

INDIRECT METHODS OF RIVER DISCHARGE MEASUREMENT

by

J. S. CRAGWALL, JR.

U. S. Geological Survey, Washington, D. C.

INTRODUCTION

The United States Geological Survey is one of the federal agencies designated to collect data pertaining to the water resources of this country. Among its responsibilities is measurement of stream flow. The determination of peak discharges of floods, no small part of the stream-gaging program, becomes an important and challenging task.

Stream flow is usually measured by the familiar current-meter method. Oftentimes during floods, and for many good reasons, however, current-meter measurements cannot be obtained. For definition of floods under such conditions, peak discharges can frequently be measured by so-called indirect methods. It is the purpose of this paper to describe these indirect methods of river-discharge measurement.

BASIS OF INDIRECT METHODS

Discharge in a reach of channel is related to the water-surface profile and the hydraulic characteristics of the channel. Definition of water-surface profiles from high-water marks, and channel characteristics from a survey of channel size, geometry, and roughness constitute the basis of indirect methods of river-discharge measurement.

The water-surface profile is an important element in an indirect measurement. Not only must its elevation be known, but even more important, its changes within the reach of channel under consideration. These changes in profile are primarily the result of (1) energy losses due to bed roughness, eddies, etc., and (2) acceleration. For a relatively uniform reach of stream channel, the change in water-surface profile results largely from bed roughness. At sudden contractions, such as bridges, culverts, and dams, the surface profile reacts to the influence of acceleration; that is, the change in profile reflects primarily a change in energy from potential to kinetic.

NECESSITY FOR INDIRECT METHODS

The necessity for indirect methods of measurement becomes apparent following any widespread flood of unusual magnitude. The great flood of July 1951 in Kansas and Missouri is a striking example. That flood reached such extremes of stage and discharge that it was found impossible to carry on the normal stream-gaging program in the flooded area. Travel during and immediately following the flood period was at a standstill. Even had it been possible to reach gaging sites, the structures from which measurements would normally have been made were, for the most part, destroyed, overflowed, or bypassed by wide and swift overbank flows, making impossible the measurement of discharge by current meter. Because of these and other difficulties, few direct measurements of peak, or near-peak, flows were made. For a flood of such extreme and widespread proportions, however, it was imperative that discharges be adequately defined.

Soon after the recession of the flood, operations were begun to determine peak discharges by indirect methods at gaging stations and other critical points in the flood area. Engineers from Geological Survey offices all over the country—men experienced in indirect methods of measurement—were quickly dispatched to the flooded area. This group, with the assistance of some personnel from other agencies, State and Federal, manned numerous field parties for securing the necessary surveys, computed the flood records, and prepared reports thereof.

The magnitude of the task and the coverage obtained are illustrated on Fig. 1, a map of the Kansas River basin. Shown thereon are 88 sites where discharge was determined, most of them at established gaging stations. At 50 of those sites, peak flows were defined by indirect methods. Over the entire area encompassed by the flood, discharge was defined at 182 sites, at which, for 68, indirect methods of measurement were applied in defining flood peaks.

This concerted effort was rewarding. In later basin-wide flow comparisons using flood-routing techniques, flood discharges (peaks and volumes) were found consistent to an encouraging degree. Where possible, comparisons with stage-discharge ratings defined by current meter indicated that ratings defined by indirect methods were reliable.

The Kansas-Missouri floods of July 1951 are reported in *Water-Supply Paper 1139* [1]. That the flood data contained therein were, to a great extent, based on indirect methods of flow measurement, indicates the practical necessity for and the value of indirect

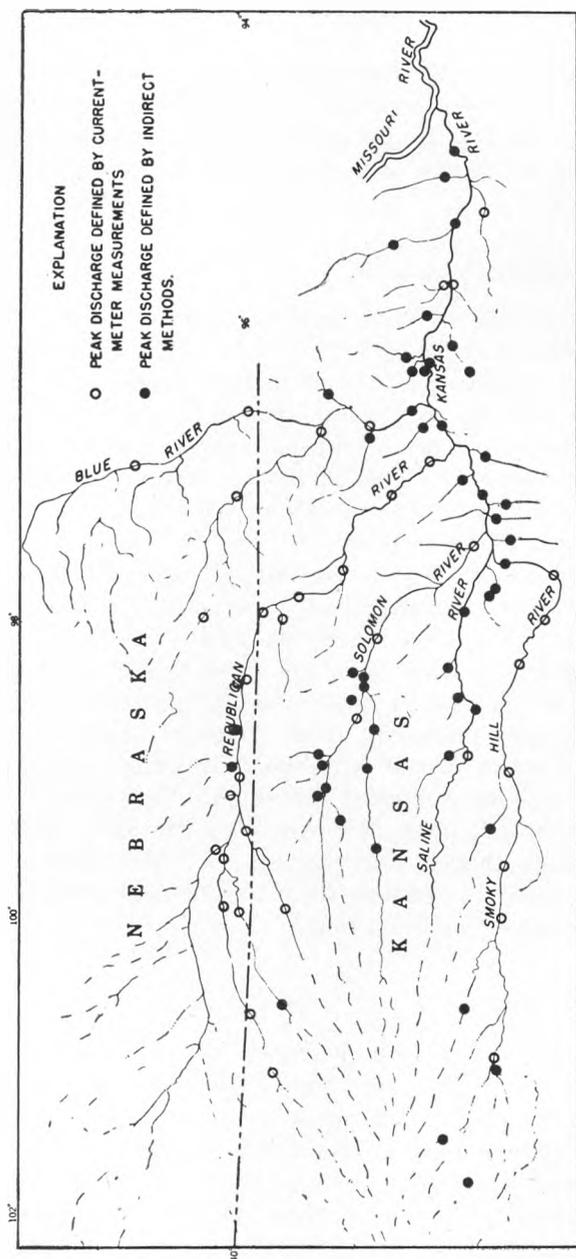


FIG. 1. MAP OF KANSAS RIVER BASIN SHOWING LOCATION OF FLOOD-DETERMINATION POINTS, FLOOD OF JULY 1951

methods. Unquestionably, the quantitative description of the Kansas-Missouri floods of 1951 would not be so complete nor the data nearly so reliable had methods of indirect flow measurement not been applied.

