Measurement and Computation of Streamflow: Volume 2. Computation of Discharge

By S. E. RANTZ *and others*

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Inch-pound	*Multiply by—*	*Metric*
acres	4.047×10^3	m^2 (square meters)
acre-ft (acre-feet)	1.233×10^3	m^3 (cubic meters)
acre-ft/yr (acre feet per year)	1.233×10^3	m^3/yr (cubic meters per year)
ft (feet)	3.048×10^{-1}	m (meters)
ft/hr (feet per hour)	3.048×10^{-1}	m/hr (meters per hour)
ft/s (feet per second)	3.048×10^{-1}	m/s (meters per second)
ft^3/s (cubic feet per second)	2.832×10^{-2}	m^3/s (cubic meters per second)
in (inches)	2.540×10	mm (millimeters)
lb (pounds)	4.536×10^{-1}	kg (kilograms)
mi (miles)	1.609	km (kilometers)
mi^2 (square miles)	2.590	km^2 (square kilometers)
oz (ounces)	2.835×10^{-2}	kg (kilograms)

MEASUREMENT AND COMPUTATION OF STREAMFLOW

VOLUME 2. COMPUTATION OF DISCHARGE

By S. E. RANTZ and others

CHAPTER 10.—DISCHARGE RATINGS USING SIMPLE STAGE-DISCHARGE RELATIONS

INTRODUCTION

Continuous records of discharge at gaging stations are computed by applying the discharge rating for the stream to records of stage. Discharge ratings may be simple or complex, depending on the number of variables needed to define the stage-discharge relation. This chapter is concerned with ratings in which the discharge can be related to stage alone. (The terms "rating," "rating curve," "stage rating," and "stage-discharge relation" are synonymous and are used here interchangeably.)

Discharge ratings for gaging stations are usually determined empirically by means of periodic measurements of discharge and stage. The discharge measurements are usually made by current meter. Measured discharge is then plotted against concurrent stage on graph paper to define the rating curve. At a new station many discharge measurements are needed to define the stage-discharge relation throughout the entire range of stage. Periodic measurements are needed thereafter to either confirm the permanence of the rating or to follow changes (shifts) in the rating. A minimum of 10 discharge measurements per year is recommended, unless it has been demonstrated that the stage-discharge relation is unvarying with time. In that event the frequency of measurements may be reduced. It is of prime importance that the stage-discharge relation be defined for flood conditions and for periods when the rating is subject to shifts as a result of ice formation (see section titled, "Effect of Ice Formation on Discharge Ratings") or as a result of the variable channel and control conditions discussed in the section titled, "Shifts in the Discharge Rating." It is essential that the stream-gaging program have sufficient flexibility to provide for the nonroutine scheduling of additional measurements of discharge at those times.

If the discharge measurements cover the entire range of stage experienced during a period of time when the stage-discharge relation is stable, there is little problem in defining the discharge rating for that period. On the other hand, if, as is usually the case, discharge measurements are lacking to define the upper end of the rating, the defined lower part of the rating curve must be extrapolated to the highest stage experienced. Such extrapolations are always subject to error, but the error may be reduced if the analyst has a knowledge of the principles that govern the shape of rating curves. Much of the material in this chapter is directed toward a discussion of those principles, so that when the hydrographer is faced with the problem of extending the high-water end of a rating curve he can decide whether the extrapolation should be a straight line, or whether it should be concave upward or concave downward.

The problem of extrapolation can be circumvented, of course, if the unmeasured peak discharge is determined by use of the indirect methods discussed in chapter 9. In the absence of such peak-discharge determinations, some of the uncertainty in extrapolating the rating may be reduced by the use of one or more of several methods of estimating the discharge corresponding to high values of stage. Four such methods are discussed in the section titled "High-flow Extrapolation."

In the discussions that follow it was generally impractical to use both English and metric units, except where basic equations are given. Consequently English units are used throughout, unless otherwise noted.

STAGE-DISCHARGE CONTROLS

The subject of stage-discharge controls was discussed in detail in chapter 3, but a brief summary at this point is appropriate.

The relation of stage to discharge is usually controlled by a section or reach of channel downstream from the gage that is known as the station control. A section control may be natural or manmade; it may be a ledge of rock across the channel, a boulder-covered riffle, an overflow dam, or any other physical feature capable of maintaining a fairly stable relation between stage and discharge. Section controls are often effective only at low discharges and are completely submerged by channel control at medium and high discharges. Channel control consists of all the physical features of the channel that determine the stage of the river at a given point for a given rate of flow. These features include the size, slope, roughness, alinement, constrictions and expansions, and shape of the channel. The reach of channel that acts as the control may lengthen as the discharge increases, introducing new features that affect the stage-discharge relation.

Knowledge of the channel features that control the stage-discharge relation is important. The development of stage-discharge curves where more than one control is effective, and where the number of measurements is limited, usually requires judgment in interpolating between measurements and in extrapolating beyond the highest measurements. That is particularly true where the controls are not permanent and the various discharge measurements are representative of changes in the positioning of segments of the stage-discharge curve.

GRAPHICAL PLOTTING OF RATING CURVES

Stage-discharge relations are usually developed from a graphical analysis of the discharge measurements plotted on either rectangular-coordinate or logarithmic plotting paper. In a preliminary step the discharge measurements available for analysis are tabulated and summarized on a form such as that shown in figure 139. Discharge is then plotted as the abscissa, corresponding gage height is plotted as the ordinate, and a curve or line is fitted by eye to the plotted points. The plotted points carry the identifying measurement numbers given in figure 139; the discharge measurements are numbered consecutively in chronological order so that time trends can be identified.

At recording-gage stations that use stilling wells, systematic and significantly large differences between inside (recorded) gage heights and outside gage heights often occur during periods of high stage, usually as a result of intake drawdown (see section in chapter 4 titled, "Stilling Wells"). For stations where such differences occur, both inside and outside gage heights for high-water discharge measurements are recorded on the form shown in figure 139, and in plotting the measurements for rating analysis, the outside gage readings are used first. The stage-discharge relation is drawn through the outside gage readings of the high-water discharge measurements and is extended to the stage of the outside high-water marks that are observed for each flood event. The stage-discharge relation is next transposed to correspond with the inside gage heights obtained from the stage-recorder at the times of discharge measurement and at flood peaks. It is this transposed stage-discharge relation that is used with recorded stages to compute the discharge.

The rationale behind the above procedure is as follows. The outside gage readings are used for developing the rating because the hydraulic principles on which the rating is based require the use of the true stage of the stream. The transposition of the rating to inside (recorded) stages is then made because the recorded stages will be used with the rating to determine discharge. The recorded stages are used for discharge determination because if differences exist between inside and outside gage readings, those differences will be known only for those times when the two gages are read concurrently. If the

FORM 9-207
(Rev. 11-69)

UNITED STATES DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY (WATER RESOURCES DIVISION)
DISCHARGE MEASUREMENT SUMMARY SHEET

Station No 11-4770

Discharge measurements of Eel River at Scotia, Calif. during the year ending Sept 30, 1966

No	Date	Made by—	Width	Area	Mean velocity	Gage height	Discharge	Rating: Shift adj	Rating: Percent diff	Method	Number meas sections	Gage height change	Time	Meas rated	Water temp °C	Outside gage	REMARKS
	1965		Feet	Sq ft	Fps	Feet	Cfs	Feet				Feet	Hr				
495	Aug. 31	Hammond McGuire	153	W 199	0.78	8.92	155			0.6	34	-.01	1.2	G	74	8.98	
496	Oct. 5	La Rue	155	W 148	0.82	8.79	121			.6	28	0	.5	G	61	8.87	Zero flow = 8.5 ± 0.05
497	Nov 2	do	154	W 158	0.85	8.88	135			.6	29	0	.5	G	60	8.89	Zero flow = 8.5 ± 0.05
498	30	La Rue Crumrine	454	2,030	2.47	12.52	5,010			.6; .2/.8	33	-.01	1.1	G	46	12.55	
499	1966 Jan. 3	La Rue	553	6,790	7.19	21.03	48,800			.2/.8	29	+0.52	1.75	G/F	46	21.05	
500	25	Hammond Crumrine	512	2350	1.73	12.74	4,050			.6, .2/.8	25	-.02	1.3	P	45	12.80	peak (Jan 5) 45.84
501	25	do	506	2270	1.71	12.73	3,880			.6, .2/.8	28	0	2	F	—	12.80	
502	31	La Rue	543	3960	4.18	16.08	16,500			.6, .2/.8	32	-.07	1.25	G	—	16.13	
503	Feb. 21	do	533	3000	2.70	13.85	8,100			.2/.8	36	-.03	1.4	G	48	13.86	
504	Mar. 31	do	2-Channels		—	13.59	7,420			.2/.8	37	.00	1.6	G	57	13.59	
505	Apr 28	do	454	1890	1.90	12.13	3,600			.6, .2/.8	36	-.01	1.3	G	58	12.18	
506	June 1	Palmer	434	1200	0.83	10.62	1,000			.6, .2/.8	40	0	1.7	G	62	10.64	
507	July 6	do	196	193	1.56	9.82	302			.6	39	0	.9	G	70	9.83	
508	Aug. 8	do	184	W 126	1.11	9.40	140			.6	38	0	.75	F/P	67	—	Zero flow = 9.05 ± 0.05
509	Sept. 12	Hammond	77	W 74.1	1.48	9.26	110			.6	31	0	.8	F	66	9.3 ± .02	Zero flow = 9.0 ± 0.0[illegible]
510	Oct. 4	Palmer	184	116	1.01	9.30	117			.6	34	0	.7	F	64	9.3 ± .01	

Copied by   Computed by W-Wading   Checked by

FIGURE 139.—Example of form used for tabulating and summarizing current-meter discharge measurements.

outside gage heights were used with the rating to determine discharge, variable corrections, either known or assumed, would have to be applied to recorded gage heights to convert them to outside stages. We have digressed here to discuss differences between inside and outside gage heights, because in the discussions that follow no distinction between the two gages will be made.

The use of logarithmic plotting paper is usually preferred for graphical analysis of the rating because in the usual situation of compound controls, changes in the slope of the logarithmically plotted rating identify the range in stage for which the individual controls are effective. Furthermore, the portion of the rating curve that is applicable to any particular control may be linearized for rational extrapolation or interpolation. A discussion of the characteristics of logarithmic plotting follows.

The measured distance between any two ordinates or abscissas on logarithmic graph paper, whose values are printed or indicated on the sheet by the manufacturer of the paper, represents the difference between the logarithms of those values. Consequently, the measured distance is related to the ratio of the two values. Therefore, the distance between pairs of numbers such as 1 and 2, 2 and 4, 3 and 6, 5 and 10, are all equal because the ratios of the various pairs are identical. Thus the logarithmic scale of either the ordinates or the abscissas is maintained if all printed numbers on the scale are multiplied or divided by a constant. This property of the paper has practical value. For example, assume that the logarithmic plotting paper available has two cycles (fig. 140), and that ordinates ranging from 0.3 to 15.0 are to be plotted. If the printed scale of ordinates is used and the bottom line is called 0.1, the top line of the paper becomes 10.0, and values between 10.0 and 15.0 cannot be accommodated. However, the logarithmic scale will not be distorted if all values are multiplied by a constant. For this particular problem, 2 is the constant used in figure 140, and now the desired range of 0.3 to 15.0 can be accommodated. Examination of figure 140 shows that the change in scale has not changed the distance between any given pair of ordinates; the position of the ordinate scale has merely been transposed.

We turn now to a theoretical discussion of rating curves plotted on logarithmic graph paper. A rating curve, or a segment of a rating curve, that plots as a straight line of logarithmic paper has the equation,

$$Q = p(G - e)^N, \tag{53}$$

where

Q is discharge;

$(G - e)$ is head or depth of water on the control—this value is indicated by the ordinate scale printed by the manufacturer or

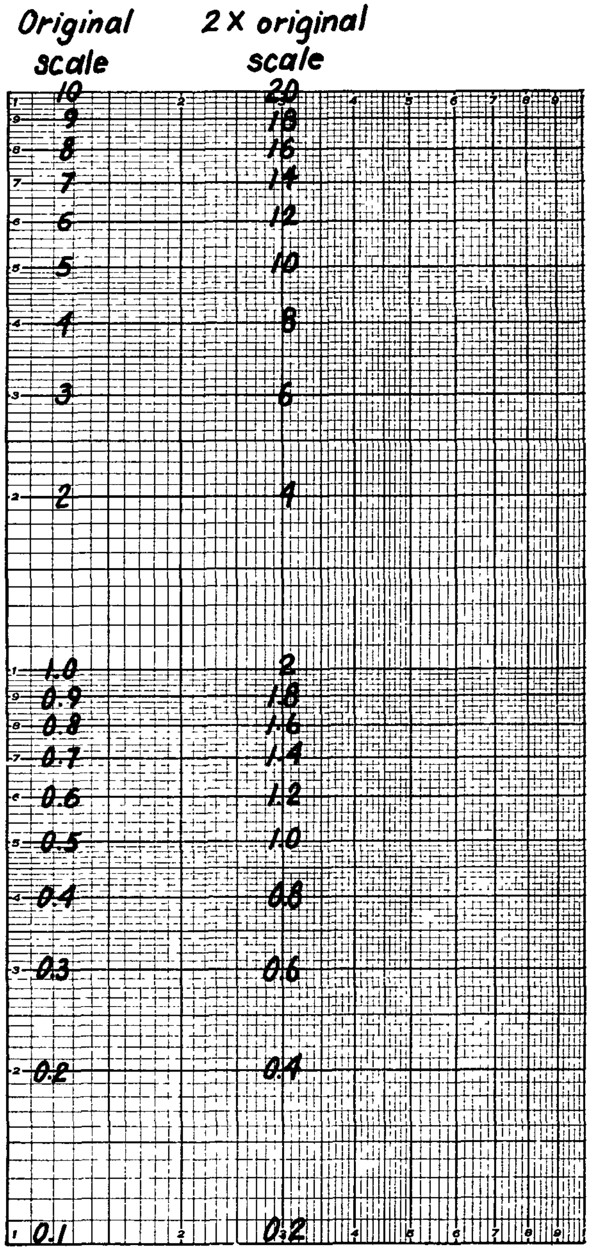


FIGURE 140.—Example showing how the logarithmic scale of graph paper may be transposed.

by the ordinate scale that has been transposed, as explained in the preceding paragraph;

G is gage height of the water surface;

e is gage height of zero flow for a section control of regular shape, or the gage height of effective zero flow for a channel control or a section control of irregular shape;

p is a constant that is numerically equal to the discharge when the head $(G - e)$ equals 1.0 ft or 1.0 m, depending on whether English or metric units are used; and

N is slope of the rating curve. (Slope in equation 53 is the ratio of the horizontal distance to the vertical distance. This unconventional way of measuring slope is necessary because the dependent variable Q is always plotted as the abscissa.)

We assume now that a segment of an established logarithmic rating is linear, and we examine the effect on the rating of changes to the control. If the width of the control increases, p increases and the new rating will be parallel to and to the right of the original rating. If the width of the control decreases, the opposite effect occurs; p decreases and the new rating will be parallel to and to the left of the original rating. If the control scours, e decreases and the depth $(G - e)$ for a given gage height increases; the new rating moves to the right and will no longer be a straight line but will be a curve that is concave downward. If the control becomes built up by deposition, e increases and the depth $(G - e)$ for a given gage height decreases; the new rating moves to the left and is no longer linear but is a curve that is concave upward.

When discharge measurements are originally plotted on logarithmic paper, no consideration is given to values of e. The gage height of each measurement is plotted using the ordinate scale provided by the manufacturer or, if necessary, an ordinate scale that has been transposed as illustrated in figure 140. We refer now to figure 141. The inside scale $(e = 0)$ is the scale printed by the paper manufacturer. Assume that the discharge measurements have been plotted to that scale and that they define the curvilinear relation between gage height (G) and discharge (Q) that is shown in the topmost curve. For the purpose of extrapolating the relation, a value of e is sought, which when applied to G, will result in a linear relation between $(G - e)$ and Q. If we are dealing with a section control of regular shape, the value of e will be known; it will be the gage height of the lowest point of the control (point of zero flow). If we are dealing with a channel control or section control of irregular shape, the value of e is the gage height of *effective* zero flow. The gage height of effective zero flow is not the gage height of some identifiable feature on the irregular section control or in the channel but is actually a mathematical constant that is considered as a gage height to preserve the concept of a logarithmically linear head-discharge relation. Effective zero flow is usually determined by a method of successive approximations.

In successive trials, the ordinate scale in figure 141 is varied for e values of 1, 2 and 3 ft, each of which results in a different curve, but each new curve still represents the same rating as the top curve. For example, a discharge of 30 ft^3/s corresponds to a gage height (G) of 5.5 ft on all four curves. The true value of e is 2 ft, and thus the rating plots as a straight line if the ordinate scale numbers are increased by that value. In other words, while even on the new scale a discharge of 30 ft^3/s corresponds to a gage height (G) of 5.5 ft, the head or depth on the control for a discharge of 30 ft^3/s is ($G-e$), or 3.5 ft; the linear rating marked e = 2 crosses the ordinate for 30 ft^3/s at 5.5 ft on the new scale and at 3.5 ft on the manufacturer's, or inside, scale. If values of e smaller than the true value of 2 ft are used, the rating curve will be concave upward, if values of e greater than 2 ft are used, the curve will be concave downward. The value of e to be used for a rating curve, or for a segment of a rating curve, can thus be determined by adding or subtracting trial values of e to the numbered scales on the logarithmic plotting paper until a value is found that results in a straight-line plot of the rating. It is important to note that if the logarithmic ordinate scale must be transposed by multiplication or division to accommodate the range of stage to be plotted, that transposition must be made before the ordinate scale is manipulated for values of e.

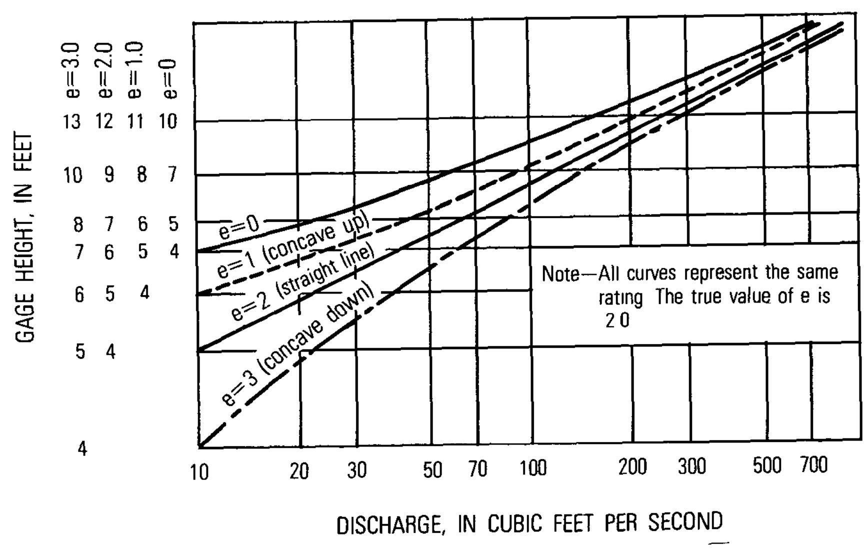


FIGURE 141.—Rating-curve shapes resulting from the use of differing values of effective zero flow.

A more direct solution for e, as described by Johnson (1952) is illustrated in figure 142. A plot of G versus Q has resulted in the solid-line curve which is to be linearized by subtracting a value of e from each value of G. The part of the rating between points 1 and 2 is chosen, and values of G_1, G_2, Q_1 and Q_2 are picked from the coordinate scales. A value of Q_3 is next computed, such that

$$Q^2_3 = Q_1 Q_2.$$

From the solid-line curve, the value of G_3 that corresponds to Q_3 is picked. In accordance with the properties of a straight line on logarithmic plotting paper,

$$(G_3 - e)^2 = (G_1 - e)(G_2 - e). \tag{54}$$

Expansion of terms in equation 54 leads to equation 55 which provides a direct solution for e.

$$e = \frac{G_1 G_2 - G_3^2}{G_1 + G_2 - 2G_3} \tag{55}$$

A logarithmic rating curve is seldom a straight line or a gentle curve for the entire range in stage. Even where a single cross section of the channel is the control for all stages, a sharp break in the

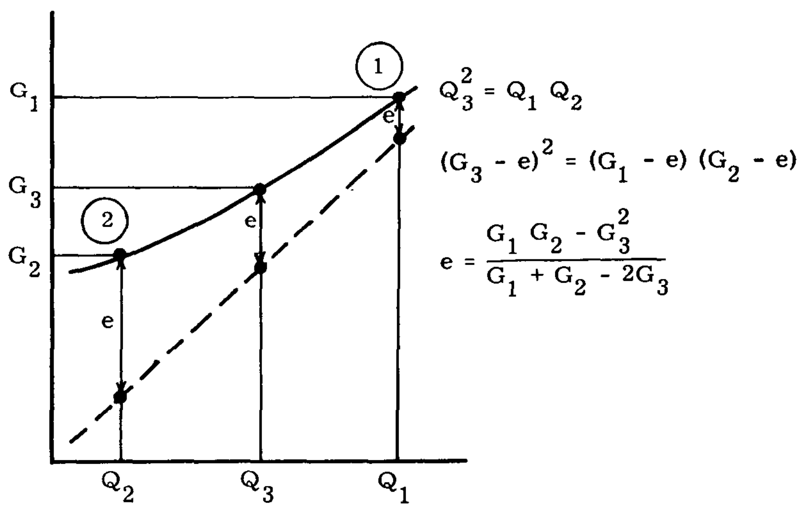


FIGURE 142.—Schematic representation of the linearization of a curve on logarithmic graph paper.

contour of the cross section, such as an overflow plain, will cause a break in the slope of the rating curve. Commonly, however, a break in slope is due to the low-water control being drowned out by a downstream section control becoming effective or by channel control becoming effective.

The use of rectangular-coordinate paper for rating analysis has certain advantages, particularly in the study of the pattern of shifts in the lower part of the rating. A change in the low-flow rating at many sites results from a change in the elevation of effective zero flow (e), which means a constant shift in gage height. A shift of that kind is more easily visualized on rectangular-coordinate paper because on that paper the shift curve is parallel to the original rating curve, the two curves being separated by a vertical distance equal to the change in the value of e. On logarithmic paper the two curves will be separated by a variable distance which decreases as stage increases. A further advantage of rectangular-coordinate paper is the fact that the point of zero flow can be plotted directly on rectangular-coordinate paper, thereby facilitating extrapolation of the low-water end of the rating curve. That cannot be done on logarithmic paper because zero values cannot be shown on that type of paper.

As a general rule logarithmic plotting should be used initially in developing the general shape of the rating. The final curve may be displayed on either type of graph paper and used as a base curve for the analysis of shifts. A combination of the two types of graph paper is frequently used with the lower part of the rating plotted on an inset of rectangular-coordinate paper or on a separate sheet of rectangular-coordinate paper.

SECTION CONTROLS

ARTIFICIAL CONTROLS

At this point we digress from the subject of logarithmic rating curves to discuss the ratings for artificial section controls. A knowledge of the rating characteristics of controls of standard shape is necessary for an understanding of the rating characteristics of natural controls, almost all of which have irregular shapes. On pages that follow we first discuss thin-plate weirs, then broad-crested weirs, and finally flumes.

Thin-plate weirs are generally used in small clear-flowing streams, particularly where high accuracy is desired and adequate maintenance can be provided, as in small research watersheds. Flumes are preferred for use in small streams and canals that carry sediment and debris, and in other situations where the head loss (backwater) associated with a thin-plate weir is unacceptable. Most types of flume may also be used under conditions of submergence, as opposed to free-flow conditions, thereby permitting them to operate with even smaller head loss but with some loss of accuracy of the stage-discharge relation. The broad-crested weirs are commonly used in the larger streams.

TRANSFERABILITY OF LABORATORY RATINGS

Standard shapes or dimensions are commonly used in building artificial controls, and many of these standard structures have been rated in laboratory model studies (World Meteorological Organization, 1971). The transfer of a laboratory discharge rating to a structure in the field requires the existence, and maintenance, of similitude between laboratory model and prototype, not only with regard to the structure, but also with regard to the approach channel. For example, scour and (or) fill in the approach channel will change the head-discharge relation, as will algal growth on the control structure. Both the structure and the approach channel must be kept free from accumulations of debris, sediment, and vegetal growth. Flow conditions downstream from the structure are significant only to the extent that they control the tailwater elevation, which may influence the operation of structures designed for free-flow conditions.

Because of the likelihood of the existence or development of conditions that differ from those specified in a laboratory model study, the policy of the Geological Survey is to calibrate the prototype control in the field by discharge measurements for the entire range of stage that is experienced. (See section in chapter 3 titled, "Artificial Controls.") In-place calibration is sometimes dispensed with where the artificial control is a standard thin-plate weir having negligible velocity of approach.

THIN-PLATE WEIRS

The surface of the weir over which the water flows is the crest of the weir. A thin-plate weir has its crest beveled to a chisel edge and is always installed with the beveled face on the downstream side. The crest of a thin-plate weir is highly susceptible to damage from floating debris, and therefore such weirs are used as control structures almost solely in canals whose flow is free of floating debris. Thin-plate weirs are not satisfactory for use in canals carrying sediment-laden water because they trap sediment and thereby cause the gage pool to fill with sediment, sometimes to a level above the weir crest. The banks of the canal must also be high enough to accommodate the increase in stage (backwater) caused by the installation of the weir, the weir plate being an impedance to flow in the canal. The commonly used shapes for thin-plate weirs are rectangular, trapezoidal, and triangular or V-notch.

The information needed to compute the discharge over a thin-plate weir is as follows:

1. Static head (h), which is the difference in elevation between the weir crest and the water surface at the approach section; the approach section is located upstream from the weir face a distance equal to about $3h$ or more. (See section in chapter 2 titled, "Considerations in Specific Site Selection" for discussion of location of gage intakes.)
2. Length of crest of weir (b) if weir is rectangular or trapezoidal.
3. Width of channel in the plane of the weir face (B).
4. Angle of side slopes if weir is triangular or trapezoidal.
5. Average depth of streambed below elevation of weir crest (P). P is measured in the approach section.

RECTANGULAR THIN-PLATE WEIR

Flow over a rectangular thin-plate weir is illustrated in figure 143. The discharge equation for this type of weir is:

$$Q = Cbh^{3/2}, \tag{56}$$

where

Q = discharge,
C = discharge coefficient,
b = length of weir crest normal to flow, and
h = static or piezometric head on a weir, referred to the weir crest.

Information on discharge coefficients for rectangular thin-plate weirs is available from the investigations of Kindsvater and Carter (1959) and others, and is given in the previously cited WMO Technical Note No. 117 (1971). Those investigations show that the coefficient for free discharge is a function of certain dimensionless ratios which describe the geometry of the channel and the weir;

$$C = f\left(\frac{h}{P}, \frac{b}{B}, E\right), \tag{57}$$

where E is the slope of the weir face; the other variables are depicted in figure 143.

The relation between C, h/P and E for weirs with no side contraction ($b/B = 1.0$) is shown in figure 144, where each of the four curves corresponds to a particular value of E. The coefficient is defined in the range of h/P from 0 to 5. The value of the coefficient becomes uncertain at high values of h/P. The greater the value of h/P, the greater the velocity of approach, and therefore the greater the coefficient. The coefficients in figure 144 are for use with English units, where all linear measurements are expressed in feet and discharge is in cubic feet per second. If linear measurements are expressed in meters and discharge is in cubic meters per second, all values of C must be multiplied by the factor 0.552.

Side contractions reduce the effective length of the weir crest. That effect is accounted for by multiplying the value of C from figure 144 by a correction factor that is a function of b/B, h/P, and the degree of rounding of the upstream vertical edge of the weir-notch abutments. Rounding is a factor only in the situation where the horizontal weir crest is set between vertical abutments. For a rectangular thin-plate weir with sharp-edged entry, the correction factor is k_c; appropriate values of k_c are obtained from the curves in figure 145. For a

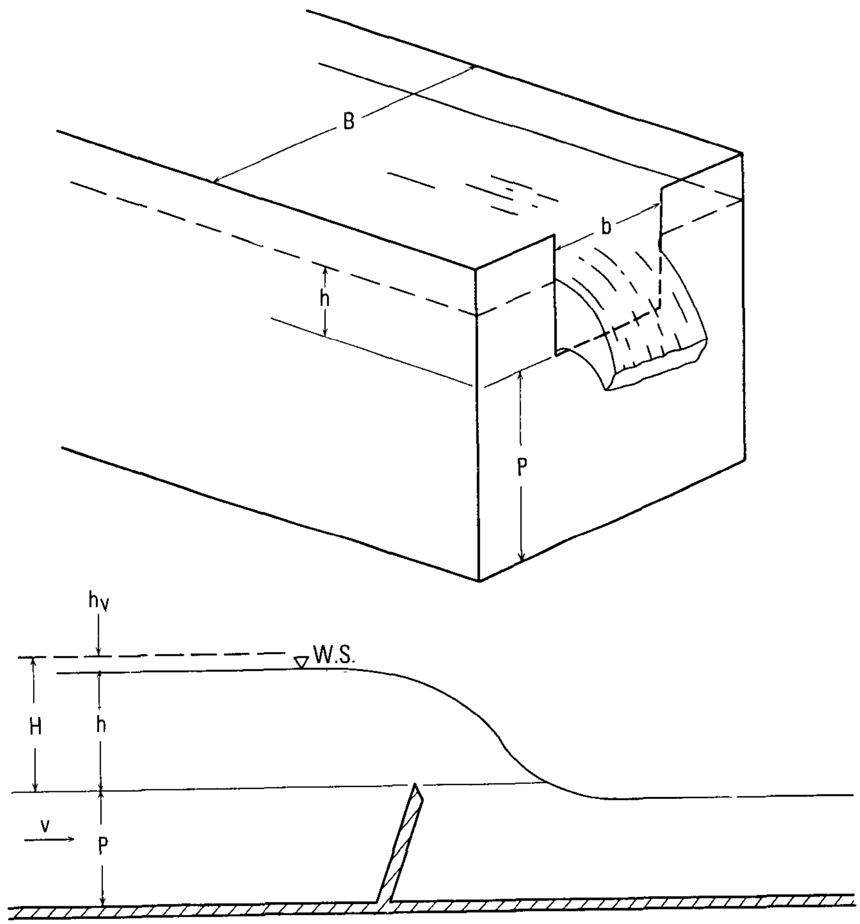


FIGURE 143.—Definition sketch of a rectangular thin-plate weir.

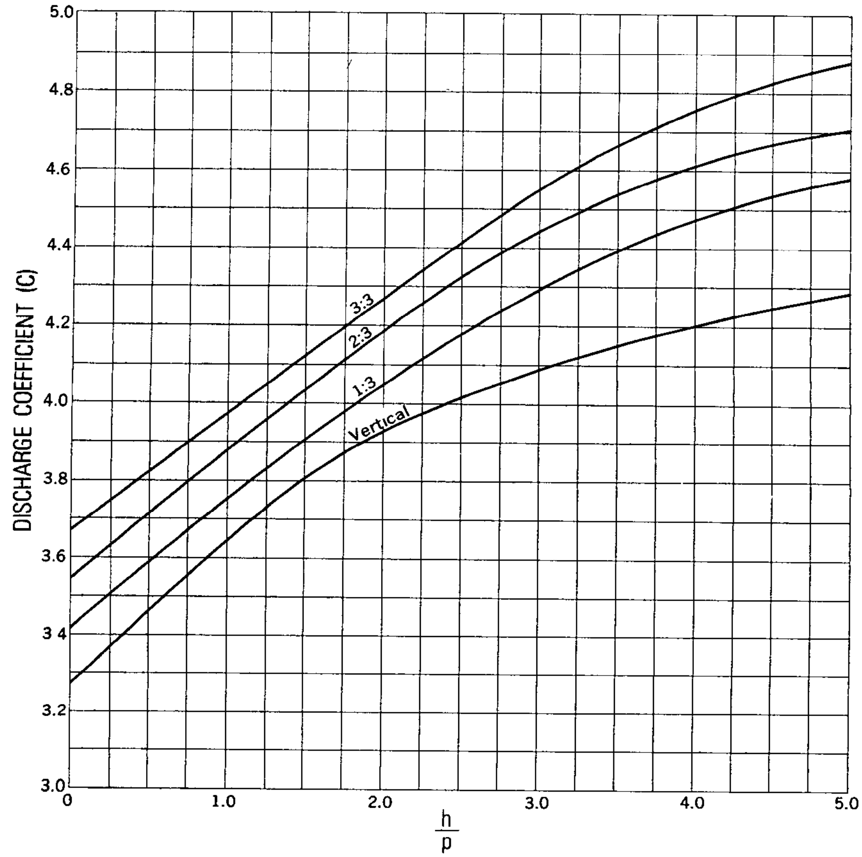


FIGURE 144.—Discharge coefficients for full-width, vertical and inclined, rectangular thin-plate weirs.

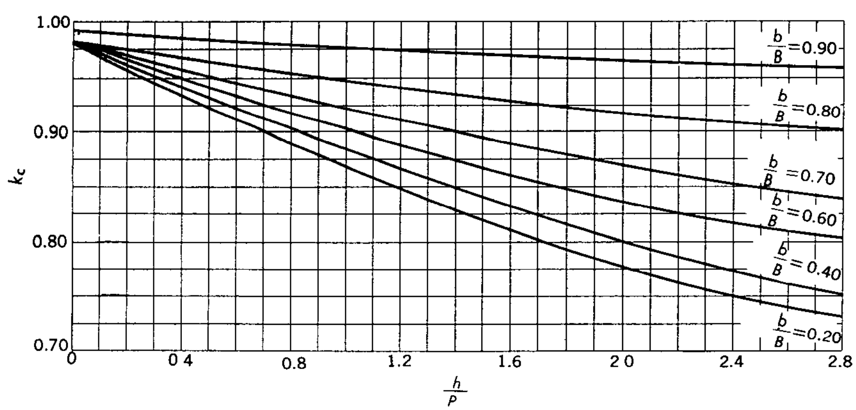


FIGURE 145.—Definition of adjustment factor, k_c, for contracted rectangular thin-plate weirs.

rectangular thin-plate weir having vertical abutments rounded with radius r, the correction factor, when $r/b \geqslant 0.12$, is assumed to equal $(1 + k_c)/2$, where again k_c is read from figure 145. For rounded abutments having a lesser value of r/b, the correction factor is obtained by interpolating between the appropriate k_c value from figure 145 and the value $(1 + k_c)/2$. In other words, for given values of b/B, h/P, and r/b between 0 and 0.12, we interpolate linearly between the value of k_c (corresponds to $r/b=0$) and the value of $(1 + k_c)/2$ (corresponds to $r/b = 0.12$).

We will now prepare the rating for a hypothetical rectangular thin-plate weir for the purpose of examining the implications of a logarithmic plot of the rating. The discharges corresponding to various stages will be computed by use of the theoretical equation for a rectangular weir, using figures 144 and 145 to obtain the constant in the equation. We will assume that the computed discharges represent the results of carefully made discharge measurements.

Assume that we have a rectangular thin-plate weir, with vertical face, and that discharge measurements (computations) have been made at heads ranging from 0.1 ft to 7.0 ft. The dimensions of the weir using the symbols in figure 142 are given in table 16. The data and computations are also shown in table 16 and should be self-explanatory. The weir constant is actually equal to $C(k_c)$. The stages and corresponding discharges are plotted on logarithmic graph paper and fitted with a curve by eye in figure 146.

Figure 146 shows that a tangent can be fitted to the plotted points at heads greater than 0.3 ft ($G=1.3$ ft). The intercept (p) of the tangent at $G - e=1.0$ ft is 67 ft^3/s and the measured slope of the tangent is 1.55. (Note that the slope of the rating curve Q/h is the ratio of the horizontal distance to the vertical distance.) In accordance with equation 53, the equation of the tangent is therefore $Q=67h^{1.55}$ However, the equation for discharge over a rectangular weir is $Q=(Ck_cb)h^{1.50}$. Therefore (Ck_cb) must vary with stage, as we know it does, and $Ck_cb=67h^{0.05}$; the exponent 0.05 is obtained by subtracting the theoretical exponent 1.50 from the empirical exponent 1.55. Because b has a constant value of 20 ft, $Ck_c=3.35h^{0.05}$; the coefficient 3.35 is obtained by dividing the original coefficient (67) by the value of b (20 ft). We can extrapolate the tangent in figure 146 with some confidence. If we wish to determine the discharge from the curve for a gage height of 11 ft ($h=10$ ft), the extrapolated value of Q is 2,380 ft^3/s; that is, if a value of 10 ft is substituted in the equation $Q=67h^{1.55}$, Q will equal 2,380 ft^3/s. That value matches the true value computed on the bottom line of table 16 for a gage height of 11 ft.

TRAPEZOIDAL THIN-PLATE WEIR

Few experimental data are available for determining the discharge

TABLE 16.—*Computation of discharge rating for a hypothetical rectangular thin-plate weir*

[*Given* P=2.0 ft, b=20 ft, B=25 ft, b/B=0.8 Gage height of weir crest (e)=1.0 ft, h=Gage height (G) minus e]

G (ft)	$h=G-e$ (ft)	h/P	C (from fig 144)	k_t (from fig 145)	Ck_t	$Ck_t b$	$h^{3/2}$	$Q=Ck_t bh^{3/2}$ (ft³/s)
1.0	0	----	----	----	----	----	----	0
1.1	0.1	0.05	3.29	0.980	3.224	64.48	0.0316	2.04
1.3	.3	.15	3.33	.977	3.253	65.06	.1643	10.7
1.5	.5	.25	3.37	.973	3.279	65.58	.3536	23.2
2.0	1.0	.50	3.46	.965	3.339	66.78	1.000	66.8
2.5	1.5	.75	3.56	.955	3.400	68.00	1.837	125
3.0	2.0	1.0	3.65	.947	3.456	69.12	2.828	195
4.0	3.0	1.5	3.81	.930	3.543	70.86	5.196	368
5.0	4.0	2.0	3.93	.920	3.616	72.32	8.000	579
6.0	5.0	2.5	4.02	.905	3.638	72.76	11.18	813
7.0	6.0	3.0	4.08	.900	3.672	73.44	14.70	1,080
Computation to check extrapolation obtained from figure 146								
11.0	10.0	5.0	4.28	.88	3.766	75.32	31.62	2,380

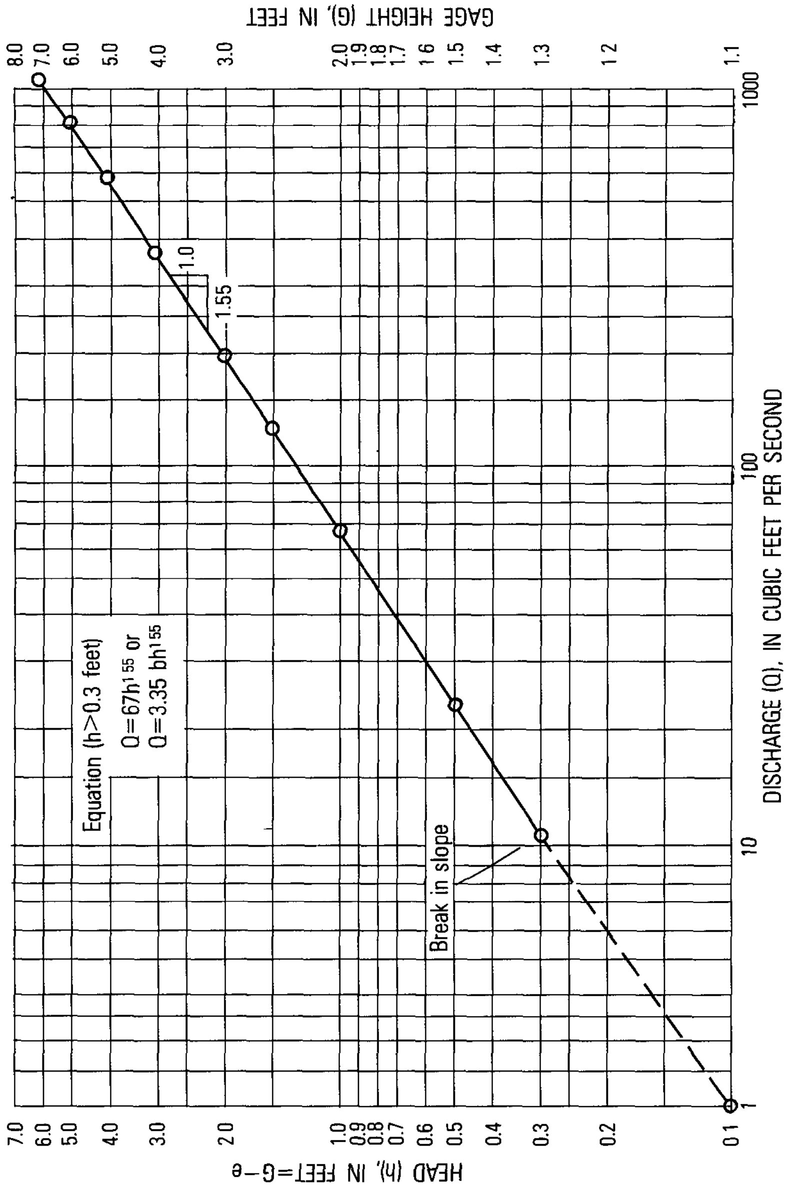


FIGURE 146.—Rating curve for hypothetical rectangular thin-plate weir.

coefficients of trapezoidal weirs (fig. 147). One exception is the vertical Cippoletti weir, which is a sharp-crested trapezoidal weir whose sides have a slope of 1 horizontal (x) to 4 vertical (y). The slope of the sides is approximately that required to obtain a discharge through the two triangular parts of the weir opening that equals the decrease in discharge resulting from end contractions. In other words, the Cippoletti weir acts as a rectangular thin-plate weir whose crest length is equal to b and whose contraction coefficient, k_c, is equal to 1.0. The dimension B (fig. 147) for a Cippoletti weir has little bearing on the discharge. The equation used to compute discharge is again

$$Q=Cbh^{3/2}, \tag{56}$$

and close approximations of values of C are obtained from figure 144. The head, h, and height of notch, P, are both measured in the approach section.

If we compute the discharge for a vertical thin-plate Cippoletti weir whose value of b is 20 ft and whose value of P is 2.0 ft, similar to the dimensions used in computing the hypothetical rating shown in table 16, the rating will approximate that obtained for the thin-plate rectangular weir of table 16. The only difference in discharge will be that attributable to the fact that the value of k_c is 1.00 for all values of head for the Cippoletti weir. A logarithmic plot of the rating (not shown here) indicates that the equation for all but the very small values of head is

$$Q=69h^{1.58}, \text{ (English units)}$$

meaning that $C=3.45h^{0.08}$.

For trapezoidal weirs other than Cippoletti weirs, the general empirical equation for discharge is

$$Q=Cb(h+h_v)^N, \tag{58}$$

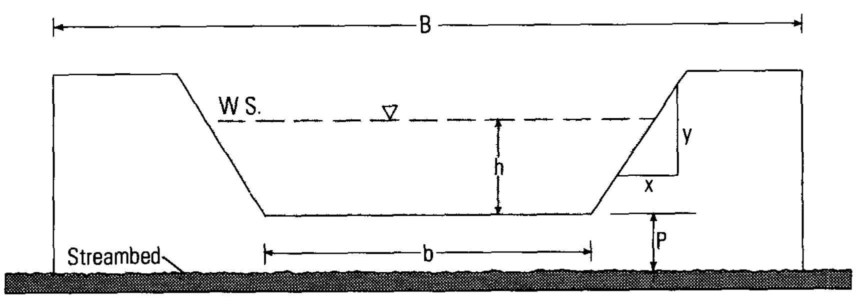


FIGURE 147.—Sketch of upstream face of a trapezoidal weir.

where h_v, the velocity head at the approach section, is equal to $V_a^2/2g$, V_a being the mean velocity in the approach section and g being the acceleration of gravity. The coefficient C and exponent N must be determined from current-meter discharge measurements that cover the entire range of stage that is experienced. If discharge measurements are not available for the highest stage experienced, a rating curve is obtained by plotting on logarithmic paper the head $(G-e)$ against discharge (Q) for the measurements that have been obtained, and then fitting a curve to the plotted points. The upper end of that curve should be a tangent, or possibly an extremely flat curve, that can be extrapolated to the highest stage experienced. Because the limiting shapes of a trapezoid are a rectangle at one extreme and a triangle at the other, the slope of the tangent will lie somewhere between 1.5, which is the theoretical slope for a rectangular weir, and 2.5 which is the theoretical slope for a triangular weir. The closer the shape is to a rectangle, the closer the slope will be to 1.5; the closer the shape is to a triangle, the closer the slope will be to 2.5.

