbulence. Channel flow, where there are abrupt changes in velocity, clearly displays turbulence by the presence of boils and vortexes. This is particularly evident when there is an abrupt and large decrease in velocity resulting in a conversion of kinetic to static head.

It has been a general practice in the Bureau of Reclamation to compute the eddy loss as a 10-percent difference in the energy heads (Δh_v) when static head is converted to kinetic head and 50 percent for a reverse condition. Thus, when the value of

$h_{v_1} - h_{v_2}$ is positive, a 10-percent correction is applied and for negative values a 50-percent correction is used. Regardless of sign, this value in the eddy loss is added to the friction loss to obtain the total head loss for the reach.

There are special cases, however, when departures from the 50-percent value quoted above may be necessary. This may apply in cases where there are extreme changes in velocities occurring in the channel reach. This condition results in transitions from a very narrow section to a wide section. Large standing waves are formed indicating that the flow is bordering the critical stage. As the flow progresses downstream, the energy is dissipated in turbulence and as the flow reaches the downstream section, it reverts to a more tranquil state. In these cases, the eddy loss could reasonably be increased to as high as 100 percent of the change in velocity heads.

By always applying a 50-percent correction in the computations, a situation can result showing a higher water surface elevation at the downstream section than at the upstream section ($Z_1 + d_1 > Z_2 + d_2$), which may be an unrealistic condition. To correct this condition, an adjustment can be made by increasing the percentage applied. Eddy losses, at best, are only estimates.

There have been no formulas developed yet for a thorough determination of bend losses in open channel flow. A number of formulas resulting from various studies are listed below:

1. $$h_b = 0.21 \frac{\text{channel width}}{\text{inner radius}} \left(h_v \right)$$

from Yarnell and Woodward [5]

2. "n" increased 0.001 for each 20° of curvature

suggested by Scobey [6]

3. $h_b = 0.25 \frac{\Delta}{90°} h_v$

where Δ = angle of bend

4. $h_b = 0.38 h_v$

by C. H. Yen [7]

5. $h_b \frac{0.0256 V^2 \text{ x width of stream}}{\text{graphic radius of curvature}}$

by Boer and Urick [8]

No criteria have been set for a practical evaluation of bend losses. Two methods are suggested; first--they could be computed by any one of several theoretical formulas, and second--"n" values could be modified to include bend loss effects. Losses determined by the first method are very theoretical and require analyses beyond the limits of the collected field data. It is recommended, therefore, that "n" coefficients be assigned values to include any significant effects of bend losses.

The hydraulic conditions at bridge crossings must be carefully analyzed to determine bridge pier losses. King's Handbook [9] recommends contraction and enlargement coefficients of 0.5 and 1.0, respectively, applied to the changes in velocity heads at the approach, bridge, and emergent sections. Nagler [10], Yarnell [11] [12], and Johnson [13] have developed formulas for computing the head loss through bridges, which may be applied under certain conditions.

Other methods for determining the backwater curves through bridges will be treated in more detail under examples of the methods for computing water surface profiles.

METHODS

In the following explanation of the various methods, it is assumed that the elevation-discharge relation is known for the most downstream section. Water surface profiles are computed, advancing in the upstream direction.

Standard Step Method

The Standard Step Method developed by Woodward and Posey is a trial and error procedure. Referring again to the diagrammatic sketch in Figure 3, the objective is to determine the water surface elevation for a given discharge at Section 2 when it is known at Section 1. This method is used when the flow is contained in one relatively uniform channel, and all portions of the flow will travel the same distance between Sections 1 and 2.

With a known water surface at Section 1, then the values of A, "n", and R are determinable for that elevation. The water surface elevation $(Z_1 + d_1)$ plus h_{v_1} determines the elevation of the energy gradient at Section 1. For Section 2, the first step is to assume a water surface elevation $(Z_2 + d_2)$. $Z_2 + d_2 + h_{v_2}$ will determine the elevation of the energy gradient at Section 2.

Examination of the energy balance equation $Z_2 + d_2 + h_{v_2} = Z_1 + d_1 + h_{v_1} + h_f$ + other losses, shows that a determination must be made for the components, h_f + other losses.

The value of h_f is determined first by computing the friction slope, S_f, at Section 1 using Manning's equation as follows:

$$S_f^{1/2} = \frac{Qn}{1.486\ AR^{2/3}}$$

letting

$$Y = \frac{1}{1.486\ R^{2/3}}$$

then

$$S_f = \left(\frac{Qn\ Y}{A} \right)^2$$

Values of Y have been tabulated in Table 19.

A similar computation is made for the friction slope at Section 2. Then the total friction head, h_f, is computed from the formula $h_f = L \left(\frac{S_{f_1} + S_{f_2}}{2} \right)$.

In the normal computation, the other loss item consists of the eddy loss. The eddy loss is computed from the formula: eddy loss $= M(h_{v_1} - h_{v_2})$ where M is a percentage figure that is arbitrarily selected as 0.10 for positive values of $h_{v_1} - h_{v_2}$ and 0.50 for negative values of $h_{v_1} - h_{v_2}$. All elements of the energy equation have been determined from the hydraulic properties established by assuming the water surface elevation at Section 2. If the equation is balanced, the assumed elevation is correct for the given discharge.

If the equation is not balanced, then other water surfaces at Section 2 are assumed until a balance is reached. In general practice, the equation is considered balanced when agreement of .1 foot is reached. Closer agreements can be attained by securing more accurate basic field measurements.

The Standard Step computation is usually set up in a tabular form for orderly solution. The form used in this guide is the same as the Corps' Method I with some of the columns omitted. A step computation and example are presented on pages 114 to 121.

Leach's Method

Leach's Method is used where the flow travel path between sections is essentially equal for the overbank and main channel segments of the flow and may be used when the cross section has considerable variation in the hydraulic characteristics of "n", A, and R, although it is better applicable to conditions approaching uniform flow. This method fulfills only the following portion of the energy balance equation

$$Z_2 + d_2 = Z_1 + d_1 + h_f .$$

Since

$$h_f = S_f L ,$$

then

$$Z_2 + d_2 = Z_1 + d_1 + S_f L .$$

In this method, S_f, the friction slope, is determined by applying the conveyance principle where the conveyance capacity, K_d, is defined for any channel cross-sectional subdivision as follows:

$$K_d = \frac{1.486}{n} AR^{2/3}$$

Substituting the above relationship in Manning's formula for discharge, there results:

$$Q = K_d \, S_f^{1/2}$$

Solving for S_f :

$$S_f = \left(\frac{Q}{K_d} \right)^2$$

In application, the K_d's of each subdivision are the first computed for common elevations. For each section, a total K_d value at a given elevation is computed by summing up the individual K_d's of each subdivision. A set of conveyance curves for each sectional subdivision is plotted on a semilogarithmic graph with elevation as the ordinate and K_d as the abscissa.

The friction slope, S_f , is computed from the above equation using an average K_d value determined from Sections 1 and 2 as follows:

$$S_f = \left(\frac{Q}{\text{Ave } K_d} \right)^2$$

where $\text{Ave } K_d = \dfrac{K_{d_1} + K_{d_2}}{2}$

h_f is then resolved from the formula, $h_f = S_f \, L$.

The Leach computation is best handled by tabular computation. The form, step computation, and example are given on pages 122 to 126.

Method I--Corps of Engineers, U. S. Army

The Corps of Engineers' Method I has been developed for computation of profiles where the flow travel between sections is essentially equal for all segments of the flow, but the cross section throughout its width has considerable variation in the hydraulic characteristics of "n", A, and R. A typical section of this type is presented in Figure 7.

The flow conveyance in a section of this type is considerably different because of a wide variation in the hydraulic parameters of "n", A, and R. This will also be evident from the variation in velocity.

To estimate an average overall "n" value for the section is very difficult from a practical standpoint. It is more convenient to estimate separate "n" values for the main channel and overbank segments.

The computation of the R parameter is considered next. When the main channel is bankfull, the R value is high. As the flow just exceeds the bankfull condition, a single section computation of R will result in a lower R value than for the bankfull condition because there is a small increase in area accompanied by a large increase in wetted perimeter. For a given energy gradient and using this lower R value, the computed discharge would tend to be much smaller than the actual discharge.

There is also the consideration of computing the velocity head h_v. An average velocity at a section computed by using the total area of the main channel and overbank sections would not be

representative of the existing velocity in an open channel. Unrealistic velocity heads would result by using such an average velocity. Usually, there are significant differences in velocities occurring in the main channel and overbank sections. A more representative velocity head is computed by weighting the flow and velocity for the various segments.

The Corps of Engineers' Method I computation is made by determining the hydraulic parameters of "n", A, and R individually for each of the segments of flow. In the computation of R, the wetted perimeter is considered as the actual water-earth boundary for subdivisions of the cross section. The vertical water depth between subdivisions does not constitute a part of the wetted perimeter.

The first step is a computation of the friction slope S_f for each section. This can be conveniently derived by assuming a slope of 0.0001 for the section. The discharge in each segment is then computed by Manning's formula using this slope. The actual friction slope, S_f, for the section can be determined from the equation $S_f = \left(\frac{Q \times .01}{Q_{.01}} \right)^2$, which is derived as follows:

$$Q_{.01} = \frac{1.486}{n} AR^{2/3} \; S_{.01}^{1/2} \; .$$

From $S_{.01} = 0.0001$, $S_{.01}^{1/2} = 0.01$, and substituting in the above equation:

$$Q_{.01} = \frac{1.486}{n} AR^{2/3} \; 0.01$$

Combining the above equation with Manning's general formula, there results:

$$\frac{Q}{Q_{.01}} = \frac{1.486/n\ AR^{2/3}\ S_f^{1/2}}{1.486/n\ AR^{2/3}\ \ 0.01}$$

Cancelling n , 1.486, A, and $R^{2/3}$ in the equation:

$$\frac{Q}{Q_{.01}} = \frac{S_f^{1/2}}{0.01}$$

which resolves to the desired equation:

$$S_f = \left(\frac{Q \times 0.01}{Q_{.01}} \right)^2$$

It must be remembered that $Q_{.01}$ represents the summation of the subdivisional flows where the slope has been assumed equal to 0.0001.

S_f may be computed from the known water surface at Section 1 and from the assumed water surface at Section 2. The h_f component of the energy balance equation is

$$h_f = L \left(\frac{S_{f_1} + S_{f_2}}{2} \right)$$

The next step is evaluating the h_v components of the energy balance equation. This is accomplished by determining the velocities existing in each of the sectional subdivisions. First, the discharge in each of the segments is found from the relationship:

$$\frac{Q}{Q_{.01}} \times (Q_{.01})_s = Q_s$$

where Q is the total discharge for the entire cross section, $Q_{.01}$ is the total discharge in the channel for an S_f of 0.0001, $(Q_{.01})_s$ is the discharge of the individual subdivision for an S_f of 0.0001, and Q_s is the actual discharge occurring in the subdivision. The velocity for each segment can then be computed by the equation

$$V_s = \frac{Q_s}{A_s}$$

where A_s is the area of the segment.

The velocity head is eventually resolved by a weighting process using the following equation:

$$h_v = \frac{\Sigma \left(V_s^2 \, Q_s \right)}{2g \, Q}$$

The remaining component of the energy equation is the other losses of which, in general, the only significant component is the eddy loss. This again is computed by the formula: eddy loss = $M\,(h_{v_1} - h_{v_2})$ where M is 0.10 for positive values of $h_{v_1} - h_{v_2}$ and 0.50 for negative values of $h_{v_1} - h_{v_2}$.

The Corps of Engineers' Method I can be conveniently handled by a tabular computation. The form used, the step computation, and example are presented on pages 105 to 113.

Method II--Corps of Engineers, U. S. Army

The Corps of Engineers' Method II is adapted to the condition of unequal travel lengths for the flow between sections and when segments of the section have variable conveyance parameters of "n", A, and R. It is easier to visualize this condition by

considering a stream with a main channel having a large meander pattern. The flow of the main channel with a large conveyance will have a much longer travel length than the overbank flow which can cut across the meander. However, overbank areas usually have a smaller conveyance because of trees, brush, or other vegetation.

Referring to the diagrammatic sketch in Figure 3, the components of the energy balance equation are $Z_2 + d_2 + h_{v_2} = Z_1 + d_1 + h_{v_1} + h_f$ + other losses. The objective is to determine the water surface elevation at Section 2 with a known water surface elevation at Section 1.

The first step in this method is determining h_f when a water surface elevation at Section 2 is assumed. In the computations, the water surface is assumed to be level, transversely, to the cross section. Regardless of the travel lengths, all segments of flow will have the same friction head, then $h_f = L\ S_f$ and $S_f = \frac{h_f}{L}$. Substituting in the Manning equation gives

$$Q = \frac{1.486}{n} AR^{2/3} \left(\frac{h_f}{L} \right)^{1/2}$$

Since h_f is common to all segments, the formula may be written:

$$Q_s = \frac{1.486}{n_s} A_s R_s^{2/3} \left(\frac{h_f}{L_s} \right)^{1/2}$$

where the subscripts refer to a specific set of segmental hydraulic properties.

Let $\dfrac{1.486\ A_s\ R_s^{2/3}}{n_s\ L_s^{1/2}}$ equal K_s''

then $Q_s = K_s''\ h_f^{1/2}$ and $h_f = \left(\dfrac{Q_s}{K_s''}\right)^2$

For convenience let $Y_s = \dfrac{1}{1.486\ R_s^{2/3}}$

K_s'' can then be written

$$K_s'' = \frac{A_s}{n_s\ Y_s\ L_s^{1/2}}$$

The values of $\dfrac{1}{1.486\ R^{2/3}}$ can be obtained from Table 19 in this guide.

When the channel cross sections are subdivided into two or more parts, the equation h_f becomes

$$h_f = \left(\frac{Q_{s_1}}{K_{s_1}''}\right)^2 + \left(\frac{Q_{s_2}}{K_{s_2}''}\right)^2 + \ldots \left(\frac{Q_{s_n}}{K_{s_n}''}\right)^2$$

$$= \frac{Q^2}{\left(K_{s_1}'' + K_{s_2}'' + \ldots K_{s_n}''\right)^2}$$

The discharge for the various segments or parts can be obtained from the formula $K_s''\ h_f^{1/2} = Q_s$. The velocity of the segments can be obtained from $V_s = Q_s/A_s$. The weighted velocity head, h_v, is obtained from the formula

$$h_v = \frac{\left(V_{s_1}^2\ Q_{s_1} + V_{s_2}^2\ Q_{s_2} + \ldots V_{s_n}^2\ Q_{s_n}\right)}{Q \times 2g}$$

The other losses, in most cases the eddy loss, are $M\,(h_{v_1} - h_{v_2})$ where M is 0.10 for positive values of $h_{v_1} - h_{v_2}$ and 0.50 for negative values of $h_{v_1} - h_{v_2}$.

The assumption of the upstream water surface Z_2 is correct when the energy equation balances. For practical purposes, a balance to the nearest .1 foot is satisfactory.

The Corps' Method II is best solved by tabular computation. The form, step computation, and example are presented on pages 146 to 153. Note, there are two computations required for each section except the first and last sections. This is necessary because of the different reach lengths.

Method A--Bureau of Reclamation

This method covers the same conditions as described under Method I, Corps of Engineers. The flow path is assumed to be equal between sections for each sectional subdivision. Variations in the "n", A, and R values are appropriately analyzed.

When "n" values are different in the main channel and overbank areas, the cross section is subdivided. The areas and hydraulic radii are computed separately for each subdivision and tabulated as shown in Tables 6 to 12. The conveyance capacity, K_d, is also computed for the main channel and overbank using the corresponding "n" value. These conveyance curves are plotted as shown in Figures 13 to 19.

In this method the friction slope, S_f, is determined by the following analysis:

$$Q = \frac{1.486}{n} AR^{2/3} S_f^{1/2}$$

since $$K_d = \frac{1.486}{n} AR^{2/3}$$

then $$Q = K_d S_f^{1/2}$$

solving for S_f

$$S_f = \left(\frac{Q}{K_d} \right)^2$$

The K_d value used in the above equation is the total conveyance capacity of the main channel and overbank areas.

The discharges occurring in each subdivision of the cross section are determined as follows:

$$Q = K_d S_f^{1/2}$$

and $$Q_s = K_{d_s} S_f^{1/2}$$

where Q = total discharge, cfs

S_f = slope of the friction gradient

K_d = total conveyance capacity

and Q_s and K_{d_s} = corresponding elements for partial discharge and conveyance capacity of the subdivision under consideration

Dividing one equation by the other

$$\frac{Q}{Q_s} = \frac{K_d}{K_{d_s}}$$

solving for Q_s :

$$Q_s = \frac{Q \times K_{d_s}}{K_d}$$

From the above analysis, the K_d value, in addition to being defined as conveyance capacity, can also be designated as the discharge occurring with a slope equal to unity.

The friction head, h_f , is determined by averaging the slopes at Sections 1 and 2 and multiplying by the length as illustrated below:

$$h_f = L \left(\frac{S_{f_1} + S_{f_2}}{2} \right)$$

The velocity head, h_v , is derived by a weighting process using the partial discharges occurring in each subdivision of the cross section. Velocities in each segment are computed by the equation $V_s = Q_s/A_s$ where A_s is the area of the segment. The velocity head results from the following equation:

$$h_v = \frac{\Sigma \left(V_s^2 \, Q_s \right)}{2g \, Q}$$

Eddy losses are computed from the formula: eddy loss = $M \, (h_{v_1} - h_{v_2})$ where M is 0.50 for negative values of $h_{v_1} - h_{v_2}$ and 0.10 for positive values.

A detailed procedure of the application of this method is contained on pages 64 to 104.

Method B--Bureau of Reclamation

Method B is adapted to conditions where flow lengths between sections are different. Consideration is also given to the variability of the hydraulic elements "n", A, and R of the cross section. Main channel flow will have a greater travel length than the overbank flow because it is more sinuous.

The sketch in Figure 3 represents the energy balance equation which is basic to this method. Knowing the elevation at Section 1, the water surface elevation at Section 2 is assumed and the energy equation is checked.

The determination of the friction head, h_f , is of first importance in this method. This is accomplished by Manning's formula as follows:

$$Q = \frac{1.486}{n} AR^{2/3} S_f^{1/2}$$

expressing S_f as $\frac{h_f}{L}$ and

substituting

$$Q = \frac{1.486}{n} AR^{2/3} \left(\frac{h_f}{L} \right)^{1/2}$$

substituting K_d in the equation

$$Q = K_d \left(\frac{h_f}{L} \right)^{1/2}$$

squaring and solving for h_f

$$h_f = \left(Q/\Sigma \frac{K_d}{L^{1/2}} \right)^2$$

A mean friction head is eventually determined from the equation $h_f = \frac{h_{f_1} + h_{f_2}}{2}$ where h_{f_1} and h_{f_2} are the friction heads computed for Sections 1 and 2, respectively.

The discharges occurring in each sectional subdivision are computed from the formula listed below:

$$Q_s = \left(K_{d_s} / L^{1/2} \right) \left(h_f \right)^{1/2}$$

The velocity head, h_v, is computed by the weighting process where the partial discharges occurring in each subdivision are used. Each subdivisional velocity is calculated by the equation $V_s = Q_s / A_s$ where A_s is the area of the segment. The velocity head is computed from the equation

$$h_v = \frac{\Sigma \left(V_s^2 \, Q_s \right)}{2g \, Q}$$

Eddy losses are computed from the formula: eddy loss = $M \, (h_{v_1} - h_{v_2})$ where M is 0.50 for negative values of $h_{v_1} - h_{v_2}$ and 0.10 for positive values.

An application of this method is illustrated on pages 125 to 145 in more detail.

Profiles by Observation

The six theoretical methods of computing water surface profiles depend upon the judgment of the engineer in the field to assign proper "n" values and locate sections representative of the average conveyance parameters between sections. The accuracy of the profiles and the elevation-discharge relation, eventually

resolved by theoretical computations, is sometimes questioned by the hydraulic engineer, depending on his ability to secure adequate field data for defining representative hydraulic parameters of the channel.

Since preparation of the profiles by theoretical methods is expensive, if sufficient time is available, observed profiles can be determined that are beyond question at a reasonable cost. Staff gages at a common datum located along the channel can be read for various discharges. The discharge at given times can be secured from the records of a nearby gaging station.

Staff gages require observation at the time the discharge in question occurs, although this is not always possible or practical. The Geological Survey has developed a Crest Stage Gage** which can be substituted for staff gages, particularly for higher flows. A design drawing of this gage is shown in Figure 4. These gages may be secured to bridge abutments, trees, rock faces, and to concrete-embedded steel-column sections. They can be referenced to a common datum the same as a staff gage. The granulated cork mixture shown in the figure will leave an indication of the maximum stage on the inside measuring rod. After recording this elevation, the observer cleans the measuring rod and the gage can be set to record any subsequent crest stage. The observer does not have to be in the vicinity of the gage during the discharge occurrence. The gages can also be set near the main channel which is not accessible in flood periods.

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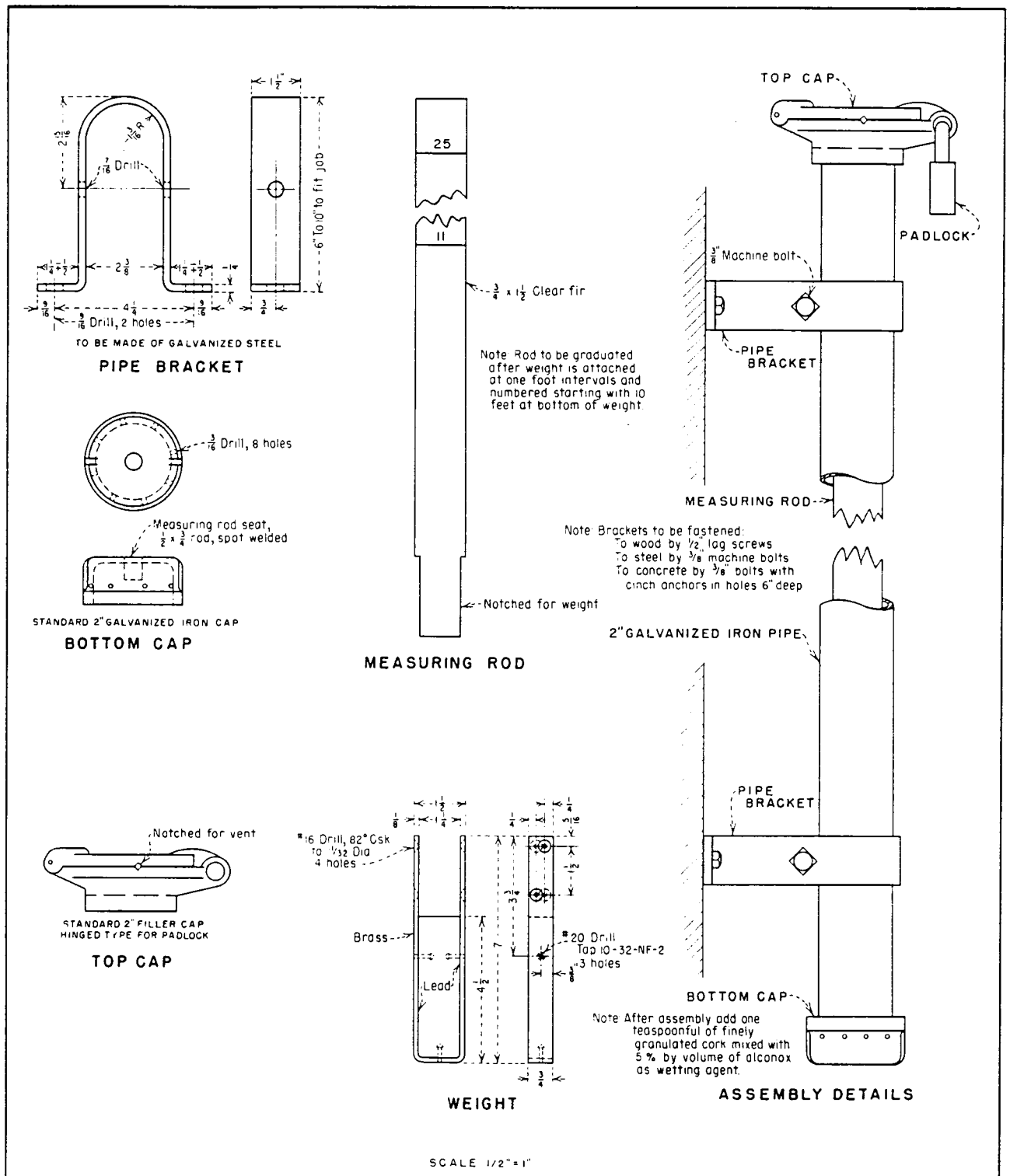
Figure 4


TO BE MADE OF GALVANIZED STEEL
PIPE BRACKET
7/16 Drill
9/16 Drill, 2 holes
6" To 10" to fit job
3/16 Drill, 8 holes
Measuring rod seat, 1/2 x 3/4 rod, spot welded
STANDARD 2" GALVANIZED IRON CAP
BOTTOM CAP
25
11
3/4 x 1 1/2 Clear fir
Note Rod to be graduated after weight is attached at one foot intervals and numbered starting with 10 feet at bottom of weight.
Notched for weight
MEASURING ROD
TOP CAP
PADLOCK
3/8" Machine bolt
PIPE BRACKET
MEASURING ROD
Note Brackets to be fastened:
To wood by 1/2" lag screws
To steel by 3/8" machine bolts
To concrete by 3/8" bolts with cinch anchors in holes 6" deep
2" GALVANIZED IRON PIPE
PIPE BRACKET
BOTTOM CAP
Note After assembly add one teaspoonful of finely granulated cork mixed with 5% by volume of alconox as wetting agent.
ASSEMBLY DETAILS
Notched for vent
STANDARD 2" FILLER CAP HINGED TYPE FOR PADLOCK
TOP CAP
#16 Drill, 82° Csk to 1/32 Dia 4 holes
Brass
Lead
#20 Drill Tap 10-32-NF-2 3 holes
WEIGHT
SCALE 1/2" = 1"
COLUMBUS DESIGN OF
CREST STAGE GAGE
AS USED TO DEFINE PEAK STAGES AT NEBRASKA HIGHWAY BRIDGES
U.S. GEOLOGICAL SURVEY LINCOLN, NEBR.
D.D. LEWIS, DISTRICT ENGINEER
PREPARED BY L.W. FURNESS JANUARY 12, 1953

The crest stage discharge and time of the event can be obtained from any nearby gaging station record, if available. If the crest stage discharge is not available, the maximum stage will furnish factual data for a slope-area method computation to determine the discharge.

It is possible to develop all the needed profiles and the tail water curve for a dam site by observation. However, it is rare that the flood events, during an observation period, are of sufficient magnitude to cover the maximum stage-discharge range required. If sufficient events have been observed to establish a partial stage-discharge relationship at the various gages, the relationship can often be extended satisfactorily.

One method for extending the stage-discharge curve involves the Chezy equation for discharge, $Q = C A \sqrt{RS}$. C is a coefficient of channel rugosity. If it is assumed that C and S are constant for the higher stages as for the defined portion of the stage-discharge relation, then Q is directly proportional to $A\sqrt{R}$. Cross sections obtained at the location of the gages will supply the $A\sqrt{R}$ information needed to extend the stage-discharge relationship. The engineer should determine in the field that the defined portion of the curve is representative of the C and S that may be expected for higher stages.

There are streams where the stage-discharge relation is not constant, therefore, computed profiles based on this relation would not be consistent or practical. Sand bedded alluvial streams like the Canadian, Rio Grande, and Platte Rivers do not have fixed

cross sections. Because of this variability, they do not lend themselves to a normal computation. These channels are wide with shallow depths that scour during floods so a rise in stage is not indicative of the total increase in discharge. The scour may not be of the same magnitude for each occurrence; hence, there is variation in the stage-discharge relationship. Regardless, the designer will require a tail water curve as a basis of design. The best practice is to install a staff gage at the site and then arrange with the Geological Survey to read the staff gage each time a discharge measurement is made at the nearest gaging station. In time, a mean stage-discharge curve can be developed which will also show the deviations from this mean curve. It may be necessary to extend this curve to higher stages, which can be done by simple logarithmic extension; the only method particularly adaptable to channels subject to extreme scour.

The establishment of staff gages is a wise investment by the project planner. The data collected by these gages can also be used in event the dam site is shifted up or downstream from the proposed site. The new location, however, should be hydraulically comparable to the site where the stage-discharge curve was developed.

Discussion of Methods

There are seven methods outlined in this guide for determining water surface profiles. The most superior of these methods, and usually the most economical if time and events permit, is the preparation of profiles or a tail water curve by observation. This method eliminates any questionable selection of "n" values and

uncertainty over whether a representative combination of sections has been used. In the case of alluvial sand bedded streams, it is the only reliable method of determining profiles or an elevation-discharge relationship.

One of the six theoretical methods should be selected for use on the basis of fit to the conditions and ease of application. All are trial and error methods. The Standard Step Method is recommended for a single uniform channel without the complication of overbank flow.

Leach's Method is recommended for use on streams having relatively uniform sections and slopes. This method will give improper results at any one section where the cross sections and slopes in a reach are variable because the water surface slope is used instead of the energy gradient. However, for long reaches of river, the resultant profiles will tend to follow those by other methods and may be satisfactory for the results required. It does provide a method for easy computation where there is an overbank condition having different conveyance parameters from the main channel. The flow travel distance should, however, be the same for main channel as the overbank portions.

The Corps of Engineers' Method I and Method A are recommended for the condition involving overbank and main channel flow with different conveyance parameters. The flow travel distance between sections should be essentially the same for the main channel as the overbank flow. Both methods use the friction slope in balancing the energy equation. Most engineers will find Method A less total work than the Corps' Method I.

The Corps of Engineers' Method II and Method B are recommended for the conditions involving overbank and main channel flow with different conveyance parameters and for variation in flow travel distance between sections for the main channel flow and the overbank flow. This condition becomes apparent when the main channel meanders in a flood plain. The overbank flow will travel a shorter distance by cutting across the meander. While both methods are based upon the same principle and give comparable results, Method B is recommended as much easier to use.

The key to the river hydraulic computations is a thorough understanding of the balance equation presented in Figure 3. It is recommended that this equation be studied before using any of the methods presented.

SPECIAL PROBLEMS

Stage-Discharge Relation for Initial Section

All of the discussions so far, except for the section on "Profiles by Observation," have assumed the elevation of the downstream water surface for a given discharge is known. This is not always true.

In the case of reservoir backwater computations, the reservoir water surface elevation is the starting elevation. Whether it be top of conservation pool, top of the flood control pool, the maximum water surface, etc., depends upon the purpose of the backwater profiles. The starting point of the backwater curve within the reservoir for any given pool elevation may be selected by

applying the criterion that the velocity head (h_v) is less than 0.05 foot through the cross-sectional area. The average velocity then should be 1.80 feet per second or less.

The starting elevation for computing the profiles of tail water elevations below dams is more arbitrary. In a few cases, the stage-discharge relationship for a stream gaging station located conveniently a short distance below the dam can be used. It may be necessary to convert the gaging station datum to that of the dam survey so the curve may be converted from a stage-discharge relation to a relationship of elevation versus discharge. It may also be necessary to extend the stage-discharge relationship for higher discharges, which can be done on the basis of the $A\sqrt{R}$ curve as explained in the section on "Profiles by Observations."

Where there is no stage-discharge relationship available, such a curve must be developed. Extreme accuracy of this relationship is not important if several section computations are made before progressing into the area where greater accuracy of the profiles is required. The computed profiles for a given discharge will converge after three or four sections even though the starting elevations vary by as much as 3 feet.

A common convenient practice in the development of a stage-discharge relationship is the application of a constant slope to the conveyance of the section. This slope may be that of an observed high- or low-water profile surveyed from high-water marks. Sometimes the slope of the streambed profile for a reach through the section is used. The development of the section conveyance is

similar to that used in the Leach Method wherein $K_d = \frac{1.486}{n} AR^{2/3}$ and $Q = K_d S^{1/2}$. An elevation versus K_d curve then needs only the application of $S^{1/2}$ to convert to an elevation-discharge curve. High-water marks with established discharges will provide check points for the curve.

Control sections with flow at critical depths make excellent starting sections, as the stage-discharge relationship is well established by the critical depth determination.

Critical Conditions

The following discussion concerns the analysis to be made when the river hydraulics produce critical flow conditions. These conditions occur where the channel slope steepens abruptly or at a natural constriction in the channel. The placement of structures raising the channel bottom or constricting channel widths can also cause critical flow.

The existence of critical conditions at a section is reflected in the computations when a balance cannot be reached in Bernoulli's equation. Before proceeding with the computations, it is necessary to determine the critical depth or elevation for the section.

There are a number of ways for checking the existence of critical or subcritical depth which occurs commensurate with critical or supercritical velocities. The basic equation for flow at critical depth is

$$\frac{Q^2}{g} = \frac{A^3}{T}$$

where

Q = discharge in cfs

g = acceleration of gravity

A = area

T = water surface width

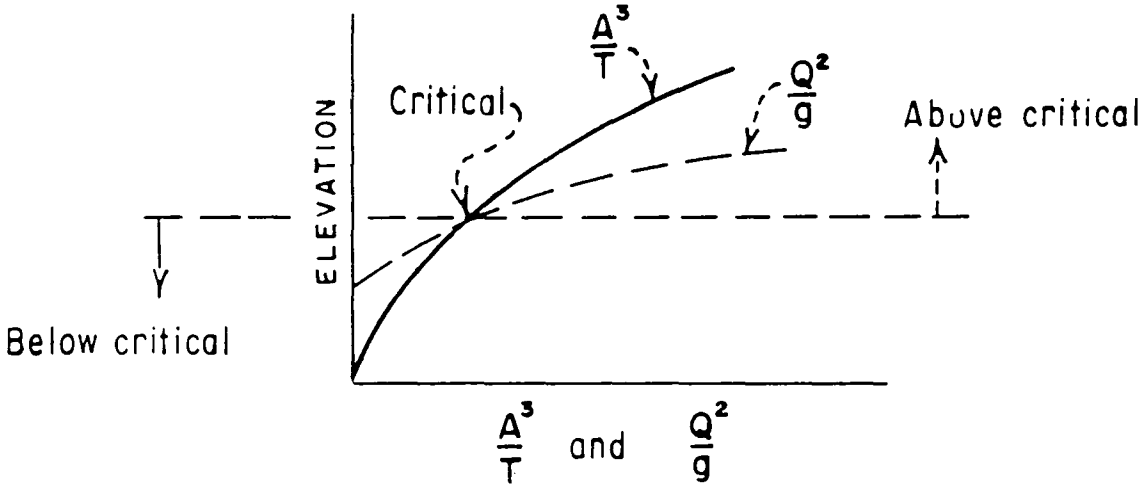
This equation must be satisfied, no matter what the shape of the channel, when flow is at critical depth.

A preliminary check can be made of critical conditions by determining the critical velocity from the following equation which results from an algebraic transformation of the above equation, letting $Q = VA$:

$$V_{cr} = 5.67 \left(\frac{A}{T} \right)^{1/2}$$

Thus, when the computed velocity exceeds V_{cr}, critical velocity, it can be assumed that critical flow exists.

In cases where the flow is suspected to approach critical throughout the range of computed discharges, it is recommended that a set of curves be plotted as shown below:



The graph portrays a condition where the flow is at subcritical depth for low stages and above critical in the higher stages. The reverse, of course, can also occur.

An alternative to the above is the plotting of a critical discharge curve which can be derived as follows:

From the equation

$$\frac{Q^2}{g} = \frac{A^3}{T}$$

solve for Q

$$Q^2 = \frac{A^3 g}{T}$$

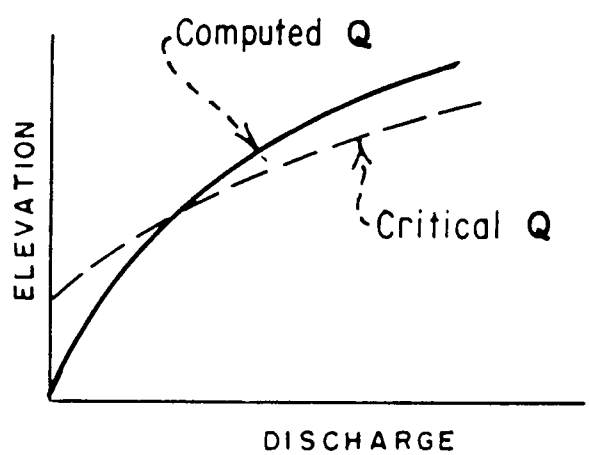
and

$$Q_{cr} = \left(\frac{A^3 g}{T} \right)^{1/2}$$

simplifying

$$Q_{cr} = 5.67\ A \left(\frac{A}{T} \right)^{1/2}$$

The curve--elevation against critical and computed discharge--would be plotted as illustrated below:



A check can be made of the critical discharge by entering the curve at the water surface elevation determined for the discharge computed. If the critical discharge at this elevation exceeds the computed discharge, the flow is at a depth above critical depth. When the reverse occurs, there will be a subcritical depth condition. Refering to the graph above, the true rating curve would be established by using the critical Q curve from the point of intersection down and computed Q curve upward from that point.

Other means can also be used in determining critical flow. However, they will not be discussed, since all are based on the various ways of manipulating the equation $Q^2/g = A^3/T$ with the data at hand. The important point of the analysis is that the critical depth elevation is used to proceed with computations when the critical discharge is less than the computed discharge. An example of a critical depth analysis will be further illustrated in the Appendix.

Bridge Openings

Bridge crossings are of special interest in the computations of water surface profiles. Establishing cross sections at the bridge and at 50 and 100 feet upstream and downstream from the bridge greatly facilitates the computations and more realistic results are obtained. If cross sections have not been established, however, the hydraulic properties of the adjacent upstream and downstream sections are assumed to apply. The Appendix will also include an example of a bridge crossing problem.

The bridge becomes a submerged obstruction at higher flood flows. Backwater elevations can be determined by computing flows by

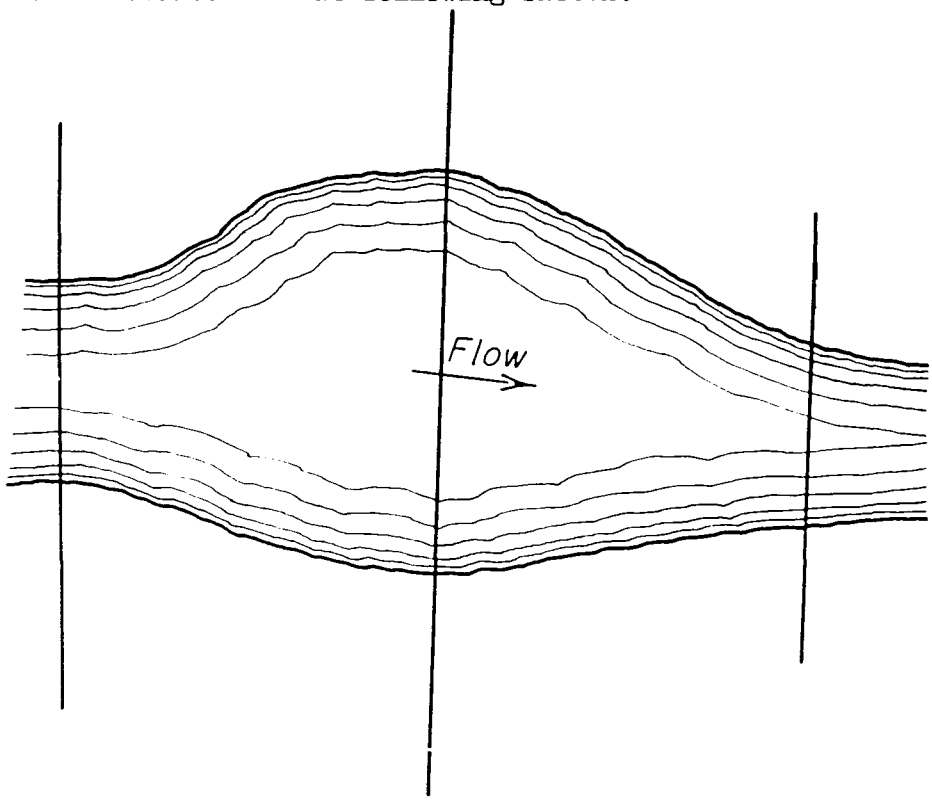
the orifice formula, $Q = CA\sqrt{2gH}$ and the broad-crested weir equation, $Q = C\ell H^{3/2}$, through the submerged bridge opening and over the roadway and bridge, respectively. Yarnell and Nagler [14] conducted model studies on the flow of flood waters over railway and highway embankments which are applicable to situations such as this. Another source dealing with broad-crested weir coefficients is the Geological Survey Water Supply Paper 200 [15]. Hollister Johnson [13] developed a method which may be used for computing the head loss through a bridge. The Corps of Engineers' Manual [4], previously referred to, also contains an approximate method for determining backwater curves through bridge sections. The engineer must make a thorough investigation of the hydraulics involved at the bridge crossing and then select the method, based on judgment, which will give the best practical solution to the problem.

FIELD DATA

Field Observations

All the methods discussed in this guide, except the method of developing profiles by observation, require determinations of the hydraulic parameters of "n", A, R, and L, as well as a general analysis of the situation for the most practical approach. Securing the field information can be very costly. It is advisable that an early reconnaissance be made of the area by a hydraulic engineer to determine the extent of the necessary data. When the need for data has been established, the cost of the field surveys can often be reduced by integrating them with securing reservoir and dam site topography, drilling, etc.

An initial step involves locating the sections for survey. Aerial photographs and topographic maps will aid in the location of sections to be surveyed. Sections should be located so the average area, hydraulic radius, and "n" values of the two sections will be representative of the reach length between. Channels that are alternately wide and narrow should have sections located in the narrow and wide reaches so the average condition can be defined. This is illustrated in the following sketch:



Sections should also be located to reflect the influence of grade changes and section controls such as bridge crossings.

The total channel length needed to define adequately natural flow profiles and the desirable distance between cross sections will vary with conditions. In general, the distance between cross sections can be longer for flat slope streams which

have gradients usually in a range less than 5 feet per mile. Two or three thousand feet between sections is common for the flat slope streams, and between 300 and 1,000 feet for the steep slope streams. Total channel lengths will vary similarly--three or four thousand feet for the steep slope (> 5 ft/mi) streams, and up to 4 or 5 miles for the flat slope streams. At times, it may be desirable to deviate from these distances to be able to tie into stream gaging stations and when there are alternate possibilities for locations of the dam sites. Other considerations include the size of the river and the discharges being investigated. Distances, therefore, can be greater than those quoted above for large rivers and higher discharges. Sections for reservoir backwater computations should extend far enough upstream to reflect the end of the reservoir backwater influence; that is, where the natural depth elevation is within a tenth foot of the backwater elevation. At this point $d_B = d_o + 0.1$ where d_B equals the backwater depth and d_o is the natural depth.

When the criteria for the section survey are set, the hydraulic engineer should first determine the maximum discharge for which the profiles will have to be computed. This will help in establishing the detail of the survey and give some idea of the elevation to which the section surveys should be carried. Also, as a general rule, the main channel has the largest conveyance within a section; therefore, definition of this segment should be good. It is often necessary to take soundings of the channel in the deeper flowing portions of the channel. A topographic map rarely will define

a section adequately for realistic profile computations, particularly in the main channel portion; hence, it is recommended that field surveys of the sections be made. It is good practice for the field survey notes to record the section stationing at the points of changing "n" values. If the hydraulic engineer will go over the sections in the field with the survey chief, the sections can be flagged for changing "n" values and the estimated "n" values given the survey chief for inclusion in the notes. When the sections are surveyed, maximum high-water marks should be recorded as well as the time and elevation of main channel flow. An example of this procedure is shown in Table 1. This information can be correlated later with stream gaging data to help in establishing "n" values. All sections should be located transversely to the bulk of the flow at the section. The main channel flow direction and the direction of drift lodged in the overbank areas will help in this location.