The foregoing is an outstanding single example of the value and necessity for using indirect methods. Their worth can further be attested by the fact that annually, on the average, the Geological Survey makes about 600 determinations of peak discharge by indirect methods.

INDIRECT METHODS CLASSIFIED

Indirect methods of river-discharge measurement are grouped into four major categories for ease of application and reference. These are (1) slope area, (2) contracted opening, (3) flow through culverts, and (4) flow over dams. The four classifications although somewhat arbitrary, have been found convenient in setting up field and office procedures. Occasionally an indirect measurement will involve a combination of methods or another method of solution outside of these general classes.

The slope-area method is the most frequently used, especially on the large rivers, primarily because a natural reach of channel acceptable as a slope-area reach can usually be found. Contracted-opening, culvert, or dam sites are used whenever conditions are favorable. The change in water-surface profile through a slope-area reach results primarily from channel roughness and, hence, the ability to select proper roughness coefficients is a measure of the accuracy of the computed discharge. The contracted-opening, flow-through-culvert, and flow-over-dam methods involve abrupt contractions and, hence, the changes in water-surface profile reflect mainly changes in energy form, and the value of the roughness coefficient becomes less important.

SLOPE-AREA METHOD

In the slope-area method, discharge is computed on the basis of a uniform flow equation involving channel characteristics, water-surface profiles, and a roughness or retardation coefficient. The change in water-surface profile for a uniform reach of channel represents losses caused by bed roughness.

In application of the slope-area method, any one of the well-known variations of the Chézy equation might well be used. The Geological Survey uses the Manning formula. This formula was originally adopted, as it has been by many engineers, because of its

simplicity of application. Many years of experience in its use have now been accumulated. That it has become a medium through which reliable results can be obtained, justifies its use.

The Manning formula, written in terms of discharge, is

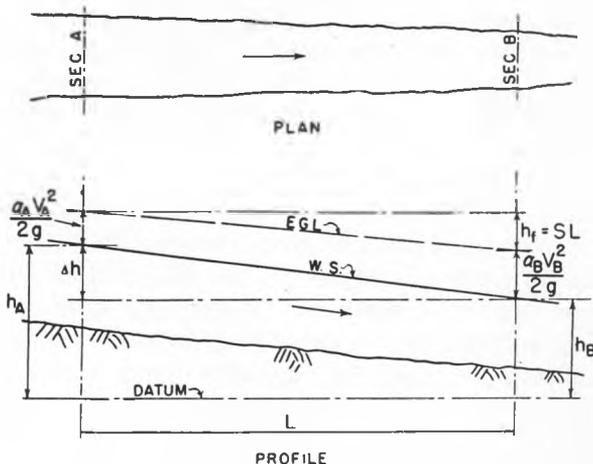
$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2} \quad (1)$$

where

- Q = discharge, in cfs
- A = cross-sectional area, in sq ft
- R = hydraulic radius in ft
- S = energy gradient, or friction slope
- n = a roughness or retardation coefficient

Manning's formula, as originally developed, was intended only for uniform flow where the water-surface profile is parallel to the stream bed and the area and hydraulic radius remain constant throughout the reach. In spite of these limitations, since a better solution is lacking, the formula is used for the nonuniform reaches that are invariably encountered in natural channels. The only justification for such use is that of necessity; however, several factors in the formula are modified in an attempt to correct for non-uniformity.

A description of the method used in computing river discharge by the slope-area method is discussed in conjunction with Fig. 2,



$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2} = K S^{1/2}$$

$$K = \sqrt{K_A \cdot K_B} \quad S = \frac{h_f}{L} = \frac{\Delta h + \left(\frac{q_A V_A^2}{2g} - \frac{q_B V_B^2}{2g} \right)}{L}$$

FIG. 2. DEFINITION SKETCH OF A TWO-SECTION SLOPE-AREA REACH (NOT TO SCALE)

a definition sketch of a two-section reach which is gradually contracting in direction of flow. Cross sections A and B are selected as being representative of the channel between A and B. The term $[(1.486/n)AR^{2/3}]$ contains the several factors descriptive of channel characteristics and is labeled "conveyance" K . For the reach A-B, because it is not truly uniform, conveyance is expressed as the geometric mean of the conveyances of the two sections, in this way: $K = \sqrt{K_A K_B}$

For the contracting reach, the drop in water-surface profile Δh is not entirely a result of friction loss h_f , but also reflects the acceleration between sections A and B. If the S term in Manning's formula is taken to represent friction slope only, then the drop in water level Δh must be adjusted by the change in velocity head. Friction slope may be expressed as

$$S = \frac{h_f}{L} = \frac{\Delta h + \left[\frac{\alpha_A V_A^2}{2g} - \frac{\alpha_B V_B^2}{2g} \right]}{L}$$

The discharge formula, therefore, may be condensed to

$$Q = KS^{1/2} \quad (2)$$

where K represents the average conveyance of the reach and S the energy gradient or friction slope.

In this manner Manning's formula is applied to gradually-varied flow in natural channels. Admittedly, assumptions have been made that greatly oversimplify the complex functions inherent in non-uniform flow. This treatment, however, provides a logical method to which experience can be related, and when a given problem is within the accumulated experience range, it has been found that reliable results can generally be obtained.

Note that a moderately contracting reach is illustrated in Fig. 2. The practice of using such reaches—reaches that are contracting gradually and to moderate degree in the direction of flow—has become standard Survey procedure. Contracting reaches have been found to yield a consistent scale of values in the Survey's roughness-investigation program. On the other hand, expanding reaches have not, probably because of unknown energy losses and velocity distributions associated with the diverging flow patterns. Uniform reaches are not intentionally avoided, but in reality can rarely be found. By searching for slightly contracting reaches, expanding reaches can at least be avoided. Contracting reaches also are favored for other reasons. For example, any errors involved in estimating n affect the discharge result to a lesser degree than in expanding

reaches, owing to the effect of the velocity-head correction involved in computing the energy gradient S .

Coefficients

Selection of the roughness coefficient n is a critical step in a slope-area measurement, so much so because personal judgment cannot be entirely eliminated. Moreover, for a natural channel the coefficient is descriptive of losses other than bed roughness. Bank irregularities, channel meanders and curvature, overhanging trees, and many other retarding influences that defy quantitative description are present. Tables of n values as found in many hydraulic texts provide little assistance to the inexperienced, unless very large errors are permissible.