The reader will note the difference in form between equations 56 and 58. Equation 56 uses static head (h), whereas equation 58 uses total head $(h+h_v)$. Velocity head is a factor in any discharge equation for a weir. In the more modern laboratory studies of weir discharge, the static-head term is used in the discharge equation, and velocity head, as indicated by a term h/P, is used directly as a variable in the determination of C. (See equation 57.) In older laboratory investigations, a more empirical approach for determining C was followed in that the total-head term was used in the discharge equation and the values of C that were determined do not vary directly with change in velocity head. Both forms of the weir-discharge equations will be found in this manual; the older type of equation is shown wherever it has not been superseded by later laboratory studies.

TRIANGULAR OR V-NOTCH THIN-PLATE WEIR

Triangular or V-notch thin-plate weirs (fig. 148) are installed at sites where low discharges occur; they are highly sensitive to low flows but have less capacity than rectangular or trapezoidal weirs. Because the area of the notch is invariably small compared to the cross-sectional area of the channel, water is pooled upstream from the weir and the approach velocity is necessarily low. The approach velocity head can usually be neglected in computing the discharge for a 90° V-notch weir ($\Theta=90°$ in fig. 148). Actually, for values of Θ equal to or less than 90°, it has been specified that velocity of approach is negligible if h/P is less than 0.4 and h/B is less than 0.2 (WMO Tech. Note 117, 1971). Whether or not the velocity head can be ignored in computing discharges for V-notch weirs having central angles greater than 90° depends on the relative size of the areas occupied by the water in the notch and the water in the approach section. Virtually no experimental work has been done with triangular weirs having significant velocity of approach, and all equations discussed below are for installations where the velocity of approach can be neglected. Furthermore the equations are applicable only for thin-plate V-notch weirs whose faces are vertical.

Both the cross-sectional area of the flow in the notch and its velocity in the notch are functions of the head (h). Consequently, the general equation for a triangular thin-plate weir is $Q=Ch^{x}$, and the constants in that equation do not vary greatly from those in the following equations:

$$Q=2.5(\tan \Theta/2)h^{5/2},$$

where h is in feet and Q is in cubic feet per second; or

$$Q=1.38(\tan \Theta/2)h^{5/2},$$

where h is in meters and Q is in cubic meters per second.
The head is measured in the approach section, a distance about $3h$ upstream from the weir face. From an earlier discussion it is apparent that the above equations will plot as straight lines on logarithmic graph paper. The slope of the ratings will be 2.5, and the intercept, where $h=1$, will be either 2.5 tan $\Theta/2$ or 1.38 tan $\Theta/2$, depending on whether English or metric units are used.

V-notch weirs are most commonly built with a central angle of 90°. Much experimental work has been done with thin-plate 90°

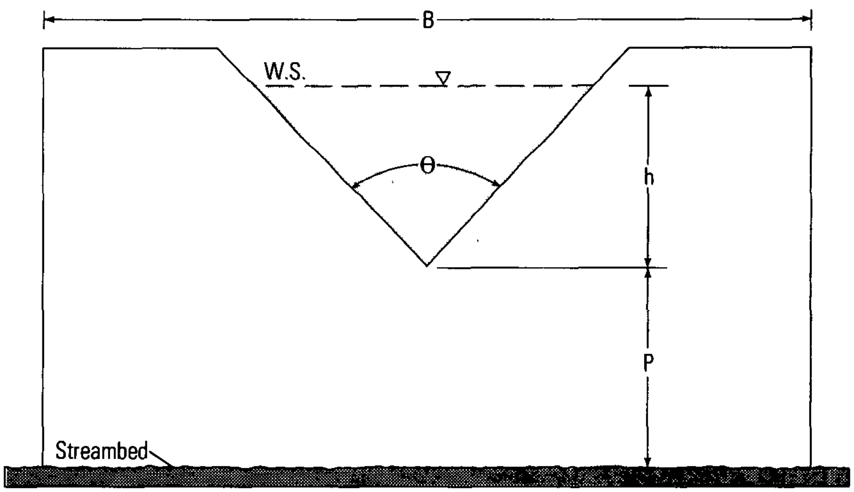


FIGURE 148.—Sketch of upstream face of a triangular or V-notch weir.

V-notches, and the discharge equation usually recommended in the U.S.A. is

$$Q=2.47h^{2.50} \text{ (English units).}$$

More precise values of the weir coefficient, which vary with h, are given for use with metric units in WMO Technical Note No. 117.

The only other central angle that is commonly used in the U.S.A. for V-notch weirs is 120°. The recommended discharge equation is

$$Q=4.35h^{2.50} \text{ (English units).}$$

SUBMERGED THIN-PLATE WEIRS

Submergence occurs at a weir when the elevation of the downstream water surface (tailwater) exceeds the elevation of the weir crest (fig. 149). The tailwater elevation is measured downstream from the turbulence that occurs in the immediate vicinity of the downstream face of the weir. The degree of submergence is expressed by the ratio h_t/h. For any given head h, submergence has the effect of reducing the discharge that would occur under the condition of free flow; the greater the submergence ratio h_t/h, the greater the reduction in discharge. Villemonte (1947) combined the results of his tests with those of several other investigators to produce the generalized relation shown in figure 150. Figure 150 is applicable to all shapes of vertical thin-plate weirs. In that figure, the abscissa is the submergence ratio raised to a power N, where N is the exponent in the free-flow discharge equation; for example, $N=1.5$ for a rectangular weir and $N=2.5$ for a triangular weir. The ordinate in figure 150 is the ratio of discharge under the submerged condition (Q_s) to free-flow discharge (Q). The relation shown in figure 150 agrees reasonably with the individual results obtained by the various investigators of submerged-weir discharge. However, if great accuracy is essential, it

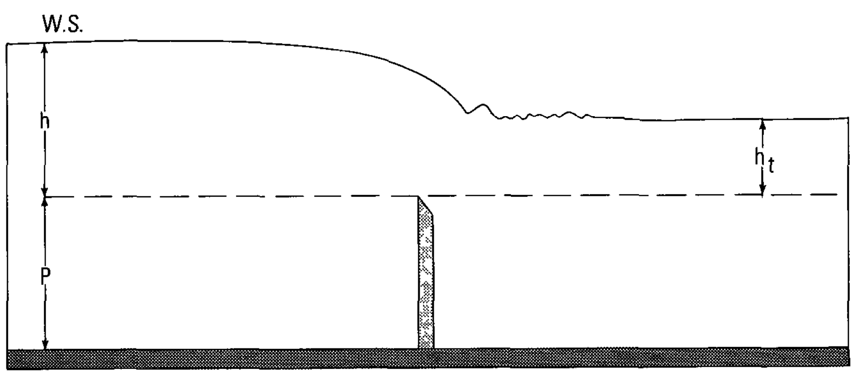


FIGURE 149.—Sketch showing submergence of a weir.

is recommended that the particular weir be calibrated in the field or in a laboratory under conditions similar to field conditions.

BROAD-CRESTED WEIRS

The term "broad-crested weir", as used here, refers to any weir that is not of the thin-plate type. The most common type of artificial control built in natural channels is the broad-crested weir. A structure of that type has the necessary strength and durability to withstand possible damage by floating debris. When installed in a stream channel that carries sediment-laden water, the weir is often built with a gently sloping upstream apron (slope: 1 vertical to 5 horizontal) so that there is no abrupt impedance to the flow and sediment is carried over the weir and not deposited in the gage pool. Because the

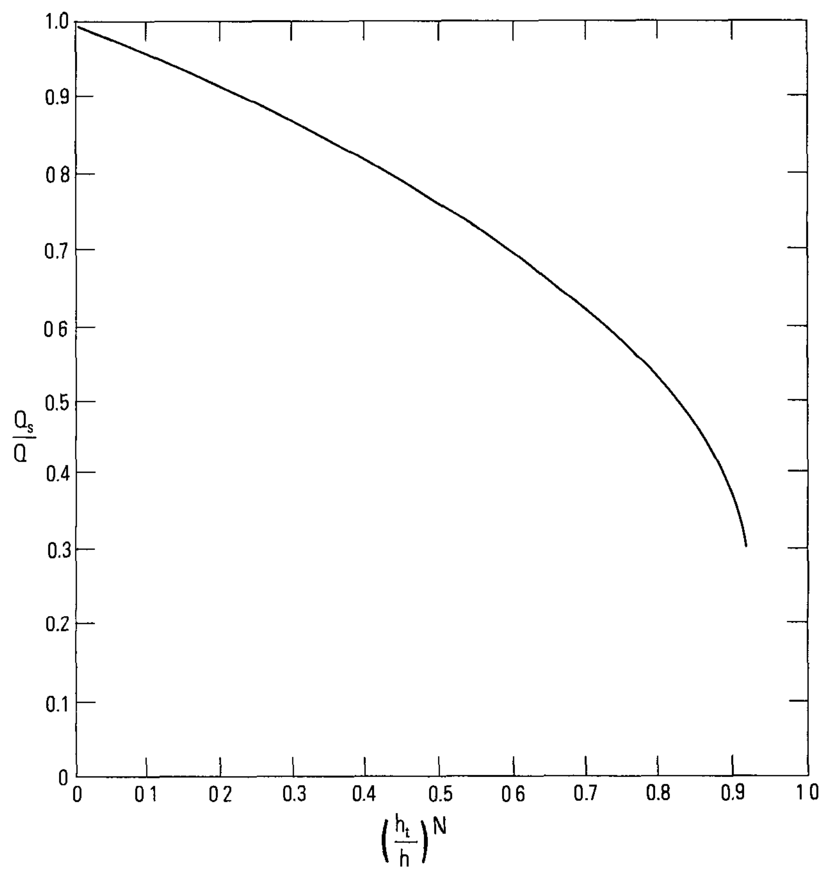


FIGURE 150.—Generalized relation of discharge ratio to submergence ratio for vertical thin-plate weirs. (After Villemonte, 1947.)

backwater caused by a high weir can aggravate flood problems along a stream, broad-crested weirs are usually built low to act as low-water controls and they become submerged at intermediate and high stages.

There are a myriad of crest shapes that can be used for broad-crested weirs, and there will be no attempt to describe the characteristics of the rating curve for each. Much of the material for such a discussion can be found in WMO Technical Note No. 117 (1971), in a report by Hulsing (1967), and in a textbook by King and Brater (1963). Instead, this section of the report will present a discussion of general principles as they apply to the definition of the discharge rating and will present the approximate ratings for broad-crested weirs commonly used as gaging-station controls in the U.S.A. The weirs are all intended to be field calibrated by current-meter discharge measurements.

Before proceeding to the discussion of broad-crested weirs commonly used in the U.S.A., it might be mentioned in passing that perhaps the most popular weir for use as a gaging-station control in Europe, and particularly in the United Kingdom, is the Crump weir (World Meteorological Organization, 1971). The Crump weir is triangular in cross section; the *upstream* face has a slope of 1 (vertical) to 2 (horizontal) and the *downstream* face has a slope of 1 (vertical) to 5 (horizontal). The crest, or apex of the triangular cross section, is usually horizontal over its entire length (b), but for greater sensitivity the crest may be given the shape of a flat Vee, the sides of which often have a slope of 1 (vertical) to 10 (horizontal). The basic equation for the Crump weir with horizontal crest is,

$$Q = Cb\,(h + h_v)^{3/2},$$

where C equals about 3.55 when English units are used and 1.96 when metric units are used.

FLAT-CRESTED RECTANGULAR WEIR

The simplest type of broad-crested weir is one that is rectangular in cross section and whose crest is horizontal over its entire length, b. The basic discharge equation for that weir is $Q=Cb(h+h_v)^{1.5}$, where h_v is the head attributable to velocity of approach. The coefficient C will increase with stage in the manner shown in figure 151, and h_v will also increase with stage as a result of the velocity of approach increasing with stage. (Figure 151 also shows the relation of C to stage for flat-crested weirs with sloping faces.) The rating curve for a flat-crested rectangular weir, when plotted on logarithmic graph paper, will be a straight line except for extremely low stages. The equation of the line will be of the form $Q=p(G-e)^N$, where the slope of the line, N, will have a value greater than 1.5 because both the weir coefficient and the velocity head increase with stage.

Most flat-crested rectangular weirs are not sufficiently sensitive at low flows. To increase the low-flow sensitivity, the crest is often modified as shown in figure 152. Instead of the crest being horizontal over its entire length, b, the crest is given a gentle slope from one streambank to the other, or the crest is given the shape of an extremely flat Vee or catenary. As a result of this modification, the area of flow over the weir is triangular, or nearly so, at low flows and approximately rectangular at high flows. In other words, the length of weir crest that is utilized by the flow varies with stage until the stage rises high enough to flow over the entire length of the crest (b in fig. 152). In the general equation for the weir discharge, $Q=Cb(h+h_v)^{1.5}$, not only do C and h_v increase with stage, but length of weir crest, b, also increases with stage, as stated in the preceding sentence. Consequently if the weir rating plots as a straight line on logarithmic graph paper in accordance with equation, $Q=p(G-e)^N$, the slope of the line, N, will be considerably greater than 1.5, and invariably will be greater than 2.0.

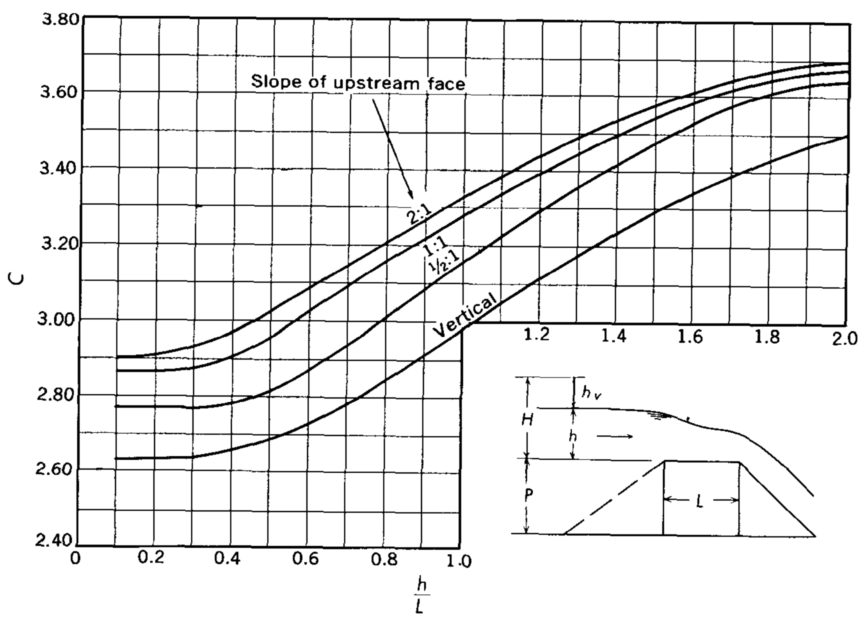


FIGURE 151.—Coefficients of discharge for full-width, broad-crested weirs with downstream slope ≤ 1:1 and various upstream slopes. (Slope is the ratio of horizontal to vertical distance.)

NOTCHED FLAT-CRESTED RECTANGULAR WEIR

Figure 153 shows the notched flat-crested rectangular weir that is the control for a gaging station on Great Trough Creek near Marklesburg, Pa.

Because there is a sharp break in the cross section at gage height 1.4 ft, a break occurs in the slope of the rating curve at that stage. The gage-height of zero flow for stages between 0.0 and 1.4 ft is 0.0 ft; for stages above 1.4 ft, the effective zero flow is at some gage height between 0.0 and 1.4 ft. If the low end of the rating is made a tangent, the gage height of zero flow (e) is 0.0 ft, and the slope of this tangent turns out to be 2.5, which, as now expected, is greater than the theoretical slope of 1.5. The upper part of this rating curve is concave upward because the value of e used (0.0 ft) is lower than the effective value of zero flow for high stages.

If the upper end of the rating is made a tangent, it is found that the value of e, or effective zero flow, must be increased to 0.6 foot. Because we have raised the value of e, the low-water end of the curve will be concave downward. The high-water tangent of the curve, principally because of increased rate of change of velocity of approach, will have a

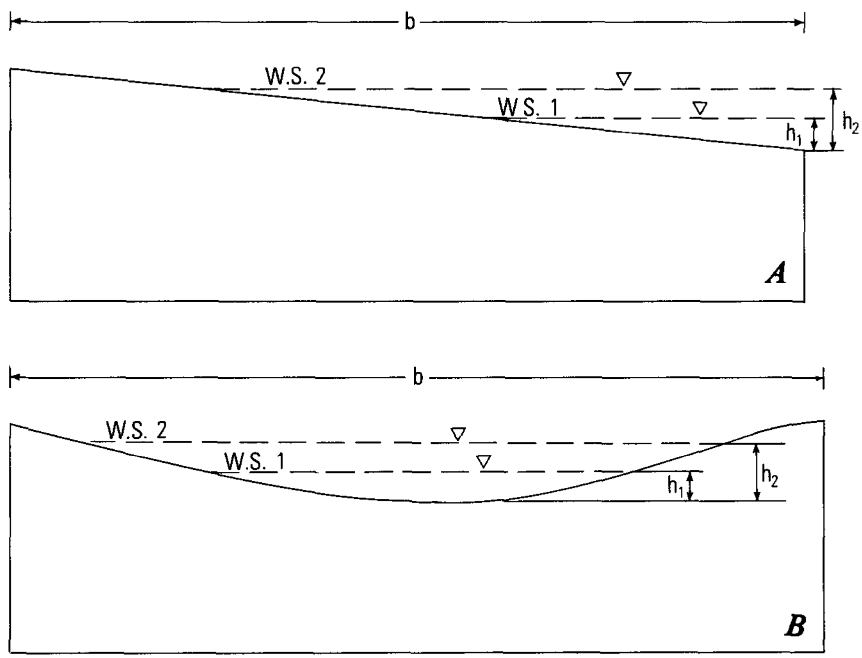


FIGURE 152.—Sketch of upstream face of flat-crested weir with (*A*) sloping crest and (*B*) catenary crest.

slope that is greater than that of the low-water tangent of the curve previously described; its slope is found to have a value of 3.0.

The low-water tangent for the notched control, which is defined by discharge measurements, warrants further discussion. Its slope of 2.5 is higher than one would normally expect for a simple flat-crested rectangular notch. One reason for the steep slope is the fact that the range of stage involved, 0.0 ft to 1.4 ft, is one in which the theoretical weir coefficient C increases very rapidly with stage. A more important reason is the geometric complexity of the notch which is not indicated in figure 153. At the downstream edge of the notch is a sharp-edged plate; its elevation is at 0.0 ft, but the sharp edge is about 0.1 ft higher than the concrete base of the notch. The details of the notch are not important to this discussion; they are mentioned here only to warn the reader not to expect a slope as great as 2.5 in the rating for a simple flat-crested rectangular notch. In fact, the sole purpose here of discussing the low-water tangent of the rating curve is to demonstrate the effect exerted on the curve by varying the applied values of e. The low-water end of a rating curve is usually well defined by discharge measurements, and if it is necessary to

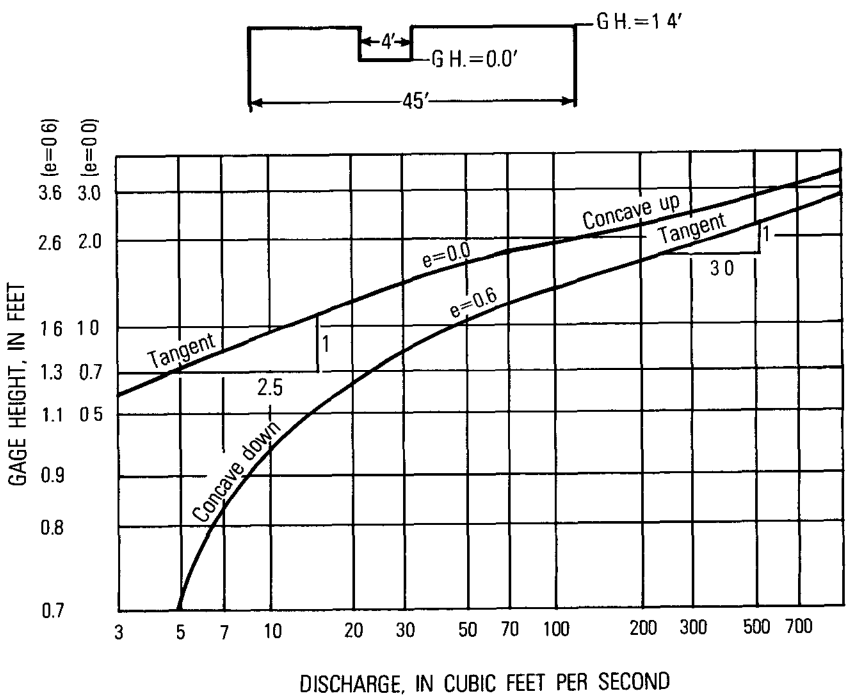


FIGURE 153.—Rating curve for a notched broad-crested control at Great Trough Creek near Marklesburg, Pa.

extrapolate the rating downward, it is best done by replotting the low-water end of the curve on rectangular-coordinate graph paper, and extrapolating the curve down to the point of zero discharge. (See section titled, "Low-Flow Extrapolation.")

TRENTON-TYPE CONTROL

The so-called Trenton-type control is a concrete weir that is frequently used in the U.S.A. The dimensions of the cross section of the crest are shown in figure 154. The crest may be constructed so as to be horizontal for its entire length across the stream, or for increased low-flow sensitivity the crest may be given the shape of an extremely flat Vee. For a horizontal crest, the equation of the stage-discharge relation, as obtained from a logarithmic plot of the discharge measurements, is commonly on the order of $Q=3.5bh^{1.65}$(English units). The precise values of the constants will vary with the height of the weir above the streambed, because that height affects the velocity of approach. The constants of the equation are greater than those for a flat-crested rectangular weir (see section titled, "Flat-crested Rectangular Weir") because the cross-sectional shape of the Trenton-type control is more efficient than a rectangle with regard to the flow of water.

When the Trenton-type control is built with its crest in the shape of a flat Vee, the exponent of h in the discharge equation is usually 2.5 or more, as expected for a triangular notch where velocity of approach

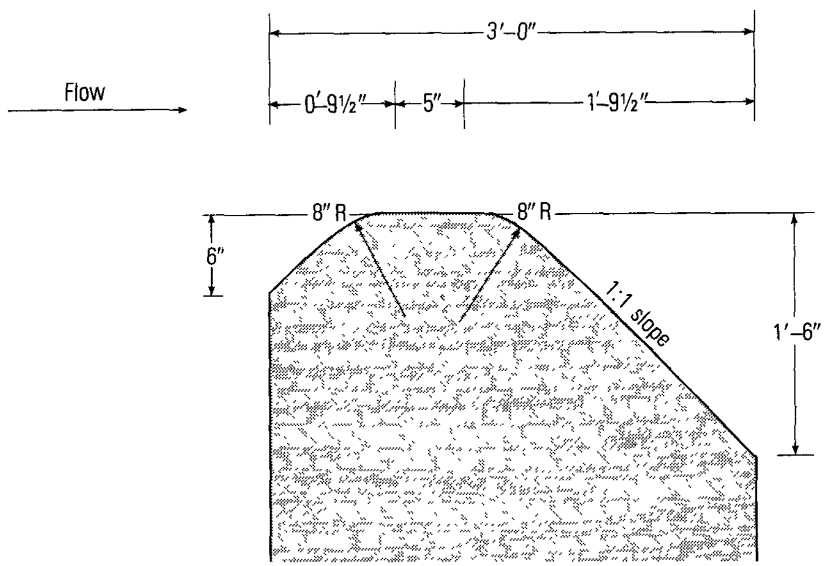


FIGURE 154.—Cross section of Trenton-type control.

is significant. Again, the precise values of the constants in the discharge equation are dependent on the geometry of the installation.

COLUMBUS-TYPE CONTROL

One of the most widely used controls in the U.S.A. is the Columbus-type control. This control is a concrete weir with a parabolic notch that is designed to give accurate measurement of a wide range of flows (fig. 155). The notch accommodates low flows; the main section, whose crest has a flat upward slope away from the notch, accommodates higher flows. The throat of the notch is convex along the axis of flow to permit the passage of debris. For stages above a head of 0.7 ft, which is the elevation of the top of the notch, the elevation of effective zero flow is 0.2 ft, and the equation of discharge is approximately,

$$Q=8.5(h-0.2)^{3.3} \text{ (English units)}$$

The precise values of the constants in the equation will vary with conditions for each installation. The shape of the crest above a stage of 0.7 ft is essentially a flat Vee for which the theoretical exponent of head is 2.5 in the discharge equation. However, the actual value of the exponent is greater than 2.5 principally because of the increase of velocity of approach with stage.

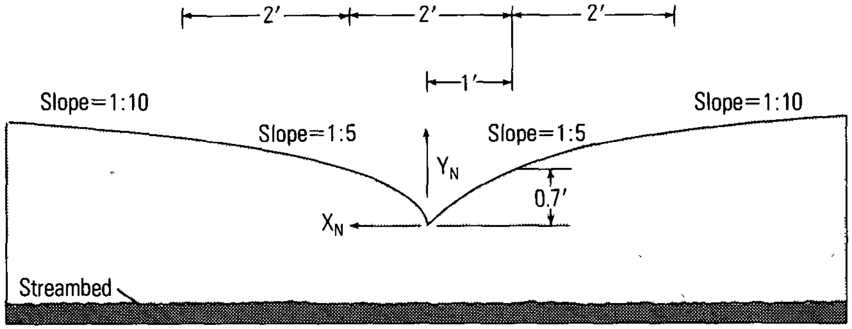
SUBMERGED BROAD-CRESTED WEIRS

Weir submergence was defined earlier in the section titled, "Submerged Thin-Plate Weirs." As in the case of thin-plate weirs, for a given static head (h) the discharge decreases as the submergence ratio (h_t/h) increases. Little quantitative data are available to define the relation of discharge ratio to submergence ratio for the many types of broad-crested weir. However, it is known that for horizontal crests the submergence ratio must be appreciable before any significant reduction in discharge occurs. This threshold value of the submergence ratio at which the discharge is first affected ranges from about 0.65 to 0.85, depending on the cross-sectional shape of the weir crest.

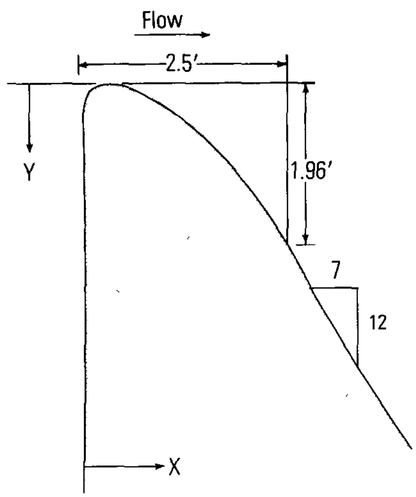
FLUMES

Flumes commonly utilize a contraction in channel width and free fall or a steepening of bed slope to produce critical or supercritical flow in the throat of the flume. The relation between stage measured at some standard cross section and discharge is thus a function only of the characteristics of the flume and can be determined, on an interim basis at least, prior to installation.

In the section in chapter 3 titled, "Artificial Controls," it was mentioned that flumes may be categorized with respect to the flow regime that principally controls the measured stage; that is, a flume may be classed as either a critical-flow flume or a supercritical-flow flume. The most commonly used critical-flow flume is the Parshall flume, and it is the only one of that type that will be described here. The



Profile of weir crest and notch



Cross section of weir

Coordinates of notch profile, in feet

X_N	Y_N
0.00	0.00
.046	.1
.108	.2
.192	.3
.302	.4
.452	.5
.665	6
1.00	7

Coordinates of cross section of weir crest, in feet

X	Y
0.0	0.126
.1	.036
2	.007
.3	.000
.4	.007
.6	.060
8	.142
1.0	.257
1.2	.397
1.4	565
1.7	.870
2.0	1.22
2 5	1.96

FIGURE 155.—Dimensions of Columbus-type control.

supercritical-flow flume is less widely used, but fills a definite need. (See section in chapter 3 titled "Choice of an Artificial Control.") Of that type of flume, the trapezoidal supercritical-flow is preferred by the Geological Survey; it too will be described here.

PARSHALL FLUME

The principal feature of the Parshall flume is an approach reach having converging sidewalls and a level floor, the downstream end of which is a critical-depth cross section. Critical flow is established in the vicinity of that cross section by having a sharp downward break in the bed slope of the flume. In other words, the bed slope downstream from the level approach section is supercritical. The primary stage measurement is made in the approach reach at some standard distance upstream from the critical-depth cross section.

The general design of the Parshall flume is shown in figure 156. Table 17 gives the dimensions corresponding to the letters in figure 156 for various sizes of flumes. The flumes are designated by the width (W) of the throat. Flumes having throat widths from 3 in. to 8 ft have a rounded entrance whose floor slope is 25 percent. The smaller

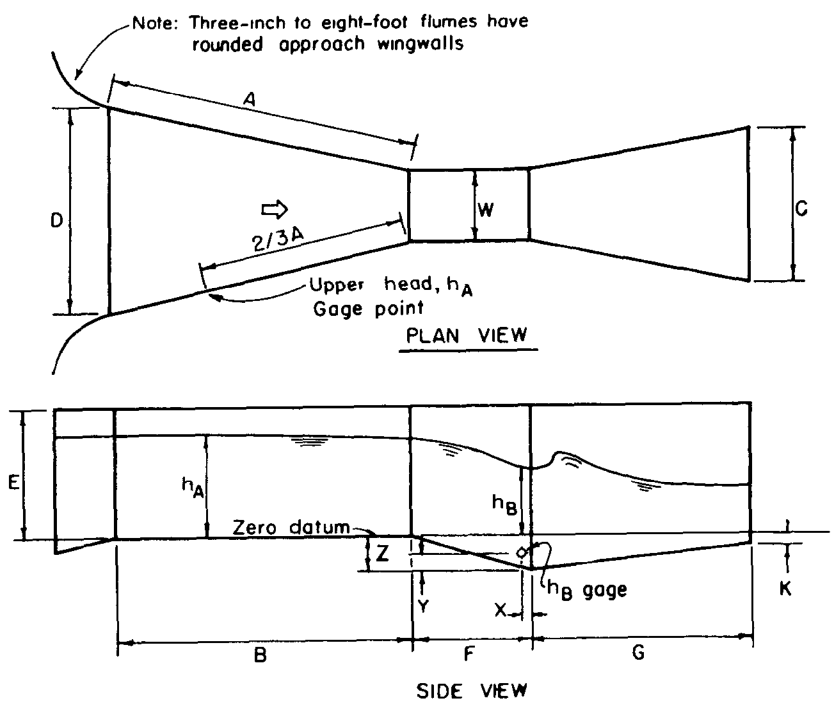


FIGURE 156.—Configuration and descriptive nomenclature for Parshall flumes.

TABLE 17.—*Dimensions and capacities of all sizes of standard Parshall flumes*

[For sizes 1 ft to 8 ft, $A=W/2+4$. For all sizes, h_A is located a distance of 2/3 A from crest, distance is converging wall length, not axial]

Size, Throat width W	Widths: Upstream end D	Widths: Downstream end C	Axial Lengths: Converging Section B	Axial Lengths: Throat Section F	Axial Lengths: Diverging Section G	Wall Depth in Converging Section E	Vertical distance below crest: Dip at Throat Z	Vertical distance below crest: Lower end of flume K	Converging wall length A	h_A, dist. upstream from crest	Gage Points h_B: X	Gage Points h_B: Y	Free Flow Capacities: Min.	Free Flow Capacities: Max.
inches	feet	feet	feet	feet	feet	feet	feet	feet	feet	feet	feet	feet	ft ^{3}/s	ft ^{3}/5s
1	0.549	0.305	1.17	0.250	0.67	0.5-0.75	0 094	0.062	1.19	0.79	0.026	0.042	0.005	0.15
2	.700	.443	1.33	.375	.83	0.50-0.83	.141	.073	1.36	.91	.052	.083	.01	.30
3	.849	.583	1.50	.500	1.00	1.00-2.00	.188	.083	1.53	1.02	.083	.125	.03	1.90
6	1.30	1.29	2.00	1.00	2.00	2.0	.375	.25	2.36	1.36	.167	.25	.05	3.90
9	1.88	1.25	2.83	1.00	1.50	2.5	.375	.25	2.88	1.93	.167	.25	.09	8.90
feet														
1.0	2.77	2.00	4.41	2.0	3.0	3.0	.75	.25	4.50	3.00	.167	.25	.11	16.1
1.5	3.36	2.50	4.66	2.0	3.0	3.0	.75	.25	4.75	3.17	.167	.25	.15	24.6
2.0	3.96	3.00	4.91	2.0	3.0	3.0	.75	.25	5.00	3.33	.167	.25	.42	33.1
3.0	5.16	4.00	5.40	2.0	3.0	3.0	.75	.25	5.50	3.67	.167	.25	.61	50.4
4.0	6.35	5.00	5.88	2.0	3.0	3.0	.75	.25	6.00	4.00	.167	.25	1.30	67.9
5.0	7.55	6.00	6.38	2.0	3.0	3.0	.75	.25	6.50	4.33	.167	.25	1.60	85.6
6.0	8.75	7.00	6.86	2.0	3.0	3.0	.75	.25	7.0	4.67	.167	.25	2.60	103.5
7.0	9.95	8.00	7.35	2.0	3.0	3.0	.75	.25	7.5	5.00	.167	.25	3.00	121.4
8.0	11.15	9.00	7.84	2.0	3.0	3.0	.75	.25	8.0	5.33	.167	.25	3.50	139.5
10	15.60	12.00	14.0	3.0	6.0	4.0	1.12	.50	9.0	6.00			6	300
12	18.40	14.67	16.0	3.0	8.0	5.0	1.12	.50	10.0	6.67			8	520
15	25.0	18.33	25.0	4.0	10.0	6.0	1.50	.75	11.5	7.67			8	900
20	30.0	24.00	25.0	6.0	12.0	7.0	2.25	1.00	14.0	9.33			10	1340
25	35.0	29.33	25.0	6.0	13.0	7.0	2.25	1.00	16.5	11.00			15	1660
30	40.4	34.67	26.0	6.0	14.0	7.0	2.25	1.00	19.0	12.67			15	1990
40	50.8	45.33	27.0	6.0	16.0	7.0	2.25	1.00	24.0	16.00			20.	2640
50	60.8	56.67	27.0	6.0	20.0	7.0	2.25	1.00	29.0	19.33			25	3280

Note: Flume sizes 3 inches through 8 feet have approach aprons rising at a 1:4 slope and the following entrance roundings: 3 through 9 inches, radius =1.33 feet; 1 through 3 feet, radius=1 67 feet, 4 through 8 feet, radius=2 00 feet and larger flumes do not have that feature, but it is doubtful whether the performance of any of the flumes is significantly affected by the presence or absence of the entrance feature as long as approach conditions are satisfactory.

Parshall flumes have provision for stage measurements both in the approach reach and in the throat reach, but the downstream gage is required only when submerged-flow conditions exist. The datum for both gages is the level floor in the approach. The raised floor, length G in figure 156, in the downstream diverging reach is designed to reduce scour downstream and to produce more consistent stage-discharge relations under conditions of submergence. The percentage of submergence for Parshall flumes is computed by the formula,

$$\frac{h_B}{h_A} \times 100.$$

Where free-flow conditions exist for all flows, the downstream gage, h_B, may be omitted and the entire diverging reach may be dispensed with if desired. That simplification has been used in the design of small portable Parshall measuring flumes. (See section in chapter 8 titled, "Portable Parshall Flume.")

Tables 18 and 19 summarize the relation of discharge to stage at h_A under conditions of free flow (low stage at h_B) for flumes of the various sizes. Although the free-flow stage-discharge relations for the various flumes were derived experimentally, all relations can be expressed closely by the following equation, (Davis, 1963),

$$Y_0 + \frac{Q_0^2}{2Y_0^2\,(1 + 0.4X_0)^2} = 1.351\,Q_0^{\,0.645} \text{ (English units)} \tag{59}$$

in which

Y_0 = nondimensional depth, y_1/b
Q_0 = nondimensional discharge, $Q/g^{1/2}b^{5/2}$
X_0 = nondimensional distance, x/b
y_1 = depth at measuring section
b = channel width at throat
Q = discharge
g = acceleration of gravity
x = distance from throat crest to measuring section.

For flumes with throat widths no greater than 6 ft, the following simplified form of the above equation (Dodge, 1963) can be used:

$$Y_0 = 1.19Q_0^{\,0.645}\,X_0^{\,0.0494} \tag{60}$$

When the stage h_B is relatively high, the free-flow discharge corresponding to any given value of h_A is reduced. The percentage of submergence, or value of $[(h_B/h_A)\times 100]$, at which the free-flow discharge is first affected, varies with the size of flume. For flumes whose throat width is less than 1 ft, the submergence must exceed 50 percent before there is any backwater effect; for flumes with throat width from 1 to 8 ft, the threshold submergence is 70 percent; for flumes with throat width greater than 10 ft, the threshold submergence is 80 percent. Figure 157 shows the discharge ratings for Parshall flumes, from 2 inches to 9 inches, under both free-flow and submergence conditions. Figure 158 shows the correction in discharge, which is always negative, that is to be applied to free-flow discharges for various percentages of submergence and various values of h_A, for flumes having throat widths between 1 and 50 feet. The appropriate correction factor (k_s) for flume size is applied to the corrections read from the graphs. In other words,

$$Q_s = Q_f - k_s Q_c,$$

where

Q_s = discharge under submergence conditions,
Q_f = discharge under free-flow conditions, and
Q_c = discharge correction unadjusted for flume size.

The stage-discharge relations, both for free-flow and submergence conditions, given in the preceding tables and graphs, should be used only as guides or as preliminary ratings for Parshall flumes built in the field. Those installations should be field-calibrated because the structural differences that invariably occur between model and prototype flume usually cause the discharge rating for the field structure to differ from the experimental ratings given in this manual.

TABLE 18.—*Discharge table for Parshall flumes, sizes 2 inches to 9 inches, for free-flow conditions*

[Discharges for standard 3-inch Parshall flumes are slightly less than those for the modified 3-inch Parshall flume discussed in chapter 8; see table 14]

h_A (feet)	2 inches (ft^3/s)	3 inches (ft^3/s)	6 inches (ft^3/s)	9 inches (ft^3/s)
0.1	0.02	0.03	0.05	0.09
2	06	08	16	26
3	11	15	31	49
4	17	24	48	.76
5	24	34	69	1.06
6	31	45	92	1.40
7	40	57	1.17	1.78
8		70	1.45	2.18
9		84	1.74	2.61
1.0		89	2.06	3.07
1.1			2.40	3.55
1.2			2.75	4.06
1.3			3.12	4.59
1.4			3.51	5.14
1.5				5.71
1.6				6.31
1.7				6.92
1.8				7.54
1.9				8.20

TABLE 19.—*Discharge table for Parshall flumes, sizes 1 foot to 50 feet for free-flow conditions*

h_A (feet)	1 foot (ft^3/s)	1.5 feet (ft^3/s)	2 feet (ft^3/s)	3 feet (ft^3/s)	4 feet (ft^3/s)	5 feet (ft^3/s)	6 feet (ft^3/s)	7 feet (ft^3/s)	8 feet (ft^3/s)
0.10	0.11	0.15							
15	20	30	0.42	0.61					
20	35	51	66	97	1.26	1.55			
25	49	.71	93	1.37	1.80	2.22	2.63	3.02	3.46
30	.64	94	1.24	1.82	2.39	2.96	3.52	4.08	4.62
4	99	1.47	1.93	2.86	3.77	4.68	5.57	6.46	7.34
5	1.39	2.06	2.73	4.05	5.36	6.66	7.94	9.23	10.5
6	1.84	2.73	3.62	5.39	7.15	8.89	10.6	12.4	14.1
7	2.33	3.46	4.60	6.86	9.11	11.4	13.6	15.8	18.0
.8	2.85	4.26	5.66	8.46	11.3	14.0	16.8	19.6	22.4
9	3.41	5.10	6.80	10.2	13.6	16.9	20.3	23.7	27.0
1.0	4.00	6.00	8.00	12.0	16.0	20.0	24.0	28.0	32.0
1.2	5.28	7.94	10.6	16.0	21.3	26.7	32.1	37.5	42.9
1.4	6.68	10.1	13.5	20.3	27.2	34.1	41.1	48.0	55.0
1.6	8.18	12.4	16.6	25.1	33.6	42.2	50.8	59.4	68.1
1.8	9.79	14.8	19.9	30.1	40.5	50.8	61.3	71.8	82.3
2.0	11.5	17.4	23.4	35.5	47.8	60.1	72.5	84.9	97.5
2.2	13.3	20.2	27.2	41.3	55.5	69.9	84.4	98.9	113.6
2.4	15.2	23.0	31.1	47.3	63.7	80.3	97.0	113.7	130.7

h_A (feet)	10 feet (ft^3/s)	12 feet (ft^3/s)	15 feet (ft^3/s)	20 feet (ft^3/s)	25 feet (ft^3/s)	30 feet (ft^3/s)	40 feet (ft^3/s)	50 feet (ft^3/s)
0.30	5.75	6.75	8.4	11.1	13.8	16.5	21.8	27.3
0.4	9.05	10.85	13.3	17.7	21.8	26.1	34.6	43.2
0.5	13.0	15.4	19.1	25.1	31.2	37.2	49.5	61.8
0.6	17.4	20.6	25.5	33.7	41.8	50.0	66.2	82.6
0.7	22.2	26.2	32.7	43.1	53.4	64.0	84.8	105.5
0.8	27.5	32.7	40.4	53.4	66.3	79.2	105	131
0.9	33.3	39.4	48.9	64.3	80.1	95.5	127	158
1.0	39.4	46.8	57.9	76.3	94.8	113.2	150	187
1.2	52.7	62.6	77.3	102.0	127.0	152	201	250
1.4	67.4	80.1	99.0	130.5	162	194	257	320
1.6	83.5	99.1	122.8	162	201	240	318	396
1.8	100.9	119.8	148.0	195	243	290	384	479
2.0	119.4	141.8	175.3	232	287	343	454	567
2.2	139.0	165.0	204	269	334	400	530	660
2.4	159.9	189.8	235	310	384	459	609	758
2.6	181.7	215.7	267	352	437	522	692	864
3.0	228.4	271.2	335	442	549	656	870	1084
3.5	294	347	429	566	703	840	1113	1387
4.0	363	430	531	700	870	1040	1379	1717
4.5	437	518	641	846	1051	1255	1664	2073
5.0	517	614	759	1002	1244	1486	1970	2453
5.5			885	1166	1448	1730	2295	2860
6.0			1016	1340	1664	1988	2638	3285

Note: Available data indicates that extension of the above ratings to greater heads is reliable

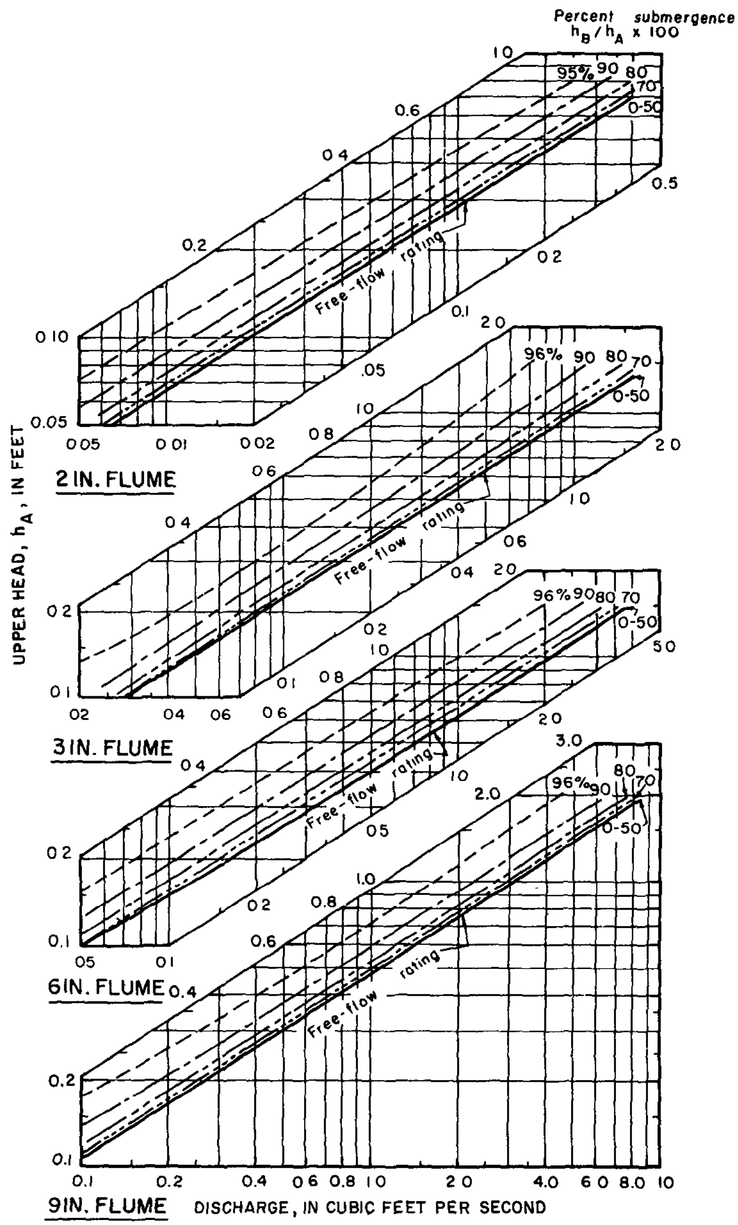


FIGURE 157.—Discharge ratings for "inch" Parshall flumes for both free-flow and submergence conditions.