The assignment of "n" values is generally a matter of experience and judgment. The "n" value should receive careful consideration since it influences the computations by the first power as contrasted with the two-thirds power of the hydraulic radius and the half power of the slope in Manning's formula. When possible, high-water marks and the main channel water surface for known discharges should be obtained in the survey to enable determination of the "n". When these data are not available, then the alternative is assignment of the "n" values. This assignment should be made in the field by the hydraulic engineer. Trees, bushes, boulders, etc., cause "n" values to differ transversely

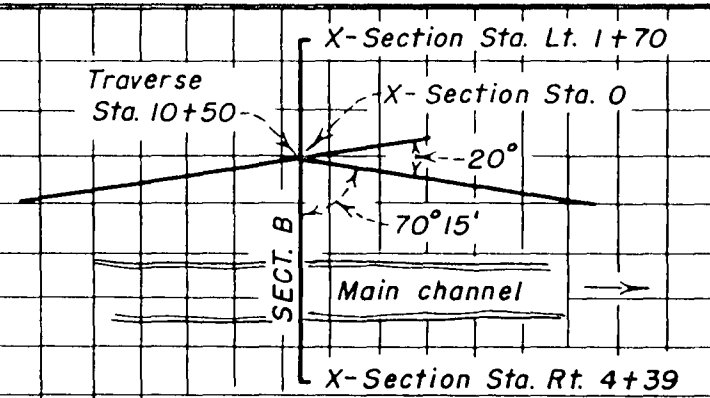
Table 1

DATE Sept. 15 19 5-

NAME OF LINE X-Section B. Tailwater Survey, Denver Dam

PARTY A. Dooks, Smith, Jones, Miller

STA.	BACK SIGHT	H. I.	FORE SIGHT T. P.	FORE SIGHT INTERMED	ELEVATION
BM # 3 STA. 10+50	4.62	5264.85			5260.23
1+00 Lt.				2.5	5262.3
TP # 10	9.63	5273.56	.92		5263.93
1+70 Lt.				1.5	5272.1
TP # 11	4.10	5267.84	9.82		5263.74
BM # 3	1.26	5261.49	7.61		5260.23
1+00 Rt.				3.6	5257.9
1+15 Rt.				8.9	5252.6
TP # 12	2.12	5251.50	12.11		5249.38
1+20 Rt.				5.3	5246.2
1+25 Rt.				8.2	5243.3
1+50 Rt.				9.1	5242.4
1+75 Rt.				10.2	5241.3
1+80 Rt.				9.3	5242.2
1+95 Rt.				3.6	5247.9
TP # 13	11.15	5261.33	1.32		5250.18
2+50 Rt.				9.5	5251.8
3+00 Rt.				1.2	5260.1
TP # 14	12.14	5272.35	1.12		5260.21
4+00 Rt.				8.5	5263.8
4+39 Rt.				1.1	5271.2
TP # 15	1.79	5262.22	11.92		5260.43
BM # 3			2.00		5260.22



Elev. BM # 3 from Level Book No. 46

Sta. 1+70 to 1+00 Left $n = .080$
large trees, light undergrowth

Sta. 1+00 Left to 1+00 Rt. $n = .040$
grassed open pasture

Main channel water surface at
Sta. 1+25 Rt. - Recorded 2:30 p.m.

Sta. 1+00 to 2+50 Rt. Main channel
$n = .035$. Earth banks, light sand bed

Sta. 2+50 Rt. to 4+39 Rt. $n = .10$
large trees, heavy undergrowth

along the cross section. It is possible to assign an average "n" for a section, but this is extremely difficult for all stages considering the changing relationship of discharge between the various segments of the section. It is easier to assign the "n" values by segments of the total section. Main channel sections may have an increasing "n" value for higher stages due to overhanging vegetation. Roughness in the main channel caused by irregularities such as sand bars, gravel, cobbles, and boulders will cause higher "n" values for low flow with corresponding shallow depth than will be found for higher discharges and greater depths. There are several guides which can be used in the assignment of "n" values. Among these are: Table 76 of King's Handbook [9], fourth edition; a set of photographs and description of channels showing various roughness factors which can be studied in the book "Low Dams" [16], pages 227 to 233; and Tables 2 and 3 included in this guide.

The cross-section surveys and assignment of "n" values define the parameters of "n", A, and R, leaving the L or length parameter to be determined. The sections normally are tied into a traverse so a plan map can be developed of the section layout. The main channel and overbank areas can be developed at each section, but this does not necessarily determine the meander length of the channel nor the path of the overbank flow.

There are three methods of obtaining these field data. If topographic maps or even reliable plan maps are available, the section plan may be plotted on these maps and channel lengths measured. A channel plan map may be developed from aerial photographs upon which

Table 2

A METHOD OF COMPUTING MEAN "n" VALUE FOR A CHANNEL

(Used by Soil Conservation Service)

Steps

1. Assume basic "n"
2. Select modifying "n" for roughness or degree of irregularity
3. Select modifying "n" for variation in size and shape of cross section
4. Select modifying "n" for obstructions such as debris deposits, stumps, exposed roots, and fallen logs
5. Select modifying "n" for vegetation
6. Select modifying "n" for meandering
7. Add Items 1 through 6

Aids in Selecting Various "n" Values

1. Recommended basic "n" values

Channels in earth	- 0.010	Channels in fine gravel	- 0.014
Channels in rock	- 0.015	Channels in coarse gravel	- 0.028

2. Recommended modifying "n" value for degree of irregularity

Smooth	- 0.000	Moderate	- 0.010
Minor	- 0.005	Severe	- 0.020

3. Recommended modifying "n" value for changes in size and shape of cross section

Gradual	- 0.000	Frequent	- 0.010 to 0.015
Occasional	- 0.005		

4. Recommended modifying "n" value for obstructions such as debris, roots, etc.

Negligible effect	- 0.000	Appreciable effect	- 0.030
Minor effect	- 0.010	Severe effect	- 0.060

5. Recommended modifying "n" values for vegetation

Low effect	- 0.005 to 0.010	High effect	- 0.025 to 0.050
Medium effect	- 0.010 to 0.025	Very high effect	- 0.050 to 0.100

6. Recommended modifying "n" value for channel meander

L_s = Straight length of reach L_m = Meander length of reach

L_m/L_s	"n"
1.0 - 1.2	0.000
1.2 - 1.5	0.15 times n_s
> 1.5	0.30 times n_s

where n_s = Items 1 + 2 + 3 + 4 + 5

Table 3

COEFFICIENT OF ROUGHNESS, AVERAGE CHANNELS

Value of "n"	Channel Condition
0.016-0.017	Smoothest natural earth channels, free from growth, with straight alinement.
0.020	Smooth natural earth channels, free from growth, little curvature.
0.0225	Average, well-constructed, moderate-sized earth channels in good condition.
0.025	Small earth channels in good condition, or large earth channels with some growth on banks or scattered cobbles in bed.
0.030	Earth channels with considerable growth. Natural streams with good alinement, fairly constant section. Large floodway channels, well maintained.
0.035	Earth channels considerably covered with small growth. Cleared but not continuously maintained floodways.
0.040-0.050	Mountain streams in clean loose cobbles. Rivers with variable section and some vegetation growing in banks. Earth channels with thick aquatic growths.
0.060-0.075	Rivers with fairly straight alinement and cross section, badly obstructed by small trees, very little underbrush or aquatic growth.
0.100	Rivers with irregular alinement and cross section, moderately obstructed by small trees and underbrush. Rivers with fairly regular alinement and cross section, heavily obstructed by small trees and underbrush.
0.125	Rivers with irregular alinement and cross section, covered with growth of virgin timber and occasional dense patches of bushes and small trees, some logs and dead fallen trees.
0.150-0.200	Rivers with very irregular alinement and cross section, many roots, trees, bushes, large logs, and other drift on bottom, trees continually falling into channel due to bank caving.

the cross-section plan may be plotted and, thus, the flow lengths determined. If aerial photographs and reliable maps are not available, then shots must be taken in the course of the traverse survey to produce the extent of the overbank areas and the main channel location from which a plan map may be developed. Unless the traverse completely represents the channel alinement, this length should not be used in the determination of L.

Photographs representative of the typical section conditions are valuable information to the computer, especially if "n" value adjustments are necessary. One or two water surface profiles through the reach are valuable aids in determination of "n" values and will certainly add to the accuracy of the profiles.

When the elevation-discharge relation is required for power installations, the low flow portion of the curve needs to be defined. This is easily accomplished by taking a discharge measurement at low flow and recording the elevation of the water surface at the time of measurement. It is sometimes possible to record only the time and elevation of a low flow and determine the discharge from a nearby gaging station record.

Processing the Field Data

The survey notes, after the routine reduction, will be the basis of developing a plan map, cross-section plots, streambed profile, section A and R curves, and a table of reach lengths.

The plan map, as discussed in the field data section, can be developed by one of the three following ways: (1) overlaying the traverse and section plot on a plan map or topographic map, (2) overlaying the traverse and section plot on a plan map developed from

aerial photographs, and (3) development of the plan completely from the survey notes. The first two methods are ways of reducing the cost of the field work; however, the third method will require more detailed field data.

The cross sections should be plotted next. The scales chosen should be such that a realistic concept of the section can be obtained. Information from the field notes regarding "n" values, low-water elevations, high-water marks, and description should be included on the plots. An example of such a plot is shown in Figure 7.

The area and hydraulic radius computations will be made from these plots; hence, the scale of the plots should be chosen relative to the methods used for computation. A and R computations are commonly made by planimetering the area and measuring the wetted perimeter, or by computing both parameters. One computational procedure, called the double area method, can be used to compute the area and determine the wetted perimeter by the Pythagorean Theorem (square root of the sum of the squares). Scales for planimetering should be selected for ease in planimeter operation. If the wetted perimeter is determined by measurement, the samescale must be used for both the abscissa and ordinate. Determination of A and R by computation requires no specific plotting other than that for a reasonable concept of the section.

The sectional parameters of area and hydraulic radius for use in the computations are best handled by drawing curves of the area and hydraulic radius versus elevation. A few determinations

of the area and R for different elevations will enable the drawing of a reasonably defined curve. An example is shown in Figure 21. The computation of area and hydraulic radius by planimetering and measurement from the plotted sections is a simple operation. However, there are cases where the vertical and horizontal distances cannot be reduced to a common scale. In such cases, computations of the area and the wetted perimeter may be easier and also afford a positive check. An example of the computation using the double area method is presented in Table 4.

The above described processing of the data will enable a straightforward computation of profiles by the Standard Step procedure, and the Corps of Engineers' Methods I and II. If Leach's Method and Bureau of Reclamation Methods A and B are used, then K_d curves must be made. An example of the computation and the curves are presented in Tables 6 to 12 and Figures 13 to 19.

From the plan map, the distance L between sections can be determined. This distance for a large channel is merely the channel length between sections. If there are large overbank flows with significant differences in the flow lengths between the overbank and main channel, then both these lengths should be determined for use in the Corps' Method II and Bureau of Reclamation Method B type of computation. It is suggested that a table be set up listing the sections and the distances between sections for easy reference in making the computations as shown in Table 5.

A profile should be plotted next. The horizontal stationing is determined from the reach length between sections and the

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COMPUTATION SHEET

SHEET ____ OF ____

SUBJECT: Area and R--Cross Section 2

PROJECT: Riesbol's Run

COMPUTED BY: Rube — DATE

CHECKED BY: Joe — DATE

SOURCE OF DATA: Cross Section 2, Drawing No. 9-476

1	2	3	4	5	6	7	8	9
Sta	Elev	Depth	Width	Sum Depth	Double Area	Δ Depth	Sum Squares	P
			Water Surface Elevation 1490					
160	1490.0	0						
155	1482.9	7.1	5	7.1	35.5	7.1	75.41	8.7
148	1477.4	12.6	7	19.7	137.9	5.5	79.25	8.9
140	1475.0	15.0	8	27.6	220.8	2.4	69.76	8.4
130	1474.0	16.0	10	31.0	310.0	1.0	101.00	10.0
115	1472.9	17.1	15	33.1	496.5	1.1	226.21	15.0
100	1474.7	15.3	15	32.4	486.0	1.8	228.24	15.1
95	1480.3	9.7	5	25.0	125.0	5.6	56.36	7.5
90	1482.6	7.4	5	17.1	85.5	2.3	30.29	5.5
87	1487.2	2.8	3	10.2	30.6	4.6	30.16	5.5
80	1490	0	7	2.8	19.6	2.8	56.86	7.5
	Total		80		1946.9			92.1
				Area	973.4			

$$R = \frac{973.4}{92.1} = 10.57$$

Water Surface Elevation is assumed section elevation for which A and R are to be computed.

Col. 1 = Cross section stationing
Col. 2 = Ground elevation at station
Col. 3 = Water surface elevation minus ground elevation
Col. 2 - Water depth at station
Col. 4 Width between stations
Col. 5 Sum of depths between stations
Col. 6 Col. 5 multiplied by Col. 4
Col. 7 Difference in depth between stations
Col. 8 Col. 7 squared plus Col. 4 squared
Col. 9 Square root of Col. 8
Area = Sum of Col. 6 divided by 2
R = Area divided by sum of Col. 9

streambed elevation taken as the lowest point in the cross section from the cross-section plots. All observed water surface and high-water mark elevations should be included on this plot. The scales for this plot should be extended to enable a later plot of the computed profiles. This profile should be considered a working draft; hence, it is recommended that the scales be large enough to obtain a reasonable concept of the profile characteristics. A final reduced profile for a finished report can be made later from this working draft.

COMPILATION OF REPORT

Compilation of Study and Report

A well-written report should be prepared and submitted with each study made. The method and computational procedure should be sufficiently described in detail to facilitate reviews of the study. This will also serve as a guide to the preparation of future studies.

The following items are recommended for inclusion in each study. Field data for use in an example of Method A have been gathered at a fictitious stream, Red Fox River, Colorado. Examples are also included showing the various charts, tables, maps, graphs, and curves prepared from the basic data for use in water surface profile computations.

1. A topographic map (if available) showing traverse and layout of the cross sections should be included as shown in Figure 5. Areal distribution should be shown of vegetation such as brush, trees, and grass, and type of land use--cultivated,

grazing, or barren, for example. For backwater studies, this map will be very helpful in determining the extent of inundated areas for the specific discharge computed. Data from the profiles in Figure 43 can be used in preparing this map by extending the range lines until they cross the ground contour equal to the water surface elevation for the range.

2. Photographs--aerial (if available) and land should be included. Notes are necessary in describing unusual hydraulic conditions in the reach.

3. High-water marks for any observed discharge should be plotted and the resulting profile drawn in.

4. When a gaging station is not located within or near the reach under study, a rating curve for the lowermost section will need to be established. In the example, a gaging station was not located nearby, therefore, a rating curve for Section 1 was developed as shown in Figure 28, using the data from Table 13 and Figure 27. Table 13 shows the computations of the rating curve covering the range of discharges to 11,100 cfs. A mean bed slope (Figure 27) was used for all stages in applying Manning's formula.

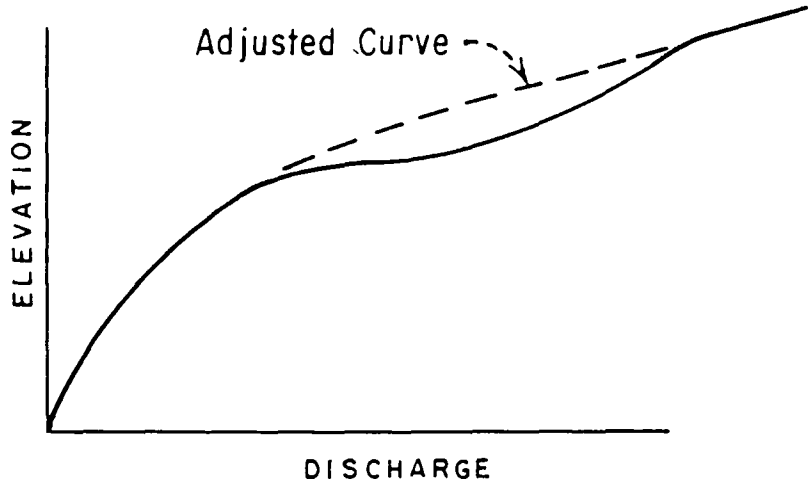
5. A plotting of the cross sections should be furnished along with a tabulation of the pertinent hydraulic properties. Examples of these are contained in Figures 6 to 12 and Tables 6 to 12, respectively. Figures 13 to 19 and 20 to 26 show the K_d and area curves needed for each section.

6. Distances between sections should be tabulated as shown in Table 5.

7. Computations should be made and included for at least five discharges within the desired range, such as shown in Tables 14 to 16.

8. A graph is needed showing the water surface profiles for each computed discharge. The bed profile should also be included. See Figure 29.

9. The rating curve of the desired tail water section should be included (Figure 30). Rating curves plotted for each cross section from the computed water surface profiles should be drawn although they can be omitted from the study. This procedure is recommended for the purpose of checking the analysis made of the hydraulics at each section. Thus, a rating curve with an unusual pattern, such as becoming concave upward, would indicate some inconsistency at the section which required further analysis. An adjustment of the curve would be necessary as indicated in the following sketch:



When such a rating curve occurs at an intervening section in the reach, the elevations must be adjusted for discharges in the affected range shown as a dotted line in the sketch. The backwater computations are then continued from these newly established elevations.

Rating curves developed for sand bottom sections by these computations are very unrealistic because of the shifts in channel bed. Hydraulic properties of the section are only temporary because of these shifts. The practical limits of accuracy of these curves should be defined by the hydraulic engineer.

Examples of the various other methods are also included using the data gathered at two fictitious streams, Red Fox and Silver Fox Rivers. An example of a backwater computation is also presented for the Alpine Reservoir.

The following Appendix contains the information necessary for preparing reports of reconnaissance and feasibility grades. It also includes examples of the various methods recommended for use in computing water surface profiles. A step procedure and corresponding form has been provided to illustrate the computational process of each method. Examples are also shown of a backwater computation, check of the head loss through a bridge, and a check for critical conditions.

GLOSSARY OF TERMS

A = cross-sectional area (in square feet)

C = empirical coefficient in various discharge formulas

cr = subscript denoting values of parameters at critical flow

d = depth of flow (in feet)

d_B = backwater depth (in feet)

d_o = natural depth (in feet)

D_1 = initial depth of flow upstream from a hydraulic jump (in feet)

D_2 = conjugate depth of flow downstream from a hydraulic jump (in feet)

g = acceleration of gravity equal to 32.2 feet per second per second

H = total head (in feet)

ΔH = change in total head (in feet)

h_b = bend loss (in feet)

h_f = friction head (in feet)

h_v = velocity head $V^2/2g$ (in feet)

Δh_v = change in velocity head (in feet)

K'' = symbol of the relation $\frac{1.486\ AR^{2/3}}{nL^{1/2}}$ or $\frac{A}{n\ (YL)^{1/2}}$

K_d = section conveyance equal to $\frac{1.486}{n} AR^{2/3}$

L = reach length between sections (in feet)

ℓ = length of weir (in feet)

M = estimated percentage of velocity head change between sections due to eddy loss

n = coefficient of channel roughness (Manning)

P = wetted perimeter (in feet)

Q = discharge (in cubic feet per second)

R = hydraulic radius which is equal to $\frac{A}{P}$ (in feet)

S = slope of the energy gradient

s = subscript denoting values of parameters for a segment or partial area

S_f = frictional slope

S_w = slope of the water surface

T = width of the water surface (in feet)

V = average velocity of flow (in feet per second)

Y = symbol of the relation $\frac{1}{1.486\ R^{2/3}}$

Z = minimum bed elevation (in feet)

Δ = angle of bend (in degrees)

Σ = symbol for summation

1 and 2 = subscripts denoting values of parameters at a downstream section (Section 1) and upstream section (Section 2)

.01 = subscript denoting values of parameters at a section assuming a slope of .0001

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APPENDIX

Reconnaissance and Feasibility Grade Studies

The information presented in the forepart of this guide pertains to the development of water surface profiles for use in final designs and specifications. The following discussions relate to the field work and computational procedures recommended for use in preparing reconnaissance and feasibility reports or in other cases where a high degree of accuracy is not required. In the following paragraphs it should be borne in mind that the term "reconnaissance and feasibility grade" refers to the required accuracy of the water surface profile and not necessarily to the type of report in which it is to be used. In outlining the required field work and selecting the method for computation, the engineer should keep in mind three factors; the use for which the water surface profile is intended, the effect of accuracy on engineering or economic feasibility, and the effect of physical condition of the stream and collected data on the accuracy of the computations. In balancing these three factors the engineer must use previous experience and exercise good judgment.

Reconnaissance

Studies of reconnaissance grade require the least amount of preparation for water surface profiles. Preliminary field observations and plans for office computational procedures can be geared to studies of feasibility grade if time permits.

When the location of the dam site has been established, it is recommended that a staff gage be installed at the site proper. Records gathered by use of this gage will define the lower range

of the tail water rating curve. Keeping in mind a feasibility study, it is suggested that a series of staff gages be also installed along the channel. These gages would supply valuable information on water surface profiles, "n" coefficients, slopes, and discharges. For large rivers where very high discharges may occur, a system of Crest-Stage gages can be installed to trace the high-water profiles. Additional information on these gages is available under "Profiles by Observation" in the forepart of this guide.

An alternate or supplemental procedure to the staff gage installation is the development of the tail water rating curve by Manning's formula. In this procedure, soundings are taken of the dam site cross section, either the water surface or flow line slope is measured, and the "n" value is estimated. Each of these field measurements furnish respectively the area-hydraulic radius, slope, and "n" values needed in Manning's formula. This procedure is discussed in detail under the section, "Stage-Discharge Relation for Initial Section," which includes the methods used for extending rating curves.

When a topographic map is available of the area under study, a tentative layout of the cross sections can be indicated on the map. Rough determinations of the cross-sectional areas can be made from the map depending on the contour interval. Flow distances between sections can be scaled from this map. The hydraulic information furnished by the map is applicable not only to tail water studies involving the reach below the dam, but also to the reach upstream from the dam when a backwater curve must be determined. The data compiled from the map are usually of reconnaissance grade.

Again, keeping a feasibility study in mind, thorough field observations of the downstream reach are very helpful in planning future surveys. At this stage, a tentative layout of the cross sections can be made using the previously discussed plan on the topographic map as a guide. Soundings may not necessarily be taken of the cross sections but their range ends pegged. From the observations, "n" coefficients may be tentatively studied and possibly assigned. It should be remembered that if flows are expected to encroach upon the overbank areas, additional hydraulic conditions are imposed on the problem. In those cases, a single "n" value may be estimated for the entire cross section for reconnaissance; but, it is often revised for feasibility studies, and each overbank is analyzed separately on the basis of its own "n" coefficient and hydraulic elements. Abrupt changes in the channel width or bed slopes can also be observed. It would be well to take a series of photographs of the various locations along the reach and to prepare a set of descriptive notes recording all the pertinent information for future reference. The preceding information applies also to the field investigations which are involved in the development of a backwater profile of the reach above the dam. This reach may extend for several miles upstream, depending on the natural channel hydraulics and post-dam hydraulic conditions.

In some cases, a gaging station may be located near the dam site axis. It is not recommended, however, that the rating curve of this station be longitudinally transferred to the dam site cross section, especially for relatively far distances. This practice is

discouraged because of the changing hydraulic conditions involving nonuniform flow between the gaging station and the dam site cross section. The transfer of this curve would be acceptable under the conditions of a relatively uniform reach which rarely occurs in natural rivers or streams for any considerable distance. The gaging station, however, furnishes the discharge information that is needed in the analysis of the upstream hydraulics.

Office computations of the tail water rating curve for reconnaissance purpose are generally restricted to a straight Manning calculation unless more detailed field data have been gathered to compute the profiles by one of the methods outlined in the first part of this guide. Backwater profile office computations will require a set of cross-sectional data which are determined from the topographic map as mentioned previously or actually measured. In these preliminary computations, changes in velocity head and eddy loss corrections can be ignored; therefore, Leach's Method, described in the forepart of this guide, can be applied. The accuracy of these computations, both for tail water and backwater, will lie in the range of ± 2 feet at the higher stages for small streams, and ± 4 feet at upper stages for large streams.

Feasibility

Feasibility grade studies should be prepared such that only minor revisions are needed to reach the status of final designs. It is important then to begin with a good reconnaissance-type study and continue to improve it by necessary revisions and modifications based on the collection of more substantial data, such as the addition of more cross sections.

How much work is involved depends on the extent of the field observations which were carried out initially. Each hydraulic factor must be defined and analyzed more critically. The cross-section layout should be more firmly established. Relocation of some sections or the addition of others may be necessary, depending on any changes in the hydraulic conditions observed afterwards. Soundings of these cross sections should be commenced as soon as practicable. The values of "n" originally assigned may need to be revised, based on observed water surfaces. Consideration should be given to the assignment of "n" values to the overbank areas. It would be well to establish these values for final designs, if possible, where high flows prevail. Keeping a running account of all discharges and measurements of the corresponding hydraulic properties at each cross section could be made when a gaging station is reasonably near the affected area. Flow line distances between cross sections now can be measured in the field. A good development of the rating curve for the beginning section could be made for feasibility and can also be used in a final design study.

Office computations are then ready to begin. Method A, described in the first part of this manual, is usually the best computational procedure to use in developing the water surface profiles. Changes in velocity head and eddy loss corrections now should be considered in the computations. At least four or five profiles for various discharges should be computed for tail water studies. The backwater curve can be traced for the highest discharge, beginning at the maximum water surface operating level of the reservoir. Backwater curves for other discharges and different reservoir water surface

levels can also be computed without any trouble. Accuracy of these curves will approach that which is required for final designs.

A study of the following sections in the first part of this guide can be made to prepare a report for feasibility grade:

1. Method A--Bureau of Reclamation
2. Profiles by Observation
3. Stage-Discharge Relation for Initial Section
4. Field Observations
5. Processing the Field Data
6. Compilation of Study and Report

It should be remembered that preparing a good feasibility grade report can lead easily into an adequate study for final designs.

Method A

Method A requires a computation of various tables and graphs before the computations are begun. The example is of a fictitious stream, Red Fox River, Colorado.

Of first importance is the preparation of Plan Map as shown in Figure 5. In this example, it was found that a topographic map existed of the area. The cross sections 1 to 7 are indicated on the map. Cross Section 7 coincides with the proposed dam axis. A bridge crossing is noted at Section 4. Sections 3 and 5 have been located 100 feet up and downstream from the bridge section. The distances between sections shown in Table 5 can be measured with sufficient practical accuracy from this map.

Cross-sectional plots in Figures 6 to 12 are taken from the field notes. It is noted that the elevation and stationing are shown in the plots.

The conveyance computations are next performed as shown in Tables 6 to 12 using the data of the graphs in Figures 6 to 12. K_d curves are prepared from these tables as shown in Figures 13 to 19. It is noted that separate K_d curves are prepared for the main channel and overbank areas.

Area curves shown in Figures 20 to 26 have been prepared also using the data of Tables 6 to 12. The hydraulic radii curves, although not used by this method, are included as a convenience for use by the other methods which are subsequently described.

Since there are no records of observed profiles for this channel, a rating curve must be developed for Section 1, the lowermost downstream section. An integral part of the computations for this rating curve is the slope. In Figure 27, the natural bed profile has been plotted along with a mean slope which is drawn in by eye. In the example, this is equal to 0.00735. Computations for the rating curve at Section 1 are shown in Table 13. Manning's equation is the basis where $Q = K_d S_f^{1/2}$. The curve is plotted in Figure 28.

The water surface profiles are computed in Tables 14 to 16 for discharges of 100, 500, 1,000, 5,000, and 11,100 cfs. These profiles are drawn as shown in Figure 29. The tail water rating curve of Section 7 in Figure 30 is the end product of the study.

Table 16 is the form recommended for use in computing water surface profiles by this method. The method shows a balance of energy by use of Bernoulli's Theorem as follows:

Rewriting as before

$$Z_2 + d_2 + h_{v_2} = Z_1 + d_1 + h_{v_1} + h_f + \text{other losses}$$

(See Figure 3)

transposing h_{v_2}, there results

$$Z_2 + d_2 = Z_1 + d_1 + (h_{v_1} - h_{v_2}) + h_f + \text{other losses}$$

Referring to the form (Table 16), the columns corresponding to the elements of the equation are as follows:

Element	Column
$Z_2 + d_2$	2
$Z_1 + d_1$	17
h_{v_1}	12
h_{v_2}	12
h_f	8
other losses	14

The following is a step procedure of the computations using Method A. A discharge of 11,100 cfs for Red Fox River has been selected for the example. Column numbers refer to those of the form in Table 16.

Step 1

Enter the cross-section designation under Column 1. Column 2 shows the assumed water surface elevation. For Section 1,

this has been established as 5714.0 from the rating curve in Figure 28. The areas of 594 and 192 are read from Figure 20 and entered in Column 3. Corresponding K_d values are taken from Figure 13 and entered under Column 5.

Step 2

The friction slope, S_f , is computed from the equation $S_f = (Q/K_d)^2$ as shown in Column 6. Thus, in the example, this is equal to $(11,100/129,150)^2 = 0.00735$. Note that the total K_d (Column 5) is used.

Step 3

Column 9 shows the subdivisional discharges as computed by the equation

$$Q_s = \frac{Q\ K_{d_s}}{\Sigma\ K_d}$$

substituting the values of the main channel in the example:

$$Q_s = \frac{(11,100)(125,000)}{129,150} = 10,700$$

A similar computation is made for the overbank area using its K_d value of 4,150.

Step 4

The velocities in Column 10 are obtained by dividing Column 9 by Column 3 for each subdivision. Velocity in the main channel equals 10,700 divided by 594.

Step 5

Column 11 is computed by squaring Column 10 and multiplying by Column 9. The values of each subdivision are totaled and entered.

Step 6

The weighted velocity head, h_v, is computed from the following relationship:

$$\frac{\Sigma\ (V^2\ Q)}{2g\ Q}$$

Thus, in the example, this is calculated by substituting as follows:

$$\frac{3{,}461{,}730}{64.4 \times 11{,}100} \quad \begin{array}{l}\text{(Column 11)} \\ \text{(Gravity doubled times discharge)}\end{array}$$

which equals 4.85 shown in Column 12.

Step 7

Enter the distance, L , between sections in Column 4. This value is read from Table 5.

Step 8

Assume an elevation for Section 2 and enter in Column 2. This is shown as 5720.9 in the example. Similar entries are made in Columns 3 and 5 as was done for Section 1 described in Step 1 above.

Step 9

The friction slope is computed as described under Step 2 above. For Section 2, this equals $(11{,}100/428{,}000)^2 = 0.000673$.

Step 10

Column 7 is the average of consecutive friction slopes computed as follows:

$$\frac{0.00735 + 0.000673}{2} = 0.00401$$

Column 8 is the friction head which equals Column 7 multiplied by Column 4.

Step 11

Columns 9, 10, 11, and 12 are computed as described in Steps 3, 4, 5, and 6, above.

Step 12

The algebraic difference in velocity heads is entered in Column 13. In the example, this equals 4.85 - 0.43 = 4.42.

Step 13

The eddy loss Column 14 is taken as 10 percent of Column 13 for positive values or 50 percent for negative values. This equals (0.10)(4.42) = 0.44 in the example.

Step 14

Column 15 is the summation of Columns 8 and 14. Column 16 equals Column 15 plus Column 13.

Step 15

Column 17 is obtained by adding the water surface of the downstream section to Column 16. For example, this equals 5714.0 + 6.86 = 5720.86. When the result in Column 17 is equal to Column 2 to the nearest tenth foot, the computations are begun at next upstream section.

Figure 5

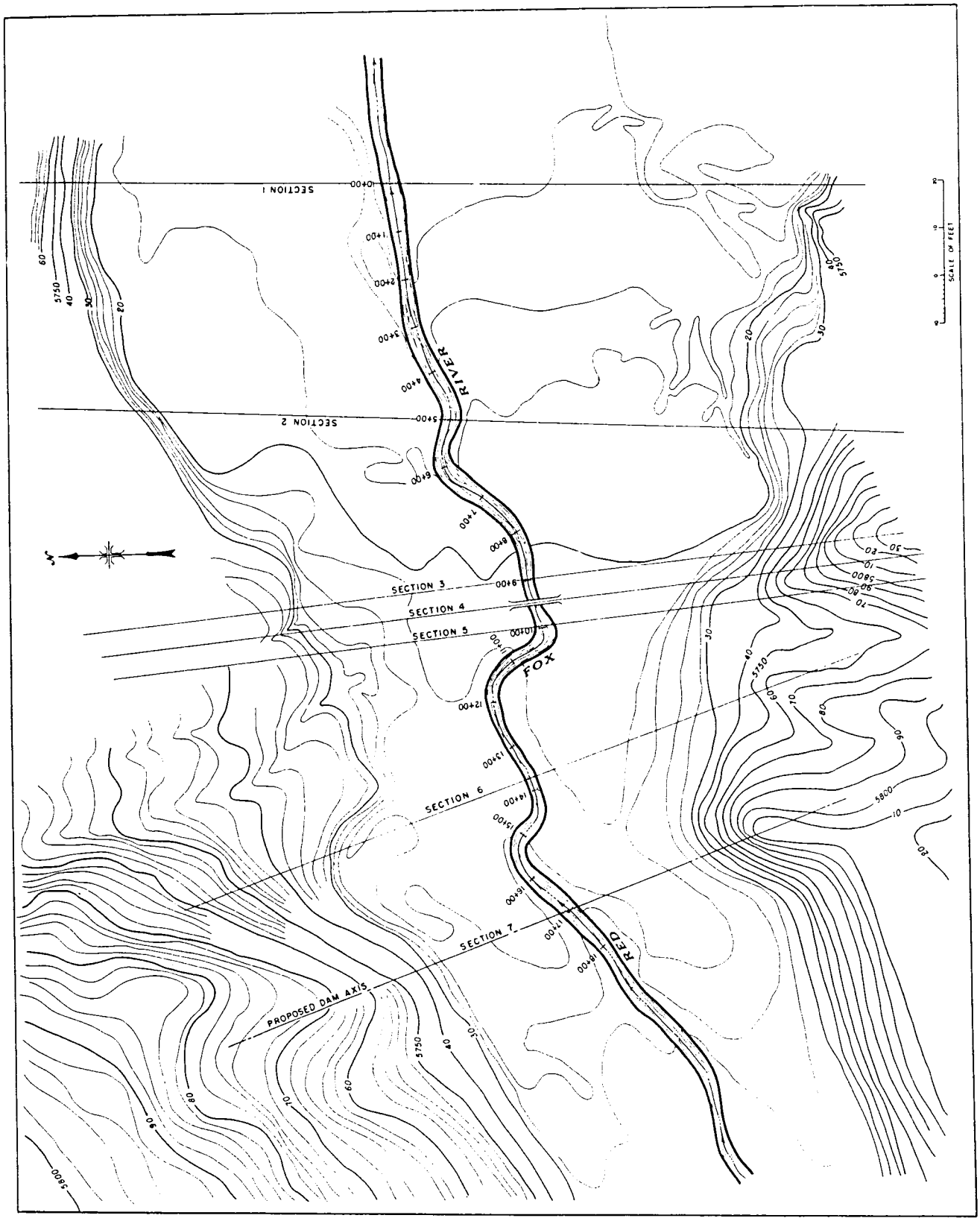

SECTION 1
SECTION 2
SECTION 3
SECTION 4
SECTION 5
SECTION 6
SECTION 7
PROPOSED DAM AXIS
RIVER
FOX
RED
SCALE OF FEET
5750
5800

Table 5

Red Fox River, Colorado

Flow Line

Section	Elevation	Length
1	5700.0	
		500
2	04.2	
		400
3	07.1	
		100
4	07.3	
		100
5	07.5	
		400
6	09.5	
		500
7	5712.5	

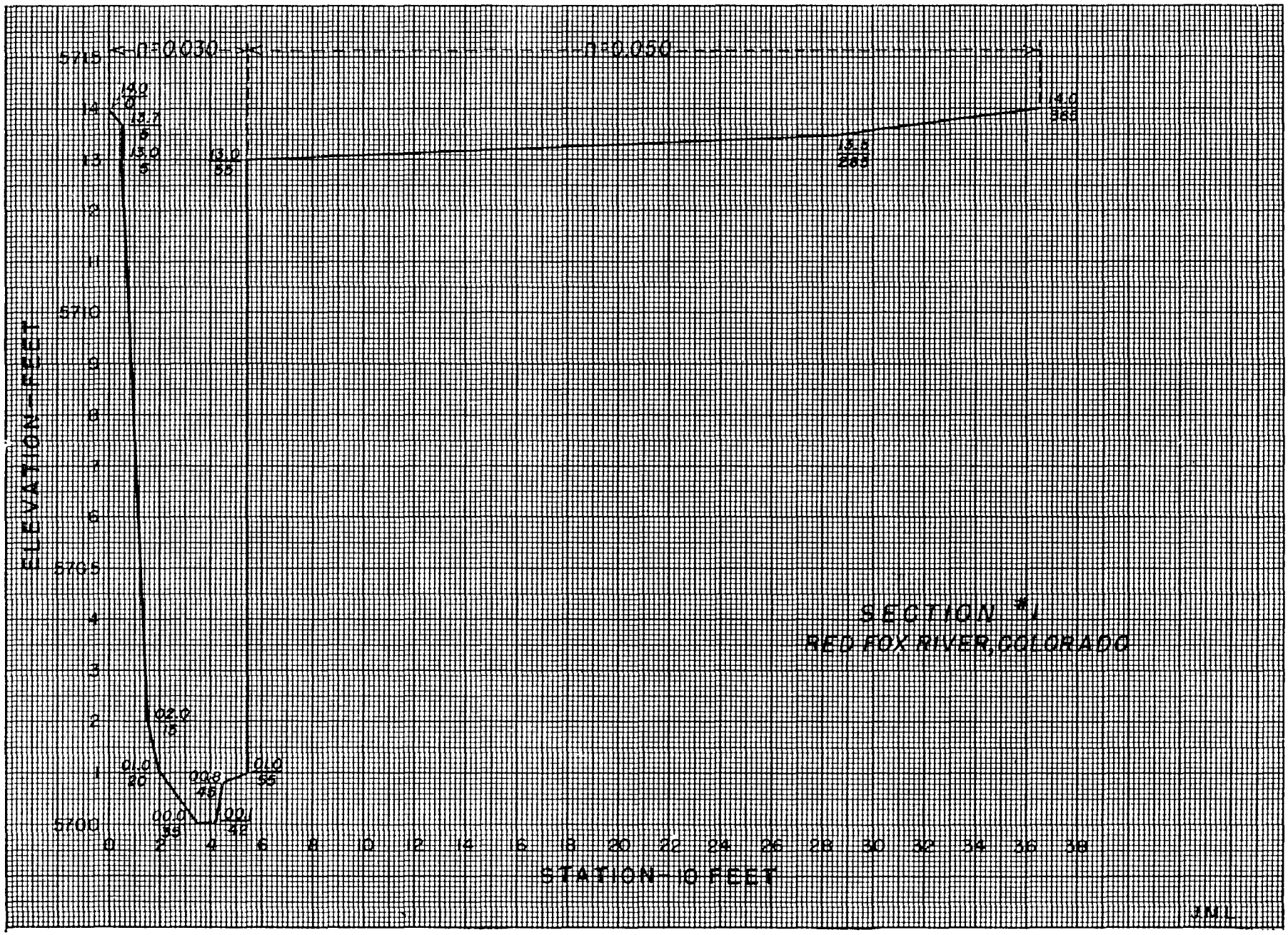

n=0.030
n=0.050
5715
5710
5705
5700
ELEVATION-FEET
STATION-10 FEET
SECTION #1
RED FOX RIVER, COLORADO

Figure 6

Figure 7

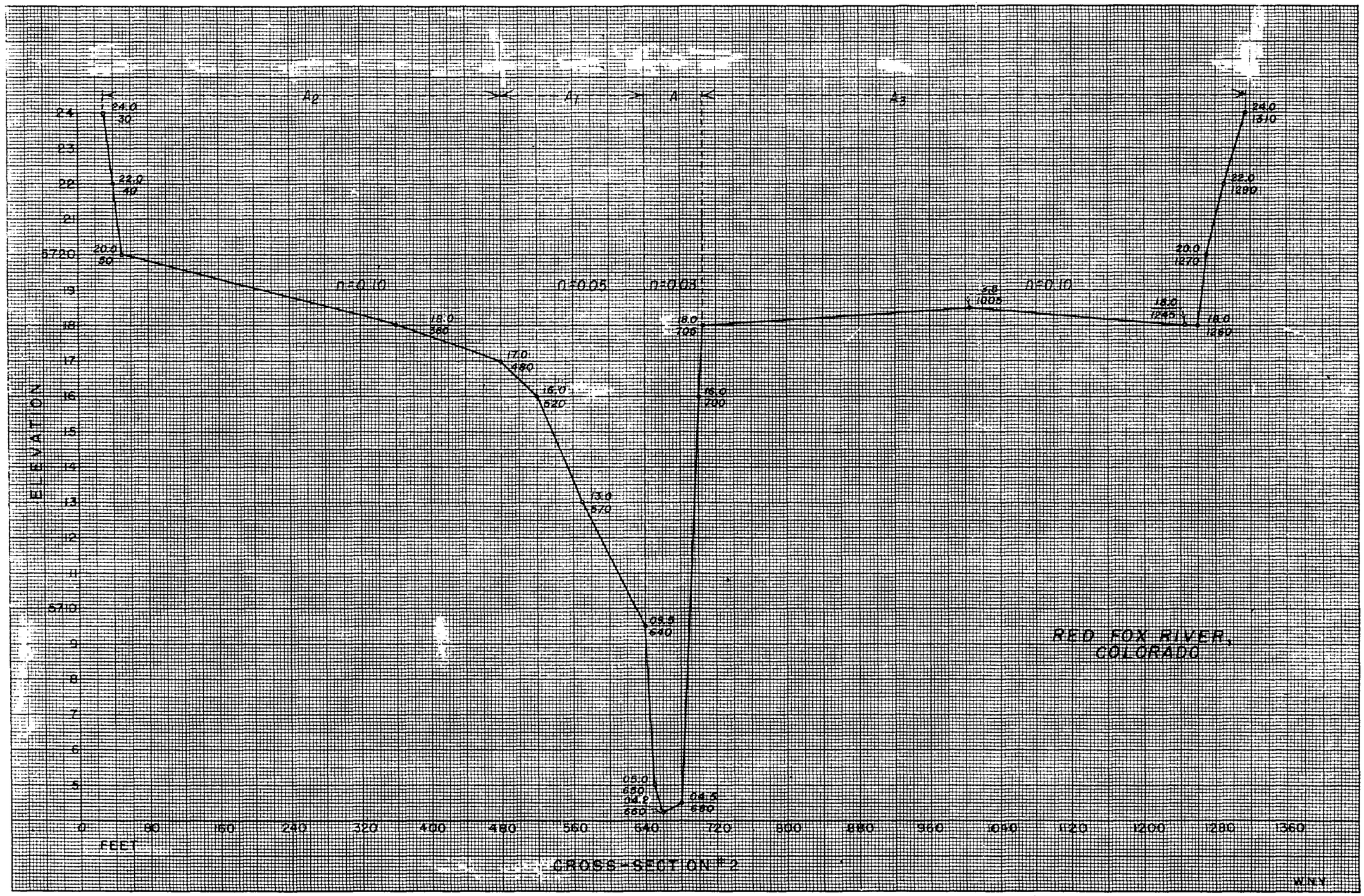
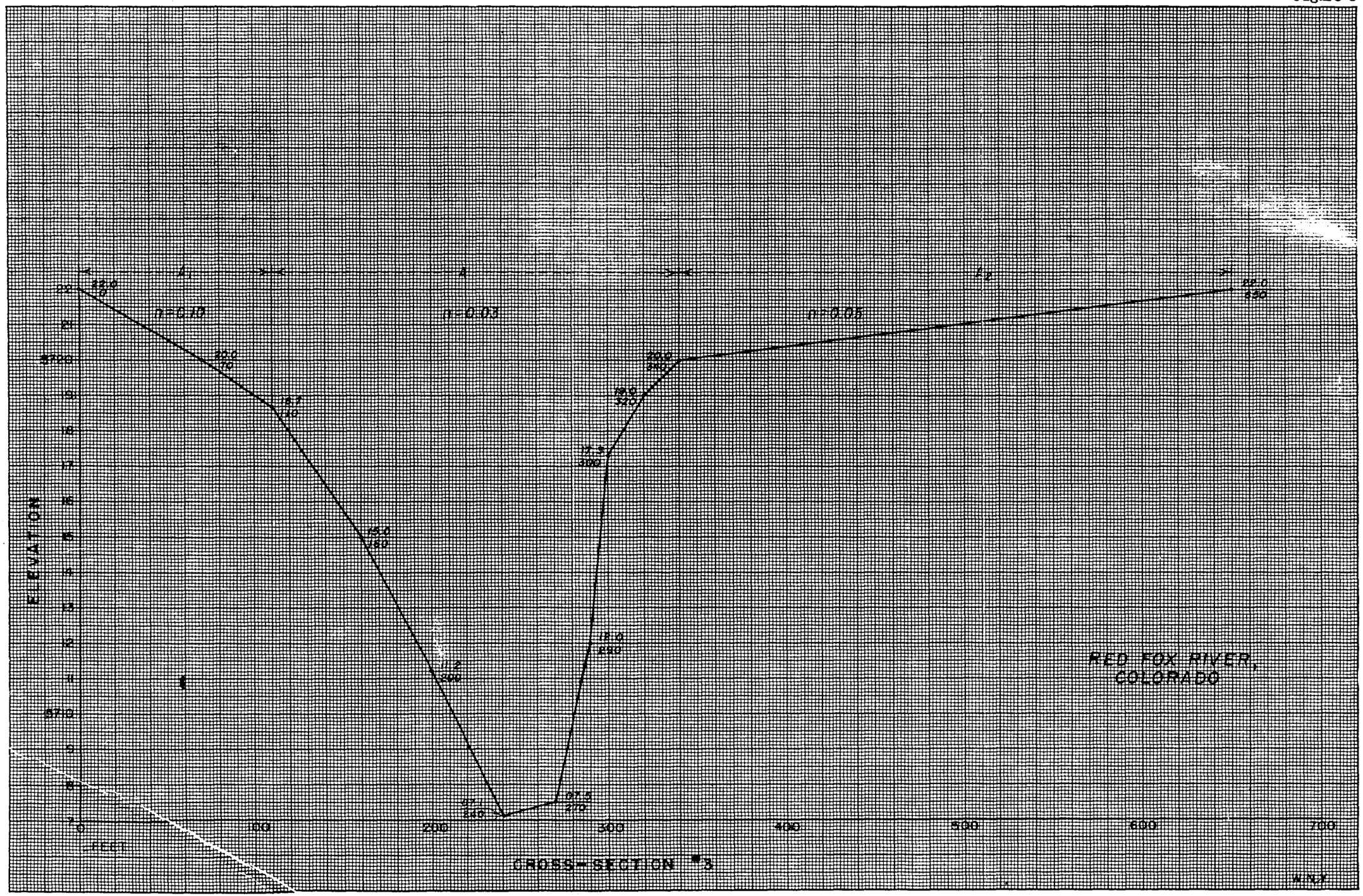