The Geological Survey has attempted to devise a useful and reliable reference index of n values. The project has been underway for several years. For streams representing a wide variety of conditions, and where peak discharges are known, slope-area measurements are obtained for the purpose of computing n values. Comprehensive photographic coverage, in three-dimensional color, is secured for each definition reach. These pictures are duplicated in sufficient quantity to place a reference slide file in each district office of the Surface Water Branch. Such a file enables the less experienced engineer to select an n value for a channel under consideration by a near-realistic and visual comparison of that channel with similar channels having defined coefficients. Values of n ranging from 0.028 to 0.075 are presently included in this reference file; additions to it are continuing.

The roughness-definition program has been limited to simple and single channels of approximate trapezoidal shape, sometimes referred to as unit channels. As would be expected a multiplicity of channel shapes are encountered in the field. Some of these, the so-called compound channels, frequently require subdivision to account for variations in shape or roughness. The conveyance K of a subdivided section is taken as the sum of the conveyances K_1, K_2, \dots of each subsection. This procedure focuses attention on the treatment given to the velocity head at each cross section. As noted on Fig. 2 and in the previous discussion on computing friction slope, the velocity-head factor includes the coefficient a . This coefficient is arbitrarily taken as unity for channels of unit shape. For compound, subdivided channels, a for a cross section is estimated by the formula

$$a = \frac{(K_1^3/A_1^2) + (K_2^3/A_2^2) + \dots}{K^3/A^2}$$

The foregoing expression, a means of separating the effect of velocity distribution, allows the coefficient n to be more nearly described as a roughness factor for all channel shapes.

Research

The nationwide program of evaluation of roughness coefficients is a continuing one. While the nucleus of a reference slide file is presently available and more examples are being processed, a broadening of the range of conditions is continually sought. For example, definitive studies on extremely rocky and steep channels and on sand channels are active field projects. Definition is needed for high values of n encountered in heavily brush-covered channels such as those encountered in the southeastern sections of this country.

The work of other investigators concerning resistance to flow in rough channels such as Powell [2, 3], and Robinson and Albertson [4], has been studied with interest. Resistance to flow in pipes has undergone intensive investigation with considerable success. It is hoped that similar treatment of open-channel flow will lead eventually to equations of maximum practical usefulness in natural-channel work.

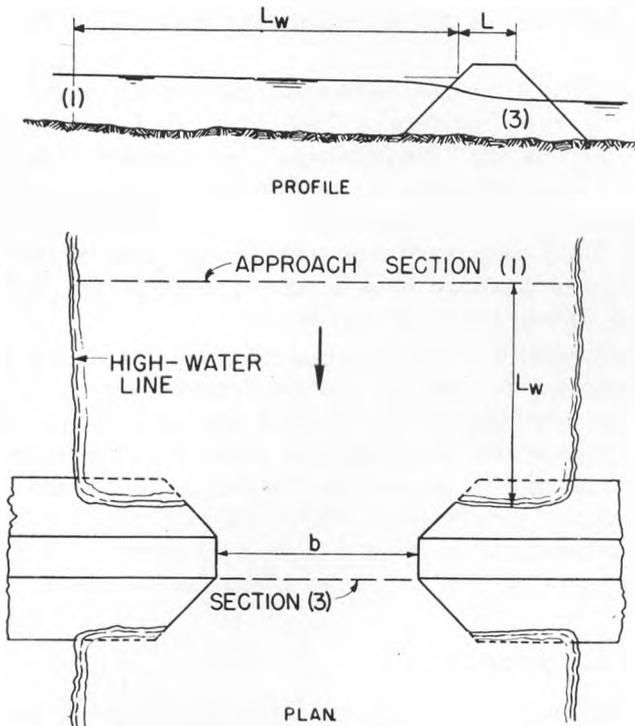
CONTRACTED-OPENING METHOD

A highway or railroad crossing of a river channel is generally so constructed as to impose an abrupt width constriction upon flood flow. At such constrictions the contracted-opening method of measuring peak discharge can be applied.

At an abrupt width constriction in a reach of channel, as shown in the definition sketch of Fig. 3, the change in water-surface profile between an approach section (1) and contracted section (3) results largely from the acceleration within the reach. Because the reach is so short, friction loss is of little importance; thus, the effect of possible errors in selecting roughness coefficients n is greatly minimized.

The drop in water surface between sections 1 and 3 is related primarily to the corresponding change in velocity. By writing the energy and continuity equations between sections 1 and 3, we obtain the discharge formula

$$Q = C A_3 \sqrt{2g \left(\Delta h + \frac{a_1 V_1^2}{2g} - h_f \right)} \quad (3)$$



$$Q = CA_3 \sqrt{2g \left(\Delta h + \frac{a_1 V_1^2}{2g} - h_f \right)}$$

FIG. 3. DEFINITION SKETCH OF AN OPEN-CHANNEL CONSTRICTION

In this expression,

- Q = discharge, in cfs
- A_3 = area of section 3, in sq ft
- Δh = difference in water-surface elevation, in ft, between sections 1 and 3
- $\frac{a_1 V_1^2}{2g}$ = weighted average-velocity head, in ft, at section 1
- h_f = friction loss, in ft, between sections 1 and 3
- C = a discharge coefficient

For many years the contracted-opening method was patterned after the methods used by the Miami Conservancy District as reported by Ivan E. Houk [5]. The method employed was essentially the same as that embodied in Eq. (3)—a form of the combined energy and continuity equations. Only limited information was

available, however, to aid in estimating values of a discharge coefficient.

In February 1951 the Geological Survey initiated a research project at Georgia Institute of Technology on flow through single-opening constrictions. This investigation resulted in a better understanding of the mechanics of flow through width constrictions in open channels and led to development of an improved method for computing peak discharge through bridge waterways. The investigation was reported by Kindsvater and Carter [6, 7], and in *Geological Survey Circular 284* [8].

The applicability of the method in general, as used previously and as expressed by Eq. (3) was confirmed. Factors largely dealing with the influence of the channel and constriction shapes not previously recognized as significant, however, were found to have an important bearing upon the discharge coefficient C . Thus, emphasis on the experimental evaluation of C for a wide range of variables generally encountered in field problems became the primary objective of the latter phase of the investigation.