TRAPEZOIDAL SUPERCRITICAL-FLOW FLUME

The principal feature of the trapezoidal supercritical-flow flume is a reach of flume (throat) whose bed has supercritical slope, upstream from which is a critical-depth cross section. The general design of the flume and the dimensions for the flumes of three throat widths that have been installed by the Geological Survey are shown in figure 159. The purpose of having the flume trapezoidal in cross section is to increase the sensitivity of the stage-discharge relation, particularly at low flows. Wide latitude exists with regard to the height (E) of the

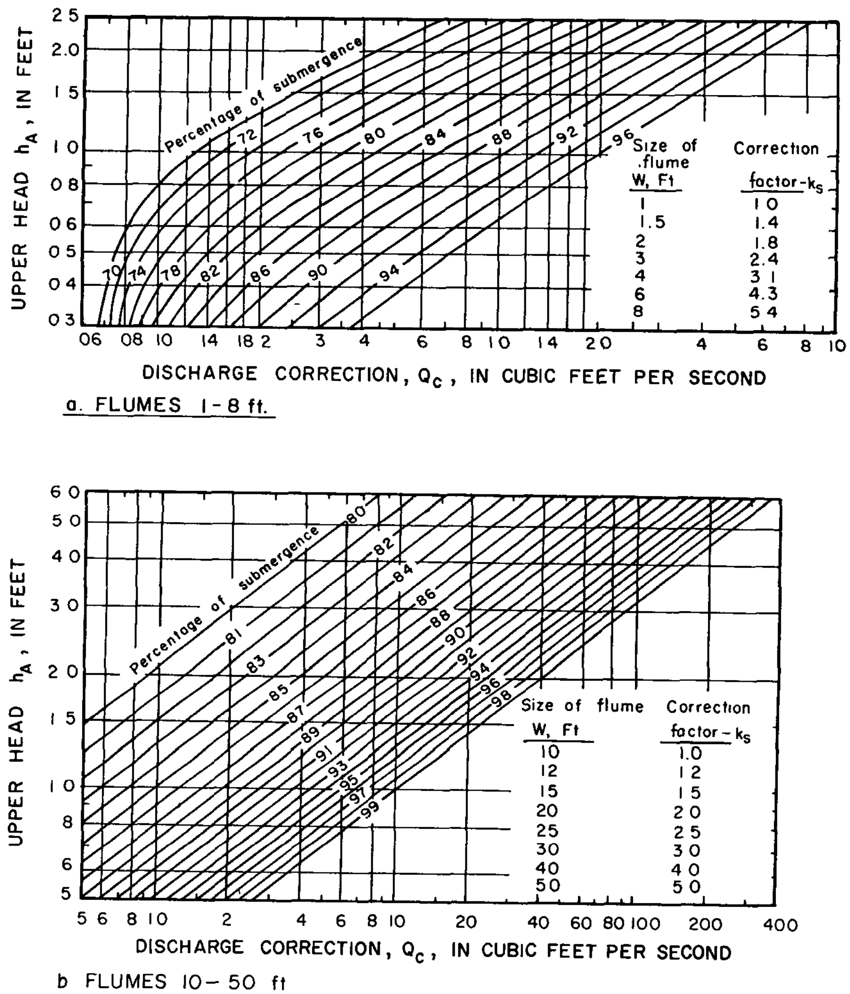


FIGURE 158.—Correction factors for submerged flow through 1- to 50-ft Parshall flumes.

sidewalls that can be used, and thus the range of discharge that can be accommodated by a supercritical-flow flume of any particular throat width is quite flexible. Stage (vertical depth of flow) is measured at a cross section at midlength of the throat reach, gage datum being the floor of the flume at the stage-measurement site. The measurement of stage must be precise because the stage-discharge relation for supercritical flow is extremely insensitive—a small change in stage corresponds to a large change in discharge.

Were it not for the severe width constriction at the downstream end of the converging reach, critical flow would occur at the break in floor slope at the downstream end of the approach reach and flow would be supercritical at all cross sections downstream from the approach reach. However for all but extremely low flows, the sharp constriction in width resulting from the use of a convergence angle (ϕ) of 21.8° (fig.159) causes backwater that extends upstream into the approach

Dimensions of trapezoidal supercritical-flow flume

Flume size, W_T feet	Width of approach reach, W_A feet	Angles		Lengths			Minimum capacity, ft³/s	Floor slopes	
		Sloping walls θ	Converging walls ϕ	Approach reach, L_A feet	Converging reach, L_C feet	Throat reach, L_T feet		Approach reach, percent	Converging and throat reaches, %
1	5	30°	21.8°	5 0	5 0	5 0	0.7	0	5
3	9	30°	21.8°	variable	7.5	10.0	2.0	0	5
8	14	30°	21.8°	variable	7.5	12 0	6.0	0	5

Note—Height of wall (E) is depenaent on magnitude of maximum discharge to be gaged

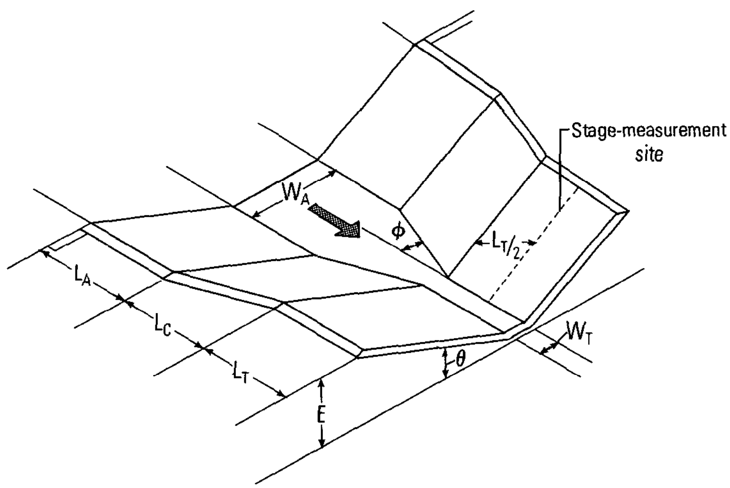


FIGURE 159.—Configuration and dimensions of trapezoidal supercritical-flow flumes of three throat widths.

reach. As a result critical depth occurs at the most constricted cross section in the converging reach, the flow being subcritical in the approach and converging reaches and supercritical in the throat reach. That is seen in figure 5 (chap. 3) which is a photograph of a 3-foot trapezoidal flume in Owl Creek in Wyoming. The purpose of the converging reach is to obtain an increased velocity at the critical-depth cross section and thereby reduce the likelihood of debris deposition at that cross section; such deposition could affect the stage-discharge relation in the throat of the flume.

The measured stage corresponding to any discharge is a function of the stage of critical depth at the head of the throat reach and the geometry of the throat reach upstream from the stage-measurement cross section. Consequently a theoretical rating for all but the smallest discharges can be computed by use of the Bernoulli or total-energy equation for the length of throat reach upstream from the stage-measurement site (fig. 160). By equating total energy at the critical-depth cross section (c) at the head of the throat reach to total energy at the stage-measurement cross section (m), we have,

$$\frac{V_c^{\,2}}{2g} + h_c + z_c = \frac{V_m^{\,2}}{2g} + h_m + z_m + h_f, \qquad (61)$$

where

V is mean velocity,
g is acceleration of gravity,
h is vertical depth,
z is elevation of flume floor above any arbitrary datum plane, and
h_f is friction loss.

We make the assumption that the friction loss h_f in the short reach is negligible and may be ignored. Then by substituting, in equation 61 values from the two equations

$$Q = A_c V_c = A_m V_m \quad \text{and} \quad \Delta Z = Z_c - Z_m,$$

we obtain

$$\frac{Q^2}{2gA^2_{\;c}} + h_c + \Delta z = \frac{Q^2}{2gA^2_{\;m}} + h_m. \qquad (62)$$

From the properties of critical-depth flow (Chow, 1959, p. 64), the critical-section factor (J) is computed by the formula

$$J = A_c \sqrt{\frac{A_c}{T_c}} \qquad (63)$$

where A_c is the area and T_c is the top width at the critical-depth cross section. The discharge (Q) at the critical-depth cross section is

$$Q = J\sqrt{g} \tag{64}$$

By assuming a depth (h_c) at the critical-depth cross section, we can compute Q and A_c, and thus the values of all terms on the left side of equation 62 will be known for any chosen value of h_c. Because h_m is uniquely related to A_m, equation 62 can be solved by trial and error to obtain the depth (stage) at the measurement cross section corresponding to the value of Q that was computed earlier.

The entire procedure is repeated for other selected values of h_c to provide a discharge rating curve for the entire range of discharge. The value of h_c corresponding to the maximum discharge to be gaged represents the height to which the sidewalls of the throat section must be built to contain that discharge. An additional height of at least 0.5 ft should be added for freeboard to accommodate surge and wave action.

The computed discharge rating should be used only until the rating can be checked by current-meter discharge measurements. The sources of error in the computed rating are uncertainty as to the exact location of the critical-depth cross section for any given discharge and neglect of the small friction loss (h_f). However, the general shape of the discharge rating curve will have been defined by the computed values, and relatively few discharge measurements should be required for any needed modification of the rating.

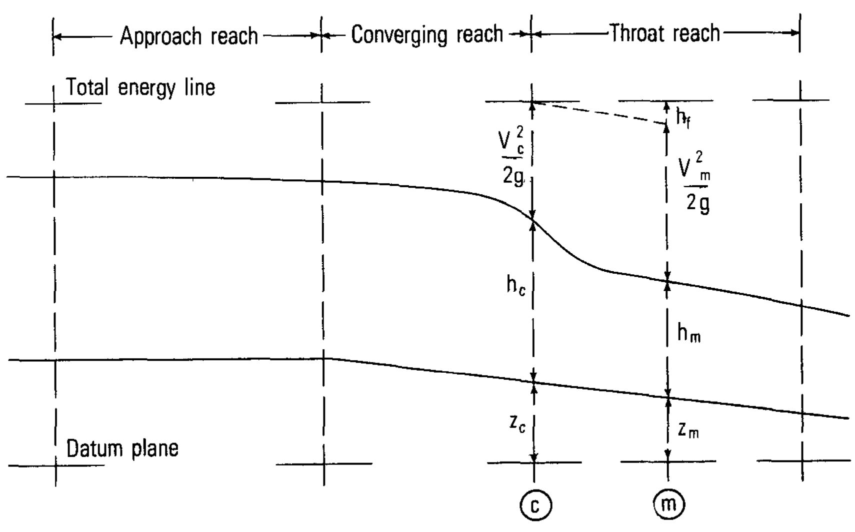


FIGURE 160.—Sketch illustrating use of the total-energy (Bernoulli) equation.

The total-energy equation (eq. 61) should also be applied to the converging reach to obtain the required height of the sidewalls at the upper end of the converging reach. The height plus an additional freeboard height of at least 0.5 ft should be used for the sidewalls in the approach reach. In applying equation 61 to the converging reach, the value of discharge used is the maximum discharge that is to be gaged, and the depth used at the lower end of the converging reach is the corresponding critical depth (h_c) that was computed earlier for the throat reach.

The solid-line curves on figures 161–163 are the theoretical discharge rating curves for the flumes of the three throat widths that have been field tested. The agreement between measured and theoretical discharges has generally been good except at extremely low stages. Nevertheless the theoretical curves should be considered as interim rating curves for newly built flumes until later measurements either corroborate the ratings or show the need for modification of the ratings. It is expected that the stage-discharge relation will not be affected by submergence, as long as submergence percentages do not exceed 80 percent. (Percentage of submergence for a given discharge is defined as the ratio, expressed as a percentage, of the stage in the natural channel immediately downstream from the throat reach to the stage at the stage-measurement site, both stages being referred to the floor elevation of the flume at the stage-measurement site.)

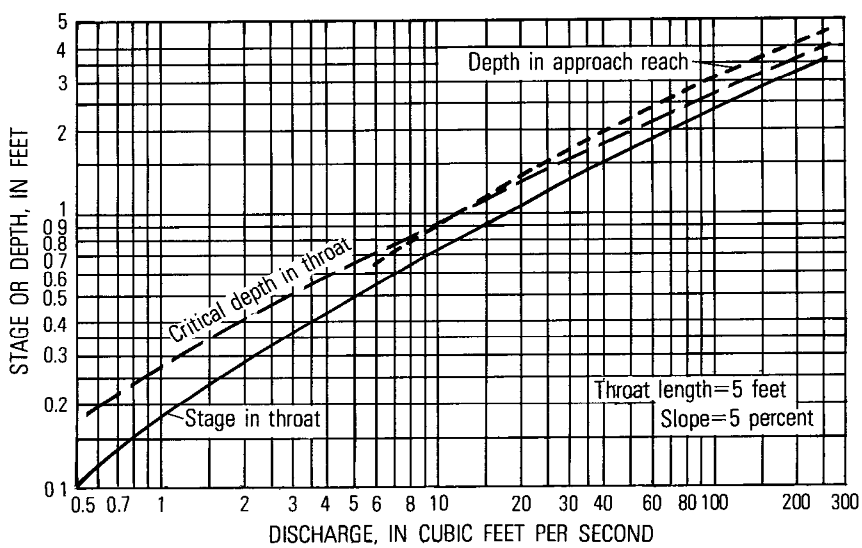


FIGURE 161.—Stage-discharge relation and significant depth-discharge relations for 1-ft trapezoidal supercritical-flow flume.

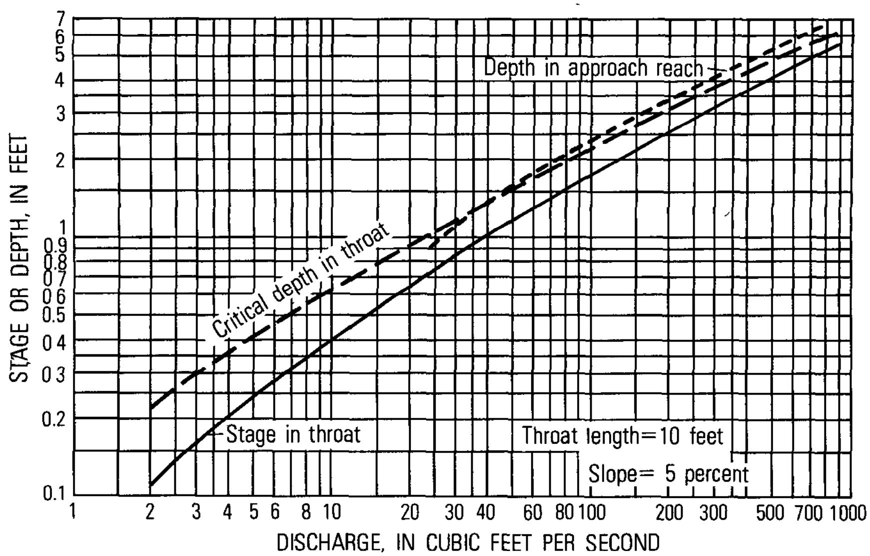


FIGURE 162.—Stage-discharge relation and significant depth-discharge relations for 3-ft trapezoidal supercritical-flow flume.

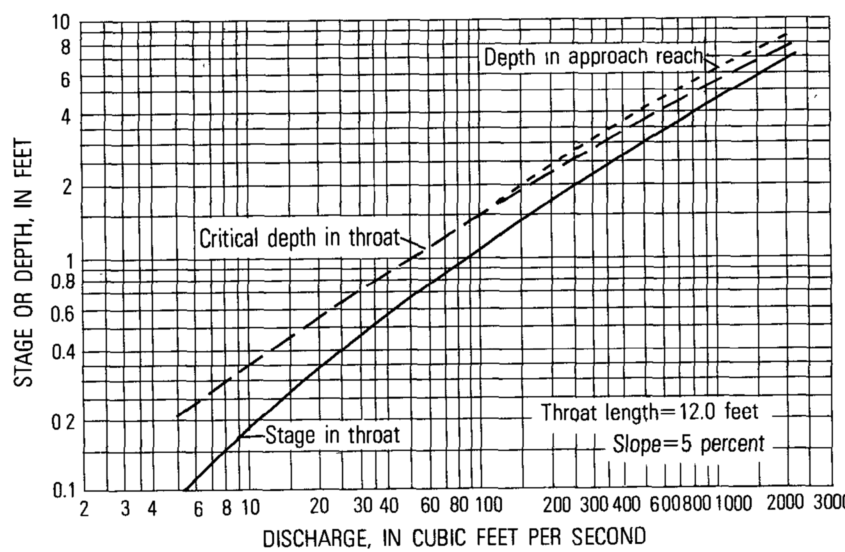


FIGURE 163.—Stage-discharge relation and significant depth-discharge relations for 8-ft trapezoidal supercritical-flow flume.

Also shown on each of the three rating-curve graphs are curves labeled "critical depth in throat" and "depth in approach reach." These curves are used to obtain the heights of sidewalls required to contain the maximum discharge for which the flume is designed. For example, let us assume that an 8-ft flume is to be built to gage a range of discharges whose maximum value is 1200 ft^3/s. Figure 163 shows that the theoretical stage for that discharge is 5.2 ft; the height of the throat sidewall (critical depth) is shown to be 6.0 ft and the height of the approach sidewall is 6.8 ft. To those sidewall heights will be added at least 0.5 ft of freeboard, and the top of the sidewalls in the converging reach will be sloped uniformly to join the tops of the sidewalls of the approach and throat reaches.

Up to this point little has been said concerning the approach reach. Its sidewalls are extended upstream from the converging reach by means of rock fill or concrete to meet the natural channel banks. As long as the approach reach provides a smooth transition from the natural channel to the converging reach, its actual geometry will have no effect on the theoretical rating. The level floor of the approach reach will provide a site for current-meter measurements of discharge and will also induce the deposition of large debris, thereby helping to keep the more vital parts of the flume structure free of sediment deposition.

NATURAL SECTION CONTROLS

Natural section controls, listed in order of permanence, are usually a rock ledge outcrop across the channel, or a riffle composed of loose rock, cobbles, and gravel, or a gravel bar. Less commonly, the section control is a natural constriction in width of the channel, or is a sharp break in channel slope, as at the head of a cascade or brink of a falls.

Where the control is a rock outcrop, riffle, or gravel bar, the stage-discharge relation, when plotted on logarithmic paper, conforms to the general principles discussed for broad-crested artificial controls. If the natural control is essentially horizontal for the entire width of the control, the head on the control is the difference between the gage heights of the water surface and the crest of the control. The exponent (N) of the head in the equation of discharge,

$$Q = p(G - e)^N \tag{53}$$

will be greater than the theoretical value 1.5, primarily because of the increase in velocity of approach with stage. If the crest of the control has a roughly parabolic profile, as most natural controls have (greater depths on the control near midstream), the exponent N will be even larger because of the increase in width of the stream with stage, as well as the increase in velocity of approach with stage. The value of N will almost always exceed 2.0. If the control is irregularly notched, as is often the case, the gage height of effective zero flow (*e*) for all but the lowest stages, will be somewhat greater than that for the lowest point in the notch. (The method of determining values of *e* was explained in the section titled, "Graphical Plotting of Rating Curves.")

The above principles are also roughly applicable to the discharge equations for an abrupt width contraction or an abrupt steepening of bed slope. The exponent *N* and the gage height of effective zero flow are influenced, as described above, by the transverse profile of the streambed at the control cross section.

An example of natural section control is treated in the following discussion.

COMPOUND SECTION CONTROLS

Where the control section is a local rise in the streambed, as at a rock outcrop, riffle, or gravel bar, that cross section is invariably a control only for low flows. The gaging station in that circumstance has a compound control, the high flows being subject to channel control. Occasionally there is a second outcrop or riffle, downstream from the low-water riffle, that acts as a section control for flows of intermediate magnitude. When the control for intermediate stages is effective it causes submergence of the low-water control. At high flows the section control for intermediate stages is in turn submerged when channel control becomes effective. An example of a compound control involving two section controls follows; an example of a compound control involving a section control that is submerged when channel control becomes effective is described in the section titled, "Compound Controls Involving Channel Control."

Figure 164 shows the rating for the compound section control at the gaging station on Muncy Creek near Sonestown, Pa. The control consists of two rock-ledge riffles, effective zero flow (e) for very low stages being at gage height 1.3 feet and for higher stages at gage height 1.2 feet. If the low end of the rating is made a tangent, it means that too large a value of *e* is used for the high end of the rating (1.3 ft vs 1.2 ft), and the high-water end of the curve becomes concave downward. Conversely, if the high end of the curve is made a tangent, the low-water end of the curve becomes concave upward. The high-water tangent of the curve has a greater value of exponent *N* than the low-water tangent of the other curve. This difference in the values of *N* reflects the effect of differences in the geometries of the two controls as well as the effect of increased rate of change of approach velocities at the higher stages. The slopes of the two tangents are 2.9 and 2.2, both values being greater than the theoretical slope of 1.5.

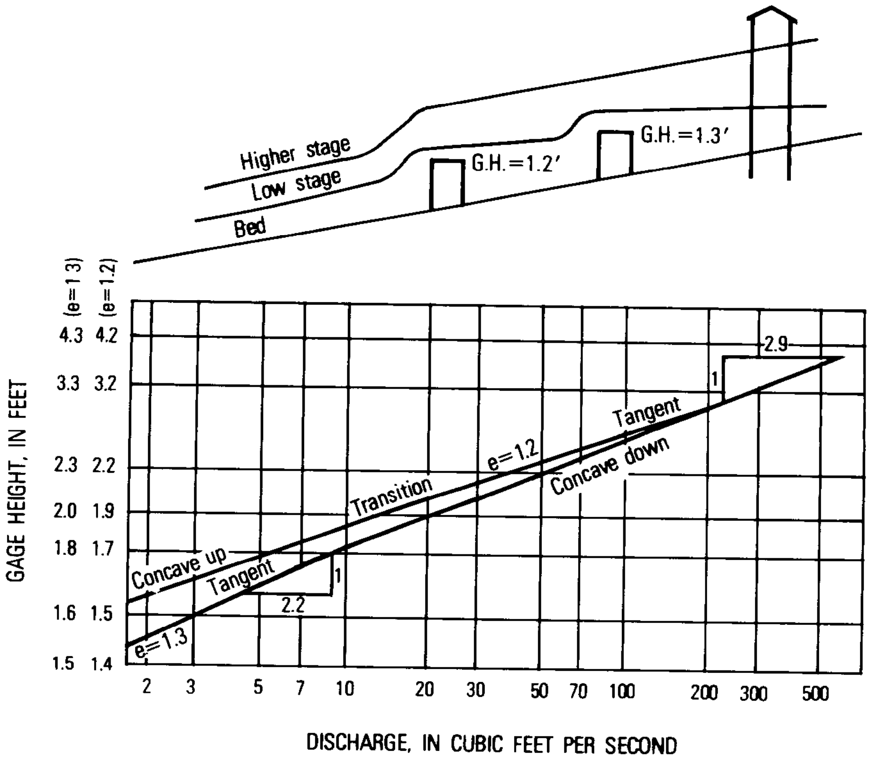


FIGURE 164.—Rating curve for a compound section control at Muncy Creek near Sonestown, Pa.

CHANNEL CONTROL

CHANNEL CONTROL FOR STABLE CHANNELS

The term "stable channels," as used in this report, is a relative term. Virtually all natural channels are subject to at least occasional change as a result of scour, deposition, or the growth of vegetation, but some alluvial channels, notably those whose bed and banks are composed of sand, have movable boundaries that change almost continuously, as do their stage-discharge relations. For the purpose of this manual, stable channels include all but sand channels. Sand channels are discussed in the section titled, "Sand-Channel Streams."

Almost all streams that are unregulated by man have channel control at the higher stages, and among those with stable channels, all but the largest rivers have section control at low stages. Because this section of the manual discusses only stable channels that have channel control for the entire range of stage experienced, the discussion is limited to the natural channels of extremely large rivers and to artificial channels constructed without section controls. The artificial channels may be concrete-lined, partly lined or rip-rapped, or unlined. Streams that have compound controls involving channel control are discussed in the section titled, "Compound Controls Involving Channel Control."

The Manning discharge equation for the condition of channel control, as discussed in chapter 9, under the heading, "Slope-Area Method," is

$$Q = \frac{1.486}{n} AR^{2/3}S^{1/2} \text{ (English units)}, \tag{65}$$

or

$$Q = \frac{1}{n} AR^{2/3}S^{1/2} \text{ (metric units)} \tag{66}$$

In analyzing an artificial channel of regular shape, whose dimensions are fixed, flow at the gage is first assumed to be at uniform depth. Consequently, for any stage all dimensions on the right side of the equations are known except n. A value of n can be computed from a single discharge measurement, or an average value of n can be computed from a pair of discharge measurements, and thus a preliminary rating curve for the artificial channel can be computed for the entire range of stage from the results of a pair of discharge measurements. If subsequent discharge measurements depart from the computed rating curve, it is likely that the original assumption of flow at uniform depth was erroneous. That means that the energy slope, S, is not parallel to the bed slope, but varies with stage, and that the value of n, which was computed on the basis of bed slope, is also in error. The rating curve must be revised to fit the plotted discharge measurements, but the preliminary rating curve may be used as a guide in shaping the required extrapolation of the rating curve. The extrapolation should also be checked by application of the conveyance-slope method of rating extrapolation, which is described in the section titled, "Conveyance-Slope Method."

To understand the principles that underlie the stage-discharge relation for channel control in a natural channel of irregular shape we return to the Manning equation and make some simplifying assumptions in that equation. We assume, not unreasonably, that at the higher stages n is a constant and that the energy slope (S) tends to become constant. Furthermore, area (A) is approximately equal to depth (D) times width (W). We make the substitution for A in equation 65 or 66, and by expressing $S^{1/2}/n$ as a constant, C_1, we obtain

$$Q = C_1 (D)(W)R^{2/3}. \text{ (approx.)}$$

If the hydraulic radius (R) is considered equal to D, and W is considered a constant, the equation becomes

$$Q = CD^{1.67} = C(G-e)^{1.67} \text{ (approx.)} \tag{67}$$

However, unless the stream is exceptionally wide, R is appreciably smaller than D. This has the effect of reducing the exponent in the last equation although this reduction may be offset by an increase of S or W with discharge. Changes in roughness with stage will also affect the value of the exponent. The net result of all these factors is a discharge equation of the form

$$Q = C\,(G-e)^N$$

where N will commonly vary between 1.3 and 1.8 and practically never reach a value as high as 2.0.

An example of a discharge rating for channel control in a natural stream is given in the following section, where compound controls that involve channel control are discussed.

COMPOUND CONTROLS INVOLVING CHANNEL CONTROL

In the preceding section mention was made of the fact that compound control of the stage-discharge relation usually exists in natural channels, section control being effective for the lower stages and channel control being effective for the higher stages. An example of that situation is shown in figure 165, the rating curve for the Susquehanna River at Harrisburg, Pa. The low-water control is a low weir with zero flow at gage height 2.2 feet. At a stage of 3.9 feet this control starts to drown out and channel control becomes effective. If the low end of the rating is made a tangent, a value of e = 2.2 ft must be used. Because the value of e for the upper end of the rating is something less than 2.2 feet, the high end becomes concave downward. If the high end of the curve is made a tangent, the effective value of e is found to be 0.0 ft. This being too low a value of e for the lower end of the curve, the low end becomes concave upward.

If the rating for a section control (low end of the curve) is a tangent, the value of the exponent N is expected to be greater than 2.0. In this example, N = 2.3. If the rating for a channel control (high end of the curve) is a tangent, the value of N is expected to be less than 2.0, and probably between 1.3 and 1.8. In this example N = 1.3. Should overbank flow occur the rating curve will bend to the right.

It can be demonstrated, nonrigorously, that straight-line rating curves for section control almost always have a slope greater than 2.0 and that those for channel control have a slope less than 2.0. It has

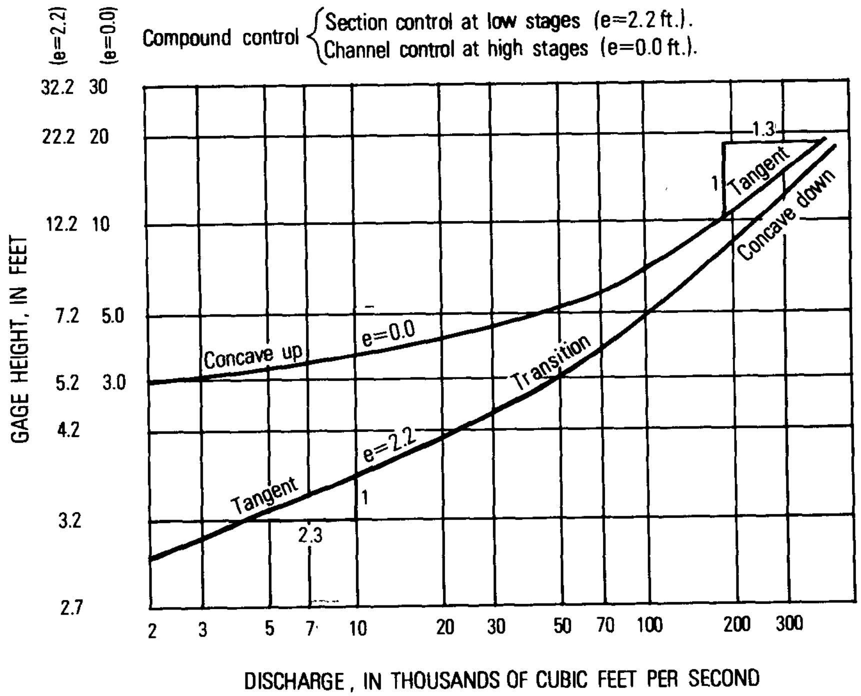


FIGURE 165.—Rating curve for a compound control at Susquehanna River at Harrisburg, Pa.

been shown that the equation for a straight-line rating on log paper is $Q = CH^N$, where N is the slope of the line. The first derivative of this equation is a measure of the change in discharge per tenth of a foot change in stage. The first derivative is:

$$\frac{dQ}{dH} = CNH^{N-1}.$$

Second differences are obtained by differentiating again. The second derivative is:

$$\frac{d^2Q}{dH^2} = CN\ (N-1)H^{N-2}.$$

Examination of the second derivative shows that second differences increase with stage when N is greater than 2.0 and decrease with stage when N is less than 2.0.

The hypothetical rating for a compound control is shown in table 20. This rating represents the condition of section control at the lower

TABLE 20.—*Hypothetical stage-discharge rating for a compound control*

[The higher values of discharge are rounded as normally used in a rating table; the precise values that required rounding are in parentheses]

Gage height (ft)	Discharge (ft^3/s)	Difference per tenth of a foot	Second difference
1.0	100		
		20	
1.1	120		1
		21	
1.2	141		1
		22	
1.3	163		2
		24	
1.4	187		2
		26	
1.5	213		3
		29	
1.6	242		3
		32	
1.7	274		4
		36	
1.8	310		3
		40 (39)	
1.9	350 (349)		2
		40 (41)	
2.0	390		2
		45 (43)	
2.1	435 (433)		2
		45 (45)	
2.2	480 (478)		2
		45 (47)	
2.3	525		1
		50 (48)	
2.4	575 (573)		1
		50 (49)	
2.5	625 (622)		1
		50 (50)	
2.6	675 (672)		1
		50 (51)	
2.7	725 (723)		1
		50 (52)	

stages and channel control at the higher stages. If two values of discharge are shown for an item in the rating table, the figure in parenthesis is the exact value and the figure without a parenthesis is the "rounded" value that normally would be used in the rating table. Experienced hydrographers will recognize the progression of discharge values in this table as being typical. Inspection of the second difference column shows the second differences to be increasing at the low-water end (section control, $N > 2$) and decreasing at the high-water end (channel control, $N < 2$). These are the results that one would predict from the discussion in the preceding paragraph.

EXTRAPOLATION OF RATING CURVES

Rating curves, more often than not, must be extrapolated beyond the range of measured discharges. The preceding material in this

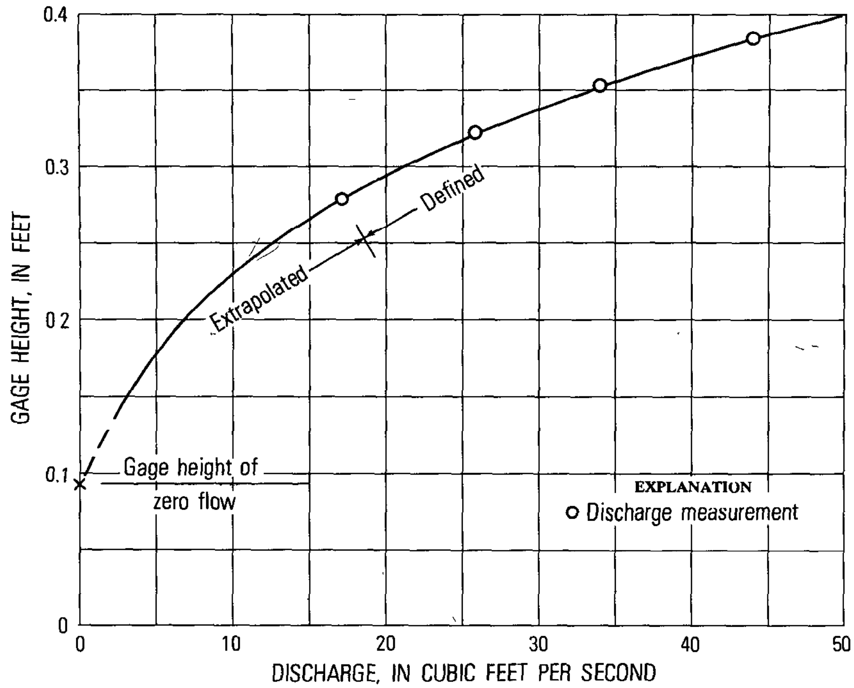


FIGURE 166.—Example of low-flow extrapolation on rectangular-coordinate graph paper.

chapter explained the principles governing the shape of logarithmic rating curves to guide the hydrographer in shaping the extrapolated segment of a rating. However, even with a knowledge of those principles, a large element of uncertainty exists in the extrapolation process. The purpose of this section of the manual is to describe methods of analysis that will reduce the degree of uncertainty.

LOW-FLOW EXTRAPOLATION

Low-flow extrapolation is best performed on rectangular-coordinate graph paper because the coordinates of zero flow can be plotted on such paper. (Zero discharge cannot be plotted on logarithmic graph paper.) An example of such an extrapolation is shown in figure 166, where the circled points represent discharge measurements plotted on the coordinate scales of gage height versus discharge. The rating in the example is defined by the measurements down to a gage height of 0.28 ft, but an extrapolation to a gage height of 0.14 ft is required. Field observation has shown the low point on the control (point of zero flow) to be at gage height 0.09 ft.

The method of extrapolation in figure 166 is self-evident. A curve has been drawn between the plotted points at gage heights 0.09 ft and 0.28 ft to merge smoothly with the rating curve above 0.28 ft. There is no assurance that the extrapolation is precise—low-flow discharge measurements are required for that assurance—but the extrapolation shown is a reasonable one.

HIGH-FLOW EXTRAPOLATION

As mentioned in the "Introduction" of this chapter, the problem of high-flow extrapolation can be avoided if the unmeasured peak discharge for the rating is determined by the use of the indirect methods discussed in chapter 9. In the absence of such peak-discharge determinations, estimates of the discharges corresponding to high values of stage may be made by using one or more of the following four techniques:

1. conveyance-slope method,
2. areal comparison of peak-runoff rates,
3. step-backwater method, and
4. flood routing.

As a matter of fact, only as a last resort should the rating curve be extrapolated beyond a discharge value equal to twice the greatest measured discharge. If a greater extrapolation is required, the hydrologist should first try to define the upper end of the rating by use of one of the indirect peak-discharge determination methods of chapter 9. If for some reason, that course of action is not feasible, he should then use at least one of the four techniques listed above.

The knowledgeable reader of this manual may notice the absence from the above list of two techniques that used to be standard practice—the velocity-area method and the Q vs $Ad^{1/2}$ method. The Q vs $Ad^{1/2}$ method was superior to the velocity-area method and largely supplanted it; similarly, the conveyance-slope method, because of its superiority, has, in the last two decades, largely supplanted the Q vs $Ad^{1/2}$ method. Of the three somewhat similar methods, only the conveyance-slope method is described here, because a description of the two earlier methods (Corbett and others, 1943, p. 91–92) would have only academic, rather than practical, value.

CONVEYANCE-SLOPE METHOD

The conveyance-slope method is based on equations of steady flow, such as the Manning equation. In the Manning equation,

$$Q = KS^{1/2}. \tag{68}$$

The conveyance, K, equals $\frac{1.486}{n} AR^{2/3}$ when English units are used, and $K = \frac{1}{n} AR^{2/3}$ when metric units are used. Values of A and R corresponding to any stage can be obtained from a field survey of the discharge-measurement cross section, and values of the coefficient n can be estimated in the field. Thus, the value of K, embodying all the elements that can be measured or estimated, can be computed for any given stage. (We shall soon see that errors in estimating n are usually not critical.) Values of gage height vs K, covering the complete range of stage up to the required peak gage height, are computed and plotted on rectangular graph paper. A smooth curve is fitted to the plotted points.

Values of slope, S, which is actually the energy gradient, are usually not available even for measured discharges. However, for the measured discharges, $S^{1/2}$ can be computed by dividing each measured discharge by its corresponding K value; S is then obtained by squaring the resulting value of $S^{1/2}$. Values of gage height vs S for the measured discharges are plotted on rectangular graph paper, a curve is fitted to the plotted points, and the curve is extrapolated to the required peak gage height. The extrapolation is guided by the knowledge that S tends to become constant at the higher stages. That constant slope is the "normal" slope, or slope of the streambed. If the upper end of the defined part of the curve of gage height vs S indicates that a constant or near-constant value of S has been attained, the extrapolation of the curve can be made with confidence. The discharge for any particular gage height will be obtained by multiplying the corresponding value of K from the K curve by the square root of the corresponding value of S from the S curve. We see that errors in estimating n will have minor effect because the resulting percentage error in computing K is compensated by a similar percentage error in the opposite direction in computing $S^{1/2}$. In other words, the constancy of S is unaffected, but if K is, say, 10 percent high, $S^{1/2}$ will be 10 percent low, and the two discrepancies are canceled when multiplication is performed. However, if the upper end of the defined part of the curve of gage height vs S has not reached the stage where S has a near-constant value, the extrapolation of the curve will be subject to uncertainty. In that situation the general slope of the streambed, as determined from a topographic map, provides a guide to the probable constant value of S that should be attained at high stages.

As mentioned in the preceding paragraph, the discharge for any particular gage height is obtained by the multiplication of appropriate values of K and $S^{1/2}$, and in that manner the upper end of the stage-discharge relation is constructed.

Figure 167 provides an example of the conveyance-slope method, as used for rating-curve extrapolation at the gaging station on Klamath River at Somes Bar, Calif. The conveyance curve is based on

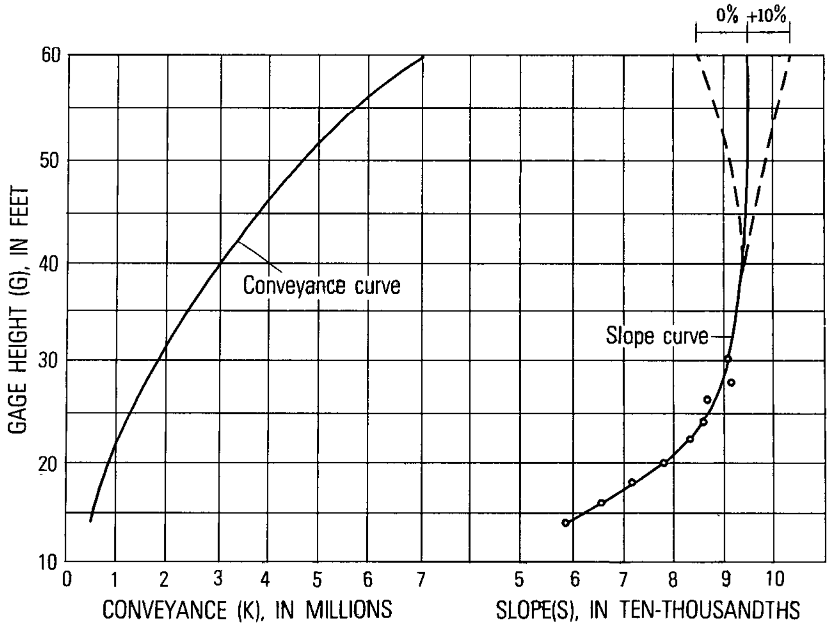


FIGURE 167.—High-flow extrapolation by use of conveyance-slope method—Klamath River at Somes Bar, Calif.

values of K computed from the geometry of the measurement cross section. The slope curve is defined to a gage height of 30 ft by discharge measurements (circled points), and extrapolated as the solid line to the peak gage height of 60 ft. It appears highly unlikely that the slope curve at a gage height of 60 ft will fall outside the limiting dashed curves shown in figure 167; in other words, it appears unlikely that the value of S at 60 ft (0.00095) is in error by more than ± 10 percent. If that is true, when the square root of S is computed and then used in a computation of peak discharge, the error for both $S^{1/2}$ and Q reduces to ± 5 percent. Although the attainment of so high an accuracy is highly improbable, the fact remains that one can place considerable confidence in the discharge computed for a gage height of 60 ft in this example. It should be mentioned here that the likelihood of a decrease in slope at high stages, as shown by the dashed curve on the left of the slope curve, is greatest when overbank flows occur.

In the above example conditions were ideal for application of the conveyance-slope method, and the example in figure 167 may therefore be misleading with regard to the general accuracy of the method. The conveyance-slope method assumes first that the geometry of the cross-section used for discharge measurements is fairly representative of that of a long reach of downstream channel. The need to meet this assumption immediately eliminates from consideration those gaging stations where discharge measurements are made at constricted cross sections, such as occur at many bridge- and cableway-measurement sections.

The conveyance-slope method also assumes that slope tends to become constant (uniform flow) at the higher stages. That is strictly true only for long, straight channels of uniform cross section, but natural channels that meet that description are virtually nonexistent. Consequently, the slope-stage relation may be anything but a vertical line at the upper stages. In the example in figure 167, a judgment decision, based on a knowledge of the channel characteristics, was made concerning the "probable" limiting positions of the stage-slope relation—the dashed lines on the graph—to give some idea of the "probable" error of the discharge computation. However, even given that knowledge of channel characteristics, if the two highest discharge measurements (two highest circles on the slope curve) had not been available it would have been impossible to position the upper end of the slope curve with any confidence. Fortunately there is a mitigating factor; an error of even as much as 40 percent in the value of slope at the upper end of the slope curve would give an error in discharge of either +18 percent ($\sqrt{1.40}-1.0=0.18$) or −23 percent ($1.0-\sqrt{0.60}=0.23$), depending on whether the estimate of slope was high or low.

In summary, the conveyance-slope method is a helpful adjunct in extrapolating rating curves, but its limitations must be understood so that it is not misused.

AREAL COMPARISON OF PEAK-RUNOFF RATES

When flood stages are produced over a large area by an intense general storm, the peak discharges can often be estimated, at gaging stations where they are lacking, from the known peak discharges at surrounding stations. Usually each known peak discharge is converted to peak discharge per unit of drainage area before making the analysis. In other words, peak discharge is expressed in terms of cubic feet per second per square mile or cubic meters per second per square kilometer.