A2
A1
A
A3
n=0.10
n=0.05
n=0.03
n=0.10
24.0
30
22.0
40
20.0
50
18.0
350
17.0
480
16.0
520
13.0
570
05.5
640
05.0
650
04.2
660
04.5
680
18.0
700
16.0
700
2.8
1005
18.0
1245
18.0
1260
20.0
1270
22.0
1290
24.0
1310
ELEVATION
FEET
RED FOX RIVER,
COLORADO
CROSS-SECTION #2
WNY

Figure 8


A1
A
A2
n=0.10
n=0.03
n=0.05
22.0
0
20.0
10
18.7
10
15.0
160
11.2
200
17.5
240
20.0
240
19.0
320
17.5
300
18.0
290
17.5
270
22.0
650
ELEVATION
22
21
20
19
18
17
16
15
14
13
12
11
0
100
200
300
400
500
600
700
FEET
RED FOX RIVER,
COLORADO
CROSS-SECTION #3

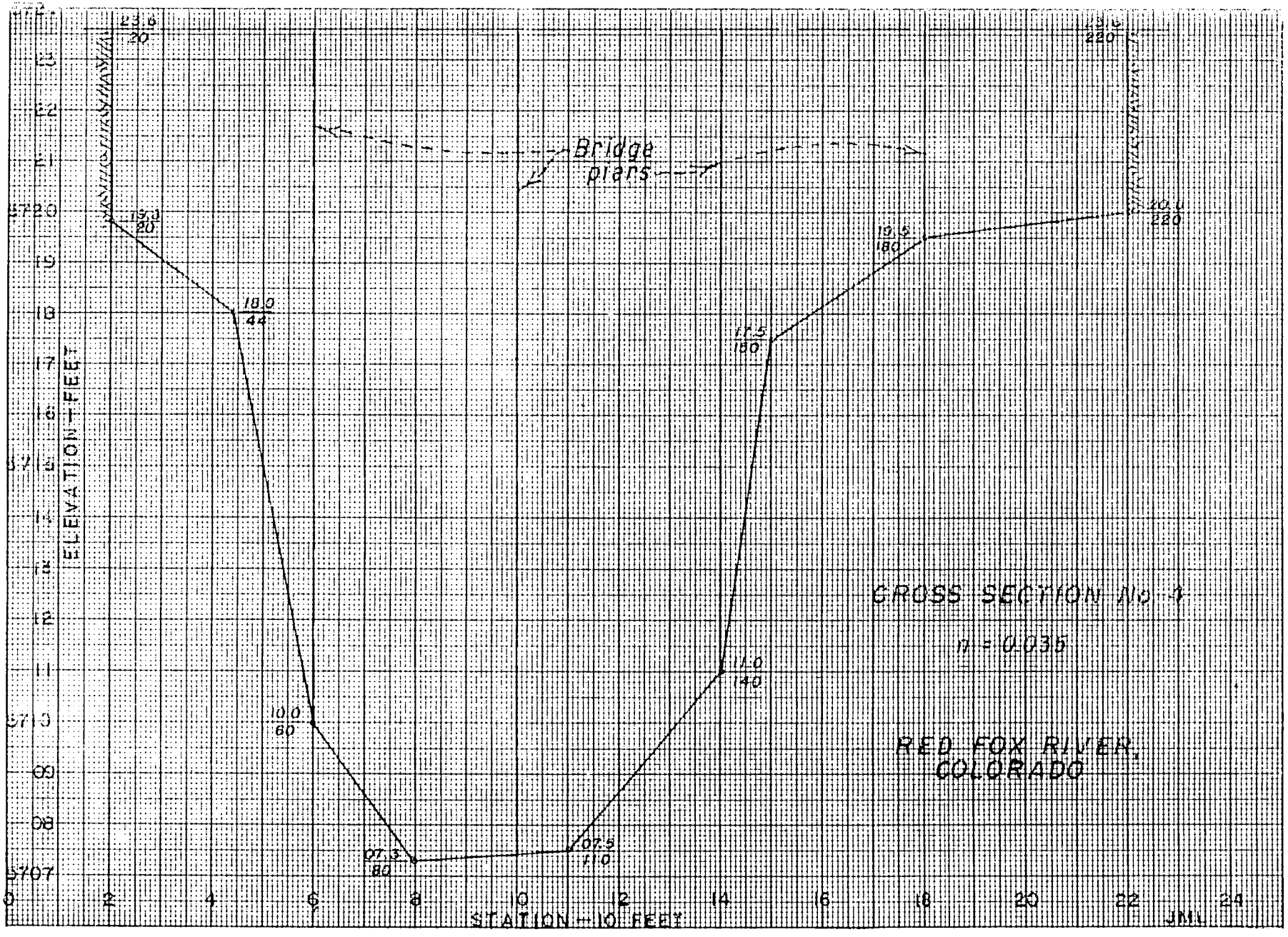

ELEVATION – FEET
STATION – 10 FEET
Bridge plans
23.5
20
19.3
20
18.0
44
10.0
60
07.3
80
07.5
110
11.0
140
17.5
180
19.5
180
20.0
220
CROSS SECTION No. 4
n = 0.035
RED FOX RIVER,
COLORADO
JML

Figure 9

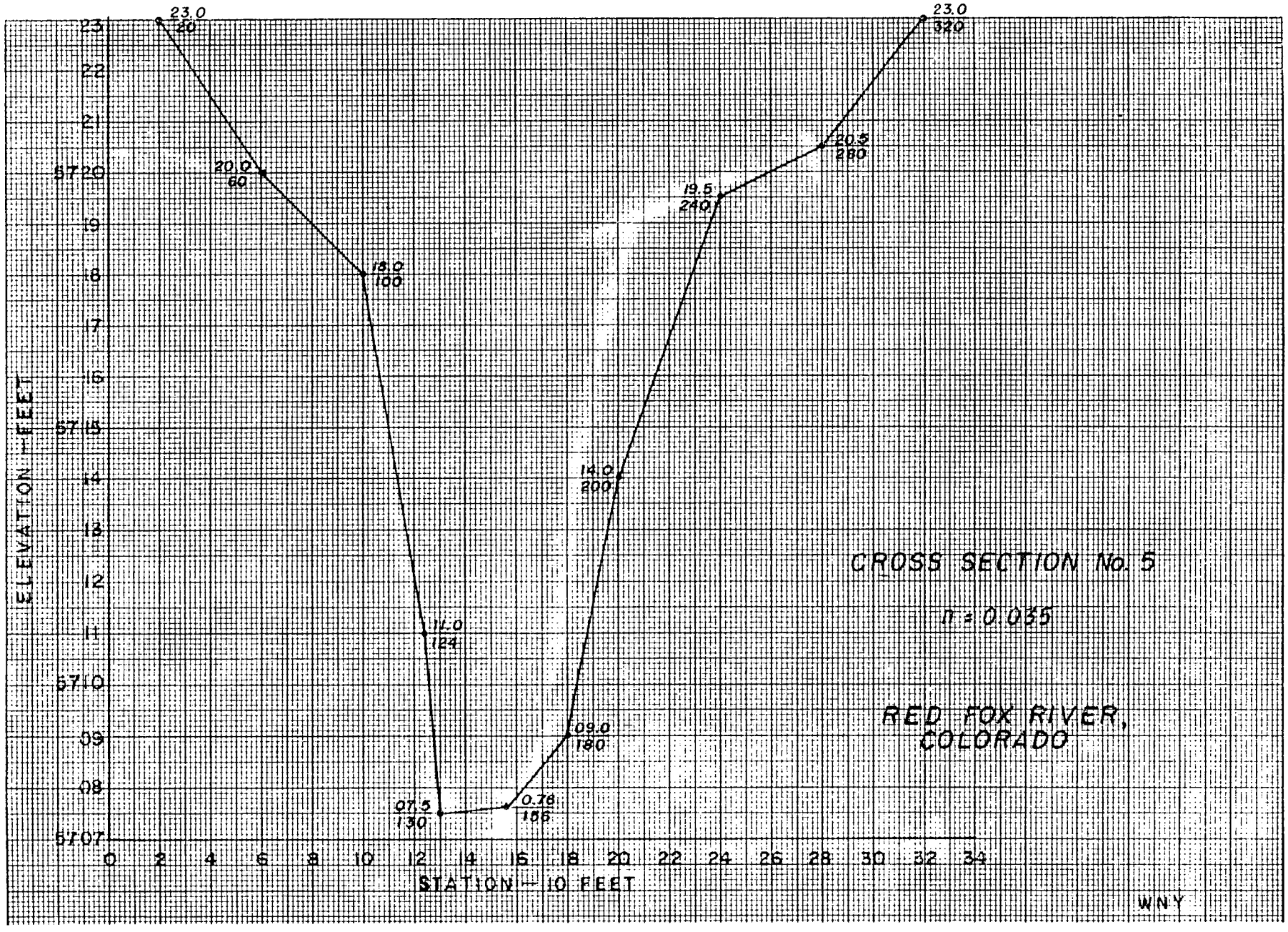

23.0
20
20.0
60
18.0
100
17.0
124
07.5
130
0.76
156
09.0
180
14.0
200
19.5
240
20.5
280
23.0
320
CROSS SECTION No. 5
n = 0.035
RED FOX RIVER,
COLORADO
ELEVATION — FEET
STATION — 10 FEET
WNY

Figure 10

Figure 11

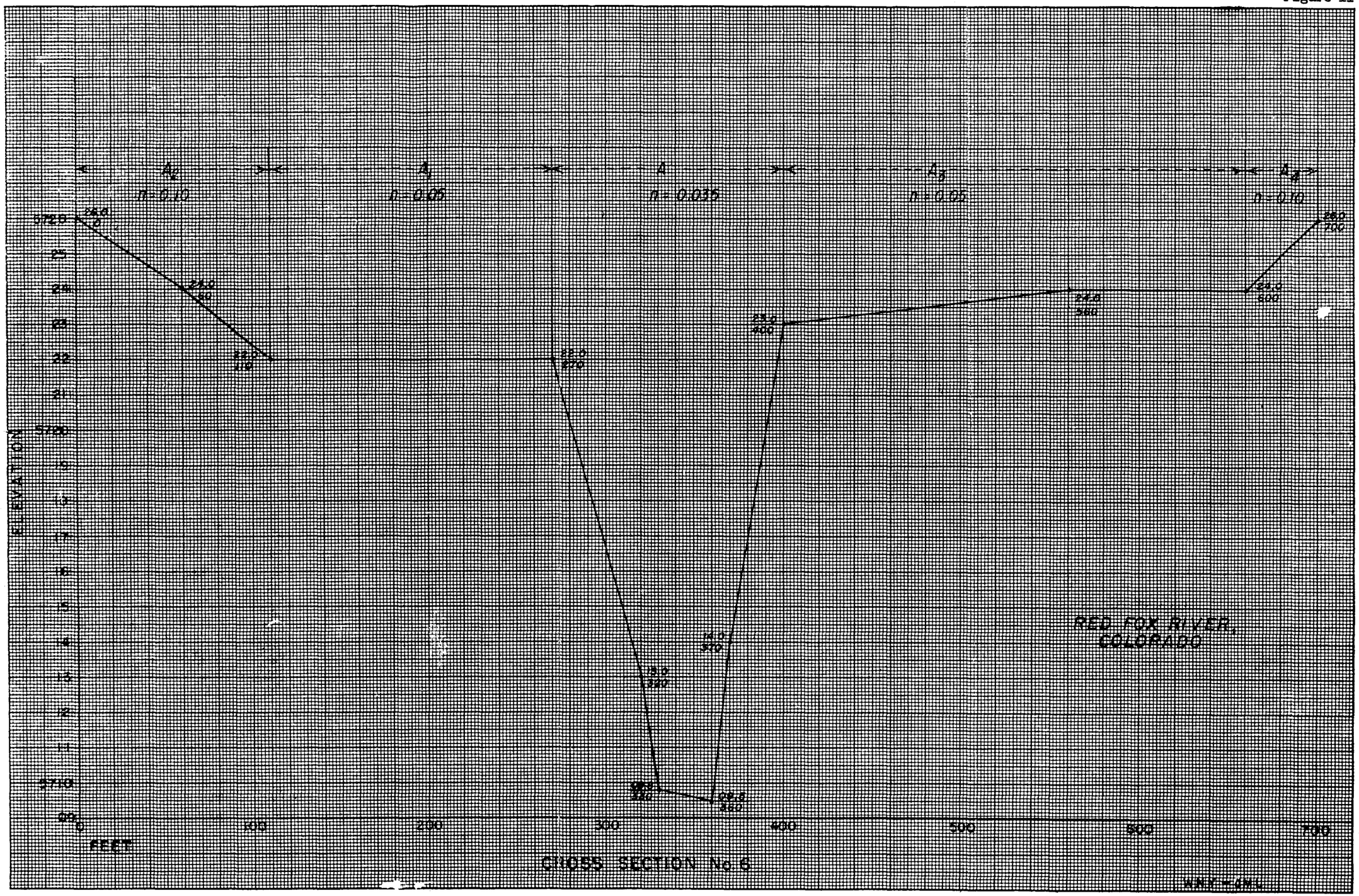

A6
n=0.10
A1
n=0.05
A
n=0.035
A3
n=0.05
A4
n=0.10
ELEVATION
FEET
0
100
200
300
400
500
600
700
5720
5710
RED FOX RIVER,
COLORADO
CROSS SECTION No. 6

Figure 12

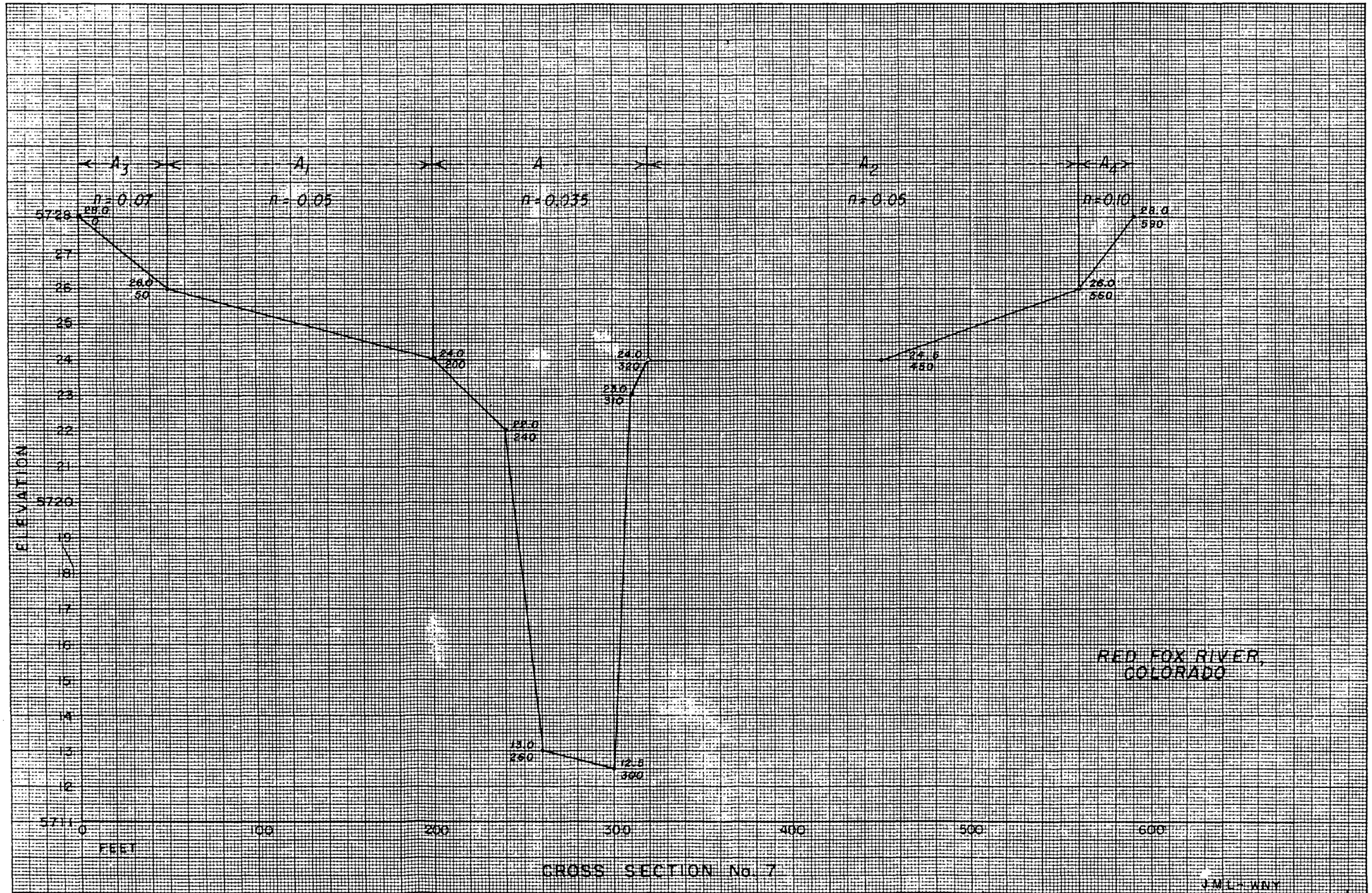

A3
n = 0.07
A1
n = 0.05
A
n = 0.035
A2
n = 0.05
A4
n = 0.10
28.0
0
28.0
50
24.0
200
22.0
240
24.0
320
23.0
310
13.0
290
12.5
300
24.6
450
26.0
550
28.0
590
5728
27
26
25
24
23
22
21
5720
19
18
17
16
15
14
13
12
5711
ELEVATION
0
100
200
300
400
500
600
FEET
RED FOX RIVER,
COLORADO
CROSS SECTION No. 7
J.M.L.-WNY

Table 6

HYDRAULIC CHARACTERISTICS

Section No. 1 — Main Channel

n = 0.030 — 1.486/n = 49.63

Elevation	A	R	$R^{2/3}$	Sub K_d	Total K_d
5701	14.5	0.48	0.613	440	440
5702	52	1.27	1.173	3,020	3,020
5707	263	4.99	2.920	38,000	38,000
5711	449	7.21	3.732	83,000	83,000
5713	547	8.17	4.057	110,000	110,000
5713.5	570	8.55	4.181	118,000	118,678
5714	594	8.77	4.253	125,000	129,150

Overbank Area A_1

n = 0.050 — 1.486/n = 29.72

5713.5	57.5	0.25	0.397	678
5714.0	192	0.62	0.727	4,150

Table 7

HYDRAULIC CHARACTERISTICS

Section No. 2 — Main Channel

n = 0.03 — 1.486/n = 49.53

Elevation	A	R	$R^{2/3}$	Sub K_d	Total K_d
5705.0	172	0.55	0.671	5,720	5,720
5709.5	197	3.84	2.452	23,900	23,900
5713.0	375	6.40	3.447	63,900	69,160
5716.0	550	8.57	4.188	114,000	141,400
5717.0	610	9.10	4.359	132,000	169,000
5718.0	674	9.72	4.555	152,000	209,627
5720.0	804	11.6	5.124	204,000	342,600
5722.0	934	13.4	5.642	261,000	551,700
5724.0	1060	15.3	6.163	324,000	820,000

Overbank Area A_1

n = 0.05 — 1.486/n = 29.72

5713.0	122	1.75	1.452	5,260
5716.0	408	3.39	2.257	27,400
5717.0	548	3.42	2.270	37,000
5718.0	705	4.40	2.685	56,200
5720.0	1030	6.42	3.454	106,000
5722.0	1345	8.42	4.139	166,000
5724.0	1670	10.4	4.765	236,000

Overbank Area A_2

n = 0.10 — 1.486/n = 14.86

5718	60	0.50	0.630	562
5720	610	1.42	1.263	11,400
5722	1480	3.36	2.243	49,300
5724	2370	5.27	3.028	107,000

Overbank Area A_3

n = 0.10 — 1.486/n = 14.86

5718	143	0.26	0.407	865
5720	985	1.74	1.447	21,200
5722	2140	3.65	2.371	75,400
5724	3320	5.49	3.112	153,000

Table 8

HYDRAULIC CHARACTERISTICS

Section No. 3 — Main Channel

n = 0.03 — 1.486/n = 49.53

Elevation	A	R	$R^{2/3}$	Sub K_d	Total K_d
5708	25.6	0.62	0.727	922	922
5709	73.5	1.33	1.209	4,401	4,401
5710	136	1.96	1.566	10,500	10,500
5712	253	2.56	1.871	23,400	23,400
5714	524	4.23	2.616	67,900	67,900
5718	1130	5.92	3.273	183,000	183,000
5718.7	1260	6.02	3.309	206,000	206,000
5719	1330	6.30	3.411	225,000	225,000
5720	1550	6.69	3.550	272,000	272,580
5721	1775	7.50	3.832	337,000	341,423
5722	2010	8.68	4.224	420,000	431,770

Overbank Area A_1

n = 0.05

5720	26	0.65	0.750	580	
5721	100	1.25	1.160	3,440	
5722	176	1.60	1.368	7,160	

Overbank Area A_2

n = 0.10

5721	105	0.50	0.630	983	
5722	310	1.00	1.000	4,610	

Table 9

HYDRAULIC CHARACTERISTICS

Section No. 4 — Bridge Section Main Channel

n = 0.035 — 1.486/n = 42.46

Elevation	A	R	$R^{2/3}$	Total K_d
5708	21.4	0.55	0.671	610
5709	65.6	1.14	1.091	3,040
5710	127	1.68	1.413	7,620
5711	200	2.29	1.737	14,750
5712	277	2.90	2.034	23,920
5713	358	3.39	2.257	34,310
5715	531	4.19	2.599	58,600
5717.5	768	5.08	2.955	96,400
5718	820	5.01	2.928	102,000
5719	942	4.74	2.822	113,000
5719.5	1010	4.72	2.814	121,000
5719.8	1062	4.27	2.632	119,000
5720	1100	4.26	2.628	123,000
5722	1484	5.36	3.063	193,000

No Overbank Flow

Table 10

HYDRAULIC CHARACTERISTICS

Section No. 5 **Main Channel**

n = 0.035 1.486/n = 42.46

Elevation	A	R	$R^{2/3}$	Total K_d
5708	13.2	0.45	0.587	329
5709	56.8	1.06	1.040	2,510
5710	111	1.91	1.539	7,220
5711	172	2.67	1.925	14,100
5712	240	3.32	2.226	22,700
5714	369	4.17	2.591	40,600
5716	563	5.72	3.198	76,400
5718	800	6.07	3.328	113,000
5719	956	6.02	3.309	134,000
5719.5	1050	6.07	3.328	148,000
5720	1150	5.62	3.161	154,000
5720.5	1250	5.45	3.097	164,000
5723	1910	6.31	3.415	277,000

Table 11

HYDRAULIC CHARACTERISTICS

Section No. 6 **Main Channel**

n = 0.035 1.486/n = 49.53

Elevation	A	R	$R^{2/3}$	Sub K_d	Total K_d
5710	10.8	0.34	0.487	223	223
5711	45.2	1.20	1.129	2,170	2,170
5712	84.4	1.97	1.571	5,630	5,630
5713	130	2.65	1.915	10,500	10,500
5714	182	3.17	2.158	18,300	18,300
5716	311	4.12	2.570	33,900	33,900
5718	476	5.07	2.951	59,700	59,700
5720	674	6.04	3.317	95,100	95,100
5722	912	7.00	3.659	141,000	141,000
5723	1040	7.78	3.926	173,000	177,760
5724	1170	8.75	4.246	211,000	228,343
5725	1300	9.70	4.548	251,000	294,960
5726	1430	10.7	4.856	295,000	379,494
		Overbank Area A_1			
		n = 0.05			
5723	160	1.00	1.000	4,760	
5724	320	2.00	1.587	15,100	
5725	485	3.00	2.080	29,500	
5726	640	4.00	2.520	47,900	
		Overbank Area A_2			
		n = 0.10			
5724	50	1.00	1.000	743	
5725	130	1.44	1.275	2,460	
5726	210	1.91	1.539	4,800	
		Overbank Area A_3			
		n = 0.05			
5724	80	0.50	0.630	1,500	
5725	340	1.31	1.191	12,000	
5726	600	2.31	1.747	31,200	
		Overbank Area A_4			
		n = 0.10			
5726	40	1.00	1.000	594	

Table 12

HYDRAULIC CHARACTERISTICS

Section No. 7 **Main Channel**

n = 0.035

Elevation	A	R	$R^{2/3}$	Sub K_d	Total K_d
5713	10.2	0.25	0.397	171	171
5715	97.8	2.02	1.598	6,630	6,630
5718	254	4.22	2.611	28,100	28,100
5720	372	5.52	3.123	49,300	49,300
5722	503	6.71	3.558	75,900	75,900
5723	582	6.04	3.317	81,900	81,900
5724	688	5.44	3.093	90,000	90,000
5725	808	6.38	3.440	118,000	122,661
5726	928	7.34	3.777	149,000	168,160

Overbank Area A_1

n = 0.050

5725	75	0.50	0.630	701	
5726	150	1.00	1.000	4,460	

Overbank Area A_2

n = 0.050

5725	158	0.85	0.897	3,960	
5726	370	1.54	1.334	14,700	

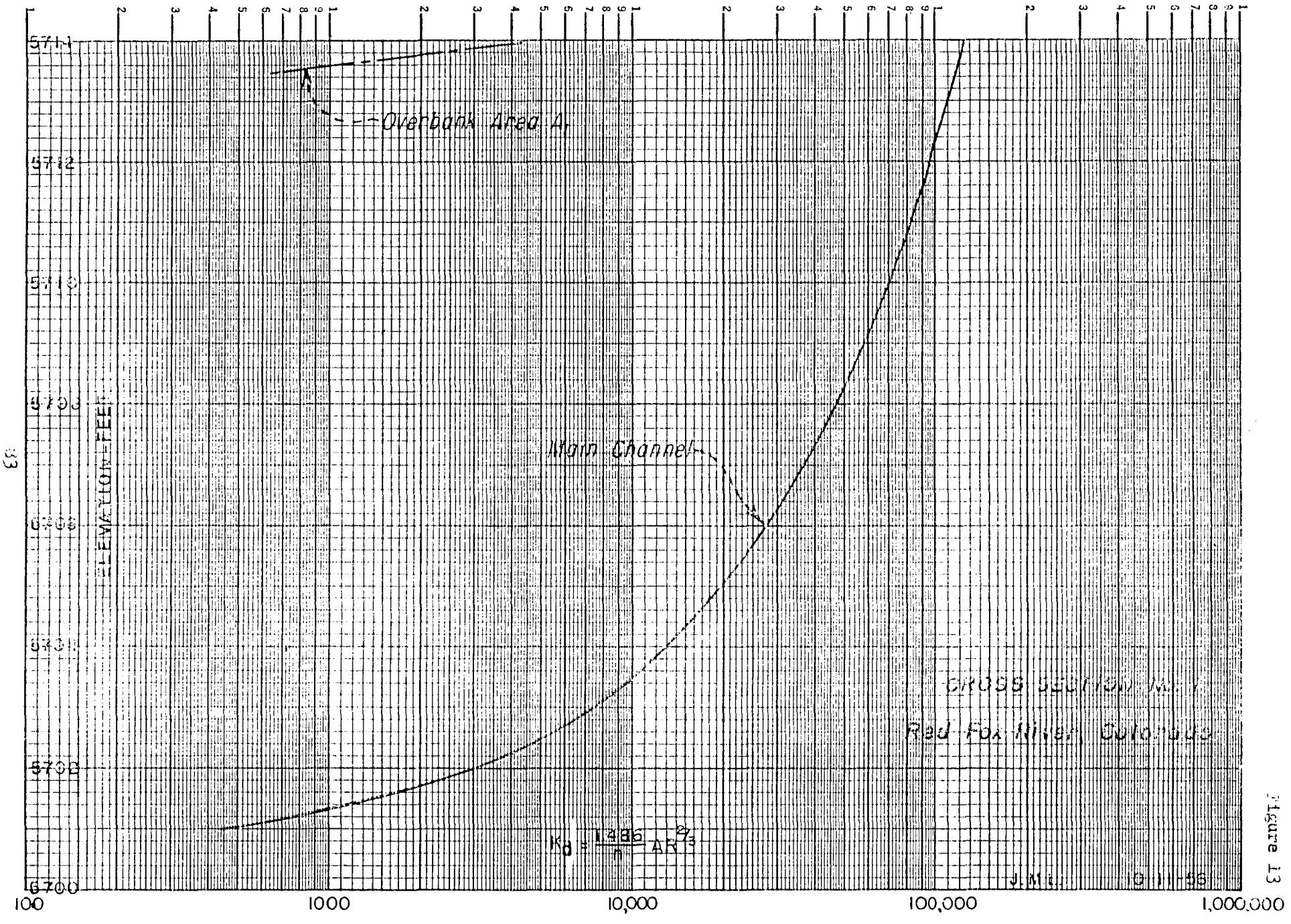
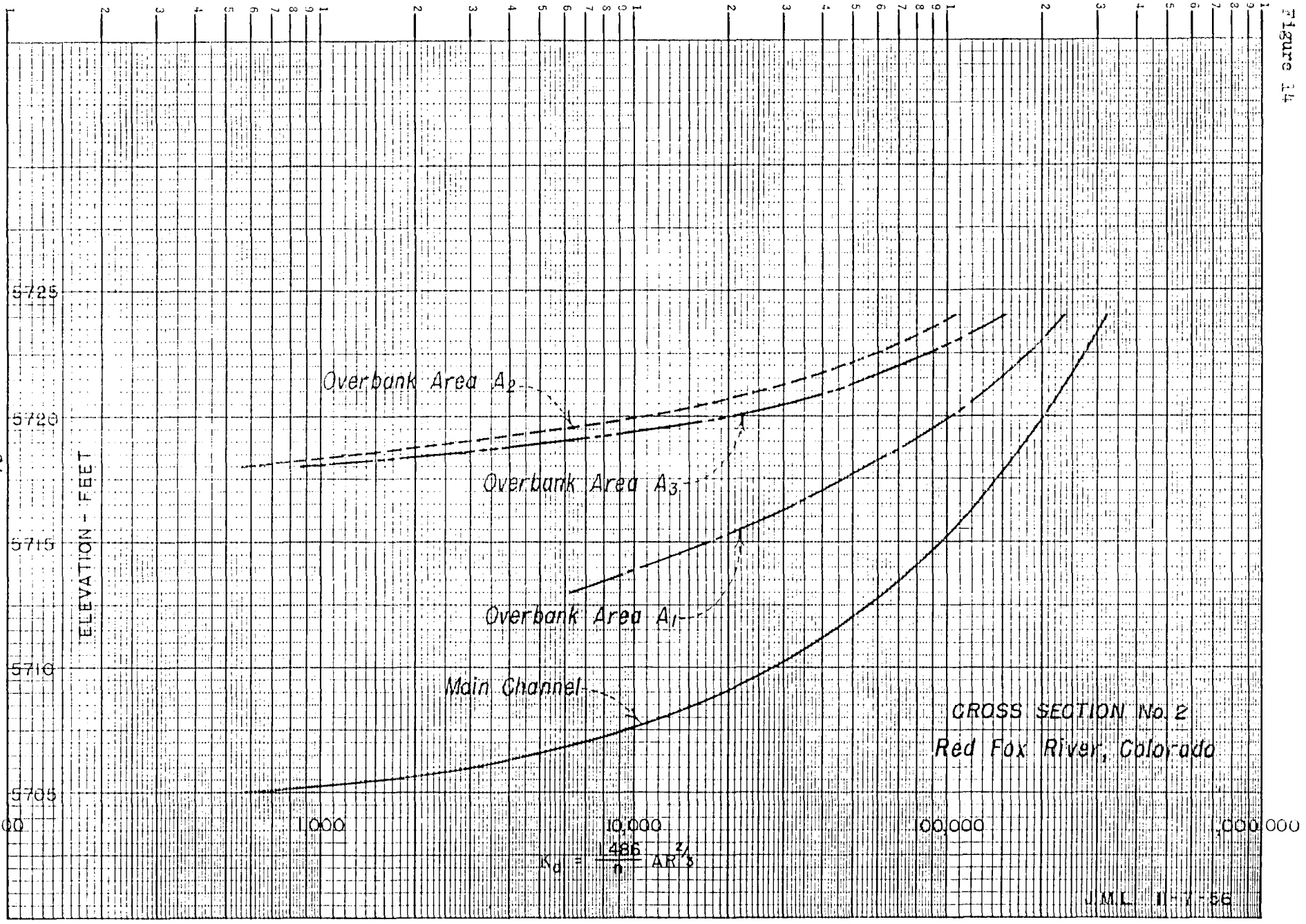

Overbank Area A_1
Main Channel
CROSS SECTION No. 1
Red Fox Creek, Colorado
$K_0 = \frac{1.486}{n} AR^{2/3}$
J.M.L.
10-11-56
ELEVATION-FEET
5712
5710
5708
5706
5704
5702
5700
6700
100
1000
10,000
100,000
1,000,000

Figure 13

Figure 14


ELEVATION - FEET
5725
5720
5715
5710
5705
100
1,000
10,000
100,000
1,000,000
$K_a = \frac{1.486}{n} AR^{2/3}$
Overbank Area A_2
Overbank Area A_3
Overbank Area A_1
Main Channel
CROSS SECTION No. 2
Red Fox River, Colorado
J.M.L. 11-7-56

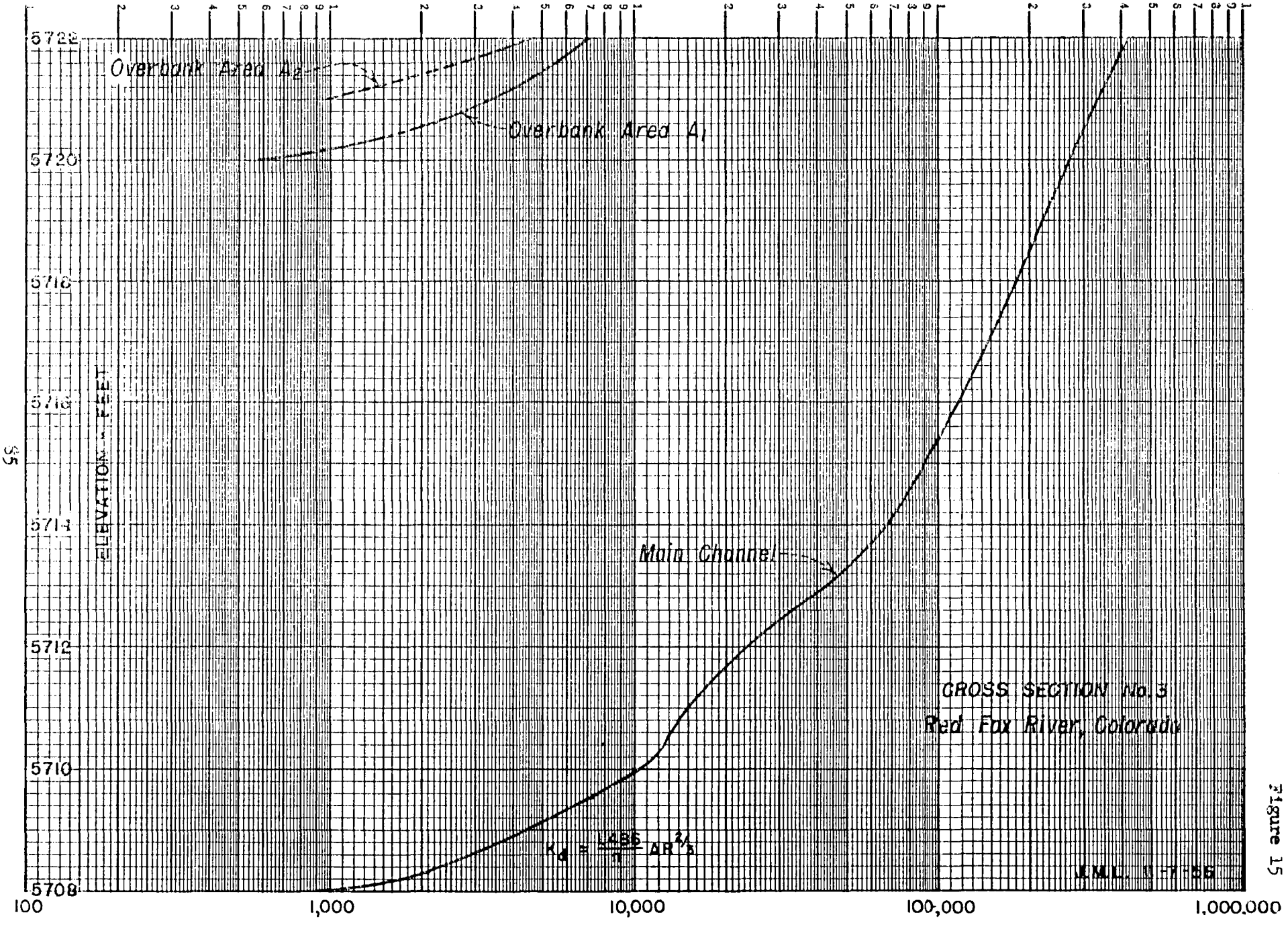

Overbank Area A_2
Overbank Area A_1
Main Channel
CROSS SECTION No. 3
Red Fox River, Colorado
$K_d = \frac{1.486}{n} AR^{2/3}$
J.M.L. 1-7-56
ELEVATION - FEET
5722
5720
5718
5716
5714
5712
5710
5708
100
1,000
10,000
100,000
1,000,000

Figure 15

Figure 16

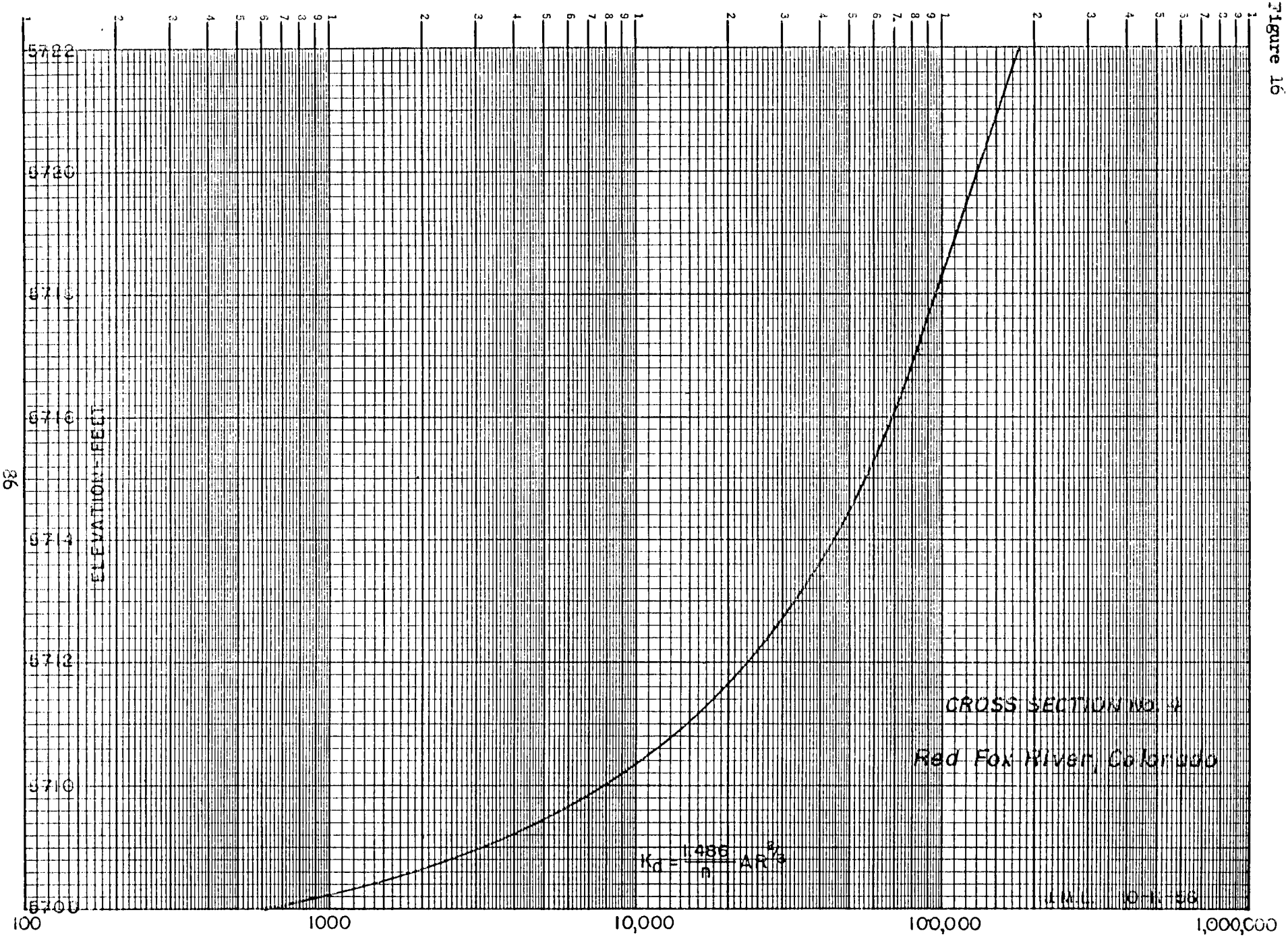

ELEVATION - FEET
5722
5720
5718
5716
5714
5712
5710
5700
100
1000
10,000
100,000
1,000,000
$K_a = \frac{1.486}{n} AR^{2/3}$
CROSS SECTION NO. 2
Red Fox River, Colorado

