284

GEOLOGICAL SURVEY CIRCULAR 284

COMPUTATION OF PEAK DISCHARGE AT CONTRACTIONS

UNITED STATES DEPARTMENT OF THE INTERIOR
Douglas McKay, Secretary

GEOLOGICAL SURVEY
W. E. Wrather, Director

GEOLOGICAL SURVEY CIRCULAR 284

COMPUTATION OF PEAK DISCHARGE AT CONTRACTIONS

By C. E. Kindsvater, R. W. Carter, and H. J. Tracy

Washington, D. C. 1953

Free on application to the Geological Survey, Washington 25, D. C.

CONTENTS

ILLUSTRATIONS

COMPUTATION OF PEAK DISCHARGE AT CONTRACTIONS

By C. E. Kindsvater, R. W. Carter, and H. J. Tracy

INTRODUCTION

Measurement of peak discharge directly by the usual current-meter method is often impossible; roads become impassable; structures from which current-meter measurements could be made are washed out, knowledge of the flood rise may not be available sufficiently in advance to permit reaching the site near the time of the peak; the flow of debris or ice may prevent use of the current meter, or the rise and fall of the stream may be too rapid to allow a complete measurement even if an engineer is at the site with the necessary equipment. Consequently, at times it is necessary to use indirect methods of determining peak discharge. This report deals with a method for determining peak discharge at abrupt contractions from a survey of high-water marks and characteristics of the channel.

The method has been in general use for many years without adequate verification of the coefficients used in the basic formula. In recent years the Geological Survey has attempted to obtain field verification of the method at sites where the discharge was already known by current-meter measurement. Certain unexplained inconsistencies brought out in these investigations resulted in a program of fundamental laboratory research on the method. This research[1] was carried out in the hydraulic laboratory at the Georgia Institute of Technology.

This report describes a new procedure for computing peak discharges at contractions based on the laboratory study. The procedure has been verified by surveys at a limited number of bridge openings where the peak discharge was known from current-meter measurement. Additional field vertification of the method is being sought and it is possible that the procedure presented in this report may be modified in the future on the basis of additional data obtained either in the field or the laboratory.

THE DISCHARGE FORMULA

When computing peak discharge at a contraction, the drop in water-surface level between an upstream section and a contracted section is related to the corresponding change in velocity. The discharge equation results from writing the energy and continuity equations for the reach between these two sections, designated section 1 and [2]3 on figure 1:

$$Q = C A_3 \sqrt{2g\left(\Delta h + \alpha_1 \frac{V_1^2}{2g} - h_f\right)} \qquad (1)$$

in which,

Q = discharge in cfs;

C = coefficient of discharge;

A_3 = gross area of section 3 in square feet; this is the minimum section parallel to the constriction between the abutments and is not necessarily at the downstream side of the bridge;

Δh = difference in elevation of the water surface between sections 1 and 3 in feet;

$\alpha_1 \frac{V_1^2}{2g}$ = weighted average velocity head in feet at section 1, where V_1 is the average velocity, Q/A_1, and α_1 (alpha) is a coefficient which takes into account the variation in velocity in that section (see page 4 for discussion of α_1).

h_f = the head loss in feet due to friction between sections 1 and 3.

The procedure recommended for evaluating each term in the discharge equation is described in detail below.

LOCATION OF SECTIONS

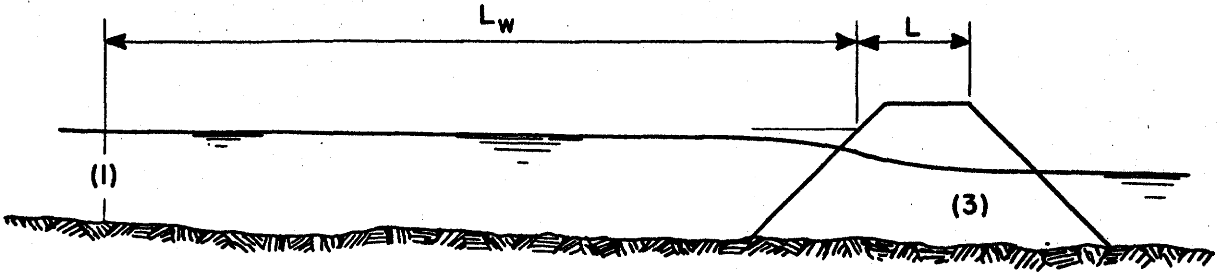
Approach Section (1)

As there is an appreciable variation in elevation of the water-surface across the channel within the drawdown zone, it is important to locate the approach section upstream from the beginning of drawdown. Figure 2, based on the laboratory tests, shows that the distance to the beginning of drawdown is related to the width of the bridge opening b and the degree of channel contraction m. It also shows that the drawdown curve is shorter along the face of the constriction than along the center line of the channel. In order that section 1 be located above the drawdown zone for all degrees of

[1] Kindsvater, C. E., and Carter, R. W., 1952 Tranquil flow through open-channel constrictions, Convention Preprint no. 21, Am. Soc. Civil Engineers.

[2] Formerly, a cross section would be taken at the upstream side of bridge and designated section 2. The method outlined in this report does not use a section 2, but the former system of designating the approach section as 1 and the contracted section as 3 has been retained.

Note.--The investigation was under the general direction of C. E. Kindsvater, Professor of Civil Engineering, Georgia Institute of Technology, acting as a consultant under part-time employment with the U. S. Geological Survey. The work was carried out by R. W. Carter and H. J. Tracy, assisted in the laboratory by personnel of the Atlanta district, Geological Survey and several student assistants.



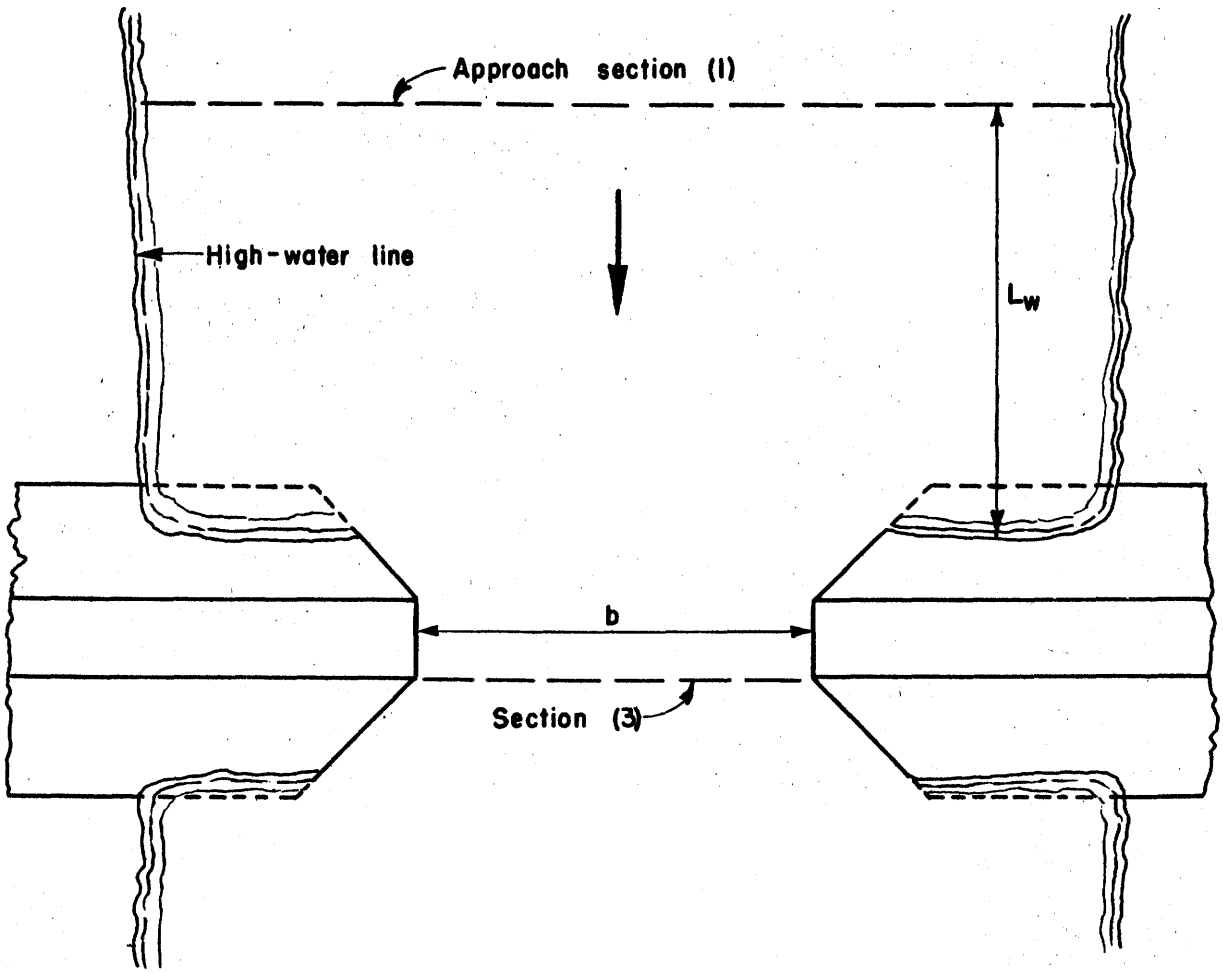


Figure 1. --Definition sketch of an open-channel constriction.

channel contraction, it is recommended that it be located at least one bridge-opening width b upstream from the constriction. This section is considered to include the entire width of the valley perpendicular to the line of flow. Where the degree of contraction is large, section 1 may be omitted if the difference in roughness at section 1 and section 3 is not great. A large degree of contraction is indicated if the water-surface profile is level for some distance along the upstream side of the embankment. For this condition the upstream water-surface elevation may be taken on the embankment at a point one b distance from the center of the opening. In absence of an approach section the factor $(A_3/A_1)^2$ is assumed to be zero, the conveyance ratio K_3/K_1 is assumed to be 1/10, the percent of channel-contraction m is assumed to be 80, and a length L_w, equal to the width of the bridge b, is assumed.

When the degree of contraction is large and the approach section lies in a zone of heavy vegetation, the friction loss from section 1 to section 3 will be a large part of the total fall and can no longer be estimated. However, surveying an approach section through such a heavily wooded zone would be a formidable task. Often a cross section along the edge of the right-of-way will be representative of a section b distance upstream insofar as ground elevations are concerned, and the ground elevation of the approach section may be approximated in this manner. The value of n selected for the approach section should be representative of the upper half of the reach from the proper location of the approach section to the bridge opening. The water-surface elevation of the approach section may be taken along the embankment at a point one b distance from the center of the bridge opening.

Where there is more than one bridge opening or if water is flowing over part of the embankment, section 1 must be divided into separate approach channels to the various openings. This is accomplished by assuming that the discharges in the subsections of section 1 are proportional to the conveyances of the subsections. An assumed discharge may thus be distributed across section 1 and the width of the approach section to each opening apportioned on basis of the assumed discharge through the opening. This process is repeated until the assumed discharge and the computed discharge are in agreement.

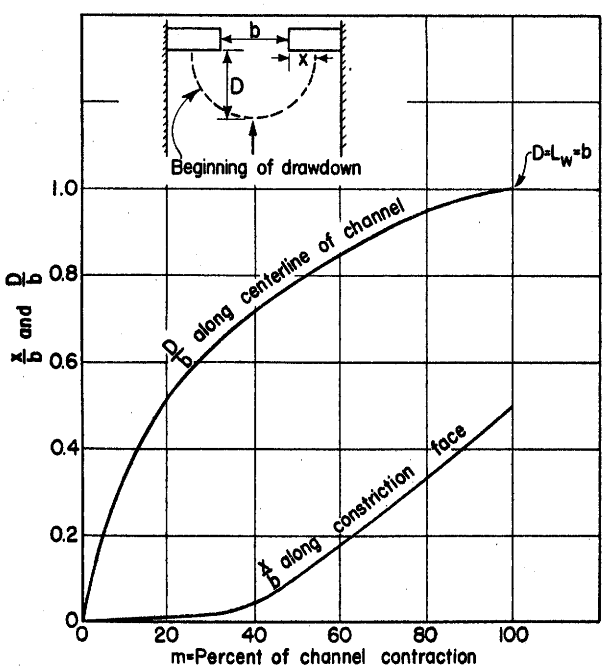


Figure 2.--Distance to beginning of drawdown along center line of channel and along the embankments.

Contracted Section (3)

Section 3 is defined by a straight line parallel to the constriction which marks the minimum area between the two abutments of the bridge. The area, A_3, to be used in the discharge equation is always the gross area of the section, without regard to the area occupied by piles or piers if they lie in the plane of the section. Similarly, if the lower portion of the bridge is submerged, the area at section 3 is considered to extend to the level of the free water surface as determined by the methods described below. However, in computing the conveyance of section 3 the areas of the piers, piles, and submerged portion of the bridge are deducted from the section.

At some sites large scour holes under the bridge make the selection of the minimum section doubtful. Under extreme conditions of scour, where a minimum section could be taken around the lip of the scour holes upstream from the embankment, the bridge geometry no longer forms the "control" for the flow through the constriction and the coefficients presented in this manual are not applicable. Such sites should be avoided until further investigation of this special problem is completed.

DETERMINATION OF WATER-SURFACE LEVELS

The determination of the water levels at sections 1 and 3 should be based on the criteria described below in order that the coefficients derived from the laboratory studies be applicable.

Average water-surface elevations at sections 1 and 3 will ordinarily be determined from high-water marks on the boundaries of the channel and the constriction. To make best use of the high-water marks, they should be plotted to yield a profile of the maximum water level along the river banks and the embankments. Elevations corresponding to sections 1 and 3 can then be taken from these profiles.

A typical water-surface configuration in the vicinity of a constriction is shown in figure 3. As shown in the illustration average level at section 1 will be clearly indicated by the high-water profiles along the upstream boundaries.

Where the ratio of abutment length L to bridge width b is small, as in figure 3 and 4, there will be an appreciable variation in level across the opening at section 3. As indicated in figure 3, however, the super-elevation of the stream at the center will disappear a short distance below the opening at the vena contracta. It is assumed in the derivation of the discharge equation that the elevation of the water surface at section 3 corresponds to the level at the vena contracta. The laboratory observations have shown that the level at the vena contracta can be very closely approximated by the level of the water on the downstream face of the embankment adjacent to the abutment. This means that the level at section 3 is conveniently determined from the high-water profiles for the downstream embankment. In this connection, one exception should be noted. Where the opening approaches the degree of eccentricity illustrated by figure 4, with a constriction on only one side of the channel, the water surface will be superelevated at the point designated (*A*) in figure 4. In this case, the level at (*B*) should be taken as the average for section 3.

COMPUTATION OF THE VELOCITY HEAD

The weighted average velocity head at section 1 is defined in the discharge equation as $\alpha_1 \frac{V_1^2}{2g}$, where V_1 is the average velocity in the section and α_1 (alpha) is a coefficient which accounts for the nonuniformity of velocity in the section. The value of α for any section is equal to

$$\alpha = \frac{\int^Q v^2 dQ}{V^2 Q}$$

This is equivalent to weighing the velocity head according to the discharge of the subsection. However, in the present application, neither the velocity distribution nor the discharge is known.