Discharge Coefficient

In the laboratory investigation, the discharge of coefficient C in Eq. (3) was taken to represent the combination of a coefficient of contraction, a coefficient accounting for eddy losses due to the contraction, and the velocity-head coefficient a_3 for the contracted section.

The discharge coefficient was found to be a function of certain governing geometric and fluid parameters; i.e.

$$C = f(\text{degree of channel contraction, geometry of constriction, Froude number})$$

Of all the factors influencing the coefficient of discharge, the degree of channel-contraction m and the length-width ratio of the constriction L/b were found the most significant.

Perhaps the most important new concept resulting from the laboratory investigation was that pertaining to definition of channel-contraction for channels of irregular shape. It was found that channel contraction could be described as a measure of that part of the total flow which is required to enter the contracted stream from the sides—from the lateral regions upstream from the embankments. Thus m , the channel-contraction ratio, is defined by the equation

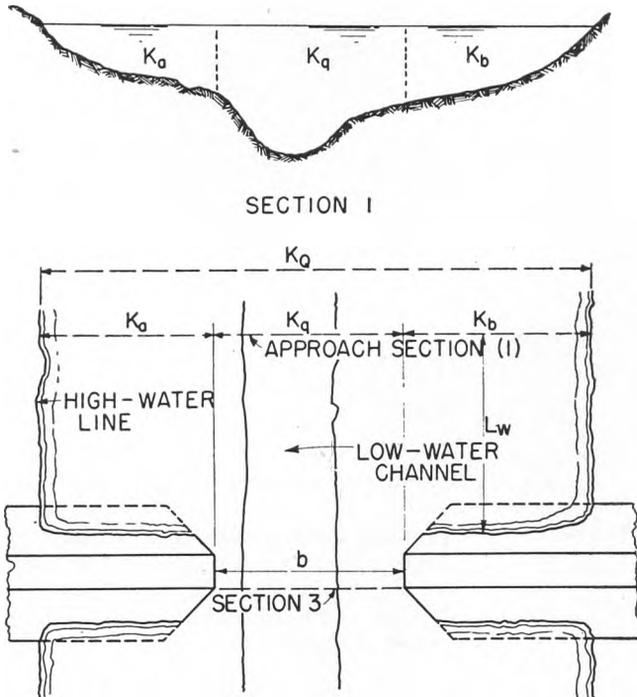
$$m = \frac{Q - q}{Q} = 1 - \frac{q}{Q}$$

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

This concept has particular advantage in connection with bridge-waterway problems in that the degree of channel-contraction for irregular, natural channels can be computed as a ratio of hydraulic conveyances. Even for considerable variations of depth, alignment, and roughness in the approach channel, a procedure for evaluating m is given by

$$m = 1 - \frac{K_q}{K_Q} = \frac{K_a + K_b}{K_Q} \quad (4)$$

The above notation is explained in the definition sketch of Fig. 4.



$$m = 1 - \frac{K_q}{K_Q} = \frac{K_a + K_b}{K_Q}$$

FIG. 4. DEFINITION SKETCH OF CHANNEL-CONTRACTION RATIO

The Froude number F was found to have a relatively minor effect upon C for most geometry types over the range of F tested. The investigation was confined to flow within the tranquil range.

The coefficient C was defined for four constriction types simulating the most frequently encountered forms of highway bridge-opening shapes. These shapes, illustrated in Fig. 5, are classified as:

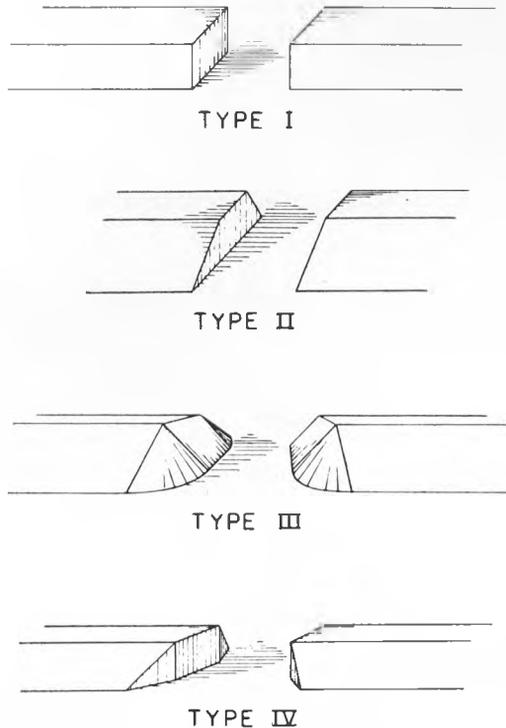


FIG. 5. CONSTRICTION GEOMETRICS CLASSIFIED

- Type I — Vertical embankments and abutments
- Type II — Sloping embankments, vertical abutments
- Type III — Sloping embankments, and abutments
- Type IV — Sloping embankments, vertical abutments with wing walls

Curves relating C to pertinent variables for each of these four types of openings may be found in *Geological Survey Circular 284* (p. 26-34) [8]. Figure 6, for a type III opening only, is included herein as an example. Inasmuch as C was found to depend primarily upon m and L/b , a standard value, C' , was related to these variables (see Fig. 6-A) for fixed values of the secondary variables.