If there has been relatively little difference in storm intensity over the area affected, peak discharge per unit area may be correlated with drainage area alone. If storm intensity has been variable, as in mountainous terrain, the correlation will require the use of some index of storm intensity as a third variable. Figure 168 illustrates a multiple correlation of that type where the independent variables used were drainage area and maximum 24-hour basinwide precipitation during the storm of December 1964 in north coastal California.

The peak discharges estimated by the above method should be used only as a guide in extrapolating the rating curve at a gaging station. The basic principles underlying the extrapolation of logarithmic rating curves are not to be violated to accommodate peak-discharge values that are relatively gross estimates, but the estimated discharges should properly be given consideration in the extrapolation process.

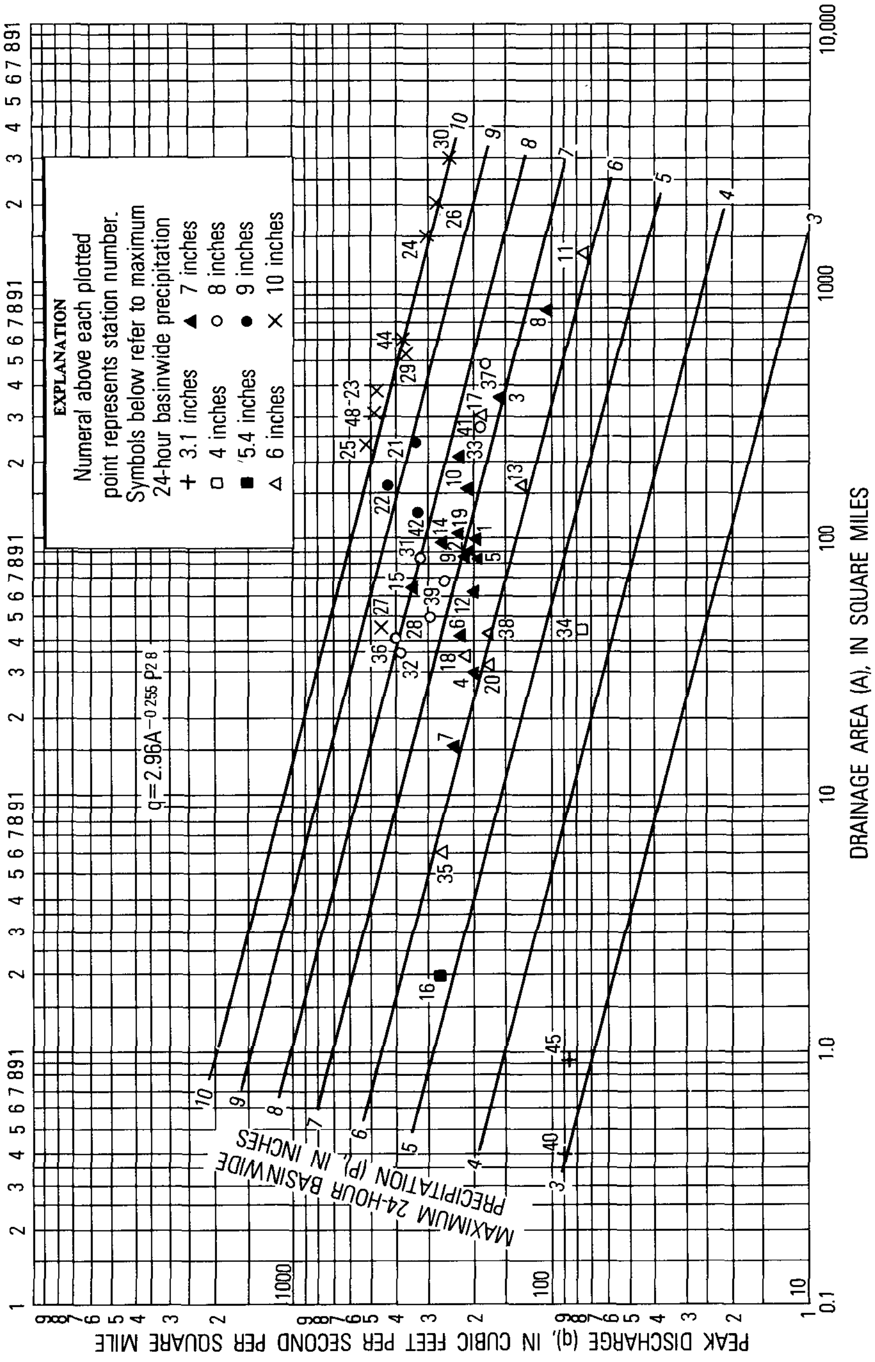
STEP-BACKWATER METHOD

The step-backwater method is a technique in which water-surface profiles for selected discharges are computed by successive approximations. The computations start at a cross section where the stage-discharge relation is known or assumed, and they proceed to the gage site whose rating requires extrapolation. If flow is in the subcritical regime, as it usually is in natural streams, the computations must proceed in the upstream direction; computations proceed in the downstream direction if flow is in the supercritical regime. In the discussion that follows, the usual situation of subcritical flow will be assumed.

Under conditions of subcritical flow, water-surface profiles converge upstream to a common profile. For example, the stage for a given discharge at a gated dam may have a wide range of values depending on the position of the gates. At a gaging station far enough upstream to be beyond the influence of the dam, the stage for that discharge will be unaffected by the gate operations. Consequently, when the water-surface profile is computed for a given discharge in the reach between the dam and the gaging station, the segment of the computed profile in the vicinity of the gage will be unaffected by the value of stage that exists at the dam. However, it will be necessary that the computations start at the dam and proceed upstream, subreach by subreach (in "steps"). It follows, therefore, that if an initial cross section for the computation of the water-surface profile is selected far enough downstream from the gage, the computed water-surface elevation at the gage, corresponding to any given discharge, will have a single value regardless of the stage selected for the initial site.

A guide for determining the required distance (L) between gaging station and initial section is found in the dimensionless graph in figure 169. The graph, (Bailey and Ray, 1966), has for its equation,

FIGURE 168.—Relation of peak discharge to drainage area and maximum 24-hour basinwide precipitation in north coastal California, December 1964.


EXPLANATION
Numeral above each plotted point represents station number. Symbols below refer to maximum 24-hour basinwide precipitation
+ 3.1 inches
□ 4 inches
■ 5.4 inches
△ 6 inches
▲ 7 inches
○ 8 inches
● 9 inches
× 10 inches
$q=2.96A^{-0.255}P^{2.8}$
MAXIMUM 24-HOUR BASINWIDE PRECIPITATION (P), IN INCHES
DRAINAGE AREA (A), IN SQUARE MILES
PEAK DISCHARGE (q), IN CUBIC FEET PER SECOND PER SQUARE MILE

$$\frac{LS_o}{\bar{d}} = 0.86 - 0.64 \left(\frac{S_o C^2}{g} \right) \tag{69}$$

where

L is the distance required for convergence,
S_0 is bed slope,
$\bar{d}$ is mean depth for the smallest discharge to be considered,
g is the acceleration of gravity, and
C is the Chezy coefficient.

If a rated cross section is available downstream from the gage, that cross section would be used as the initial section, of course, and there would be no need to be concerned with the above computation of L.

After the initial site is selected, the next step is to divide the study reach, that is, the reach between the initial section and the gaging station, into subreaches. That is done by selecting cross sections where major breaks in the high-water profile would be expected to occur because of changes in channel geometry or roughness. Those cross sections are the end sections of the subreaches. The cross sections are surveyed and roughness coefficients are selected for each subreach. That completes the field work for the study.

The first step in the computations is to select a discharge, Q, for study, and obtain a stage at the initial section for use with that value of discharge. If the initial section is a rated cross section, that stage will be known. If the initial section is not a rated cross section, an estimated stage there is computed from the estimated mean depth ($\bar{d}$) for discharge Q; $\bar{d}$ in turn is estimated by cut-and-try computations from a variation of the Chezy equation,

$$\bar{d} = \frac{Q^2}{(CA)^2 S_o} \tag{70}$$

where

C is the Chezy coefficient,
A is the cross-sectional area corresponding to $\bar{d}$, and
S_0 is the bed slope (or water-surface slope).

Step-backwater computations are then applied to the subreach farthest downstream. We have a known or estimated stage at the downstream cross section for the value of Q being considered; the object of the computations is to determine the stage at the upstream end of the subreach that is compatible with that value of Q. The computation for each subreach is based on a steady-flow equation, such as the Chezy or Manning equation, after the equation has been modified for nonuniformity in the subreach by use of the difference in velocity head at the end cross sections. (See section in chapter 9 titled,

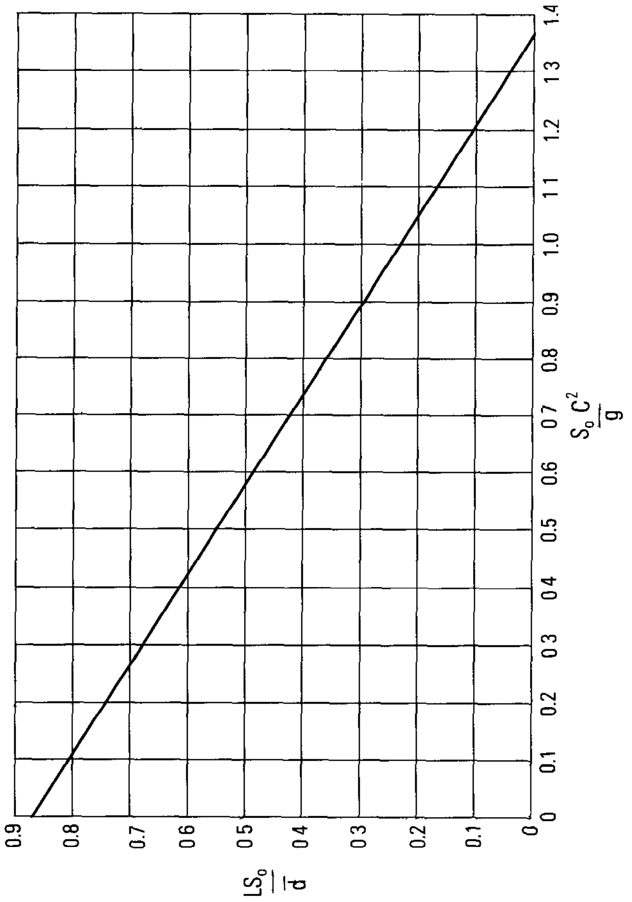


FIGURE 169.—Dimensionless relation for determining distance required for backwater profiles to converge.

"Slope-Area Method.") It will be recalled that the Chezy equation is related to the Manning equation by the formulas,

$$C = \frac{1.486}{n} R^{1/6} \text{ (English units)} \tag{71}$$

or

$$C = \frac{1}{n} R^{1/6} \text{ (metric units)} \tag{71a}$$

where n is the Manning roughness coefficient and R is the hydraulic radius.

By shifting terms in the modified Chezy equation, the following equation is obtained for the difference in water-surface elevation (Δh) between the upstream (subscript 1) and downstream (subscript 2) cross sections.

$$\Delta h = h_1 - h_2 = \frac{(\Delta L) V_1 V_2}{C^2 R_1^{1/2} R_2^{1/2}} + \frac{(\alpha_2 V_2^2 - \alpha_1 V_1^2)}{2g} \quad (1+k), \tag{72}$$

where

h is stage;

ΔL is the length of the subreach;

V is average velocity in the cross section;

g is the acceleration of gravity;

k is a constant whose value is zero when $\alpha_2 V_2^2 > \alpha_1 V_1^2$; and whose value is 0.5 when $\alpha_2 V_2^2 < \alpha_1 V_1^2$; and

α is the velocity-head coefficient whose value is dependent on the velocity distribution in the cross section.

As for α, in many countries its value is assumed to be 1.1; in the U.S.A. its value is assumed to be 1.0 for cross sections of simple shape, but its value is computed for cross sections of complex shape that require subdivision. The equation used for that purpose is

$$\alpha = \frac{\Sigma\,(K_i^3/a_i^2)}{K_T^3/A_T^2}, \tag{73}$$

where the subscript i refers to the conveyance (K) or area (a) of the individual subsections, and the subscript T refers to the conveyance (K) or area (A) of the entire cross section. With regard to conveyance, K,

$$K_i = C a_i R_i^{1/2}, \text{ and } K_T = \Sigma K_i$$

We return to our computations for the downstream subreach. A trial value of stage for discharge Q is selected for the upstream cross section, and values of A, V, and R are computed for the upstream and downstream cross sections. Those values are substituted in equations 72 and 73 and after solving for Δh, the computed value of Δh is compared with the difference between the trial value of stage at the upstream cross section and the known or assumed stage at the downstream cross section. Seldom will the two values agree after a single trial computation; if they do not agree, a second trial value of stage is selected for the upstream cross section. The computational procedure is repeated and the newly computed value of Δh is compared with its corresponding trial value. The computations are repeated as many times as are necessary to obtain agreement between the computed Δh and the difference between the trial stage at the upstream cross section and the known or assumed stage at the downstream cross section.

After a satisfactory value of stage has been determined for the upstream cross section, that cross section becomes the downstream cross section for the next subreach upstream. Computations similar to those described in the preceding paragraph are repeated for that subreach, and for each succeeding subreach, to provide a water-surface profile extending to the gaging station that is applicable to the discharge value (Q) being studied.

If the stage corresponding to discharge Q at the initial cross section was known, the stage computed for the gage is satisfactory. If the stage at the initial cross section was estimated from equation 70, it is necessary to repeat the above computations twice using other values of stage at the initial cross section for the same discharge Q. That is done to assure convergence of the water-surface profiles at the gage. The computations are repeated, first using an initial stage about 0.5 to 1.0 ft (0.15 to 0.30 m) higher than that originally used, and then using an initial stage about 0.5 to 1.0 ft lower than that originally used. All three sets of computations for discharge should result in almost identical values of stage at the gaging station for discharge Q. If they do not, the initial cross section for the step-backwater computation should be moved farther downstream, and all computations previously described must be repeated. If the three sets of computations give water-surface profiles that converge at a common stage at the gage, the entire procedure is repeated for other discharges until enough data are obtained to define the high-water rating for the gaging station.

From the preceding discussion it should be evident that the computations will be expedited if, in a preliminary step, the three relations of stage versus area (A), hydraulic radius (R), and conveyance (K), are computed for each cross section. Even then, the computations will be laborious and the use of a digital computer is therefore recommended.

The step-backwater method can be used to prepare a preliminary rating for a gaging station before a single discharge measurement is made. A smooth curve is fitted to the logarithmic plot of the discharge values that are studied. The preliminary rating can be revised, as necessary, when subsequent discharge measurements indicate the need for such revision. If the step-backwater method is used to define the high-water end of an existing rating curve, the discharge values investigated should include one or more of the highest discharges previously measured. By doing so, selected roughness coefficients can be verified, or can be modified so that step-backwater computations for the measured discharges provide stages at the gaging station that are in agreement with those observed. The computations for the high-water end of the rating can then be made with more confidence, in the knowledge that reasonable values of the roughness coefficients are being used. There will also be assurance of continuity between the defined lower part of the rating and the computed upper part.

FLOOD ROUTING

Flood-routing techniques may be used to test and improve the overall consistency of records of discharge during major floods in a river basin. The number of direct observations of discharge during such flood periods is generally limited by the short duration of the flood and the inaccessibility of certain stream sites. Through the use of flood-routing techniques, all observations of discharge and other hydrologic events in a river basin may be combined and used to evaluate the discharge hydrograph at a single site. The resulting discharge hydrograph can then be used with the stage hydrograph for that gage site to construct the stage-discharge relation for the site; or, if only a peak stage is available at the site, the peak stage may be used with the peak discharge computed for the hydrograph to provide the end point for a rating-curve extrapolation.

Flood-routing techniques, of which there are many, are based on the principle of the conservation of mass—inflow plus or minus change in storage equals outflow. It is beyond the scope of a stream-gaging manual to treat the subject of flood routing; it is discussed in most standard hydrology texts (for example, Linsley, Kohler, and Paulhus, 1949, p. 485–541).

SHIFTS IN THE DISCHARGE RATING

Shifts in the discharge rating reflect the fact that stage-discharge relations are not permanent but vary from time to time, either gradually or abruptly, because of changes in the physical features that form the control for the station. If a specific change in the rating stabilizes to the extent of lasting for more than a month or two, a new rating curve is usually prepared for the period of time during which the new stage-discharge relation is effective. If the effective period of a specific rating change is of shorter duration, the original rating curve is usually kept in effect, but during that period shifts or adjustments are applied to the recorded stage, so that the "new" discharge corresponding to a recorded stage is equal to the discharge from the original rating that corresponds to the adjusted stage. For example, assume that vegetal growth on the control has shifted the rating curve to the left (minus shift), so that in a particular range of discharge, stages are 0.05 ft higher than they originally had been. To obtain the discharge corresponding to a recorded stage of, say, 1.30 ft, the original rating is entered with a stage of 1.25 ft (1.30−0.05) and the corresponding discharge is read. The period of time during which such stage adjustments are used is known as a period of shifting control.

Frequent discharge measurements should be made during a period of shifting control to define the stage-discharge relation, or magnitude(s) of shifts, during that period. However, even with infrequent discharge measurements the stage-discharge relation can be estimated during the period of shifting control if the few available measurements are supplemented with a knowledge of shifting-control behavior. This section of the report discusses such behavior. That part of the discussion that deals with channel-control shifts does not include alluvial channels, such as sand channels, whose boundaries change almost continuously; sand channels are discussed in the section titled, "Sand-Channel Streams."

The formation of ice in the stream and on section controls causes shifts in the discharge rating, but ice effect is not discussed here; it is discussed separately in the section titled. "Effect of Ice Formation on Discharge Ratings."

DETECTION OF SHIFTS IN THE RATING

Stage-discharge relations are usually subject to minor random fluctuations resulting from the dynamic force of moving water, and because it is virtually impossible to sort out those minor fluctuations, a rating curve that averages the measured discharges within close limits is considered adequate. Furthermore, it is recognized that discharge measurements are not error-free, and consequently an average curve drawn to fit a group of measurements is probably more accurate than any single measurement that is used to define the average curve. If a group of consecutive measurements subsequently plot to the right or left of the average rating curve, it is usually clearly evident that a shift in the rating has occurred. (An exception to that statement occurs where the rating curve is poorly defined or undefined in the range of discharge covered by the subsequent measurements; in that circumstance the indication is that the original rating curve was in error and requires revision.) If, however, only one or two measurements depart significantly from a defined segment of the rating curve, there may be no unanimity of opinion on whether a shift in the rating has actually occurred, or whether the departure of the measurement (s) results from random error that is to be expected occasionally in measurements.

Two schools of thought exist with regard to identifying periods of shifting control. In the U.S.A. and many other countries, a pragmatic approach is taken that is based on certain guidelines and on the judgment of the analyst. In other countries, notably the United Kingdom, the approach used is based on statistical theory. (It is reiterated that the discussion that follows excludes the constantly shifting alluvial channels that are discussed in the section on "Sand-Channel Streams.")

In the U.S.A., if the random departure of a discharge measurement from a defined segment of the rating curve is within ±5 percent of the discharge value indicated by the rating, the measurement is considered to be a verification of the rating curve. If several consecutive measurements meet the 5-percent criterion, but they all plot on the same side of the defined segment of the rating curve, they may be considered to define a period of shifting control. It should be mentioned that when a discharge measurement is made, the measurement is computed before the hydrographer leaves the gaging station and the result is plotted on a rating curve that shows all previous discharge measurements. If the discharge measurement does not check a defined segment of the rating curve by 5 percent or less, or if the discharge measurement does not check the trend of departures shown by recent measurements, the hydrographer is normally expected to make a second discharge measurement to check his original measurement. However, at many stations the 5-percent criterion may be too stringent for low-flow measurements because of control insensitivity. At those installations departures in excess of 5 percent are generally acceptable if the indicated shift does not exceed 0.02 ft.

In making a check measurement, the possibility of systematic error is eliminated by changing the measurement conditions as much as possible. The meter and stopwatch are changed, or the stopwatch is checked against the movement of the second hand of a standard watch. If the measurements are being made from a bridge, boat, or cableway, the measurement verticals are changed by measuring at verticals between those originally used; if wading measurements are being made, a new measurement section is sought, or the measurement verticals in the original section are changed. If the check measurement checks the original rating curve or current rating trend by 5 percent or less, the original discharge measurement will be given no consideration in the rating although it is still entered in the records. If the check measurement checks, by 5 percent or less, the original discharge measurement or the trend of that measurement if the stage has changed, the two measurements are considered to be reliable evidence of a new shift in the stage-discharge relation. If the check measurement fails to check anything that has gone before, a second check measurement is made and the most consistent two of the three measurements are used for rating analysis. The need for a second check measurement is a rarity, but it may possibly occur.

Thus, in the U.S.A., a single discharge measurement and its check measurement, even if unsupported by later measurements, may mark a period of shifting control. The engineer who analyzes the rating does have the responsibility of explaining the reason for the short-lived shift—it can often be explained as having started as a result of fill (or scour) on a preceding stream rise and as having ended as a result of scour (or fill) on the recession or on a following rise.

In the United Kingdom, the analysis of the rating starts in the usual way; the chronologically numbered discharge measurements are plotted on logarithmic graph paper and are fitted by eye with a smooth curve. Where compound controls exist, there may be one or more points of inflection in the curve. In the statistical analysis that follows, each segment of the rating curve between inflection points is treated separately. The standard deviation (S_D) of the plotted points, in percent, is computed for each segment, using the standard statistical equation,

$$S_D = \sqrt{\frac{\Sigma d^2}{N-1}} \, , \tag{74}$$

where

- d is the departure of a discharge measurement from the rating curve, in percent, and
- N is the number of measurements used to define the segment of the rating curve.

Use of the standard deviation (S_D) in detecting rating shifts is explained as follows in ISO Recommendation R 1100 (1969, p. 15). On the average, 19 out of 20 measurements should depart from the particular segment of the rating curve by no more than $2S_D$ percent. Any subsequent discharge measurement that departs by a much greater percentage—say, $3S_D$ percent—can be regarded as the result of faulty measurement, except in those cases where two or more consecutive measurements, either chronologically or over a range of stage, appear to be well on one side of the $\pm 2S_D$ limit. Where that occurs, a change in the stage-discharge relation is required—either in the form of a reconstruction of the original relation using the additional discharge measurements, or in the form of a new stage-discharge relation because a shift in the control is indicated.

In the United Kingdom, additional statistical tests are given the rating to assure that : (1) the discharge measurements show no preponderance of either plus or minus departures from the rating curve; (2) the number of "runs" of successive plus or minus departures from the rating, examined in ascending order of stage, are neither excessively large nor excessively small, and (3) the average percentage departure of all measurements from the rating curve does not differ significantly from zero.

In the U.S.A. the above statistical approach is not favored for several reasons. First, it is felt that the limiting criteria of $2S_D$ percent will usually exceed the 5 percent criteria preferred in the U.S.A. Second, any statistical approach gives equal weight to all discharge measurements used in the analysis. In the U.S.A. hydrographers rate the probable accuracy of the measurements they make on the basis of measuring conditions at the time, without reference to how closely the measurements plot on the rating curve. The feeling in the U.S.A. is that more weight in the analysis should be given to measurements rated good to excellent than to measurements rated fair to poor. Third, while it is agreed that in general an average curve drawn to fit a group of measurements is probably more accurate than any single measurement that is used to define the average curve, it is also felt in the U.S.A. that any subsequent measurement that is verified by a check measurement is more accurate than the rating-curve value of discharge, particularly at a station that is historically known to have rating-curve shifts.

RATING SHIFTS FOR ARTIFICIAL CONTROLS

Weirs.—Artificial controls are not subject to scour and fill by high flows, but the streambed immediately upstream from the weir may be so affected. If scour occurs in the pool formed by the weir, the pool is deepened and velocity of approach decreases. The net result is a smaller discharge for a given stage than under pre-scour conditions; that is, the rating curve for the period of scour will shift to the left of the rating curve for pre-scour conditions. The converse occurs if the weir pool has been subjected to deposition or fill.

The effect of such scour and fill on the stage-discharge relation is usually relatively minor, and usually can be expressed by a parallel shift of most of the section-control portion of the rating curve that is plotted as a straight line on logarithmic graph paper. If only a single discharge measurement is available for defining the parallel shift curve, the shift curve is drawn to pass through that measurement. If more than one discharge measurement is available, and there is no evidence of a progressive rating shift with time, the parallel shift curve is drawn to average the discharge measurements. If the discharge measurements indicate a progressive rating shift with time, shifts are prorated with time. However, what may appear to be a gradually progressive shift, may in fact be several discrete shifts caused by individual peak flows whose occurrences are not widely separated in time. The shift in stage to be applied to recorded gage heights during the period of shifting control is determined from the vertical spacing between the original rating curve and the shift curve.

The shift, if attributable to fill, is considered to start after the peak discharge of a stream rise that preceded the first of the variant discharge measurements. Shift adjustments are therefore started on the recession of that rise. The shift, if attributable to scour, is considered to start during the high stages of a stream rise that preceded the first of the variant discharge measurements. Because those high stages generally occur when the section control is "drowned out" by channel control, the shift in the section-control segment of the rating is again commonly first applied after the peak discharge of the rise. The shifts are ended on a stream rise that follows the last variant discharge measurement, using the general principle that scour in the gage pool usually occurs during high stages and fill usually occurs during the recession of a stream rise.

The parallel shift discussed in a preceding paragraph requires some elaboration. A parallel shift of the rating curve on logarithmic graph paper indicates that for all stages the discharge changes by a fixed percentage, and that the difference in stage between the two lines increases with stage. However, it is not quite true that the discharge changes by a fixed percentage when the weir pool has scoured or filled. At extremely low flows there will be no effect because velocity of approach is negligible; that section of the original rating has a break in slope (see fig. 146; $G=1.3$ ft), and the lower end of the parallel shift curve above the break in slope should be warped to join the extreme low-water curve. The effect of scour or fill on the percentage change in discharge increases rapidly with stage to a maximum value and then slowly decreases to a percent change that does not differ greatly from the maximum percentage. The parallel shift drawn through the available discharge measurement(s) will adequately fit those relatively large percentage changes in discharge at the higher stages; the warped section of the shift curve at the lower stages will adequately fit the rapidly increasing percentage change in discharge at those lower stages. Figure 170 illustrates the above discussion; the

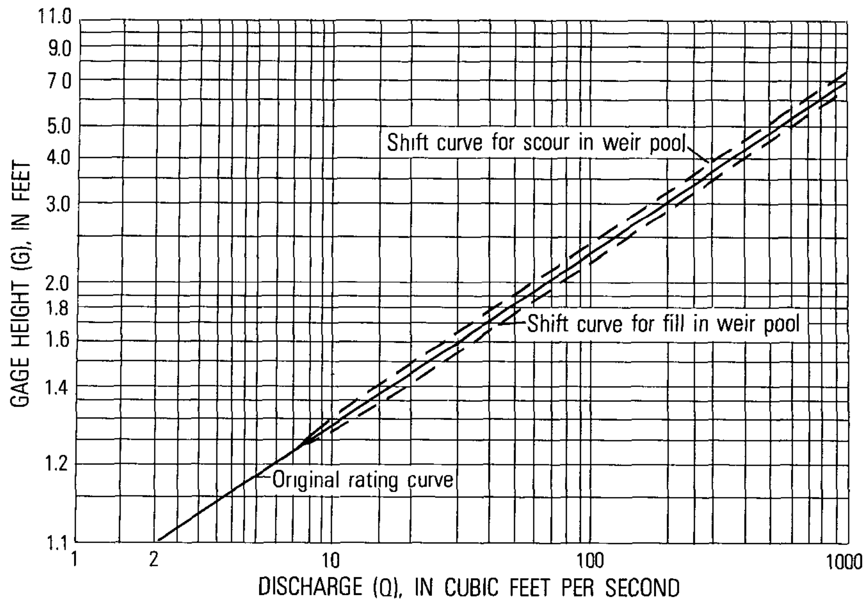


FIGURE 170.—Rating curve for hypothetical rectangular thin-plate weir, with shift curves for scour and fill in the weir pool.

original rating curve shown is a reproduction of that given in figure 146.

It has been mentioned frequently in this manual that section controls are usually submerged at high stages as a result of channel control becoming effective. The parallel shift curve described above should be extended to the stage where it either intersects the actual rating for channel control (in the case of scour in the weir pool) or can be warped into the rating for channel control (in the case of fill in the weir pool). If a shift has occurred simultaneously in the channel control (see section titled, "Rating Shifts for Channel Control"), the shift curves for the section-control and channel-control segments of the rating are drawn to form a continuous curve.

Up to now we have discussed changes in the velocity of approach that are caused only by scour and fill in the weir pool. The velocity of approach may also be affected by aquatic vegetation growing in the weir pool. Usually such an occurrence will reduce the velocity of approach by greatly increasing the friction loss, and the rating curve will shift to the left. However, the shift will not be abrupt, but will gradually increase as the growing season progresses. The aquatic growth in the pool may also encroach on the weir to the extent that the effective length (b) of the weir is reduced. The effect of a reduction in effective length of the weir is a parallel shift of the rating to the left when plotted on logarithmic graph paper. At all stages the discharge will be reduced by a percentage that is equal to the percentage change in effective length of the weir. The shift will either decrease gradually as the vegetation dies in the dormant season, or the shift may terminate abruptly if the vegetation is washed out by a stream rise.

Moss or algal growth may sometimes attach itself to a weir crest and thereby reduce the head on the weir for any given gage height (G). The head will be reduced by a constant value that is equal to the thickness of the growth. In other words, in the equation, head$=G-e$, the value of e is increased by the thickness of the growth. The reduction in head causes the rating to shift to the left, it being displaced vertically by an amount equal to the thickness of the growth. If the shift rating is plotted on rectangular-coordinate graph paper, it will be parallel to the original rating. If the shift rating is plotted on logarithmic graph paper, it will be a curve that is concave upward and asymptotic to the original linear rating curve at the higher stages. The growth of algae or moss on the weir should be removed with a wire brush before it becomes heavy enough to affect the stage-discharge relation. The effect of the shift caused by the algal growth disappears during stages when channel control becomes effective.

Flumes.—Shifts in the stage-discharge relation for flumes are most commonly caused by changes in the approach section—either in the channel immediately upstream from the flume or in the contracting section of the flume upstream from the throat. In either event the change is caused by the deposition of rocks and cobbles that are too large to pass through the flume; the flume is self-cleaning with regard to sediment of smaller size. Manual removal of the large debris should restore the original discharge rating of the flume.

The deposition of rocks and debris upstream from the flume may divert most of the flow to the gage-side of the flume and the build-up of water at the gage will result in a shift of the discharge rating to the left. Conversely, if most of the flow is diverted to the side of the flume opposite the gage, the discharge rating will shift to the right. In the above situation, the shift curve is usually drawn parallel to the original rating curve on logarithmic graph paper in much the same manner as was described earlier for shifts resulting from scour and fill in the pool behind the weir.

If rocks and cobbles are deposited at the entrance to the throat of the flume, they will cause the discharge rating to shift to the left because the stage at the gage will be raised higher than normal for any given discharge. A similar backwater effect will result from the growth of algae at the entrance to the throat.

The backwater effect, or decrease in head for a given gage height caused by deposition or algal growth at the entrance to the throat of the flume, has the effect of increasing the value of e in a linear logarithmic plot of the rating. The shift rating on logarithmic graph paper will be a curve that is concave upward and asymptotic to the original linear rating curve at the higher stages. The deposition of rocks and debris will be associated with a high-water event; the growth of algae will increase gradually with time.

Large rocks driven by high-velocity flow through the flume may erode the walls and floor of a concrete flume. The resulting increase in roughness and decrease in elevation of the concrete may cause shifts in the stage-discharge relation. The two effects tend to be compensating; an increase in roughness will shift the discharge rating to the left, and a decrease in elevation of the concrete surface will shift the discharge rating to the right. However, the latter effect usually predominates, particularly in supercritical-flow flumes.

RATING SHIFTS FOR NATURAL SECTION CONTROLS

The primary cause of changes in natural section controls is the high velocity associated with high discharge. Of those controls, a rock ledge outcrop will be unaffected by high velocities, but boulder, gravel, and sand-bar riffles are likely to shift, boulder riffles being the most resistant to movement and sand bars the least resistant. After a flood the riffles are often altered so drastically as to bear no resemblance to their pre-flood state, and a new stage-discharge relation must be defined. Minor stream rises usually move and sort the materials composing the riffle, and from the standpoint of the rating curve, the greatest effect is usually a change in the gage height of effective zero flow (e). The shift curve ideally should be defined by current-meter discharge measurements. However, if only one or two measurements are available for the purpose, they are examined and the gage-height shift that they indicate is applied to the section-control segment of the original rating curve. If the shift rating is plotted on rectangular paper, it will tend to be parallel to the original rating. The extreme low-water end of the curve can be extrapolated to the actual point of zero flow, as determined in the field when low-water discharge measurements are made. If the shift rating is plotted on logarithmic graph paper, it will be a curve that is either concave upward or downward, depending on whether the shift is to the left (increase in e) or the right (decrease in e). The shift curve will tend to be asymptotic to the linear rating at the higher stages of section control, but its precise slope in the range of stage where channel control is beginning to exert an effect, will depend on whether or not a shift has occurred in the channel-control segment of the rating curve.

(See section titled, "Rating Shifts for Channel Control.")

Vegetal growth in the approach channel of the control or on the control itself will affect the stage-discharge relation in the manner described on preceding pages, where rating shifts for weirs were discussed. Aquatic vegetation in the approach channel will affect the velocity of approach, and if the channel growth encroaches on the control, it may reduce the effective length of the control. Aquatic growth on the control itself will reduce the discharge corresponding to any given stage by reducing the head on the control and increasing the resistance to flow, and (or) by reducing the effective length of the control. The shifts associated with vegetal growth are cyclic and therefore change with time. The growth increases as the growing season progresses and declines during the dormant season, but shifts may terminate abruptly if the vegetation is washed out by a stream rise.

In temperate climates, accumulations of water-logged fallen leaves on section controls each autumn clog the interstices and raise the effective elevation of all section controls. The effect of an increase in the gage height of effective zero flow (e) is explained on a preceding page in the discussion of moss and algal growth on weirs. The build-up of water-logged leaves is progressive starting with the first killing frost (usually in October in the Northern Hemisphere) and reaching a maximum when the trees are bare of leaves. The first ensuing stream rise of any significance usually clears the control of fallen leaves.

Two other causes of backwater (increased gage height for a given discharge), unassociated with hydrologic events, also warrant discussion. Vacationers in the summer often use the gage pool for swimming, and they will often pile rocks on the control to create a deeper pool. This change in the height of the control manifests itself in the record of stage as an abrupt increase in gage height, usually during a rainless period, without any corresponding decline in stage that would be associated with the passage of a stream rise. The abrupt rise in stage fixes the time when the shift in the rating occurred; the magnitude of the change in stage is a measure of the change in the value of e. In some regions another cause of backwater is the construction of dams by beavers. These dams are built of boughs, logs, stones, and mud to create a pool that is part of the beavers' habitats. Again, the time of occurrence and the effect on the stage of the stream can be detected in the gage-height record which will show a gradual rise, usually over a period of a few days as the dam is being built, without the corresponding decline in stage that would be associated with a stream rise. The beaver dams usually remain in place until washed out by a high discharge.

RATING SHIFTS FOR CHANNEL CONTROL

As mentioned earlier, most natural streams have compound controls—section control for low-stages and channel control for high stages. The shifts in section control that were described on the preceding pages are commonly accompanied by shifts in channel control.

The most common cause of a shifting channel control, in a relatively stable channel, is scour or fill of the streambed caused by high-velocity flow. The scour usually occurs during a stream rise and fill usually occurs on the recession, but that statement is an oversimplification of the highly complex process of sediment transport. The degree of scour in a reach is dependent not only on the magnitude of the discharge and velocity, but also on the sediment load coming into the reach. On some streams it has been found that when scour is occurring in a pool at a meander bend there is simultaneous filling on the bar or riffle at the crossover, or point of inflection between successive meander bends; on other streams scour has been found to take place simultaneously through relatively long reaches of channel, both in pools and over bars. A further complication is the fact that the length of channel that is effective as a control is not constant, but increases with discharge.

From the preceding discussion it should be apparent that there is no really satisfactory substitute for discharge measurements in defining shifts in the channel-control segment of the rating; of particular importance are measurements made at or near the peak stage that occurs during periods of shifting control. However, in the usual situation a few (or less) measurements made at medium stages are the only ones available for analyzing channel-control shifts, and the shifts must be extrapolated to peak stages. The assumptions usually made in the rating analysis are those discussed below. The results are accepted unless they are shown to be invalid by a determination of peak discharge as described in chapter 9, or are shown to be invalid by use of one or more of the methods of rating-curve extrapolation as described in the section on "High-flow Extrapolation."

If a single predominantly large stream rise occurred shortly before the first measurement that indicated a shift, the shifts are assumed to have been caused solely by that rise. If more than one large stream rise occurred shortly before the first shift measurement, the shift curve may be prorated between rises. For example, if two rises of almost equal magnitude occurred just before the first shift measurement, and if the shift curve indicates a shift of 0.30 ft at a given stage, the shift to be used during the period between the two rises would be 0.15 ft at the given stage. It is often helpful to plot the shifts indicated by the discharge measurements against the observed stage of those measurements to obtain the trend of the shifts.

The pattern of scour and fill in the control channel determines whether the shift will increase with stage, decrease with stage, or be relatively constant at all stages. Figure 171 (graph *A*) illustrates a common situation where the shifts, either plus in the case of scour or minus in the case of fill, increase in absolute value as stage decreases. The highest value of the shift is assumed to be only slightly greater than the maximum value observed in order to avoid "overcorrecting" the original rating. Graph *B* of figure 171 shows the shift ratings corresponding to measurements nos. 1 and 2. The ratings have been plotted on rectangular-coordinate graph paper because the shifts are more easily visualized, at least by the inexperienced hydrographer, on that type of plotting paper. The stage-shift curve is usually plotted on rectangular-coordinate paper, but the rating curves are usually plotted on logarithmic graph paper. On logarithmic paper the shift curves in this example would converge more rapidly toward the original rating curve at high stages. The shift curves at low stages would be shaped to join smoothly with the shift curve for section control. The period for applying the shifts would be terminated on the stream rise following the last shift measurement; the original rating would be used on the recession from that rise.

In analyzing shifts there is no substitute for experience with a given stream because the shift pattern can often be interpreted logically in more than one way. For example, refer to the shift curve for channel fill in graph *B* of figure 171. Assume that measurements nos. 1 and 2 were made on a stream recession, and the measurement no. 1 was made a few days before measurement no. 2. Measurement no. 2 shows the effect of greater fill than measurement no. 1; fill usually occurs on a recession; therefore it is possible that the shifts should have been made to vary with time or to vary with time and stage, rather than with stage alone as shown in figure 171*A*. In the absence of additional knowledge the simplest interpretation is generally made, as was done here. Given more discharge measurements or a better knowledge of the behavior of the particular stream, a more accurate analysis can be made.

Figure 172 (graph *A*) illustrates a less common situation where the shifts increase as stage increases. Again the highest value of shift is assumed to be only slightly greater than the maximum value observed in order to prevent "overcorrecting" the original rating. Graph *B* of figure 172 shows the shift ratings corresponding to measurements nos. 1 and 2. The period for applying shifting-control corrections would be terminated on the stream rise following the last shift measurement; the original rating would be used on the rising limb of that rise. As in the case of figure 171, in the absence of additional knowledge, more than one interpretation can be given to

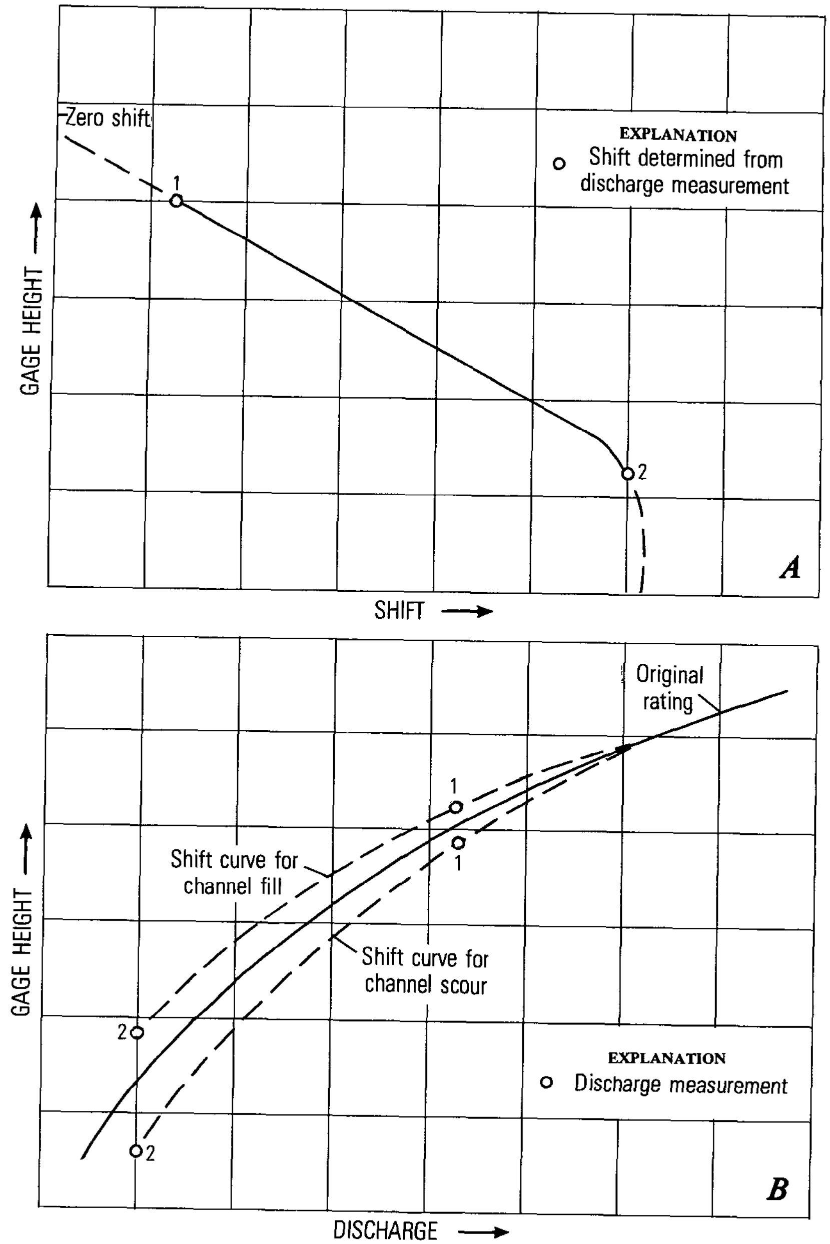


FIGURE 171.—First example of a stage-shift relation and the corresponding stage-discharge relation caused by scour or fill in the control channel.