α_1 may be approximated if the cross section is first divided into subsections, *a*, *b*, c .... *n*, of approximately constant hydraulic properties. Alpha is then computed from the equation

$$\alpha_1 = \frac{\frac{K_a^3}{A_a^2} + \frac{K_b^3}{A_b^2} \cdots\cdots\cdots + \frac{K_n^3}{A_n^2}}{\frac{K_1^3}{A_1^2}} \qquad (2)$$

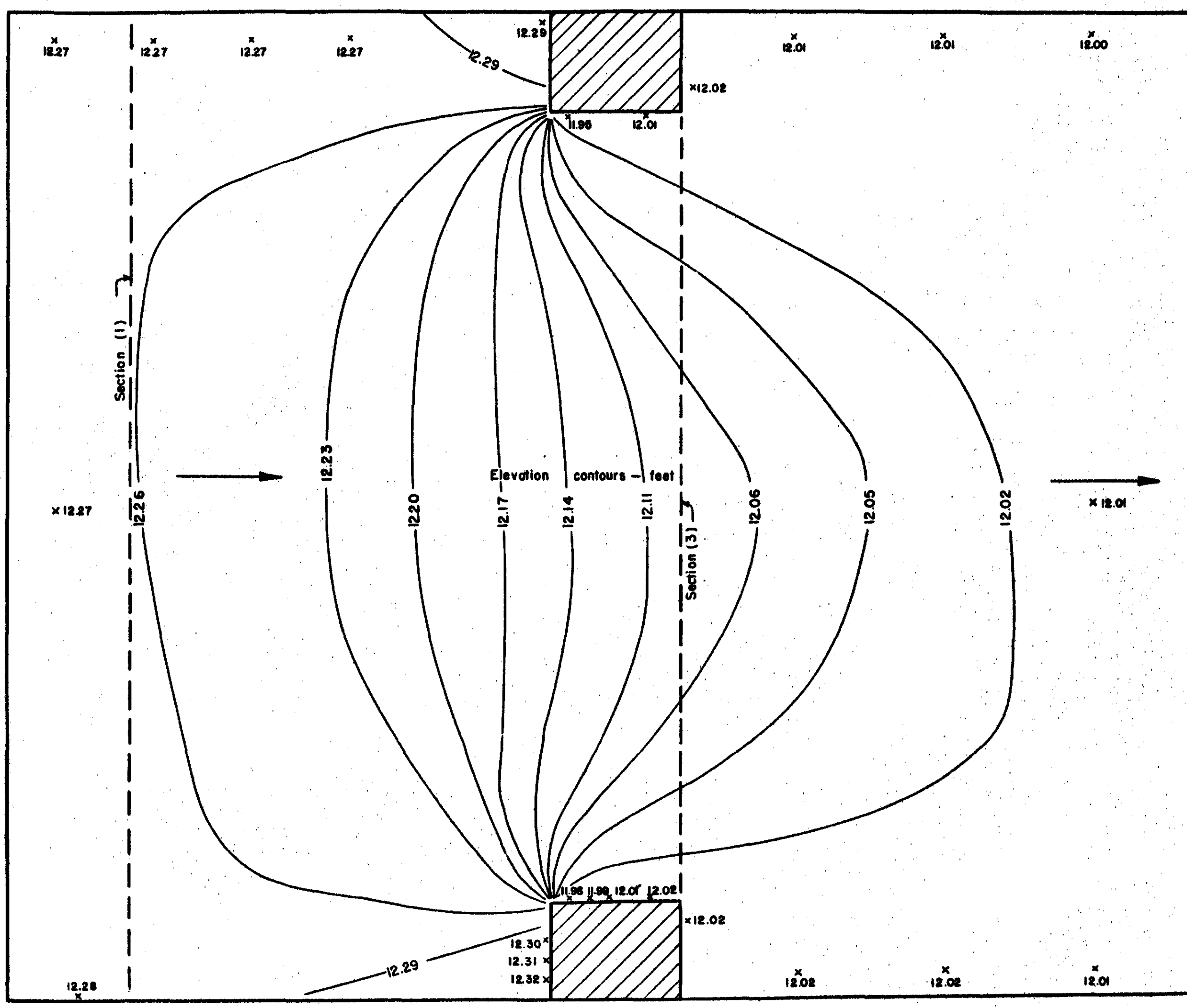


Figure 3. --Typical water-surface configuration in the vicinity of an open-channel constriction.

where K_a, K_b, and K_n, are the conveyances of the component sections of areas A_a, A_b, and A_n, respectively: $K_1 = K_a + K_b \ldots + K_n$ = the total area of section 1; and $A_1 = A_a + A_b \ldots + A_n$ = the total area of section 1. Here conveyance is defined in terms of the Manning formula as $K = \frac{1.486}{n} AR^{2/3}$, where R is the hydraulic radius and A is the area of the section. This method assumes that the velocity within subsection a, b, c, or n is constant and that the discharge of each subsection is proportional to its relative conveyance.

COMPUTATION OF THE FRICTION LOSS

The friction loss term, h_f, in the discharge equation is defined as the total loss of head due to friction between section 1 and 3. This loss is computed in accordance with standard Survey procedure. The distance between the two sections is divided into two reaches: the approach reach from section 1 to the upstream side of the bridge opening, and the bridge opening reach (fig. 1). The average friction slope in the approach reach is taken to be the geometric mean of the computed slopes at the end sections. It follows that the total head loss due to friction will be obtained from the equation:

$$h_f = L_w\left(\frac{Q}{K_w}\right)^2 + L\left(\frac{Q}{K_3}\right)^2 \qquad (3)$$

where L_w is the length of the approach reach, L is the length of the abutment, and $K_w = \sqrt{K_1 K_3}$, in which K_1 and K_3 are the total conveyances of sections 1 and 3. If section 3 contains piers or piles, or if the bridge is partially submerged, the section is subdivided at the edge of each pier or pile bent, and the area term in the conveyance equation will be computed as the gross area, A_3, minus the total submerged area of the obstructions projected on the plane defined by section 3. In this case, too, the wetted perimeter used in computing the hydraulic radius, R, will include the lengths of the sides of the piles, piers, or bridge surfaces in contact with the water. It is again noted that section 3 is always taken parallel to the bridge.

Laboratory tests indicate that about one-half the total fall from section 1 to section 3 occurs between section 1 and the upstream side of the bridge. This may be used as a rule of thumb in deciding whether the lower bridge members were submerged when it cannot be determined from field evidence.

THE DISCHARGE COEFFICIENT

Definitions

In the derivation of the discharge equation on the basis of the Bernoulli energy equation and the equation of continuity, the coefficient C is taken to represent the combination of (1) a coefficient of contraction, (2) a coefficient which takes into account the eddy losses due to the contraction, and (3) the velocity-head coefficient, α_3, for the contracted section. It follows from a dimensional analysis of the factors which influence the flow pattern that C can be expressed as a

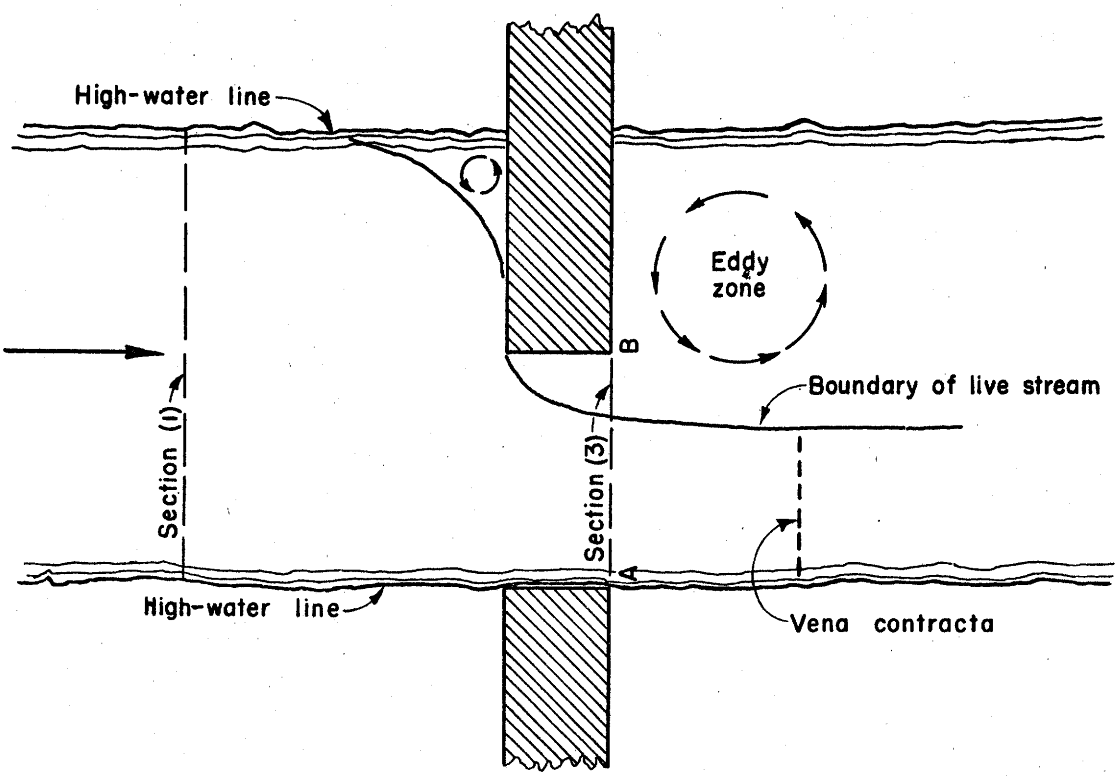


Figure 4. --Definition sketch of an open-channel constriction with a limiting condition of the eccentricity ratio (e = 0)

function of certain governing geometric and fluid parameters. Thus, in functional notation

$$C = f\left(m, \frac{L}{b}, \frac{r}{b}, \frac{W}{h}, \theta, \frac{x}{b}, \frac{y_a + y_b}{2b}, \frac{t}{y_3 + \Delta h}, j, F, \phi, e, E\right) \quad (4)$$

The terms contained in the right-hand side of equation (4) are illustrated in figures 5-12, without which the following brief word definitions are necessarily insufficient:

m = channel-contraction ratio, expressed as percent of channel contraction (figs. 11-12);

b = width of bridge opening, defined as the distance between abutment faces (figs. 5-8);

L = length of abutment, variously defined for different types of bridge openings (figs. 5-8);

r = radius of rounding of entrance corner of abutment for vertical-faced constrictions (fig. 22);

W = a convenient measure of the length of a wing wall or chamfer (fig. 8);

θ = the acute angle between a wing wall and the plane of the constriction (fig. 8);

x = horizontal distance from the point of intersection of the abutment and embankment slopes to a point on the upstream embankment having the same elevation as the water surface at section 1 (fig. 7);

y_a, y_b = depths of water at the toe of each abutment (fig. 6);

y_3 = average depth of water in section 3, $= \frac{A_3}{b_t}$, where A_3 is the gross area and b_t is the top width of section 3 (fig. 7);

t = vertical distance between water level at section 1 and lowest horizontal member of a partially submerged bridge, (fig. 30B);

j = ratio of the projected area of the submerged portions of piers or piles in the bridge opening to the gross area of the bridge opening = A_j/A_3;

F = the Froude number of the contracted section = $Q/(A_3\sqrt{gy_3})$

It is necessary to assume a discharge for initial computation of the Froude number. The final value of the Froude number can be determined by successive approximation. If the value of the Froude number is greater than 0.8, critical velocities may occur in section 3 and the method outlined in this report should not be used to determine the peak discharge.

Φ = the acute angle between the plane of the constriction and a line normal to the thread of the stream (fig. 9);

e = an eccentricity ratio = $K_a/K_b \leqq 1.0$ (fig. 10);

E = a symbol representing the slope of the embankments expressed as, for example, 2 : 1, this being a ratio of the horizontal distance to

Δh = difference in elevation of water surface between section 1 and section 3.

Other symbols contained in the figures but not included in equation (4) are:

A_j = the area of piers or piles in section 3;

h = the height of water surface above an arbitrary datum;

z = the difference between h and the average depth, y, at a section.

b_t = width of bridge opening at the water surface.

Classification of Bridge Openings

The only ratios included in equation (4) which are common to all types of bridge openings are m, L/b, and F. Of these, m is the most critical and F is of the least general significance as determined by the laboratory studies. Thus, m and L/b have been selected as primary variables for the presentation of the experimental results. The remaining ratios in equation (4) are descriptive of the geometric properties of the various types of bridge openings. In this manual, therefore, it is convenient to classify all bridge openings into four categories representing the distinctive features of their major geometric characteristics.

Type I--vertical embankments, vertical abutments (fig. 5).

Type II--sloping embankments, vertical abutments (fig. 6).

Type III--sloping embankments, sloping abutments (fig. 7).

Type IV--sloping embankments, vertical abutments with wing walls (fig. 8).

Percent of Channel Contraction

The importance of m as a primary independent variable governing the magnitude of the discharge coefficient is pointed out above. It is the function of the channel-contraction ratio to describe the relative degree of contraction imposed by the constriction on the normal river channel. As used here, m is a measure of the proportion of the total flow which is required to enter the contraction from the lateral regions or the regions upstream from the embankments. Thus,

$$m = \frac{Q - q}{Q} = 1 - \frac{q}{Q},$$

where Q is the total discharge and q is the discharge that could pass through the opening without contraction.

For the normal river-channel cross section composed of one or more deep channels and adjacent shallows, the total discharge is assumed to be distributed across the approach section in proportion to the conveyance of the arbitrarily defined subsections. This assumption is based on the approximation that the energy slope is constant across the section. Thus, as illustrated in figures 11 and 12, if K_q is taken to be the conveyance of the subsection occupied by q and K_Q is the total conveyance of the approach section, $m = 1 - K_q/K_Q$, or, expressed as a percentage in order to indicate "percent of channel contraction",

$$m = 100\left(1 - \frac{K_q}{K_Q}\right) \quad (5)$$

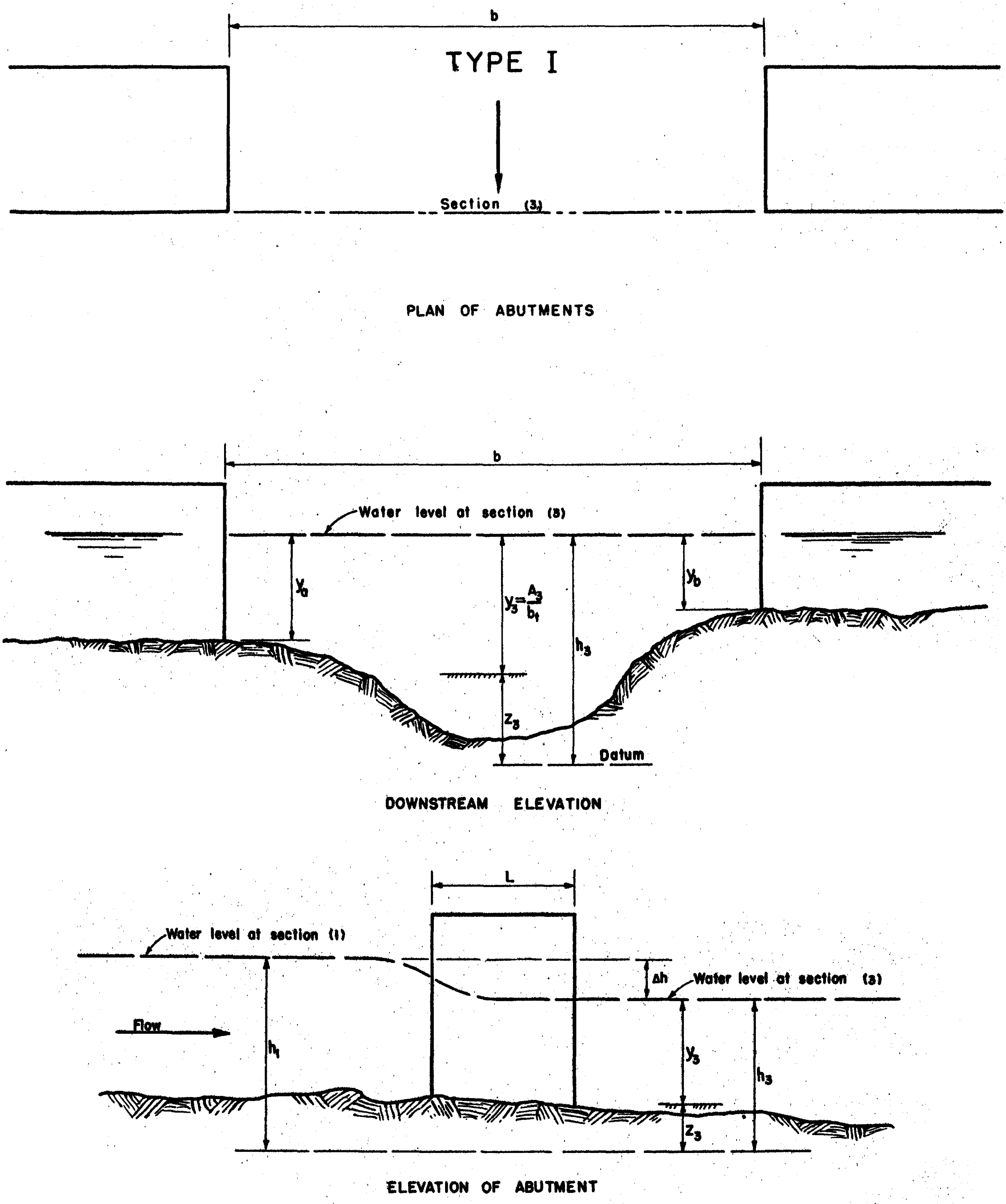


Figure 5. --Definition sketch of a Type I opening, vertical embankments and vertical abutments.