DISCHARGE-COEFFICIENT CURVES

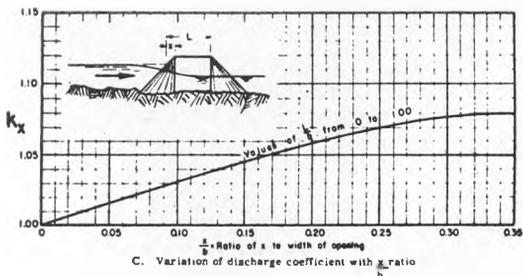
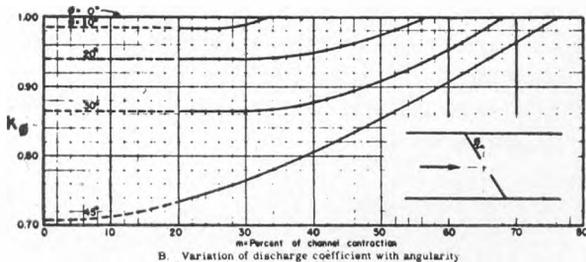
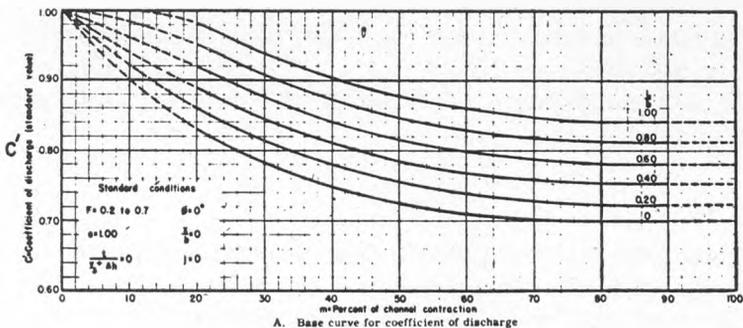


FIG. 6. TYPE III OPENING, EMBANKMENT AND ABUTMENT SLOPE 2 TO 1

If, in a particular problem, the secondary variables deviate from the fixed values shown in the box of Fig 6-A, the value of C' is successively adjusted for each deviation from the standard. For example, the final discharge coefficient is computed as

$$C = C' k_{\phi} k_x$$

where a given problem departs from the fixed secondary values only with respect to angularity (Fig. 6-B) and the x/b ratio (Fig. 6-C). Adjustment-coefficient curves for other secondary variables such as effect of area of piles and piers, eccentricity, submergence, etc., are not shown on Fig. 6.

For any combination of variables the limiting value of C is taken as 1.00.

Research

Laboratory investigation of flow through single-opening constrictions, with minor exceptions, was concluded in 1953. Since then a program of field verification of the method has been carried on, with favorable results. Twenty-two contracted-opening surveys at sites where discharges are known have been obtained to date. These cover the wide variety of conditions normally encountered in practice. The following table indicates the comparison of results by indirect-method computation, based upon discharge defined by current-meter measurement.

<i>Difference by indirect method (percentage range)</i>	<i>Number of verifications</i>
+15 to +20	1
+10 to +15	1
+ 5 to +10	7
0 to + 5	3
0 to — 5	7
— 5 to —10	1
—10 to —15	1
—15 to —20	1
	22 total

It is encouraging to note from the above tabulation that about 80 percent of the field verifications made to date give results within 10 percent of the respective known discharges. Considering the possibilities of error inherent in the field data and in the discharges measured by current meter, this is believed to be remarkably good.

A laboratory investigation of flow through multiple-opening constrictions is now underway. It is anticipated that the problems

and complexities inherent in this more complicated geometrical form will be many. Several years, no doubt, will have elapsed before definitive results are forthcoming.

FLOW-THROUGH-CULVERT METHOD

A culvert often can be used as a convenient device for measurement of peak discharge by indirect methods. This is indeed fortunate because of the many difficulties involved in making current-meter measurements of flood flow on the smaller streams.

The flow-through-culvert method is similar to the contracted-opening method in that the change in water-surface profile in the reach between the approach and constricted sections reflects largely the effect of acceleration. Again, friction losses are generally of minor importance. A culvert constriction, however, is such that it may act as a control section; that is, flow may pass through critical depth at the culvert entrance or outlet. The method, therefore, covers conditions of rapid as well as tranquil flow.

Flow classification

The discharge characteristics of a culvert depend upon an evaluation of energy changes between an approach section upstream from the culvert and the control section. Depending upon the location of the control section and the relative height of headwater and tailwater, most culvert flow patterns may be classified in six types. These are described below and defined also in the sketches of Figs. 7-9.

Type I, critical depth at inlet: As indicated on Fig. 7, flow passes through critical depth d_c near the culvert entrance. Culvert barrel flows part full. The headwater-diameter ratio $\frac{h_1 - z}{D}$ is limited to a maximum of 1.5. The slope of the culvert barrel S_0 must be greater than the critical slope S and the tailwater elevation h_4 must be less than the elevation of water surface at the control section h_2 .

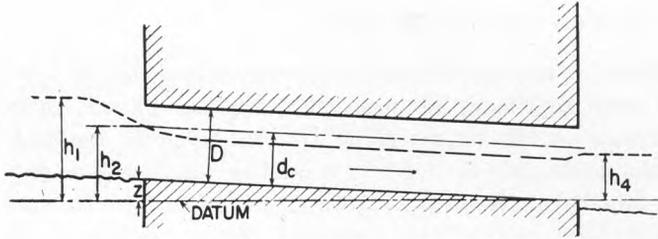
The discharge equation is

$$Q = C A_c \sqrt{2g \left(h_1 - z + \frac{V_1^2}{2g} - d_c - h_{1,2} \right)} \quad (5)$$

where A_c is the flow area, in sq ft, at the control section. Other notation is evident in Fig. 7, or has been previously explained.

TYPE I
CRITICAL DEPTH AT INLET

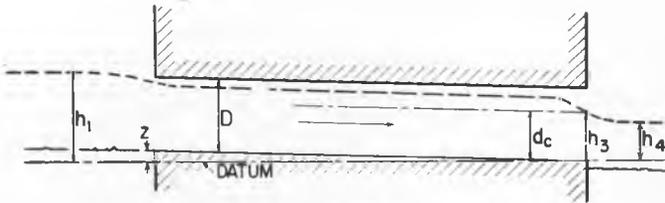
$$\frac{h_1 - z}{D} < 1.5 \quad s < s_0 \quad h_4 < h_2$$



$$Q = CA_c \sqrt{2g \left(h_1 - z + \frac{V_1^2}{2g} - d_c - h_{f_{1-2}} \right)}$$

TYPE II
CRITICAL DEPTH AT OUTLET

$$\frac{h_1 - z}{D} < 1.5 \quad s > s_0 \quad h_4 < h_3$$



$$Q = CA_c \sqrt{2g \left(h_1 + \frac{V_1^2}{2g} - d_c - h_{f_{1-2}} - h_{f_{2-3}} \right)}$$

FIG. 7. CLASSIFICATION OF CULVERT FLOW, TYPES I AND II

Type II, critical depth at outlet: Flow passes through critical depth at culvert outlet (see Fig. 7.) Culvert barrel flows part full. The headwater-diameter ratio does not exceed 1.5. Slope of the culvert is less than critical. Tailwater elevation does not exceed the elevation of water surface at the control section h_3 .