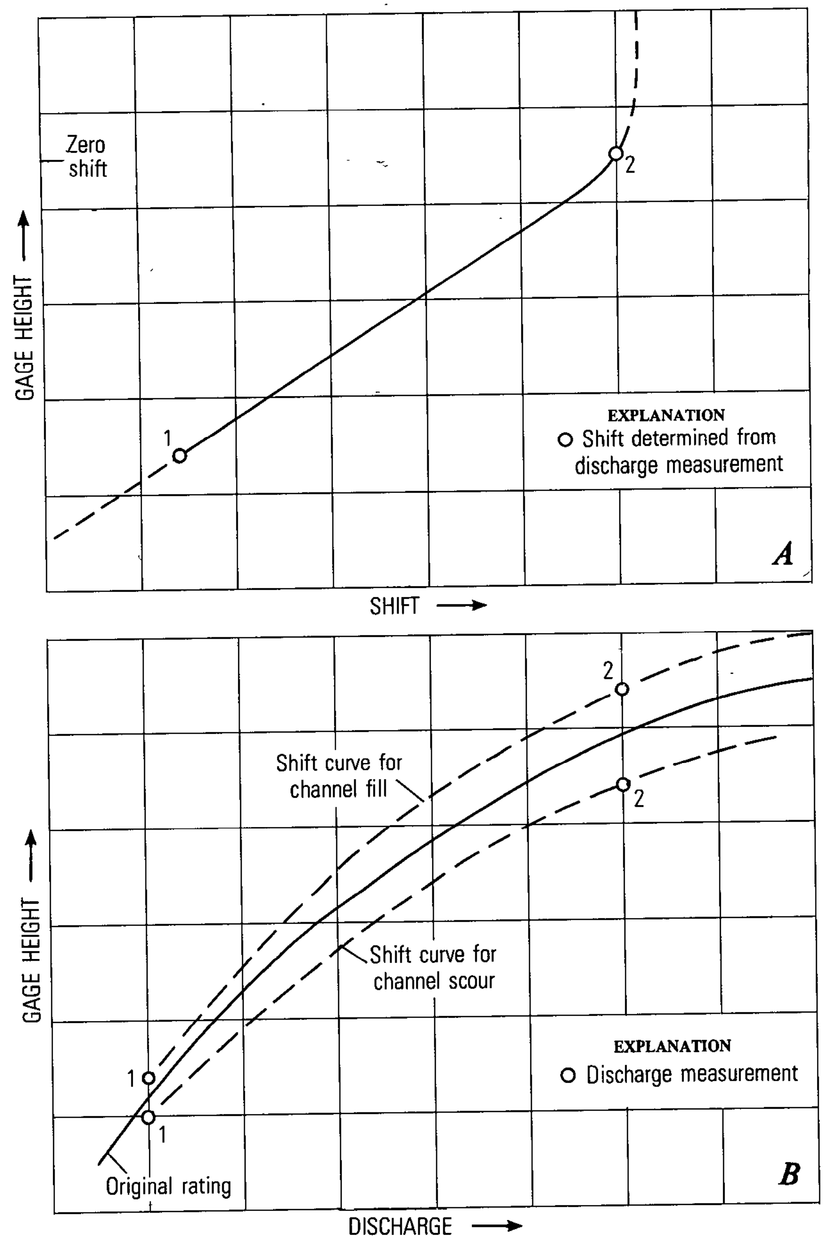


FIGURE 172.—Second example of a stage-shift relation and the corresponding stage-discharge relation caused by scour or fill in the control channel.

shifts shown by measurements nos. 1 and 2, depending on the relative times when the measurements were made and the fact that scour generally occurs on stream rises and fill generally occurs on stream recessions.

If there had been an additional major rise, one that occurred between the pairs of measurements shown in figures 171 and 172, other courses of action would be available. If the analyst had no additional data on which to base a judgment, he could assume that two separate shift events occurred, each attributable to the rise that preceded a discharge measurement. For each shift period, he could use a constant shift, equal to that shown by the discharge measurement made during that shift period. If, however, the analyst has had experience in the past with shifting control at the station caused by scour and fill in the control channel and if that experience had shown that shifts tend to vary with stage, another course of action would suggest itself. For each of the stage periods, the analyst could use a stage-shift relation of average shape that passed through the shift value shown by the appropriate discharge measurement. The above discussion would also apply to the situation of a single shift period and the availability of only a single discharge measurement made during that period. (It is assumed that the single discharge measurement would be accompanied by a check measurement to verify its accuracy, as discussed in the section on "Detection of Shifts in the Rating.")

If, during a period of shifting control, several measurements had been made but few of them could be fitted with a smooth shift curve, it would then be necessary to prorate the shifts with both time and stage, or possibly with time alone, based on the average shape of a stage-shift relation.

As mentioned earlier, scour in the control channel causes a plus shift because depth, and therefore discharge, is increased for a given gage height. Deposition or fill in the control channel causes a minus shift, because depth, and therefore discharge, is decreased for a given gage height. Thus the effect on the discharge of scour or fill in a channel control is opposite to that of scour and fill in a weir pool, which affects only the velocity of approach. Therefore, if a permanent weir is part of a compound control, scour in both the weir pool and in the channel control will cause a minus shift in the rating for section control and a plus shift in the rating for channel control. The converse is true when fill occurs in both the weir pool and the channel control. That situation is compatible with the stage-shift relation shown in figure 172, where a further decrease in stage would change the sign of the shifts. If the section control is a natural riffle, that riffle is likely to scour when the channel scours and fill when the channel fills, a situation that is compatible with the stage-shift relation shown in figure 171. In any event, the shift curves for low stages of channel control should be shaped to join smoothly with the shift curves for high stages of section control where a compound control exists.

Up to now the discussion of channel-control shifts has been confined to shifts caused by streambed scour and deposition. Shifts may also be caused by changes in the width of the channel. Even in a relatively stable channel the width of the channel may be increased during intense floods by widespread bank-cutting, and in some areas (for example, north coastal California) channel widths may be constricted by widespread landslides that occur when steep streambanks are undercut. In meandering streams changes in channel width occur as point bars are built up by deposition and later eroded by flood flows. The effect of a change in channel width on the stage-discharge relation, unaccompanied by a change in streambed elevation, is to change the discharge, for a given gage height, by a fixed percentage. When the original rating curve for channel control is plotted linearly on logarithmic graph paper, in accordance with the equation,

$$Q = p(G-e)^N, \tag{53}$$

the value of p increases with an increase in width and decreases with a decrease in width. The shift curve for a change in width alone will therefore plot on logarithmic graph paper as a straight line that is parallel to the original linear rating curve. Under those conditions a single discharge measurement is sufficient for constructing a shift curve for channel control.

When a change in channel width occurs concurrently with a change in streambed elevation, the effects of the two changes are compounded. The resulting shift curve is complex and requires at least several discharge measurements for its definition.

The growth of vegetation in a stream channel will affect the stage-discharge relation by reducing the discharge for a given gage height. The shift rating will therefore plot to the left (minus shift) of the original rating. The vegetation will increase the roughness coefficient of the channel and will tend to constrict the effective or unobstructed width of the channel. Both those factors reduce the value of p in equation 53 , and if the changes in roughness coefficient and effective width are unvarying with stage, the shift curve will be parallel to, and to the left of, the original rating curve that has been plotted linearly on logarithmic graph paper. Usually, however the changes are not independent of stage. If the growth consists of aquatic weeds, the weeds will be overtopped and bent over by high water; if the growth consists of alders and willows, the backwater effect will be greater at higher stages when the tree crowns as well as when the tree trunks are submerged. The rating shift caused by channel vegetation is, of course, variable with time as the growth spreads and increases in size.

EFFECT OF ICE FORMATION ON DISCHARGE RATINGS

GENERAL

The formation of ice in stream channels or on section controls affects the stage-discharge relation by causing backwater that varies in effect with the quantity and nature of the ice, as well as with the discharge. Because of the variability of the backwater effect, discharge measurements should be made as frequently as is feasible when the stream is under ice cover, particularly during periods of freeze-up and thaw when flow is highly variable. (Procedures for making measurements under ice cover are described in the section in chapter 5 titled, "Current-Meter Measurements from Ice Cover.") In midwinter the frequency of measurements will depend on climate, accessibility, size of stream, winter runoff characteristics, and required accuracy of the discharge record. As a general rule, two measurements per month is the recommended frequency. At stations below powerplants that carry a variable load, it may be necessary to make two measurements during each winter visit—one at the high stage of the regulated flow and the other at the low stage. The backwater effects may be markedly different at the two stages. In very cold climates where winter ice-cover persists and winter discharge shows a relatively smooth recession, fewer winter measurements are needed than in a climate that promotes the alternate freezing and thawing of river ice.

Knowledge of the three types of ice formation—frazil, anchor, and surface ice—and their possible effects, is helpful in analyzing streamflow records for ice-affected periods. With regard to the type of stage recorder that is preferred for use at ice-affected stations, the graphic recorder, described under that heading in Chapter 4, is by far the best because the recorder graph generally provides dependable evidence of the presence and type of ice formation.

FRAZIL

Frazil is ice in the form of fine elongated needles, thin sheets, or cubical crystals, formed at the surface of turbulent water, as at riffles. The turbulence prevents the ice crystals from coalescing to form sheet ice. The crystals may form in sufficient numbers to give the water a milky appearance. When the crystals float into slower water they come together to coalesce into masses of floating slush. When the current carries slush ice under a sheet of downstream surface ice, the slush may become attached to the underside of the surface ice, thereby increasing the effective depth of the surface ice. Most of the slush that adheres to the surface ice does so near the upstream end of the ice sheet.

Frazil or floating slush has no effect on the stage-discharge relation, but it may interfere with the operation of a current meter. It is particularly troublesome to operators of hydroelectric plants; by adhering and building up on trash racks the ice may effectively reduce the flow to the turbines.

ANCHOR ICE

Anchor ice is an accumulation of spongy ice or slush adhering to the rocks of a streambed. In former years the theory was held that the ice resulted from loss of heat by longwave radiation from streambed to outer space, because anchor ice generally formed on clear cold nights on the streambeds of open reaches of river. This theory has been shown to be invalid, because all of the long-wave radiation that can be lost from the bed of a stream at 0°C would be absorbed in less than 1 cm of water. Anchor ice is now commonly believed to be either (1) frazil that turbulent currents have carried to the streambed where the ice adhered to the rocks, or (2) ice that formed as the result of supercooled water finding nucleating agents on the streambed on which to crystallize. The ice crystals first formed on the rocks act as a nucleating agents for the continued growth of the ice mass.

Regardless of how anchor ice forms, it cannot form or exist when the rocks are warmed by shortwave radiation from the sun which penetrates the water. When the morning sun strikes anchor ice that had formed the night before and the streambed is warmed by the incoming solar radiation, the anchor ice is released and floats to the surface, often carrying small stones that it has picked up from the bed. For the next few hours the stream will be full of floating slush released in a similar manner upstream.

Anchor ice on the streambed or on the section control may build up the bed and (or) control to the extent that a higher than normal stage results from a given discharge. The solid-line graph in figure 173 shows a typical effect of anchor ice on a water-stage recorder graph. The rise starts in late evening or early morning, many hours after the sun has set, when ice begins to adhere to the rocks and raise the water level. By 10 a.m. the sun has warmed the streambed sufficiently to release the ice and the stage starts to fall. The distinguishing feature of the "anchor-ice hump" is that the rise is slow compared to the fall, whereas an actual increase in streamflow would occur in the opposite sequence, or at least the rise would be as rapid as the fall.

The small rises in actual discharge in the late afternoon, shown by the short-dashed lines in figure 173, probably result from water being released from channel storage when anchor ice upstream goes out. There may also be some runoff from the melting of snow and ice during the warmer part of the day.

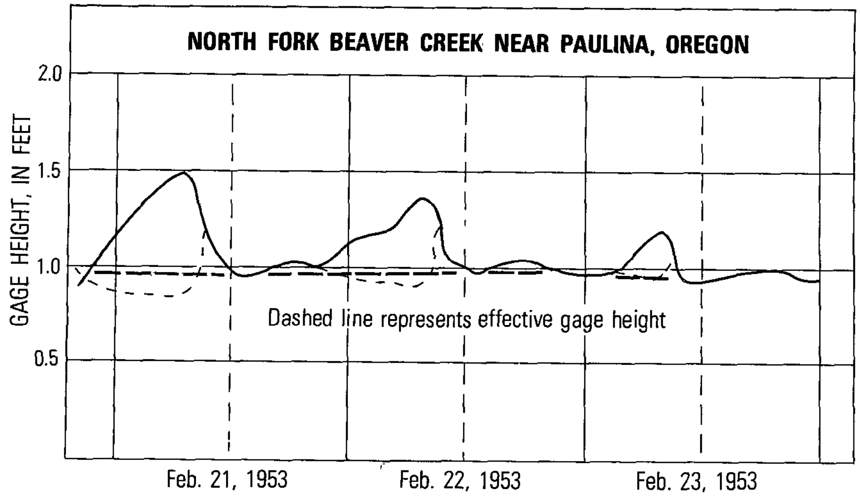


FIGURE 173.—Typical anchor-ice rises. (After Moore, 1957.)

SURFACE ICE

FORMATION OF ICE COVER

As the name implies, surface ice forms on the surface, first as a fringe of shore ice, which then, if the stream is not too turbulent, spreads to form a continuous ice cover spanning the stream from bank to bank. A description of the formation of surface ice follows.

With the onset of cold weather, the water in a stream is gradually cooled. Along the banks where the water is quiescent, temperature stratification occurs as in a lake. Because depths near the bank are usually very shallow, temperatures reach the freezing point more quickly there; ice crystals form and adhere to the banks, twigs, and projecting rocks, and a thin ice sheet forms. In the open part of the channel, temperature stratification is generally absent because of turbulent mixing, and the entire water body must reach 0°C before any freezing will occur. In the absence of nuclei or foreign material on which the ice crystals may form, there may be slight supercooling of the surface layer before any ice crystals are produced.

The ice sheet builds out from the shore as supercooled water, or water carrying ice crystals, impinges on the already-formed shore ice, and the transported or newly formed ice crystals adhere to the sheet. In the center of the stream, turbulence prevents coalescence of the ice crystals (frazil) that form. In the less turbulent areas, groups of crystals coalesce to form small pans of floating slush. These pans and (or) individual ice crystals are carried by the currents until they too impinge and adhere to existing ice sheets. In this manner an ice sheet finally forms across the entire stream. The ensuing increase in thickness of the ice sheet occurs almost entirely at the interface of ice and water.

On a fairly wide stream there is no great buildup of pressure as a result of the ice cover because the ice is, to a large degree, in floatation. Ice is weak in tension. If the stage rises or if the ice thickens considerably, the increased upward force of the water causes tension cracks to appear at the banks. The ice floats up to a position in equilibrium with the water, and water fills the tension cracks and freezes. The result is again a solid sheet in equilibrium with the river. If the stage drops, the unsupported weight of the ice again causes tension cracks, especially at the banks, and the ice drops to an equilibrium position with respect to the water. Water again fills the tension cracks, freezes, and again a solid sheet of ice results.

On narrow streams the ice may be in floatation, bridged, or under pressure. If the stream is so narrow or the ice so thick that the ice can resist the tensile stress placed on it by changes in stage, the ice will not change position regardless of change in stage. At high stages the stream, in effect, will be flowing in a pressure conduit; at low stages the ice sheet will be bridged so that it makes no contact with the water. This is particularly true when there are large boulders in the stream to which the ice is frozen, thereby reducing the length of the unsupported free span.

EFFECT OF SURFACE ICE ON STREAM HYDRAULICS

Surface ice when in contact with the stream may, in effect, change streamflow from open-channel flow to closed-conduit flow. Frictional resistance is increased because a water-ice interface replaces the water-air interface, hydraulic radius is decreased because of the additional wetted perimeter of the ice, and the cross-sectional area is decreased to a degree by the thickness of the ice. The stage will therefore increase for a given discharge. Figure 174 shows the water-stage recorder graph for a gaging station as the formation of surface ice begins to cause backwater effect. In this example, daily mean discharge remained about the same as before the freezeup, although the discharge undoubtedly fluctuated somewhat during each day. It can be seen from figure 174 that surface ice can cause much uncertainty regarding the discharge because the stage-discharge relation becomes indeterminate. It is evident in figure 174 that backwater effect exists and is increasing, because the rise looks very unnatural, but the amount of backwater effect cannot be determined directly from the recorder chart.

Surface ice can also cause siphon action when it forms on a section control, but that effect is not very common. In figure 175 when water filled the entire space between control and ice, siphon action began

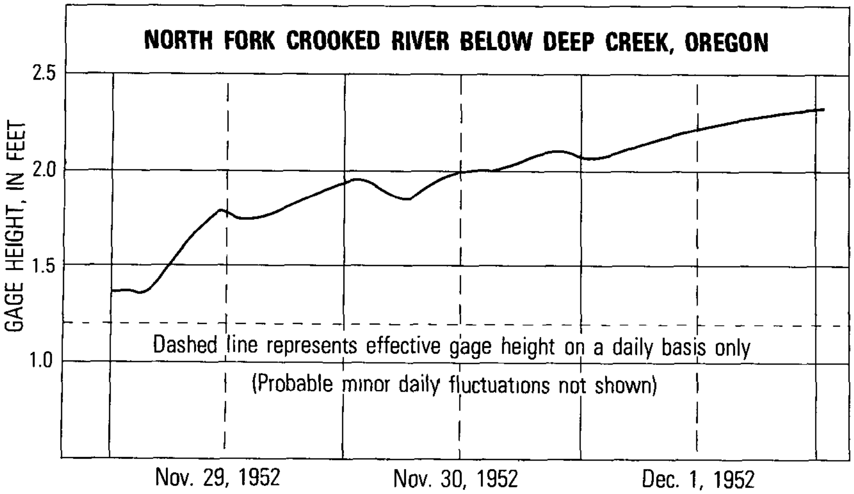


FIGURE 174.—Typical rise as complete ice cover forms. (After Moore, 1957.)

and water flowed over the control faster than it entered the gage pool. The gage pool was pulled down 0.3 ft below the point of zero flow before air entered the system and broke up the siphon action. Discharge ceased and then became a trickle while the inflow again filled the gage pool. When the entire space between control and ice was filled once more, siphon action began again. Siphon action is easily recognizable from the rapid fluctuations of the stage record. If the gaging station is visited at that time, the discharge measurement should be made far enough upstream from the gage pool to be beyond the effect of the fluctuating pool level.

If the section control is open and the gage is not too far removed from the control, there will probably be no backwater effect even though the entire pool is ice covered. The only effect of the ice cover will be to slow up the velocity of approach, and this effect will probably be minor. If the gage, however, is a considerable distance upstream from the riffle, surface ice on the pool may cause backwater as the covered reach of pool becomes a partial channel control.

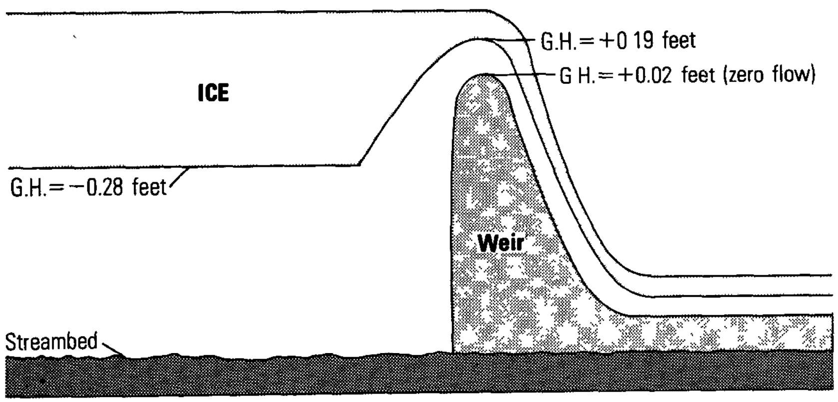
Ice forming below an open-section control may jam and raise the water level sufficiently to introduce backwater effect at the control.

COMPUTATION OF DISCHARGE DURING PERIODS OF BACKWATER FROM ANCHOR ICE

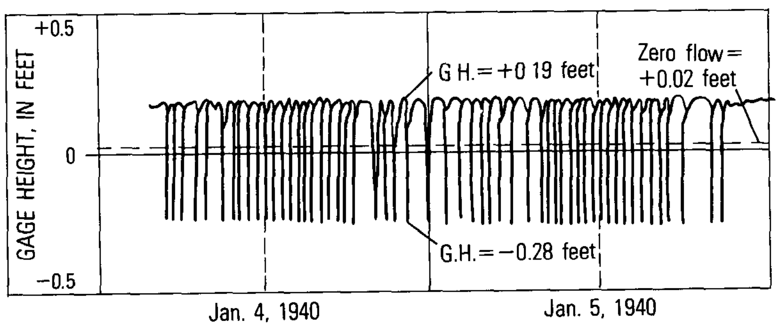
Discharge measurements are usually not made when anchor ice is present for the following reasons. First, adjustment of the stage record for the effect of anchor ice can be made quickly and reliably. Second, a discharge measurement made at that time is of little help in the analysis because discharge is highly variable with time as a result of water entering or leaving channel storage.

Anchor-ice rises are clearly recognizable on the recorder chart. In computing discharge for periods of anchor-ice effect, adjustments to gage-height are made directly on the gage-height graph. In figure 173 the long-dashed line connecting the low points of the "anchor-ice hump" is the effective gage height to use during the hours when the hump was recorded. Actually, the true effective gage height is shown by the short-dashed line. As the anchor ice builds up, the flow decreases faster than the normal recession shown by the long-dashed line, because some of the flow is going into storage as a result of the increased stage.

When the anchor ice goes out at about 9 or 10 a.m., a slug of water is released from storage and the true effective gage height rises. It can be seen however, that the areas formed by the short-dashed lines



Cross-sectional view of weir showing extent of ice cover, January 4–5, 1940



Gage height record for period January 4–5, 1940

FIGURE 175.—Effect of siphon action at artificial control in Sugar Run at Pymatuning, Pa., January 4–5, 1940.

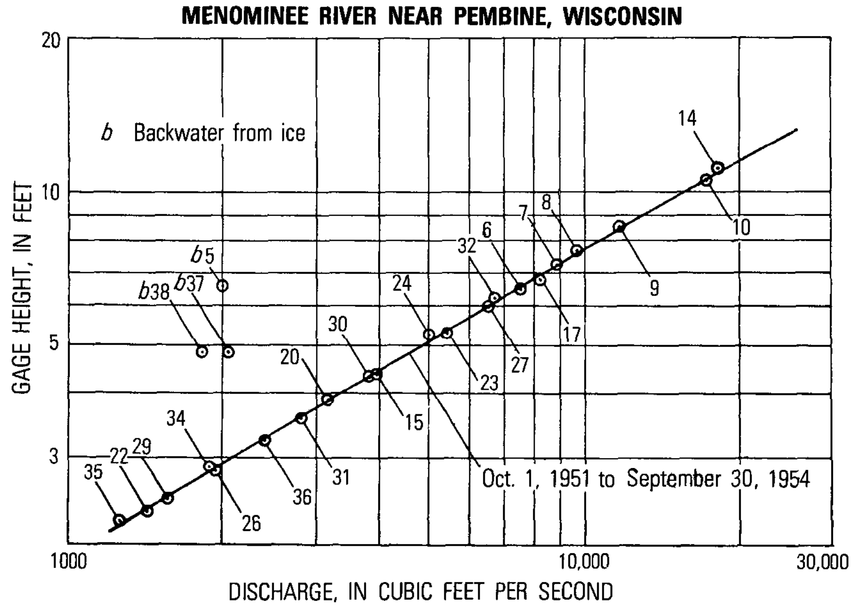


FIGURE 176.—Rating curve for Menominee River near Pembine, Wis. (After Moore, 1957.)

above and below the long-dashed line balance, and we would get identical daily mean values from use of either of the dashed lines. The rule then for obtaining effective gage height during anchor-ice periods is to cut off the hump with a straight line connecting the low points of the gage-height graph.

COMPUTATION OF DISCHARGE DURING PERIODS OF BACKWATER FROM SURFACE ICE

Figure 176 is an example of how discharge measurements (nos. 5, 37, 38), made during periods of ice effect, plot on a rating curve. Figure 174 is an example of a gage-height graph as complete ice cover forms. It is apparent from figure 174 that the backwater effect from surface ice cannot be determined directly from the recorder chart. The recorder chart is very helpful, however, in determining which periods during the winter are affected by ice. Complete notes describing ice conditions at the times the station was visited are also very valuable. Most important of all are discharge measurements made during ice-affected periods. A discharge measurement gives a definite point on a hydrograph plot of daily mean discharge versus date (fig. 177) through which the graph of estimated true daily discharge must pass. If little change in stage occurred during the day the discharge measurement was made, the measured discharge is considered to be the daily mean discharge. If a significant change in stage occurred that

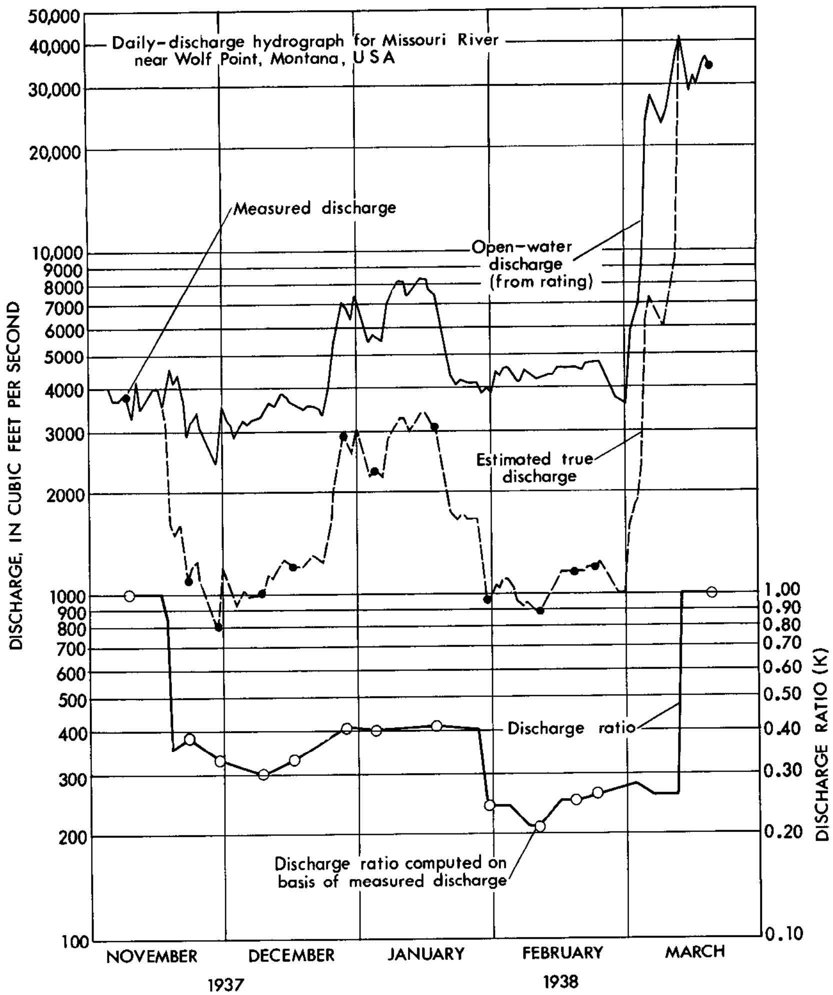


FIGURE 177.—Example of discharge-ratio method for correcting discharge record for ice effect.

day, the daily mean discharge (Q) is computed from the formula,

$$Q = Q_a \left(\frac{Q_m}{Q_r} \right), \tag{75}$$

where

Q_a is the discharge from the open-water (ice-free) rating curve corresponding to the daily mean gage height,

Q_m is the measured discharge and

Q_r is the discharge from the open-water rating curve corresponding to the gage height of the discharge measurement.

Three methods of correcting open-water discharge for ice effect are in use. (The term "open-water discharge", as used in this section of the manual, refers to the discharge for ice-free conditions obtained by applying the gage height record to the rating or shift curve that was in use immediately before the start of the ice-affected period.) The three methods are:

1. discharge-ratio method (sometimes known in the U.S.A. as the Lithuanian method),
2. shifting-control method or Stout method, and
3. hydrographic-and climatic-comparison method.

The reliability of each of the methods varies almost directly with the number of discharge measurements that were made during the ice-affected period that is being studied. Regardless of the method used, the corrected hydrograph of daily discharge, if possible, should be checked for consistency with other records. If the station being studied is on a stream that carries natural flow (flow not significantly affected by manmade development), its corrected record is compared with those for nearby streams that likewise carry natural flow. Particularly useful for that purpose are the hydrographs of streams that are unaffected by ice. If the station being studied is on a regulated stream, its corrected hydrograph is compared with the record of upstream reservoir releases or upstream hydroelectric generation, expressed either in units of discharge or in units of power output.

DISCHARGE-RATIO METHOD

In the discharge-ratio method which is used in many European countries, the open-water daily mean discharge is multiplied by a variable factor K to give the corrected discharge during periods of ice cover. A value of K is computed for each discharge measurement as the ratio of measured discharge (Q_m) to the open-water discharge (Q_r). Because K varies during the winter with time, as changes occur in the ice cover, the value of K for use on any given day is obtained by interpolation, on the basis of time, between K values computed for consecutive discharge measurements. Meteorological data are generally used to modify the simple interpolation between K values for consecutive discharge measurements; for example, during a period of extremely low temperatures the values of K indicated by simple interpolation would be reduced because the discharge usually decreases sharply at such times. The dates on which ice effect begins and ends are based on the observed or deduced beginning and end of ice cover.

An example of the discharge-ratio method is shown in figure 177. Note that discharge is plotted on a logarithmic scale. The upper daily hydrograph shows open-water discharges and the solid circles are discharge measurements; the lower graph shows the *K* values obtained from discharge measurements (open circles) and the interpolation between those values; the middle graph is the hydrograph of estimated true daily discharges, obtained by multiplying concurrent values from the upper and lower graphs. The nonlinear interpolations for *K* values during the periods November 9–23, January 18 to February 19, and February 24 to March 20, were based on the observer's notes concerning ice conditions and on temperature and precipitation records (not shown in fig. 177).

SHIFTING-CONTROL METHOD

The shifting-control method, at one time the standard method used in the U.S.A., is seldom used here now, but it is still used in other countries. In the shifting-control method, recorded gage heights are reduced by a variable backwater value to obtain the effective daily gage heights. The effective gage heights are then applied to the open-water rating to obtain estimated true daily discharges. The backwater correction on days when discharge measurements are made is computed as the difference between the actual gage height and the effective gage height—effective gage height being the gage height from the open-water rating that corresponds to the measured discharge. The backwater correction for use on any given day is obtained by interpolation, on the basis of time, between the backwater corrections computed for consecutive discharge measurements. As in the discharge-ratio method, the interpolation is subject to modification on the basis of meteorological records, and the dates on which ice effect begins and ends are based on the observed or deduced beginning and end of ice cover.

An example of the shifting-control method is shown in figure 178. The method is applied to the same gaging station used in the example in figure 177. Note that a natural (not logarithmic) scale is used in figure 178. The upper daily hydrograph in figure 178 shows recorded gage heights and the solid circles are the effective gage heights for discharge measurements; the lower graph shows the backwater corrections obtained from discharge measurements (open circles) and the interpolation between those values; the middle graph is the hydrograph of effective gage height, obtained by subtracting values on the lower graph from concurrent values on the upper graph. The nonlinear interpolations for backwater corrections during various periods were based on the observer's notes concerning ice conditions and on temperature and precipitation records (not shown in fig. 178). As mentioned in the preceding paragraph, the effective gage heights (middle graph) are applied to the rating curve to obtain estimated true daily discharges.

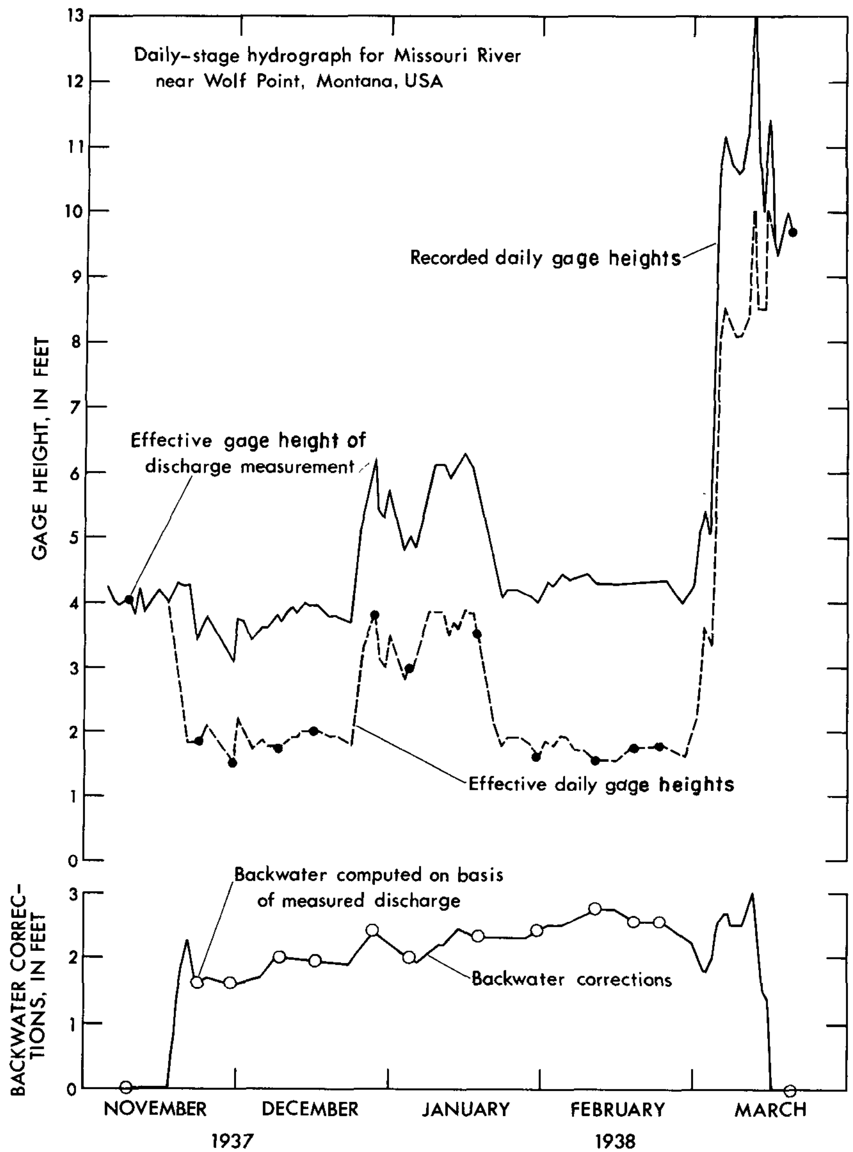


FIGURE 178.—Example of shifting-control method for adjusting stage record for ice effect.

HYDROGRAPHIC- AND CLIMATIC-COMPARISON METHOD

The method of hydrographic and climatic comparison has been favored in the U.S.A. for the last 30 years. The mechanics of the method differ from those of the discharge-ratio method, but both methods basically correct the daily open-water discharge by a variable percentage.

The first step is to compute the station discharge record for the entire year as though there were no ice effect at any time. The daily hydrograph of open-water discharge and the discharge measurements are then plotted, using a logarithmic discharge scale, and notes concerning ice conditions are entered on the graph. At this point the hydrograph sheet resembles the upper graph in figure 177. If a measurement of ice-affected discharge is not representative of the daily mean discharge because of changing stage during the day, the daily mean discharge, as computed by equation 75, is also plotted. All is then in readiness for estimating the true daily discharge directly on the hydrograph sheet, and that is done on the basis of three comparisons:

1. comparison with records for nearby gaging stations,
2. comparison with weather records, and
3. comparison with the base-flow recession curve for the gaging station that is being studied.

COMPARISON WITH RECORDS FOR NEARBY GAGING STATIONS

Comparison with other discharge records is the most important basis for determining the probable discharge for periods between discharge measurements. Even though the record used for comparison may also have been corrected for ice effect, its use provides an additional independent set of basic data—another stage record and another set of current-meter measurements. Without a nearby record that compares well with the record being studied, the accuracy of the daily discharges estimated between the dates of discharge measurements may be greatly reduced. However, hydrographic comparisons are not infallible because the relation between the flow of two streams may vary significantly during the year; hence the importance of making many discharge measurements during ice-affected periods.

In making the hydrographic comparison, the nearby station with the most reliable winter streamflow record is selected for use as a reference station. The reliability of the reference station may have been established by the fact that its discharge is unaffected by ice or is affected by ice for only a relatively short period, or by the fact that many winter measurements have been made at the station and the true discharge between the dates of measurement can be estimated from weather records. (See discussion below on use of weather records.) A hydrograph of daily discharge, corrected for ice effect if necessary, is prepared for the reference station on a separate sheet of graph paper, similar to that used for plotting the daily hydrograph for the station being studied.

A light table is used in comparing the two hydrographs. A light table is a glass-topped table that is illuminated by light from below the glass top, so that when one hydrograph is superposed on the other on the table top, both hydrographs can be viewed simultaneously. The hydrograph for the reference station is taped to the top of the light table. The hydrograph for the study station is then superposed on that of the reference station and positioned laterally so that the date lines of the two hydrographs coincide. The period preceding the first measurement (no. 1) that showed ice effect at the study station is the period first selected for consideration. The hydrograph for the study station is positioned vertically so that hydrographs for the two stations roughly coincide for the period immediately preceding the day or days when the start of ice effect is suspected. A comparison of the hydrographs and an inspection of the weather records should fix the date when ice effect started. That date will be preceded by a period of subfreezing weather, and on that date—usually a rainless day—the hydrograph for the study station will start a gradual rise not shown by the hydrograph for the reference station. For an appreciable period thereafter the hydrograph for the study station will remain above that of the reference station.

After the starting date (A) of ice-effect at the study station has been selected, the vertical position of the hydrograph for the study station is changed slightly, if necessary, to make the two hydrographs coincide on that date. If that positioning causes measurement no. 1 to fall directly on the hydrograph for the reference station, the hydrograph for the reference station between date A and measurement no. 1 is traced with dashed lines on the hydrograph sheet for the study station. The daily discharges indicated by the dashed lines are the estimated true discharges at the study station during the period between date A and measurement no. 1.

However, only rarely does measurement no. 1 coincide with the reference hydrograph when discharges at the two stations are made to coincide on date A; measurement no. 1 will usually lie above or below the hydrograph for the reference station. In that situation, as discharges from the reference hydrograph are being transferred to the sheet bearing the study hydrograph, the study sheet will in effect be moved up or down, as the case may be, so that when the transfer of discharge points reaches measurement no. 1, measurement no. 1 will coincide exactly with the reference hydrograph. If the temperature record shows no great fluctuation from day to day during the period between date A and measurement no. 1, the vertical displacement of the sheet bearing the study hydrograph will be made uniformly during the transfer process. If the temperature record does fluctuate from day to day during the period, the vertical displacement will be made at a variable rate to reflect the fact that the ratio of true discharge to open-water discharge usually decreases during sharp drops in temperature; the ratio increases during sharp rises in temperature. In other words, the vertical distance between open-water discharge and true discharge increases on the study-hydrograph sheet during sharp drops in temperature; the vertical distance decreases during sharp rises in temperature. Observer's notes concerning major changes in the ice cover, particularly where complete cover is intermittent during the winter, are also very helpful in estimating the degree of ice effect.

After correcting the discharge between date *A* and measurement no. 1, the process is repeated for the period between discharge measurement no. 1 and the next successive discharge measurement (no. 2). The two hydrographs are made to coincide at measurement no. 1 and the transfer of discharge points to the study hydrograph proceeds to measurement no. 2. In that manner the open-water discharge for the study station is corrected until the date is reached when ice effect ceases.

Comparison with Weather Records

Records of air temperature and precipitation are a most valuable aid in making corrections for ice effect. The temperature record helps the engineer decide whether the precipitation is rain or snow—snow will have no immediate effect on the runoff. The temperature record also helps the engineer decide whether ice cover is forming, increasing, or dissipating. For stations for which there are no nearby discharge records for comparison and for which the recorder chart does not provide dependable clues to the fluctuation of discharge, it may be necessary to correct open-water discharges for ice effect almost solely on the basis of weather records and available measurements of discharge. Discharge usually follows closely the "ups-and-downs" of the air temperature record, and the discharge measurements help fix, within reasonable limits, the estimated rises and falls of the "true" discharge hydrograph. An exception to that statement is found in regions of extreme cold, such as the Arctic, that become blanketed with a heavy snow cover. The snow acts as an insulator for the underlying ground, and it then requires a prolonged change in temperature to significantly change the slow uniform recession of streamflow during the winter.

It should be mentioned here that a water-temperature recorder is a helpful adjunct to a gaging station. When the water temperature is above the freezing level, there is little likelihood of ice effect.

Comparison with Base-Flow Recession Curves

During periods of subfreezing weather, virtually all the flow in a stream is base flow; that is, water that comes out of ground-water storage to sustain the flow of the stream during periods when there is no surface runoff. It will often be found that during cold ice-affected periods, the flow of the stream will be declining at a rate similar to the rate of recession shown by that stream during ice-free periods. Thus if we have a known discharge of say, 20 ft^3/s, on some day during the ice-affected period and we wish to estimate daily discharge during the next 10 days, all of which were free of rain or snowmelt, we look for an ice-free period elsewhere in the record for the study station when there was no surface runoff, and choose a day whose discharge is 20 ft^3/s. We then note the receding values of discharge for the following 10 days, and use those same discharges for the 10 days to be estimated. The ice-free period that is used for an index should preferably be in the nongrowing season because the use of water by vegetation affects the rate of base-flow recession.

It is possible that daily discharges estimated from the base-flow recession for a warmer period may be somewhat high because extremely cold weather reduces the rate at which water percolates through the ground, and because some of the water that does reach the stream may go into storage behind ice dams. Nevertheless a standard base-flow recession curve provides a valuable guide to the probable flow during recession periods when the stream is ice-covered. Because the discharge during periods of base flow originates as ground water, a record of the fluctuations of ground-water levels of wells in the area can be useful as an index for estimating the true discharge during those periods.

An example of the application of the hydrographic- and climatic-comparisons method is illustrated in figures 179 and 180. Figure 179 shows a portion of a plotted hydrograph of daily mean discharge for the gaging station on North Fork John Day River at Monument, Oreg. The solid line represents open-water discharge obtained by applying recorded gage heights to the rating curve, and the X on January 26 represents the open-water discharge corresponding to the gage height of discharge measurement C made on that date. The open-water discharge is almost 10 times as great as the measured discharge on January 26. The dashed line on figure 179 represents the estimated true daily discharge obtained by comparison with the hydrograph of daily mean discharge for John Day River at Service Creek, Oreg. and by comparison with the record of daily maximum and minimum temperature at Dayville, Oreg. The reference hydrograph and temperature record used for the comparison are shown in figure 180. Actually the precipitation record at Dayville was also

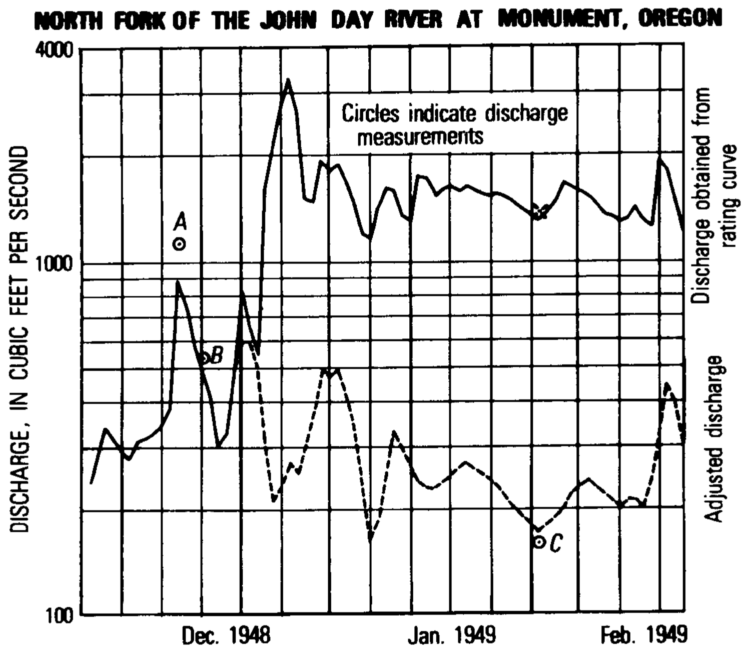


FIGURE 179.—Daily hydrographs for open-water discharge and for discharge corrected for ice effect. (After Moore, 1957.)

considered, but because all precipitation during the study period occurred as snow and therefore had no immediate effect on the runoff, the precipitation record is not shown in figure 180.

Also shown on figure 180 is the corrected hydrograph for the study station on North Fork John Day River at Monument; the hydrograph of open-water discharge at that station has been omitted to reduce clutter in the illustration. The discharge for the reference station on John Day River at Service Creek was unaffected by ice. The shapes of the two hydrographs are not identical, but useful comparison between the hydrographs for two stations does not require that their shapes be identical, as long as their discharge trends are similar. It can be seen on figure 180 that both hydrographs respond to the effect of air-temperature fluctuations during the winter period.