Figure 17

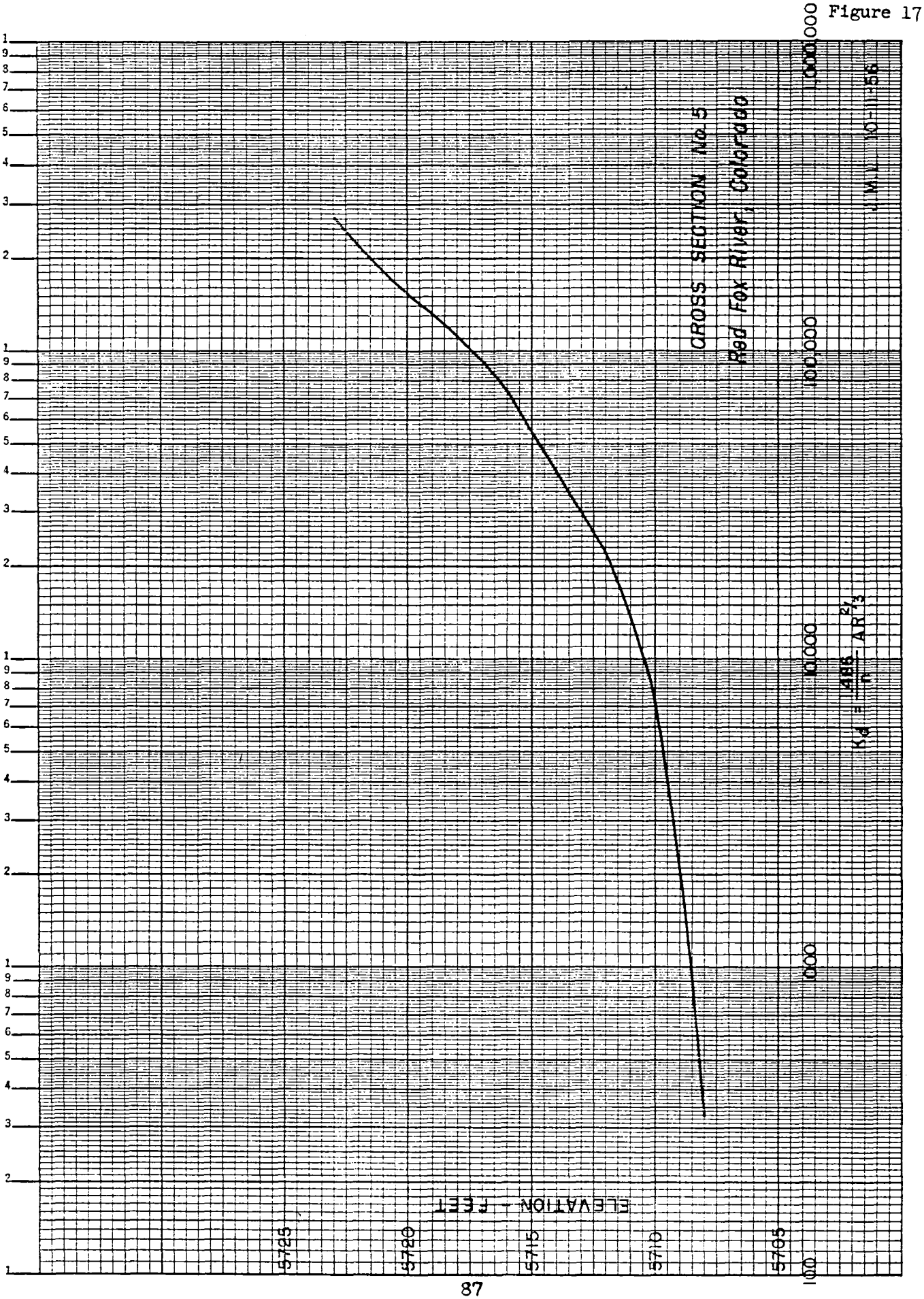
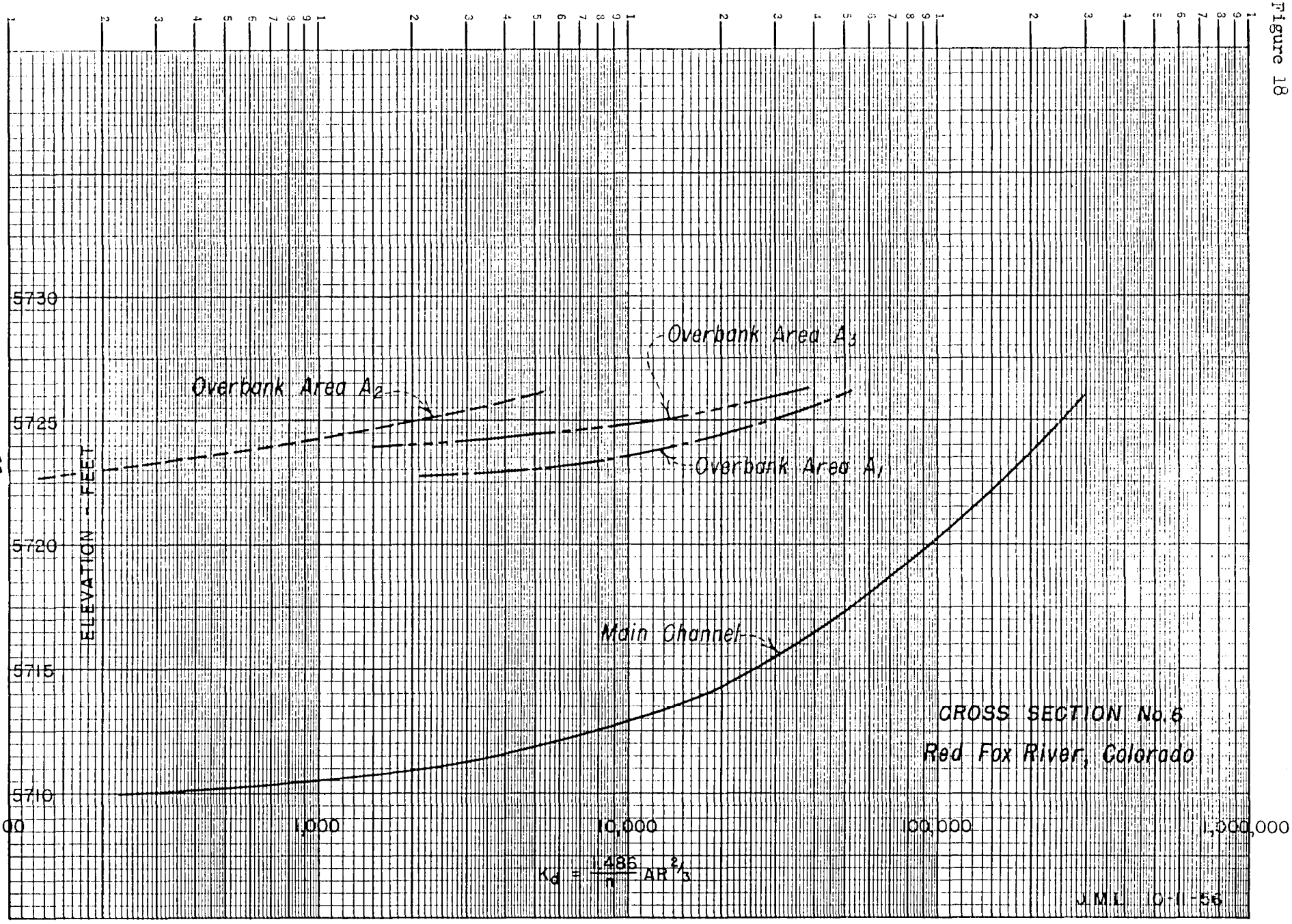

CROSS SECTION No. 5
Red Fox River, Colorado
J.M.L. 10-11-56
$K_d = \frac{486}{n} AR^{2/3}$
1,000,000
100,000
10,000
1,000
100
ELEVATION - FEET
5725
5720
5715
5710
5705

Figure 18


ELEVATION - FEET
5730
5725
5720
5715
5710
100
1,000
10,000
100,000
1,000,000
Overbank Area A_3
Overbank Area A_2
Overbank Area A_1
Main Channel
CROSS SECTION No. 6
Red Fox River, Colorado
$K_d = \frac{.486}{n} AR^{2/3}$
J.M.L. 10-1-56

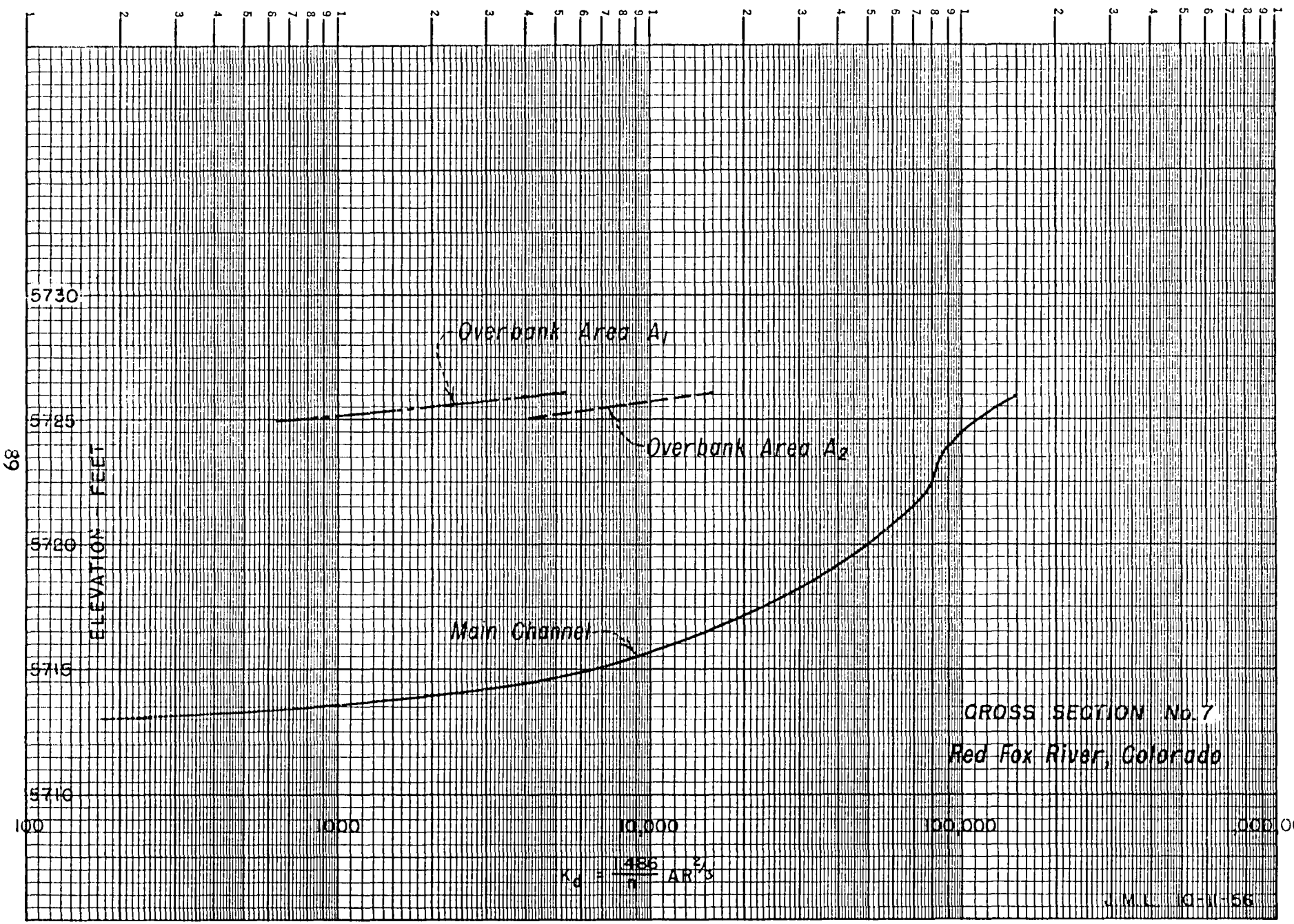

5730
5725
5720
5715
5710
ELEVATION — FEET
Overbank Area A_1
Overbank Area A_2
Main Channel
CROSS SECTION No. 7
Red Fox River, Colorado
100
1000
10,000
100,000
1,000,000
$K_d = \frac{1.486}{n} AR^{2/3}$
J.M.L. 10-11-56

Figure 19

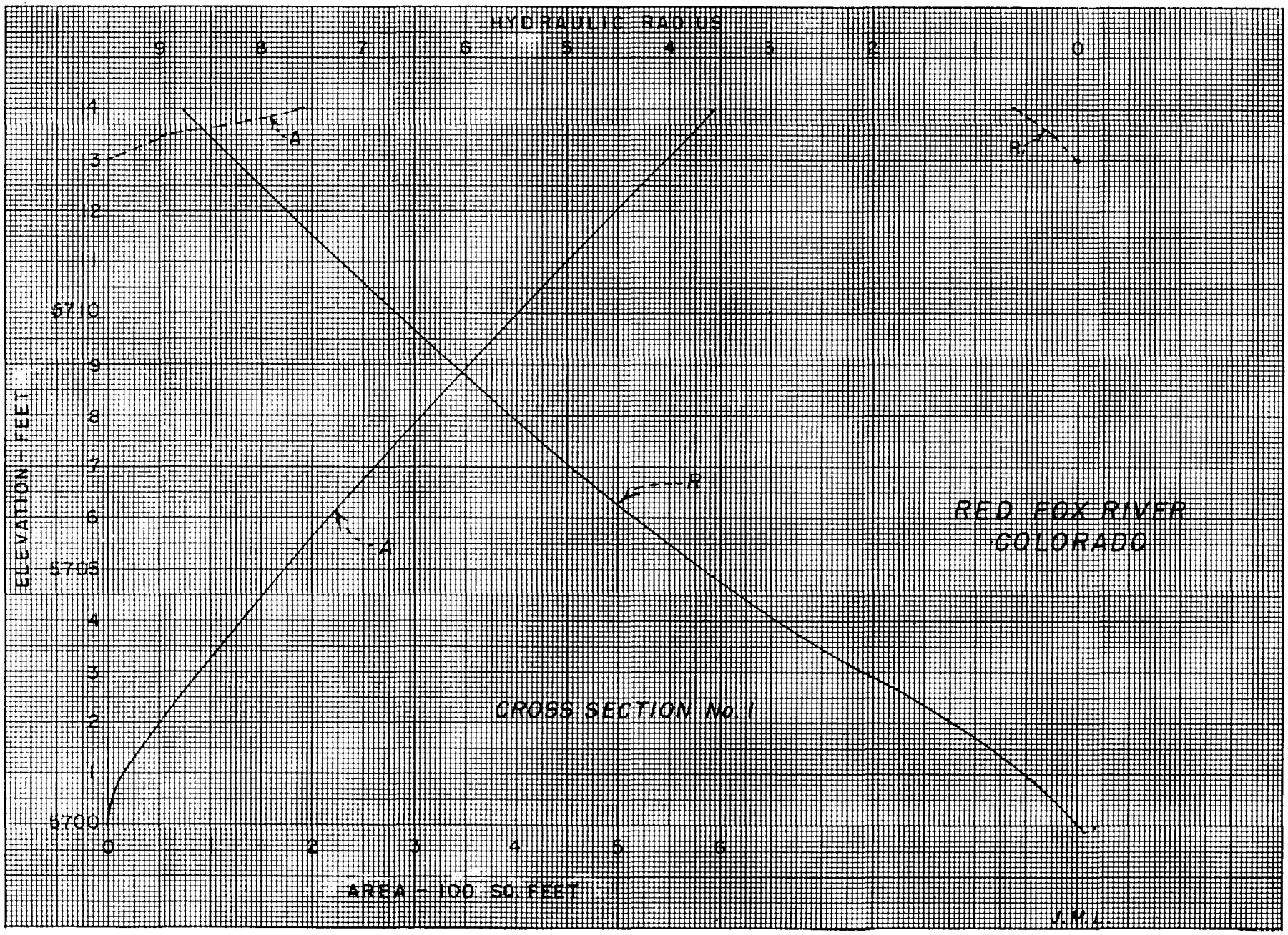
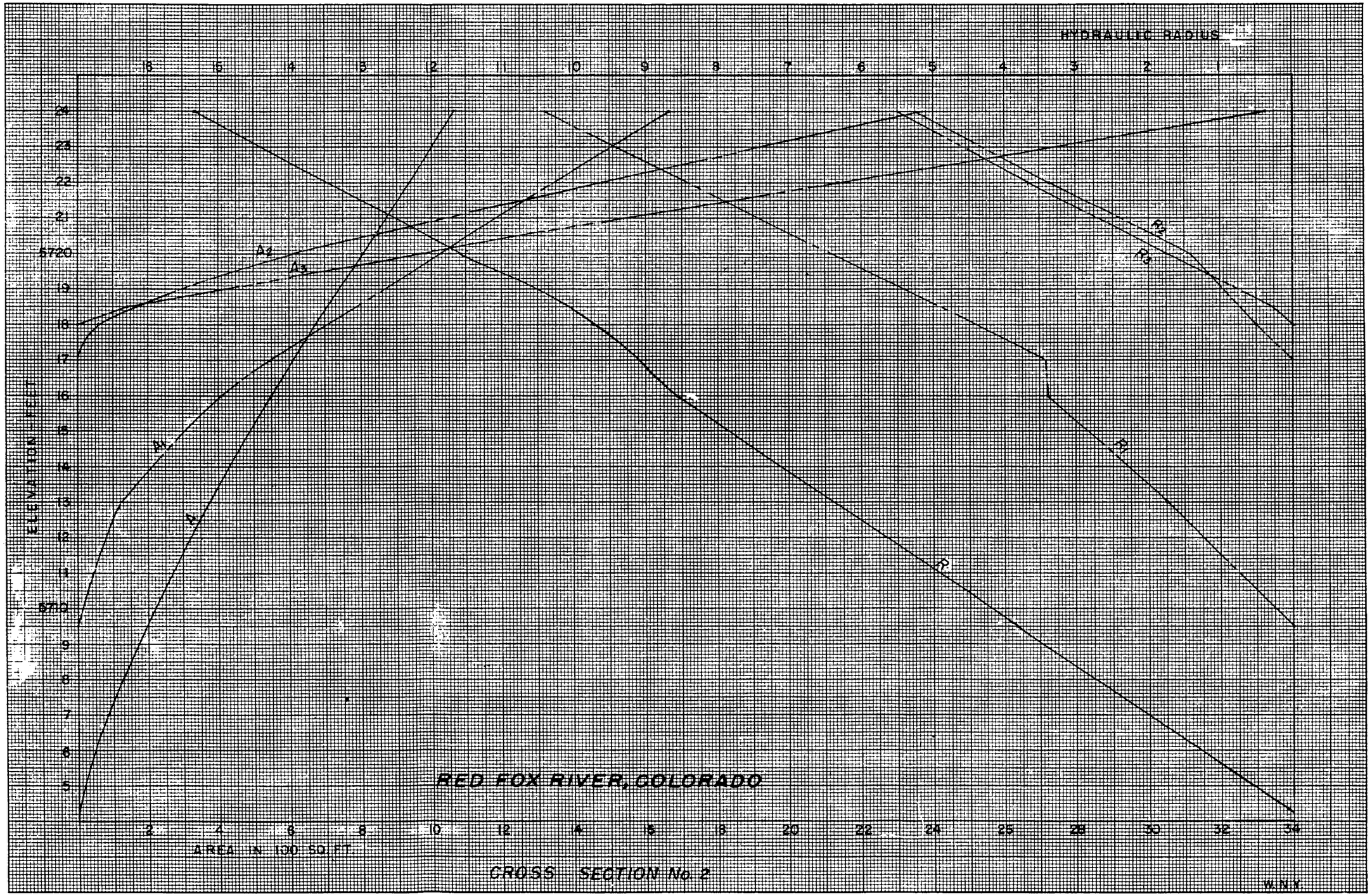

HYDRAULIC RADIUS
9
8
7
6
5
4
3
2
0
ELEVATION - FEET
14
13
12
11
5710
9
8
7
6
5705
4
3
2
1
5700
0
1
2
3
4
5
6
AREA - 100 SQ. FEET
A
A
R
R
RED FOX RIVER
COLORADO
CROSS SECTION No. 1
J.M.L.

Figure 20

Figure 21


HYDRAULIC RADIUS
16 15 14 13 12 11 10 9 8 7 6 5 4 3 2 1
ELEVATION FEET
24 23 22 21 5720 19 18 17 16 15 14 13 12 11 5710 9 8 7 6 5
A1
A2
A3
R1
R2
R3
RED FOX RIVER, COLORADO
2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34
AREA IN 100 SQ. FT.
CROSS SECTION No. 2
W.N.Y.

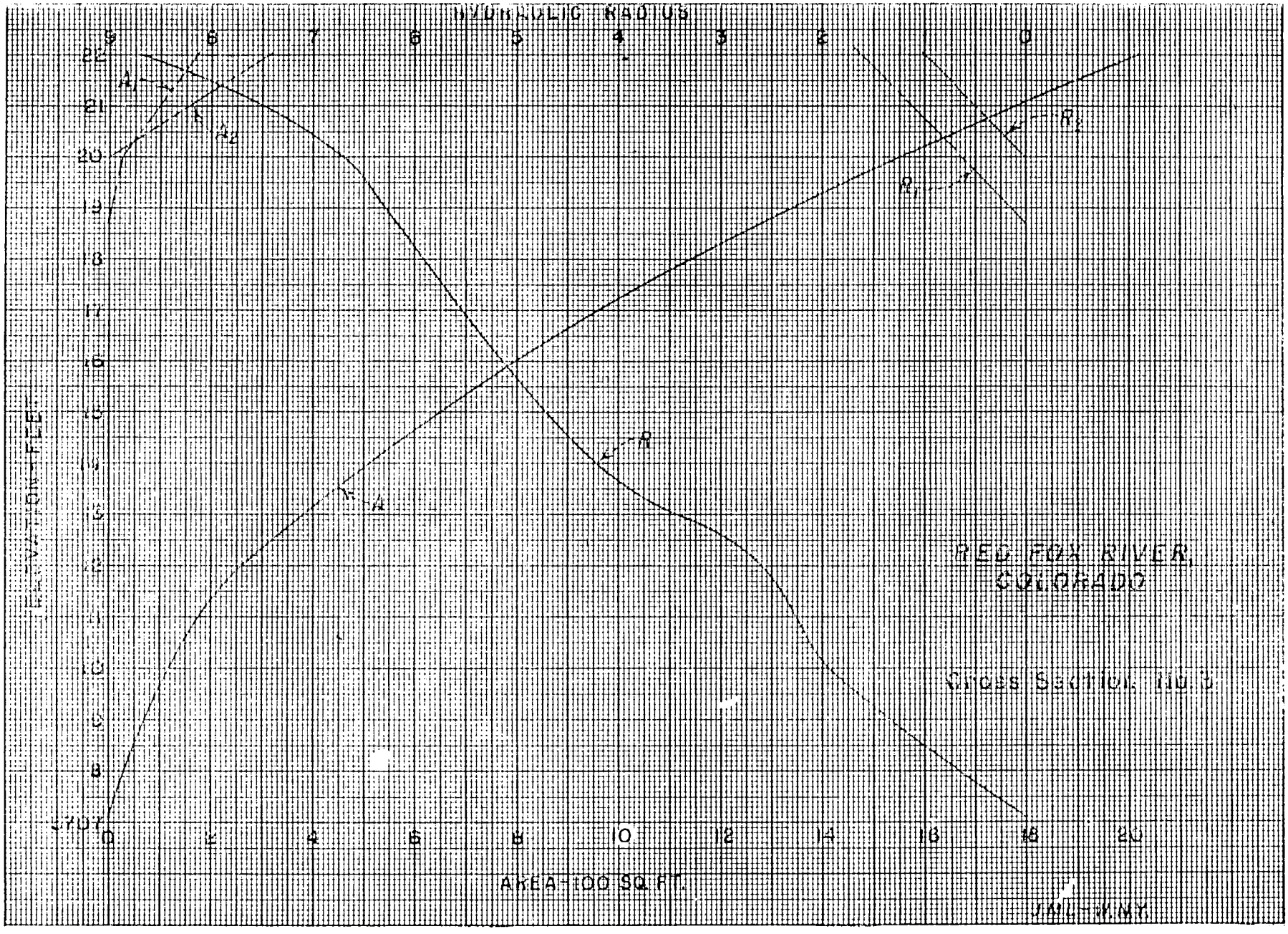

HYDRAULIC RADIUS
ELEVATION - FEET
AREA - 100 SQ. FT.
RED FOX RIVER,
COLORADO
Cross Section No. 1
A
A1
A2
R
R1
R2

Figure 22

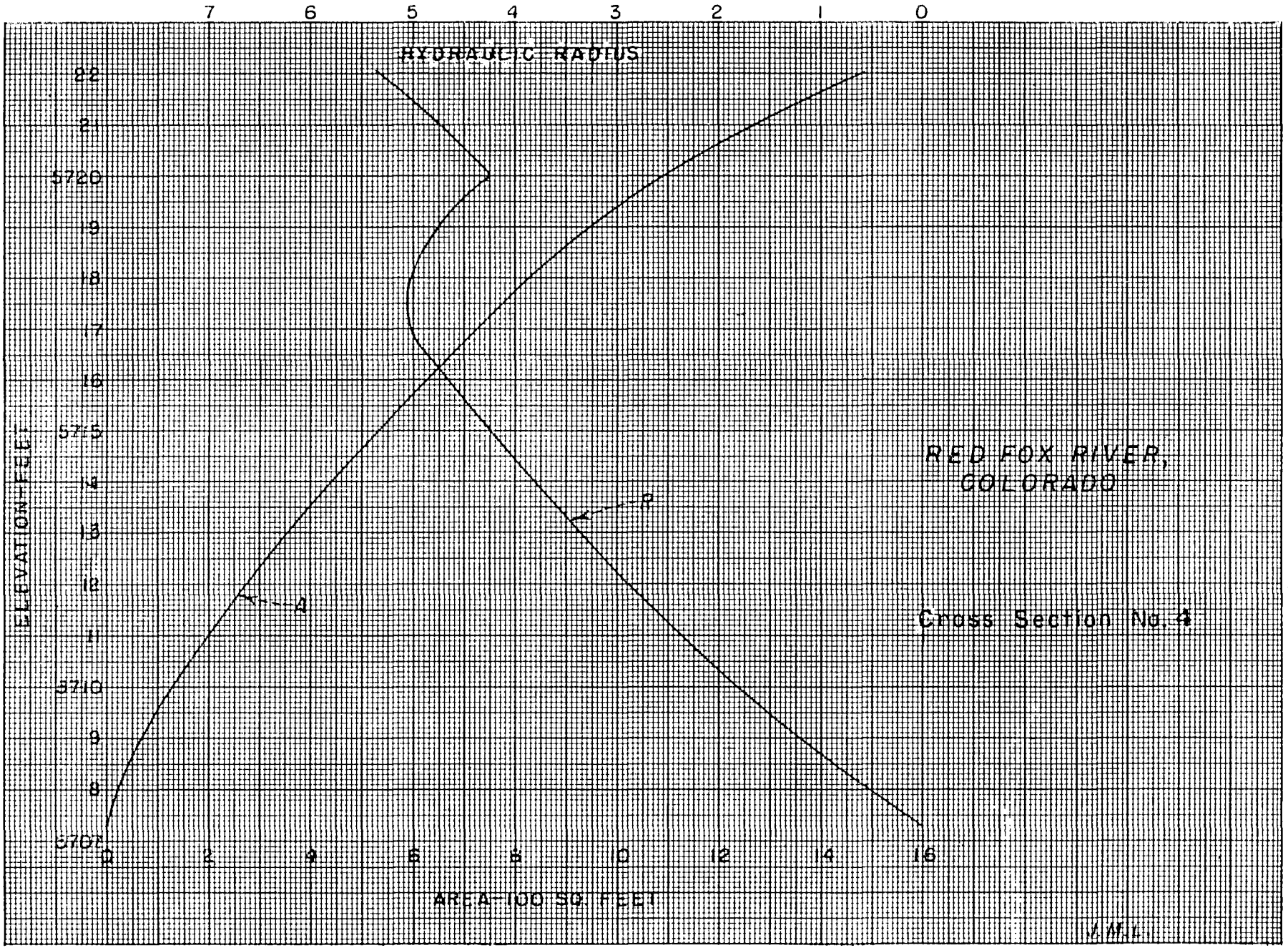

HYDRAULIC RADIUS
7
6
5
4
3
2
1
0
ELEVATION-FEET
5720
5710
5700
22
21
20
19
18
17
16
15
14
13
12
11
10
9
8
A
B
RED FOX RIVER,
COLORADO
Cross Section No. 4
0
2
4
6
8
10
12
14
16
AREA-100 SQ. FEET
J.M.L.

Figure 23

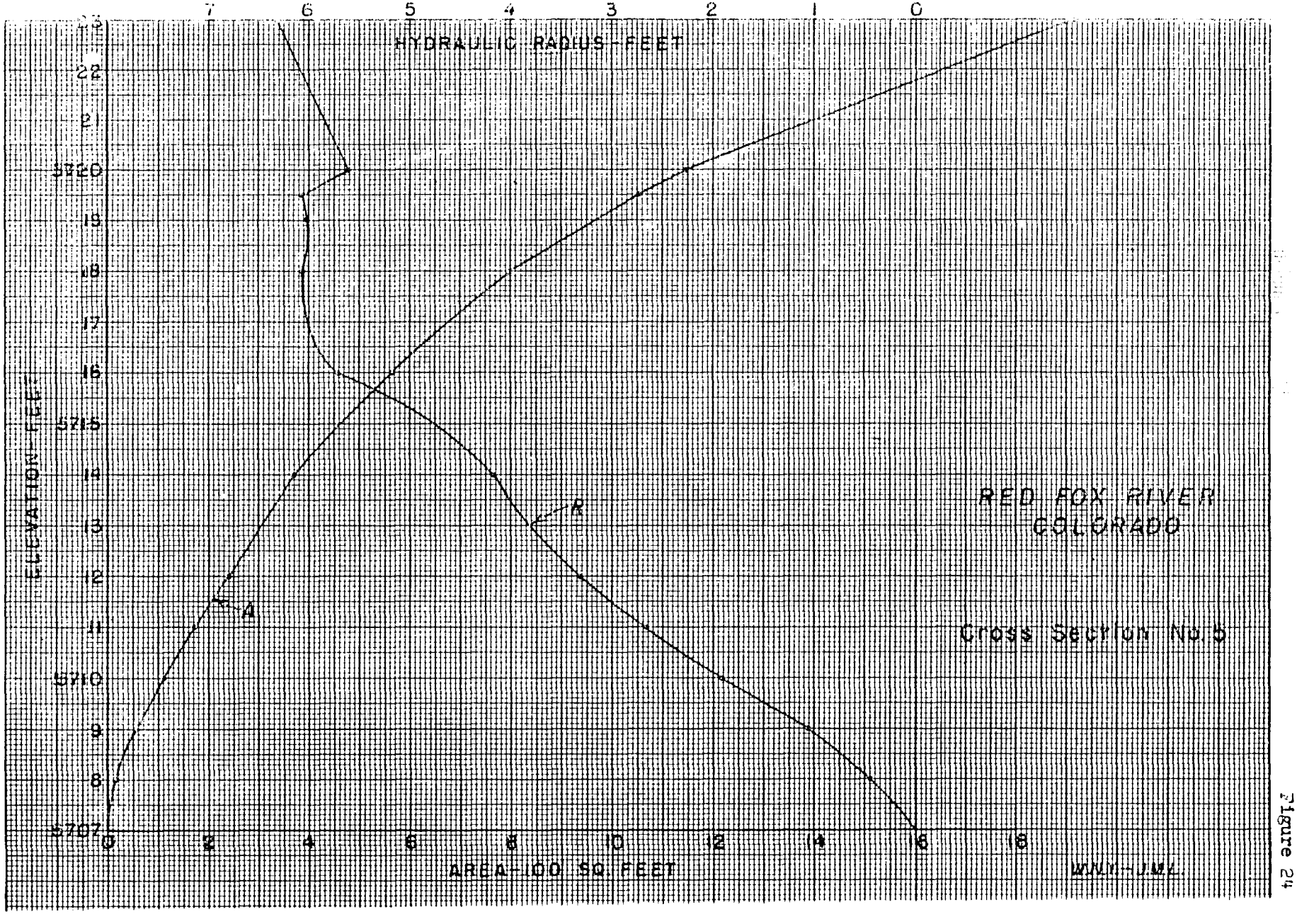

HYDRAULIC RADIUS-FEET
7
6
5
4
3
2
1
0
ELEVATION-FEET
23
22
21
5720
19
18
17
16
5715
14
13
12
11
5710
9
8
5707
A
R
RED FOX RIVER
COLORADO
Cross Section No. 5
0
2
4
6
8
10
12
14
16
18
AREA-100 SQ. FEET
WWH-JML

Figure 24

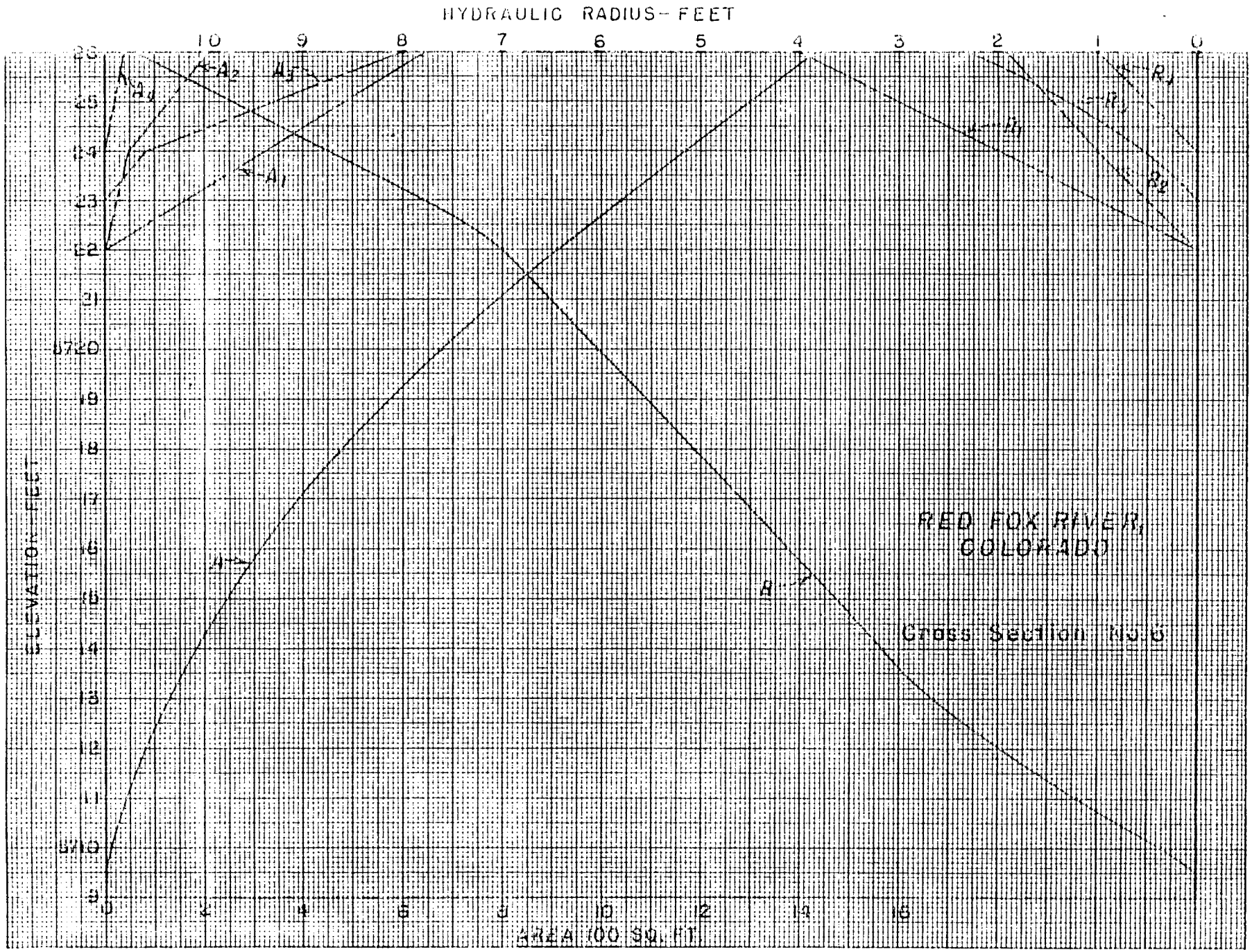

HYDRAULIC RADIUS—FEET
10
9
8
7
6
5
4
3
2
1
0
ELEVATION—FEET
26
25
24
23
22
21
5720
19
18
17
16
15
14
13
12
11
5710
9
0
2
4
6
8
10
12
14
16
AREA (00 SQ. FT)
A_1
A_2
A_3
A_4
R_1
R_2
R_3
R_4
RED FOX RIVER
COLORADO
Cross Section No. 6

Figure 55

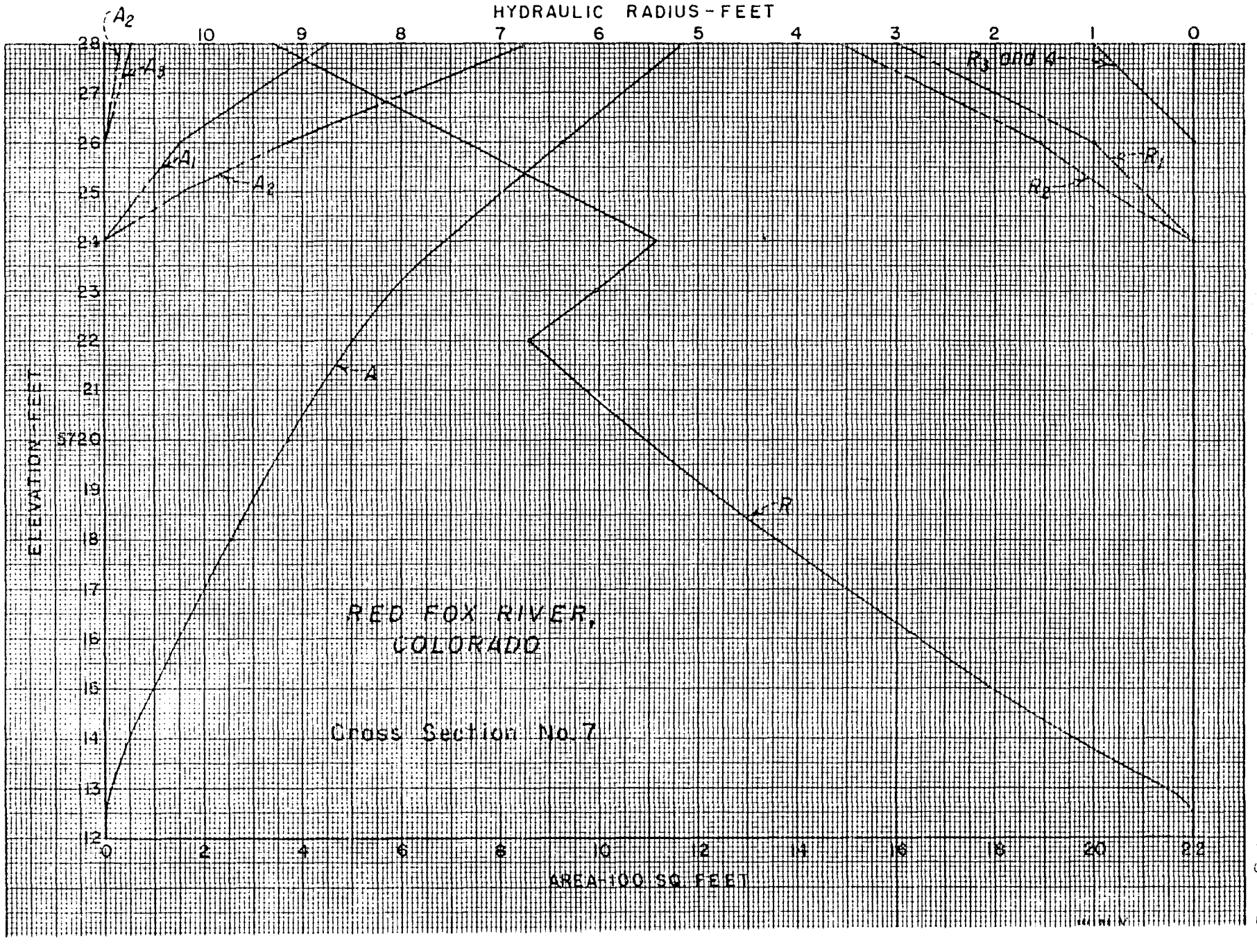

HYDRAULIC RADIUS - FEET
10
9
8
7
6
5
4
3
2
1
0
ELEVATION - FEET
28
27
26
25
24
23
22
21
5720
19
18
17
16
15
14
13
12
0
2
4
6
8
10
12
14
16
18
20
22
AREA-100 SQ. FEET
A_2
A_3
A_1
A_2
A
R
R_3 and A
R_1
R_2
RED FOX RIVER,
COLORADO
Cross Section No. 7

Figure 2

ELEVATION - FEET
STATIONS
LEFT BANK
RIVER BED PROFILE
Section No. 1
Section No. 2
Section No. 3
Section No. 4 (Bridge)
Section No. 5
Section No. 6
Section No. 7 (Axis of Dam)