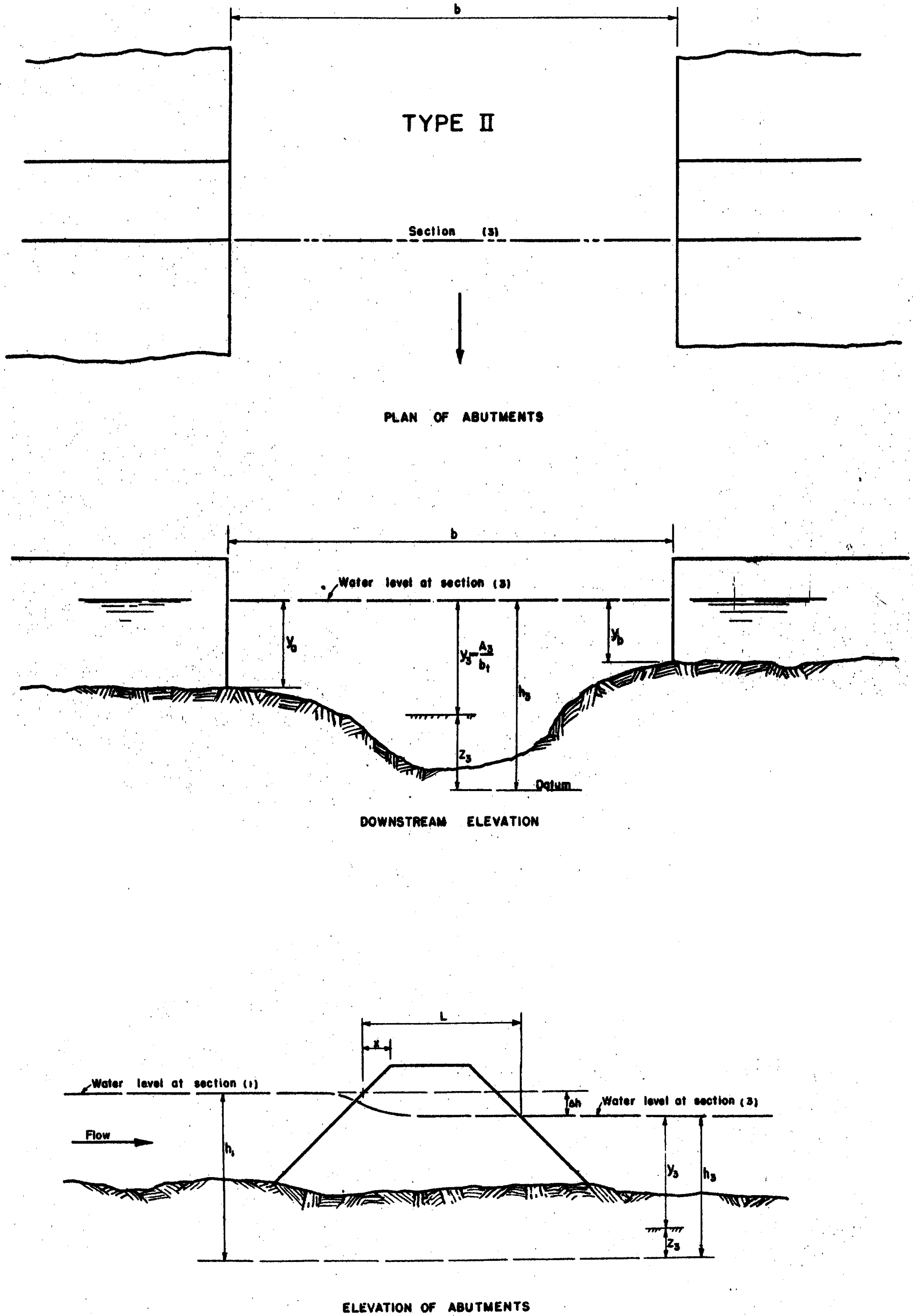


Figure ,.--Definition sketch of a Type II opening, sloping embankments and vertical abutments.

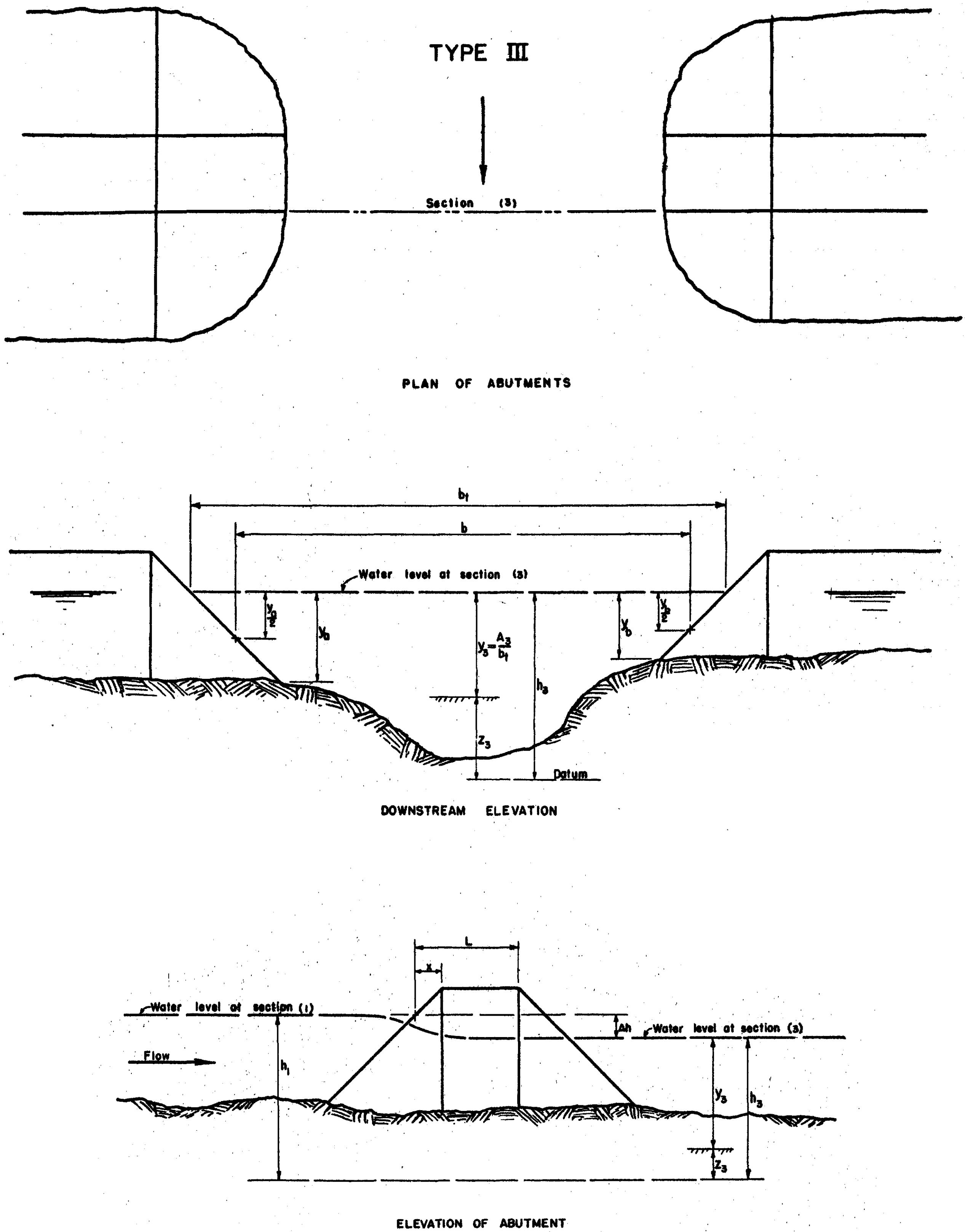


Figure 7.--Definition sketch of a Type III opening, sloping embankments and sloping abutments.

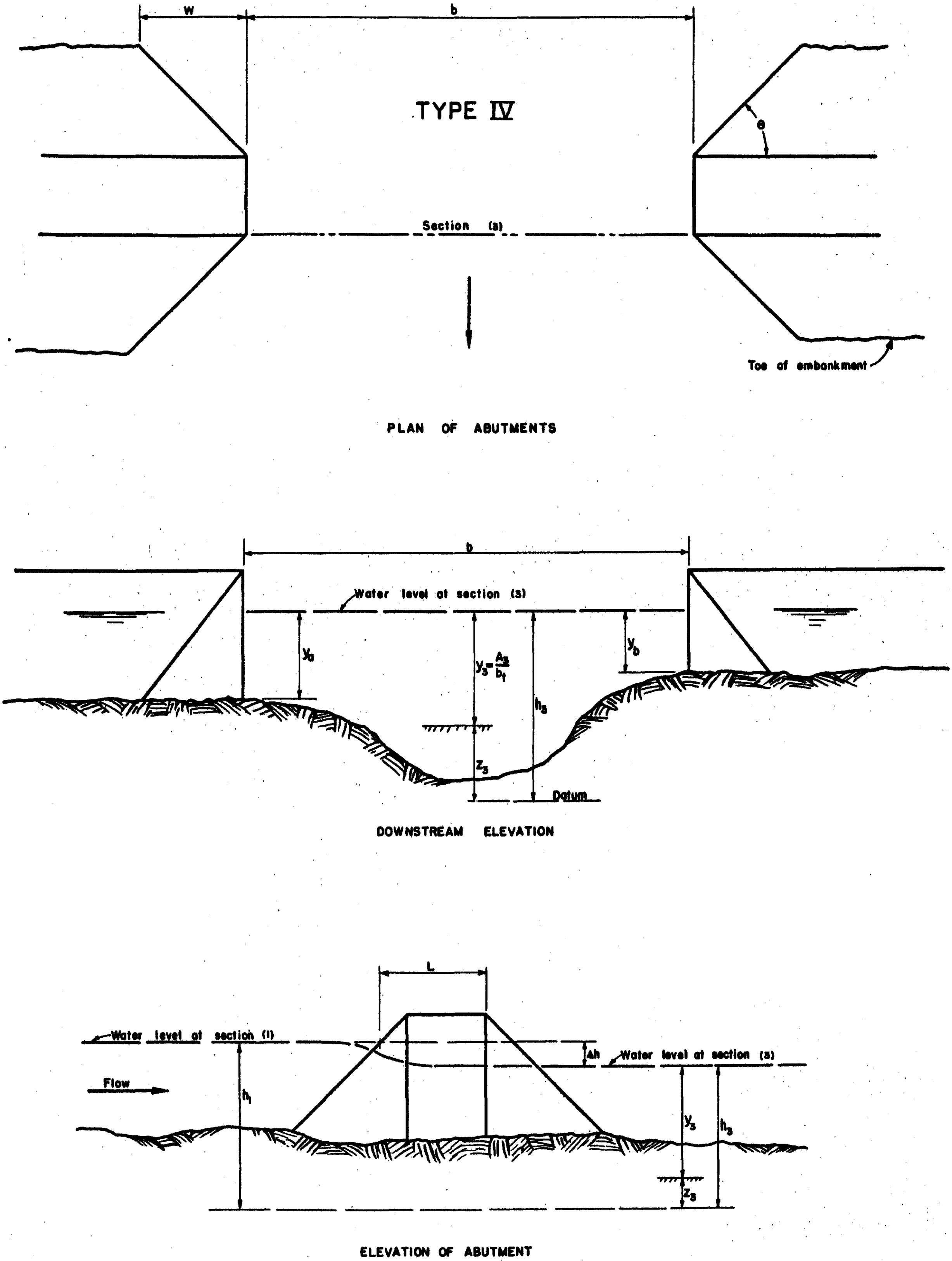


Figure 8. --Definition sketch of Type IV opening, sloping embankments and vertical abutments with wing walls.

Evaluation of the Discharge Coefficient

The process of evaluating the discharge coefficient, C, involves, first, the determination of a standard value, C', corresponding to the given values of the two primary variables, m and L/b, and certain arbitrary "standard" values of all the other pertinent variables. These standard-value relationships are called base curves and are shown in figures 22-30 for the four types of bridge openings. The standard values of the secondary variables used for each type bridge opening are shown on each base curve. These secondary variables listed under "standard conditions" on the base curves are the only variables to be considered for each type abutment.

After C' is determined from the base curve it is successively adjusted for the effect of each of the secondary variables on the basis of adjustment curves also shown in figures 22-30. The adjustment process consists of multiplying C' by the adjustment coefficients. For example, if a certain bridge departs from the standard conditions only with respect to the Froude number and angularity, then the discharge coefficient will be obtained from the equation,

$$C = C' \cdot k_F \cdot k_\Phi$$

where:

C' = the standard value of the coefficient of discharge corresponding to the given values of m and L/b and the standard values of each of the remaining variables, including F and φ;

k_F = a coefficient that adjusts C' for the influence of a nonstandard value of F;

k_Φ = a coefficient that adjusts C' for the influence of angularity.

For those bridges which involve sloping embankments or abutments, curves are presented for 1 : 1 and 2 : 1 slopes. The vertical-faced constriction (0 : 1 slope) is a limiting slope condition. When an intermediate embankment slope is encountered it will be necessary to obtain C by interpolation. When the abutment slope is different than the embankment slope, an average slope should be used. An average slope for the two ends of the bridge may also be used. Where the two abutments are of different type, a C should be computed for each side, a and b, and the final C obtained by

$$C = \frac{C_a K_a + C_b K_b}{K_a + K_b}$$

It is emphasized that certain combinations of the empirical adjustment coefficients applied to C' will appear to yield a value of C greater than 1.0. This is unreasonable, of course. A value of $C = 1.0$ should be taken as the maximum under all circumstances.

Many bridge openings cannot be classified exactly in any of the four major categories described above. The discharge coefficient must be obtained by interpolation based on a knowledge of the relative effects of the factors that influence the flow pattern. For example, m and L/b are the most critical variables and a reasonable estimate of the adjustment coefficients will give results which are within the range of accuracy expected of the method.

The discharge equation on page 1 requires a trial and error solution. However, if Q/A_1 is substituted for V_1 and $L_w\left(Q^2/K_1K_3\right) + L\left(Q/K_3\right)^2$ for h_f an equation which may be solved directly is obtained. Thus

$$Q = 8.02\, CA_3 \sqrt{\frac{\Delta h}{1 - \alpha \cdot C^2 \left(A_3/A_1\right)^2 + 2gC^2 \left(A_3/K_3\right)^2 \left(L + L_w K_3/K_1\right)}}$$

The equation could be simplified further, but in its present form the effect of the approach velocity and friction loss can be identified.

FIELD AND OFFICE PROCEDURE

A step-by-step outline of the procedure for determining the peak discharge at constrictions is given to aid in obtaining the proper information in the field survey and to serve as a guide in the computation of peak discharge.

Field Survey

It is recommended that a transit be used in making a location and elevation survey of the site. The two-peg method of surveying is recommended, and if this method is not used, as where only one instrument set-up is made, the adjustment of the transit should be checked before beginning the survey.

(1) Locate and obtain the elevation of floodmarks near the constriction. Obtain floodmarks in the vicinity of the approach section (about one b distance upstream) and along the upstream and downstream side of the embankment.

(2) Locate the river channel, the bridge opening and all features pertinent to the hydraulics of the site.

(3) Obtain ground elevations for cross sections at sections 1 and 3. Section 1 should extend completely across the channel and the flood plain and should be one "b" distance upstream from the bridge opening. For exceptions to this rule see page 3. 3

Section 3 is the minimum section parallel to the constriction between the abutments and is not necessarily at the downstream side of the bridge.

(4) Survey the bridge abutments and embankments in order that both plan and elevation drawings can be made. Accurate field definition of embankment properties is necessary, including elevation of roadway, top width of embankment, side slopes, abutment slopes, and elevation of toe of abutment slopes.

(5) Select values of Manning's n for sections 1 and 3.

(6) Photograph views of the channel, the embankments, the abutments, and the bridge structure. A stereo camera is recommended for this work.

Computation Procedure

Steps of procedure

(1) Plot location sketch, high-water profiles, detail of abutments and embankments, and cross sections 1 and 3. See figures 13-16 for example.

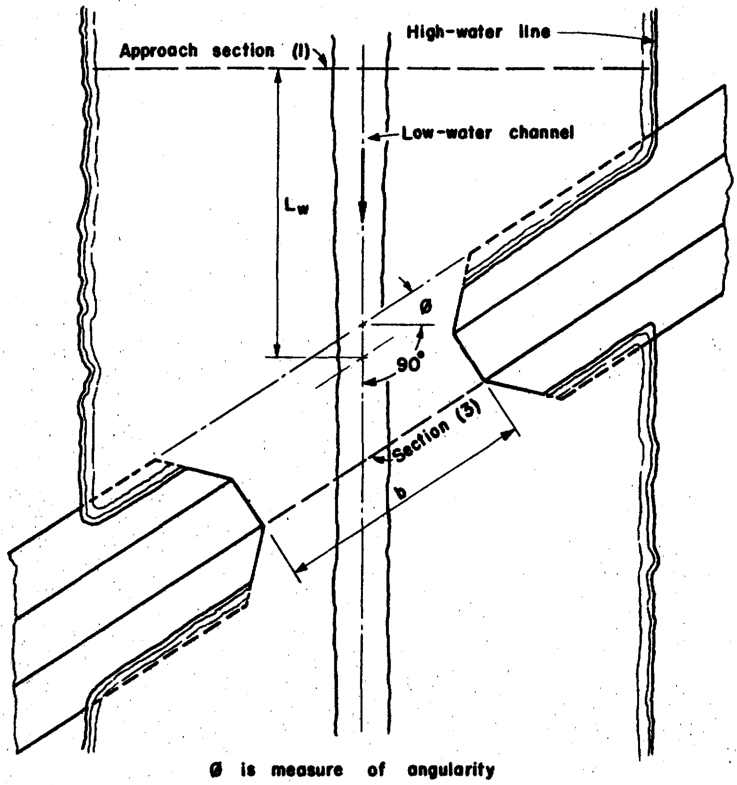


Figure 9. --Definition sketch showing a constriction at an angle to the stream.