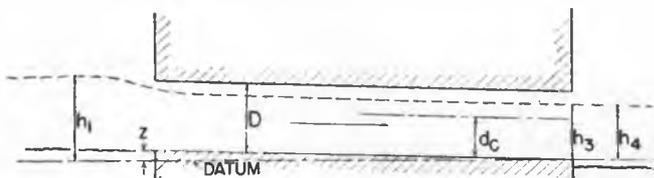
The discharge equation is

$$Q = CA_c \sqrt{2g \left(h_1 + \frac{V_1^2}{2g} - d_c - h_{f_{1-2}} - h_{f_{2-3}} \right)} \quad (6)$$

Type III, tranquil flow throughout: Culvert barrel flows part full, with the headwater-diameter ratio less than 1.5 (see Fig. 8). The tailwater elevation does not submerge culvert outlet, but does

TYPE III
TRANQUIL FLOW THROUGHOUT

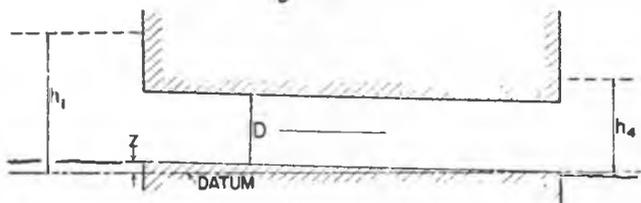
$$\frac{h-z}{D} < 1.5 \quad h_4 \bar{z} D \quad h_3 > d_c$$



$$Q = C A_3 \sqrt{2g \left(h + \frac{V_1^2}{2g} - h_3 - h_{f_{1-2}} - h_{f_{2-3}} \right)}$$

TYPE IV
SUBMERGED CULVERT

$$\frac{h_4}{D} > 10$$



$$Q = C A_o \sqrt{\frac{2g (h_1 - h_4)}{1 + \frac{29C^2 n^2 L}{R^{4/3}}}}$$

FIG. 8. CLASSIFICATION OF CULVERT FLOW, TYPES III AND IV

exceed the elevation of critical depth at the control section. The discharge equation for this condition is

$$Q = C A_3 \sqrt{2g \left(h_1 + \frac{V_1^2}{2g} - h_3 - h_{f_{1-2}} - h_{f_{2-3}} \right)} \quad (7)$$

Type IV, submerged culvert—The tailwater elevation is high enough to submerge culvert outlet; hence the culvert is submerged and flows full (see Fig. 8). The discharge equation is

$$Q = C A_o \sqrt{\frac{2g (h_1 - h_4)}{1 + \frac{29C^2 n^2 L}{R^{3/4}}}} \quad (8)$$

where

A_o = full area, in sq ft, of culvert barrel
 L = length, in ft, of culvert barrel

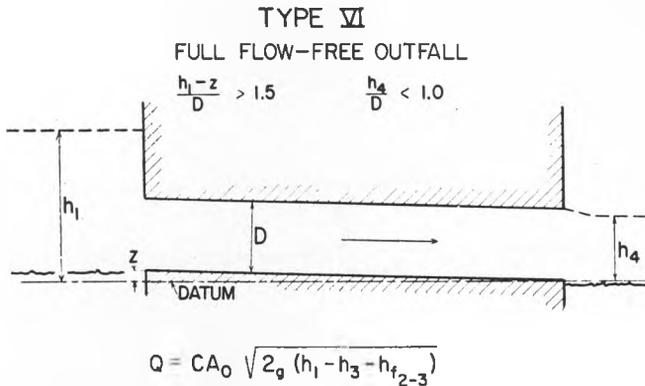
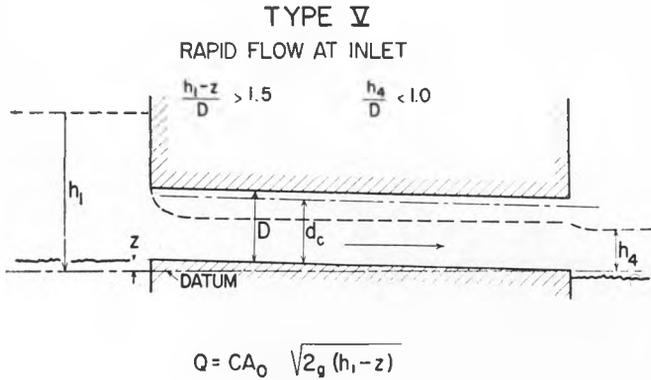


FIG. 9. CLASSIFICATION OF CULVERT FLOW, TYPES V AND VI

Type V, rapid flow at inlet: The headwater-diameter ratio exceeds 1.5 (see Fig. 9). The culvert entrance is such that flow is contracted in a manner similar to sluice- or orifice-type flow. Culvert barrel flows part full and at depth less than critical depth. The tailwater elevation does not submerge culvert outlet. The discharge equation is

$$Q = C A_o \sqrt{2g (h_1 - z)} \quad (9)$$

Type VI, full flow-free outfall: The headwater-diameter ratio exceeds 1.5 (see Fig. 9). The culvert barrel flows full under pressure. The tailwater elevation does not submerge culvert outlet. The discharge equation is

$$Q = C A_o \sqrt{2g (h_1 - h_3 - h_{f_{2-3}})} \quad (10)$$

For flow-types IV-VI a ponded condition in the headwater pool was assumed in writing Eqs: (8) to (10).

During recent years the program of gaging the flow of small streams has been expanded greatly. The use of culverts as a means of indirect measurement of peak discharge focused attention on the need for further research. In problems sometimes encountered the difficulty of distinguishing between flow-types V and VI on basis of field data ordinarily available was apparent. In addition, while discharge coefficients applicable to flow-types IV and VI had been previously defined by Yarnell [9] and Straub and Morris [10, 11], coefficients for flow types I, II, III, and V were somewhat uncertain. In order to fill in certain gaps in published data, during the past year an investigation of flow through culverts has been conducted at the Georgia Institute of Technology by the Geological Survey. The laboratory phase of the investigation has now been completed. The results of the investigation will soon be published in a form suitable for general distribution. The problem of distinguishing between types V and VI flow was not completely solved. The occurrence of flow-types V and VI, however, can be predicted, within range of geometries tested, from a knowledge of entrance geometry of the culvert, the length, slope and roughness of the culvert barrel, and the headwater-diameter ratio.