In applying the method of hydrographic and climatic comparison, the hydrograph of "true" daily discharge, plotted on a logarithmic scale, was displaced from the open-water hydrograph by a variable vertical distance. That means, in effect, that discharge ratios, variable with time, were applied to the open-water discharges, and there-

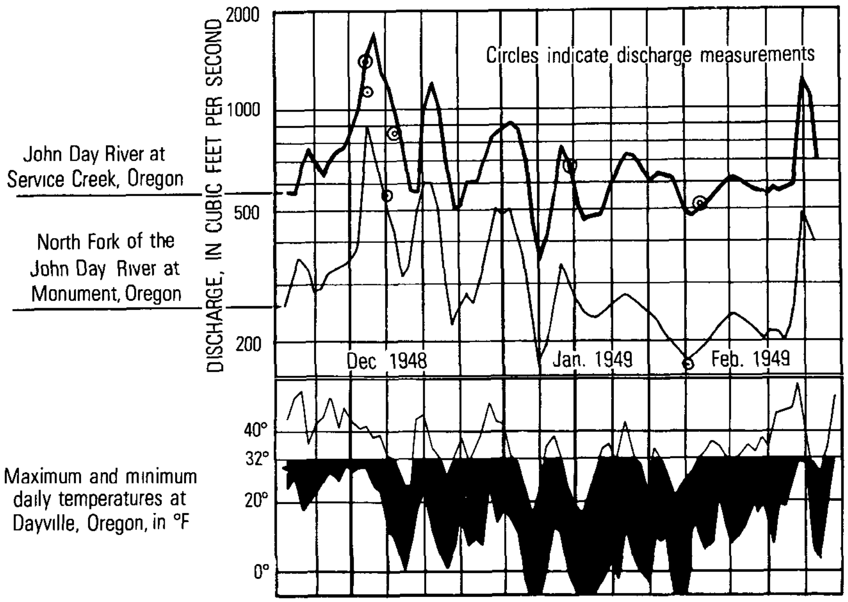


FIGURE 180.—Comparison of daily winter discharge at two gaging stations showing their response to air-temperature fluctuations. (After Moore, 1957.)

fore a basic similarity exists between the hydrographic-comparison method and the discharge-ratio method. It appears to the author that application of the hydrographic-comparison method would be greatly facilitated if the hydrograph of open-water discharge for the study station were first adjusted by the discharge-ratio method because application of that method is relatively simple. The adjusted hydrograph would then be refined by using it, rather than the open-water hydrograph, in the hydrographic-comparison method. It is much simpler to apply the hydrographic-comparison method for refining discharge estimates than it is to apply that method for making original discharge estimates.

SAND-CHANNEL STREAMS

In fixed channels, well-defined stage-discharge relations can usually be developed that show only minor shifting at low flow. In sand-channel streams, however, stage-discharge relations are continually changing with time because of scour and fill and because of changes in the configuration of the channel bed. These changes cause the shape and position of the stage-discharge relation to vary from time to time and from flood to flood, and it becomes very difficult to explain the apparent haphazard scatter of discharge measurements available to define the rating. Familiarity with the results of research studies as reported by Colby (1960), Dawdy (1961), Simons and Richardson (1962), Beckman and Furness (1962), and Culbertson and Dawdy (1964) will greatly assist the analyst in defining the discharge rating.

For a stream with rigid boundaries, the best site for a stream-gaging station is upstream from a constriction because the constriction will provide a stable and sensitive control. An opposite effect occurs, however, at a constriction on a sand-channel stream; the rating will be unstable there because the constricted section will experience maximum streambed scour and fill. In fact, any contracting reach on a sand-channel stream is undesirable for use as a gaging-station site, and a straight uniform reach should be sought. Preferably both the gage and the cableway site for high-water discharge measurements should be located in a reach suitable for the determination of peak discharge by the slope-area method (chap. 9). This will permit the use of high-water current-meter measurements to verify computed peak discharges as well as develop the hydraulics of the stage-discharge relation. The fieldwork for a sand-channel stream should also include the collection of samples of bed materials at the stream-gaging site.

BED CONFIGURATION

On the basis of laboratory investigation, Simons and Richardson (1962) described the bed configuration of sand-channel streams as ripples, dunes, plane bed, standing waves, and antidunes. This sequence of bed configurations occurs with increasing discharge. When the dunes wash out, and the sand is rearranged to form a plane bed, there is a marked decrease in resistance to flow which may result in an abrupt discontinuity in the stage-discharge relation. The forms of bed roughness, as shown in figure 181 and described in table 21, are grouped according to the two separate conditions of depth-discharge relationship that are evident in a given channel. The sequence of configurations described in table 21 is developed by continually increasing discharge. The lower regime occurs with lower discharges; the upper regime with higher discharges; an unstable discontinuity in the depth-discharge relationship appears between these two more stable regimes.

The presence of fine sediment in the flow influences the configuration of the sand bed and thus the resistance to flow. It has been found by Simons and Richardson (1962, p. 4) that with concentrations on the order of 40,000 milligrams per liter of fine material, resistance to flow in the dune range is reduced as much as 40 percent. The effect is less pronounced in the upper regime, but fine sediment may change a standing-wave condition into a breaking antidune which will increase the resistance to flow. Thus the stage-discharge relation for a

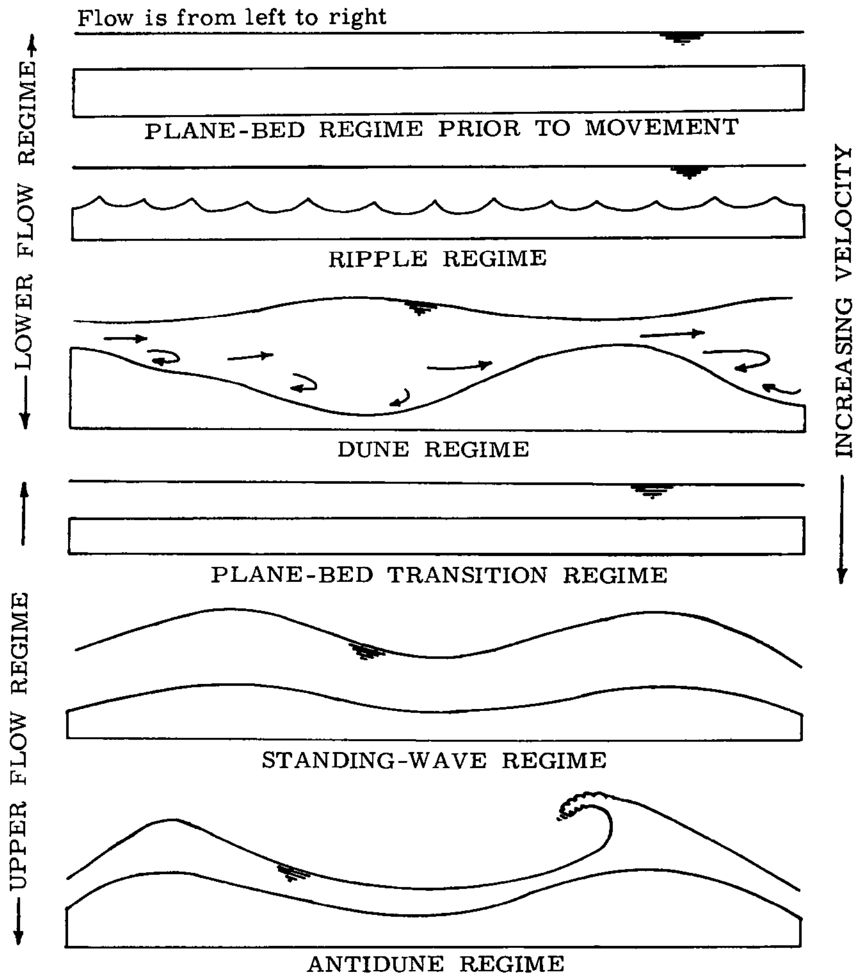


FIGURE 181.—Idealized diagram of bed and water-surface configuration of alluvial streams for various regimes of flow.

stream may vary with sediment concentration if the flow is heavily laden with fine sediment.

Changes in temperature can also alter the form of bed roughness, and, hence, the resistance to flow. Lowering the temperature increases the viscosity of the water and increases the mobility of the sand. If, for example, the form of bed roughness is in transition or nearly so, and if there is a reduction in the temperature of the water, the increased mobility of the sand may cause the dunes to wash out and the bed to become plane. This phenomenon is reversible.

Changes in bed forms do not occur instantaneously with increasing or decreasing discharge. The time lag between change in bed form

TABLE 21.—*Surface and bed descriptions for the various flow regimes*

Type of configuration	Description	
	Bed	Flow
Lower regime flow:		
Plane bed	Plane; no sediment movement.	Plane surface; little turbulence.
Ripples	Small uniform waves; no sediment movement.	Plane surface; little turbulence.
Dunes	Large, irregular, sawtoothed waves formed by sediment moving downstream; waves move slowly downstream.	Very turbulent; large boils.
Upper regime of flow:		
Plane bed	Dunes smoothed out to plane bed.	Plane surface; little turbulence.
Standing waves	Smooth sinusoidal waves in fixed position.	Standing sinusoidal waves in phase with bed waves; termed "sand waves."
Antidunes	Symmetrical sinusoidal waves progressing upstream and increasing in amplitude; suddenly collapse into suspension then gradually reform.	Symmetrical sand waves progressing upstream in phase with bed waves; amplitude increases until wave breaks, whole system collapses then gradually reforms.

and change in discharge may result in loop rating curves. For example, if bed configuration is initially dunes, the dunes will persist on rising stages to a discharge that is greater than the discharge at which the dunes will reform on falling stages. Thus at a given stage, the discharge may be greater when the stage is falling. Because the form of each loop curve depends on the initial condition of bed configuration and the rate of change of discharge, an infinite number of different loop curves, and even multiple-loop curves, may occur for a given reach of channel across the transition from dunes to plane bed. The stage-discharge relation within the transition band may be indeterminate. An example of a loop curve, typical of some channels, is shown in figure 182.

RELATION OF MEAN DEPTH TO DISCHARGE

A plot of stage against discharge in sand-channel streams often obscures any underlying hydraulic relationship because neither the bottom nor sides of these streams are fixed. Figure 183 shows as an extreme example the stage-discharge plot for Huerfano River near Undercliffe, Colo., for 1941 and 1942. The relation between stage and discharge is indeterminate. However, the underlying hydraulic relation may be revealed by a change in variables. The effect of variation in bottom elevation is eliminated by replacing stage by mean depth or hydraulic radius. The effect of variation in width is eliminated by

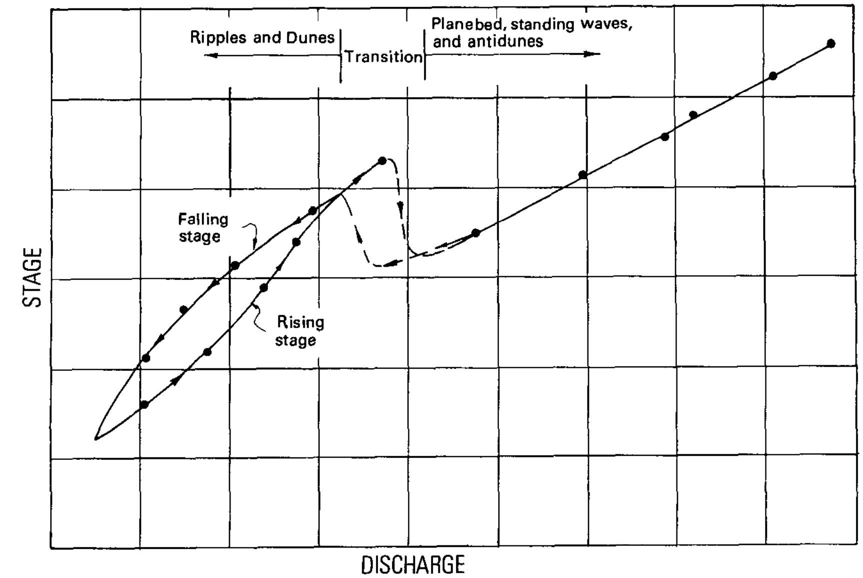


FIGURE 182.—Typical loop curve of stage versus discharge for a single flood event in a sand channel. (After Stepanich, Simons, and Richardson, 1964.)

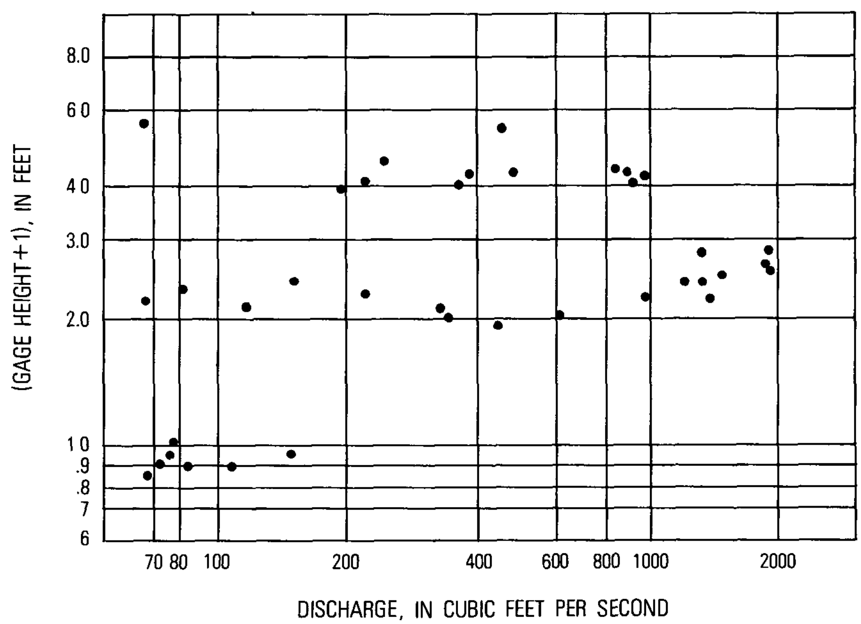


FIGURE 183.—Stage-discharge relation for Huerfano River near Undercliffe, Colo.

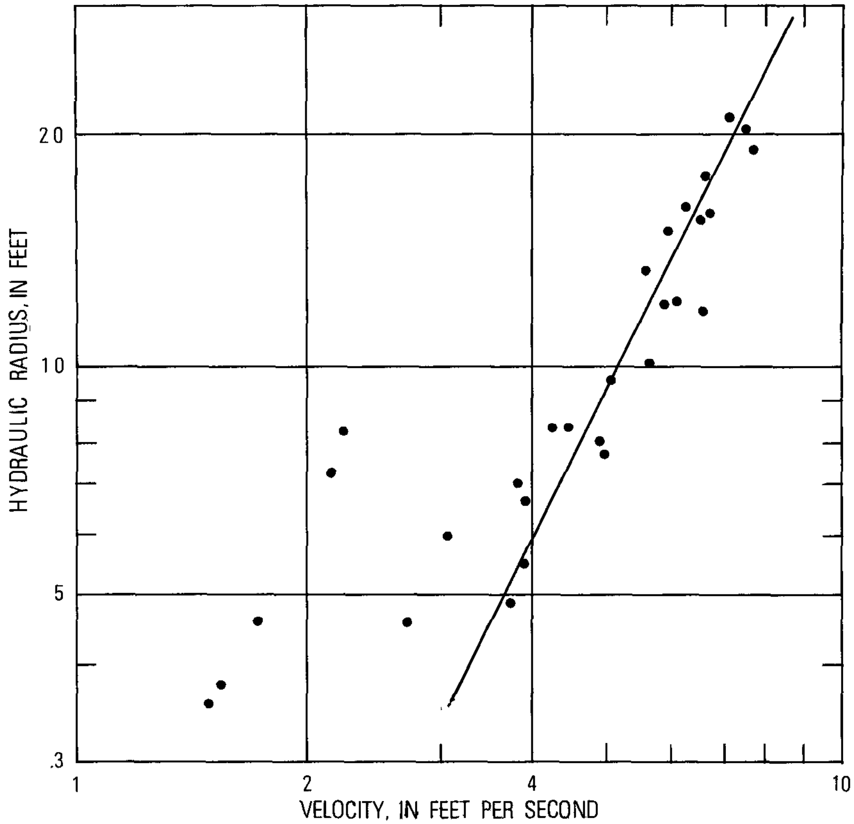


FIGURE 184.—Relation of velocity to hydraulic radius for Huerfano River near Undercliffe, Colo.

using mean velocity. Figure 184 shows most of the same measurements for Huerfano River that were plotted in figure 183, now replotted on the basis of velocity and hydraulic radius. Measurements for this stream with a hydraulic radius greater than one foot define a single curve with bed forms corresponding to the upper regime. Measurements in the transition range from dunes to plane bed scatter wildly as would be expected from the previous discussion.

The discontinuity in the depth-discharge relation is further illustrated in figure 185 which shows a plot of hydraulic radius against velocity for Rio Grande near Bernalillo, N. Mex. The measurements plotted on the left represent bed configurations of ripples and dunes and the curve on the right represents bed configurations of plane bed, standing waves, or antidunes.

According to Dawdy (1961), the curve representing the upper regime in a true sand-bed stream usually fits the following relation,

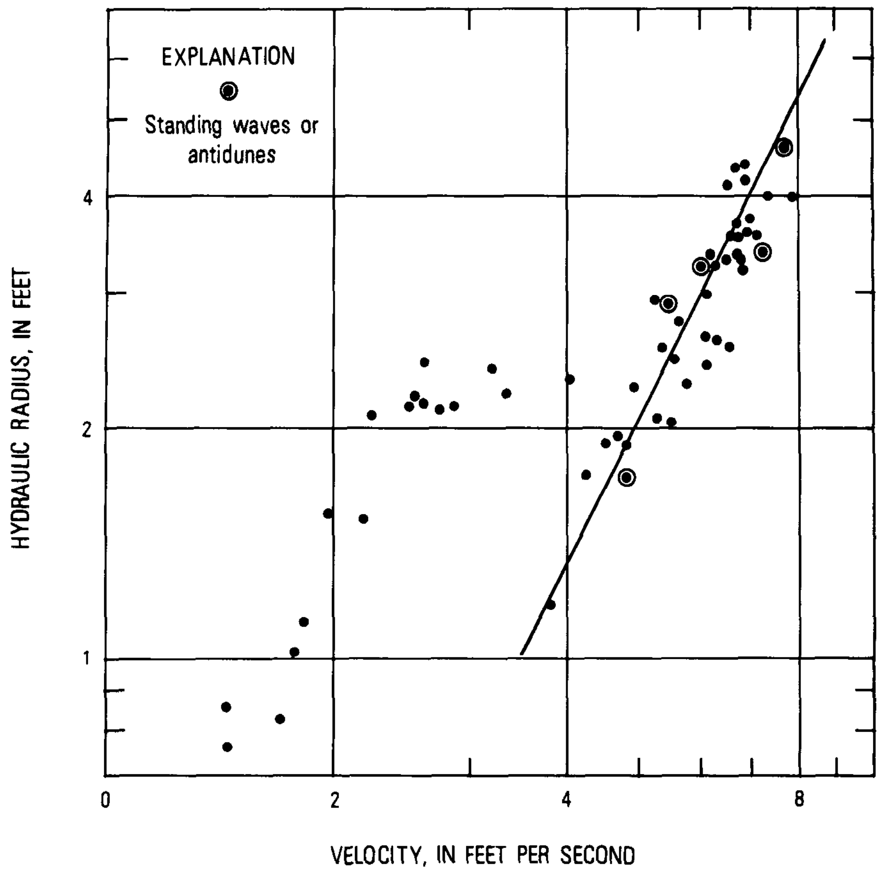


FIGURE 185.—Relation of velocity to hydraulic radius for Rio Grande near Bernalillo, N. Mex.

$$V = kR^{1/2},$$

where V is the mean velocity, k is a constant, and R is the hydraulic radius. He found this relation to be applicable for 26 of the 27 streams used in his study. More recent study has shown that the exponent of R ranges from ⅔, as in the Manning equation, to ½, the larger exponents being associated with the coarser grain sizes.

DEVELOPMENT OF DISCHARGE RATING

Plots of mean depth or hydraulic radius against mean velocity or discharge per foot of width are valuable in the analysis of stage-discharge relations. These plots clearly identify the regimes of bed configuration and assist in the identification of the conditions represented by individual discharge measurements. For example, only

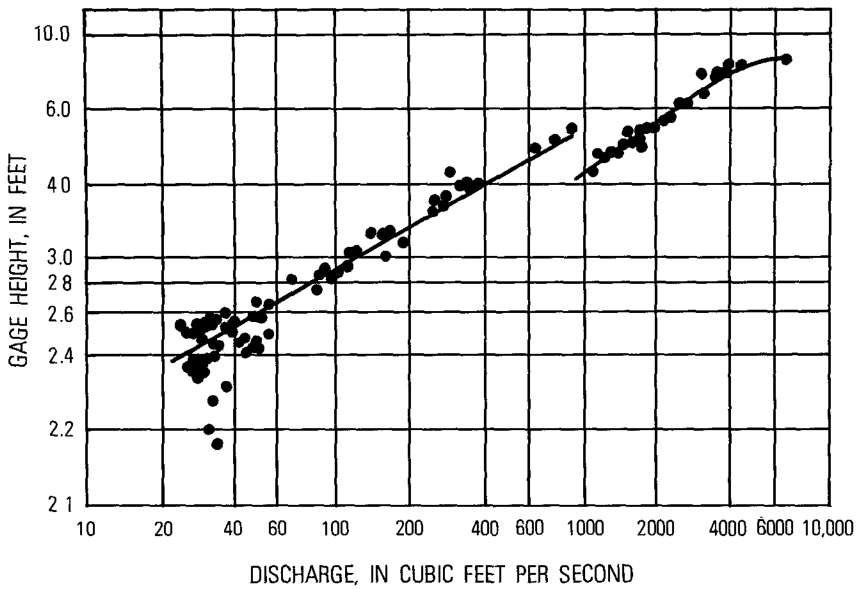


FIGURE 186.—Stage-discharge relation for station 34 on Pigeon Roost Creek, Miss. (After Colby, 1960.)

those measurements identified with the upper regime should be used to define the position and slope of the upper portion of the stage-discharge curve; similarly, only those measurements identified with the lower regime should be used to define the lower portion of the stage-discharge curve. Measurements made in the transition zone may be expected to scatter widely but do not necessarily represent shifts in more stable portions of the rating.

Plots of stage against mean depth and stage against width are also helpful in developing a mean stage-discharge relation and in analyzing the cause of shifts from the mean relation. In the upper regime the use of these plots in conjunction with the plot of velocity versus mean depth or hydraulic radius raised to the ½ to ⅔ power, depending on grain size, may be useful in establishing a reasonable slope to the upper part of the stage-discharge relation.

The stage-discharge relation developed by Colby (1960) for Pigeon Roost Creek, Miss., is shown in figure 186. This stream is about 75 ft wide, the banks are relatively stable, and the median size of the bed material is 0.4 mm. The mean elevation of the channel bed does not change appreciably with time or discharge. The discontinuity in the stage-discharge relation is very abrupt. Discharges from 900 to 1,800 ft^3/s may occur at a stage of 5.3 ft.

According to Colby (1960, p. 19, 20), stage-discharge relations may be expected to have a discontinuity if the reach has all of the following characteristics:

1. a bed of uniform and readily shifting sediment that does not form distinct pools and riffles,
2. at some flows almost all of the stream-bed is covered with loose sand dunes,
3. at higher flows the bed of the stream is mostly plane or has antidunes,
4. the depth of flow at the point of discontinuity is sufficiently great so that changes in the stage-discharge relation at the discontinuity can be distinguished from changes caused by small local shifts of the channel bottom, and
5. the lateral distribution of depths and velocities is sufficiently uniform for the bed configuration to change across most of the streambed in a relatively short time.

The above conditions are very restrictive. Many streams with sand beds have well developed pools and riffles at the stage where the discontinuity might otherwise occur. Streams do not generally have uniform sediment sizes; many have large sorting coefficients. A few streams having suitable bed material may never show the discontinuity because dunes exist even at the highest flow rates. Others may have such high slopes that the lower regime cannot be defined by discharge measurements because of the shallow depths at which the discontinuity occurs. Winding streams seldom have uniform lateral distribution of velocity and depths. Some streams have such gradual or inconsistent transitions between dunes and plane bed that the discontinuity may be difficult, if not impossible, to define. Dunes may exist near the banks at the same time that a plane bed exists near the center of the stream. The transition in this case may occur so gradually with increasing stage that the discontinuity in rating is eliminated. However, at any station where dunes exist at low flows and a plane bed exists at higher flows, there is a major change in bed roughness. Knowledge of the bed forms that exist at each stage or discharge can be very helpful in developing the discharge rating.

EVIDENCES OF BED FORMS

Evidence of the bed forms that exist at a given time at a particular station can be obtained in several ways, a listing of which follows.

1. Visual observation of the water surface will reveal one of several conditions: large boils or eddies, which indicate dunes; a very smooth water surface, which indicates a plane bed; standing waves, which indicate smooth bed waves in phase with the surface waves; or breaking waves, which indicate antidunes. Visual observations of the water surface should be recorded on each discharge measurement.

2. Noting whether the sand in the bed is soft or firm. A soft bed often indicates lower regime conditions. The streambed during upper regime flow will usually be firm.

3. Measurements of bed elevations in a cross section will usually indicate the type of bed forms. A large variation in depth indicates dunes, and a small variation in depth a plane bed. The small variation in depths for a plane-bed (upper-regime) configuration should not be confused with small variations caused by ripples or by small dunes, both of which are definitely lower-regime configurations. A large variation in bed elevation at a particular point in the cross section during a series of discharge measurements indicates the movement of dunes.

4. The amount of surge on a recorder chart may also indicate the configuration of the channel bed. Medium surge may indicate dunes, little or no surge may indicate a plane bed, and violent surge may indicate standing waves or antidunes. The transition from plane bed to dunes during a discharge recession may cause a secondary hump on the gage-height trace if the transition occurs over a short time period.

5. Relations that define the occurrence of bed forms as a function of hydraulic radius (R), slope (S), mean velocity (V), and grain size (d), are useful in developing discharge ratings. A relation of that type, presented by Simons and Richardson (1962), is shown in figure 187. In that figure the dimension of R is feet; that of V is feet per second. Recent studies suggest that: the lower regime of bed forms will occur when the ratio,

$$\frac{V^4}{g^2 D^{1/2} d_{50}{}^{3/2}}, \tag{76}$$

is less than 1×10^3; the upper regime of bed forms will occur when the ratio is greater than 4×10^3; the bed will be in transition if the ratio is between those values. In the above ratio, V is the mean velocity in feet per second, g is the acceleration of gravity in feet per second per second, D is the mean depth in feet, and d_{50} is the median grain size of bed material in feet.

SHIFTING CONTROLS

The upper part of the stage-discharge relation is relatively stable if it represents the upper regime of bed forms. Rating shifts that occur in upper-regime flow can be analyzed in accordance with the methods or principles discussed in the section titled, "Rating Shifts for Channel Control." However, the shift ratings after minor stream rises will generally have a strong tendency to parallel the base rating when plotted on rectangular-coordinate graph paper; that is, the equation

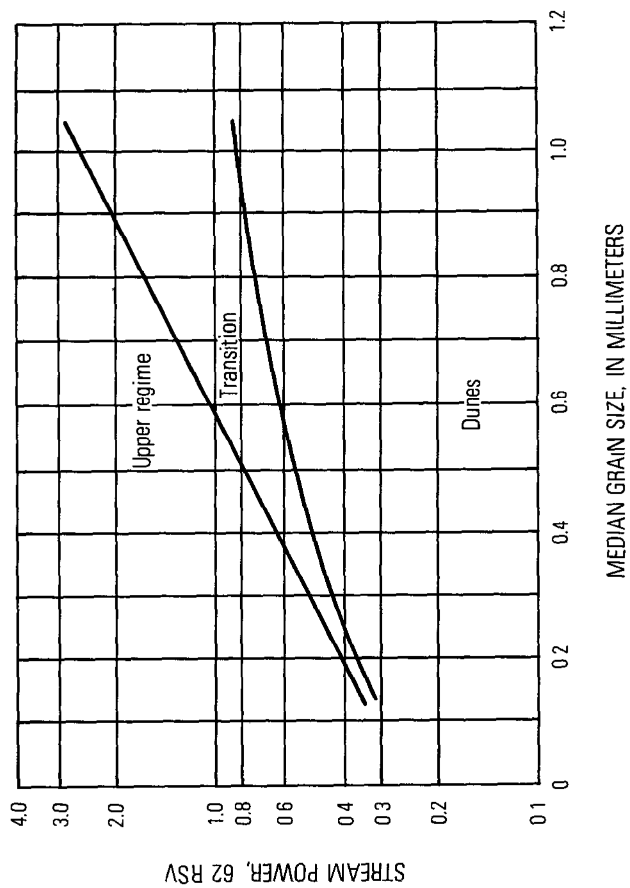


FIGURE 187.—Relation of stream power and median grain size to form of bed roughness.

for each shift curve will differ from that of the base rating by a change in the value of e in the basic equation,

$$Q = p(G-e)^N. \qquad (53)$$

The shifts will change on stream rises and will often vary with time between rises. Major stream rises may also change the value of p in equation 53.

The lower part of the rating is usually in the dune regime and the stage-discharge relation varies almost randomly with time. Frequent discharge measurements are necessary to define the stage-discharge relation, and for some streams they are necessary to determine the variation of discharge with time in the absence of any usable relation between stage and discharge. In the U.S.A. a frequency of three discharge measurements per week is often recommended, but for some streams, even daily measurements barely suffice.

A mean curve for the lower regime is frequently used with shifts as defined by discharge measurements. In some instances the shift defined by a single discharge measurement represents only the temporary position of a dune moving over a partial section control. A series of discharge measurements made at short time intervals over the period of a day may define a pattern of shifts caused by dune movement. When discharge is constant but the stage fluctuates, the changing gage-height trace generally reflects dune movement.

Continuous definition of the stage-discharge relation in a sand channel stream at low flow is a very difficult problem. The installation of a control structure should be considered if at all feasible.

ARTIFICIAL CONTROLS FOR SAND CHANNELS

When conventional controls are installed in sand channels, they are seldom satisfactory, even those designed to be self-cleaning. The principal difficulty is that for such controls in a sand channel, discharge is dependent not only on water-surface elevation, but also on the bed elevation and flow regime upstream from the structure. A satisfactory control is one whose stage-discharge relation is unaffected by bed configuration. A few successful low-water controls have been designed for use in sand channels; one example is the weir designed for the gaging station on the Rio Grande conveyance channel near Bernardo, N. Mex. (Richardson and Harris, 1962). That structure will not be described here because generalizations concerning control shape are meaningless; each control structure must be individually designed for compatibility with channel and flow conditions that exist at the proposed site for the control. A laboratory model study involving a reach of channel is therefore needed for each site investigated. Efforts continue to design low-water controls that are both relatively cheap and that have satisfactory operating characteristics when installed in sand channels (Stepanich and others, 1964).

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CHAPTER 11—DISCHARGE RATINGS USING SLOPE AS A PARAMETER

GENERAL CONSIDERATIONS

If variable backwater or highly unsteady flow exists at a gaging station, the energy slope is variable at a given stage and the discharge rating cannot be defined by stage alone.

Variable backwater is most commonly caused by variable stage at a downstream confluence for a given discharge upstream or by the manipulation of gates at a downstream dam. The discharge under those conditions is a function of both stage and slope of the energy gradient. If the rate of change of stage is sufficiently great, the acceleration head must also be considered, but this chapter deals only with situations where the acceleration head has insignificant effect and can be neglected.

The unsteady-flow situation treated in this chapter is that of a natural flood wave, in which the flow maintains a stable wave profile as it moves down the channel. That type of wave is known as a uniformly progressive wave, and it often produces a loop rating at the gaging station; that is, for a given stage the discharge is greater when the stream is rising than it is when the stream is falling. The difference between the two discharges is significant only when the flow is highly unsteady. The term "highly unsteady", when associated only with the property of producing loop ratings, is a relative term, because channel slope is of equal importance in determining whether or not loop ratings will occur. A flood wave in a steep mountain channel will have a simple stage-discharge relation; that same flood wave in a flat valley channel may have a loop rating. The sections of this chapter that deal with unsteady flow are concerned only with loop ratings whose definition requires the use of slope, as well as stage, in a relation with discharge.

When a new gaging station is established, the need for a slope parameter in the rating can often be anticipated from the rating procedures used for existing stations nearby in a similar hydrologic and hydraulic environment. At other times the need for a slope parameter is not as evident. However, a plot of a series of discharge measurements made at medium and high stages will indicate the type of rating required for the station and will dictate whether or not an auxiliary gage is necessary to continuously measure water-surface slope.

If a pair of gages is needed, the locations of the base and auxiliary gage are based on the characteristics of the slope reach. The length of the reach should be such that ordinary errors that occur in the determination of gage heights at stage stations will cause no more than minor error in computing the fall in the reach. A fall of about 0.5 ft (0.15 m) is desirable but satisfactory records can often be obtained in reaches where the minimum fall is considerably less than 0.5 ft. Channel slope in the reach should be as uniform as possible. The reach should be as far upstream from the source of backwater as is practicable, and inflow between the two gages should be negligible. If possible, reaches with frequent or appreciable overbank flow should be avoided, as should reaches with sharp bends or unstable channel conditions. If the reach includes a natural control for low stages, the upstream (base) gage should be located just upstream from that control so that a simple stage-discharge relation will apply at low stages. Rarely will a slope reach be found that has all of the above attributes, but they should be considered in making a selection from the reaches that are available for slope measurement.

THEORETICAL CONSIDERATIONS

Variable slopes that affect flow in open channels are caused by variable backwater, by changing discharge, or by variable backwater in conjunction with changing discharge. The pair of differential equations given below provides a general solution to both gradually varied and unsteady flow.

$$\frac{Q^2}{K^2} = -\frac{\partial H}{\partial x} - \frac{1}{g}\frac{\partial V}{\partial t} \tag{77}$$

$$\frac{\partial Q}{\partial x} = -B\frac{\partial h}{\partial t} \tag{77a}$$

In the equations Q is the discharge, K is the conveyance of the cross section, H is the total energy head, x the distance along the channel, g the acceleration of gravity, V the mean velocity, t the time, B the top width of the channel, and h is the water-surface elevation. A solution to these equations in uniform channels may be obtained by approximate step methods after the conveyance term has been evaluated by discharge measurements.

In those practical problems of determining flow in open channels that require application of equation 77 the increment of slope due to the acceleration head $\frac{1}{g}\frac{\partial V}{\partial t}$ is, in general, so small with respect to the other two terms that its effect may be neglected. Thus, in equation 77 the terms that remain in addition to discharge (Q), are conveyance (K) which is a function of stage, and energy gradient ($\partial H/\partial x$) which is related to water-surface slope. At those sites where tidal action or variation in power production cause the acceleration head to be large, approximate methods of integration of equations 77 and 77a are used in conjunction with an electronic computer. Those methods are described briefly in chapter 13 of this manual.

The discussion of stage-fall-discharge ratings presented in the present chapter draws heavily on previously published reports. The three primary references used are Corbett and others (1945), Eisenlohr (1964), and Mitchell (1954).

VARIABLE SLOPE CAUSED BY VARIABLE BACKWATER

The stage at a gaging station for a given discharge, under the usual subcritical flow conditions, is influenced by downstream control elements. A brief discussion of those elements is now in order.

Previous discussions of controls in this manual have dealt primarily with such elements as natural riffles, weirs and dams, flumes, and the physical properties of the stream channel. It had also been explained that a control may act independently for some range of stage or it may act in concert with one or more other controls. However, it had also been mentioned in appropriate places in this manual that the stage at downstream stream confluences may affect the stage-discharge relation at a gaging station. Where that occurs, the confluent stream must be classed as a control element that acts in concert (partial control) with the control(s) in the gaged stream. Furthermore, when a confluent stream acts as a control element, it usually does so as a variable element. That is, the stage at the gaging station will no longer be related solely to the discharge of the main stream, but will also vary with variation of the discharge in the confluent stream.

At gaging stations on tide-affected streams, the tide itself must be considered as a variable control element because of its effect on the stage-discharge relation at the gaging station. As mentioned earlier tide-affected stage-discharge relations are treated in chapter 13.

A less clear-cut situation with regard to control elements exists in many streams in southeastern United States. These streams have extremely wide flood plains that are crossed in places by highway embankments whose bridge openings locally constrict the flow severely. At high flow if water occupies the flood plain, the stage-discharge relation at the bridge is affected; for a given discharge through the bridge the corresponding stage will vary, depending on whether streamflow is entering the overbank areas as on a rising stage, or whether water is returning to the main channel from the overbank areas as on a falling stage. In that situation the overbank flow itself is acting as a variable control element in concert with the "more conventional" and more stable control elements, such the geometry of the bridge opening and the geometry and roughness of the downstream main channel and overbank areas. The streamflow that is entering the overbank areas acts, in effect, as an extremely wide downstream distributary; the overbank flow that is returning to the stream acts, in effect, as an extremely wide downstream tributary. The streams usually have extremely flat gradients and the rating may possibly be complicated by the effect of changing discharge on streams of flat slope. However, as explained in the section titled, "Variable Slope Caused by a Combination of Variable Backwater and Changing Discharge," streams affected by both variable backwater and changing discharge are treated as though they were affected by variable backwater alone.

The control elements that affect the stage-discharge relation for a stream have now been identified and their descriptions have been amplified for the discussion of backwater that follows. At any given discharge the effect on the stage at the gaging station that is attributable to the operative control element(s) is known as backwater. As long as the control elements are unvarying, the backwater for a given discharge is unvarying, and the discharge is a function of stage only; the slope of the water surface at that stage is also unvarying. If some of the control elements are variable—for example, movable gates at a downstream dam or the varying stage at a downstream stream confluence—for any given discharge the stage at the station and the slope are likewise variable. In a preceeding discussion titled "Theoretical Considerations," it was demonstrated that for the above variable conditions, discharge can be related to stage and slope. Because the slope between two fixed points is measured by the fall between those points, it is more convenient to express discharge as a function of stage and fall.

Stage-fall-discharge ratings are usually determined empirically for observations of (1) discharge, (2) stage at the base gage, which is usually the upstream gage, and (3) the fall of the water surface between the base gage and an auxiliary gage. The general procedure used in developing the ratings is as follows:

1. A base relation between stage and discharge for uniform flow or for a fixed backwater condition is developed from the observations. The discharge from that relation is termed Q_r.
2. The corresponding relation between stages and the falls for conditions of uniform flow or fixed backwater is developed. Those falls are termed rating falls, F_r. Figure 188 shows schematically three forms the stage-fall relation may have.
3. The ratios of discharges Q_m, measured under conditions of variable backwater, to Q_r, are correlated with the ratios of the measured falls F_m to the rating falls F_r. Thus,

$$\frac{Q_m}{Q_r} = f\left(\frac{F_m}{F_r}\right). \tag{78}$$

The form of the relation depends primarily on the channel features that control the stage-discharge relation. The relation commonly takes the form,

$$\frac{Q_m}{Q_r} = \left(\frac{F_m}{F_r}\right)^N, \tag{78a}$$

where N varies from 0.4 to 0.6, the theoretical value of N being 0.5. Generally speaking, the stage-fall-discharge rating can be extrapolated with more confidence when the data are such that they fit equation 78a best when an N value of 0.5 is used.

The fall between the base and auxiliary gage sites, as determined from recorded stages at the two gages, may not provide a true representation of the slope of the water surface between the two sites. That situation may result from the channel and gaging conditions that are described below.

First, the water surface in any reach affected by backwater is not a plane surface between points in the reach, as sinuosity of the channel

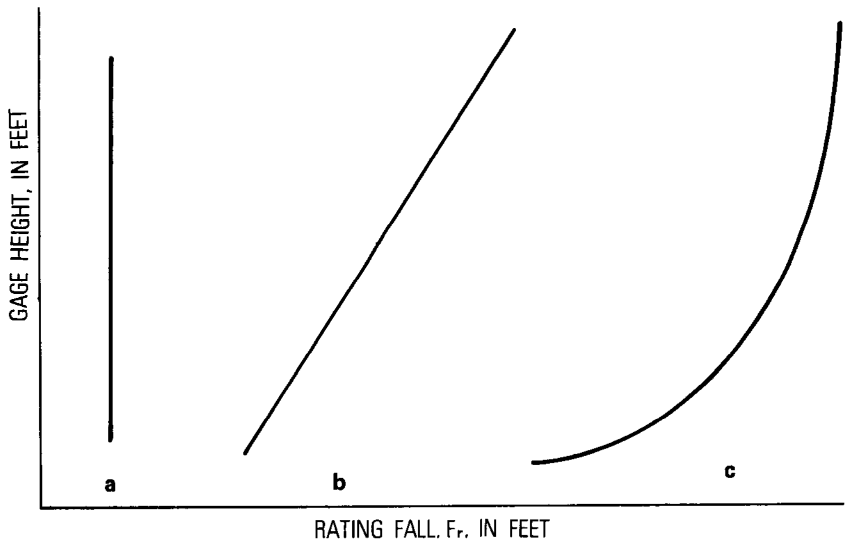

FIGURE 188.—Schematic representation of typical stage-fall relations. Curve (a), rating fall constant; curve (b), rating fall a linear function of stage; curve (c), rating fall a curvilinear function of stage.

will produce variations in the height of the water surface, both across and along the reach; variations in channel cross section and the effects of backwater also tend to produce curvature of the water surface. The slope determined from observed differences in stages is that of a chord connecting the water-surface elevations at points at the ends of a reach. It may not represent the slope of the water surface at either end of the reach but may be parallel to a line that is tangent to the water surface at some point in the reach.

Second, no reach of a natural stream selected for the determination of slope is completely uniform. The area of the cross section may vary considerably from point to point in the reach, but more important is the effect that shoals, riffles, rapids, or bends in the stream channel within the reach may have on the slope of the water surface, as well as on the energy gradient.

Third, the positions of the gages at the ends of the reach with respect to the physical features of the channel may have a material effect on the recorded gage heights and hence on the indicated slope. For example, if one gage is on the inside of a rather sharp bend and the other on the outside of a similar bend, the slope computed from records of stages at those gages may be widely different from the average slope of the water surface. Also, if differing drawdown effects exist at the intakes of the two gages, the two stage records obtained may not provide a true index of the water-surface slope.

Fourth, both gages may not be set to exactly the same datum, the difference in datum possibly being a large percentage of the total fall if the fall is small. The slope determined from gages not set to the same datum would not indicate the true water-surface slope because the computed slope would include the quantity y/L, where y is the difference in datum and L is the length of the reach.

Because of those conditions, theoretical relations between stage, fall, and discharge cannot be directly applied, and the relations must be empirically defined by discharge measurements made throughout the range of backwater conditions. Thus, the "best" value of the exponent of F_m/F_r in equation 78a will often be found to be in the range from 0.4 to 0.6, rather than having the theoretical value of 0.5; or, it may even be necessary to depart from a pure exponential curve in order to fit the plotted points satisfactorily. At other times the substitution of a term, $F+y$, for F values in equation 78a will improve the discharge relation. The use of a constant, y, whose best value is determined by trial computations, compensates in part for the inaccuracies in the value of F that were discussed above.

It is convenient to classify stage-fall-discharge ratings according to the types of relation that may be developed between stage and rating fall. The two types are:

1. *Rating fall constant.*—This type of relation (curve *a* in fig. 188) may be developed for channels that tend to be uniform in nature and for which the water-surface profile between gages does not have appreciable curvature.

2. *Rating fall a function of stage.*—This type of relation (curves *b* and *c* in fig. 188) may be developed if any of the following conditions exist:

a. appreciable curvature occurs in the water-surface profile between gages;
b. the reach is nonuniform;
c. a submerged section control exists in the reach between gages, but the control does not become completely drowned by channel control even at high discharges; and
d. a combination of some of the conditions listed above.

It is not uncommon for variable backwater to be effective for only part of the time. That follows from the two general principles that apply to backwater effect. The first states that for a given stage at the variable control element, backwater decreases at the base gage as discharge increases. For example, in a long gage reach of fairly steep slope, a given stage at the variable control element may cause significant backwater at the base gage when the discharge in the gaged stream is low but cause no backwater during periods when the discharge is high. The second principle states that for a given discharge, backwater decreases at the base gage as stage decreases at the variable control element. For example, at a given discharge in the gaged stream a high stage at the variable control element may cause significant backwater at the base gage, but a low stage at the variable control element may cause no backwater.

Other basic principles and detailed procedures used in defining stage-fall-discharge ratings are discussed on the pages that follow. The discussions are arranged in accordance with the preceding classification of stage-rating fall relations. A knowledge of the hydraulic principles applicable to a given slope reach is essential as a guide to the empirical analysis of the data.

RATING FALL CONSTANT

GENERAL DISCUSSION OF RATING PRINCIPLES

In uniform channels the water-surface profile is parallel to the bed; the slope, and therefore the fall, is the same for all discharges. The rating fall, F_r, for the condition of no variable backwater (uniform-flow conditions) would be the same at any stage. The stage-discharge relation with no backwater could be described by the Chezy equation,

$$Q_0 = CA_0\sqrt{R_0 S_0},$$

where the subscripts denote uniform flow; or by the equation,

$$Q_r = CA\sqrt{RF_r/L}, \tag{79}$$

where the subscripts denote the base rating conditions.