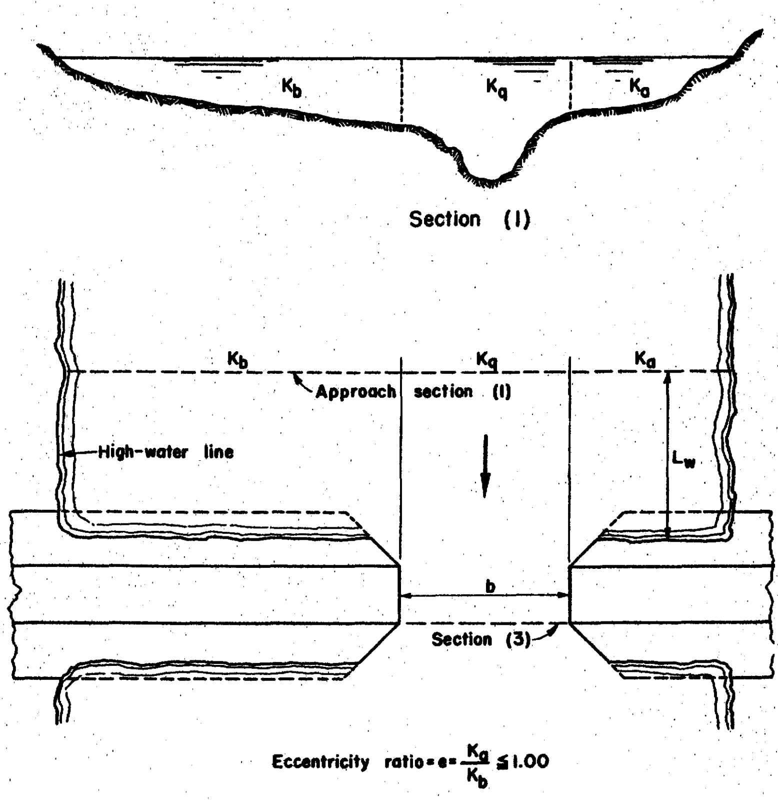


Figure 10. --Definition sketch showing the eccentricity of a constriction.

(2) Determine from high-water profiles the elevation of the water surface at sections 1 and 3.

(3) Determine b from the drawing of section 3 and lay out a length equal to b on section 1. The center of the low-water channel should occupy the same relative position on b in sections 1 and 3. The entire b distance is used regardless of the angle of skew (ϕ).

The length, b, on the approach section should embrace the flow which could pass through the opening without contraction. At sites where the low-flow channel is winding above the constriction, judgement will be required to lay out the b length in the proper position.

(4) Subdivide the approach section at the ends of b length and at abrupt changes in hydraulic properties.

(5) List the properties of section 1 on form 9-191 and compute the area and wetted perimeter.

(6) Add the areas and wetted perimeters for each of the subsections and list as shown in figure 20. Compute the conveyance of each subsection on the form. Denote whether the subsections lie within the b length or to the left or right of it by the symbols K_a, K_q, and K_b.

(7) Measure the slope of the embankment and the abutment and list the average slope as shown in figure 20.

(8) Classify the abutments as to type I, II, III, or IV (see page 6). If the abutments are different types, the value of C should be computed for each abutment as explained on page 11.

(9) Find the discharge coefficient base curve corresponding to the type and slope of the abutments as classified in (8). If the slope is intermediate between the slopes for which base curves are shown, the final value of C should be determined by interpolation.

(10) Note the "standard conditions" for which the base curve was drawn. These are the only secondary variables which must be considered in determining the discharge coefficient.

(11) List the value of the items necessary to compute the ratios shown under "standard conditions" as shown in figure 20.

(12) Compute the pertinent ratios as shown in figure 20.

(13) Determine C' from the base curve and the adjustment, k's, from the secondary curves.

(14) Determine C by multiplying C' by the proper k's.

(15) List water-surface elevations at sections 1 and 3, the discharge coefficient, the gross area of section 3, and the lengths L and L_w as shown in figure 21.

(16) Ignoring the subdivision at the ends of length b on section 1, sum the areas and wetted perimeters of the subsections and list as shown in figure 21. Compute the conveyances of the subsections. Compute α_1.

(17) List the properties of section 3 on form 9-191 and compute the areas and wetted perimeters. Subdivide the section at the edge of each pier or pile bent and at abrupt changes in hydraulic properties.

(18) Sum the areas and wetted perimeters for each subsection of section 3 and list as shown in figure 21. Compute the conveyance of each subsection.

(19) Compute the discharge by substituting in the formula as shown in figure 21.

(20) Compute the mean velocity in sections 1 and 3.

Details

The following example is given to illustrate the determination of a peak discharge at a contraction.

From information obtained in the field survey, the location sketch, the high-water profiles, the cross sections at 1 and 3, and the detailed sketch of the abutments were plotted. The drawings are shown as figures 13-16.

The subsequent steps in the computations, as illustrated on the computation forms (figs. 17-21) are as follows:

Determination of C

1. For purpose of computing the channel contraction ratio, section 1 was divided into subsections of conveyances K_a, K_b, and K_q as illustrated in figure 12. The section was also subdivided at points of abrupt change in hydraulic properties.

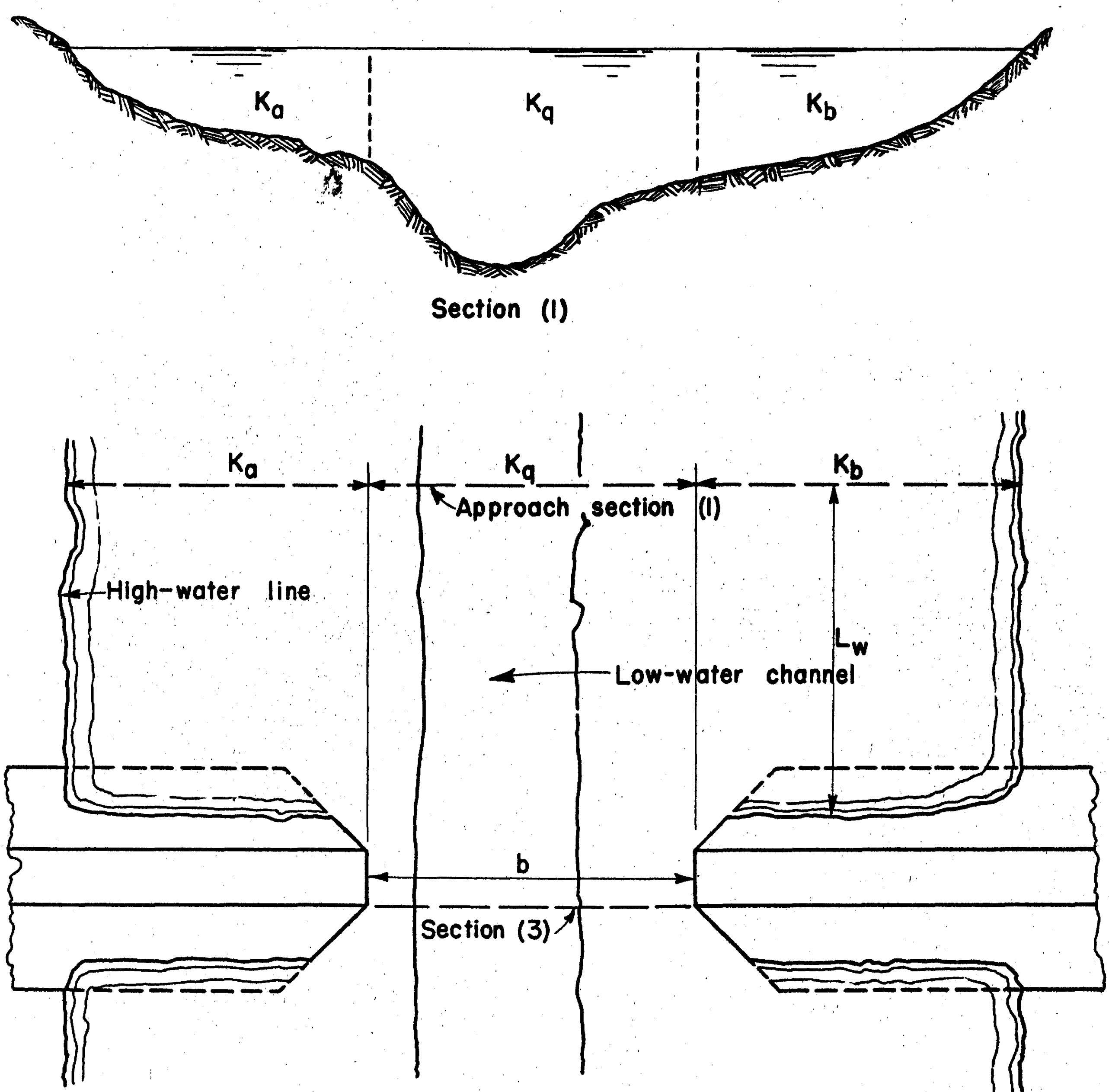
2. The embankments and abutments were illustrated in figure 16. The bridge opening was classified as a Type III opening with 1 1/2 : 1 slopes.

3. The ratios pertinent to the determination of C for this type opening are m and L/b and the ratios listed under "standard conditions" on the base curves, F, e, $\frac{t}{y_3 + \Delta h}$, ϕ, $\frac{x}{b}$, j. The dimensions necessary for the computation of these ratios were taken from the drawings.

4. For computed values of m and L/b, standard values of the coefficient of discharge, C', were determined from the Type III curves (figs. 26-27) for 1 : 1 and 2 : 1 slopes.

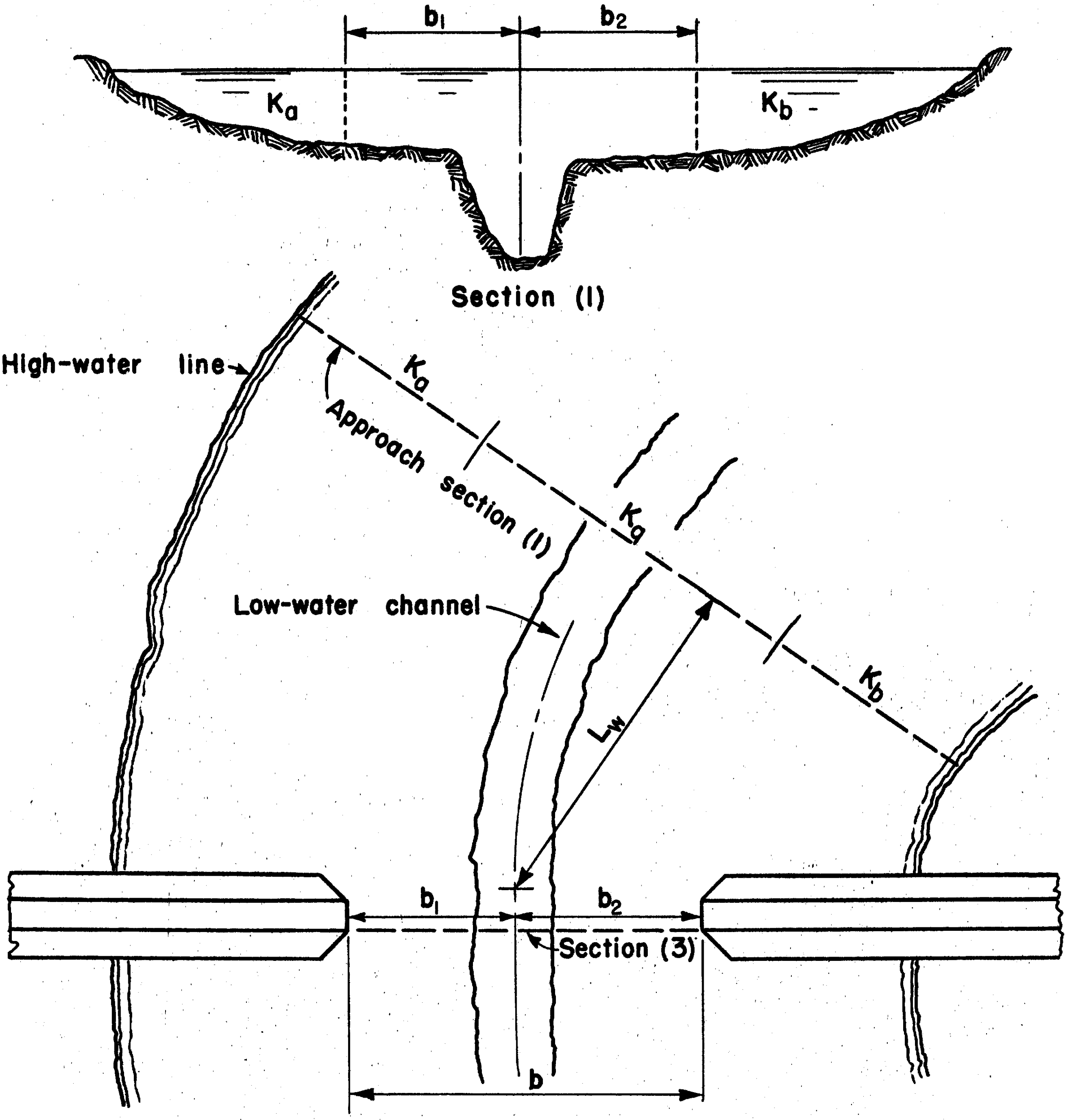
Corresponding values of k_ϕ and k_x were determined from the adjustment curves (figs. 26-27); the value of k_j was determined from the curve as shown in figure 30. The ratios F, e, and $t/(y_3+\Delta h)$ have no effect because (1) the Froude number was within the range given under "standard conditions", (2) $k_e \doteq 1.00$, and (3) the lowest bridge chord was above the water-surface elevation of the approach section.

5. The values of C for slopes of 1 : 1 and 2 : 1 were computed from the equation $C = C' \cdot k_\phi \cdot k_x \cdot k_j$. Then C for the 1 1/2 : 1 slope was taken as the average (straight line interpolation on basis of the denominator of the slope ratio).



$$m = \left(1 - \frac{K_q}{K_a + K_q + K_b}\right)100$$

Figure 11. --Definition sketch showing degree of channel contraction.



$$m = \left(1 - \frac{K_q}{K_a + K_q + K_b}\right)100$$

Figure 12.--Definition sketch showing degree of channel contraction at a bend in the stream channel.

Computation of Discharge

6. Elevations of the water surface at sections 1 and 3 which are listed on the discharge computation form were obtained from the high-water profiles. The lengths L and L_w were taken from figures 13-16. It is emphasized that the area of the submerged portions of the bridge piers was not deducted from the gross area of the contracted section, A_3.

7. To compute the total conveyance, section 3 was subdivided at the edge of each pier. For computation of conveyance of section 3, the submerged areas of the bridge piers were subtracted from the gross area of the section. Similarly, the lengths of the sides of the piers were included in the computation of the wetted perimeter.

8. The discharge was computed directly from the equation shown in figure 21. The equation is simply an algebraic transformation of the general discharge equation given in the manual.

DISCHARGE-COEFFICIENT CURVES

Base curves that give the relation between C', L/b, and m are presented for various embankment and abutment slopes. The base curves were defined under "standard conditions" of the secondary variables as listed on the figures 22-30. The value of C' from the base curve is multiplied by the various k factors when any of the secondary variables are nonstandard. The applicable k factors are defined on the same page as the base curve except for the factors k_e, k_t, and k_j, which are used with all the base curves.

The value of k_j, which accounts for the effect of bridge piles, is a function of three variables, and the use of the diagram, figure 30, may not be clear. To define k_j enter the horizontal scale at the proper value of m and move vertically to the value of L/b; then move horizontally to the line marked $j = 0.10$, then vertically to the value of j and horizontally to the value of k_j.

The following curves for definition of the discharge coefficient are presented:

Type I opening, vertical embankment, vertical abutment.

A. Base curve.

B. Variation with Froude number.

C. Variation with entrance rounding.

D. Variation with 45° wing walls.

E. Variation with 60° wing walls.

F. Variation with 30° wing walls.

G. Variation with angularity.

Type II opening, embankment slope 1 to 1, vertical abutment.

A. Base curve.

B. Variation with entrance geometry.

C. Variation with angularity.

Type II opening, embankment slope 2 to 1, vertical abutment.

A. Base curve.

B. Variation with entrance geometry.

C. Variation with angularity.

Type III opening, embankment and abutment slope 1 to 1.

A. Base curve.

B. Variation with angularity.

C. Variation with entrance geometry.

Type III opening, embankment and abutment slope 2 to 1.

A. Base curve.

B. Variation with angularity.

C. Variation with entrance geometry.

Type IV opening, embankment slope 1 to 1, vertical abutment with wing walls.

A. Base curve.

B. Variation with angularity.

C. Variation with wing-wall angle.

Type IV opening, embankment slope 2 to 1, vertical abutment with wing walls.

A. Base curve.

B. Variation with angularity.

C. Variation with Froude number.

D. Variation with wing-wall angle.

Type I through Type IV openings, k_e, k_t, and k_j curves.