Discharge coefficients

The discharge-coefficient phase of the laboratory investigation, as previously stated, was confined primarily to flow-types I, II, III, and V. It was found that discharge coefficients vary in magnitude from 0.40 to 0.98, and are a function of the culvert geometry and the degree of channel contraction for any flow type.

Laboratory observation indicated that the discharge coefficients could be grouped for certain combinations of flow types. Coefficients for flow-types I, II, and III form one group, types IV and VI another, and type V, the third. Thus the same form of discharge equation is applicable to each group.

Entrance geometries tested were limited to those types most commonly used in highway-engineering practice. Tests included (1) flush entrances in vertical headwall with varying degrees of entrance rounding or beveling, (2) standard wing-wall entrance, (3) projecting entrance, and (4) mitered entrance set flush with sloping embankments. Results of these tests will be included in the forthcoming report.

Research

The Geological Survey has no immediate plans for further extensive laboratory research on flow through culverts. Other investigators, however, are conducting research on culvert hydraulics. Many of the continuing investigations are pointed to culvert design, the objective being to achieve an entrance form which will insure full-barrel flow (type VI) for low degrees of entrance submergence. Much of this work, undoubtedly, will enable wider usage of culvert structures for indirect methods of flow measurement.

FLOW-OVER-DAM METHOD

The hydraulics of channel controls and spillways has long been a basic tool of the hydraulician and designer. These principles may be adapted to indirect computation of discharge over such structures.

A dam forms a control section at which the discharge is related to the water-surface elevation upstream. Friction loss between an upstream approach section and the control section is generally of minor importance.

For computing peak discharges, the method consists simply of determining by field survey the head on the spillway from high-water profiles, approach-section characteristics, and spillway geometry. Discharge is computed by the well known weir formula,

$$Q = C b \left(h + \frac{V_1^2}{2g} \right)^{3/2} \quad (11)$$

where

Q = discharge, in cfs

C = coefficient of discharge

b = spillway width, in ft

h = head on spillway, in ft, adjusted when necessary for friction loss between the approach and spillway sections

$\frac{V_1^2}{2g}$ = velocity head, in ft, in approach section

It is readily apparent that reliability of discharge computed by this method depends in large measure upon selection of the proper coefficient C . For most determinations this coefficient must be estimated by comparison with calibrated spillways of similar shape.

Spillways encountered in the field in indirect-measurement work may be sharp-crested, but more generally are broad or round-crested, or of ogee or irregular shape. For most problems, therefore, the

best data available in existing literature must be used in estimating C . Among many possible references, the Survey has found most frequent use for the published works of R. E. Horton [12], the Bureau of Reclamation, [13] and J. N. Bradley [14].

There is frequent need for computing flow over highway embankments. The basic broad-crested weir formula is used, with C selected on basis of results reported by Yarnell and Nagler [15] for flow over railway and highway embankments. The original data of Yarnell and Nagler recently have been reanalyzed by the Survey (for its own use) to define two dimensionless curves; one a curve relating C to the head-breadth ratio for free-fall conditions, and a second, relating correction for submergence to degree of submergence. This dimensionless treatment is more generally applicable than the original method of presentation, insofar as indirect measurement of flow over highway fills is concerned. The weakness of using these relations based on the slightly different geometry representative of railway embankments remains.

Research

The U. S. Geological Survey has an interest in the discharge characteristics of all forms of weirs, whether they be small V-notch plates, masonry mill dams, highway embankments, or major dam spillways. Knowledge of the head-discharge relationships for weirs of all types is essential to the work of the organization, and may occasionally provide the only means of determining an important flood-flow magnitude.

In some cases where peak discharges should be known accurately, the scarcity of information on the discharge characteristics of many forms of weirs and spillways limits the flow-over-dam method to an approximation. In general the discharge coefficient must be determined by experiment, and fortunately, through the application of the principles of dynamic similarity, tests on scale models in the laboratory can be applied to the prototype with confidence.

In 1907 the Geological Survey published *Water Supply Paper* 200 [12] by Robert E. Horton. The importance of Horton's work is demonstrated by the fact that the publication has been reprinted several times and copies have been widely distributed. Horton's work was a complete summary of theoretical and experimental knowledge on the subject as of 1907. In the 48 years since, our theoretical knowledge of fluid behavior and, certainly, our store of empirical knowledge of the discharge of weirs of all forms have increased extensively. But until now, no one has attempted to col-

lect, analyze, correlate, or publish this material in a form which will make it available to the engineering profession.

This task is again being undertaken by the Geological Survey. The project has been designed by and is being carried on under the direction of Professor Carl E. Kindsvater, of the Georgia Institute of Technology. As in the first instance, only a limited amount of original research will be involved. The greater part of the project will consist of collecting, examining, and evaluating all of the data which can be located in this country, and, to some extent, abroad. One of the significant objectives of the proposed study is to correlate these empirical data by methods based on the techniques of modern fluid mechanics. It is envisaged that several years will be required to complete the compilation and research project on coefficients for dams.

An investigation of coefficients applicable to flow over highway embankments is now an active laboratory project at the Georgia Institute of Technology. Various phases of the project will be subjects for masters' theses. The current phase is supported by the J. Waldo Smith fellowship of the American Society of Civil Engineers.

SUMMARY

Indirect methods of measurement offer a practical solution to the determination of peak discharges of natural streams where current-meter measurements cannot be made. While admittedly there are weaknesses and shortcomings in the methods, the necessity for their use and the overall reliability of results fully justify their being called measurements.

Of the four methods described herein, the slope-area method is applied most frequently, with the contracted-opening, flow-through-culvert and flow-over-dam methods following in about that order. Increased reliability of results by the contracted-opening and flow-through-culvert methods is certainly to be expected as a consequence of the recent research. All methods give good results when applied to channel reaches of acceptable standards. Without any doubt it can be said that the overall accuracy of stream flow data published today is greater than ever before. A major contributing factor has been improvement in and greater use of indirect methods of river discharge measurement.