Figure 41

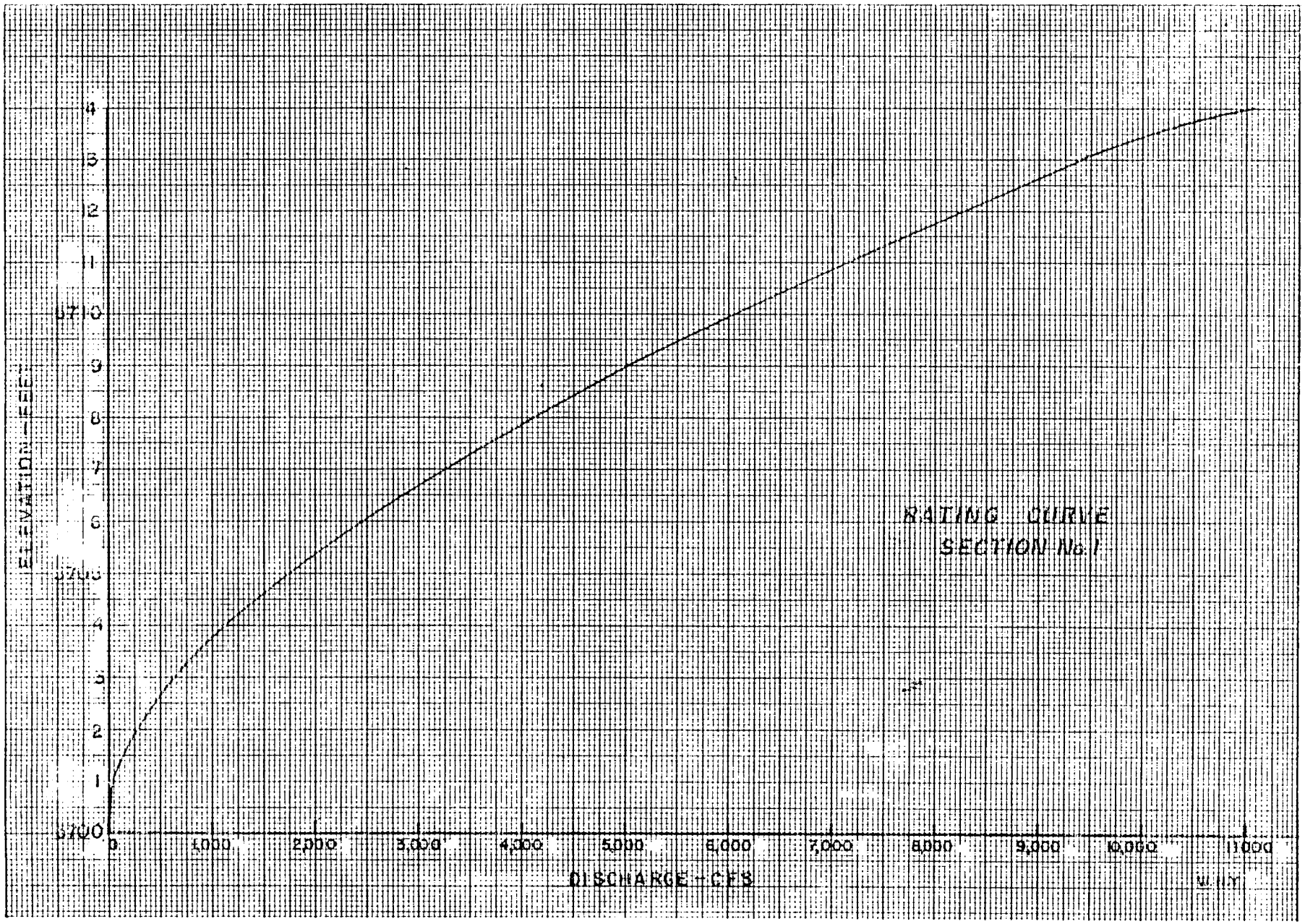
RED FOX RIVER, COLORADO

Rating Curve--Section 1

$S = 0.00735$ $S^{\frac{1}{2}} = .0857$

Elev.	K_d	$S^{\frac{1}{2}}$	Q
5701	440	.0857	38
5702	3,020	.0857	259
5703	7,330	.0857	628
5704	13,200	.0857	1,130
5706	28,500	.0857	2,440
5707	38,000	.0857	3,260
5709	58,500	.0857	5,010
5711	83,000	.0857	7,110
5713	110,000	.0857	9,430
5714	129,150	.0857	11,100

Figure 28


RATING CURVE
SECTION No. 1
ELEVATION - FEET
DISCHARGE - CFS
0
1,000
2,000
3,000
4,000
5,000
6,000
7,000
8,000
9,000
10,000
11000
4
3
2
1
0
9
8
7
6
5
4
3
2
1
0

RED FOX RIVER, COLORADO

Rating Curve--Section 1

$S = 0.00735$ $S^{\frac{1}{2}} = .0857$

Elev.	K_d	$S^{\frac{1}{2}}$	Q
5701	440	.0857	38
5702	3,020	.0857	259
5703	7,330	.0857	628
5704	13,200	.0857	1,130
5706	28,500	.0857	2,440
5707	38,000	.0857	3,260
5709	58,500	.0857	5,010
5711	83,000	.0857	7,110
5713	110,000	.0857	9,430
5714	129,150	.0857	11,100

Figure 28

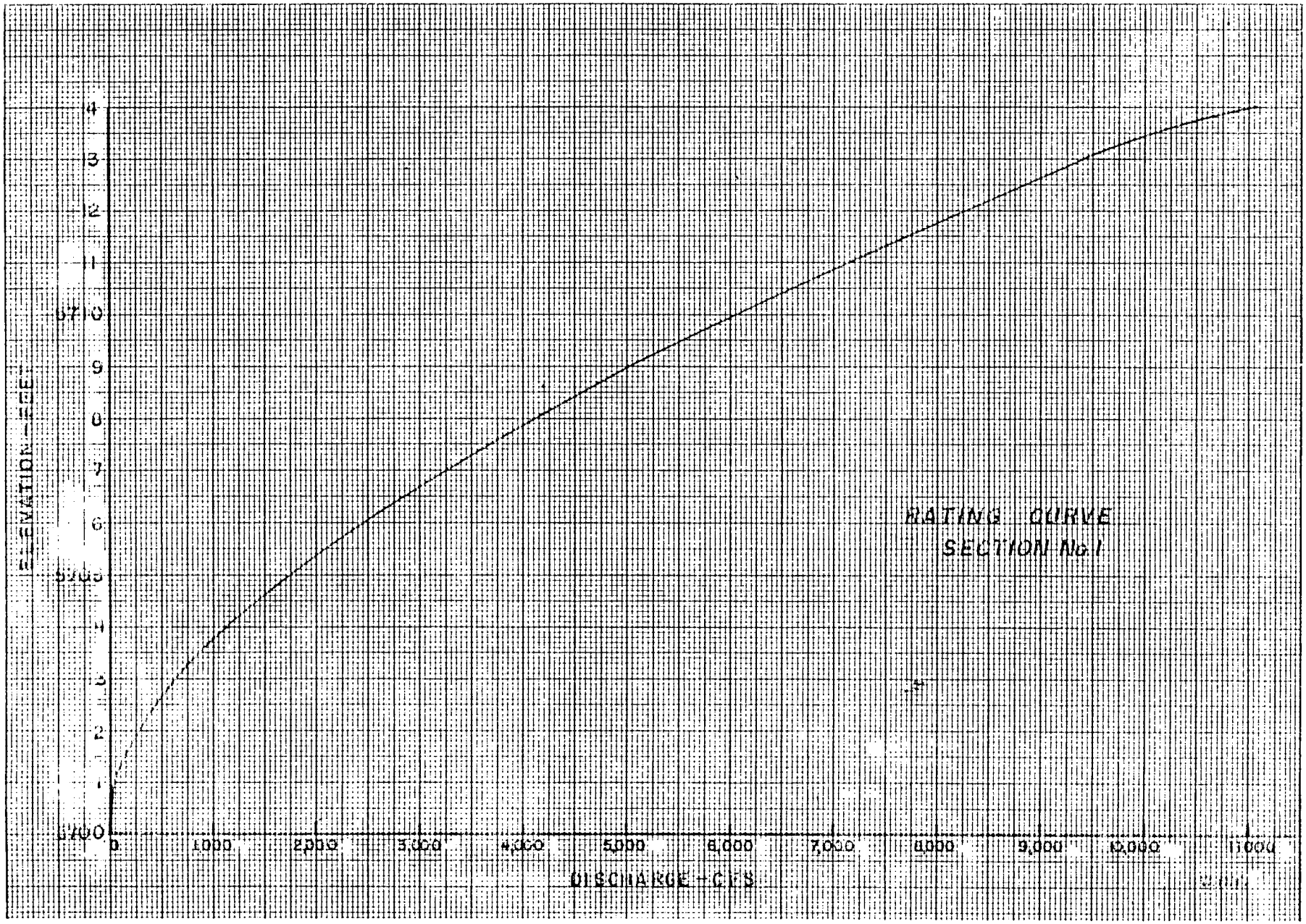

RATING CURVE
SECTION No. 1
ELEVATION - FEET
DISCHARGE - CFS
0
1000
2000
3000
4000
5000
6000
7000
8000
9000
10000
11000

Table 14

WATER SURFACE PROFILE COMPUTATIONS – METHOD A

SUBJECT: Red Fox River, Colorado — Discharges of 100 and 500 cfs

COMPUTED BY: JML DATE _____ CHECKED BY: _____ DATE: _____ SHEET ____ OF ____

1	2	3.	4	5	6	7	8	9	10	11	12	13	14	15	16	17
STATION	Assumed Water Surface Elev.	Area A	Dist. L	$K_d = \frac{1.486}{n} AR^{2/3}$	$S_f = \left(\frac{Q}{K_d}\right)^2$	Mean S_f	Mean S_f x L = h_f	Q	V	V^2Q	h_v	$h_{v_1} - h_{v_2}$	Eddy Loss	Total Loss	Δ H	W.S. El.
For Q = 100	cfs															
Section 1	5701.4	28		1,165	0.00736			100	3.57		0.20					5701.4
			500													
Section 2	*5705.3	26		1,070	.00871	0.00804	4.02	100	3.85		.23	-0.03	0.02	4.04	4.01	5705.41
			400													
Section 3	5708.3	44		2,090	.00228	.00550	2.75	100	2.27		.08	+ .15	.02	2.77	2.92	5708.33
			100													
Section 4	5708.6	45		1,840	.00294	.00261	.26	100	2.22		.08	.00	.00	.26	.26	5708.59
			100													
Section 5	5708.9	49		2,080	.00231	.00272	.27	100	2.04		.06	+ .02	.00	.29	.29	5708.83
			400													
Section 6	*5710.6	30		1,160	.00743	.00487	1.95	100	3.33		.17	- .11	.06	2.01	1.90	5710.78
			500													
Section 7	*5713.7	37		1,520	.00433	.00588	2.94	100	2.70		.11	+ .06	.01	2.95	3.01	5710.79
	*To the nearest 0.10 foot															
For Q = 500	cfs															
Section 1	5702.7	77		5,700	.00770			500	6.49		.65					5702.7
			500													
Section 2	5706.7	73		5,400	.00856	.00813	4.07	500	6.85		.73	- .08	.04	4.11	4.03	5706.73
			400													
Section 3	5709.7	101		7,300	.00470	.00663	2.65	500	4.95		.38	+ .35	.04	2.69	3.04	5709.77
			100													
Section 4	5710.3	149		10,000	.00250	.00360	.36	500	3.36		.18	+ .20	.02	.38	.58	5710.35
			100													
Section 5	5710.6	148		11,200	.00199	.00224	.22	500	3.38		.18	.00	.00	.22	.22	5710.57
			400													
Section 6	5712.1	89		6,070	.00676	.00439	1.76	500	5.62		.49	- .31	.16	1.92	1.61	5712.18
			500													
Section 7	5715.2	106		7,400	.00456	.00566	2.83	500	4.72		.34	+ .15	.02	2.85	3.00	5715.18

WATER SURFACE PROFILE COMPUTATIONS – METHOD A

SUBJECT: Red Fox River Discharges of 1,000 and 5,000 cfs

COMPUTED BY: _____ DATE _____ CHECKED BY: _____ DATE: _____ SHEET ____ OF ____

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
STATION	Assumed Water Surface Elev.	Area A	Dist. L	$K_d = \frac{1.486}{n} AR^{2/3}$	$S_f = \left(\frac{Q}{K_d}\right)^2$	Mean S_f	Mean S_f x L = h_f	Q	V	V^2Q	h_v	$h_{v_1} - h_{v_2}$	Eddy Loss	Total Loss	Δ H	W.S. El.
For Q =	1,000	cfs														
Section 1	5703.8	122		11,900	0.00705				8.20		1.04					5703.8
			500													
Section 2	5707.7	114		10,500	.00905	0.00805	4.02	1,000	8.77		1.19	-0.15	0.08	4.10	3.95	5707.75
			400													
Section 3	5711.2	195		16,100	.00386	.00645	2.58	1,000	5.13		0.41	+0.78	.08	2.66	3.44	5711.19
			100													
Section 4	5711.7	250		20,500	.00238	.00312	0.31	1,000	4.00		0.25	+0.16	.02	0.33	0.49	5711.68
			100													
Section 5	5711.9	233		21,500	.00212	.00225	0.23	1,000	4.29		0.28	-0.03	.00	0.23	0.20	5711.88
			400													
Section 6	5713.3	144		12,900	.00600	.00413	1.65	1,000	6.94		0.75	-0.47	.24	1.89	1.42	5713.30
			500													
Section 7	5716.3	163		13,900	.00518	.00559	2.80	1,000	6.14		0.59	+0.16	.02	2.82	2.98	5716.28
For Q =	5,000	cfs														
Section 1	5709.0	355		59,000	.00720			5,000	14.08		3.08					5709.0
			500													
Section 2	5713.2	383		66,000				4,520	11.80	630,000						
		142		7,000				480	3.38	5,480						
				73,000	.00469	.00595	2.98	5,000		635,480	1.97	+1.11	.11	3.09	4.20	5713.20
			400													
Section 3	5716.0	795		117,000	.00183	.00326	1.30	5,000	6.29		0.61	+1.36	.14	1.44	2.80	5716.00
			100													
Section 4	5716.0	626		69,000	.00525	.00354	0.35	5,000	7.98		0.99	-0.38	.03	0.38	0.00	5716.00
			100													
Section 5	5716.4	608		82,000	.00372	.00449	0.45	5,000	8.22		1.05	-0.06	.03	0.48	0.42	5716.42
			400													
Section 6	5718.2	497		63,000	.00630	.00501	2.00	5,000	10.05		1.57	-0.52	.26	2.26	1.74	5718.16
			500													
Section 7	5721.1	440		64,000	.00610	.00620	3.10	5,000	11.36		2.00	-0.43	.22	3.32	2.89	5721.05

WATER SURFACE PROFILE COMPUTATIONS – METHOD A

SUBJECT: Red Fox River, Colorado Discharge = 11,100 cfs

COMPUTED BY: _____ DATE _____ CHECKED BY: _____ DATE: _____ SHEET ____ OF ____

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
STATION	Assumed Water Surface Elev.	Area A	Dist. L	$K_d = \frac{1.486}{n} AR^{2/3}$	$S_f = \left(\frac{Q}{K_d}\right)^2$	Mean S_f	Mean S_f x L = h_f	Q	V	V^2Q	h_v	$h_{v_1} - h_{v_2}$	Eddy Loss	Total Loss	Δ H	W.S. El.
Section 1	5714.0	594		125,000				10,700	18.0	3,460,000						
		192		4,150				400	2.08	1,730						
				129,150	0.00735			11,100		3,461,730	4.85					5714.0
			500													
Section 2	5720.9	863		230,000				5,960	6.80	276,000						
		1,170		130,000				3,370	2.88	28,000						
		1,000		25,000				650	0.65	274						
		1,500		43,000				1,120	0.75	630						
				428,000	0.000673	0.00401	2.00	11,100		304,904	0.43	+4.42	0.44	2.44	6.86	5720.86
			400													
Section 3	5721.2	1,832		360,000				11,000	6.00	396,000						
		115		2,700				70	0.61	26						
		185		1,250				30	0.16	8						
				363,950	0.00108	0.000877	0.35	11,100		396,034	0.56	-0.13	0.07	0.42	0.29	5721.15
			100													
Section 4	5721.2	1,300		158,000	0.00493	0.00301	0.30	11,100	8.54		1.13	-0.57	0.27	0.57	0.00	5721.20
			100													
Section 5	5722.1	1,681		220,000	0.00254	0.00374	0.37	11,100	6.60		0.68	+0.45	0.05	0.42	0.87	5722.07
			400													
Section 6	5722.8	1,018		174,000				10,900	10.7	1,250,000						
		130		3,380				190	1.46	405						
		20		161				10	0.50	3						
				177,541	0.00391	0.00328	1.31	11,100		1,250,408	1.75	-1.07	0.54	1.85	0.78	5722.85
			500													
Section 7	5725.3	844		124,000				10,500	12.5	1,640,000						
		98		1,400				110	1.12	138						
		220		5,800				490	2.23	2,440						
				131,200	0.00714	0.00553	2.77	11,100		1,642,578	2.30	-0.55	0.28	3.05	2.50	5725.35

Figure 29

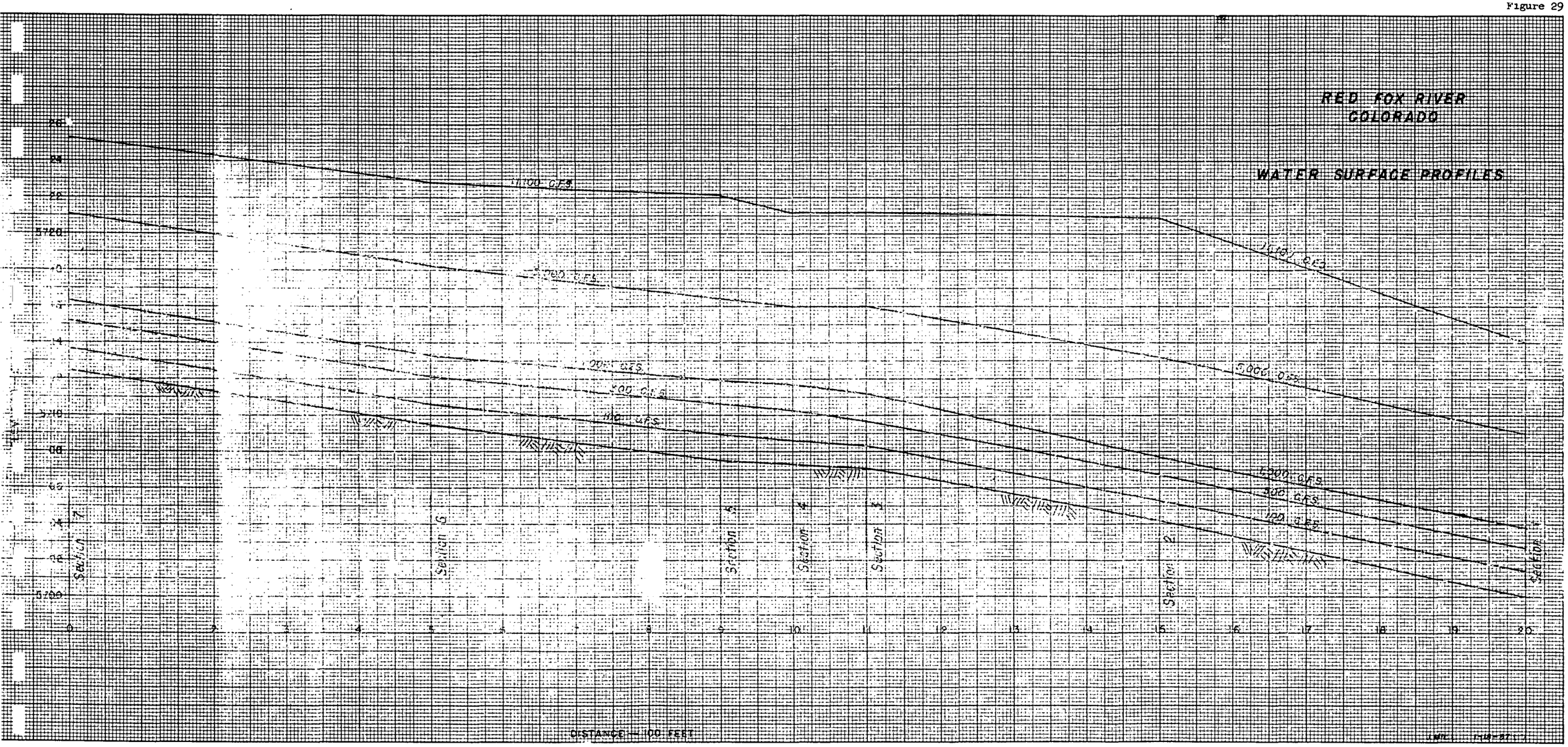
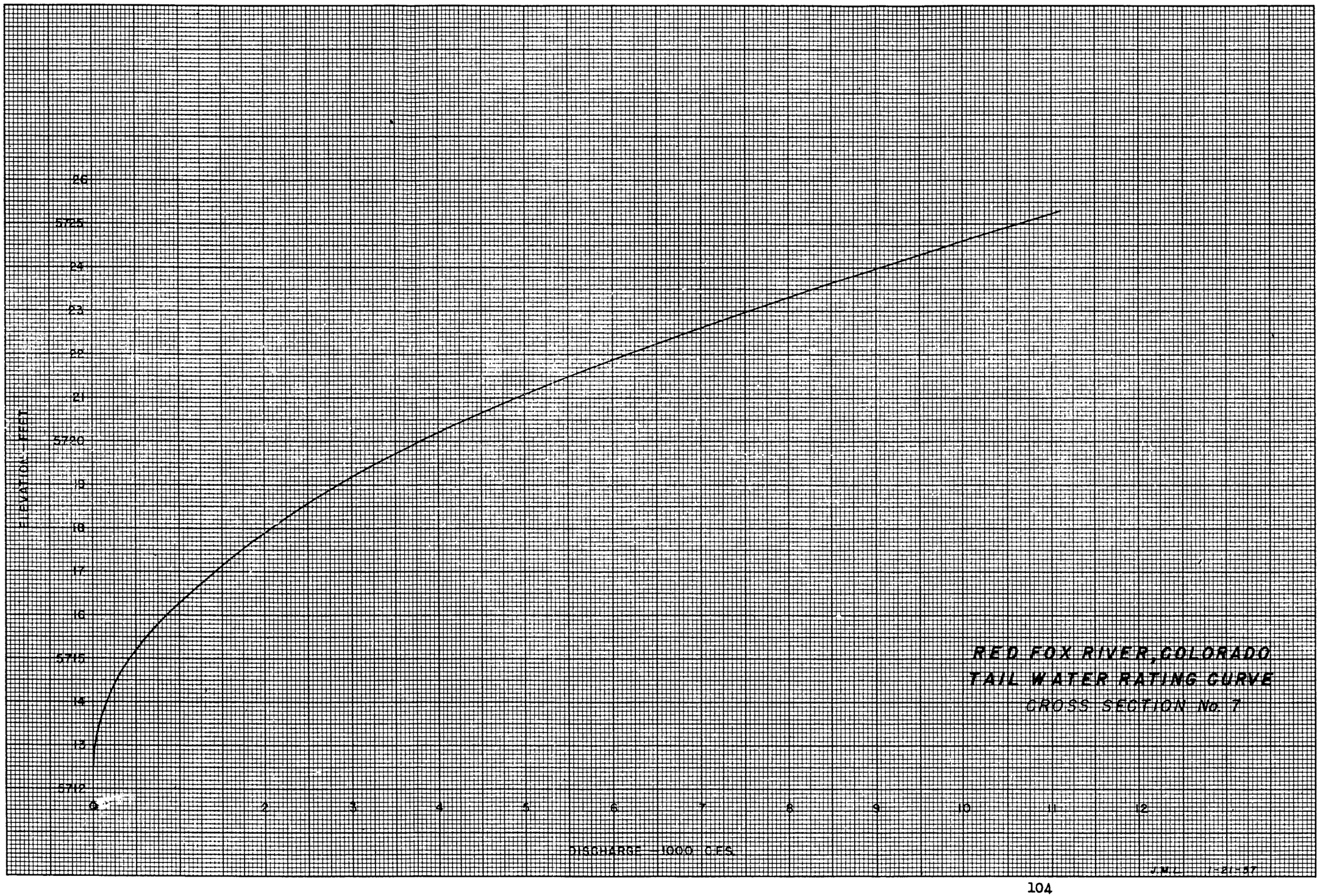

RED FOX RIVER
COLORADO
WATER SURFACE PROFILES
11,100 CFS
5,000 CFS
500 CFS
100 CFS
Section 7
Section 6
Section 5
Section 4
Section 3
Section 2
Section 1
DISTANCE — 100 FEET

Figure 30


RED FOX RIVER, COLORADO
TAIL WATER RATING CURVE
CROSS SECTION No. 7
ELEVATION - FEET
26
5725
24
23
22
21
5720
19
18
17
16
5715
14
13
5712
0
2
3
4
5
6
7
8
9
10
11
12
DISCHARGE - 1000 C.F.S.
J.M.L.
1-21-57

Method I

This method employs a similar analysis as Method A. The balance of energy by Bernoulli's Theorem is achieved in the same manner. There is a slight difference between this method and Method A in deriving the friction slope.

In the following step computations using this method, the same data as used in Method A are employed--Red Fox River for a discharge of 11,100 cfs. Only the area and hydraulic radius curves are needed as shown in Figures 20 to 26 under the description of Method A. The example covers one discharge only. The column numbers refer to the columns of Table 18.

Step 1

Enter the information at the top of the form (Table 18) relative to the name and location of the stream and the discharge being computed. In the example, these are Red Fox River, Colorado, and discharge of 11,100 cfs.

Step 2

Under Column 1, the designation of the beginning station is entered. This has been classified as Section 1 for the example.

Step 3

The water surface elevation for Section 1 will not have to be assumed (Column 2) since it has already been established (see Figure 28). Assumptions will be necessary for the other sections as the computations proceed upstream. For example, at Section 2 the initial water surface elevation was assumed as 5718.0 and found to be wrong. The elevation should have been assumed at least 5718.93

as will be discussed in the next paragraph. The next assumption of 5720.9 proved to be correct.

At the beginning of the computations, the novice often makes four to five assumptions before the energy balance is reached. The following guide is offered as a suggestion to the beginner. From Bernoulli's Theorem, it is evident that the elevation at Section 2 should be at least 5718.93 (Column 2 plus Column 15 for Section 1). An estimate of the friction head loss (Column 11) can be made and added to the elevation 5718.93. In the example, Section 2 is an expanding section which would have a smaller friction slope than Section 1 (Column 9), therefore, it can be estimated as 0.00090 for Section 2. The friction head loss (Column 11) is then computed to be $\frac{0.00090 + 0.00735}{2} = 0.00412$ (average slope--Column 10) x 500 = 2.06 feet. This is added to 5718.93, giving the trial elevation of 5720.99 for Section 2. The same approach may be used for Section 3 and so forth upstream. With more practice and experience, the beginner will reduce these assumptions to one or two trials.

Step 4

Areas for the corresponding elevations are entered in Column 3. Where the cross section has been subdivided as shown in Sections 1 and 2, the appropriate hydraulic elements corresponding to each subdivision are entered. Section 1 shows a main channel and one overbank area while Section 2 has three separate overbank areas and the main channel (see Figures 6 and 7).

Step 5

Under Column 4, the length between sections is noted. See Table 5.

Step 6

The hydraulic radii are listed under Column 5. (Read from Figures 20 and 21 for Sections 1 and 2, respectively). The two-thirds power of the hydraulic radii are entered in Column 6. These can be read from Table 17. The entry in Column 6 is optional, as will be explained later under Step 8.

Step 7

Under Column 7, Manning's roughness coefficient, "n", is entered. Note that each subdivision has its own particular "n" value.

Step 8

The purpose of Column 8 is twofold. First, it is conveniently used to determine the friction slope, S_f , in Column 9. Secondly, it provides the data needed in weighting the velocity head, h_v , Column 15. Two optional methods can be used to determine the velocity, V , in Column 8. The first involves a computational process using Manning's formula thus:

$$V = \frac{1.466}{n} R^{2/3} S^{1/2}$$

For a given "n" and S = 0.0001, the elements $1.486/n \times S^{1/2}$ reduce to a constant which is multiplied by $R^{2/3}$ (Column 6) to give the velocity. The second method is a handbook approach. From

the Hydraulic Tables [17] prepared by the Corps of Engineers, the velocity can be determined knowing R and "n". Column 6 could be omitted if this method is used.

Of the two procedures described above, the first has probably the most practical advantage. This is particularly true when an odd "n" value, say 0.037, has been determined for the section. The Hydraulic Tables quoted above do not include velocity tabulations for these odd "n" coefficients. It is the only method to use, of course, when such tables are not available. The engineer, therefore, may select the method he feels will facilitate and expedite the computations.

The discharge, Q , in Column 8 is determined by multiplying the velocities (computed above) by the corresponding area in Column 3.

Step 9

The friction slope, S_f , shown in Column 9, is calculated from the formula

$$S_f = \left(\frac{0.01\ Q}{Q_{.01}} \right)^2$$

where Q is the total discharge and $Q_{.01}$ is the total of the discharges from Column 8. For example, at Section 2

$$S_f = \left[\frac{(0.01)(11,100)}{4287} \right]^2 = 0.00067$$

The above formula results from the relationship in Manning's equation that each discharge varies directly with the half power of its friction slope.

Step 10

Column 10 shows the average friction slope for the reach between sections. This value is multiplied by the distance, L , (Column 4) to compute the friction head, h_f , in Column 11.

Step 11

The subdivisional discharges under Column 13 are determined by the proportionality as follows: Using the data of Section 2 where the "n" value is 0.03 (main channel)

$$Q = \frac{(11,100)(2296)}{4287}$$

$$Q = 5944$$

The remainder of the subdivisional discharges is similarly computed and all must total 11,100 cfs. Slight adjustments, however, may be necessary.

Step 12

The subdivisional velocities in Column 12 are computed from the equation $V = Q/A$. For example, the main channel velocity of Section 2 is determined thus:

$$V = \frac{5944}{863}$$

$$V = 6.89$$

Step 13

Subdivisional V^2Q's in Column 14 are computed by squaring Column 12 and multiplying by Column 13.

Step 14

Column 15 shows the weighted velocity heads computed by dividing the total of Column 14 by the total of Column 13 multiplied

by 2g or 64.4, better illustrated as follows:

$$\text{Column 15} = \frac{\Sigma \text{ Column 14}}{(\Sigma \text{ Column 13})(64.4)}$$

Step 15

The change in velocity heads $h_{v_1} - h_{v_2}$ is recorded in Column 16. It is important to note that positive or negative signs are inserted. The algebraic sign of this value is retained in the final summation of the losses as will be described later. As a matter of convenience, it is repeated that h_{v_1} and h_{v_2} are the downstream and upstream velocity heads, respectively (see Figure 3).

Step 16

Under Column 17, eddy losses are recorded. These losses are computed by taking an assumed percentage of the change in velocity heads (Column 16). In this example, 10 and 50 percent were used for converging and diverging sections, respectively. Corrections such as 0.04 for Section 5 could be neglected since it is beyond the usual practical limits. It is included in the computations merely to illustrate the application of an eddy loss.

Step 17

The total loss Column 18 is the total of Columns 11 and 17. Column 19 (ΔH) equals the algebraic sum of 16 and 18.

Step 18

Column 20 shows the computed water surface elevation which is determined by adding Column 19 to the preceding water surface elevation (Column 20 above). When the elevations in Columns 2 and 20 agree, usually to the nearest 0.1 foot, the energy balance has been attained.

HYDRAULIC TABLES

Two-thirds powers of numbers

Number	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
0.0	0.000	0.046	0.074	0.097	0.117	0.136	0.153	0.170	0.186	0.201
0.1	.215	.229	.243	.256	.269	.282	.295	.307	.319	.331
0.2	.342	.353	.364	.375	.386	.397	.407	.418	.428	.438
0.3	.448	.458	.468	.477	.487	.497	.506	.515	.525	.534
0.4	.543	.552	.561	.570	.578	.587	.596	.604	.613	.622
0.5	.630	.638	.647	.655	.663	.671	.679	.687	.695	.703
0.6	.711	.719	.727	.735	.743	.750	.758	.765	.773	.781
0.7	.788	.796	.803	.811	.818	.825	.832	.840	.847	.855
0.8	.862	.869	.876	.883	.890	.897	.904	.911	.918	.925
0.9	.932	.939	.946	.953	.960	.966	.973	.980	.987	.993
1.0	1.000	1.007	1.013	1.020	1.027	1.033	1.040	1.046	1.053	1.059
1.1	1.065	1.072	1.078	1.085	1.091	1.097	1.104	1.110	1.117	1.123
1.2	1.129	1.136	1.142	1.148	1.154	1.160	1.167	1.173	1.179	1.185
1.3	1.191	1.197	1.203	1.209	1.215	1.221	1.227	1.233	1.239	1.245
1.4	1.251	1.257	1.263	1.269	1.275	1.281	1.287	1.293	1.299	1.305
1.5	1.310	1.316	1.322	1.328	1.334	1.339	1.345	1.351	1.357	1.362
1.6	1.368	1.374	1.379	1.385	1.391	1.396	1.402	1.408	1.413	1.419
1.7	1.424	1.430	1.436	1.441	1.447	1.452	1.458	1.463	1.469	1.474
1.8	1.480	1.485	1.491	1.496	1.502	1.507	1.513	1.518	1.523	1.529
1.9	1.534	1.539	1.545	1.550	1.556	1.561	1.566	1.571	1.577	1.582
2.0	1.587	1.593	1.598	1.603	1.608	1.613	1.619	1.624	1.629	1.634
2.1	1.639	1.645	1.650	1.655	1.660	1.665	1.671	1.676	1.681	1.686
2.2	1.691	1.697	1.702	1.707	1.712	1.717	1.722	1.727	1.732	1.737
2.3	1.742	1.747	1.752	1.757	1.762	1.767	1.772	1.777	1.782	1.787
2.4	1.792	1.797	1.802	1.807	1.812	1.817	1.822	1.827	1.832	1.837
2.5	1.842	1.847	1.852	1.857	1.862	1.867	1.871	1.876	1.881	1.886
2.6	1.891	1.896	1.900	1.905	1.910	1.915	1.920	1.925	1.929	1.934
2.7	1.939	1.944	1.949	1.953	1.958	1.963	1.968	1.972	1.977	1.982
2.8	1.987	1.992	1.996	2.001	2.006	2.010	2.015	2.020	2.024	2.029
2.9	2.034	2.038	2.043	2.048	2.052	2.057	2.062	2.066	2.071	2.075
3.0	2.080	2.085	2.089	2.094	2.099	2.103	2.108	2.112	2.117	2.122
3.1	2.126	2.131	2.135	2.140	2.144	2.149	2.153	2.158	2.163	2.167
3.2	2.172	2.176	2.180	2.185	2.190	2.194	2.199	2.203	2.208	2.212
3.3	2.217	2.221	2.226	2.230	2.234	2.239	2.243	2.248	2.252	2.257
3.4	2.261	2.265	2.270	2.274	2.279	2.283	2.288	2.292	2.296	2.301
3.5	2.305	2.310	2.314	2.318	2.323	2.327	2.331	2.336	2.340	2.345
3.6	2.349	2.353	2.358	2.362	2.366	2.371	2.375	2.379	2.384	2.388
3.7	2.392	2.397	2.401	2.405	2.409	2.414	2.418	2.422	2.427	2.431
3.8	2.435	2.439	2.444	2.448	2.452	2.457	2.461	2.465	2.469	2.474
3.9	2.478	2.482	2.486	2.490	2.495	2.499	2.503	2.507	2.511	2.516
4.0	2.520	2.524	2.528	2.532	2.537	2.541	2.545	2.549	2.553	2.558
4.1	2.562	2.566	2.570	2.574	2.579	2.583	2.587	2.591	2.595	2.599
4.2	2.603	2.607	2.611	2.616	2.620	2.624	2.628	2.632	2.636	2.640
4.3	2.644	2.648	2.653	2.657	2.661	2.665	2.669	2.673	2.677	2.681
4.4	2.685	2.689	2.693	2.698	2.702	2.706	2.710	2.714	2.718	2.722
4.5	2.726	2.730	2.734	2.738	2.742	2.746	2.750	2.754	2.758	2.762
4.6	2.766	2.770	2.774	2.778	2.782	2.786	2.790	2.794	2.798	2.802
4.7	2.806	2.810	2.814	2.818	2.822	2.826	2.830	2.834	2.838	2.842
4.8	2.846	2.850	2.854	2.858	2.862	2.865	2.869	2.873	2.877	2.881
4.9	2.885	2.889	2.893	2.897	2.901	2.904	2.908	2.912	2.916	2.920

HYDRAULIC TABLES

Two-thirds powers of numbers—Continued

Number	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
10.0	4.642	4.645	4.648	4.651	4.654	4.657	4.660	4.663	4.666	4.669
10.1	4.672	4.676	4.679	4.682	4.685	4.688	4.691	4.694	4.697	4.700
10.2	4.703	4.706	4.709	4.712	4.716	4.719	4.722	4.725	4.728	4.731
10.3	4.734	4.737	4.740	4.743	4.746	4.749	4.752	4.755	4.758	4.762
10.4	4.765	4.768	4.771	4.774	4.777	4.780	4.783	4.786	4.789	4.792
10.5	4.795	4.798	4.801	4.804	4.807	4.810	4.813	4.816	4.819	4.822
10.6	4.825	4.828	4.832	4.835	4.838	4.841	4.844	4.847	4.850	4.853
10.7	4.856	4.859	4.862	4.865	4.868	4.871	4.874	4.877	4.880	4.883
10.8	4.886	4.889	4.892	4.895	4.898	4.901	4.904	4.907	4.910	4.913
10.9	4.916	4.919	4.922	4.925	4.928	4.931	4.934	4.937	4.940	4.943
11.0	4.946	4.949	4.952	4.955	4.958	4.961	4.964	4.967	4.970	4.973
11.1	4.976	4.979	4.982	4.985	4.988	4.991	4.994	4.997	5.000	5.003
11.2	5.006	5.009	5.012	5.015	5.018	5.021	5.024	5.027	5.030	5.033
11.3	5.036	5.039	5.042	5.044	5.047	5.050	5.053	5.056	5.059	5.062
11.4	5.065	5.068	5.071	5.074	5.077	5.080	5.083	5.086	5.089	5.092
11.5	5.095	5.098	5.101	5.104	5.107	5.110	5.113	5.115	5.118	5.121
11.6	5.124	5.127	5.130	5.133	5.136	5.139	5.142	5.145	5.148	5.151
11.7	5.154	5.157	5.160	5.163	5.166	5.168	5.171	5.174	5.177	5.180
11.8	5.183	5.186	5.189	5.192	5.195	5.198	5.201	5.204	5.207	5.209
11.9	5.212	5.215	5.218	5.221	5.224	5.227	5.230	5.233	5.236	5.238
12.0	5.241	5.244	5.247	5.250	5.253	5.256	5.259	5.262	5.265	5.268
12.1	5.271	5.273	5.276	5.279	5.282	5.285	5.288	5.291	5.294	5.297
12.2	5.300	5.302	5.305	5.308	5.311	5.314	5.317	5.320	5.323	5.326
12.3	5.329	5.331	5.334	5.337	5.340	5.343	5.346	5.349	5.352	5.354
12.4	5.357	5.360	5.363	5.366	5.369	5.372	5.375	5.377	5.380	5.383
12.5	5.386	5.389	5.392	5.395	5.398	5.400	5.403	5.406	5.409	5.412
12.6	5.415	5.418	5.421	5.423	5.426	5.429	5.432	5.435	5.438	5.440
12.7	5.443	5.446	5.449	5.452	5.455	5.458	5.461	5.463	5.466	5.469
12.8	5.472	5.475	5.478	5.480	5.483	5.486	5.489	5.492	5.495	5.498
12.9	5.500	5.503	5.506	5.509	5.512	5.515	5.517	5.520	5.523	5.526
13.0	5.529	5.532	5.534	5.537	5.540	5.543	5.546	5.549	5.551	5.554
13.1	5.557	5.560	5.563	5.566	5.568	5.571	5.574	5.577	5.580	5.582
13.2	5.585	5.588	5.591	5.594	5.597	5.600	5.602	5.605	5.608	5.611
13.3	5.614	5.616	5.619	5.622	5.625	5.628	5.630	5.633	5.636	5.639
13.4	5.642	5.644	5.647	5.650	5.653	5.656	5.658	5.661	5.664	5.667
13.5	5.670	5.672	5.675	5.678	5.681	5.684	5.686	5.689	5.692	5.695
13.6	5.698	5.700	5.703	5.706	5.709	5.712	5.714	5.717	5.720	5.723
13.7	5.725	5.728	5.731	5.734	5.737	5.739	5.742	5.745	5.748	5.751
13.8	5.753	5.756	5.759	5.762	5.765	5.767	5.770	5.773	5.776	5.778
13.9	5.781	5.784	5.787	5.790	5.792	5.795	5.798	5.801	5.803	5.806
14.0	5.809	5.812	5.814	5.817	5.820	5.823	5.825	5.828	5.831	5.834
14.1	5.836	5.839	5.842	5.845	5.847	5.850	5.853	5.856	5.859	5.861
14.2	5.864	5.867	5.870	5.872	5.875	5.878	5.881	5.883	5.886	5.889
14.3	5.892	5.894	5.897	5.900	5.902	5.905	5.908	5.911	5.913	5.916
14.4	5.919	5.922	5.924	5.927	5.930	5.933	5.935	5.938	5.941	5.944
14.5	5.946	5.949	5.952	5.955	5.957	5.960	5.963	5.965	5.968	5.971
14.6	5.974	5.976	5.979	5.982	5.985	5.987	5.990	5.993	5.995	5.998
14.7	6.001	6.004	6.006	6.009	6.012	6.014	6.017	6.020	6.023	6.025
14.8	6.028	6.031	6.034	6.036	6.039	6.042	6.044	6.047	6.050	6.052
14.9	6.055	6.058	6.061	6.063	6.066	6.069	6.071	6.074	6.077	6.080

HYDRAULIC TABLES

Two-thirds powers of numbers—Continued

Number	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
15.0	6.082	6.085	6.088	6.090	6.093	6.096	6.098	6.101	6.104	6.106
15.1	6.109	6.112	6.115	6.117	6.120	6.123	6.125	6.128	6.131	6.133
15.2	6.136	6.139	6.141	6.144	6.147	6.150	6.152	6.155	6.158	6.160
15.3	6.163	6.166	6.168	6.171	6.174	6.176	6.179	6.182	6.184	6.187
15.4	6.190	6.193	6.195	6.198	6.200	6.203	6.206	6.208	6.211	6.214
15.5	6.217	6.219	6.222	6.225	6.227	6.230	6.233	6.235	6.238	6.241
15.6	6.243	6.246	6.249	6.251	6.254	6.257	6.259	6.262	6.265	6.267
15.7	6.270	6.273	6.275	6.278	6.281	6.283	6.286	6.289	6.291	6.294
15.8	6.297	6.299	6.302	6.305	6.307	6.310	6.312	6.315	6.318	6.320
15.9	6.323	6.326	6.328	6.331	6.334	6.336	6.339	6.342	6.344	6.347
16.0	6.350	6.352	6.355	6.358	6.360	6.363	6.365	6.368	6.371	6.373
16.1	6.376	6.379	6.381	6.384	6.387	6.389	6.392	6.394	6.397	6.400
16.2	6.402	6.405	6.408	6.410	6.413	6.416	6.418	6.421	6.423	6.426
16.3	6.429	6.431	6.434	6.437	6.439	6.442	6.444	6.447	6.450	6.452
16.4	6.455	6.458	6.460	6.463	6.465	6.468	6.471	6.473	6.476	6.479
16.5	6.481	6.484	6.486	6.489	6.492	6.494	6.497	6.500	6.502	6.505
16.6	6.507	6.510	6.513	6.515	6.518	6.520	6.523	6.526	6.528	6.531
16.7	6.533	6.536	6.539	6.541	6.544	6.546	6.549	6.552	6.554	6.557
16.8	6.560	6.562	6.565	6.567	6.570	6.573	6.575	6.578	6.580	6.583
16.9	6.586	6.588	6.591	6.593	6.596	6.599	6.601	6.604	6.606	6.609
17.0	6.611	6.614	6.617	6.619	6.622	6.624	6.627	6.630	6.632	6.635
17.1	6.637	6.640	6.643	6.645	6.648	6.650	6.653	6.655	6.658	6.661
17.2	6.663	6.666	6.668	6.671	6.673	6.676	6.679	6.681	6.684	6.686
17.3	6.689	6.692	6.694	6.697	6.699	6.702	6.705	6.707	6.710	6.712
17.4	6.715	6.717	6.720	6.723	6.725	6.728	6.730	6.733	6.735	6.738
17.5	6.740	6.743	6.746	6.748	6.751	6.753	6.756	6.758	6.761	6.764
17.6	6.766	6.769	6.771	6.774	6.776	6.779	6.782	6.784	6.787	6.789
17.7	6.792	6.794	6.797	6.799	6.802	6.805	6.807	6.810	6.812	6.815
17.8	6.817	6.820	6.822	6.825	6.828	6.830	6.833	6.835	6.838	6.840
17.9	6.843	6.845	6.848	6.850	6.853	6.856	6.858	6.861	6.863	6.866
18.0	6.868	6.871	6.873	6.876	6.878	6.881	6.884	6.886	6.889	6.891
18.1	6.894	6.896	6.899	6.901	6.904	6.906	6.909	6.911	6.914	6.917
18.2	6.919	6.922	6.924	6.927	6.929	6.932	6.934	6.937	6.939	6.942
18.3	6.944	6.947	6.949	6.952	6.954	6.957	6.960	6.962	6.965	6.967
18.4	6.970	6.972	6.975	6.980	6.982	6.985	6.987	6.990	6.992	6.995
18.5	6.995	6.997	7.000	7.002	7.005	7.007	7.010	7.013	7.015	7.018
18.6	7.020	7.023	7.025	7.028	7.030	7.033	7.035	7.038	7.040	7.043
18.7	7.045	7.048	7.050	7.053	7.055	7.058	7.060	7.063	7.065	7.068
18.8	7.070	7.073	7.075	7.078	7.080	7.083	7.085	7.088	7.090	7.093
18.9	7.095	7.098	7.100	7.103	7.105	7.108	7.110	7.113	7.115	7.118
19.0	7.120	7.123	7.125	7.128	7.130	7.133	7.135	7.138	7.140	7.143
19.1	7.145	7.148	7.150	7.153	7.155	7.158	7.160	7.163	7.165	7.168
19.2	7.170	7.173	7.175	7.178	7.180	7.183	7.185	7.188	7.190	7.193
19.3	7.195	7.198	7.200	7.203	7.205	7.208	7.210	7.212	7.215	7.217
19.4	7.220	7.222	7.225	7.227	7.230	7.232	7.235	7.237	7.240	7.242
19.5	7.245	7.247	7.250	7.252	7.255	7.257	7.260	7.262	7.265	7.267
19.6	7.270	7.272	7.275	7.277	7.280	7.282	7.284	7.287	7.289	7.292
19.7	7.294	7.297	7.299	7.302	7.304	7.307	7.309	7.311	7.314	7.316
19.8	7.319	7.321	7.323	7.326	7.328	7.331	7.333	7.336	7.338	7.341
19.9	7.343	7.346	7.348	7.351	7.353	7.356	7.358	7.361	7.363	7.366