A. Variation with eccentricity.

B. Variation with submergence.

C. Variation with bridge piles.

D. Variation with bridge piers.

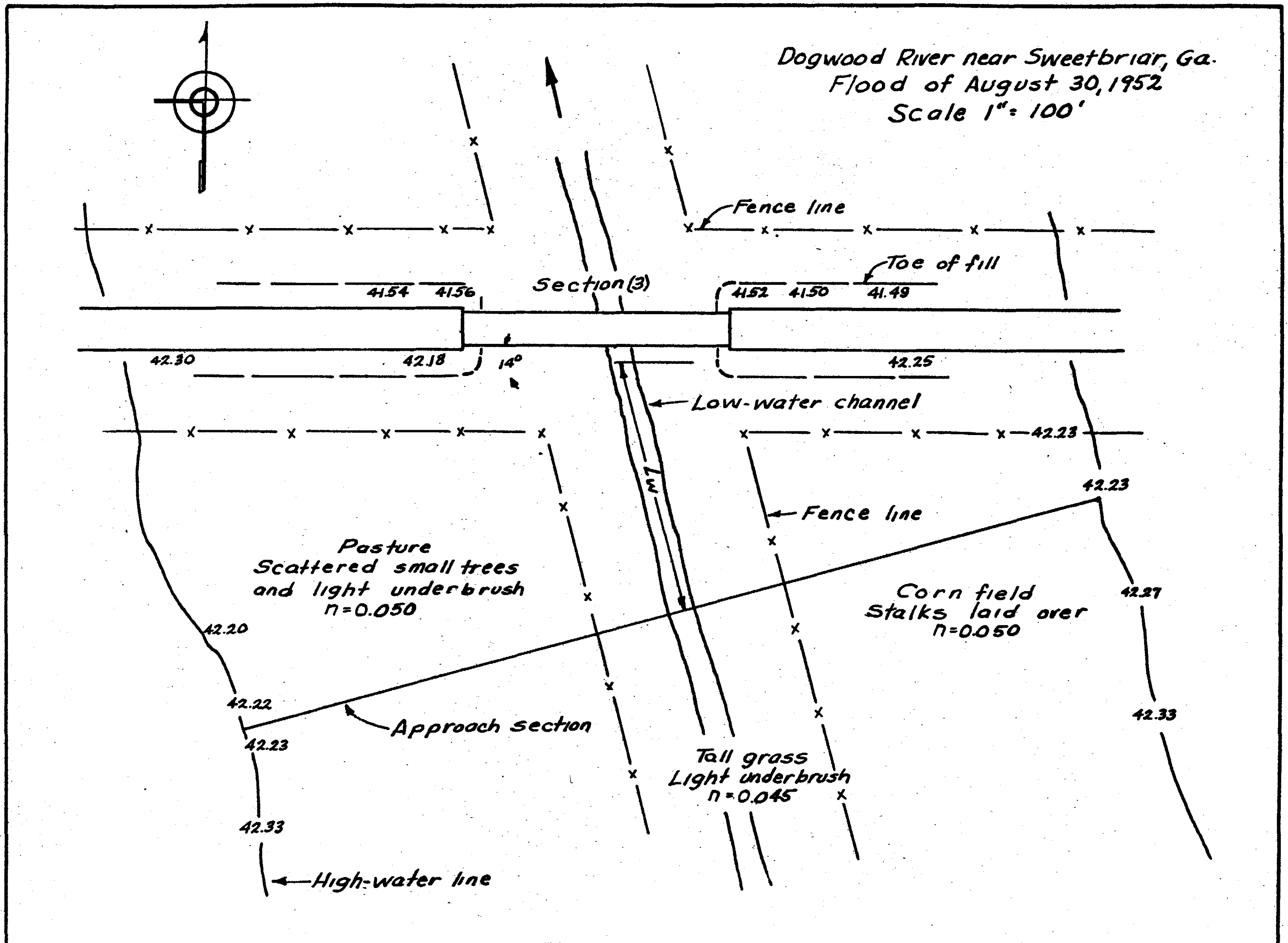
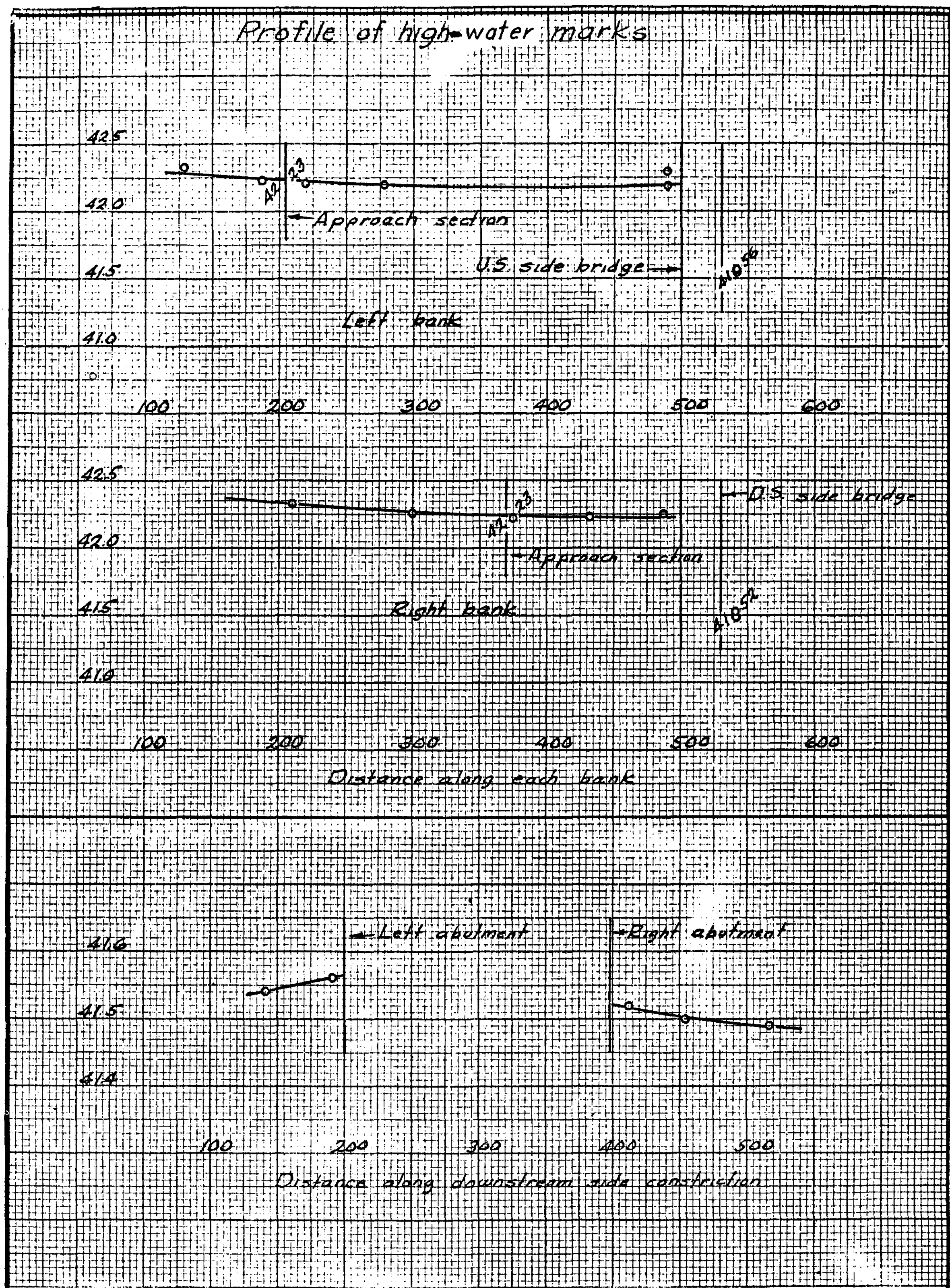


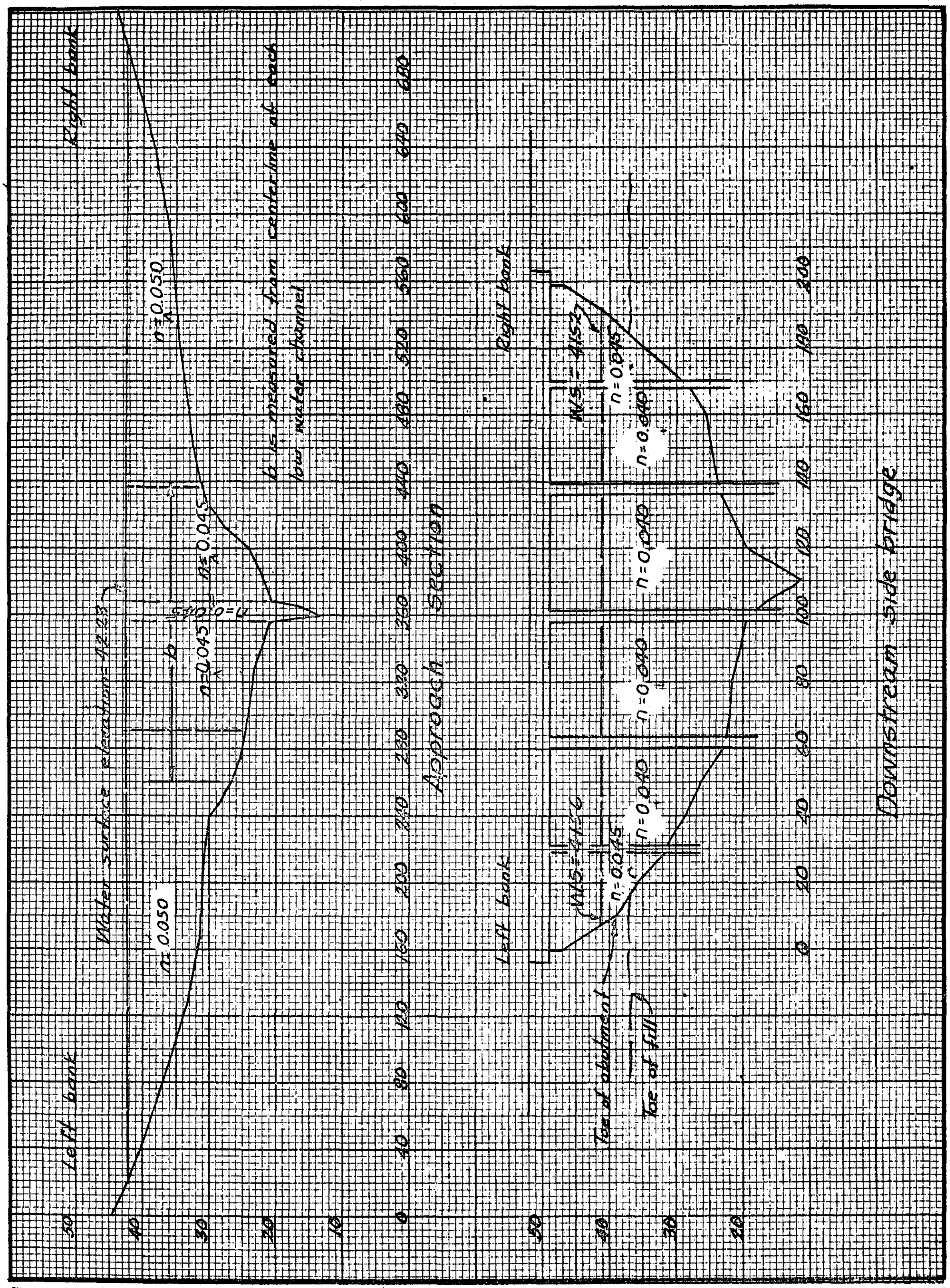
Figure 13.--Location sketch, Dogwood River near Sweetbriar, Ga.



Sheet No. .... of .... Sheets. Prepared by H.J.T. Date 11-28-52 Checked by M.N.K. Date 11-29-52

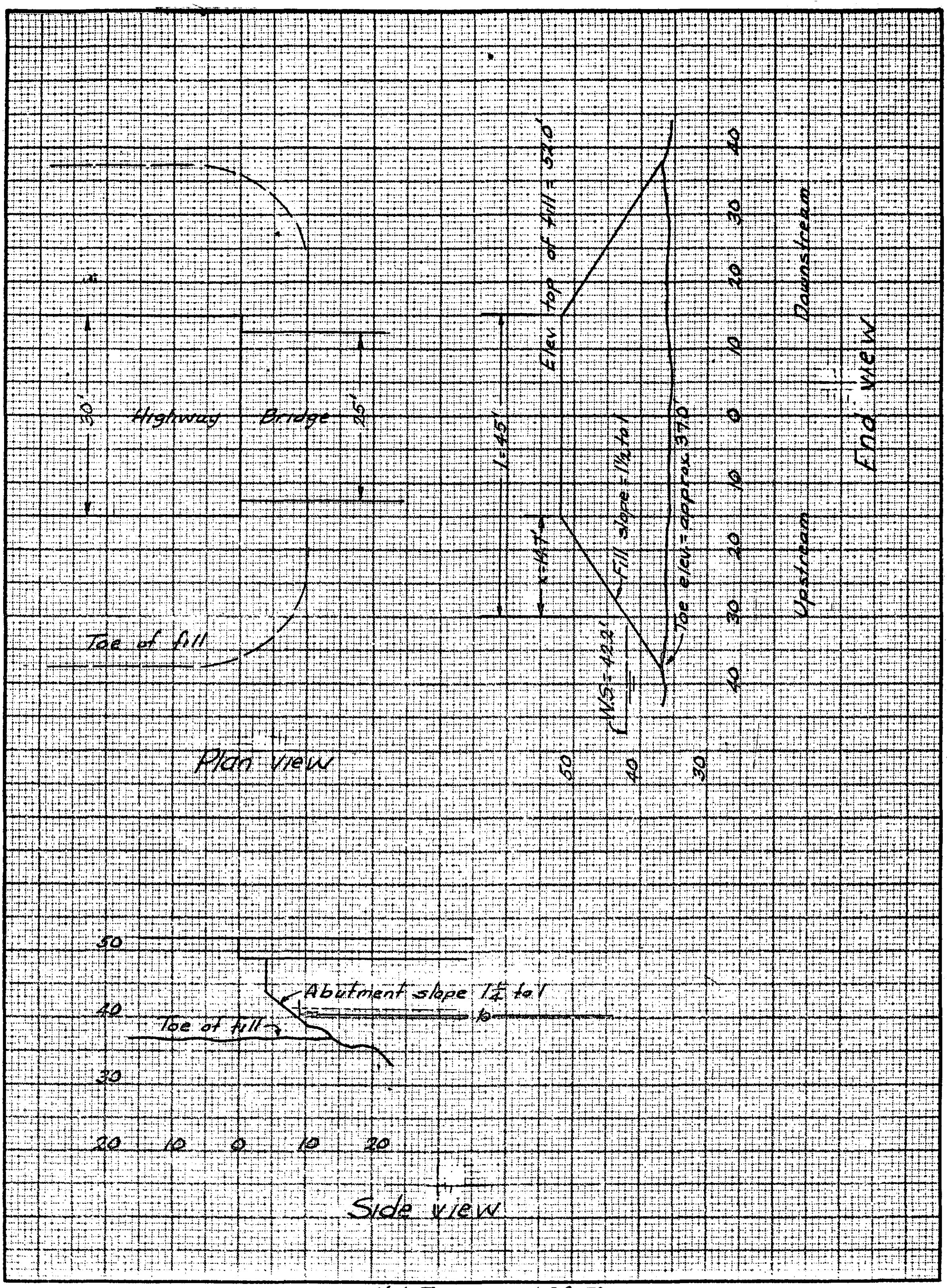
GPO : 1951 O - 94162

Figure 14.--Profile of high-water marks, Dogwood River near Sweetbriar, Ga.



Sheet No. .... of .... Sheets. Prepared by H.J.T. Date 11-28-52 Checked by M.N.K. Date 11-29-52

Figure 15.--Channel cross sections, Dogwood River near Sweetbriar, Ga.



Sheet No. .... of .... Sheets. Prepared by H.J.T. Date 11-28-52 Checked by M.N.K. Date 11-28-52

Figure 16. --Details of embankment and abutment, Dogwood River near Sweetbriar, Ga.

High-water marks

Left bank sta.*	Left bank elev.	Left bank remarks	Right bank sta.*	Right bank elev.	Right bank remarks
128	42.33		210	42.33	
186	42.23		300	42.27	
205		approach section	370		approach section
218	42.22		375	42.23	
278	42.20		432	42.23	
490	42.30	in stagnation zone	488	42.25	in stagnation zone
490	42.18		500		upstream side bridge
500		upstream side bridge	530		downstream side bridge
530		downstream side bridge	541	41.49	
542	41.54		541	41.50	
542	41.56		541	41.52	

* stationing along each bank approximately parallel to high-water lines

Along downstream side constriction

Left bank sta.	Left bank elev.	Left bank remarks	Right bank sta.	Right bank elev.	Right bank remarks
142	41.54		399		right abutment
192	41.56		411	41.52	
200		left abutment	456	41.50	
			516	41.49	

listed HJT 11-28-52

ckd. M.N.K. 11-29-52

Figure 17.--Station number of high-water marks.