If variable backwater is imposed on the reach by a downstream tributary, the measured fall, F_m, and measured discharge, Q_m, would be less at a given stage than indicated by the uniform-flow rating. If the slope or fall as measured truly represents the slope at the base gage, those measurements would define, as shown in figure 189, a family of stage-discharge curves, each for a constant but different value of fall. The relation of each curve in the family to the curve for base rating conditions according to equation 79, is expressed by the equation,

$$\frac{Q}{Q_r} = \sqrt{\frac{F}{F_r}} \tag{80}$$

The discharge under variable backwater conditions may be computed as the product of (a) the discharge Q_r from the base rating and (b) the

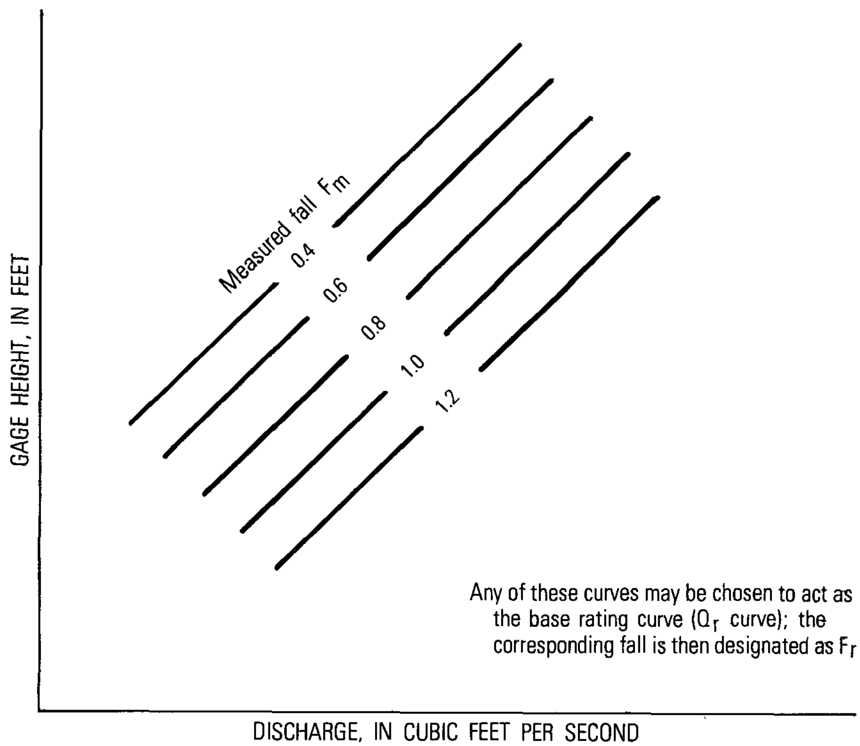

FIGURE 189.—Schematic representation of family of stage-discharge curves, each for a constant but different value of fall.

square root of the ratio of the measured fall to the constant-value rating fall.

A constant rating fall may also exist at sites where the base rating is controlled by a dam downstream from the reach in which fall is measured. If the curvature in the backwater profile is not significant, and if the channel is uniform, the water-surface profile will approximately parallel the channel-bed profile at all discharges. For example, the curve in figure 189 for a constant fall of 1.2 ft may be taken to represent the base stage-discharge relation for a fixed or stable control element. The curve for lesser falls that might result from variable submergence of the dam, are theoretically related to this base curve by the square root of the fall ratios, as described above. Quite commonly a constant value of 1.0 ft is used for F_r in equation 80. That special case of the constant rating-fall method, usually referred to as the unit-fall method, simplifies the computations because equation 80 then reduces to

$$Q_r = Q/\sqrt{F}. \tag{81}$$

A constant rating fall is not the usual case encountered in natural streams. However, if discharge measurements cover the entire range of flow conditions and if such measurements conform to a constant rating fall, there is no need to use a more complicated technique. If profile curvature and velocity-head increments are truly negligible, the relation between the discharge ratio and fall ratio should resolve into a single curve; otherwise the relation may be a family of curves with stage as a third variable.

PROCEDURE FOR ESTABLISHING THE RATING

The general procedure used in establishing a stage-fall-discharge rating with constant rating fall is outlined as follows:

1. Plot all discharge measurements using stages at the base gage as ordinates and discharges as abscissas, and note the measured fall (F_m) beside each plotted point. If the information on this plot indicates a family of curves, each corresponding to a constant value of fall (fig. 189), the use of a constant rating fall should be investigated.
2. The most satisfactory type of constant-fall rating, from the standpoint of high-water extrapolation, is one whose discharge ratio-fall ratio relation is a pure parabolic relation, as in equation 80, with the exponent equal to, or nearly equal to, 0.5. If such a relation fits the measured discharges, the results are unaffected by whatever value of constant fall (F_r) is used. For convenience, unit fall is used, as in equation 81.

3. For each discharge measurement (Q_m), compute Q_r by use of the equation $Q_r = Q_m/(F_m)^{0.5}$.

4. Plot values of gage height versus Q_r for each discharge measurement and fit a curve to the plotted points to obtain the Q_r discharges from the Q_r rating curve.

5. Compute and tabulate the percentage departures of the plotted Q_r discharges from the Q_r rating curve.

6. Repeat steps 3–5, using exponents of F_m other than, but close to, 0.5. Try exponents equal to 0.40, 0.45, 0.55, and 0.60.

7. Compare the five Q_r rating curves and select the curve that best fits the plotted points used to define it. In steps 8 and 9 that follow, the discharges from that "best" rating curve will be referred to as Q_{rd}, and the corresponding exponent of F_m will be referred to as d.

8. If the plotted discharges closely fit the Q_{rd} rating curve, that curve and the relation of (Q_m/Q_{rd}) to F_m are accepted for use.

9. If the plotted discharges do not closely fit the Q_{rd} rating curve repeat steps 3–5, using the exponent d but substituting the term (F_m+y) for F_m. Several values of y, a small quantity that may be either positive or negative, are tried to obtain a Q_r rating curve that closely fits the plotted discharge.

10. Compare the various Q_r rating curves obtained from step 9 and select the curve that best fits the plotted points used to define it. If the plotted discharges closely fit that Q_r rating curve, that rating curve and the corresponding relation of (Q_m/Q_r) to (F_m+y) are accepted for use. If the fit is not considered to be sufficiently close, the use of a pure parabolic relation, such as equation 81, is abandoned and the strictly empirical approach described in the following steps is used.

11. From the family of stage-discharge curves discussed in step 1, select one as the base Q_r curve and use the constant fall for this curve as F_r.

12. Compute the ratios Q_m/Q_r and F_m/F_r, plot the discharge ratios as ordinates and the fall ratios as abscissas, and draw an average curve through the plotted points that passes through the point whose coordinates are 1.0, 1.0.

13. Adjust each measured discharge by dividing it by the discharge ratio corresponding to the fall ratio on the above curve. Plot these computed values of Q_r against stage, and draw an average curve (Q_r curve) through the plotted points.

14. Repeat steps 11–13 using alternative constant values of F_r until the best relation between stage, fall, and discharge is established.

15. If the best relation derived from the application of steps 11–14 is still unsatisfactory, use the more flexible method described in the section titled, "Rating Fall a Function of Stage."

EXAMPLE OF RATING PROCEDURE

The stage-fall-discharge rating for Tennessee River at Guntersville, Ala. is presented in figure 190 as an example of a rating with constant rating fall. The upper gage is a water-stage recorder installed in a well attached to a pier of a highway bridge. The lower gage is a water-stage recorder installed on the right bank 43,700 ft below the upper gage and 3,300 ft above Guntersville Dam. The channel conditions in this reach are reasonably uniform. Variable backwater is caused by the operations at Guntersville Dam.

A satisfactory relation between stage, fall, and discharge could not be established for the upper (base) gage by use of the procedures for a pure parabolic fall-ratio curve that are described in steps 1–10. The empirical approach described in steps 11–14 was therefore used. The best rating was obtained by using a value of F_r equal to 1.5 ft. The fall-ratio curve in figure 190 approximately fits equation 80 for all fall ratios no greater than 1.0; for fall ratios greater than 1.0 the curve is flatter than a parabola defined by equation 80.

To plot, on the Q_r rating curve, a subsequent discharge measurement (Q_m) having a fall F_m, the fall ratio, F_m/F_r or $F_m/1.5$, is first computed. The fall-ratio curve is then entered with the computed fall ratio, and the discharge ratio, Q_m/Q_r, is read. Q_m is then divided by that value of the discharge ratio to give the value of Q_r to be plotted.

The method of obtaining the discharge corresponding to a given gage height and a given fall (F_m) is explained in the section titled, "Determination of Discharge from Relations for Variable Backwater."

RATING FALL A FUNCTION OF STAGE

GENERAL DISCUSSION OF RATING PRINCIPLES

Where variable backwater is a factor in the discharge rating, it will generally be found that fall is a function of stage. The average relation between fall and discharge may be linear, or fall may be a complex function of stage. Rating principles are best discussed by reference to examples.

The right-hand graph in figure 191 for the Columbia River at The Dalles, Oreg., is an example of a linear relation between stage and fall. The stage-discharge relation at the base gage is affected by reservoir operations at Bonneville Dam, more than 80 miles downstream. The auxiliary gage is located at Hood River bridge, 19 miles downstream from the base gage. Within the range of measured discharges, fall increases linearly with stage.

A much more complex stage-fall relation is shown in the right-hand graph in figure 192 for the Ohio River at Metropolis, Ill. At the downstream (auxiliary) gage, the stage-discharge relation is affected only at the lower stages by a constriction, the backwater from which causes fall to decrease with stage in the slope reach. At the higher

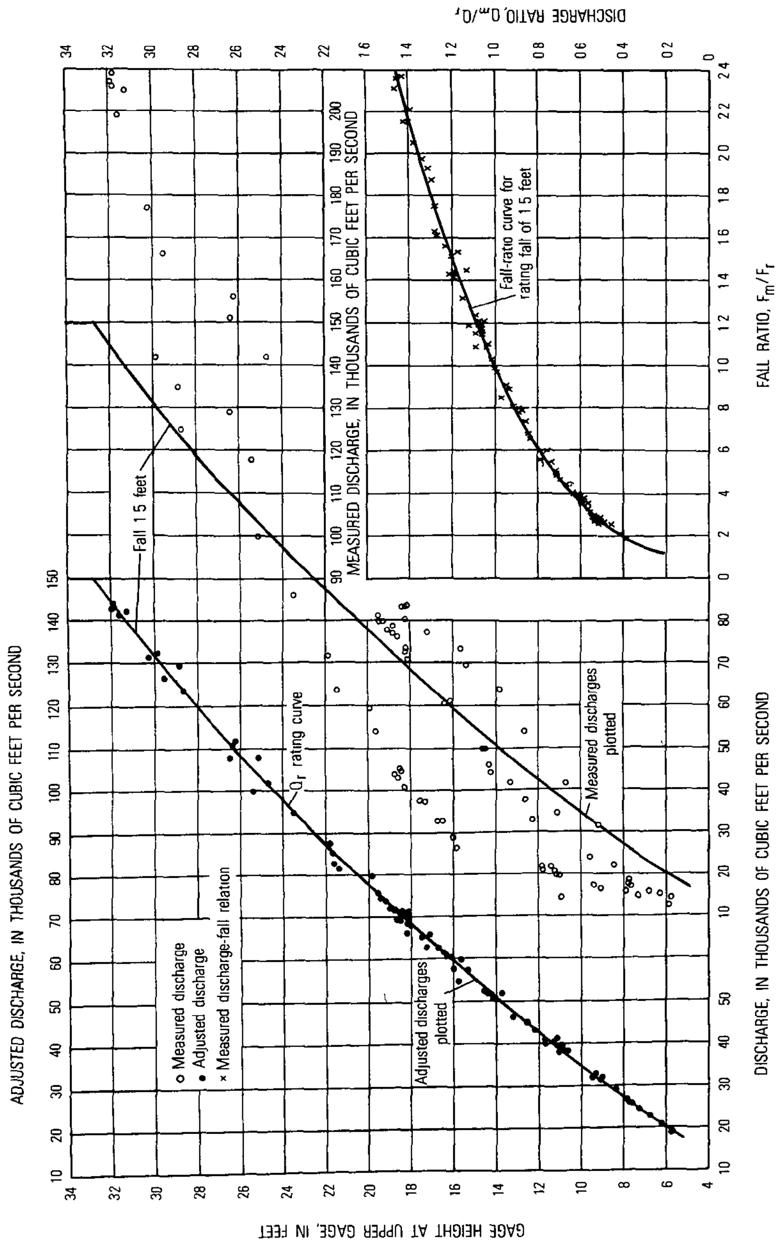

FIGURE 190.—Stage-fall-discharge relations for Tennessee River at Guntersville, Ala.

stages the constriction has little effect and fall increases with stage.

Another example of a complex stage-fall relation is shown in the right-hand graph in figure 193 for Kelly Bayou near Hosston, La. The base gage for this rating is about 2.7 miles upstream from the mouth of Kelly Bayou. The auxiliary gage is on Black Bayou, 4.2 miles downstream from the base gage. At low stages, fall increases with stage; at medium and high stages the backwater effect from Black Bayou is more pronounced and fall tends to assume a constant value.

Where a section control exists just downstream from the base gage, it is necessary to identify those situations when backwater effect is absent at the base gage. Obviously there will be no backwater when the tailwater at the section control is below the crest of the control. Most artificial controls are broad-crested, and submergence is generally effective only when tailwater rises to a height above the crest that is equal to or greater than 0.7 times the head on the control. Looked at another way, submergence is effective only when the fall between the upstream and downstream stages is equal to or less than 0.3 times the head on the control. Thus a straight line of initial submergence may be drawn on the curve of stage versus fall; the line passes through the coordinates representing the elevation of the control crest and zero fall, with a slope of 3 ft of stage per foot of fall.

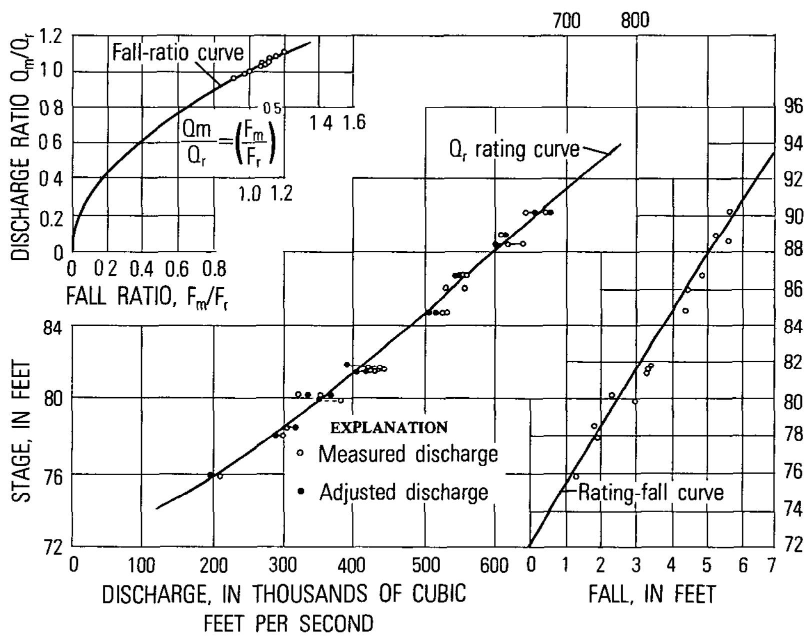

FIGURE 191.—Stage-fall-discharge relations for Columbia River at The Dalles, Oreg.

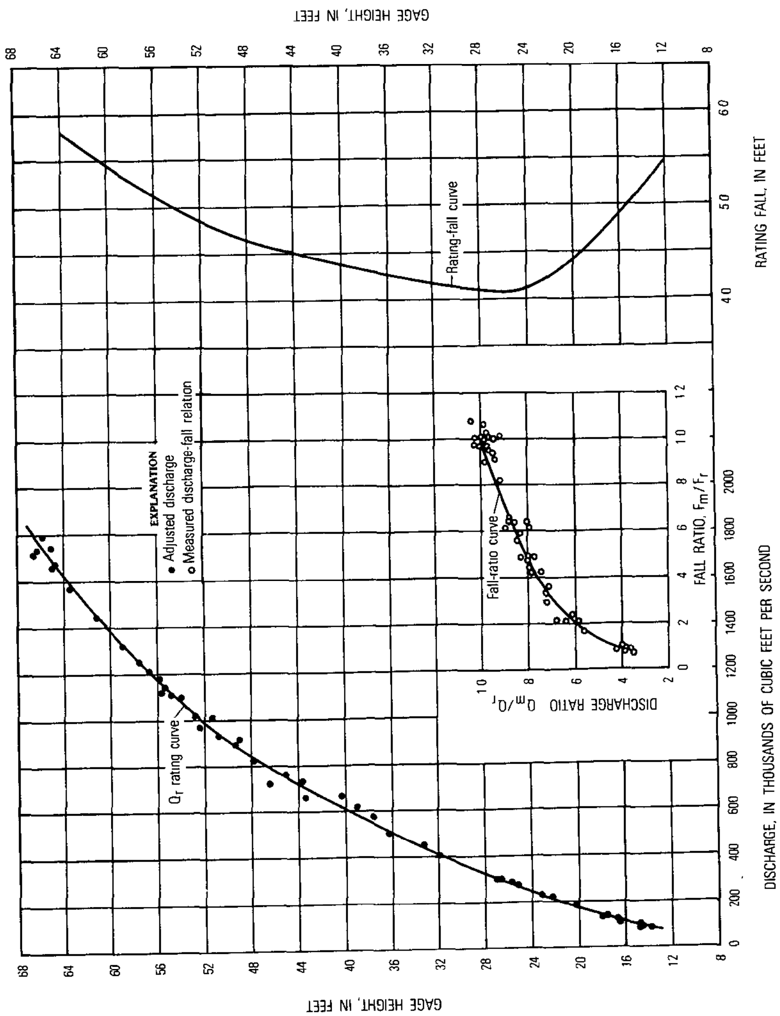

FIGURE 192.—Stage-fall-discharge relations for Ohio River at Metropolis, Ill.

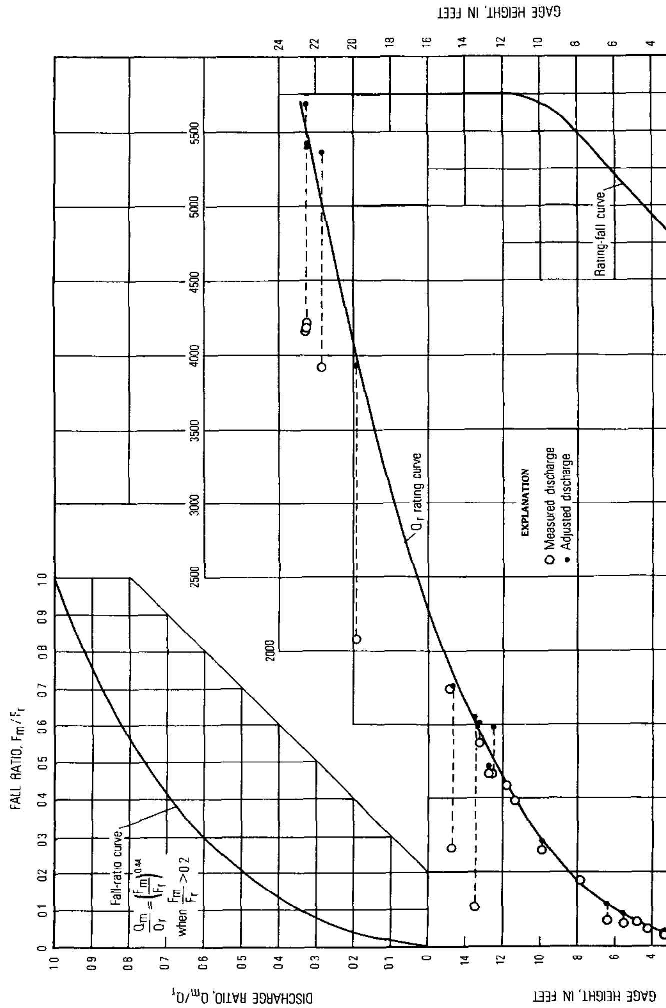

FIGURE 193.—Stage-fall-discharge relations for Kelly Bayou near Hosston, La.

The precise position and slope of the line will depend on the location of the downstream auxiliary gage with respect to the section control. If the auxiliary gage is immediately downstream from the control, the line of initial submergence will have the position and slope stated above. If the auxiliary gage is far downstream from the control, the line on the stage-fall graph will intersect the elevation of the control crest at a value of fall greater than zero, and the slope of the line will depend on the hydraulic features of the station; field observation will be necessary to define the graph coordinates of the line of initial submergence. All observed or recorded values of fall that lie below the line of initial submergence indicate free-fall discharge (discharge unaffected by the tailwater elevation); all observed or recorded values of fall that lie above the line of initial submergence indicate discharge affected by variable backwater. Furthermore, if the auxiliary (tailwater) gage is close to the control, the fall-ratio curve for discharges affected by backwater should closely fit the theoretical equation,

$$(Q_m/Q_r) = (F_m/F_r)^{0.5}.$$

If the auxiliary (tailwater) gage is distant from the control, the fall-ratio curve will depart from the theoretical equation.

The right-hand graph in figure 194 shows the stage-fall relation for Colusa Weir near Colusa, Calif. The base gage for the station is a short distance upstream from an ungated weir which acts as a section control, and the auxiliary gage is a short distance downstream from the control. There is no pool immediately upstream from Colusa Weir, the streambed being at the elevation of the weir crest; there is a drop of about 2 ft immediately downstream from the weir. The line of initial submergence shown crossing the lower part of the stage-fall relation has the theoretical position and slope discussed above. Colusa Weir is at the downstream end of a large natural detention basin along the left bank of the Sacramento River, and water that passes over the weir immediately enters the river. Because the river stage rises faster than the stage of the detention pool, fall decreases with stage at the base gage, as shown by the rating-fall curve.

The right-hand graph in figure 195 is a plot of stage versus fall for the Kootenay River at Grohman, B.C., Canada. The base gage for this station is on the west arm of Kootenay Lake about 2 miles upstream from Grohman Narrows. Downstream from the narrows is the forebay of the Corra Linn powerplant, and in the forebay is the auxiliary gage, about 8 miles downstream from the base gage. Grohman Narrows is the control for the base gage, but operations of Corra Linn Dam cause variable submergence of the control when the stage of the

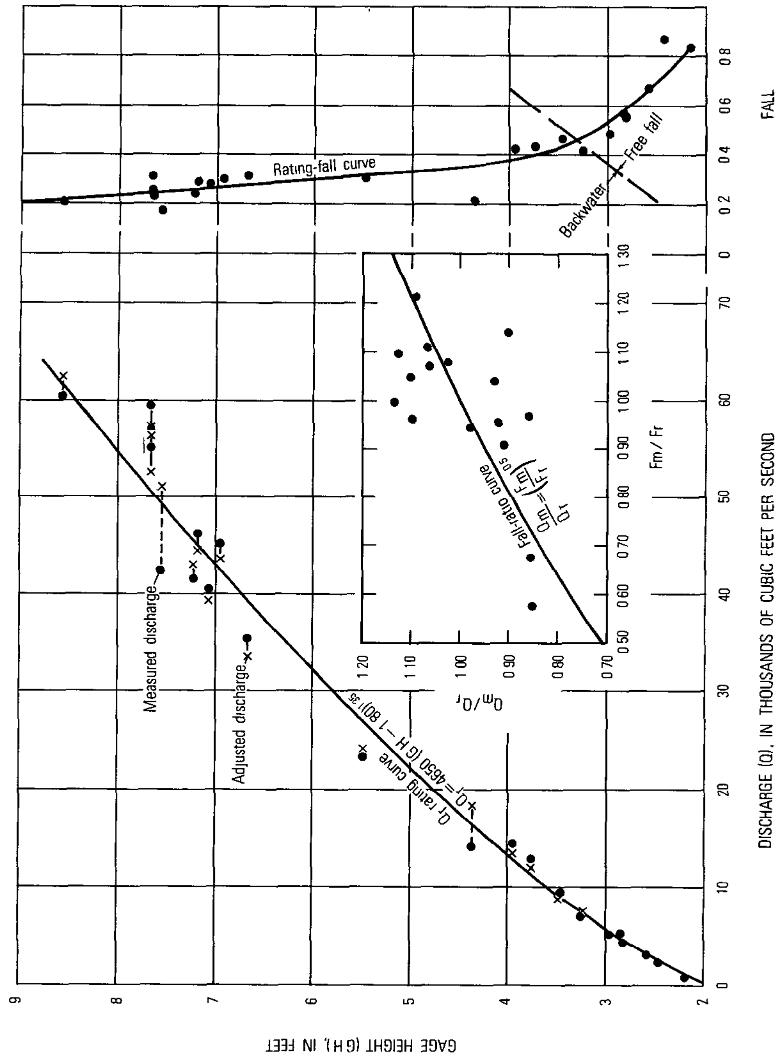

FIGURE 194.—Stage-fall-discharge relations for Colusa Weir near Colusa, Calif

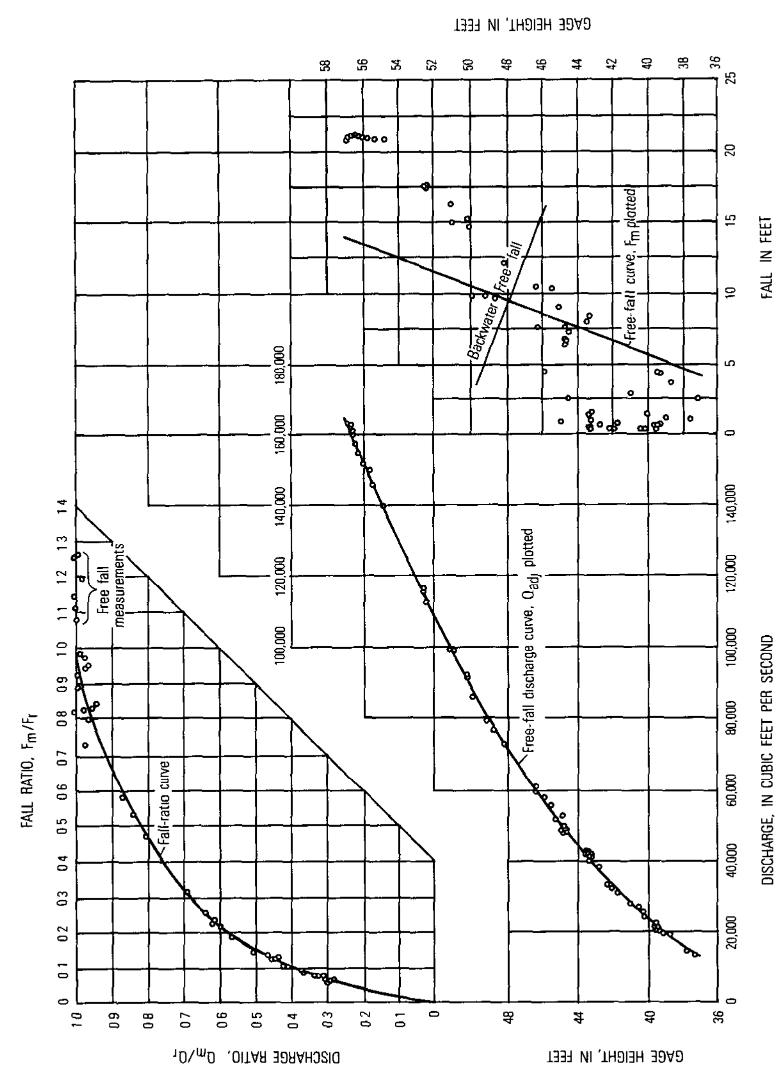

FIGURE 195.—Stage-fall-discharge relations for Kootenay River at Grohman, British Columbia, Canada. Measurements with falls less than 0.4 feet not plotted.

forebay is sufficiently high. The line of initial submergence, shown as the free-fall curve in figure 195, was determined from observation and discharge measurements. Discharge measurements whose values of fall plot below, or to the right of, the free-fall curve are unaffected by backwater and those discharges are therefore independent of fall. Discharge measurements whose values of fall plot above, or to the left of, the free-fall curve are affected by variable backwater. For those measurements the graph shows no apparent relation between stage and fall, and the free-fall curve (line of initial submergence) was used as the rating-fall curve for the measurements affected by variable backwater.

The rating for a gaging station whose base gage has no section control is analyzed in a manner similar to that previously described in the section on "Rating Fall Constant—Procedure for Establishing the Rating," the principal difference being that instead of using a constant value of rating fall, the rating fall for any stage is obtained from the rating-fall curve. The rating for a gaging station whose base gage has a section control is analyzed in two separate steps. The free-fall part of the rating (no variable backwater) is analyzed as explained in chapter 10, where simple stage-discharge relations are discussed. That part of the rating that is affected by variable backwater is analyzed as though no section control existed. It is not necessary to use the free-fall rating curve as the basis for establishing that part of the rating that is affected by variable backwater although that course of action is commonly followed.

Summary.—In view of the many different and complex situations that exist in natural channels, it is difficult to give general guidelines for establishing stage-fall-discharge relations. The analyst should make every effort to acquaint himself with the physical characteristics of the channel and the source of variable backwater. The best position of the relation curves that comprise the discharge rating must be determined by trial and error. The complexity of those relations determines, to a large degree, the number of discharge measurements necessary to define the discharge rating. Although the methods are empirical, experience has shown that there may be found a stage-discharge relation (the Q_r curve) which, taken in conjunction with its associated stage-fall relation (the rating-fall curve), will give close approximation to the true discharge under all possible combinations of stage and fall, by the application of a single-curve relation, Q_m/Q_r versus F_m/F_r. It is desirable, but not always possible, to have that relation take the theoretical form,

$$Q_m/Q_r = (F_m/F_r)^{0.5} \qquad (80)$$

PROCEDURE FOR ESTABLISHING THE RATING

The general procedure used in establishing a stage-fall-discharge rating with variable fall is outlined as follows:

1. Plot all discharge measurements using stages at the base gage as ordinates and discharges (Q_m) as abscissas, and note the measured fall (F_m) beside each plotted point.
2. On another graph plot the measured fall (F_m) for each discharge measurement against stage at the base gage, using stage as the ordinate.
3. If the base gage has a section control, determine the position of the line of initial submergence on the plot of stage versus measured fall. Its position is based on discharge measurements known to have been made under conditions of free fall. Those measurements, plotted against stage on logarithmic graph paper, are fitted with a free-fall rating curve which is extrapolated in accordance with the principles discussed in chapter 10. The remaining measurements are added to the logarithmic rating plot; those measurements that plot to the left of the extrapolation are considered to be affected by backwater. That knowledge, along with a knowledge of the probable degree of submergence required to cause backwater effect, enables the analyst to fix the position of the line of initial submergence. Only those measurements that plot above, or to the left of, the line of initial submergence are used in the analysis of the rating for variable backwater that is discussed in the steps that follow.
4. Fit a curve, Q_r rating curve, to the stage-discharge plot in step 1, and another curve, F_r, or rating-fall curve, to the stage-fall plot in step 2.
5. From the curves in step 4 obtain values of Q_r and F_r corresponding to the stage of each discharge measurement.
6. Compute the ratios Q_m/Q_r and F_m/F_r for each discharge measurement.
7. Plot Q_m/Q_r as ordinate against F_m/F_r as abscissa, and on that graph draw the curve $Q_m/Q_r=(F_m/F_r)^{0.5}$.
8. On the basis of the scatter of the plotted points about the curve in step 7, adjust the Q_r and F_r curves (step 4) to obtain revised values of Q_r and F_r (step 5), such that the new ratios of Q_m/Q_r and F_m/F_r fit the curve in step 7 as closely as possible. The adjustments to the Q_r and F_r curve should not be so drastic that the adjusted curves are no longer smooth curves.
9. Repeat steps 4–8, using exponents of (F_m/F_r) other than, but close to 0.5. Try exponents equal to 0.40, 0.45, 0.55, and 0.60.
10. Compare the five plots of Q_m/Q_r versus F_m/F_r and select the one which shows the best fit between curve and plotted points. (The ratio of plotted values of Q_m/Q_r to curve values of Q_m/Q_r is identical with

the ratio of measured discharge to discharge obtained from the stage-fall-discharge relations.) In steps 11 and 12 that follow, the exponent of that best fall-ratio curve will be referred to as d.

11. If the plotted ratios closely fit the curve $(Q_m/Q_r) = (F_m/F_r)^d$, that curve and the corresponding Q_r and F_r curves are accepted for use.

12. If the plotted ratios do not closely fit the curve $(Q_m/Q_r) = (F_m/F_r)^d$, repeat steps 4–8, using the exponent d but substituting the terms $(F_m + y)$ for F_m and $(F_r + y)$ for F_r. Several values of y, a small quantity that may be either positive or negative, are tried to obtain a close fit between plotted points and the curve $(Q_m/Q_r) = \left[(F_m+y)/(F_r+y)\right]^d$.

13. Compare the various plots of the fall-ratio graph obtained from step 12 and select the one showing the best fit between curve and plotted points. If the fit is satisfactory, that curve and the corresponding Q_r and F_r curves are selected for use. If the fit is not considered to be sufficiently close, the use of a pure parabolic relation, such as

$$Q_m/Q_r = (F_m/F_r)^d \tag{82}$$

or

$$Q_m/Q_r = \left[(F_m+y)/(F_r+y)\right]^d \tag{83}$$

is abandoned and the strictly empirical approach described in the following steps is used.

14. Select one of the trial Q_r and F_r curves, such as were constructed in step 4, along with the corresponding values of Q_r, F_r, Q_m/Q_r, and F_m/F_r, such as were obtained in steps 5 and 6.

15. Plot the discharge ratios as ordinates and the fall ratios as abscissas, and draw an average curve through the plotted points that passes through the point whose coordinates are (1.0, 1.0).

16. On the basis of the scatter of the plotted points about the curve in step 15, adjust the Q_r and F_r curves (step 14), as well as the fall-ratio curve. Again, the reminder that the adjusted curves must remain smooth curves.

17. Repeat steps 14–16, using other trial curves of Q_r, F_r, and fall ratio versus discharge ratio, until the best relation is established between stage, fall, and discharge; in other words, until a close fit is obtained between plotted points and the fall-ratio curve.

18. After having obtained acceptable Q_r, F_r, and fall-ratio curves, plot adjusted values of the discharge measurements on the Q_r rating curve. The adjusted values are computed as follows: Given a measured discharge (Q_m) and a measured fall (F_m). Enter the F_r curve (stage-fall relation) with the gage height of the discharge measurement and read F_r. Next, compute the fall ratio, F_m/F_r, and enter

the fall-ratio curve to obtain the discharge ratio, Q_m/Q_r. Obtain the value Q_r to be plotted by dividing Q_m by (Q_m/Q_r).

The method of obtaining the discharge corresponding to a given gage height and a given fall (F_m) will be explained in the section titled, "Determination of Discharge from Relations for Variable Backwater."

EXAMPLES OF RATING PROCEDURE

Figures 191–195 are examples of stage-fall-discharge relations for slope stations where fall is a function of stage.

Figure 191 for a Columbia River station shows that excellent results were achieved in the range of discharge that was measured. The linear trend of fall increasing with stage is clearly evident, and the fall-ratio curve not only is represented by the theoretical equation 80, but is closely fitted by the plotted points. Where the rating-fall curve (stage versus fall) is so well defined, the first estimate of the Q_r curve is usually made by the use of equation 80, in which Q would represent the measured discharges. The computed Q_r values for the discharge measurements would then be plotted against stage, and a curve fitted to the plotted points would represent the first trial Q_r curve.

Figure 192 for an Ohio River station is an extremely complex example, as can be seen from the shape of the rating-fall curve. It is not surprising that the fall-ratio curve could not be expressed by a simple parabolic equation such as equation 82 or 83.

Figure 193 for a station on Kelly Bayou shows that there is relatively minor effect from variable backwater at low stages. At medium and high stages, the variable stage of Black Bayou causes variable backwater at the base gage. The rating-fall used during high-water periods has the constant value of 10.0 ft. The fall-ratio curve, for values of F_m/F_r greater than 0.2, has the equation

$$Q_m/Q_r = (F_m/F_r)^{0.44}.$$

Because the exponent 0.44 does not differ greatly from its theoretical value of 0.5, the Q_r rating curve can be extrapolated with some confidence.

Figure 194 for Colusa Weir is an example of the stage-fall-discharge relation for a station whose base gage has a section control. There is no variable backwater at low flow, as shown by the 6 discharge measurements that plot below the line of initial submergence on the graph of stage versus fall. The remaining 16 discharge measurements show the effect of variable backwater. While the fit of adjusted measured discharges to the Q_r rating curve is not completely satisfactory, there is some satisfaction to be derived from the facts that the equation of the fall-ratio curve is theoretically correct and the fall-ratio curve balances the plotted points.

Figure 195 for a station on the Kootenay River is an example of the stage-fall-discharge relation for a station whose base gage has a control that is unsubmerged at high stages. Of the 59 discharge measurements shown, 23 were made under free-fall conditions; they plot below, or to the right of, the line of initial submergence on the graph of stage versus fall. The remaining 36 discharge measurements are affected by variable backwater and were used in the stage-fall-discharge analysis. Because the line of initial submergence was used as F_r in the analysis, the value of F_m for any measurement affected by backwater is less than F_r. Consequently the fall-ratio curve was fitted empirically to the plotted points and is not expressed by a simple parabolic equation such as equation 82 or 83.

DETERMINATION OF DISCHARGE FROM RELATIONS FOR VARIABLE BACKWATER

After the three necessary graphical relations are available—stage versus rating fall (F_r), stage versus rating discharge (Q_r), and Q_m/Q_r versus F_m/F_r—the graphs are converted to tables. The determination of discharge (Q_m) corresponding to a given stage and a given fall (F_m) proceeds as follows:

1) From the stage-fall table determine the rating fall, F_r, for the known stage.
2) Compute the ratio F_m/F_r.
3) From the table of discharge ratios, (Q_m/Q_r) and fall ratios (F_m/F_r), determine the value of the ratio Q_m/Q_r.
4) From the stage-discharge table, determine the rating discharge, Q_r, for the known stage.
5) Compute Q_m by multiplying the ratio Q_m/Q_r by the value of Q_r.

Much emphasis has been placed on obtaining a purely parabolic function, such as equation 82 or 83, for the relation between fall ratio and discharge ratio. Such a relation not only permits the analyst to extrapolate the Q_r curve with more confidence, but it also expedites the computation of discharge. For example equation 82 may be transposed to

$$Q_m = \left(\frac{Q_r}{F_r^{N}}\right)\left(F_m^{N}\right). \qquad (82a)$$

Two tables can be prepared, one giving the values of the quantity (Q_r/F_r^{N}) corresponding to stage, and the other giving values of (F_m^{N}) corresponding to values of F_m. The discharge is then computed as the

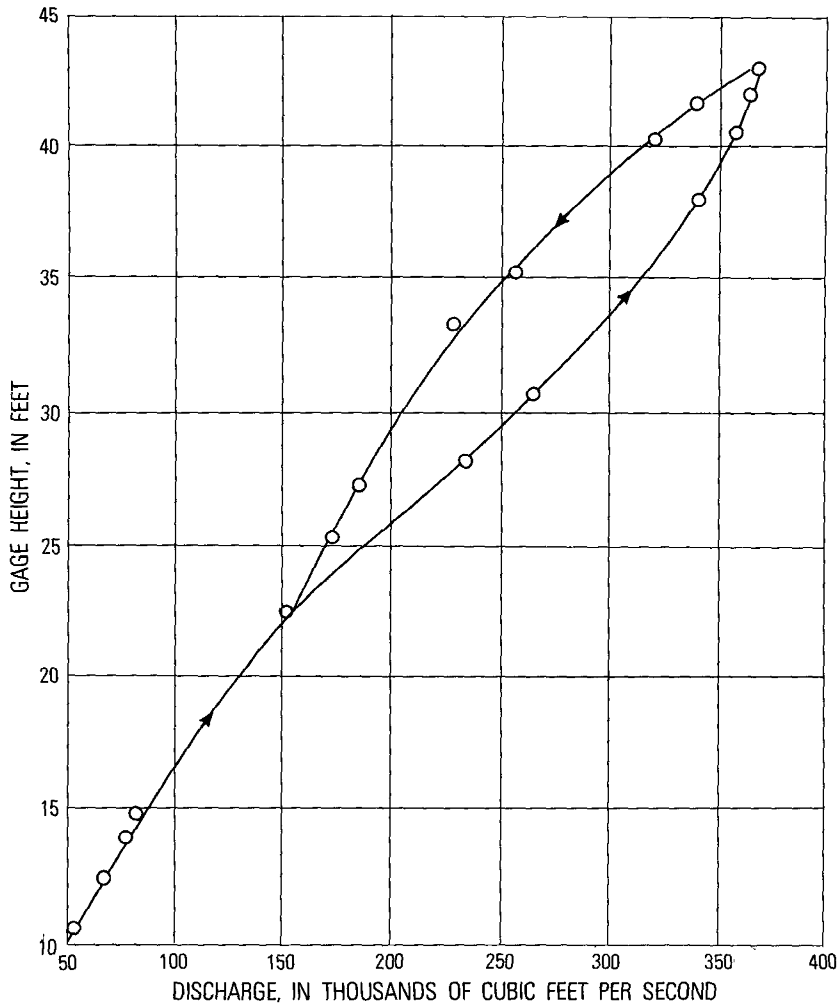

FIGURE 196.—Stage-discharge loop for the Ohio River at Wheeling, W. Va., during the flood of March 14–27, 1905.

product of the two values picked from the tables. Equation 83 may be transposed in a similar way.

VARIABLE SLOPE CAUSED BY CHANGING DISCHARGE

THEORETICAL CONSIDERATIONS

Where channel control is effective, the effect of changing discharge on a graph of the stage-discharge relation is such as to produce a loop curve (fig. 196), on which the discharge for a given stage is greater when the stream is rising than it is when the stream is falling. In other words, given a simple stage-discharge relation for steady flow—that is, a rating that averages all discharge measurements—it will be found that the measurements made on a rising stage plot to the right of the curve and those made on a falling stage plot to the left. The discharge measurements for individual flood waves will commonly describe individual loops in the rating. The departure of measurements from the rating curve for steady flow is of significant magnitude only if the slope of the stream is relatively flat and the rate of change of discharge is rapid. For gaging stations where this scatter of discharge measurements does occur, the discharge rating must be developed by the application of adjustment factors that relate steady flow to unsteady flow. (Unsteady flow refers to discharge at a site that changes appreciably with time, as in the passage of a flood wave.)

The relation between the discharges for steady and unsteady conditions at the same stage can be derived from the general equations for unsteady flow (Rouse, 1950). A simplified equation shown below may also be derived by neglecting all terms representing change of velocity head or acceleration.

$$\frac{Q_m}{Q_c} = \sqrt{1 + \frac{1}{S_c\, v_w}\,\frac{dh}{dt}} \tag{84}$$

where Q_m is the discharge for unsteady flow, Q_c and S_c are the discharge and energy slope for steady flow at the same stage, v_w is the wave velocity, and dh/dt is the rate of change of stage with respect to time (dh is positive for rising stages).

Because equation 84 is basic to the methods commonly used for adjusting discharge ratings for the effect of changing discharge, it is appropriate to elaborate on its derivation. The ratio of the magnitudes of two discharges that occur at a given stage is equal to the ratio of the square roots of their energy slopes. That principle can be expressed in the following basic equation, which is similar to equation 80 that was used in preceding sections of the manual.

$$\frac{Q_m}{Q_c} = \frac{\sqrt{S_m}}{\sqrt{S_c}} \tag{85}$$

where S_m is the energy slope for unsteady flow at the time of Q_m; the remaining terms are defined above for equation 84.