Table 17 (2 of 2)

HYDRAULIC TABLES

Two-thirds powers of numbers—Continued

Number	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
20.0	7.368	7.371	7.373	7.375	7.378	7.380	7.383	7.385	7.388	7.390
20.1	7.393	7.395	7.397	7.400	7.402	7.405	7.407	7.410	7.412	7.415
20.2	7.417	7.420	7.422	7.424	7.427	7.429	7.432	7.434	7.437	7.439
20.3	7.442	7.444	7.446	7.449	7.451	7.454	7.456	7.459	7.461	7.464
20.4	7.466	7.468	7.471	7.473	7.476	7.478	7.481	7.483	7.485	7.488
20.5	7.490	7.493	7.495	7.498	7.500	7.503	7.505	7.507	7.510	7.512
20.6	7.515	7.517	7.520	7.522	7.524	7.527	7.529	7.532	7.534	7.537
20.7	7.539	7.541	7.544	7.546	7.549	7.551	7.554	7.556	7.558	7.561
20.8	7.563	7.566	7.568	7.571	7.573	7.575	7.578	7.580	7.583	7.585
20.9	7.587	7.590	7.592	7.595	7.597	7.600	7.602	7.604	7.607	7.609
21.0	7.612	7.614	7.617	7.619	7.621	7.624	7.626	7.629	7.631	7.633
21.1	7.636	7.638	7.641	7.643	7.645	7.648	7.650	7.653	7.655	7.658
21.2	7.660	7.662	7.665	7.667	7.670	7.672	7.674	7.677	7.679	7.682
21.3	7.684	7.686	7.689	7.691	7.694	7.696	7.698	7.701	7.703	7.706
21.4	7.708	7.710	7.713	7.715	7.718	7.720	7.722	7.725	7.727	7.730
21.5	7.732	7.734	7.737	7.739	7.742	7.744	7.746	7.749	7.751	7.754
21.6	7.756	7.758	7.761	7.763	7.766	7.768	7.770	7.773	7.775	7.778
21.7	7.780	7.782	7.785	7.787	7.790	7.792	7.794	7.797	7.799	7.801
21.8	7.804	7.806	7.808	7.811	7.813	7.816	7.818	7.820	7.823	7.825
21.9	7.828	7.830	7.832	7.835	7.837	7.840	7.842	7.844	7.847	7.849
22.0	7.851	7.854	7.856	7.859	7.861	7.863	7.866	7.868	7.870	7.873
22.1	7.875	7.878	7.880	7.882	7.885	7.887	7.889	7.892	7.894	7.897
22.2	7.899	7.901	7.904	7.906	7.908	7.911	7.913	7.916	7.918	7.920
22.3	7.923	7.925	7.927	7.930	7.932	7.934	7.937	7.939	7.942	7.944
22.4	7.946	7.949	7.951	7.953	7.956	7.958	7.960	7.963	7.965	7.968
22.5	7.970	7.972	7.975	7.977	7.979	7.982	7.984	7.986	7.989	7.991
22.6	7.994	7.996	7.998	8.001	8.003	8.005	8.008	8.010	8.012	8.015
22.7	8.017	8.019	8.022	8.024	8.027	8.029	8.031	8.034	8.036	8.038
22.8	8.041	8.043	8.045	8.048	8.050	8.052	8.055	8.057	8.059	8.062
22.9	8.064	8.066	8.069	8.071	8.074	8.076	8.078	8.081	8.083	8.085
23.0	8.088	8.090	8.092	8.095	8.097	8.099	8.102	8.104	8.106	8.109
23.1	8.111	8.113	8.116	8.118	8.120	8.123	8.125	8.127	8.130	8.132
23.2	8.134	8.137	8.139	8.141	8.144	8.146	8.148	8.151	8.153	8.155
23.3	8.158	8.160	8.162	8.165	8.167	8.169	8.172	8.174	8.176	8.179
23.4	8.181	8.183	8.186	8.188	8.190	8.193	8.195	8.197	8.200	8.202
23.5	8.204	8.207	8.209	8.211	8.214	8.216	8.218	8.221	8.223	8.225
23.6	8.228	8.230	8.232	8.235	8.237	8.239	8.242	8.244	8.246	8.249
23.7	8.251	8.253	8.255	8.258	8.260	8.262	8.265	8.267	8.269	8.272
23.8	8.274	8.276	8.279	8.281	8.283	8.286	8.288	8.290	8.293	8.295
23.9	8.297	8.300	8.302	8.304	8.306	8.309	8.311	8.313	8.316	8.318
24.0	8.320	8.323	8.325	8.327	8.330	8.332	8.334	8.337	8.339	8.341
24.1	8.343	8.346	8.348	8.350	8.353	8.355	8.357	8.360	8.362	8.364
24.2	8.366	8.369	8.371	8.374	8.376	8.378	8.380	8.383	8.385	8.387
24.3	8.390	8.392	8.394	8.396	8.399	8.401	8.403	8.406	8.408	8.410
24.4	8.413	8.415	8.417	8.419	8.422	8.424	8.426	8.428	8.431	8.433
24.5	8.435	8.438	8.440	8.442	8.445	8.447	8.449	8.452	8.454	8.456
24.6	8.458	8.461	8.463	8.465	8.468	8.470	8.472	8.474	8.477	8.479
24.7	8.481	8.484	8.486	8.488	8.490	8.493	8.495	8.497	8.500	8.502
24.8	8.504	8.507	8.509	8.511	8.513	8.516	8.518	8.520	8.522	8.525
24.9	8.527	8.529	8.532	8.534	8.536	8.538	8.541	8.543	8.545	8.548

HYDRAULIC TABLES

Two-thirds powers of numbers—Continued

Number	.0	.1	.2	.3	.4	.5	.6	.7	.8	.9
30	9.655	9.676	9.698	9.719	9.741	9.762	9.783	9.805	9.826	9.847
31	9.868	9.889	9.911	9.932	9.953	9.974	9.995	10.02	10.04	10.06
32	10.08	10.10	10.12	10.14	10.16	10.18	10.20	10.23	10.25	10.27
33	10.29	10.31	10.33	10.35	10.37	10.39	10.41	10.43	10.45	10.47
34	10.50	10.52	10.54	10.56	10.58	10.60	10.62	10.64	10.66	10.68
35	10.70	10.72	10.74	10.76	10.78	10.80	10.82	10.84	10.86	10.88
36	10.90	10.92	10.94	10.96	10.98	11.00	11.02	11.04	11.06	11.08
37	11.10	11.12	11.14	11.16	11.18	11.20	11.22	11.24	11.26	11.28
38	11.30	11.32	11.34	11.36	11.38	11.40	11.42	11.44	11.46	11.48
39	11.50	11.52	11.54	11.56	11.58	11.60	11.62	11.64	11.66	11.68
40	11.70	11.72	11.74	11.75	11.77	11.79	11.81	11.83	11.85	11.87
41	11.89	11.91	11.93	11.95	11.97	11.99	12.01	12.03	12.04	12.06
42	12.08	12.10	12.12	12.14	12.16	12.18	12.20	12.22	12.24	12.25
43	12.27	12.29	12.31	12.33	12.35	12.37	12.39	12.41	12.43	12.44
44	12.46	12.48	12.50	12.52	12.54	12.56	12.58	12.60	12.61	12.63
45	12.65	12.67	12.69	12.71	12.73	12.75	12.76	12.78	12.80	12.82
46	12.84	12.86	12.88	12.89	12.91	12.93	12.95	12.97	12.99	13.01
47	13.02	13.04	13.06	13.08	13.10	13.12	13.13	13.15	13.17	13.19
48	13.21	13.23	13.24	13.26	13.28	13.30	13.32	13.34	13.35	13.37
49	13.39	13.41	13.43	13.45	13.46	13.48	13.50	13.52	13.54	13.55
50	13.57	13.59	13.61	13.63	13.64	13.66	13.68	13.70	13.72	13.73
51	13.75	13.77	13.79	13.81	13.82	13.84	13.86	13.88	13.90	13.91
52	13.93	13.95	13.97	13.99	14.00	14.02	14.04	14.06	14.07	14.09
53	14.11	14.13	14.15	14.16	14.18	14.20	14.22	14.23	14.25	14.27
54	14.29	14.30	14.32	14.34	14.36	14.37	14.39	14.41	14.43	14.44
55	14.46	14.48	14.50	14.51	14.53	14.55	14.57	14.58	14.60	14.62
56	14.64	14.65	14.67	14.69	14.71	14.72	14.74	14.76	14.78	14.79
57	14.81	14.83	14.85	14.86	14.88	14.90	14.91	14.93	14.95	14.97
58	14.98	15.00	15.02	15.04	15.05	15.07	15.09	15.10	15.12	15.14
59	15.16	15.17	15.19	15.21	15.22	15.24	15.26	15.28	15.29	15.31
60	15.33	15.34	15.36	15.38	15.39	15.41	15.43	15.45	15.46	15.48
61	15.50	15.51	15.53	15.55	15.56	15.58	15.60	15.61	15.63	15.65
62	15.66	15.68	15.70	15.72	15.73	15.75	15.77	15.78	15.80	15.82
63	15.83	15.85	15.87	15.88	15.90	15.92	15.93	15.95	15.97	15.98
64	16.00	16.02	16.03	16.05	16.07	16.08	16.10	16.12	16.13	16.15
65	16.17	16.18	16.20	16.22	16.23	16.25	16.27	16.28	16.30	16.32
66	16.33	16.35	16.36	16.38	16.40	16.41	16.43	16.45	16.46	16.48
67	16.50	16.51	16.53	16.55	16.56	16.58	16.59	16.61	16.63	16.64
68	16.66	16.68	16.69	16.71	16.73	16.74	16.76	16.77	16.79	16.81
69	16.82	16.84	16.86	16.87	16.89	16.90	16.92	16.94	16.95	16.97
70	16.98	17.00	17.02	17.03	17.05	17.07	17.08	17.10	17.11	17.13
71	17.15	17.16	17.18	17.19	17.21	17.23	17.24	17.26	17.27	17.29
72	17.31	17.32	17.34	17.36	17.37	17.39	17.40	17.42	17.43	17.45
73	17.47	17.48	17.50	17.51	17.53	17.55	17.56	17.58	17.59	17.61
74	17.63	17.64	17.66	17.67	17.69	17.71	17.72	17.74	17.75	17.77
75	17.78	17.80	17.82	17.83	17.85	17.86	17.88	17.89	17.91	17.93
76	17.94	17.96	17.97	17.99	18.01	18.02	18.04	18.05	18.07	18.08
77	18.10	18.11	18.13	18.15	18.16	18.18	18.19	18.21	18.22	18.24
78	18.26	18.27	18.29	18.30	18.32	18.33	18.35	18.36	18.38	18.40
79	18.41	18.43	18.44	18.46	18.47	18.49	18.50	18.52	18.54	18.55

HYDRAULIC TABLES

Two-thirds powers of numbers—Continued

Number	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
25.0	8.550	8.552	8.554	8.557	8.559	8.561	8.564	8.566	8.568	8.570
25.1	8.573	8.575	8.577	8.580	8.582	8.584	8.586	8.589	8.591	8.593
25.2	8.595	8.598	8.600	8.602	8.605	8.607	8.609	8.611	8.614	8.616
25.3	8.618	8.620	8.623	8.625	8.627	8.629	8.632	8.634	8.636	8.639
25.4	8.641	8.643	8.645	8.648	8.650	8.652	8.654	8.657	8.659	8.661
25.5	8.664	8.666	8.668	8.670	8.673	8.675	8.677	8.679	8.682	8.684
25.6	8.686	8.688	8.691	8.693	8.695	8.697	8.700	8.702	8.704	8.706
25.7	8.709	8.711	8.713	8.716	8.718	8.720	8.722	8.725	8.727	8.729
25.8	8.731	8.734	8.736	8.738	8.740	8.743	8.745	8.747	8.749	8.752
25.9	8.754	8.756	8.758	8.761	8.763	8.765	8.767	8.770	8.772	8.774
26.0	8.776	8.779	8.781	8.783	8.785	8.788	8.790	8.792	8.794	8.797
26.1	8.799	8.801	8.803	8.806	8.808	8.810	8.812	8.815	8.817	8.819
26.2	8.821	8.824	8.826	8.828	8.830	8.833	8.835	8.837	8.839	8.842
26.3	8.844	8.846	8.848	8.851	8.853	8.855	8.857	8.859	8.862	8.864
26.4	8.866	8.868	8.871	8.873	8.875	8.877	8.880	8.882	8.884	8.886
26.5	8.889	8.891	8.893	8.895	8.897	8.900	8.902	8.904	8.906	8.909
26.6	8.911	8.913	8.915	8.918	8.920	8.922	8.924	8.927	8.929	8.931
26.7	8.933	8.935	8.938	8.940	8.942	8.944	8.947	8.949	8.951	8.953
26.8	8.956	8.958	8.960	8.962	8.964	8.967	8.969	8.971	8.973	8.976
26.9	8.978	8.980	8.982	8.984	8.987	8.989	8.991	8.993	8.996	8.998
27.0	9.000	9.002	9.004	9.007	9.009	9.011	9.013	9.016	9.018	9.020
27.1	9.022	9.024	9.027	9.029	9.031	9.033	9.036	9.038	9.040	9.042
27.2	9.044	9.047	9.049	9.051	9.053	9.055	9.058	9.060	9.062	9.064
27.3	9.067	9.069	9.071	9.073	9.075	9.078	9.080	9.082	9.084	9.086
27.4	9.089	9.091	9.093	9.095	9.098	9.100	9.102	9.104	9.106	9.109
27.5	9.111	9.113	9.115	9.117	9.120	9.122	9.124	9.126	9.128	9.131
27.6	9.133	9.135	9.137	9.139	9.142	9.144	9.146	9.148	9.150	9.153
27.7	9.155	9.157	9.159	9.162	9.164	9.166	9.168	9.170	9.173	9.175
27.8	9.177	9.179	9.181	9.184	9.186	9.188	9.190	9.192	9.195	9.197
27.9	9.199	9.201	9.203	9.206	9.208	9.210	9.212	9.214	9.216	9.219
28.0	9.221	9.223	9.225	9.227	9.230	9.232	9.234	9.236	9.238	9.241
28.1	9.243	9.245	9.247	9.249	9.252	9.254	9.256	9.258	9.260	9.263
28.2	9.265	9.267	9.269	9.271	9.274	9.276	9.278	9.280	9.282	9.284
28.3	9.287	9.289	9.291	9.293	9.295	9.298	9.300	9.302	9.304	9.306
28.4	9.308	9.311	9.313	9.315	9.317	9.319	9.322	9.324	9.326	9.328
28.5	9.330	9.333	9.335	9.337	9.339	9.341	9.343	9.346	9.348	9.350
28.6	9.352	9.354	9.356	9.359	9.361	9.363	9.365	9.367	9.370	9.372
28.7	9.374	9.376	9.378	9.380	9.383	9.385	9.387	9.389	9.391	9.393
28.8	9.396	9.398	9.400	9.402	9.404	9.407	9.409	9.411	9.413	9.415
28.9	9.417	9.420	9.422	9.424	9.426	9.428	9.430	9.433	9.435	9.437
29.0	9.439	9.441	9.443	9.446	9.448	9.450	9.452	9.454	9.456	9.459
29.1	9.461	9.463	9.465	9.467	9.469	9.472	9.474	9.476	9.478	9.480
29.2	9.482	9.485	9.487	9.489	9.491	9.493	9.495	9.498	9.500	9.502
29.3	9.504	9.506	9.508	9.511	9.513	9.515	9.517	9.519	9.521	9.524
29.4	9.526	9.528	9.530	9.532	9.534	9.537	9.539	9.541	9.543	9.545
29.5	9.547	9.549	9.552	9.554	9.556	9.558	9.560	9.562	9.565	9.567
29.6	9.569	9.571	9.573	9.575	9.577	9.580	9.582	9.584	9.586	9.588
29.7	9.590	9.593	9.595	9.597	9.599	9.601	9.603	9.605	9.608	9.610
29.8	9.612	9.614	9.616	9.618	9.621	9.623	9.625	9.627	9.629	9.631
29.9	9.633	9.636	9.638	9.640	9.642	9.644	9.646	9.648	9.651	9.653

HYDRAULIC TABLES

Two-thirds powers of numbers—Continued

Number	.0	.1	.2	.3	.4	.5	.6	.7	.8	.9
80	18.57	18.58	18.60	18.61	18.63	18.64	18.66	18.67	18.69	18.71
81	18.72	18.74	18.75	18.77	18.78	18.80	18.81	18.83	18.84	18.86
82	18.87	18.89	18.91	18.92	18.94	18.95	18.97	18.98	19.00	19.01
83	19.03	19.04	19.06	19.07	19.09	19.10	19.12	19.13	19.15	19.16
84	19.18	19.20	19.21	19.23	19.24	19.26	19.27	19.29	19.30	19.32
85	19.33	19.35	19.36	19.38	19.39	19.41	19.42	19.44	19.45	19.47
86	19.48	19.50	19.51	19.53	19.54	19.56	19.57	19.59	19.60	19.62
87	19.63	19.65	19.66	19.68	19.69	19.71	19.72	19.74	19.75	19.77
88	19.78	19.80	19.81	19.83	19.84	19.86	19.87	19.89	19.90	19.92
89	19.93	19.95	19.96	19.98	19.99	20.01	20.02	20.04	20.05	20.07
90	20.08	20.10	20.11	20.13	20.14	20.16	20.17	20.19	20.20	20.22
91	20.23	20.25	20.26	20.28	20.29	20.31	20.32	20.34	20.35	20.36
92	20.38	20.39	20.41	20.42	20.44	20.45	20.47	20.48	20.50	20.51
93	20.53	20.54	20.56	20.57	20.59	20.60	20.62	20.63	20.64	20.66
94	20.67	20.69	20.70	20.72	20.73	20.75	20.76	20.78	20.79	20.81
95	20.82	20.83	20.85	20.86	20.88	20.89	20.91	20.92	20.94	20.95
96	20.97	20.98	21.00	21.01	21.02	21.04	21.05	21.07	21.08	21.10
97	21.11	21.13	21.14	21.15	21.17	21.18	21.20	21.21	21.23	21.24
98	21.26	21.27	21.29	21.30	21.31	21.33	21.34	21.36	21.37	21.39
99	21.40	21.41	21.43	21.44	21.46	21.47	21.49	21.50	21.52	21.53

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Bureau of Reclamation

Table 18

WATER SURFACE PROFILE COMPUTATIONS METHOD I

SUBJECT: Red Fox River, Colorado, Discharge 11,100 cfs

COMPUTED BY: W.N.Y. DATE: CHECKED BY: JML DATE: SHEET 1 OF 1

1	2	3	4	5	6	7	8		9	10	11	12	13	14	15	16	17	18	19	20
STATION	Assumed Water Surface Elev.	Area A	Dist. L	R	$R^{2/3}$	n	$S_f^{1/2} = 0.01$ V	Q	S_f	Mean S_f	Mean S_f x L = h_f	V	Q	V^2Q	h_v	$h_{v_1} - h_{v_2}$	Eddy Loss	Total Loss	Δ H	W.S. El.
Section No 1	5714.0	594		8.77	4.25	.03	2.10	1247				18.1	10,743	3,520,000						
		192		0.62	.727	.05	.216	41				1.86	357	1,230						
								1288	.00735				11,100	3,521,230	4.93					5714.00
			500																	
Section No 2	~~5718.0~~	~~674~~		~~9.72~~	~~4.56~~	~~.03~~	~~2.26~~	~~1520~~				~~12.0~~	~~8,070~~	~~1,160,000~~						
		~~710~~		~~4.40~~	~~2.68~~	~~.05~~	~~.795~~	~~565~~				~~4.22~~	~~3,000~~	~~53,400~~						
		~~60~~		~~0.50~~	~~.63~~	~~.10~~	~~.~~	~~6~~				~~.~~	~~30~~	~~7~~						
								~~2041~~	~~.00282~~	~~.00568~~	~~2.54~~		~~11,100~~	~~1,213,407~~	~~1.70~~	~~+3.23~~	~~0.32~~	~~2.86~~	~~6.09~~	~~5720.69~~
	5720.9	863		12.42	5.363	.03	2.66	2296				6.89	5,944	282,174						
		1170		7.32	3.770	.05	1.12	1310				2.90	3,393	28,535						
		1000		2.30	1.742	.10	0.259	259				0.671	671	302						
		1500		2.60	1.891	.10	0.281	422				0.728	1,092	579						
								4287	.000670	.00401	2.00		11,100	311,590	0.44	+4.49	0.45	2.45	6.94	5720.94
			400																	
Section No 3	5721.2	1832		7.67	3.889	.03	1.93	3536				5.96	10,920	387,896						
		115		1.22	1.142	.05	0.339	39				1.05	120	132						
		185		0.60	0.711	.10	0.106	20				0.33	60	6						
								3595	.000953	.000812	0.32		11,100	388,034	0.54	-0.10	0.05	0.37	0.27	5721.21
			100																	
Section No 4	5721.2	1300		4.85	2.865	.035			.00517	.00306	0.31	8.54	11,100		1.13	-0.59	0.28	0.59	0.00	5721.20
			100																	
Section No 5	5722.1	1681		6.11	3.342	.035			.00216	.00366	0.37	6.60	11,100		0.68	+0.45	0.04	0.41	0.86	5722.06
			400																	
Section No 6	5722.8	1018		7.60	3.866	.035	1.641	1670				10.68	10,872	1,240,090						
		130		0.80	0.862	.05	0.256	33				1.67	217	605						
		20		0.40	0.543	.10	0.081	2				0.53	11	3						
								1705	.00424	.00320	1.28		11,100	1,240,698	1.74	-1.06	0.53	1.81	0.75	5722.81
			500																	
Section No 7	5725.3	844		6.69	3.550	.035	1.507	1272				12.28	10,366	1,563,200						
		98		0.65	0.750	.05	0.223	22				1.82	179	593						
		220		1.06	1.040	.05	0.309	68				2.52	555	3,524						
								1362	.00664	.0054.	2.72		11,100	1,567,317	2.19	-0.45	0.22	2.94	2.49	5725.30

Standard Step Method

An illustration of the Standard Step Method is contained in Table 20 (Form HD-60). This method follows practically the same procedure as Method I of the Corps of Engineers. Using the hydraulic data of the Red Fox River, as described under Method A, and a discharge of 500 cfs, the following step procedure is outlined for a normal computation in Table 20. It is noted that only the area and hydraulic radius curves are needed in this method.

Step 1

Enter the information at the top of the form relative to the name and location of the stream and the discharge being computed. In the example, these are Red Fox River, Colorado, and discharge of 500 cfs.

Step 2

Under Column 1, the designation of the beginning station is entered, such as Section 1 in the example.

Step 3

Since the water surface for Section 1 has already been established from Figure 28, no assumption is made under Column 2. The elevations will have to be assumed, however, for the other sections as the computations are carried out for the upstream stations. The elevation of 5706.7 is assumed for Section 2 and entered in Column 2.

Step 4

An area of 73 square feet is entered in Column 3 as read from Figure 20 (under Method A).

Step 5

Under Column 4, the length between sections is noted. See Table 5 (under Method A).

Step 6

The hydraulic radius of 1.82 read from Figure 21 is entered in Column 5. The two-thirds power of the hydraulic radius is entered under Column 6. This is equal to 1.491 taken from Table 17 (under Method I).

Step 7

Manning's roughness coefficient, "n", is entered under Column 7.

Step 8

Column 8 can be omitted since all the flow is contained in the main channel and only one "n" value is applied. S_f in Column 9 is found for Section 2 as follows:

$$S_f = \left(\frac{Qn}{1.486\ AR^{2/3}} \right)^2 = \left(\frac{1}{1.486\ R^{2/3}} \right)^2 \left(\frac{Qn}{A} \right)^2$$

since $$Y = \frac{1}{1.486\ R^{2/3}}$$

$$S_f = Y^2 \left(\frac{Qn}{A} \right)^2 = \left(\frac{YQn}{A} \right)^2$$

Hydraulic data for Section 2

$R = 1.82 \quad A = 73 \quad n = 0.03 \quad Q = 500$

Read $Y = 0.452$ from Table 19

Substituting

$$S_f \left[\frac{(0.452)(500)(0.030)}{73} \right]^2$$

$$= (0.0926)^2 = 0.00857$$

Step 9

Column 10 shows the average friction slope for the reach between sections. This value is multiplied by the distance, L , (Column 4) to compute the friction head, h_f , in Column 11. Thus, in the example .00813 x 500 = 4.06.

Step 10

The velocity of 6.85 is computed by dividing the discharge 500 by the area 73 (Column 3). The discharge is entered in Column 13.

Step 11

Column 14 is omitted since only one velocity exists (main channel). Column 15 contains the velocity head, h_v , computed by squaring the velocity and dividing by twice the gravitational acceleration. In the example, this is computed as follows:

$$(6.85)^2/64.4 = 0.73.$$

Step 12

The change in velocity heads, $h_{v_1} - h_{v_2}$ is recorded in Column 16. It is important to note that positive or negative signs are inserted. The algebraic sign of this value is retained in the final summation of the losses as covered in the ensuing discussion.

Step 13

Under Column 17, eddy losses are recorded. These losses are computed by taking an assumed percentage of the change in velocity

heads (Column 16). In this example, 10 and 50 percent were used for converging and diverging reaches, respectively. Thus, an eddy loss of 0.04 is computed from 0.50 x 0.08. Corrections such as .02 for Sections 4 and 7 could be neglected since they are beyond the usual practical limits and are not realistic. They are included in the computations merely to illustrate the application of an eddy loss.

Step 14

The total loss Column 18 is the sum of Columns 11 and 17. Column 19 (ΔH) equals the algebraic sum of 16 and 18.

Step 15

Column 20 shows the computed water surface elevation which is determined by adding Column 19 to the preceding water surface elevation (Column 20 above) as 4.02 + 5702.7 = 5706.72 in the example. When the elevations in Columns 2 and 20 agree, usually to the nearest 0.1 foot, the energy balance has been reached.

Values of $^1/_{1.486}\ R^{2/3} = Y$

R	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
.1	3.12	2.93	2.77	2.62	2.50	2.38	2.28	2.19	2.11	2.04
.2	1.97	1.91	1.85	1.79	1.74	1.70	1.65	1.61	1.57	1.54
.3	1.50	1.47	1.44	1.41	1.38	1.36	1.33	1.30	1.28	1.26
.4	1.24	1.22	1.20	1.18	1.16	1.14	1.13	1.11	1.10	1.08
.5	1.07	1.05	1.04	1.03	1.01	1.00	.990	.979	.967	.957
.6	.946	.935	.926	.916	.906	.897	.888	.879	.870	.862
.7	.854	.846	.838	.830	.823	.815	.808	.801	.794	.787
.8	.781	.775	.768	.762	.756	.750	.744	.738	.733	.727
.9	.722	.717	.711	.706	.701	.696	.691	.687	.682	.677
1.0	.673	.669	.664	.660	.656	.651	.647	.643	.640	.636
1.1	.632	.628	.624	.620	.616	.613	.610	.606	.602	.599
1.2	.596	.592	.589	.587	.583	.580	.577	.574	.571	.567
1.3	.565	.562	.559	.557	.554	.551	.549	.546	.543	.540
1.4	.538	.535	.533	.530	.528	.525	.522	.521	.519	.516
1.5	.514	.511	.509	.507	.505	.503	.500	.498	.496	.494
1.6	.492	.490	.488	.486	.484	.482	.480	.479	.476	.474
1.7	.472	.470	.469	.467	.465	.464	.462	.460	.458	.456
1.8	.455	.453	.452	.449	.448	.446	.445	.444	.442	.440
1.9	.438	.437	.436	.435	.432	.431	.430	.428	.427	.425
2.0	.424	.423	.421	.420	.418	.417	.416	.415	.414	.411
2.1	.410	.409	.407	.406	.405	.404	.402	.401	.400	.399
2.2	.397	.396	.395	.394	.394	.392	.391	.390	.389	.387
2.3	.386	.385	.383	.383	.382	.381	.379	.378	.378	.377
2.4	.376	.374	.373	.373	.371	.370	.369	.369	.367	.366
2.5	.365	.365	.363	.362	.362	.361	.359	.359	.358	.356
2.6	.356	.355	.354	.354	.352	.352	.351	.349	.349	.348
2.7	.346	.346	.345	.345	.344	.344	.342	.341	.341	.339
2.8	.339	.338	.338	.336	.336	.335	.335	.333	.333	.332
2.9	.330	.330	.329	.329	.329	.327	.327	.326	.326	.324
3.0	.324	.322	.322	.321	.321	.319	.319	.318	.318	.318
3.1	.316	.316	.315	.314	.314	.313	.313	.312	.311	.310
3.2	.310	.309	.309	.308	.307	.307	.306	.305	.305	.304
3.3	.303	.303	.302	.302	.301	.301	.300	.299	.299	.298
3.4	.298	.297	.296	.296	.295	.295	.294	.294	.293	.292
3.5	.292	.291	.291	.290	.290	.289	.289	.288	.288	.287

Table 19 Sheet 1 of 3

Values of $^{1}/_{1.486}\ R^{2/3} = Y$ (cont'd)

R	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
3.6	.287	.286	.285	.285	.284	.284	.283	.283	.282	.282
3.7	.281	.281	.280	.280	.279	.279	.278	.278	.277	.277
3.8	.276	.276	.275	.275	.274	.274	.273	.273	.273	.272
3.9	.272	.271	.271	.270	.270	.269	.269	.268	.268	.268
4.0	.267	.267	.266	.266.	.265	.265	.264	.264	.264	.263
4.1	.263	.262	.262	.261	.261	.261	.260	.260	.259	.259
4.2	.258	.258	.258	.257	.257	.257	.256	.256	.255	.255
4.3	.255	.254	.254	.253	.253	.253	.252	.252	.251	.251
4.4	.251	.250	.250	.249	.249	.249	.248	.248	.248	.247
4.5	.247	.247	.246	.246	.245	.245	.245	.244	.244	.244
4.6	.243	.243	.243	.242	.242	.241	.241	.241	.240	.240
4.7	.240	.240	.239	.239	.239	.238	.238	.237	.237	.237
4.8	.236	.236	.236	.236	.235	.235	.235	.234	.234	.234
4.9	.233	.233	.233	.232	.232	.232	.231	.231	.231	.230
5.0	.230	.230	.230	.229	.229	.229	.228	.228	.228	.227

Values of $^{1}/1.486\ R^{2}/3 = Y$ (cont'd)

R	.0	.1	.2	.3	.4	.5	.6	.7	.8	.9
5	.230	.227	.224	.221	.219	.216	.213	.211	.209	.206
6	.204	.201	.199	.197	.195	.193	.191	.189	.187	.186
7	.184	.182	.181	.179	.177	.175	.174	.173	.171	.170
8	.168	.167	.166	.164	.163	.162	.160	.159	.158	.157
9	.156	.154	.153	.152	.151	.150	.149	.148	.147	.146
10	.145	.144	.143	.142	.141	.140	.139	.139	.138	.137
11	.136	.135	.135	.134	.133	.132	.132	.131	.130	.129
12	.128	.128	.127	.126	.126	.125	.124	.124	.123	.122
13	.122	.121	.120	.120	.119	.119	.118	.117	.117	.117
14	.116	.115	.115	.114	.114	.113	.113	.112	.112	.111
15	.110	.110	.110	.109	.109	.108	.108	.107	.107	.106
16	.106	.105	.105	.105	.104	.104	.103	.103	.102	.102
17	.102	.101	.101	.100	.100	.0998	.0994	.0991	.0987	.0983
18	.0980	.0976	.0973	.0969	.0965	.0962	.0959	.0955	.0952	.0949
19	.0945	.0942	.0939	.0935	.0932	.0929	.0926	.0922	.0919	.0917
20	.0913	.0910	.0907	.0904	.0901	.0898	.0896	.0893	.0890	.0887
21	.0884	.0881	.0879	.0876	.0873	.0871	.0868	.0865	.0863	.0860
22	.0857	.0854	.0852	.0850	.0847	.0844	.0842	.0840	.0837	.0834
23	.0832	.0829	.0827	.0825	.0823	.0820	.0818	.0815	.0813	.0811
24	.0809	.0807	.0804	.0802	.0800	.797	.0796	.0794	.0791	.0789
25	.0787	.0785	.0783	.0781	.0779	.0777	.0775	.0773	.0771	.0769
26	.0767	.0765	.0763	.0761	.0759	.0757	.755	.0753	.0752	.0750
27	.0748	.0746	.0744	.0742	.0740	.0739	.0737	.0735	.0733	.0731
28	.0730	.0728	.0727	.0725	.0723	.0721	.0720	.0718	.0716	.0715
29	.0713	.0711	.0710	.0708	.0706	.0705	.0704	.0702	.0700	.0699
30	.0697	.0696	.0694	.0692	.0691	.0689	.0688	.0686	.0685	.0683

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Bureau of Reclamation

Table 20

WATER SURFACE PROFILE COMPUTATIONS - METHOD I

SUBJECT: Red Fox River, Discharge 500 cfs

COMPUTED BY: _____ DATE: _____ CHECKED BY: _____ DATE: _____ SHEET 1 OF _____

1	2	3	4	5	6	7	8		9	10	11	12	13	14	15	16	17	18	19	20
STATION	Assumed Water Surface Elev.	Area A	Dist. L	R	$R^{2/3}$	n	$S_f^{1/2} = 0.01$ V	Q	S_f	Mean S_f	Mean S_f x L = h_f	V	Q	V^2Q	h_v	$h_{v_1} - h_{v_2}$	Eddy Loss	Total Loss	Δ H	W.S. El.
Section 1	5702.7	77		1.82	1.491	.030			.00769			6.49	500		0.065					5702.7
			500																	
Section 2	5706.7	73		1.82	1.491	.030			.00857	0.00813	4.06	6.85	500		0.73	-0.08	0.04	4.10	4.02	5706.72
			400																	
Section 3	5709.7	101		1.76	1.458	.030			.00469	.00663	2.65	4.95	500		0.38	+0.35	0.04	2.69	3.04	5709.76
			100																	
Section 4	5710.3	149		2.00	1.587	.035			.00248	.00358	0.36	3.36	500		0.18	+0.20	0.02	0.38	0.58	5710.34
			100																	
Section 5	5710.6	148		2.38	1.782	.035			.00199	.00224	0.22	3.38	500		0.18	0.00	0.00	0.22	0.22	5710.56
			400																	
Section 6	5712.1	89		2.04	1.608	.035			.00675	.00437	1.75	5.62	500		0.49	-0.31	0.16	1.91	1.60	5712.16
			500																	
Section 7	5715.1	106		2.13	1.655	.035			.00450	.00562	2.81	4.72	500		0.34	+0.15	0.02	2.83	2.98	5715.14

Leach's Method

An illustration of a computation, using Leach's Method, is shown in Table 21. The profile is shown for a discharge of 11,100 cfs. K_d curves in Figures 13 to 19 and hydraulic data of the Red Fox River, described under Method A, are used. This method involves an application of the Manning formula as follows:

$$Q = \frac{1.486}{n} AR^{2/3} S^{1/2}$$

let $$K_d = \frac{1.486}{n} AR^{2/3}$$

where K_d is the total conveyance capacity of the stream channel.

Then $$Q = K_d S^{1/2}$$

S_f can then be computed by

$$S_f = \left(\frac{Q}{K_d}\right)^2$$

Then $$h_f = L\left(\frac{S_{f_1} + S_{f_2}}{2}\right) \text{ or } h_f = L\left(\frac{Q}{\text{Ave } K_d}\right)^2$$

The Leach equation is thus completed when $Z_2 + d_2 = Z_1 + d_1 + h_f$.

The following step procedure is used to compute the water surface profiles by this method.

Step 1--Station and Elevation

A sample computation is shown in Table 21 on the Red Fox River for a profile of 11,100 cfs. Column 1 is the stationing or the cross-section designation. Column 2 is the water surface elevation.

Section 1 is the farthest downstream section and the starting point of the profile computation. The water surface elevation-discharge relationship has been established for Section 1, as previously described, and is presented in Figure 28 under Method A. From Figure 28, the water surface elevation is 5714.0 for a discharge of 11,100 cfs. The water surface for Section 2 is assumed until a balance is obtained at the end of the computation. The beginner will eventually reduce the number of trial assumptions as he becomes more familiar and experienced with this computational procedure. In the example, 5716.8 has been assumed as the water surface elevation for Section 2.

Step 2--Section K_d

Column 3 is the section K_d. K_d curves are always prepared for each section prior to starting the profile computations and are part of processing the field data as explained in that chapter. Figures 13 to 19 present the K_d curves for the example as computed in Tables 6 to 12. From Figure 13, the total K_d for Section 1 is 125,000 (main channel) + 4,150 (overbank) = 129,150 for an elevation of 5714.0. Similarly, for Section 2 the K_d for elevation 5716.8 is 166,500 as read from the curves in Figure 14.

Step 3--Average K_d

Column 4 is the average K_d. The friction head is computed from the relation

$$h_f = L\left(\frac{Q}{\text{Ave } K_d}\right)^2 .$$

In the example, the average K_d is $\frac{129,150 + 166,500}{2} = 147,825$ for Sections 1 and 2. For clarity in documenting the computations, the average K_d is tabulated on a line between the sections left for that purpose.

Step 4--Length L

Column 5 is the length L between sections. The determination of the distance between sections results from processing the field data. In the example, the length L between Sections 1 and 2 is 500 feet as obtained from Table 5 (under Method A).

Step 5--Computation of S_f (Friction Slope)

Column 6 is S_f, the friction slope, which is determined from $\left(\frac{Q}{K_d}\right)^2$ for any cross section using the conveyance characteristics of that section. The friction slope between two sections is the average S_f of the two sections. However, the average S_f can be computed by using the average conveyance, K_d, of the two sections in the formula $S_f = \left(\frac{Q}{\text{Ave } K_d}\right)^2$. In the example, S_f for the reach bounded by Sections 1 and 2 is $\left(\frac{11,100}{147,825}\right)^2$ and equal to .00564. The average K_d is obtained from Column 4.

Step 6--Computation of h_f (Friction Head)

Column 7 is h_f, the friction head, which is computed from the formula $h_f = LS_f$. In the example, h_f is computed by multiplying the values in Columns 5 and 6. 500 x .00564 = 2.82.

Step 7--Computed Water Surface Elevation at the Upstream Section

Column 8 is the computed water surface elevation. Referring to Leach's formula, $Z_2 + d_2 = Z_1 + d_1 + h_f$, the water surface elevation for Section 2 is the elevation in Column 8 (Section 1) plus h_f in Column 7. Note that at the starting section (Section 1), the elevations in Columns 2 and 8 are the same. In the example, this computation is 5714 + 2.82 = 5716.82. The computation between sections is complete when the computed water surface for Section 2 in Column 8 equals the assumed water surface elevation in Column 2. In general, the balance is considered satisfactory when the difference in the two values does not exceed .1 foot. If the balance is unsatisfactory, then other water surface elevations for Section 2 are assumed until a satisfactory balance is reached. The example presents a complete profile computation through a reach with seven sections. The computations are begun at the next upstream section using the same procedure outlined in the seven steps above.

Method B

Method B requires the preparation of various graphs and tables from the collected field data.

The example presented in this computation is of a fictitious stream, Silver Fox River. A plan map has been prepared as

WATER SURFACE PROFILE COMPUTATIONS — LEACH'S METHOD

SUBJECT: Red Fox River Q = 11,100 cfs

COMPUTED BY: JML CHECKED BY: KBS DATE: SHEET 1 OF 1

1	2	3	4	5	6	7	8
STA.	ELEV.	K_d	AVE. K_d	L	Sf $\left(\frac{Q}{Ave. K_d}\right)^2$	h_f	ELEV.
Sec No. 1	5714.0	129,150					5714.0
			147,825	500	.00564	2.82	
Sec No. 2	5716.8	166,500					5716.82
			180,750	400	.00377	1.51	
Sec No. 3	5718.3	195,000					5718.33
			152,000	100	.00533	.53	
Sec No. 4	5718.8	109,000					5718.86
			137,000	100	.00656	.66	
Sec No. 5	5719.5	165,000					5719.52
			157,500	400	.00497	1.99	
Sec No. 6	5721.5	150,000					5721.51
			139,832	500	.00630	3.15	
Sec No. 7	5724.7	129,664					5724.66

Table 21

shown in Figure 31. Seven cross sections were laid out in the field as indicated on the map. It is noted that two flow lines are shown indicating different lengths between sections. One represents the main channel flow length and the other shows the overbank flow length. These are the lengths used in the computations.

Area and hydraulic radius curves are presented in Figures 32 to 34. These were developed as described in Method A, therefore, no cross-sectional plottings are included. By the same token, K_d curves shown in Figures 35 to 41 and the rating curve at the beginning section, Station 209+40 in Figure 42, were similarly developed.

An application of Method B is shown in the following step computational procedure. Again, as in Method A, only the area and conveyance curves are needed in the computations. The hydraulic radius curves are included for use by Method II which is described next. Only one example of the computations is included to demonstrate the procedure.

Step 1

Referring to Table 22, which is the computational form to be used for this method, enter the pertinent data in the heading. In the example, these are the Silver Fox River water surface profile computations for a discharge of 50,000 cfs.

Step 2

Enter the section number of stationing under Column 1, shown as 209+40 in the example.

Step 3

Under Column 2, the main channel and overbank reach lengths between sections are entered. These are noted as 2,260 and 1,955 feet between Sections 209+40 and 186+80 for the main channel and overbank reaches as taken from Figure 31, which is the plan map.

Step 4

The known water surface elevation of 94.67, from Figure 42 is entered in Column 3 for Section 209+40. Areas from Figure 32 for each subdivision are entered in Column 4.

Step 5

K_d values for the main channel and overbank areas are read from Figure 35. These are equal to 973,000 and 604,000 in the example.

Step 6

Column 6 is determined by dividing the individual values of Column 5 by the square root of Column 2. For example, the main channel value is computed as follows:

$$\frac{K_d}{L^{1/2}} = \frac{973,000}{(2,260)^{1/2}} = 20,500$$

Step 7

The friction head, h_f , in Column 7 is found by squaring the quantity of the total discharge divided by the summation of the values in Column 6, as follows:

$$h_f = \left(Q/\Sigma \frac{K_d}{L^{1/2}} \right)^2 = \left(\frac{50,000}{34,200} \right)^2 = 2.13$$

Step 8

The subdivisional flows listed in Column 8 are computed by multiplying the individual values of Column 6 by the square root of Column 7. For example, the overbank area discharge is computed as follows:

$$13{,}700 \times 2.13^{1/2} = 20{,}000$$

Step 9

Subdivisional velocities in Column 9 are computed by the equation $V = Q/A$. Thus, for the main channel,

$$V = 30{,}000/3{,}390 = 8.85 \text{ feet per second.}$$

Step 10

The values determined by squaring the individual velocities and multiplying by their respective discharges are entered in Column 10. For the overbank area, $V^2Q = (1.72)^2(20{,}000) = 59{,}000$.