DISCUSSION

Mr. Boyer initiated the discussion by presenting some of his own work on the slope-area method for correlating the Manning coefficient n with the bed roughness, abstracted from a paper published in the Transactions of the AGU in December 1954. Two figures from his paper, plots of y_0/k against $n/y_0^{1/6}$ and $V_{0.2}/V_{0.8}$ against $n/y_0^{1/6}$, were presented as slides. Here k is the average height of bed roughness in the stream and $V_{0.2}$ and $V_{0.8}$ are the velocities at 0.2 and 0.8 depth. He stated that Mr. Stevens of the Soil Conservation Control Council of New Zealand has verified the former plot very well and the latter fairly well, and found that the latter may be a satisfactory method of getting n from lower discharge measurements than those made at the peak of the flood.

Mr. Albertson asked about methods suitable for application to alluvial beds which can continue to scour to equilibrium. Mr. Cragwall replied that the application of indirect methods for computing discharge in alluvial streams has not been encouraging. The Geological Survey has undertaken some studies of scour, which is a difficult problem itself from the instrumentation standpoint, but, so far, it appears that the slope-area method must be used with caution in erodible channels of any type.

Mr. Laursen pointed out two difficulties in the determination of discharge after a flood; first, that the section of an alluvial stream has been changed by the flood, and secondly, where a vegetable screen is present along the low-water bank, as is often the case, it is possible that the water in the overbank is not at the same elevation as the water in the stream. In reply Mr. Cragwall stated that the interpretation of the significance of the high-water marks is included in the estimate of the value of n ; that in their roughness verification program the high-water mark is measured, so that the effect of the nature of the bank is taken into account. No unexplainable variations in n due to this cause have arisen.

Mr. Kolupaila observed that correction factors for the slope of a channel have been mentioned, and inquired as to what is being done about estimating the velocity-head factor a , which is a difficult problem for open-channels and rivers. Mr. Cragwall pointed out that the procedure used in estimating a had been mentioned in the paper. Such estimates were made only for compound channels, i.e. channels with considerable overbank flow, where roughness varies considerably across the channel. In normal channels, a appears to vary little between upstream and downstream sections. In compound channels, conveyances for 5 or 6 subsections, the number

depending upon the shape of the section and the variable roughness across it, are used to calculate a . Thus an attempt is made to incorporate these effects into the estimate of n .

Mr. Barbarossa reported that the Corps of Engineers had measured n values for high flows in alluvial streams as low as 0.013, which is of the order of magnitude of that for concrete. This was attributed to the disappearance of form roughness in such flows. This was mentioned because it appeared to contradict a statement by Mr. Cragwall that he informed his young engineers that n values varied between 0.03 and 0.06. Mr. Cragwall agreed that it is frequently necessary to choose values outside of the range of n from 0.03 to 0.075, and that, in some cases, values as low as 0.012 have been selected by experienced men.

Mr. Banks remarked that there is a recent publication which tends to confirm the Einstein sidewall-elimination method which was published about 14 years ago. He asked how the U.S.G.S. takes sidewall into account in arriving at a final n value. Mr. Cragwall replied that it is hard to describe how n is chosen. He indicated that he has a mental picture of good natural channel of $n = 0.03$, and if other channels differ he increases or decreases this value according to his experience. The method is qualitative, but with the help of 3-dimensional slides it is becoming more consistent.

Mr. Izzard asked Mr. Barbarossa whether actual measurements of the cross-section were taken when the low values of n had been observed, implying that the variation in n might be due to the fact that the bed is lower during higher stages. Mr. Barbarossa replied that that possibility had already been considered, but that the amount of sediment that could be transported could not degrade the bed sufficiently to account for the total change.

REFERENCES

1. "Kansas-Missouri Floods of July 1951," U. S. Geological Survey, *Water-Supply Paper 1139*, 1952.
2. Powell, R. W., "Flow in a Channel of Definite Roughness," *Trans. A.S.C.E.*, v. 111, p. 531-566, 1946.
3. "Resistance to Flow in Rough Channels," *Trans. A.G.U.*, v. 31, p. 575-582, 1950.
4. Robinson, A. R., and Albertson, M. L., "Artificial Roughness Standard for Open Channels," *Trans. A.G.U.*, v. 33, p. 881-888, 1952.
5. Houk, I. E., "Calculation of Flow in Open Channels," State of Ohio, The Miami Conservancy District, Technical Reports, Part IV, 1918.

6. Kindsvater, C. E., and Carter, R. W., "Tranquil Flow through Open-Channel Constrictions," A.S.C.E. convention preprint No. 21, 1952.
7. Kindsvater, C. E., "Tranquil Flow through Open-Channel Constrictions," *A.S.C.E. Separate* 467, 1954.
8. Kindsvater, C. E., Carter, R. W., and Tracy, H. J., "Computation of Peak Discharge at Contractions," *U. S. G. S. Circ.* 284, 1953.
9. Yarnell, D. L., "The Flow of Water through Culverts," Univ. Iowa, Bull. 1, 1926.
10. Straub, L. G., and Morris, H. M., "Hydraulic Tests on Concrete Culvert Pipes," Univ. Minn., Tech. Paper 3, Ser. B, 1950.
11. Straub, L. G., "Hydraulic Tests on Corrugated Metal Pipes," Univ. Minn. Tech. Paper 5, Ser. B, 1950.
12. Horton, R. E., "Weir Experiments Coefficients, and Formulas," *U. S. G. S. Water-Supply Paper* 200, 1907.
13. "Studies of Crests for Overfall Dams," U. S. Bureau of Reclamation, Boulder Canyon Reports, Part VI, Hydraulic Investigations, Bull. 3, 1948.
14. Bradley, J. N., "Discharge Coefficients for Irregular Overfall Spillways," U. S. Bur. Reclamation, Engineering Monographs No. 9, 1952.
15. Yarnell, D. L., and Nagler, F. A., "Flow of Flood Water over Railway and Highway Embankments: U. S. Dept. Agriculture, Bur. Public Roads, Public Roads, v. 11, p. 30-34, 1930.