SECTION.— Approach

STA.	DIST.	W. S. ELEV.	ELEV.	DEPTH	MEAN DEPTH	AREA		W. P.		
L. B. 0		42.2	44.5							
20	20		42.2	0	0					
40	20		40.2	2.0	1.0	20		20.1		
80	40		37.0	5.2	3.6	144		40.1		
128	48		33.1	9.1	7.2	345		48.1		
167	39		31.2	11.0	10.0	390		39.0		
216	49		30.9	11.3	11.2	548		49.0		
240	24		30.0	12.2	11.8	283		24.0		r: 9.28
248	8		28.1	14.1	13.2	105		8.2		
256	8		27.2	15.0	14.6	117		8.1		
260	4		26.9	15.3	15.2	61		4.0		
274	14		25.2	17.0	16.2	227		14.1		
290	16		24.7	17.5	17.2	275	2515	16.0	270.7	
328	38		23.1	19.1	18.3	695		38.0		r: 19.1
354	26		21.0	21.2	20.2	525		26.1		
356	2		20.8	21.4	21.3	43	1263	2.0	66.1	
358	2		17.5	24.7	23.0	46		3.9		
360	2		13.2	29.0	26.8	58		4.8		r: 15.4
366	6		17.0	25.2	27.1	162		7.1		
368	2		20.7	21.5	23.4	47	313	4.2	20.0	
385	17		22.3	19.9	20.7	352		17.1		
400	15		24.0	18.2	19.0	285		15.1		r: 16.3
412	12		27.9	14.3	16.2	194		12.6		
428	16		30.6	11.6	13.0	208		16.3		
437	9		31.0	11.2	11.4	103	1142	9.0	70.1	
440	3		31.2	11.0	11.1	33		3.0		
460	20		32.4	9.8	10.4	208		20.0		r: 6.35
486	26		33.2	9.0	9.4	244		26.1		
520	34		34.5	7.7	8.4	286		34.0		
588	68		36.0	6.2	7.0	476		68.0		
640	52		38.2	4.0	5.1	265		52.0		
690	50		42.2	0	2.0	100	1612	50.0	253.1	
700	10	42.2	43.8							

Sheet No.......... of .... sheets. Prepared by H J T Date 11-28-52 Checked by 11-29-52 Date M.N.K.

U. S. GOVERNMENT PRINTING OFFICE 16—66957-1

Figure 18. --Cross-section properties of approach section.

SECTION.— Contracted (downstream side bridge)

STA.	DIST.	W. S. ELEV.	ELEV.	DEPTH	MEAN DEPTH	AREA	Pier area	W. P.		
L. B. 0		41.5	47.0							
7	7		41.5	0						a: 113
10	3		39.0	2.5	1.2	4		3.9		r: 3.24
20	10		36.0	5.5	4.0	40		10.4		
29	9		31.8	9.7	7.6	69		19.5	33.8	
Pier							20			
31	0		31.2	10.3				10.3		
40	9		28.6	12.9	11.6	104		9.4		a: 414
50	10		26.2	15.3	14.1	141		10.3		r: 7.02
60	10		23.0	18.5	16.9	169		29.0	59.0	
Pier							56			
63	0		22.8	18.7				18.7		
70	7		22.0	19.5	19.1	134		7.1		a: 709
80	10		21.8	19.7	19.6	196		10.0		r: 9.29
90	10		20.2	21.3	20.5	205		10.1		
98	8		19.3	22.2	21.8	174		30.3	76.2	
Pier							68			
101	0		18.1	23.4				23.4		
104	3		16.2	25.3	24.4	73		3.6		a = 824
108	4		12.8	28.7	27.0	108		5.3		r: 10.1
112	4		12.5	29.0	28.8	115		4.0		
120	8		19.4	22.1	25.6	205		10.6		
136	16		23.2	18.3	20.2	323		34.8	81.7	
Pier							54			
139	0		23.8	17.7				17.7		a: 468
160	21		25.8	15.7	16.7	351		21.0		r: 7.76
168	8		28.0	13.5	14.6	117		21.8	60.5	
Pier							26			
170	0		29.0	12.5				12.5		a: 176
180	10		35.0	6.5	9.5	95		11.7		r = 4.65
192	12		41.5	4.5	5.5	66		13.6	37.8	
199	7	41.5	47.0	0	2.2	15				
Total						2704	224	349.0		

Sheet No. .... of .... sheets. Prepared by H J T Date 11-28-52 Checked by M. N. K. Date 11-29-52

Figure 19. --Cross-section properties of contraction section

APPROACH SECTION PROPERTIES

Subsection	n	$\frac{1.486}{n}$	a	W.P.	r	$r^{2/3}$	$K = \frac{1.486}{n} a r^{2/3}$
K_b Sta. 20-260	0.050	29.7	2,013	240.6	8.39	4.14	247,000
K_q Sta. 260-290	.050	29.7	502	30.1	16.7	6.53	97,000
K_q Sta. 290-437							603,000
K_q Sta. 437-440	.050	29.7	33	3.0	11.0	4.95	4,850
K_a Sta. 440-690	.050	29.7	1,579	250.1	6.31	3.40	159,000

1,108,500

CHARACTERISTICS OF CONSTRICTION

Embankment and abutment slope $1\frac{1}{2}$ to 1

Type of abutment III

ITEMS AND RATIOS

Items		Ratios	
Δh	.69	$m = 1 - \frac{K_q}{K_{total}}$	.37
b	180'	L/b	0.25
L	45'	r/b	
r		W/b	
W		x/b	.081
x	14.7'	$\frac{y_a + y_b}{2b}$	
y_a		$\frac{t}{y_3 + \Delta h}$	
y_b		$F = V_3/\sqrt{gy_3}$	
t		$j = A_j/A_3$	.077
y_3		$e = K_a/K_b$	.65
A_j	224	ϕ	14°
A_3	2,928	θ	
K_a	159,000		
K_b	247,000		

	C'	k_ϕ	k_x	k_j	k	k	
C =	0.780 x	1.00 x	1.12 x	.965 x	x		= 0.843 (1 to 1)
C =	0.800 x	.980 x	1.025 x	.965 x	x		= 0.776 (2 to 1)

0.810

Computed by HJF Date 11-26 Checked by F.N.K. Date 11-29-52

Figure 20.--Computation of coefficients.

File ....

Meas. No.......................

Contracted opening measurement of ....*Dogwood River near Sweetbriar, Ga.*....................

....for....*flood of August 30, 1952*..........

	Left	Right	Average
Water-surface elevation at approach section	42.23	42.23	42.23
Water-surface elevation at contracted section	41.56	41.52	41.54
Fall in water surface		Δh =	0.69

Gage height *Not gaged* feet

Discharge *15,100* second-feet

Drainage area ______ square miles

Unit runoff ______ cfs per sq. mile

C (discharge coefficient) *0.810* L_w (above contraction) *188* feet

A_3 (gross area) *2,928* L (through contraction) *45* feet

APPROACH SECTION

Subsection	n	$\frac{1.486}{n}$	a	W.P.	r	$r^{2/3}$	$K = \frac{1.486}{n} a\, r^{2/3}$	K^3/a^2	α_1
Sta. 20-290	0.050	29.7	2,515	270.7	9.28	4.43	331,000	5,730,000,000	
Sta. 290-356	.045	33.0	1,263	66.1	19.1	7.15	298,000	16,600,000,000	
Sta. 356-368	.045	33.0	313	20.0	15.4	6.19	63,900	2,680,000,000	
Sta. 368-437	.045	33.0	1,142	70.1	16.3	6.42	242,000	10,850,000,000	
Sta. 437-690	.050	29.7	1,612	253.1	6.35	3.46	160,000	1,570,000,000	
								37,430,000,000	
Total			6845			K_3 =	1,094,900	28,100,000,000	1.33

CONTRACTED SECTION $\alpha_1 = \sum(K^3/a^2) \div K^3_{total}/A^2_{total}$

Subsection	n	$\frac{1.486}{n}$	a	W.P.	r	$r^{2/3}$	K		
Sec. 1	0.045	33.0	113	33.8	3.34	2.24	8,350		
Sec. 2	.040	37.1	414	59.0	7.02	3.67	56,400		
Sec. 3	.040	37.1	709	76.2	9.30	4.44	117,000		
Sec. 4	.040	37.1	824	81.7	10.1	4.66	142,000		
Sec. 5	.040	37.1	468	60.5	7.74	3.90	67,800		
Sec. 6	.045	33.0	176	37.8	4.65	2.78	16,200		
Total						K_3 =	407,750		

Use net areas for contracted subareas

V_1 = *2.21* V_3 = *5.58*

COMPUTATION OF DISCHARGE

$$Q = 8.02\, C A_3 \sqrt{\frac{\Delta h}{1 - \alpha_1 C^2 (A_3/A_1)^2 + 2gC^2(A_3/K_3)^2(L + L_w K_3/K_1)}}$$

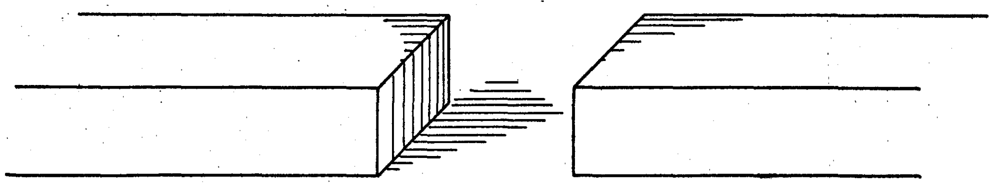
$$= \frac{8.02 \times .810 \times 2928 \times \sqrt{0.69}}{\sqrt{1 - 1.33(.810)^2\left(\frac{2928}{6845}\right)^2 + \frac{64.3(.810)^2(2928)^2}{(407{,}750)^2}\left(45 + 188\,\frac{407{,}750}{1{,}094{,}900}\right)}} = \frac{15{,}800}{\sqrt{1 - .159 + .249}}$$

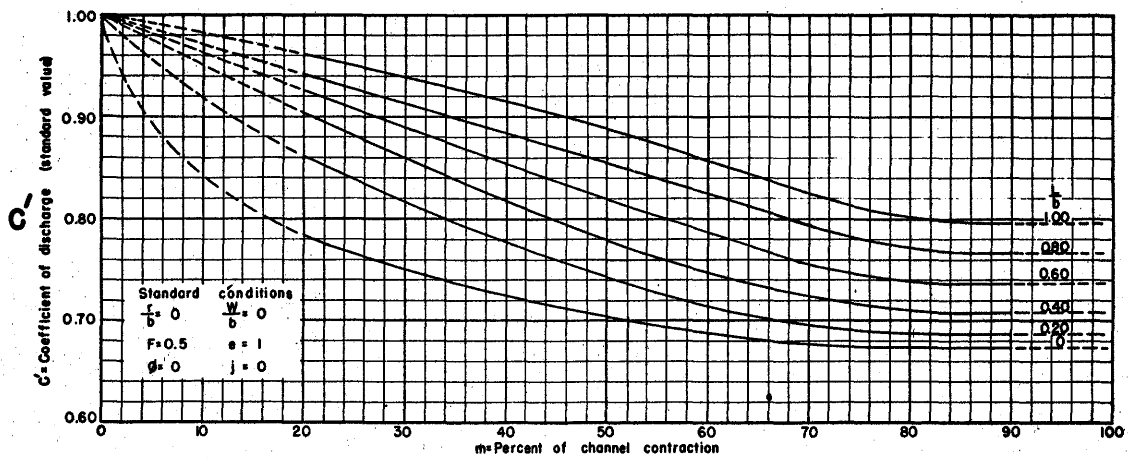
$$= 15{,}100$$

Remarks: ____

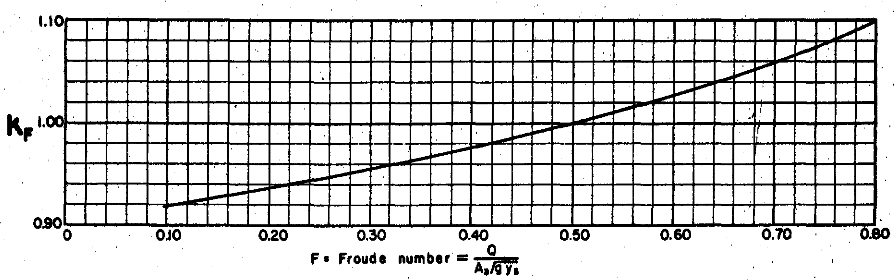
Sheet ___ of ___ sheets Computed by *HJT* Date *11-28-52* Checked by *MNK* Date *11-29-52*

Figure 21. --Computation of discharge.

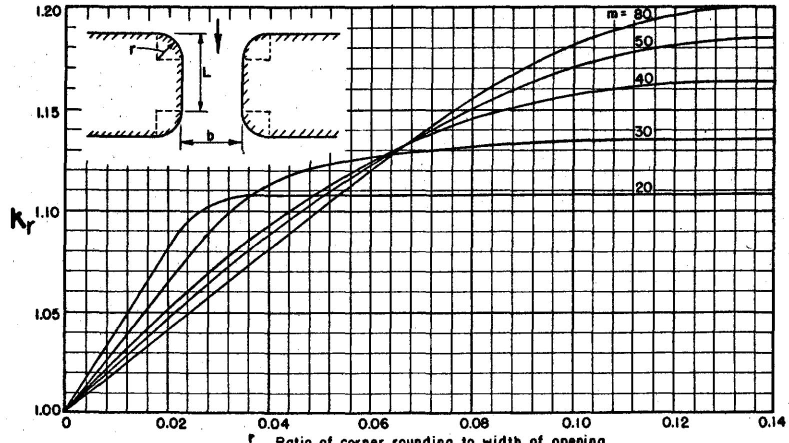




A. Base curve for coefficient of discharge

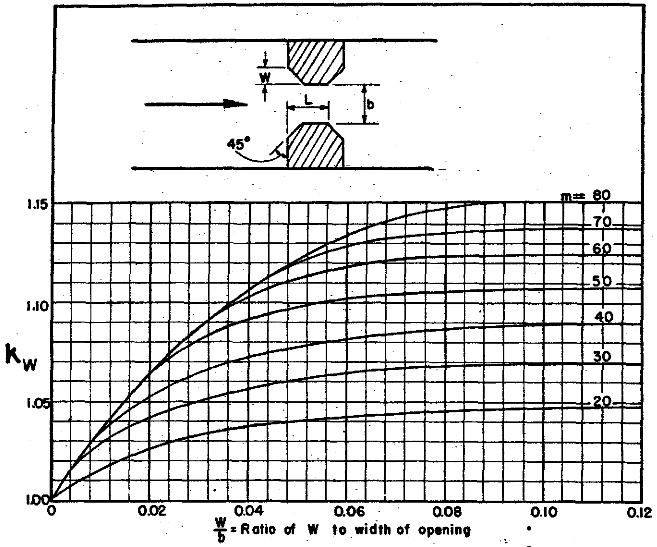


B. Variation of discharge coefficient with Froude number

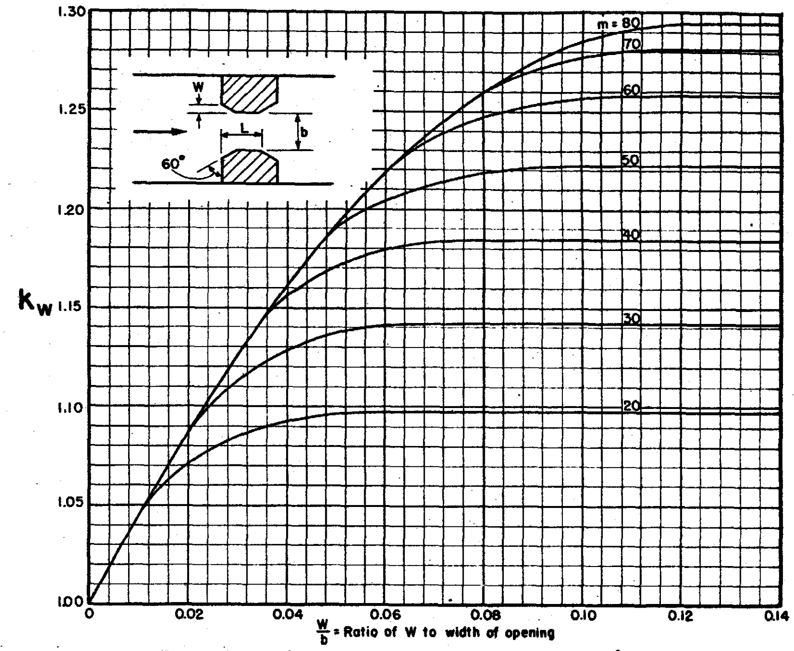


C. Variation of discharge coefficient with entrance rounding

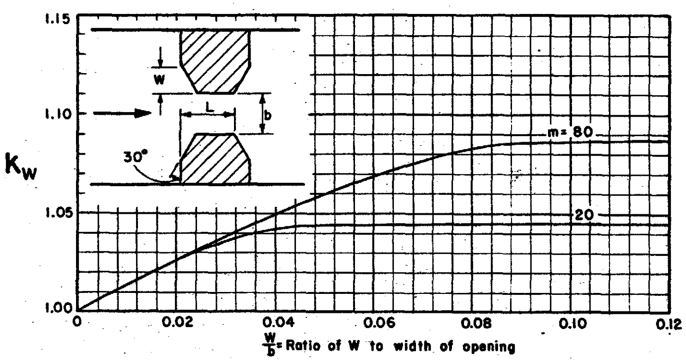
Figure 22. --Type I opening, vertical embankment, vertical abutment.