Step 11

Column 11 shows the weighted velocity head which is calculated by dividing Column 10 by the product of 50,000 and 64.4(2g). Thus, in the example:

$$h_v = \frac{V^2Q}{Q2g} = \frac{2{,}409{,}000}{50{,}000 \times 64.4} = 0.75$$

Step 12

The remainder of the columns are omitted since Section 209+40 is the beginning station. The next step, therefore, consists of assuming the water surface elevation for the next upstream section, 186+80. This has been assumed as 96.90 in the example. Similar

computations are made through Column 11 as described for Section 209+40. The velocity head, h_v , for Section 186+80 computes to 0.45. The algebraic difference in velocity heads is entered in Column 12; thus, in the example

$$h_{v_1} - h_{v_2} = 0.75 - 0.45 = + 0.30 \quad .$$

Step 13

The reach is diverging between Sections 209+40 and 186+80, therefore, a 10-percent correction is applied to the change in velocity heads to determine the eddy loss in Column 13. In the example, this amounts to 0.1 x 0.30 = 0.03. A convenient rule to follow in the eddy loss determination is to apply a 10-percent correction to positive values in Column 12, and 50 percent to negative values.

Step 14

Column 14 is the mean friction head loss, h_f , computed by averaging the values in Column 9. Thus, in the example:

$$h_f = \frac{2.13 + 1.62}{2} = 1.88$$

Step 15

The total loss in Column 15 is determined by adding Columns 13 and 14.

Step 16

Column 16, ΔH, is the algebraic sum of Columns 12 and 15.

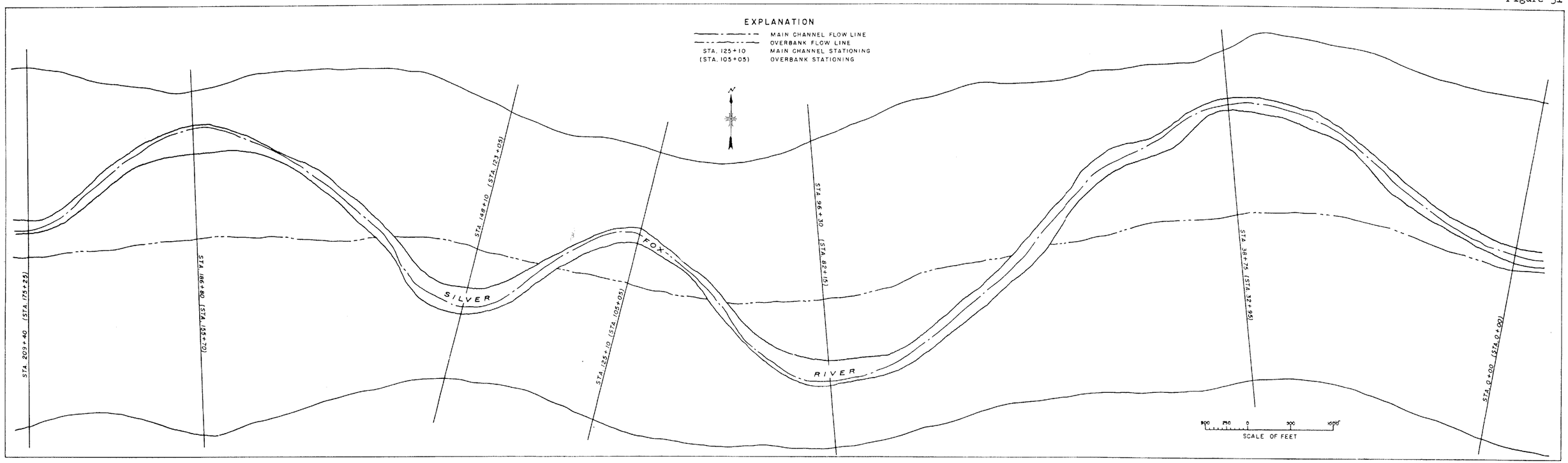
Step 17

The water surface elevation in Column 17 is calculated by adding Column 16 to the preceding water surface elevation of the downstream section. In this example, it is computed as follows: 94.67 + 2.21 = 96.88.

When a balance to the nearest tenth foot is obtained between Columns 17 and 3, the computations are complete.

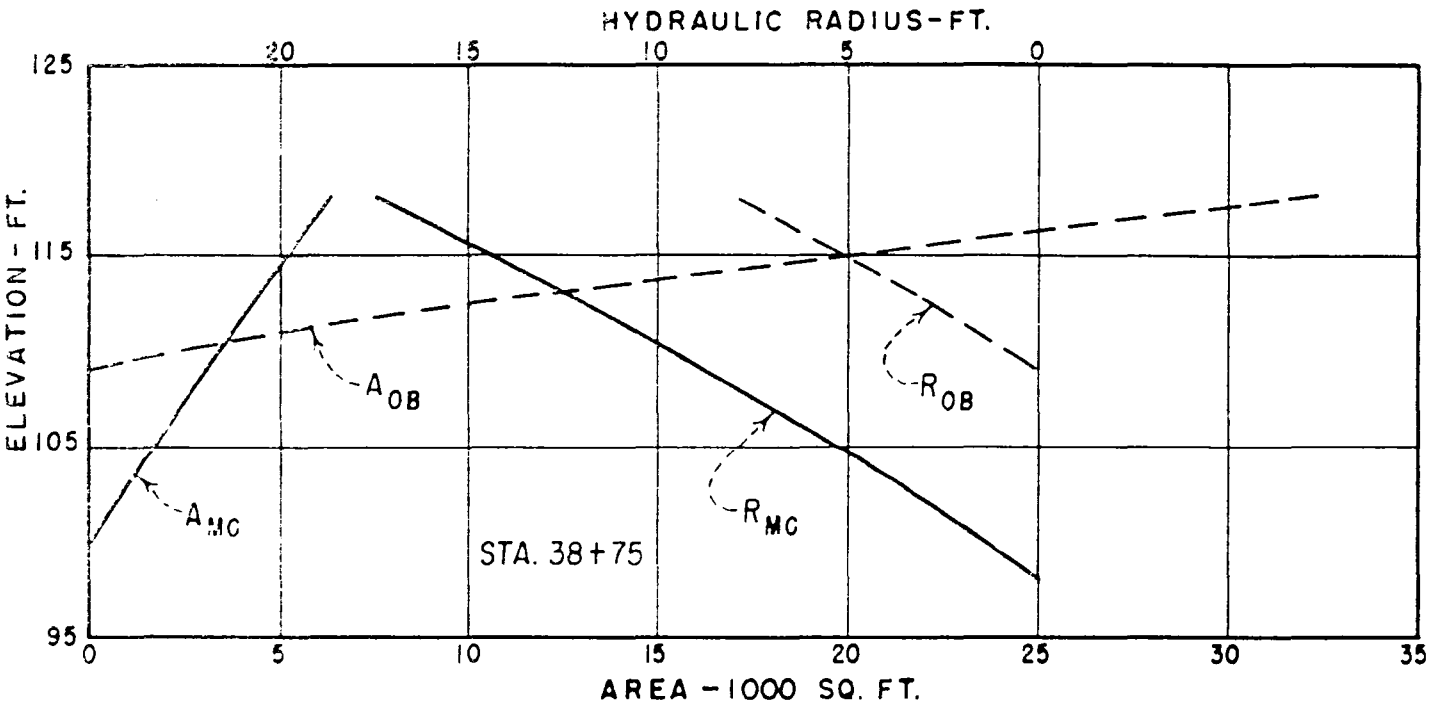
The next step involves a computation of the friction head, h_f , between Sections 186+80 and 148+10 (next upstream section) using the lengths of 3,870 and 3,265 as shown for the main channel and overbank. The water surface elevation at Section 148+10 is then assumed and a new cycle of computations is made as outlined in the above step procedure.

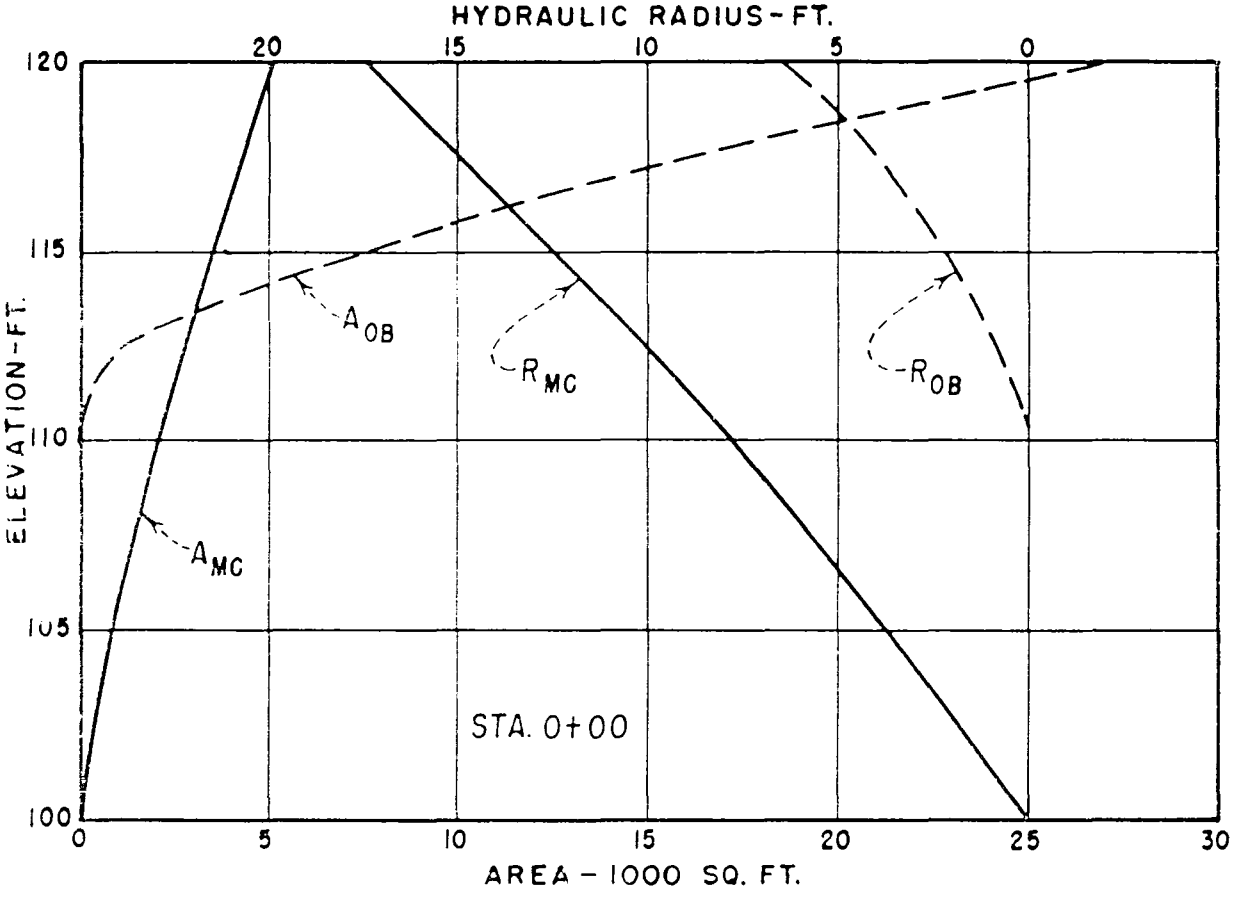
Figure 31



PLAN MAP
SILVER FOX RIVER

Figure 32

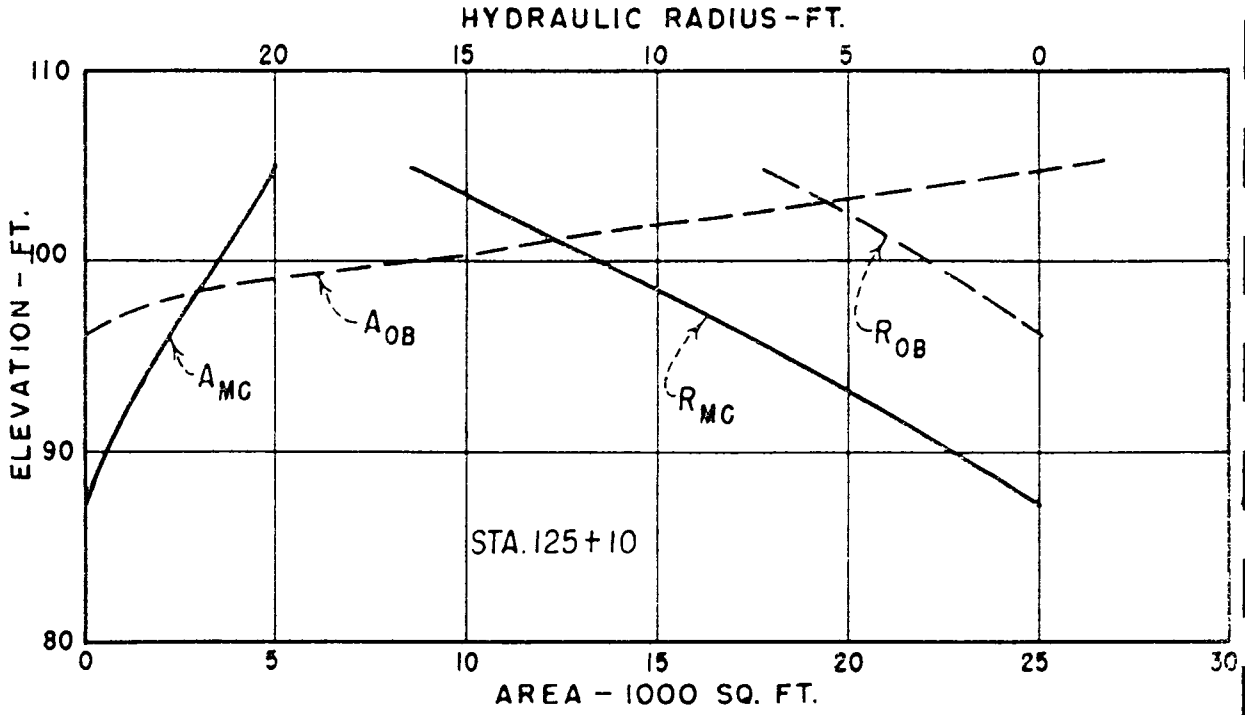


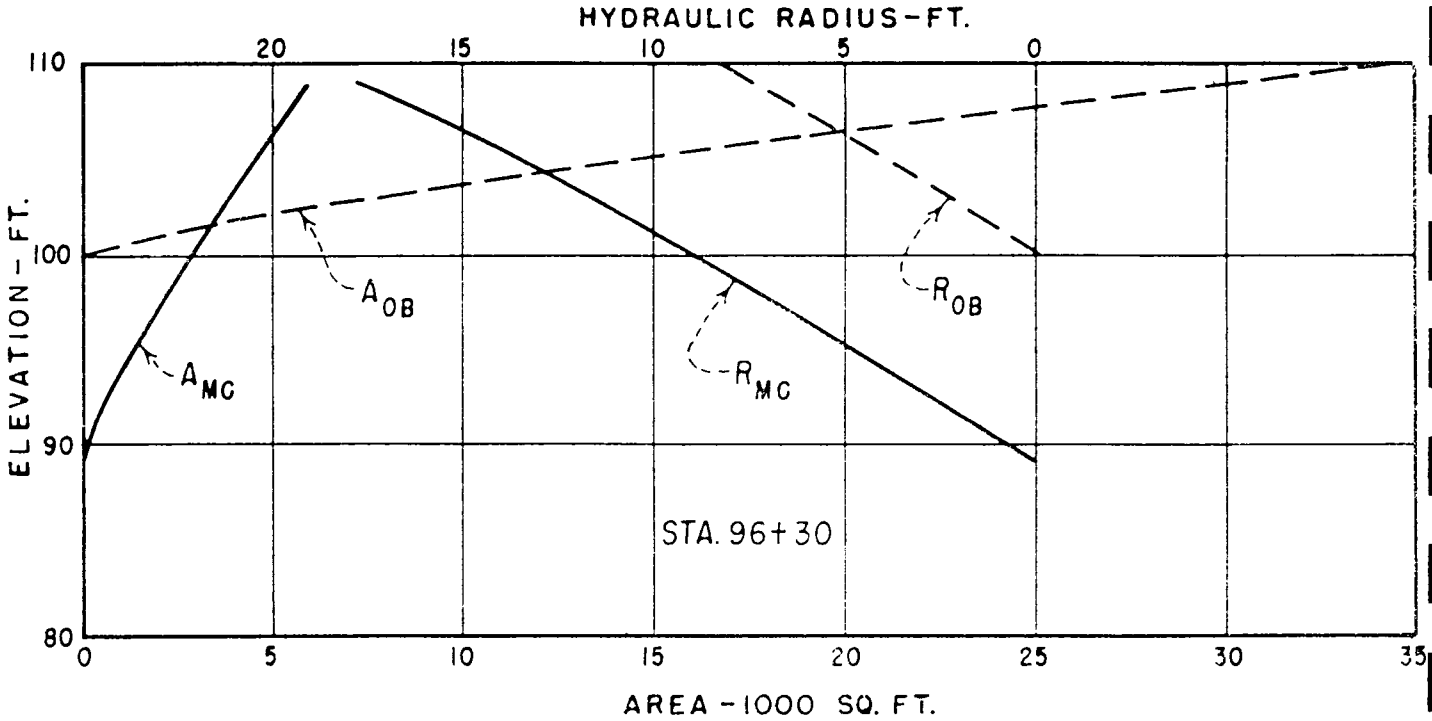


SILVER FOX RIVER

MC - Main Channel
OB - Overbank

Figure 33

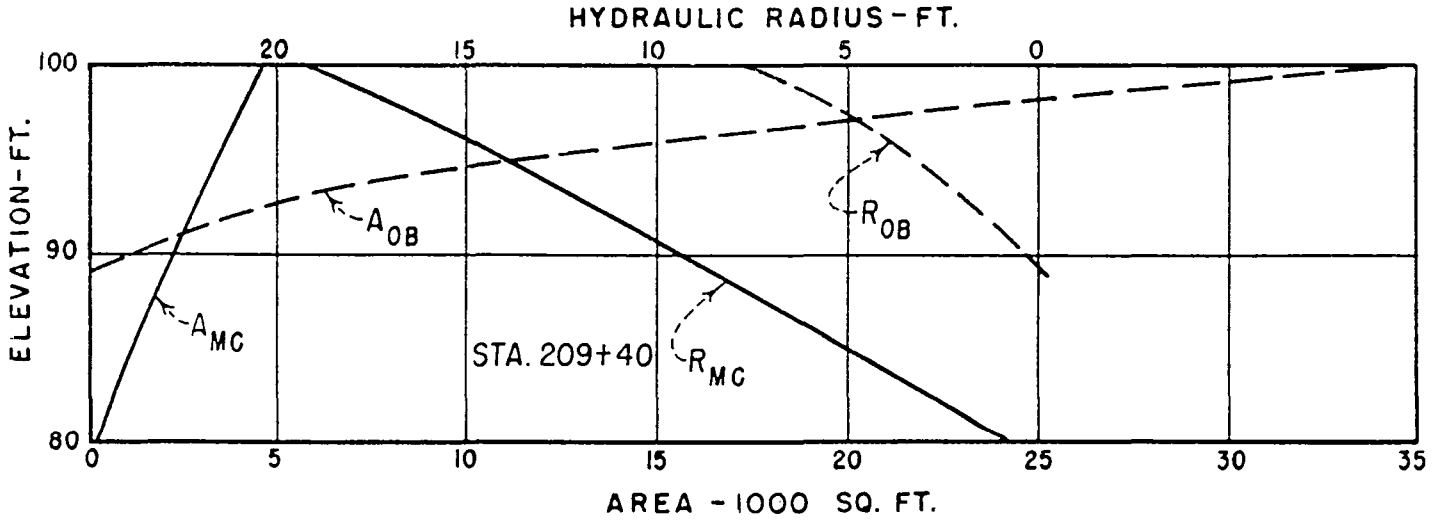


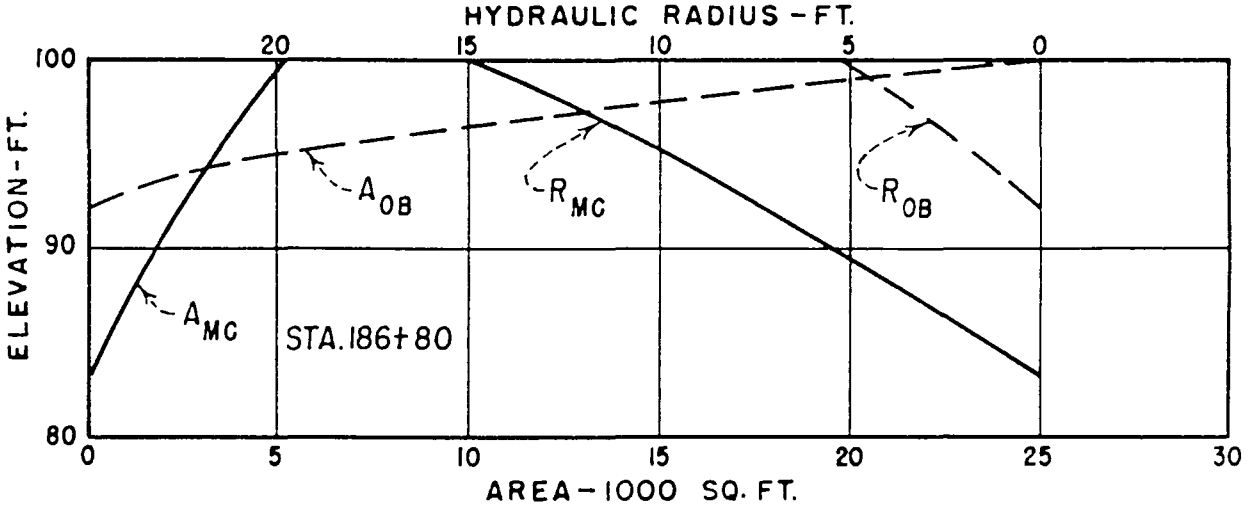


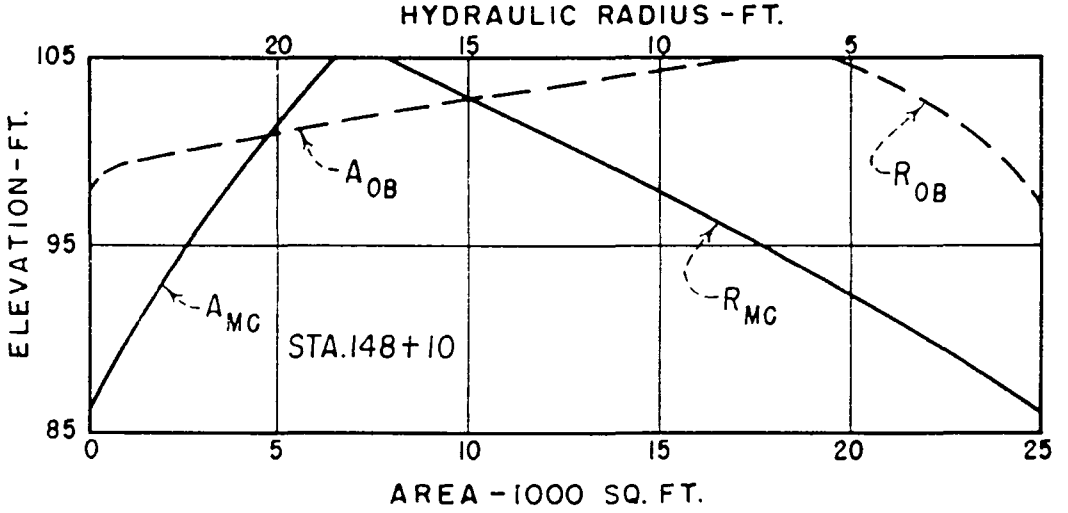
SILVER FOX RIVER

MC – Main Channel
OB – Overbank

Figure 34







SILVER FOX RIVER

MC - Main Channel
OB - Overbank

Figure 35

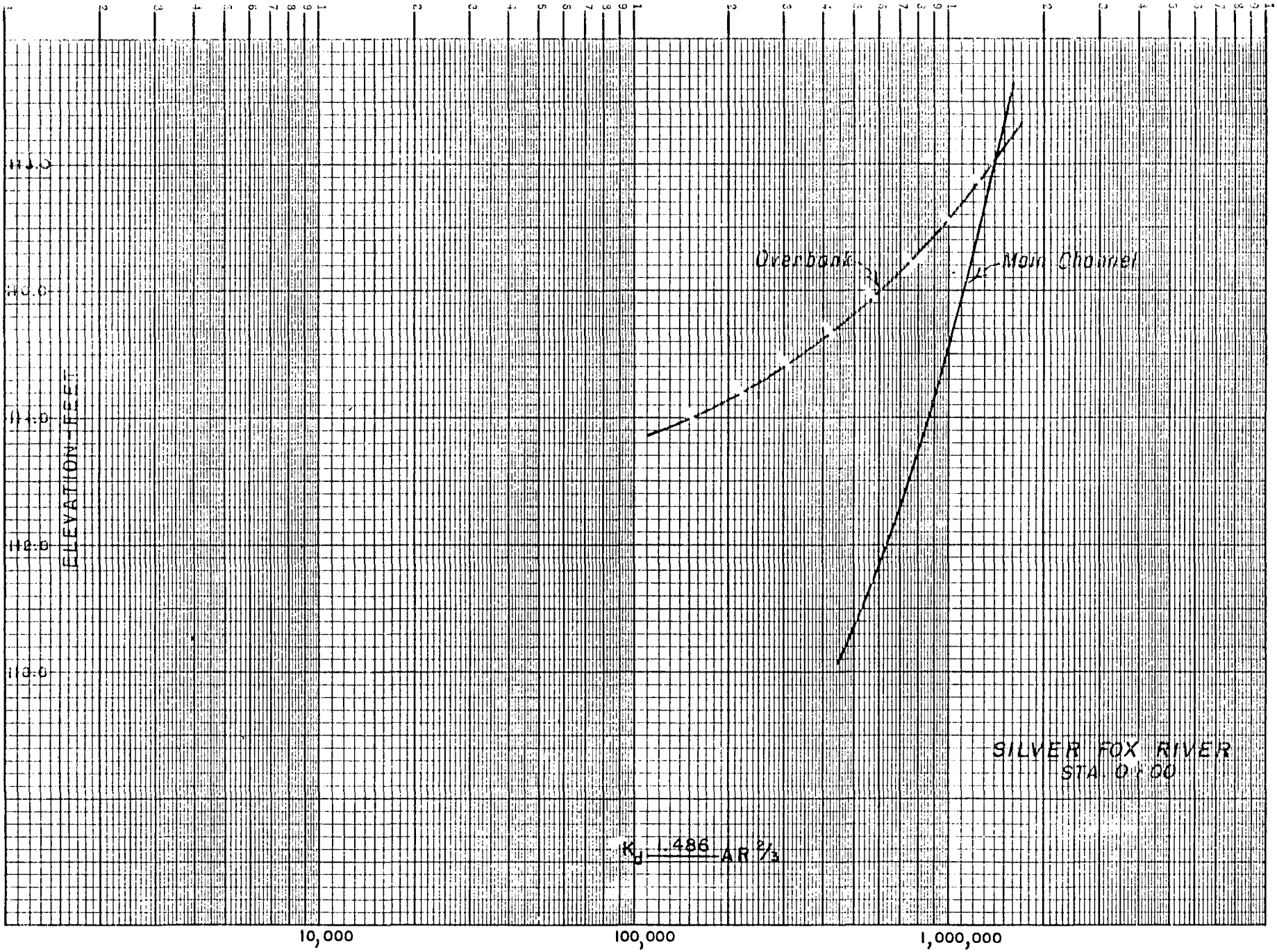

ELEVATION - FEET
Overbank
Main Channel
SILVER FOX RIVER
STA. 0+00
$K_d = \frac{1.486}{n} AR^{2/3}$
10,000
100,000
1,000,000

Figure 36

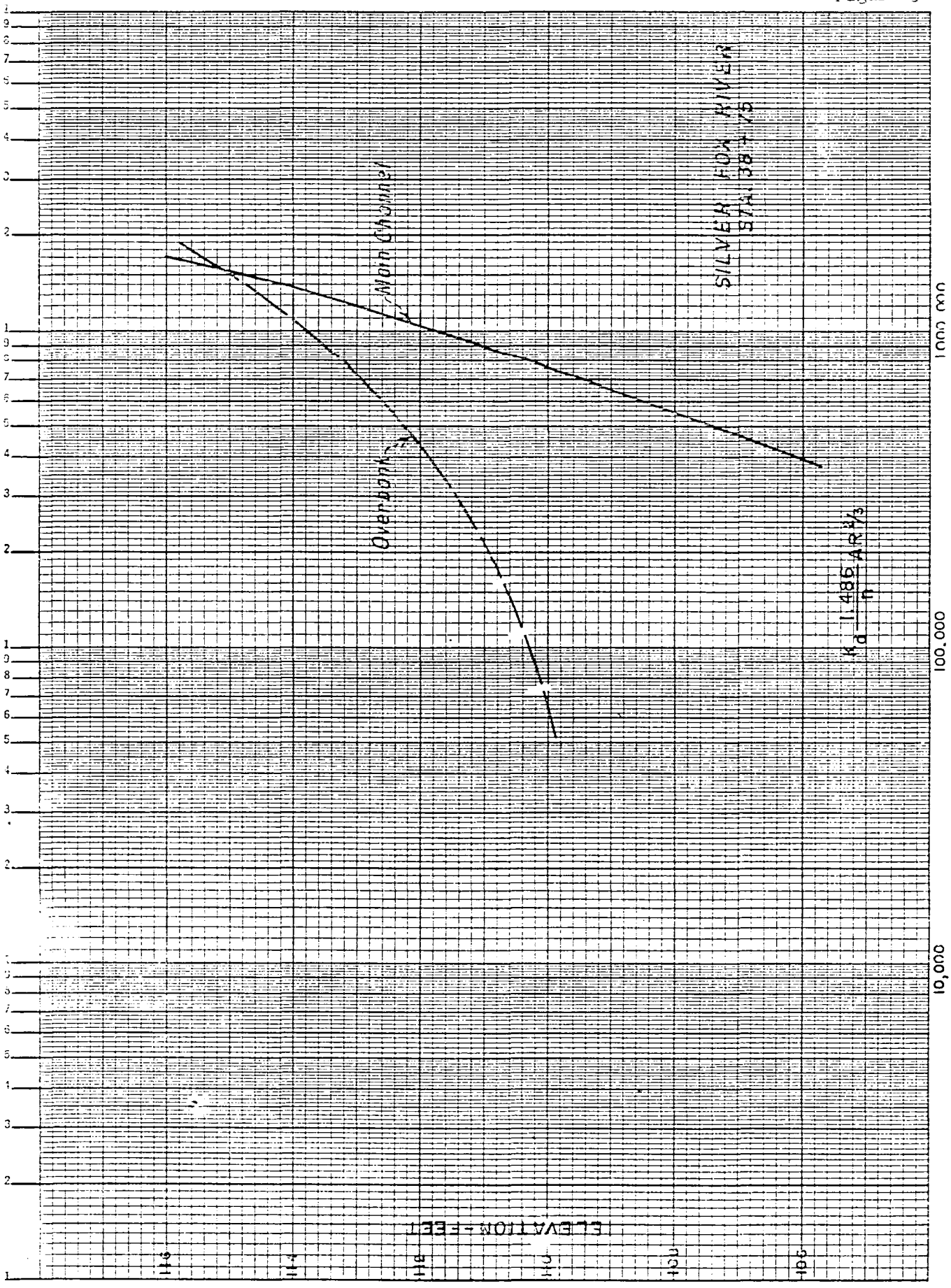
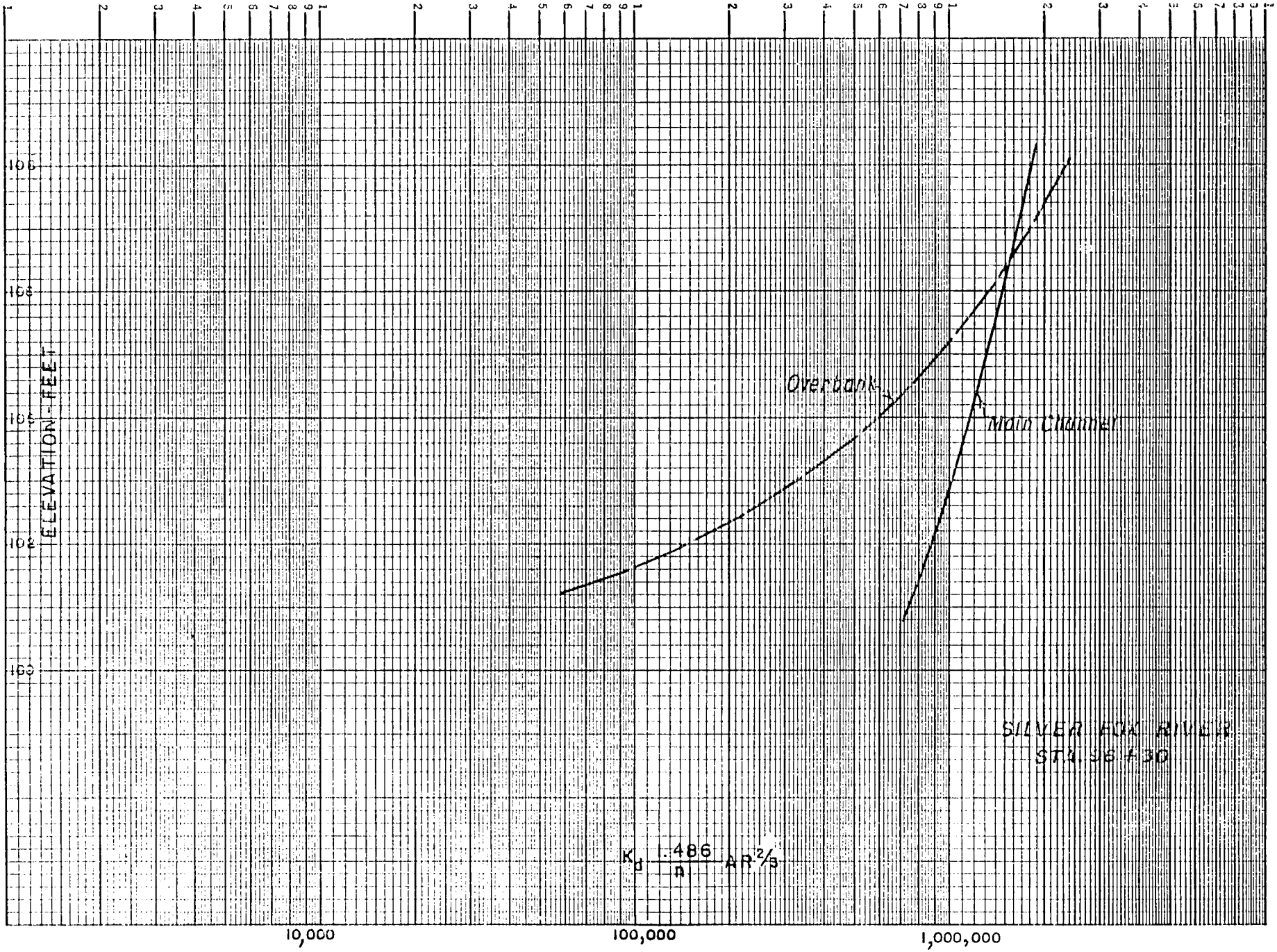

SILVER FOX RIVER
STA. 38+75
Main Channel
Overbank
$K_d = \frac{1.486}{n} AR^{2/3}$
ELEVATION-FEET
1,000,000
100,000
10,000

Figure 37


ELEVATION - FEET
10,000
100,000
1,000,000
Overbank
Main Channel
SILVER FOX RIVER
STA. 96 + 30
$K_d \frac{.486}{n} AR^{2/3}$

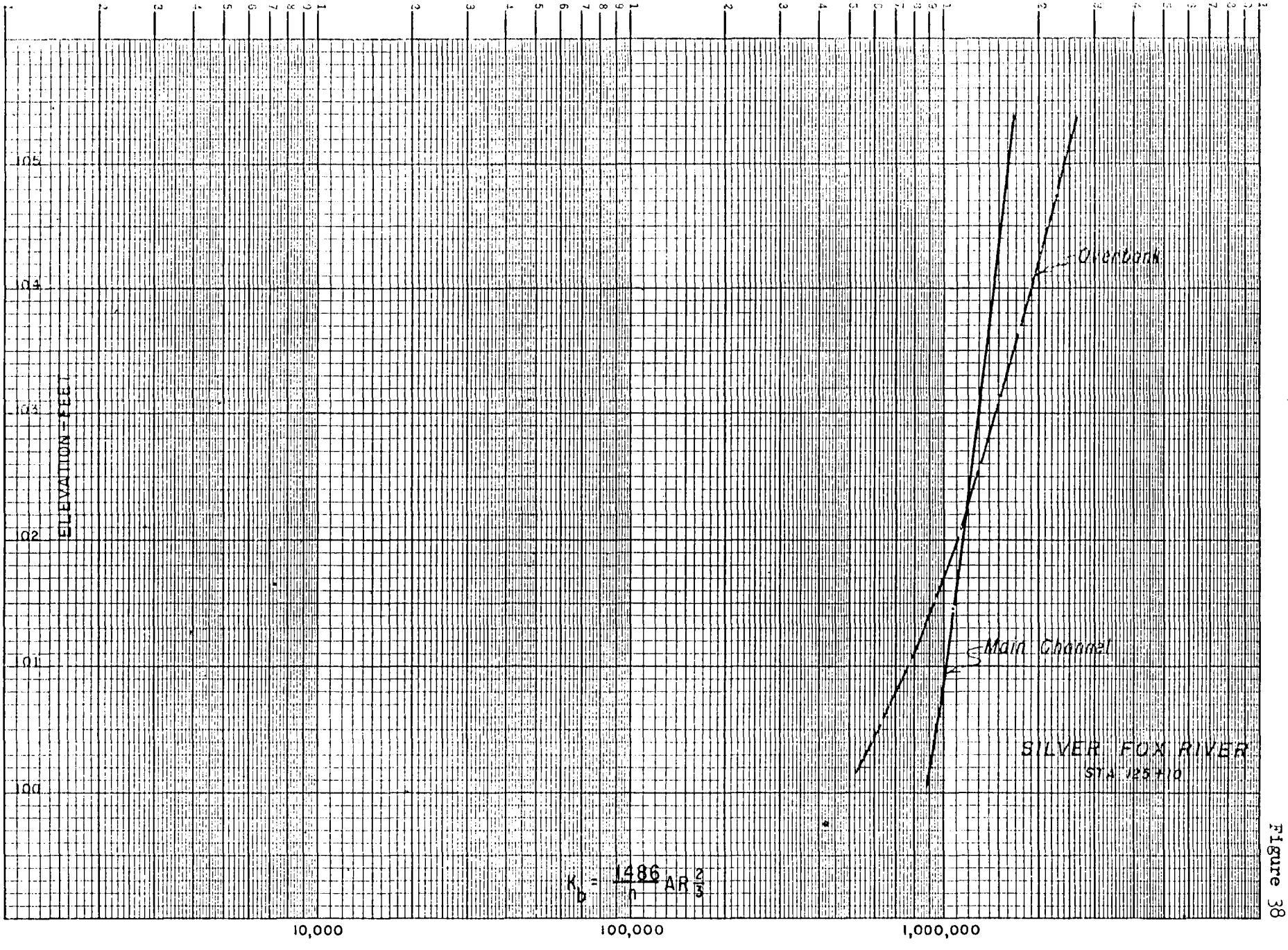

ELEVATION - FEET
105
104
103
102
101
100
10,000
100,000
1,000,000
$K_b = \frac{1486}{n} AR^{\frac{2}{3}}$
Overbank
Main Channel
SILVER FOX RIVER
STA. 125+10

Figure 38

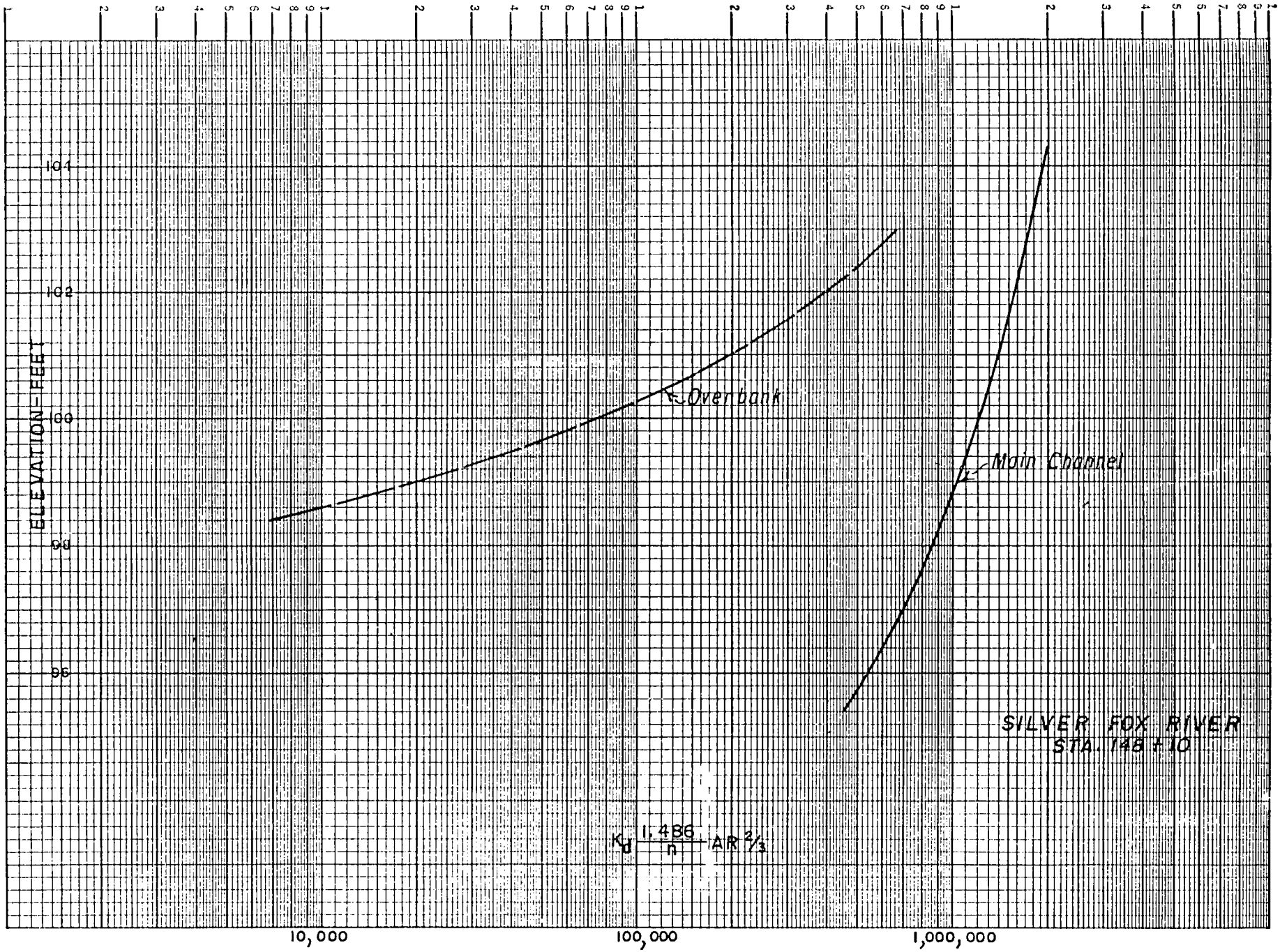

ELEVATION-FEET
104
102
100
98
96
Overbank
Main Channel
SILVER FOX RIVER
STA. 148 + 10
$K_d \frac{1.486}{n} AR^{2/3}$
10,000
100,000
1,000,000