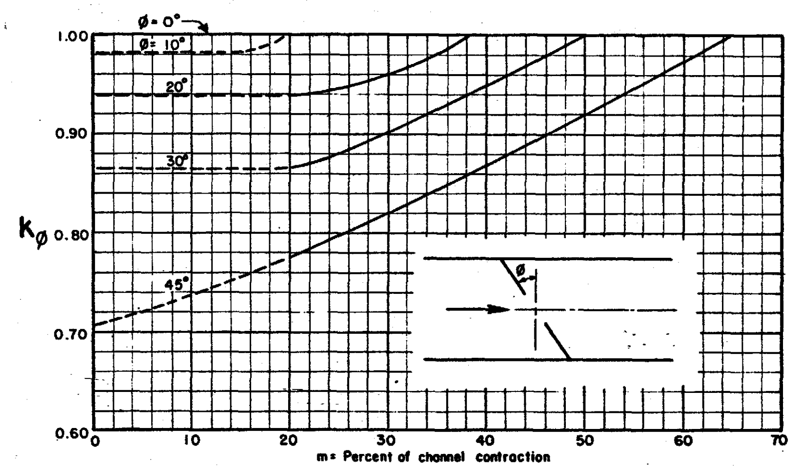
A. Variation of discharge coefficient with length of 45° wing walls or chamfers



B. Variation of discharge coefficient with length of 60° wing walls

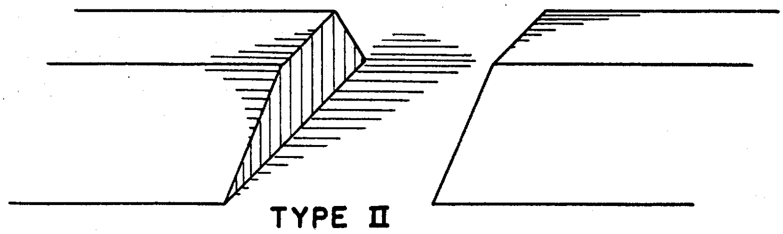


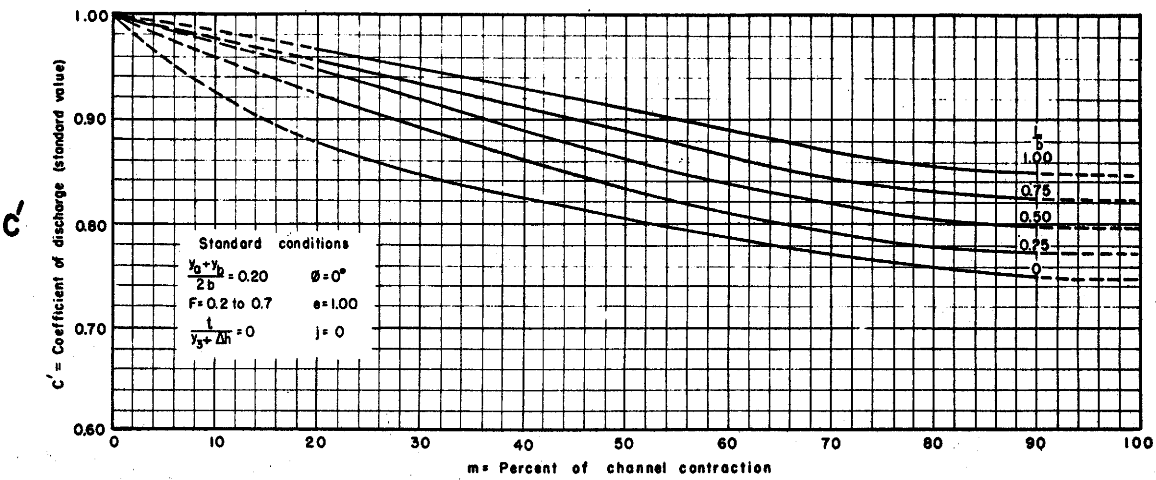
C. Variation of discharge coefficient with length of 30° wing walls



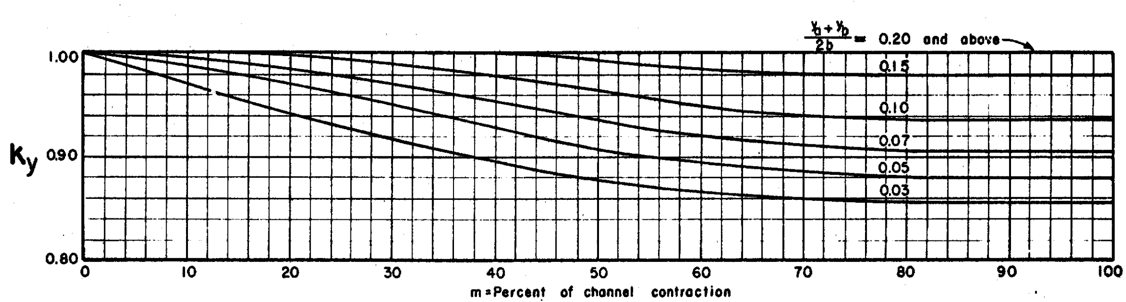
D. Variation of discharge coefficient with angularity

Figure 23. -- K_w and $K_Ø$ curves for vertical embankment and abutment of type I opening. See figure 30.

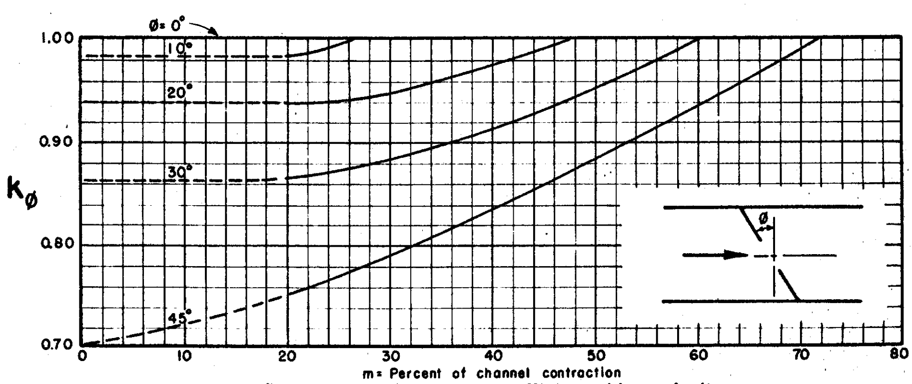




A. Base curve for coefficient of discharge

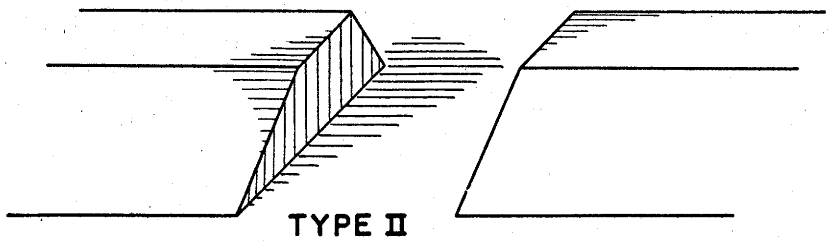


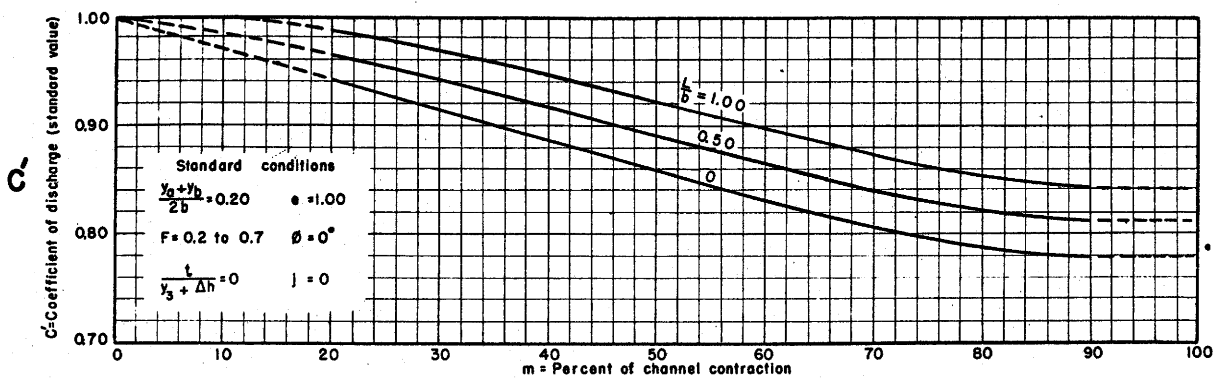
B. Variation of discharge coefficient with $\frac{y_a + y_b}{2b}$ ratio



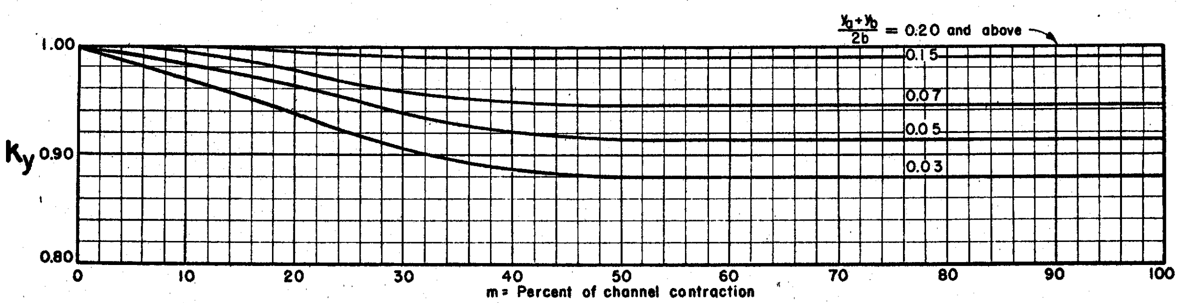
C. Variation of discharge coefficient with angularity

Figure 24.--Type II opening, embankment slope 1 to 1, vertical abutment. See figure 30.

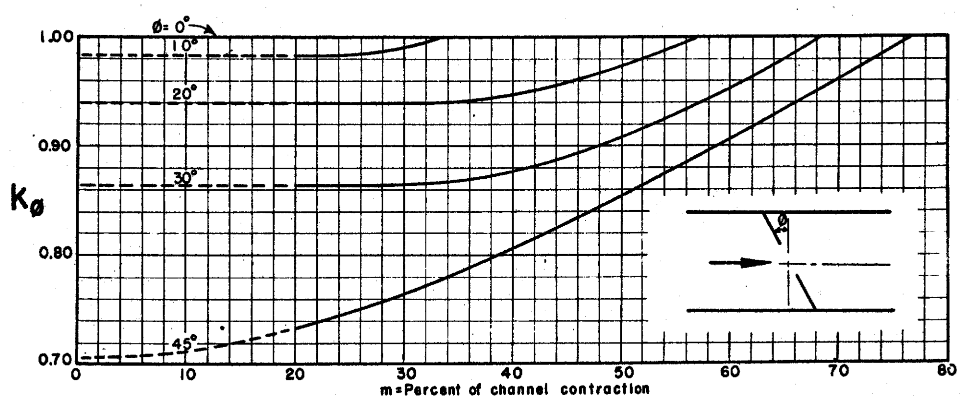




A. Base curve for coefficient of discharge

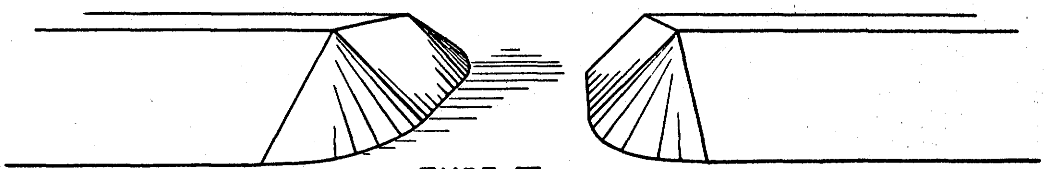


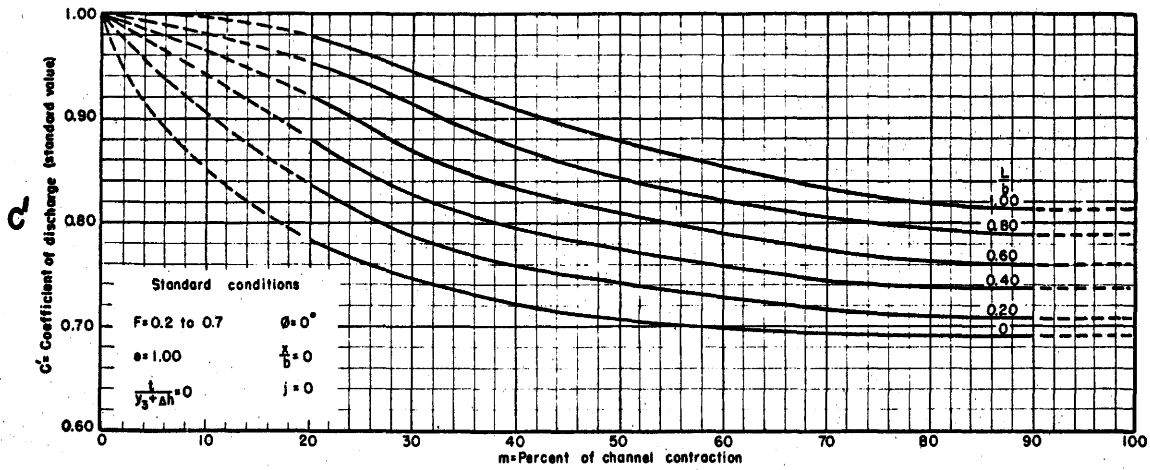
B. Variation of discharge coefficient with $\frac{y_a + y_b}{2b}$ ratio



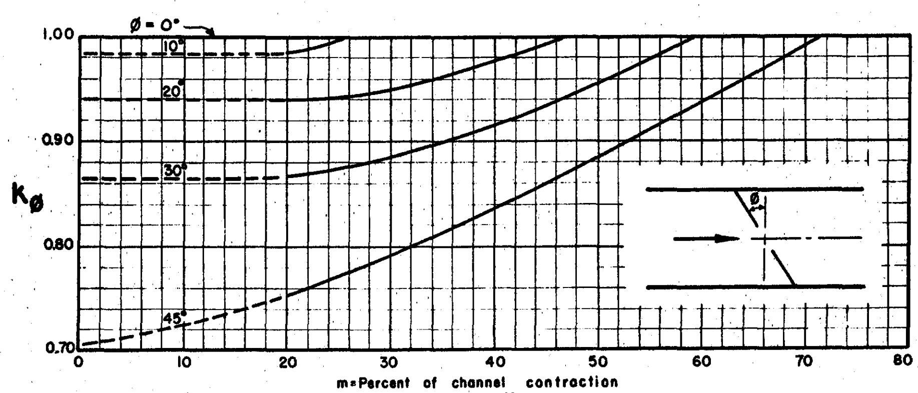
C. Variation of discharge coefficient with angularity

Figure 25.--Type II opening, embankment and abutment slope 2 to 1, vertical abutment. See figure 30.

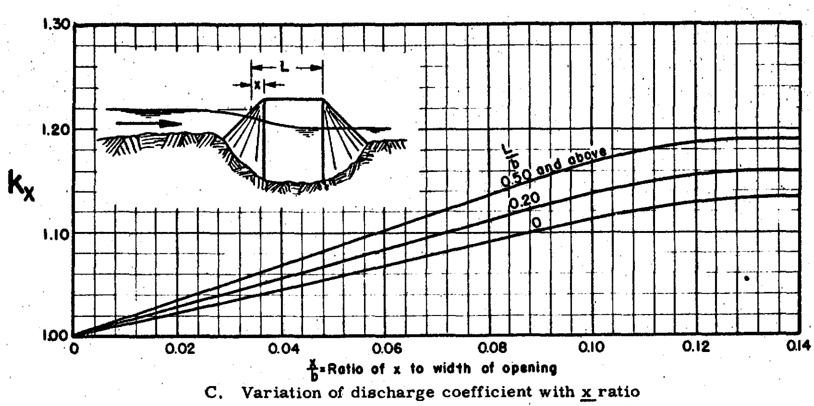




A. Base curve for coefficient of discharge

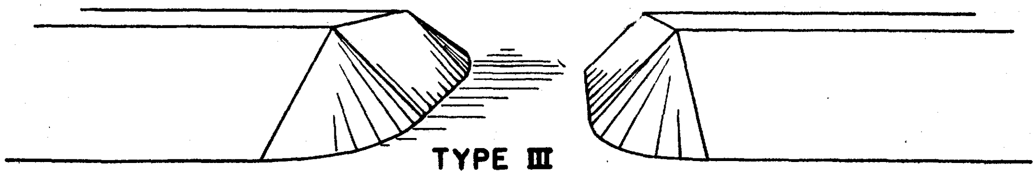


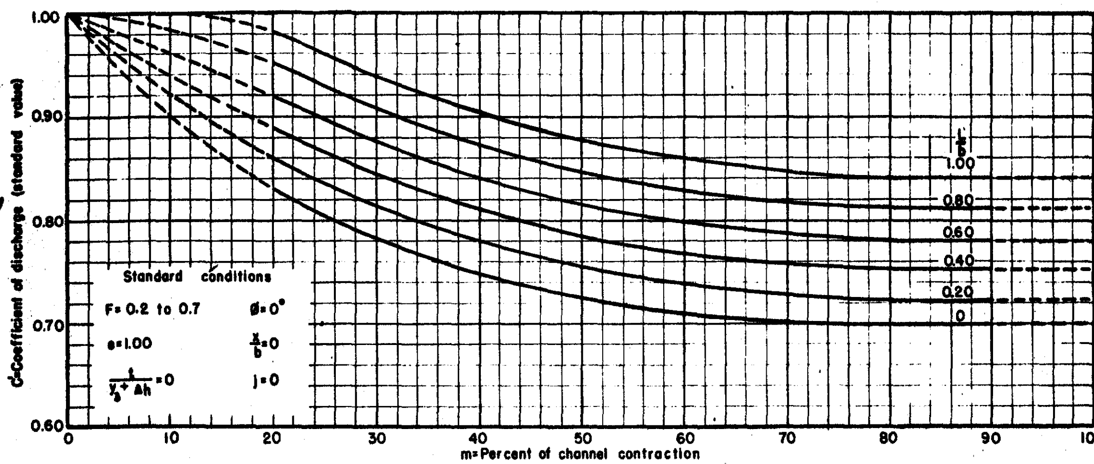
B. Variation of discharge coefficient with angularity



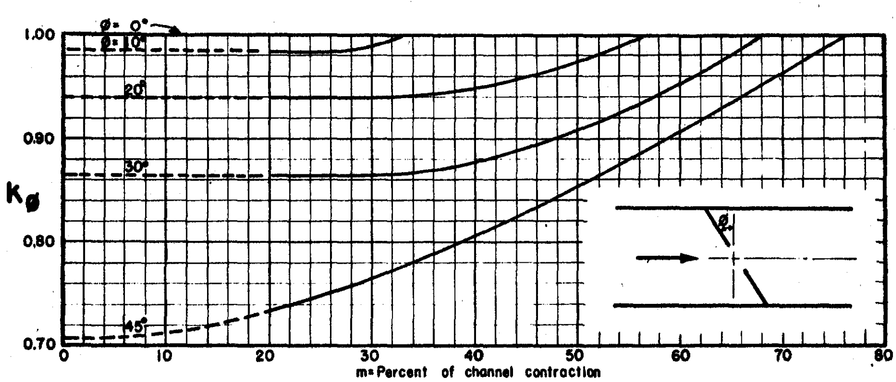
C. Variation of discharge coefficient with $\frac{x}{b}$ ratio

Figure 26.--Type III opening, embankment and abutment slope 1 to 1. See figure 30.

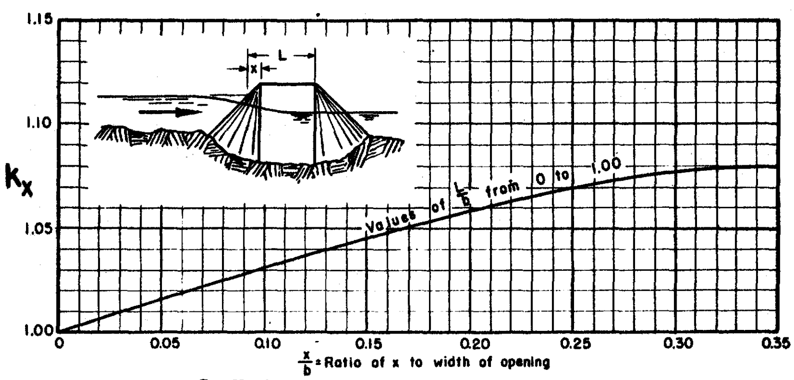




A. Base curve for coefficient of discharge

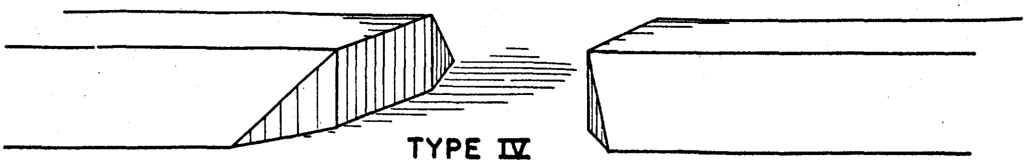


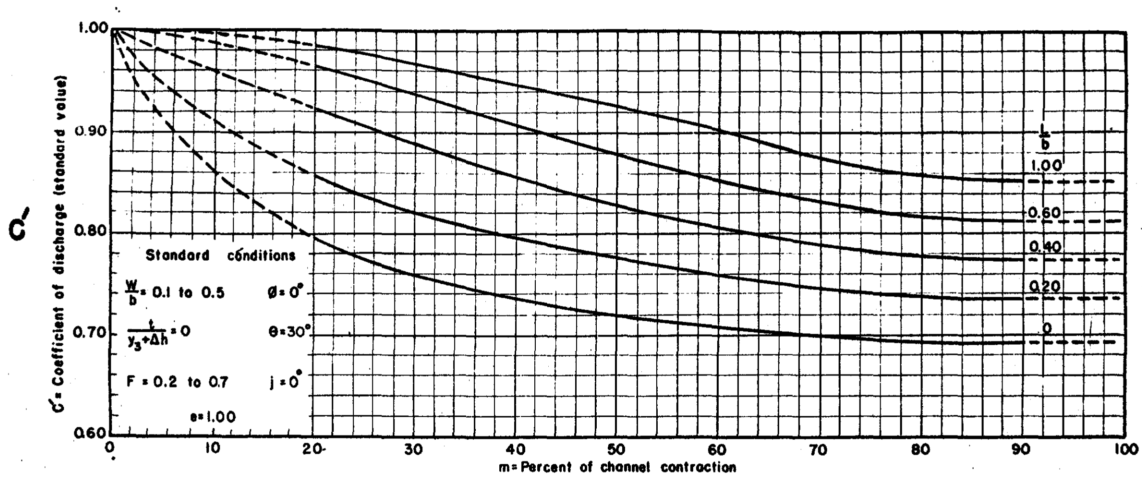
B. Variation of discharge coefficient with angularity



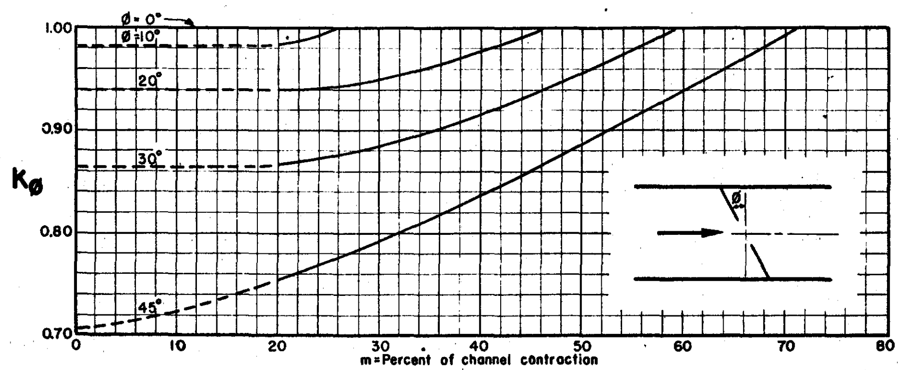
C. Variation of discharge coefficient with x/b ratio

Figure 27.--Type III opening, embankment and abutment slope 2 to 1. See figure 30.

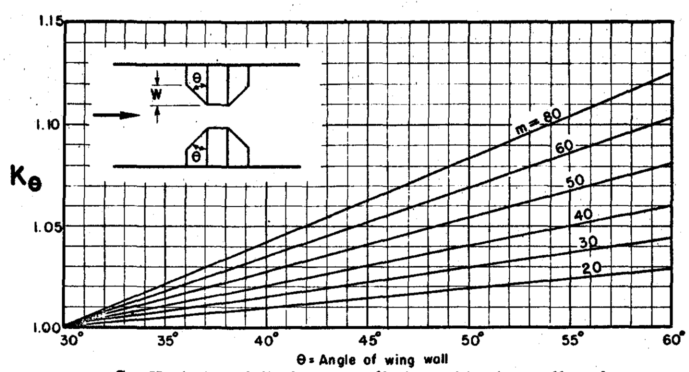




A. Base curve for coefficient of discharge

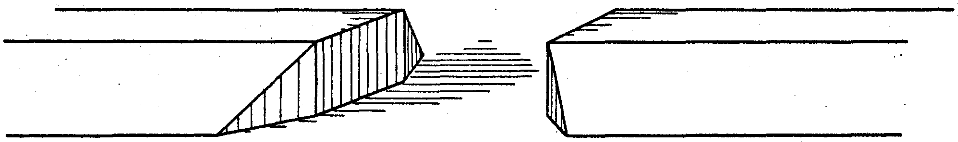


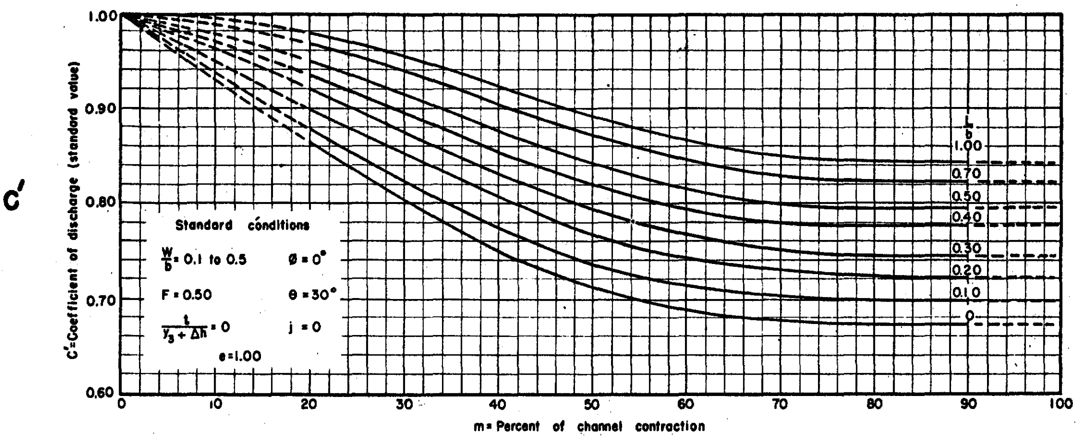
B. Variation of discharge coefficient with angularity



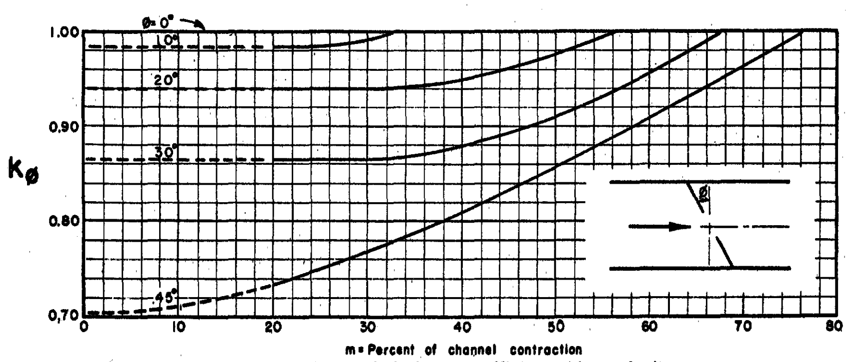
C. Variation of discharge coefficient with wing-wall angle

Figure 28.--Type IV opening, embankment slope 1 to 1, vertical abutment with wing walls. See figure 30.

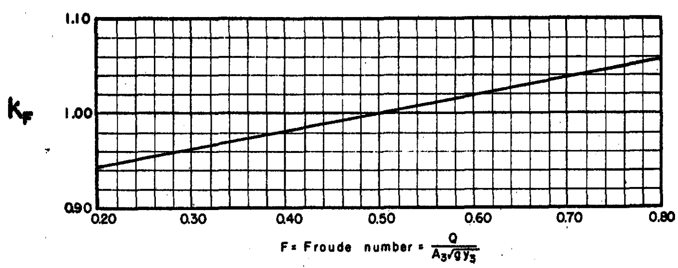




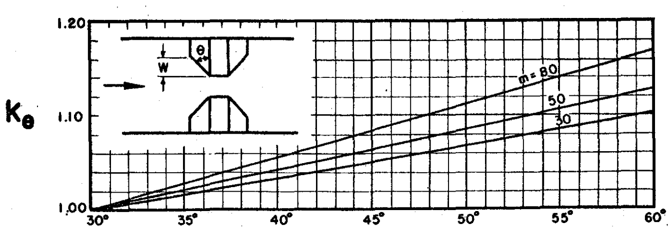
A. Base curve for coefficient of discharge



B. Variation of discharge coefficient with angularity

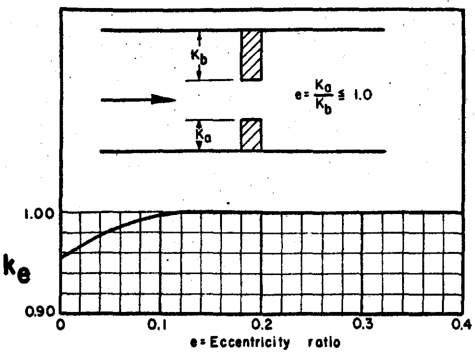


C. Variation of discharge coefficient with Froude number

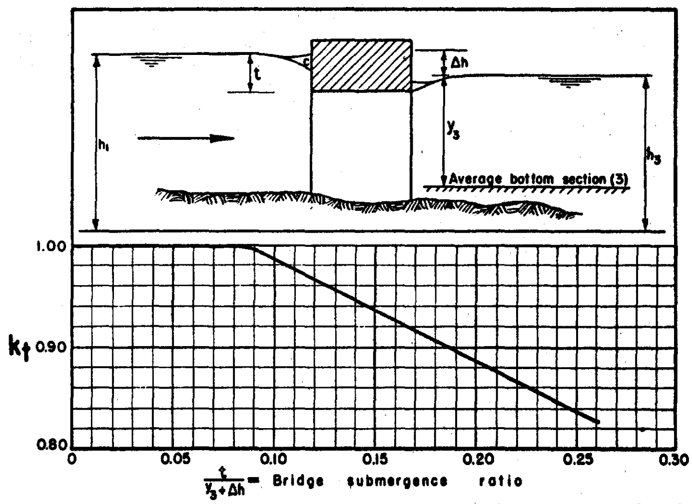


D. Variation of discharge coefficient with wing-wall angle

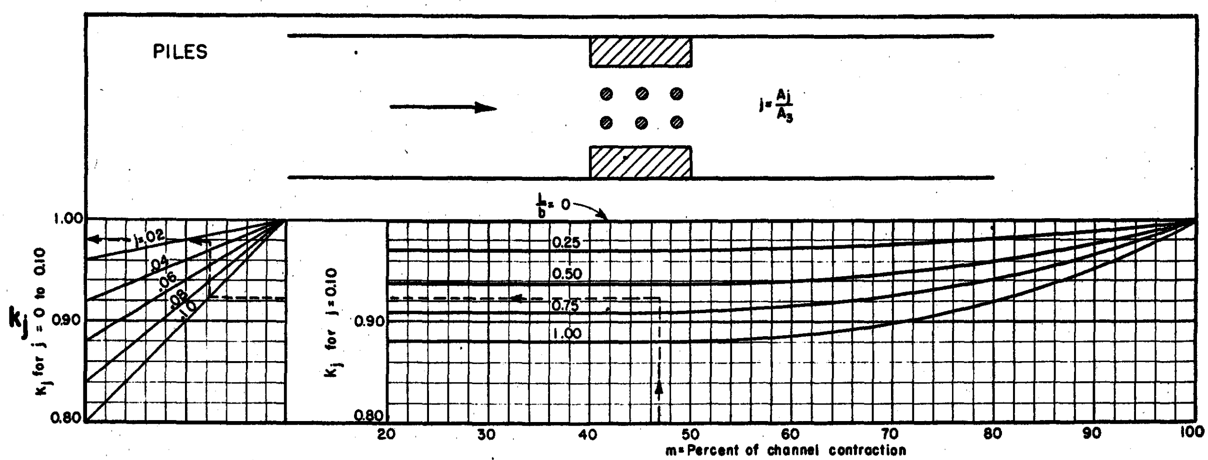
Figure 29. --Type IV opening, embankment slope 2 to 1, vertical abutment with wing walls. See figure 30.



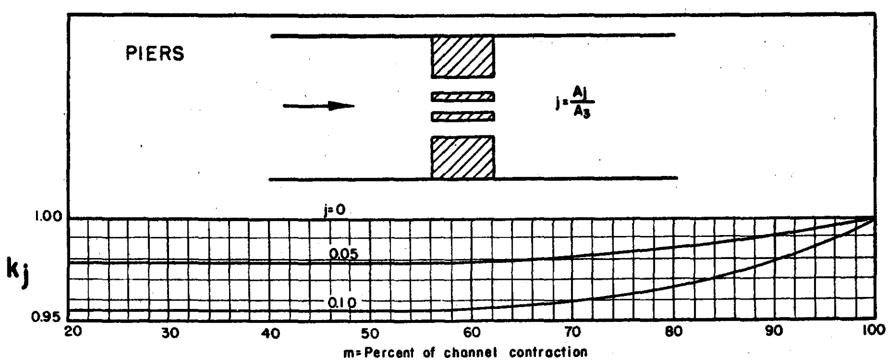
A. Variation of discharge coefficient with eccentricity



B. Variation of discharge coefficient with degree of submergence of bridge



C. Variation of discharge coefficient with area of bridge piles



D. Variation of discharge coefficient with area of bridge piers

Figure 30.--Types I-IV openings, k_e, k_t, and k_j curves.