Figure 39

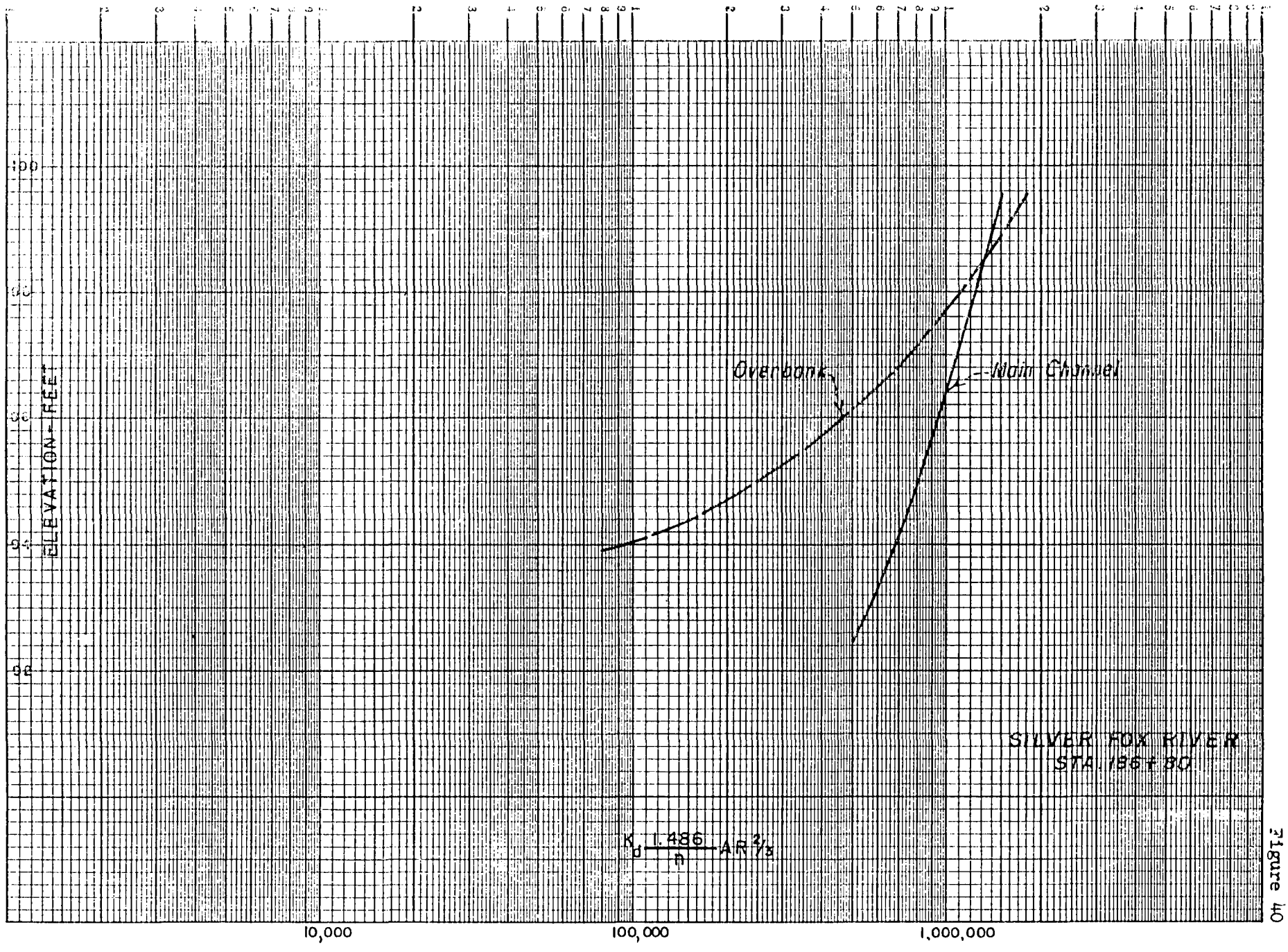

ELEVATION - FEET
Overbank
Main Channel
SILVER FOX RIVER
STA. 186 + 80
$K_u \frac{1.486}{n} AR^{2/3}$
10,000
100,000
1,000,000

Figure 40

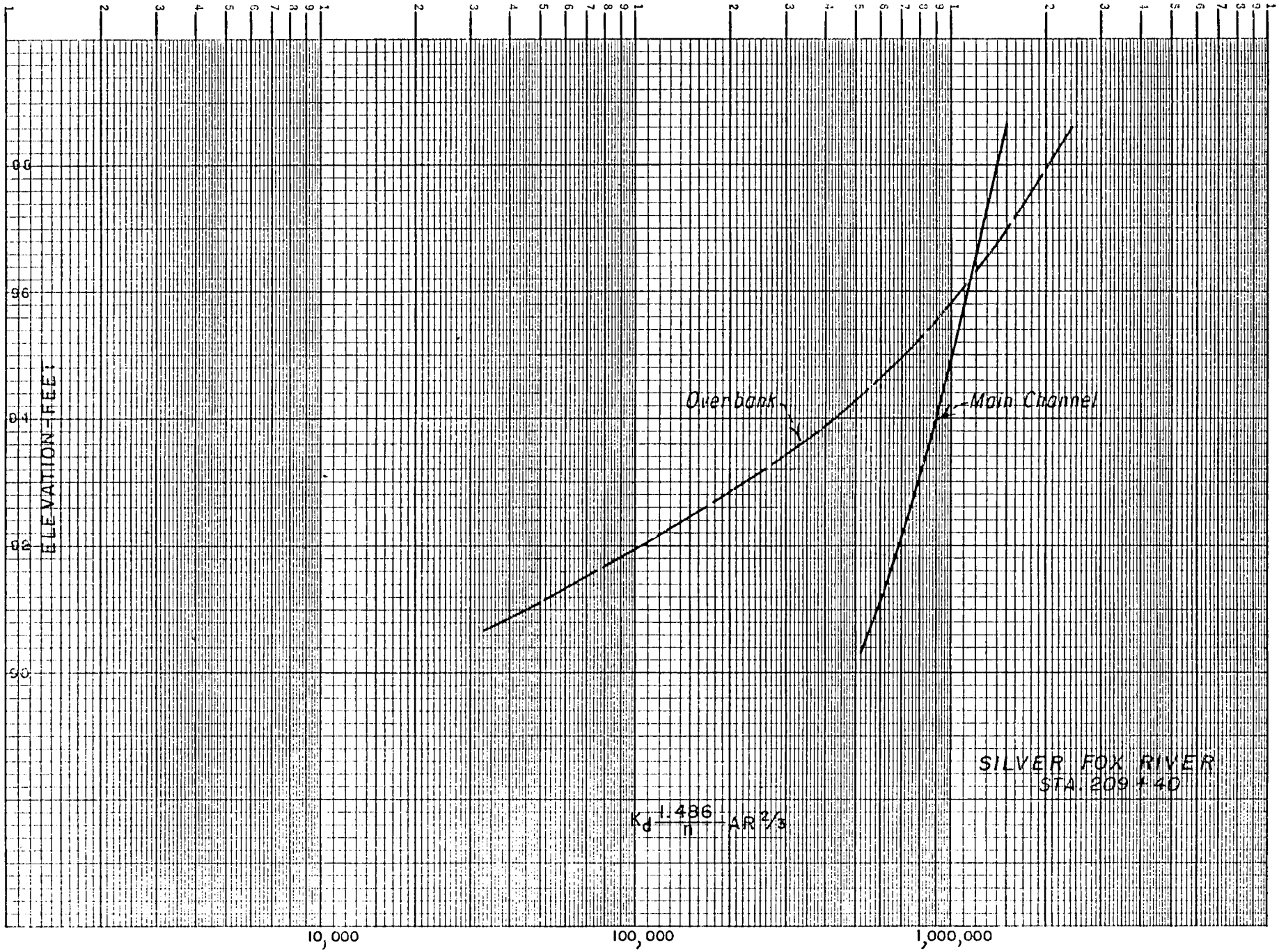
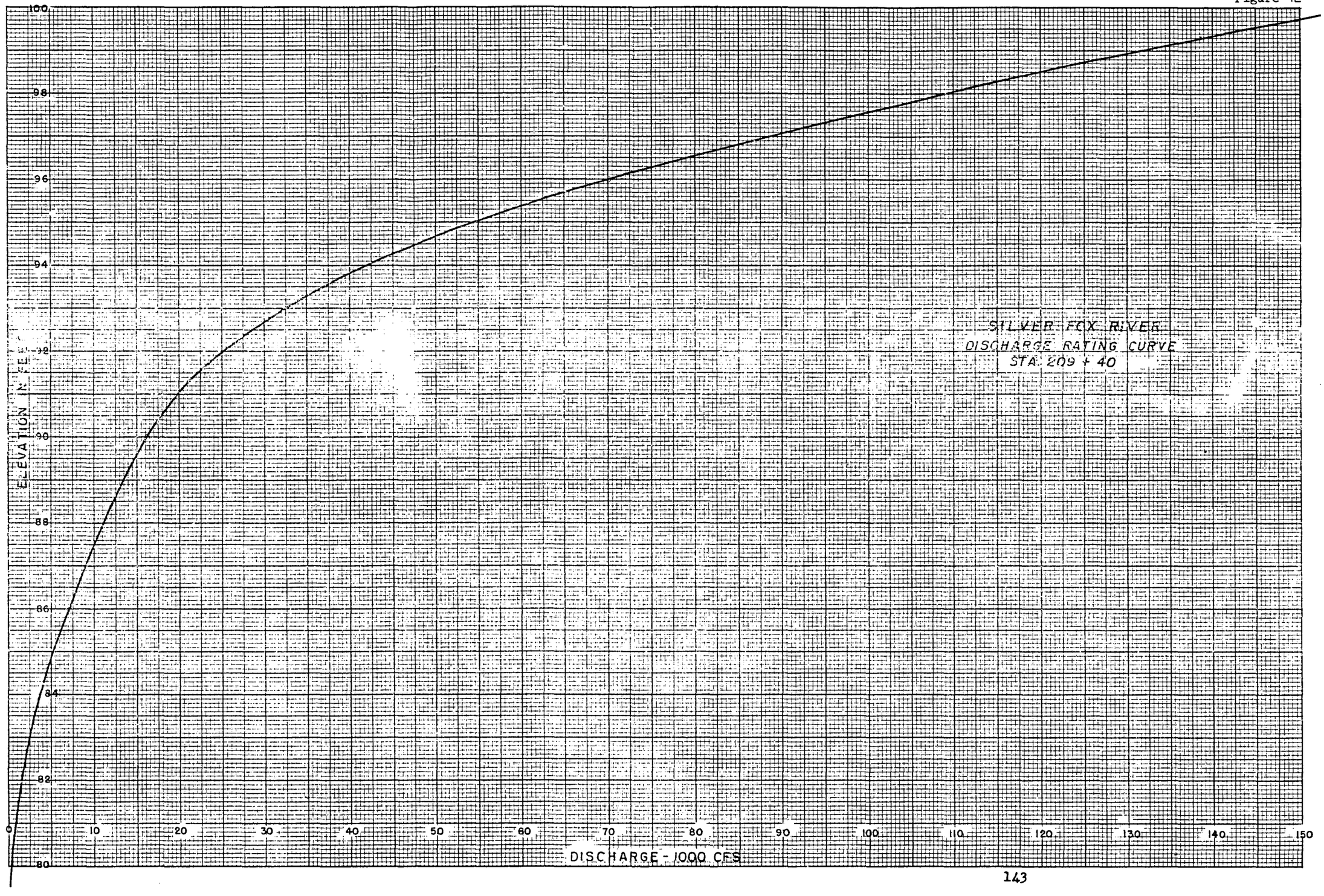

ELEVATION-FEET
99
96
94
92
90
10,000
100,000
1,000,000
Overbank
Main Channel
SILVER FOX RIVER
STA. 209 + 40
$K_d \frac{1.486}{n} AR^{2/3}$

Figure 41

Figure 42


SILVER FOX RIVER
DISCHARGE RATING CURVE
STA. 209 + 40
ELEVATION IN FEET
100
98
96
94
92
90
88
86
84
82
80
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
DISCHARGE - 1000 CFS

WATER SURFACE PROFILE COMPUTATIONS – METHOD B

SUBJECT: Silver Fox River Discharge = 50,000 cfs

COMPUTED BY: _____ DATE _____ CHECKED BY: _____ DATE: _____ SHEET 1 OF 2

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
SECTION	Reach Length L	Assumed W.S. Elevation	Area A	$K_d = \frac{1.486}{n} AR^{2/3}$	$K_d / L^{1/2}$	$\left(Q / \Sigma \frac{K_d}{L^{1/2}}\right)^2 = h_f$	$(K_d / L^{1/2})(h_f)^{1/2} = Q$	V	$\dot{V}^a Q$	h_v	$h_{v_1} - h_{v_2}$	Eddy Loss	Mean h_f	Total Loss	Δ H	Water Surface Elevation
209+40	2,260	94.67	3,390	973,000	20,500		30,000	8.85	2,350,000							94.67
	1,955		11,600	604,000	13,700		20,000	1.72	59,000							
					34,200	2.13	50,000		2,409,000	0.75						
186+80	2,260	96.90	4,140	1,070,000	22,600		28,700	6.93	1,380,000							
	1,955		13,650	739,000	16,700		21,300	1.56	52,000							
					39,300	1.62	50,000		1,432,000	0.45	+0.30	0.03	1.88	1.91	2.21	96.88
186+80	3,870	96.90	4,140	1,070,000	17,200		28,600	6.91	1,365,000							
	3,265		13,650	739,000	12,900		21,400	1.57	53,000							
					30,100	2.76	50,000		1,418,000	0.44						
148+10	3,870	100.35	4,660	1,258,000	20,200		45,700	9.80	4,390,000							
	3,265		3,520	110,200	1,920		4,300	1.22	6,400							
					22,120	5.10	50,000		4,396,400	1.37	-0.93	0.47	3.93	4.40	3.47	100.35
148+10	2,300	100.35	4,660	1,258,000	26,200		45,500	9.76	4,330,000							
	1,800		3,520	110,200	2,600		4,500	1.28	7,360							
					28,800	3.01	50,000		4,337,360	1.35						
125+10	2,300	103.40	4,520	1,350,000	28,100		21,000	4.64	453,000							
	1,800		20,700	1,642,000	38,800		29,000	1.40	56,900							
					66,900	0.56	50,000		509,900	0.16	+1.19	0.12	1.79	1.91	3.10	103.45
125+10	2,880	103.40	4,520	1,350,000	25,100		21,100	4.67	460,000							
	2,290		20,700	1,642,000	34,400		28,900	1.40	56,400							
					59,500	0.71	50,000		516,400	0.16						
96+30	2,880	104.50	4,440	1,238,000	23,100		29,800	6.71	1,340,000							
	2,290		13,100	750,000	15,700		20,200	1.54	48,000							
					38,800	1.66	50,000		1,388,000	0.43	-0.27	0.14	1.19	1.33	1.06	104.51
96+30	5,755	104.50	4,440	1,238,000	16,300		30,200	6.80	1,400,000							
	4,920		13,100	750,000	10,700		19,800	1.51	45,000							
					27,000	3.42	50,000		1,445,000	0.45						
38+75	5,755	111.10	3,840	918,000	12,100		39,300	10.22	4,020,000							
	4,920		5,900	232,000	3,310		10,700	1.81	35,000							
					15,410	10.55	50,000		4,055,000	1.26	-0.81	0.41	6.99	7.40	6.59	111.10

WATER SURFACE PROFILE COMPUTATIONS – METHOD B

SUBJECT: Silver Fox River Discharge = 50,000 cfs

COMPUTED BY: _____ DATE _____ CHECKED BY: _____ DATE: _____ SHEET 2 OF 2

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
SECTION	Reach Length L	Assumed W.S. Elevation	Area A	$K_d = \frac{1.486}{n} AR^{2/3}$	$K_d / L^{1/2}$	$\left(\frac{Q}{\Sigma K_d / L^{1/2}}\right)^2 = h_f$	$(K_d/L^{1/2})(h_f)^{1/2} = Q$	V	V^2Q	h_v	$h_{v_1} - h_{v_2}$	Eddy Loss	Mean h_f	Total Loss	Δ H	Water Surface Elevation
38+75	3,875	111.10	3,840	918,000	14,750		39,200	10.20	4,080,000							
	3,295		5,900	232,000	4,040		10,800	1.83	36,000							
					18,790	7.08	50,000		4,116,000	1.28						
0+00	3,875	116.60	4,130	1,192,000	19,200		28,800	6.98	1,405,000							
	3,295		14,000	807,000	14,100		21,200	1.51	48,000							
					33,300	2.26	50,000		1,453,000	0.45	+0.83	0.08	4.67	4.75	5.58	116.63

Method II

This method can be used for computing water surface profiles under the same conditions as for the preceding Method B. An application of Method II is shown in the following step computations. The same field data of Method B are used. The computation is for one discharge of 50,000 cfs, Silver Fox River.

Step 1

Referring to Table 23, which is the computational form to be used for this method, enter the pertinent data in the heading. In this example, these are the Silver Fox River water surface profile computations for a discharge of 50,000 cfs.

Step 2

Enter the section number or stationing under Column 1. This has been designated as 209+40 for the first section in the example.

Step 3

Under Column 2, the main channel and overbank reach lengths between sections are entered. These are noted as 2,260 and 1,955 feet between Sections 209+40 and 186+80 for the main channel and overbank reaches as taken from Figure 31 (see Method B).

Step 4

The water surface elevation in Column 3 will not have to be assumed for the Section 209+40. An elevation of 94.67 has been determined for this section as established from the rating curve in Figure 42 (see Method B). The areas and hydraulic radii of the

main channel and overbank sections are entered in Columns 4 and 5. These are read from Figure 32 (see Method B) for an elevation of 94.67.

Step 5

The Y values in Column 6 are taken from Table 19 (see Standard Step Method). These are read as 0.116 and 0.319 for respective R values of 13.95 and 3.05.

Step 6

Manning's roughness coefficients of 0.030 and 0.060 have been determined for the main channel and overbank sections.

$$K'' = \frac{A}{nYL^{1/2}}$$

Using the data for the main channel, this is computed as follows:

$$K'' = \frac{3390}{(0.030)(0.116)(2260)^{1/2}} = 20{,}500$$

A, n, Y, and L are read from Columns 4, 7, 6, and 2, respectively.

Step 8

The friction head, h_f, in Column 9 is computed from the equation:

$$h_f = \left(\frac{Q}{K''}\right)^2$$

Using the total K'' in Column 8, this resolves to:

$$h_f = \left(\frac{50{,}000}{34{,}200}\right)^2 = 2.14$$

Step 9

Column 10 shows the partial discharges existing in the main channel and overbank area. These are determined from the formula:

$$Q = K'' \left(h_f \right)^{1/2}$$

Substituting main channel data:

$$Q = (20,500)(2.14)^{1/2} = 30,000$$

Step 10

Subdivisional velocities in Column 11 are computed by the equation $V = Q/A$. Thus, for the main channel, $V = 30,000/3390 =$ 8.85 feet per second.

Step 11

The individual velocities are squared and multiplied by their respective discharges and entered in Column 12. For the main channel $V^2Q = (8.85)^2\ (30,000) = 2,350,000$.

Step 12

Column 13 shows the average weighted velocity squared computed as follows:

$$V^2_{ave} = \frac{V^2Q}{Q}$$

which is determined in the example, using the total V^2Q from Column 12:

$$V^2_{ave} = \frac{2,409,000}{50,000} = 48.2$$

Step 13

The weighted velocity head is entered in Column 14 as computed by dividing Column 13 by 2g or 64.4. Thus, in the example:

$$h_v = \frac{V^2_{ave}}{64.4} = \frac{48.2}{64.4} = 0.75$$

Step 14

The next step consists of assuming a water surface elevation for the next upstream section 186+80. This has been assumed as 96.90 in the example. Similar computations are made through Column 14 as described above for Section 209+40. The velocity head, h_v, for Section 186+80 computes to 0.45. In Column 15, the algebraic difference in velocity heads is entered. Thus, $h_{v_1} - h_{v_2} = 0.75 - 0.45 = +0.30$.

Step 15

The reach between Sections 209+40 and 186+80 is diverging; therefore, a 10-percent correction is applied to the change in velocity heads to determine the eddy loss in Column 16. In the example, this amounts to 0.1 x 0.30 = 0.03. A convenient rule to follow in the eddy loss determination is to apply a 10-percent correction to positive values in Column 15, and 50 percent to negative values.

Step 16

Column 17 is the mean friction head loss, h_f, computed by averaging the consecutive values of Column 9. This is computed as follows:

$$\text{Mean } h_f = \frac{2.14 + 1.62}{2} = 1.88$$

Step 17

The total loss in Column 18 is determined by adding Columns 16 and 17.

Step 18

Column 19, ΔH, is the algebraic sum of Columns 15 and 18.

Step 19

The water surface elevation in Column 20 is calculated by adding Column 19 to the preceding water surface elevation of the downstream section. In this example, this is computed as follows:

$$\begin{array}{r} 94.67 \\ +\ \underline{2.21} \\ 96.88 \end{array}$$

When a balance to the nearest tenth foot is obtained between Columns 20 and 3, the computations are complete. The next step involves a computation of the friction head, h_f , between Sections 186+80 and 148+10 (next upstream section) using the lengths of 3,870 and 3,265 as shown for the main channel and overbank. The water surface elevation at Section 148+10 is then assumed and a new cycle of computations is begun following the above outlined step procedure.

A short cut in computing K'' values in Column 8 for the upstream reach consists of multiplying the K'' value of downstream reach by the inverse ratio of the square roots of the lengths in the two reaches. To illustrate this procedure, refer to

Section 186+80 in the example. The main channel length between Sections 209+40 and 186+80 is 2,260 feet, but between Sections 148+10 and 186+80 it is 3,870 feet. Thus, to compute the K'' value for the main channel of the upstream reach (3,870 feet) for this section, the following calculation is made:

$$\frac{(2260)^{.5}}{(3870)^{.5}} \times 22{,}600 = 17{,}300$$

A similar computation can be made for the overbank as follows:

$$\frac{(1955)^{.5}}{(3265)^{.5}} \times 16{,}750 = 12{,}930$$

Table 23

WATER SURFACE PROFILE COMPUTATIONS—METHOD II

SUBJECT: Silver Fox River Water Surface Profile Computations for Discharge of 50,000 cfs

Computed By: TD Date: Checked By: JML Date: Sheet 1 of 2

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
Section	Reach Length L	Assumed W.S. Elevation	Area A	Hydraulic Radius R	$\frac{1}{1.486R^{2/3}} = Y$	n	$\frac{A}{nY\sqrt{L}} = K''$	$\left(\frac{Q}{K''}\right)^2 = h_f$	$K''(h_f)^{1/2} = Q$	V	V^2Q	V^2 ave	$\frac{V^2 \text{ ave}}{64.4} = h_v$	$h_{v_1} - h_{v_2}$	Eddy Loss	Mean h_f	Total Loss	ΔH	Water Surface Elevation
209+40	2,260	94.67	3,390	13.95	0.116	.030	20,500		30,000	8.85	2,350,000								94.67
	1,955		11,600	3.05	.319	.060	13,700		20,000	1.72	59,000								
							34,200	2.14	50,000		2,409,000	48.2	0.75						
186+80	2,260	96.90	4,140	12.00	.128	.030	22,600		28,700	6.93	1,380,000								
	1,955		13,650	3.23	.308	.060	16,750		21,300	1.56	52,000								
							39,350	1.62	50,000		1,432,000	28.7	0.45	+0.30	0.03	1.88	1.91	2.21	96.88
186+80	3,870	96.90	4,140	12.00	.128	.030	17,300		28,600	6.91	1,365,000								
	3,265		13,650	3.23	.308	.060	12,930		21,400	1.57	53,000								
							30,230	2.74	50,000		1,418,000	28.4	0.44						
148+10	3,870	100.35	4,660	12.70	.124	.030	20,200		45,660	9.80	4,400,000								
	3,265		3,500	1.42	.533	.060	1,920		4,340	1.24	7,000								
							22,120	5.10	50,000		4,407,000	88.3	1.37	-0.93	.47	3.92	4.39	3.46	100.34
148+10	2,300	100.35	4,660	12.70	.124	.030	26,300		45,500	9.75	4,330,000								
	1,800		3,500	1.42	.533	.060	2,580		4,500	1.29	7,000								
							28,880	3.00	50,000		4,337,000	86.6	1.35						
125+10	2,300	103.40	4,520	15.00	.110	.030	28,400		20,600	4.57	430,000								
	1,800		20,700	6.00	.204	.060	40,500		29,400	1.42	59,000								
							68,900	0.53	50,000		489,000	9.8	0.15	+1.20	.12	1.77	1.89	3.09	103.43
125+10	2,880	103.40	4,520	15.00	.110	.030	25,400		20,900	4.63	450,000								
	2,290		20,700	6.00	.204	.060	35,400		29,100	1.41	58,000								
							60,800	0.68	50,000		508,000	10.2	0.16						
96+30	2,880	104.50	4,440	13.35	.120	.030	23,050		29,700	6.69	1,330,000								
	2,290		13,100	3.52	.291	.060	15,700		20,300	1.55	49,000								
							38,750	1.65	50,000		1,379,000	27.6	0.43	-0.27	.14	1.17	1.31	1.04	104.47
96+30	5,755	104.50	4,440	13.35	.120	.030	16,350		30,200	6.80	1,400,000								
	4,920		13,100	3.52	.291	.060	10,700		19,800	1.51	45,000								
							27,050	3.41	50,000		1,445,000	28.9	0.45						
38+75	5,755	111.10	3,840	10.60	.139	.030	12,100		39,300	10.22	4,020,000								
	4,920		5,900	2.00	.424	.060	3,300		10,700	1.81	35,000								
							15,400	10.55	50,000		4,055,000	81.0	1.26	-0.81	.41	6.98	7.39	6.58	111.05

Table 23

WATER SURFACE PROFILE COMPUTATIONS—METHOD II

SUBJECT: Silver Fox River Water Surface Profile Computations for Q = 50,000 cfs

Computed By: _____ Date: _____ Checked By: _____ Date: _____ Sheet 2 of 2

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
Section	Reach Length L	Assumed W.S. Elevation	Area A	Hydraulic Radius R	$\frac{1}{1.486 R^{2/3}} = Y$	n	$\frac{A}{nY\sqrt{L}} = K''$	$\left(\frac{Q}{K''}\right)^2 = h_f$	$K''(h_f)^{1/2} = Q$	V	$V^2 Q$	V^2 ave	$\frac{V^2 \text{ ave}}{64.4} = h_v$	$h_{v_1} - h_{v_2}$	Eddy Loss	Mean h_f	Total Loss	ΔH	Water Surface Elevation
38+75	3,875	111.10	3,840	10.60	0.139	.030	14,750		39,200	10.20	4,080,000								
	3,295		5,900	2.00	.424	.060	4,040		10,800	1.83	36,000								
							18,790	7.08	50,000		4,116,000	82.4	1.28						
0+00	3,875	116.60	4,130	14.10	.115	.030	19,200		28,800	6.98	1,405,000								
	3,295		14,000	3.55	.289	.060	14,080		21,200	1.51	48,000								
							33,280	2.26	50,000		1,453,000	29.1	0.45	+0.83	0.08	4.67	4.75	5.58	116.63

Example of a Backwater Computation

An illustration of a backwater computation is shown in Table 25 for a fictitious reservoir, the Alpine Reservoir. The example is computed for a discharge of 25,000 cfs using Method A as previously discussed.

Of first importance is the computation of the water surface profile for 25,000 cfs under natural conditions. These calculations are shown in Table 24. Again, Method A has been used. The natural profile has been computed to the Range 1-A, which is 8,043 feet upstream from the beginning Range 6. It is noted from the computation that the elevation at this Range (1-A) is 5505.8. It is initially assumed that the backwater profile should be approaching this profile near this point.

The next step consists of computing the backwater profile for this discharge. This has been done in Table 25. From a predetermined reservoir elevation of 5500.0, the computations are begun at Range 6. It is noted under Column 12 that the beginning velocity head is less than 0.05 which is the recommended criteria for a starting point. The computations are carried out in the usual manner to the Range 3-A. At this point, it is noted that the computations show the elevation is within 0.1 foot of the elevation under natural conditions. This is within the required limit; the computations, therefore, can be discontinued since the next upstream point would be nearer than the 0.1-foot difference indicated at Range 3-A. Theoretically, however, the two profiles never coincide.

The water surface profiles computed for the two conditions are drawn as shown in Figure 43. Additional profiles can be computed when warranted for other conditions and also included in this graph.

WATER SURFACE PROFILE COMPUTATIONS – METHOD A

SUBJECT: Water surface profile computations for natural conditions--Alpine Reservoir Q = 25,000 cfs

COMPUTED BY: DATE CHECKED BY: DATE: SHEET 1 OF 1

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
STATION	Assumed Water Surface Elev.	Area A	Dist. L	$K_d = \frac{1.486}{n} AR^{2/3}$	$S_f = \left(\frac{Q}{K_d}\right)^2$	Mean S_f	Mean S_f x L = h_f	Q	V	V^2Q	h_v	$h_{v_1} - h_{v_2}$	Eddy Loss	Total Loss	Δ H	W.S. El.
Range 6	5482.1	6,150		489,000	.00261			25,000	4.07		.26					5482.1
			863													
Range 5	5484.4	5,720		477,000	.00275	.00268	2.31	25,000	4.37		.30	-.04	.02	2.33	2.29	5484.39
			1,096													
Range 4	5487.4	4,830		470,000	.00283	.00279	3.06	25,000	5.18		.42	-.12	.06	3.12	3.00	5487.39
			2,437													
Range 3	5494.3	4,270		465,000	.00289	.00286	6.97	25,000	5.85		.53	-.11	.06	7.03	6.92	5494.31
			1,102													
Range 2	5497.5	3,960		455,000	.00301	.00295	3.25	25,000	6.31		.62	-.09	.05	3.30	3.21	5497.52
			964													
Range 1	5500.5	3,720		444,000	.00317	.00309	2.98	25,000	6.72		.70	-.08	.04	3.02	2.94	5500.46
			770													
Range 4-A	5502.8	3,200		442,000	.00320	.00319	2.46	25,000	7.81		.95	-.25	.13	2.59	2.34	5502.80
			392													
Range 3-A	5504.1	3,140		438,000	.00325	.00323	1.27	25,000	7.96		.98	-.03	.02	1.29	1.26	5504.06
			170													
Range 2-A	5504.6	2,940		390,000	.00411	.00368	0.63	25,000	8.50		1.12	-.14	.07	0.70	0.56	5504.62
			249													
Range 1-A	5505.8	2,870		325,000	.00592	.00502	1.25	25,000	8.71		1.18	-.06	.03	1.28	1.22	5505.84

WATER SURFACE PROFILE COMPUTATIONS – METHOD A

SUBJECT: Water surface profile computations for natural conditions--Alpine Reservoir Q = 25,000 cfs

COMPUTED BY: DATE CHECKED BY: DATE: SHEET 1 OF 1

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
STATION	Assumed Water Surface Elev.	Area A	Dist. L	$K_d = \frac{1.486}{n} AR^{2/3}$	$S_f = \left(\frac{Q}{K_d}\right)^2$	Mean S_f	Mean S_f x L = h_f	Q	V	V^2Q	h_v	$h_{v_1} - h_{v_2}$	Eddy Loss	Total Loss	Δ H	W.S. El.
Range 6	5482.1	6,150		489,000	.00261			25,000	4.07		.26					5482.1
			863													
Range 5	5484.4	5,720		477,000	.00275	.00268	2.31	25,000	4.37		.30	-.04	.02	2.33	2.29	5484.39
			1,096													
Range 4	5487.4	4,830		470,000	.00283	.00279	3.06	25,000	5.18		.42	-.12	.06	3.12	3.00	5487.39
			2,437													
Range 3	5494.3	4,270		465,000	.00289	.00286	6.97	25,000	5.85		.53	-.11	.06	7.03	6.92	5494.31
			1,102													
Range 2	5497.5	3,960		455,000	.00301	.00295	3.25	25,000	6.31		.62	-.09	.05	3.30	3.21	5497.52
			964													
Range 1	5500.5	3,720		444,000	.00317	.00309	2.98	25,000	6.72		.70	-.08	.04	3.02	2.94	5500.46
			770													
Range 4-A	5502.8	3,200		442,000	.00320	.00319	2.46	25,000	7.81		.95	-.25	.13	2.59	2.34	5502.80
			392													
Range 3-A	5504.1	3,140		438,000	.00325	.00323	1.27	25,000	7.96		.98	-.03	.02	1.29	1.26	5504.06
			170													
Range 2-A	5504.6	2,940		390,000	.00411	.00368	0.63	25,000	8.50		1.12	-.14	.07	0.70	0.56	5504.62
			249													
Range 1-A	5505.8	2,870		325,000	.00592	.00502	1.25	25,000	8.71		1.18	-.06	.03	1.28	1.22	5505.84

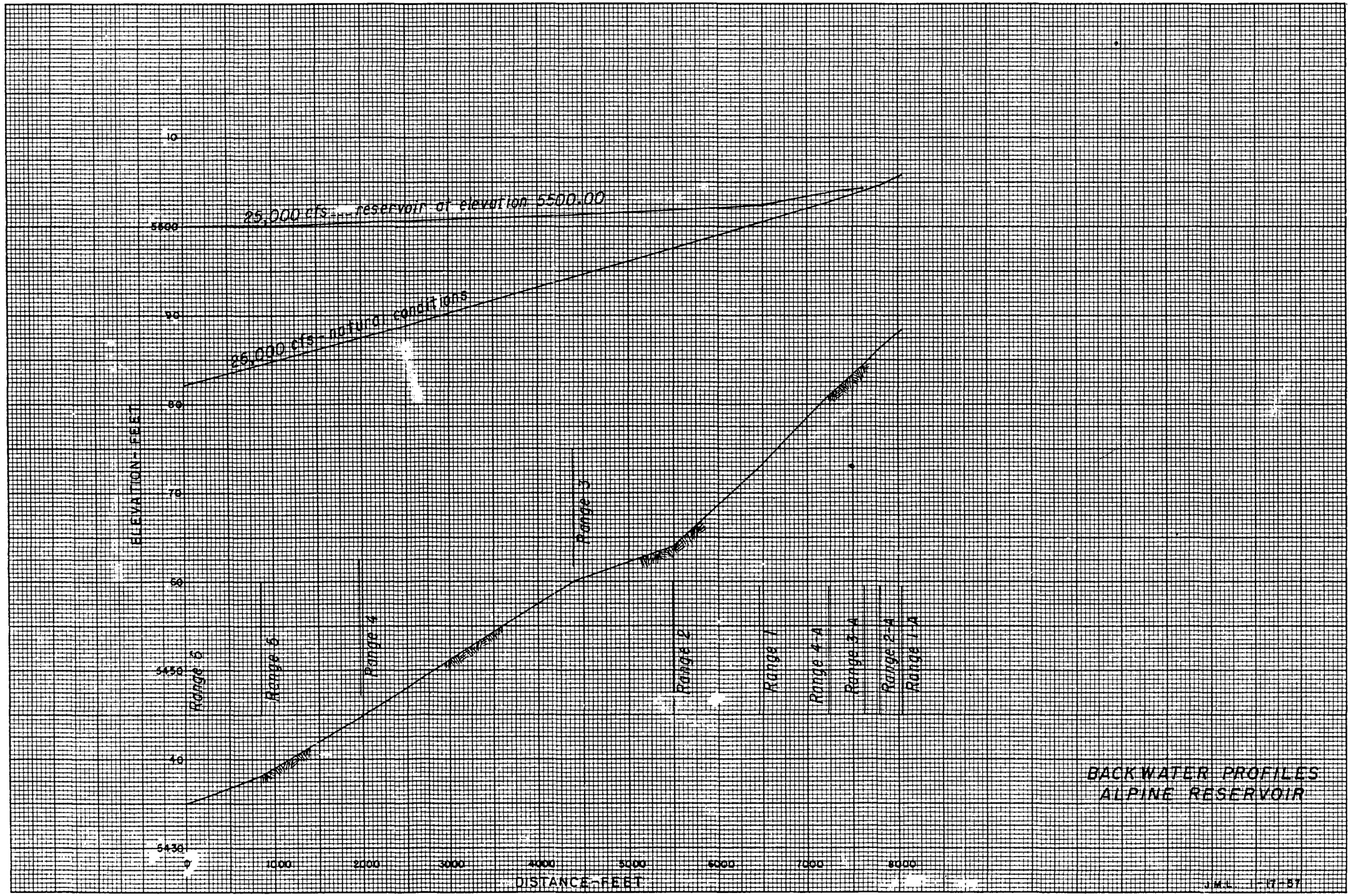
WATER SURFACE PROFILE COMPUTATIONS – METHOD A

SUBJECT: Backwater Computations--Alpine Reservoir--Elevation at 5500.0 n = 0.050 Q = 25,000 cfs

COMPUTED BY: _____ DATE _____ CHECKED BY: _____ DATE: _____ SHEET 1 OF 1

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
STATION	Assumed Water Surface Elev.	Area A	Dist. L	$K_d = \frac{1.486}{n} AR^{2/3}$	$S_f = \left(\frac{Q}{K_d}\right)^2$	Mean S_f	Mean S_f x L = h_f	Q	V	V^2Q	h_v	$h_{v_1} - h_{v_2}$	Eddy Loss	Total Loss	Δ H	W.S. El.
Range 6	5500.0	14,800		2,140,000	.000137			25,000	1.69		0.04					5500.0
			863													
Range 5	5500.1	13,100		1,840,000	.000185	.000161	0.14	25,000	1.91		0.06	-0.02	0.01	0.15	0.13	5500.13
			1,096													
Range 4	5500.4	11,200		1,470,000	.000289	.000237	0.26	25,000	2.23		0.08	-0.02	0.01	0.27	0.25	5500.38
			2,437													
Range 3	5501.2	10,200		1,300,000	.000370	.000320	0.84	25,000	2.45		0.09	-0.01	0	0.84	0.83	5501.21
			1,102													
Range 2	5501.8	8,340		954,000	.000688	.000529	0.58	25,000	3.00		0.14	-0.05	0.03	0.61	0.56	5501.77
			964													
Range 1	5502.4	5,280		800,000	.000980	.000755	0.73	25,000	4.73		0.31	-0.21	0.11	0.84	0.63	5502.40
			770													
Range 4-A	5503.4	4,370		569,000	.00193	.00146	1.12	25,000	5.71		0.51	-0.20	0.10	1.22	1.02	5503.42
			392													
Range 3-A	5504.2	3,740		480,000	.00272	.00233	0.91	25,000	6.09		0.69	-0.18	0.09	1.00	0.82	5504.24

Figure 43


25,000 cfs - reservoir at elevation 5500.00
25,000 cfs - natural conditions
ELEVATION - FEET
DISTANCE - FEET
Range 6
Range 5
Range 4
Range 3
Range 2
Range 1
Range 4-A
Range 3-A
Range 2-A
Range 1-A
5500
5450
5430
0
1000
2000
3000
4000
5000
6000
7000
8000
BACKWATER PROFILES
ALPINE RESERVOIR
J.M.L. 1-17-57

Check on Bridge or Contracted-Opening Losses

A method for checking the computed head loss through a bridge or contracted opening has been developed by using the contracted-opening discharge computation. This method, known as the "Contracted-Opening Method" by Hollister Johnson [13], is easily adapted to Method A or Method B as a check on the initial computations through a bridge. This method consists of solving the contracted-opening formula for the computed discharge. If the assumed discharge and the computed discharge agree, the total head loss at the bridge section is correct.

The contracted-opening formula, expressed in terms of this guide is:

$$Q = 8.02\ kA_1 \left(\Delta H + h_{v_2} - h_f \right)^{1/2}$$

where A_1 is the total area at the bridge section

ΔH is the total head loss between the upstream approach section and the bridge section

h_{v_2} is the weighted velocity head at the upstream approach section

h_f is the frictional head loss between the upstream approach section and the bridge section

Q is the computed discharge

k is a value for use in evaluating entrance losses on the basis of weighted perimeters of certain bridge characteristic factors as follows:

0.65 for sharp edges such as on piers, submerged structural members, etc.

0.95 for rounded edges of piers, embankments, 45-degree wing walls, etc.

1.00 for suppressed or no contraction, or no disturbance to flow as in continuous bed and banks

Using Section 4 and Section 5 of Method A (Table 16) to check Method A computations through the bridge, the computed Q is calculated as follows:

1. Evaluation of k value. From Cross Section 4, Figure 19, assuming the water surface at 5721.2

Perimeter	Length	k value	kL
a	1.4	0.95	1.3
b	24.1	1.00	24.1
c	17.9	1.00	17.9
d	20.2	1.00	20.2
e	30.0	1.00	30.0
f	30.2	1.00	30.2
g	11.9	1.00	11.9
h	30.1	1.00	30.1
i	40.0	1.00	40.0
j	1.2	0.95	1.1
k	11.2	0.65	7.3
l	11.2	0.65	7.3
m	13.8	0.65	9.0
n	13.8	0.65	9.0
o	10.2	0.65	6.6
p	10.2	0.65	6.6
q	1.7	0.65	1.1
r	1.7	0.65	1.1
Total	280.8		254.8

Weighted k value $= \frac{254.8}{280.8} = 0.91$

2. Check on discharge. From Table 16

$$A_1 = 1{,}300 \text{ ft.}^2$$

$$\Delta H = 0.87$$

$$h_{v_2} = 0.68$$

$$h_f = 0.37$$

$$\text{Assumed } Q = 11{,}100 \text{ cfs}$$

$$\text{Computed } Q = 8.02\ (0.91)(1300)\ (0.87 + 0.68 - 0.37)^{1/2}$$

$$Q = 9490\ (1.18)^{1/2}$$

$$Q = 10{,}300 \text{ cfs}$$

The computed discharge checks the assumed discharge by 7 percent which is within the accuracy of tail water data and computations. Any adjustment that would be necessary because the two discharges did not check would be made in the eddy loss.

Check for Critical Conditions

The following is an example for checking critical conditions.

In the course of the computations, it has been found that for a discharge of 60,000 cfs at Section X5, the computed elevation is 3511.3. The velocity of 25.6 feet per second is noted to be very high. The following check is made for critical velocity:

El. 3511.3

A = 2,340 square feet T = 129 feet

$$V_{cr} = 5.67 \left(\frac{A}{T}\right)^{1/2} = 5.67 \left(\frac{2{,}340}{129}\right)^{1/2}$$

$$V_{cr} = 24.2 \text{ feet per second} < 25.6 \text{ feet per second}$$

CHECK FOR CRITICAL CONDITIONS AT SECTION 4-5
DISCHARGE OF 60,000 CFS

Figure 44

where A = cross-sectional area

T = water surface width

V_{cr} = critical velocity

Since the critical velocity is less than the computed velocity, the next step consists of tabulating the following data in order to determine the elevation at the point of minimum specific energy. This is designated as the critical elevation for the purpose of the analysis.

Elevation	A	T	A/T	$(A/T)^{1/2}$	$5.67\ (A/T)^{1/2}$	Q_{cr}
3510.0	2190	126	17.4	4.17	23.6	51,700
10.5	2250	127	17.7	4.21	23.9	53,800
11.0	2310	128	18.0	4.25	24.1	55,700
11.5	2380	129	18.4	4.29	24.3	57,800
12.0	2450	130	18.8	4.34	24.6	60,300
3512.5	2530	131	19.3	4.40	24.9	63,000

The critical discharge curve is prepared as shown in Figure 44.

Entering the curve with a discharge of 60,000 cfs, the critical elevation of 3511.95 is determined. This is the elevation to be used in continuing the computations. It is noted that this elevation is about 0.7 foot higher than the one originally computed.

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