During changing discharge, the slope of the water surface increases or decreases by an increment of slope (ΔS), where

$$\Delta S = \frac{1}{v_w}\frac{dh}{dt} \cdot \tag{86}$$

If we assume that the increment of slope by which the energy gradient changes is likewise equal to ΔS, then

$$S_m = S_c + \Delta S = S_c + \frac{1}{v_w}\frac{dh}{dt} \cdot \tag{87}$$

By combining equations 85 and 87,

$$\frac{Q_m}{Q_c} = \left(\frac{S_c + \frac{1}{v_w}\frac{dh}{dt}}{S_c}\right)^{1/2}, \tag{88}$$

or

$$\frac{Q_m}{Q_c} = \left(1 + \frac{1}{S_c\, v_w}\frac{dh}{dt}\right)^{1/2} . \tag{84}$$

The wave velocity v_w in the above equations may be evaluated by the Seddon principle (Seddon, J. E., 1900).

$$v_w = \frac{1}{B}\frac{dQ}{dh},$$

where B is the width of the channel at the water surface, and dQ/dh is the slope of the stage-discharge curve for constant-flow conditions. From examination of formulas for mean velocity (V_m) in open channels, the ratio of wave velocity to mean velocity may be shown to vary as follows,

	Ratio v_w/V_m	
Channel Type	*Manning*	*Chezy*
Triangular	1.33	1.25
Wide rectangular	1.67	1.50
Wide parabolic	1.44	1.33

Experience seems to indicate that the most probable value of the ratio in natural channels is 1.3.

Equation 84 explains why the effect of changing discharge is significant only on flat streams during rapid changes in discharge; that combination is necessary to make the right-hand side of the equation differ significantly from unity. During rapid changes in discharge, absolute values of dh/dt are large. On flat streams both energy slope (S_e) and wave velocity (v_w) are small. The combination of a large value of dh/dt and small values of S_e and v_w gives the right-hand side of the equation a value that is significantly larger than unity during a rising stage (dh/dt is positive) and significantly smaller than unity during a falling stage (dh/dt is negative).

METHODS OF RATING ADJUSTMENT FOR CHANGING DISCHARGE

The two methods used to adjust discharge for the effect of changing slope attributable to changing discharge are the Boyer method and the Wiggins method. Both methods are based on equation 84. The knowledgeable reader of this manual may notice that the Jones and Lewis methods are not included among the techniques for adjusting discharge. Those two methods have been supplanted by the somewhat similar Boyer method and therefore are not described here. For a description of the Jones and Lewis methods the interested reader is referred to the manual by Corbett (1943, p. 159–165).

BOYER METHOD

The Boyer method provides a solution of equation 84 without the necessity for individual evaluation of v_w and S_e. The method requires numerous discharge measurements made under the conditions of rising and falling stage. Measured discharge (Q_m) is plotted against stage in the usual manner, and beside each plotted point is noted the value of dh/dt for the measurement. For convenience dh/dt is expressed in feet or meters per hour and the algebraic sign of dh/dt is included in the notation—plus for a rising stage and minus for a falling stage. A trial Q_c rating curve, representing the steady-flow condition where dh/dt equals zero, is fitted to the plotted discharge measurements, its position being influenced by the values of dh/dt noted for the plotted points. Values of Q_c from the curve corresponding to the stage of each discharge measurement, are used in equation 84, along with the measured discharge (Q_m) and observed change in stage (dh/dt), to compute corresponding values of the adjustment factor, $1/S_e v_w$. The computed values of $1/S_e v_w$ are then plotted against stage and a smooth curve is fitted to the plotted points. If the plotted values of $1/S_e v_w$ scatter widely about the curve, the Q_c curve is modified to produce some new values of $1/S_e v_w$ that can be better fitted by a smooth curve. The modifications of the curves of Q_c and $1/S_e v_w$ should not be so drastic that the modified curves are no longer smooth curves, nor should the modified shape of the Q_c rating curve violate the principles underlying rating curves, as discussed in chap-

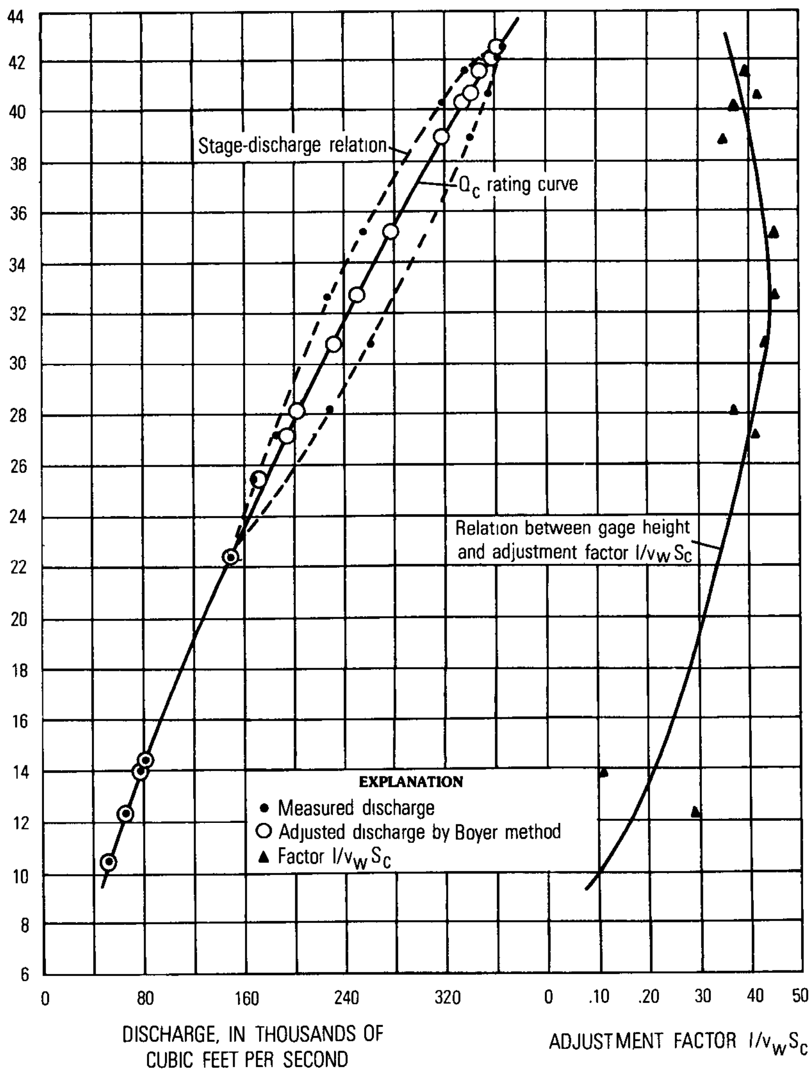

FIGURE 197.—Adjustment of discharge measurements for changing discharge, Ohio River at Wheeling, W. Va., during the period March 14–27, 1905.

ter 10. Construction of the two curves completes the rating analysis. Figure 197 is an example of such an analysis.

To adjust the value of subsequent discharge measurements for plotting on the Q_c rating curve, the adjustment-factor curve is first entered with the stage of the measurement to obtain the appropriate value of the factor, $1/S_c v_w$. Next, the observed value of dh/dt is used with that factor to compute the term, $\left(1+\frac{1}{S_c v_w}\frac{dh}{dt}\right)^{0.5}$. That term is then divided into the measured discharge (Q_m) to obtain the required value of Q_c.

To determine discharge from the Q_c rating curve and adjustment-factor curve, during a period when the stage and rate of change of stage are known, the procedure described above is used to obtain the value of the term $\left(1+\frac{1}{S_c v_w}\frac{dh}{dt}\right)^{1/2}$. That term is then multiplied by Q_c, which is obtained by entering the Q_c rating curve with the known stage. The product is the required discharge (Q_m).

WIGGINS METHOD

The Wiggins method is convenient for adjusting measured discharge (Q_m) for the effect of changing discharge to obtain the corresponding steady-flow discharge (Q_c). However, the reverse procedure of computing discharge for unsteady flow (Q_m) from the steady-flow discharge rating is rather complicated. Consequently, the Wiggins method is used only for those stations where only occasional adjustment of measured discharge at high stages is required. If the discharge is affected by changing stage on numerous days each year, the more accurate Boyer method of discharge adjustment should be used. Unlike the Boyer method, application of the Wiggins method does not require numerous discharge measurements that have been made under conditions of both rising and falling stage.

The discharge measurement adjusted by the Wiggins method are used to define the steady-flow rating, and that rating is used directly with the gage-height record to obtain daily values of discharge. That course of action is justifiable for those streams whose discharge is affected by changing discharge on only a few days each year. For that type of stream, it will generally be found that the discharge adjustment is less than 10 percent. On the affected days, the discharge obtained from the steady flow rating will be underestimated by a small percentage when the discharge is rising rapidly, and overestimated by a small percentage when the discharge is falling rapidly. The discrepancies are compensating, and if only few days are involved, the streamflow record is not significantly impaired. The advantage of applying the adjustment to discharge measurements made under unsteady-flow conditions is that the scatter of discharge measurements on the rating curve is reduced, and the rating curve can therefore be more precisely defined.

Application of the Wiggins method has been simplified by the preparation of diagrams that eliminate much of the computational labor. Figures 198*A*-*D* are used to determine the value of the energy slope

(S_m) at the time of the discharge measurement (Q_m), for combinations of values of mean velocity (V_m) and hydraulic radius (R). The Manning equation was used in preparing the graphs, and each of the four sheets is applicable for a particular value of Mannings n, as shown in the following tabulation:

Figure 198A—n=0.025 Smooth bed and banks.
198B—n=0.035 Fairly smooth.
198C—n=0.050 Rough.
198D—n=0.080 Very rough.

Figure 199 is used to determine the increment of energy slope $\left(\frac{1}{v_w}\frac{dh}{dt}\right)$ attributable to changing discharge, for combinations of values of flood-wave velocity (v_w) and rate of change of stage (dh/dt). Flood-wave velocity is assumed to equal $1.3V_m$.

Figures 200*A* and *B* are used to determine the factor to apply to the measured discharge (Q_m) to obtain the steady-flow discharge (Q_c). The factor, which is equal to

$$\left[\frac{S_m-\left(\frac{1}{v_w}\right)\left(\frac{dh}{dt}\right)}{S_m}\right]^{0.5},$$

is given for combinations of values of S_m from figure 198 and of $\left(\frac{1}{v_w}\frac{dh}{dt}\right)$ from figure 199. (Note that the factor differs from that given in equation 88, because S_m is used here as the base slope, rather than S_c as in equation 88.) Figure 200*A* is used for rising stages and figure 200*B* is used for falling stages.

An example of the use of the Wiggins diagrams follows.

Given: a discharge measurement with the following data for a stream with fairly smooth bed (n=0.035);

Q_m=23,000 ft³/s
Area=53,900 ft²
Width= 2,700 ft
V_m=4.27 ft/s
Change in stage=0.87 ft in 1.5 hours (rising)

Compute adjusted discharge to be plotted on rating curve.

First compute:
$$R=\frac{\text{Area}}{\text{Width}}=\frac{53{,}900}{2{,}700}=20 \text{ ft}$$
$$v_w=1.3\,V_m=1.3\times 4.27=5.55 \text{ ft/s}$$
$$\frac{dh}{dt}=\text{change in stage per hour}=\frac{0.87}{1.5}=0.58 \text{ ft/hr}$$

Then: (a) Enter figure 198*B* with V_m = 4.27 and R = 20 and read S_m = 0.00018

(b) Enter figure 199 with $\frac{dh}{dt}$ = 0.58 and v_w = 5.55 and read slope increment $(\frac{1}{v_w}\frac{dh}{dt})$ = 0.000029

(c) Enter figure 200*A* (rising stage) with S_m = 0.00018 and slope increment = 0.000029 and read factor = 0.915.

Adjusted discharge = 0.915 × 230,000 = 210,000 ft^3/s.

Because the stage was rising, the unadjusted discharge would plot to the right of the rating curve. The computed adjustment moves the measurement to the left.

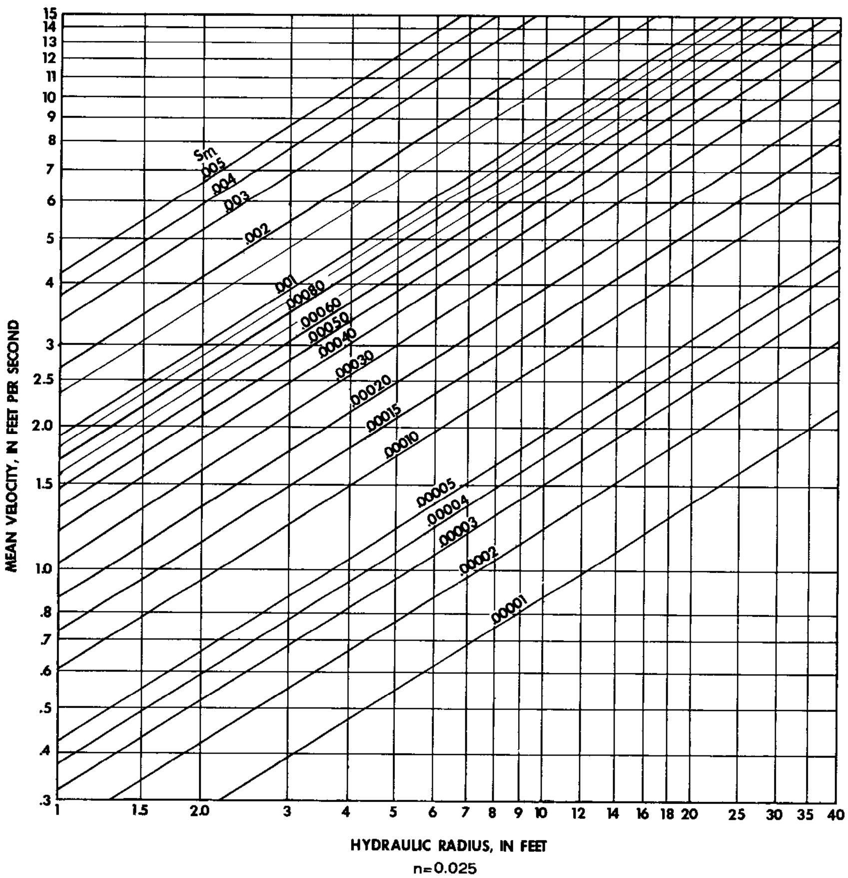

FIGURE 198*A*.—Diagram for solution of the Manning equation to determine S_m. Smooth bed and banks (n=0.025).

Both the measured (Q_m) and adjusted (Q_c) discharges are entered in the list of discharge measurements and both are plotted on the rating curve. Suitable symbols are used, however, to differentiate between the measured and adjusted discharges.

VARIABLE SLOPE CAUSED BY A COMBINATION OF VARIABLE BACKWATER AND CHANGING DISCHARGE

Where the rating for a gaging station is affected by a combination of variable backwater and changing discharge, the rating should be analyzed as though it were affected by variable backwater only, using the fall-rating methods described in the section titled, "Rating Fall a Function of Stage." The basic equation for variable-backwater adjustments (eq. 80) and that for changing-discharge adjustments (eq.

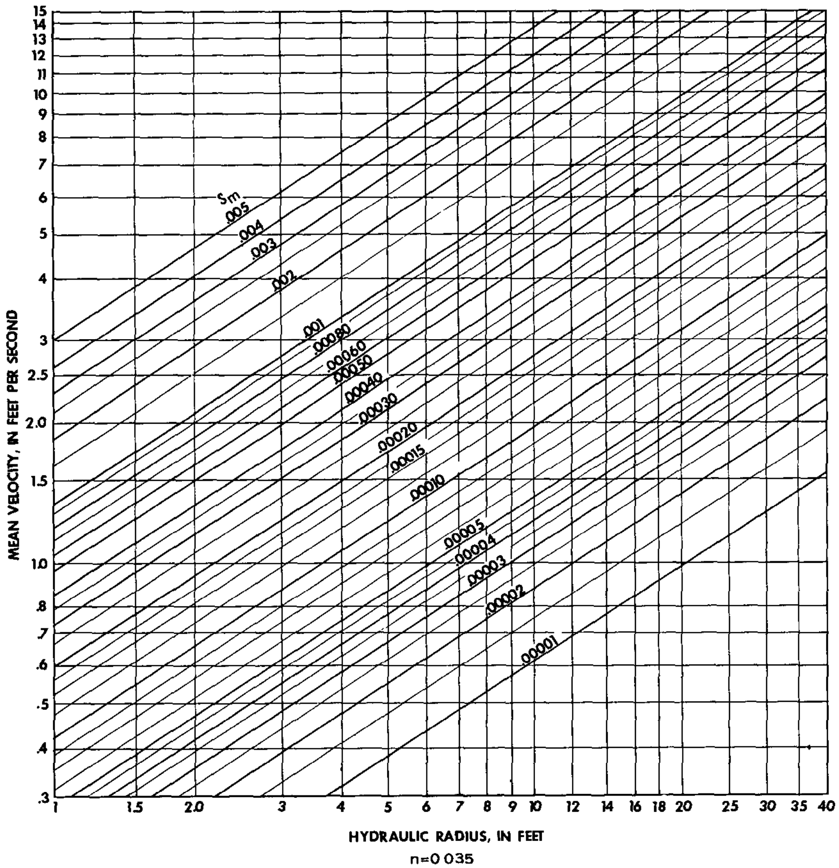

FIGURE 198*B*.—Diagram for solution of the Manning equation to determine S_m. Fairly smooth bed (n = 0.035).

85) are similar, but only the fall-rating methods are versatile enough to handle the combined effect of the two factors.

SHIFTS IN DISCHARGE RATINGS WHERE SLOPE IS A FACTOR

Changes in channel geometry (scour or fill) and (or) changes in flow conditions (vegetal growth) will cause shifts in the discharge rating where slope is a factor, just as they cause shifts in simple stage-discharge relations. When discharge measurements indicate a shift in the rating for a slope station, the shifts should be applied to the Q_r rating curve if the station is affected by variable backwater, or to the Q_c rating curve if the station is affected by changing discharge. Extrapolation of the shift curves should be performed in accordance with the principles discussed in chapter 10 for shifts in simple stage-

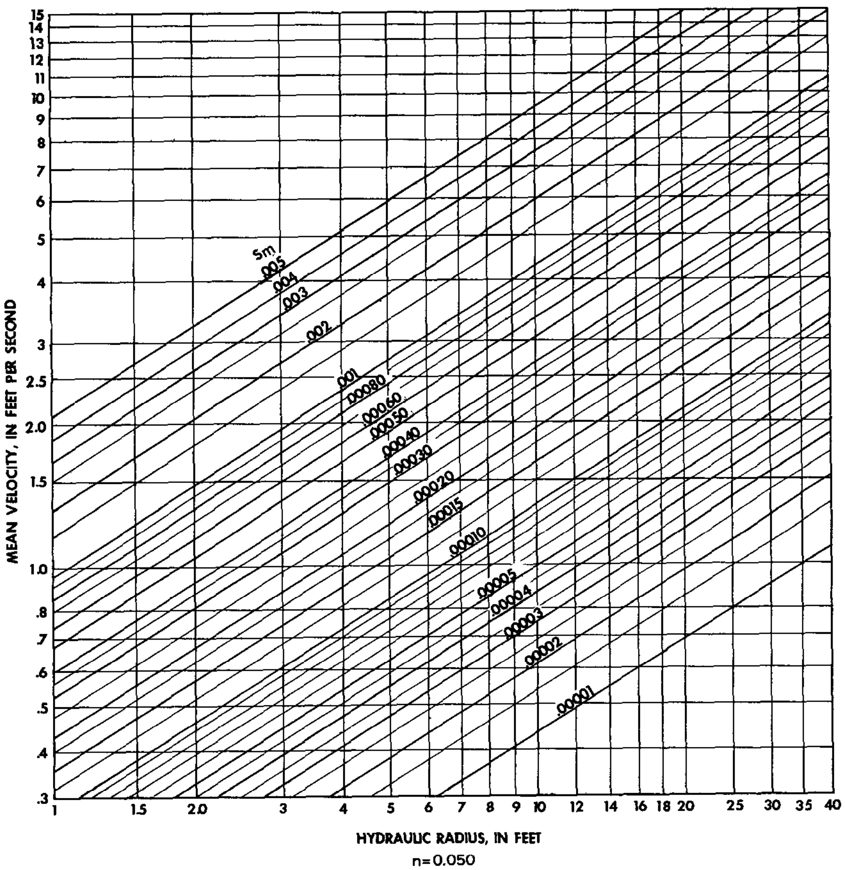

FIGURE 198*C*.—Diagram for solution of the Manning equation to determine S_m Rough bed (n=0.050).

discharge relations. (See section in chapter 10 titled, "Shifts in the Discharge Rating.")

A SUGGESTED NEW APPROACH FOR COMPUTING DISCHARGE RECORDS FOR SLOPE STATIONS

Now that the use of electronic computers has become commonplace, it appears that a fresh approach might be tried with regard to computing streamflow records for gaging stations equipped with a stage-recorder at each end of a slope reach. Instead of using the various graphical empiricisms that were described in this chapter, a computer program could be written to compute discharge for the reach by the Manning equation or by some similar equation for open-channel flow. (It is assumed that acceleration head can be neglected.) Dis-

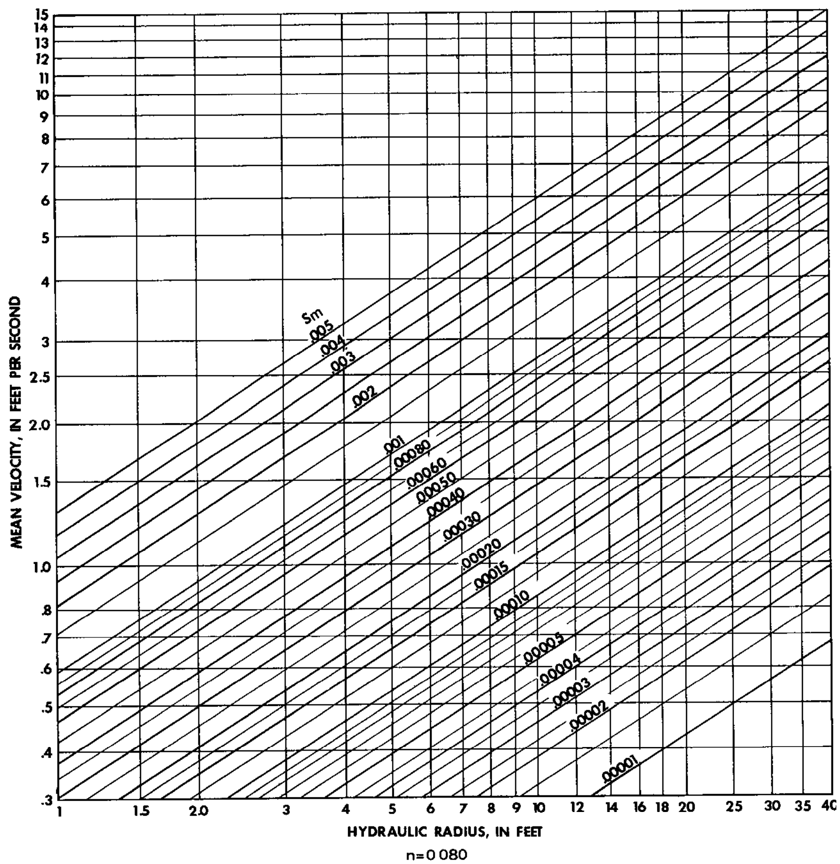

FIGURE 198*D*.—Diagram for solution of the Manning equation to determine S_m. Very rough bed (n=0.080).

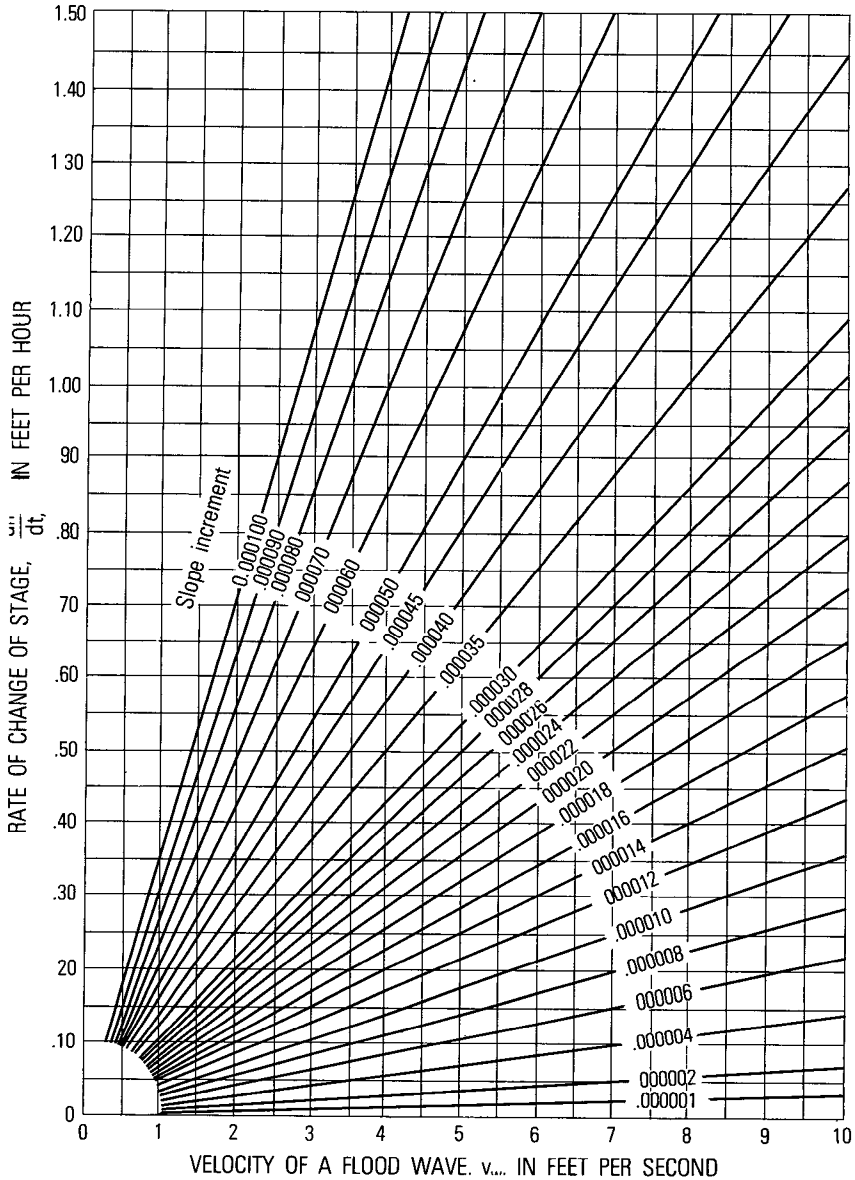

FIGURE 199.—Diagram for determining slope increment resulting from changing discharge.

charge measurements would be made solely for the purpose of determining the Manning roughness coefficient (n) from the measured discharge, thereby obtaining the only unkown factor needed to compute the conveyance (K) at each end of the slope reach.

The value of n computed from a discharge measurement usually would not represent the true value of the roughness coefficent but would actually be a "catchall" value that included the effect of error in the computed value of the energy slope in the reach. The computed values of n would likely vary with stage.

The discharge computations would proceed along the following lines. The basic form of the Manning equation is

$$Q = KS^{1/2} \tag{89}$$

where

Q is discharge;
K is conveyance, which is equal to $\frac{1.49}{n} AR^{2/3}$ (A is cross-sectional area and R is hydraulic radius); and
S is the energy gradient.

Equation 89 can be expanded to

$$Q = K_2 \sqrt{\frac{F}{\frac{K_2}{K_1} L + \frac{K_2^2}{2gA_2^2}\left[-\alpha_1\left(\frac{A_2}{A_1}\right)^2(1-k)+\alpha_2(1-k)\right]}} \tag{90}$$

where
F is fall in the reach,
L is length of reach,
g is the acceleration of gravity,
α is the velocity-head coefficient whose value is dependent on the velocity distribution in the cross section,
k is a coefficient of energy loss whose value is considered to be zero for contracting reaches and 0.5 for expanding reaches;
subscript 1 refers to the upstream cross section, and
subscript 2 refers to the downstream cross section.

For the cross section at each end of the slope reach, relations would be prepared between stage and each of the following three elements: K, A, and α. A computer program would be written to solve equation 90. Then, given the stage at each end of the reach, the computer would compute F, A, K, α, and finally, Q.

For those slope stations where the change in velocity head in the reach is so minor an item that it can be neglected, the conventional constant-fall method (see section titled, "Rating Fall Constant") could be continued in use; computer computation would be optional.

It is emphasized that the above method of computing discharge records is as yet untried, but it is suggested that it be tested.

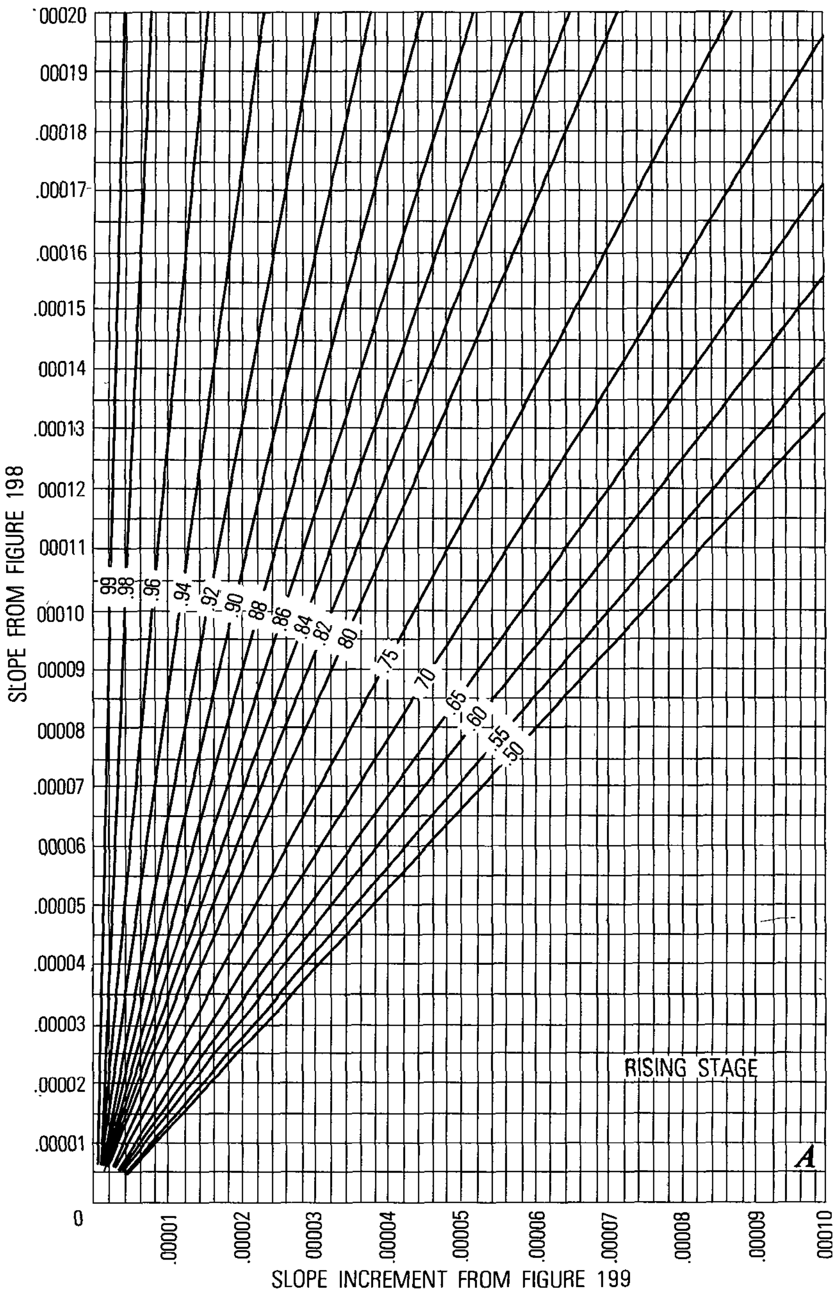

FIGURE 200*A*.—Diagram for determining factor to apply to measured discharge—rising stage.

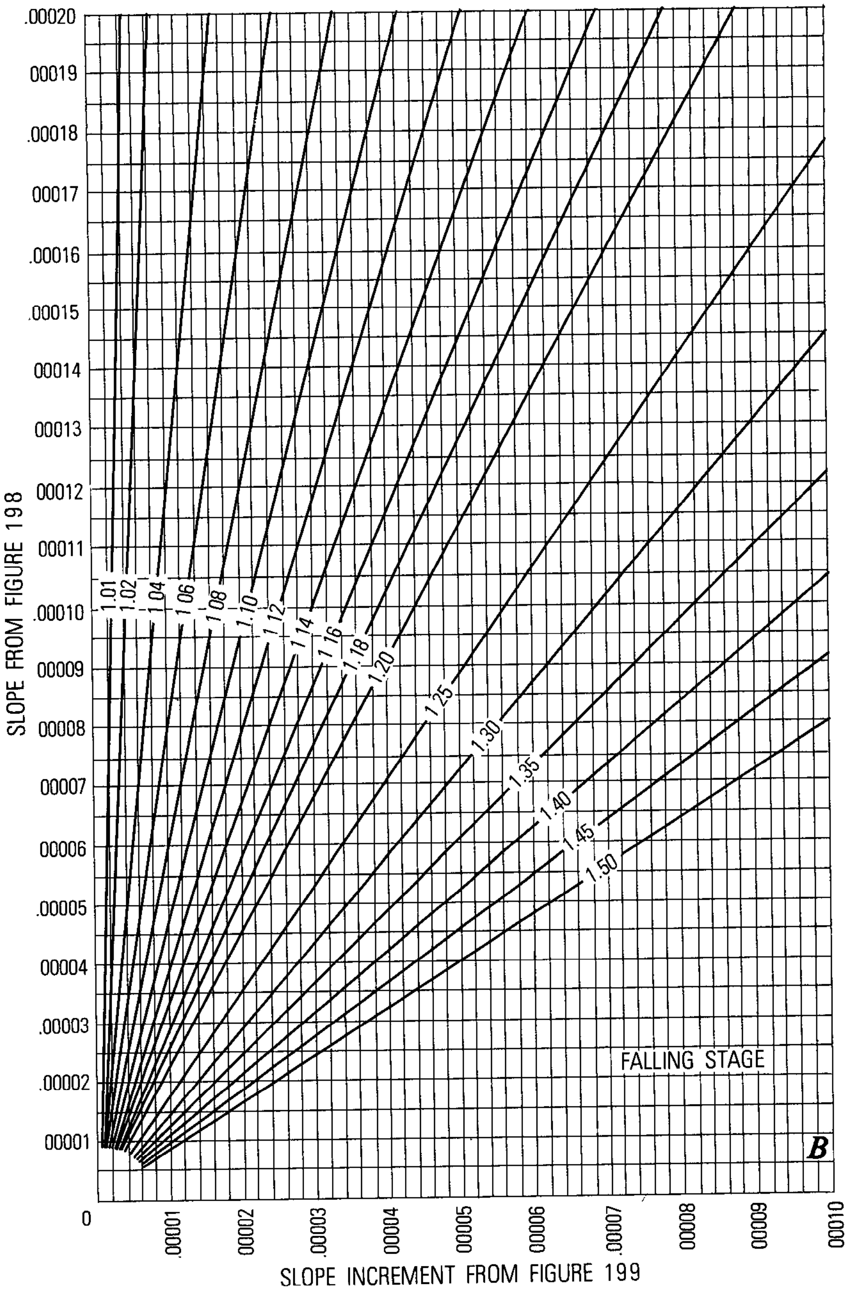

FIGURE 200*B*.—Diagram for determining factor to apply to measured discharge—falling stage.

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CHAPTER 12—DISCHARGE RATINGS USING A VELOCITY INDEX AS A PARAMETER

INTRODUCTION

Chapter 11 discussed the use of a slope parameter for developing discharge ratings at gaging stations where the use of stage alone was inadequate for rating purposes. However, it is not feasible to use a slope parameter for all stations for which no simple stage-discharge relation can be developed. Often slopes are so flat that the available reach of channel for developing slope is too short to give sufficiently accurate values of fall in the reach. At other sites, as on tidal streams or on some streams used for hydroelectric power generation, the acceleration head (p. 391) in the equations of unsteady flow is of such magnitude that it cannot be ignored as was done in chapter 11. In those situations it is often possible to develop a discharge rating by using a velocity index in a stage-velocity-discharge relation.

The principle behind a stage-velocity-discharge relation is simple enough. A continuous stage record provides a means of obtaining a continuous record of cross-sectional area from a relation of area to stage. If a continuously recorded velocity index, at a point or in a transverse line, can be related to stage and mean velocity in the cross section, the product of cross sectional area and mean velocity gives the discharge at any time. The calibration of the velocity relation—that is, the relation of recorded index velocity to stage and mean velocity—requires discharge measurements for the determination of mean velocity. The discharge measurements also furnish the values of cross-sectional area to be used in the stage-area relation.

Four types of instrumentation have been used to provide an index of mean velocity in a measurement cross section. They are:

1. standard current meter,
2. deflection meter,
3. acoustic velocity meter, and
4. electromagnetic velocity meter.

The simplest instruments for recording velocity at a fixed point in the cross section are the standard current meter and the deflection meter. Their use is limited to the smaller streams and canals where the hazard of damage by boats or debris is minimal. The acoustic velocity meter integrates the velocity along a transverse line in the stream. It has been used in large rivers to provide an index to mean velocity in the measurement cross section. The use of an electromagnetic velocity meter is still (1980) in the experimental stage, and its use has been limited mostly to the smaller streams. Experimental work in the U.S.A. with the electromagnetic current meter has been largely in the use of the meter to obtain a continuous record of velocity at a point; in several European countries the experimental work has been largely in the use of the meter to obtain a continuous record of an index value of integrated mean velocity in the entire measurement cross section.

STANDARD CURRENT-METER METHOD

The use of an unattended standard current meter, securely anchored in a fixed position in the stream below the minimum expected stage, is attractive because of the simplicity of the device. The most desirable location for the meter will be in the central core of the flow, away from the influence of the banks or any other impediment to flow, where streamlines are parallel and at right angles to the measurement cross section. For streams of irregular alignment or cross section, it may be necessary to experiment with meter location to determine the most suitable site for the meter.

Any of several schemes may be used for recording revolutions of the current meter. For example, one might use a modification of the system for recording velocity that was described earlier for the moving-boat method of measuring discharge (see section in chapter 6 titled, "Rate Indicator and Counter"). In that system a clock-activated moving chart is automatically marked after each occurrence of a predetermined number of meter revolutions. In another system that might be used, the current meter would be connected to a digital recorder and at predetermined time intervals—say, 15 minutes—the number of revolutions that occurred in the preceding 15 minutes would be punched. In either system the current-meter rating equation would be used to convert revolutions per time interval to average velocity during the time interval.

As mentioned earlier, discharge measurements would be used to calibrate the stage-velocity-discharge relation. The cross-sectional areas shown by the discharge measurements would be used with stage to define the stage-area relation, which could be extrapolated by the use of data obtained in a field survey. The mean velocities shown by the discharge measurements would be used in a graphical relation of mean velocity to stage and to the index velocities indicated by the fixed current meter. Extrapolation of that relation would be aided if a vertical-velocity curve were obtained at the site of the index current meter at the time of each discharge measurement, and if the mean velocity in the vertical at the index meter site, as computed from each vertical velocity curve, were related to mean velocity in the measurement cross section. The use of such relations is illustrated in the hypothetical example that follows where, for simplicity, it is assumed that the relations can be expressed mathematically.

Assume that the vertical-velocity curves at the index site can consistently be defined by the equation,

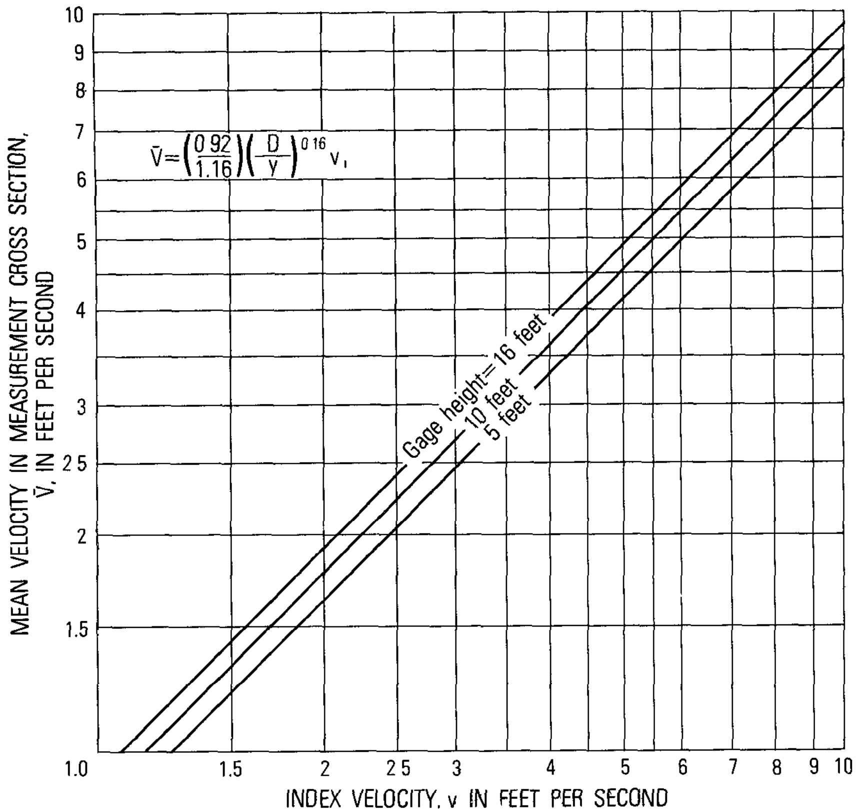


FIGURE 201.—*Hypothetical relation of mean velocity in measurement cross section to stage and index velocity.*

$$v_i = 1.16\ V_m\ (y/D)^{0.16},$$

where

V_m is the mean velocity in the vertical,

D is the depth, and

v_i is the velocity at a height, y, above the streambed.

Assume further that the ratio of mean velocity in the measurement cross section to mean velocity in the vertical at the index-meter site is consistently 0.92. It is also assumed that gage height and depth are equivalent, that stage is expected to range from 6 to 16 ft, and that the index meter is set at an elevation 5 ft above the streambed. Under those assumptions, the relation of mean velocity in the cross section to stage and index velocity would be that shown in figure 201. The mean velocity obtained by the use of figure 201 would be multiplied by the appropriate cross-sectional area to obtain the required discharge.

The utilization of a standard current meter to obtain an index of mean velocity has certain disadvantages that inhibit its use. The meter is susceptible to damage or impairment by submerged drift, but even where that hazard is negligible, there is a strong tendency for the meter to become fouled, after long immersion, by algae and other aquatic growth that becomes attached to the meter. Stoppage or impaired operation of the meter invariably results from the attachment of such growth, and constant servicing of the meter is usually a necessity. Suspended sediment in the stream also adversely affects the operation of an unattended current meter.

DEFLECTION-METER METHOD

GENERAL

Deflection meters are used to provide a velocity index in small canals and streams where no simple stage-discharge relation can be developed. The inability to develop a simple stage-discharge relation usually results from tide effect or from downstream gate operations to regulate the flow. At such gaging stations a recording stage-gage is operated in conjunction with the deflection meter.

The deflection meter has a submerged vane that is deflected by the force of the current. The amount of deflection, which is roughly proportional to the velocity of the current impinging on the vane, is transmitted either mechanically or electrically to a recorder. Values of the mean velocity of the stream are determined from discharge measurements, and mean velocity is then related to deflection and stage.

The ideal location for a deflection meter is in midchannel of a straight reach. However, it seldom is feasible to install the meter in midchannel; a site close to the bank of a straight reach is usually used.

Through the years, two basic types of deflection vane have evolved—the vertical-axis and the horizontal-axis types. The vertical-axis type has been most commonly used. Both types are described in the sections that follow.

VERTICAL-AXIS DEFLECTION VANE

The vertical-axis deflection vane is attached to a vertical shaft that is free to pivot about its vertical axis. Figure 202 shows two variations of the vertical-axis deflection vane. Vane A on the left is designed to sample a "point" or local velocity; vane B on the right is designed to integrate velocities throughout the greater part of a vertical. Vane B is used particularly in tidal streams where at times during a tidal cycle, stratification and density currents occur. At those times the denser salt water at the bottom of the channel flows upstream while fresh water in the upper zone starts to flow seaward.

Vane B extends from about 6 inches above the streambed to an elevation just below the water surface at low tide. While vane B is used in other circumstances, it cannot be used in a narrow channel where velocities are high, because a hydraulic jump may occur on the downstream side of the vane and affect the meter rating.

The force of the current acting on a vertical-axis vane turns the vertical shaft and the motion is transmitted to a graphic or digital

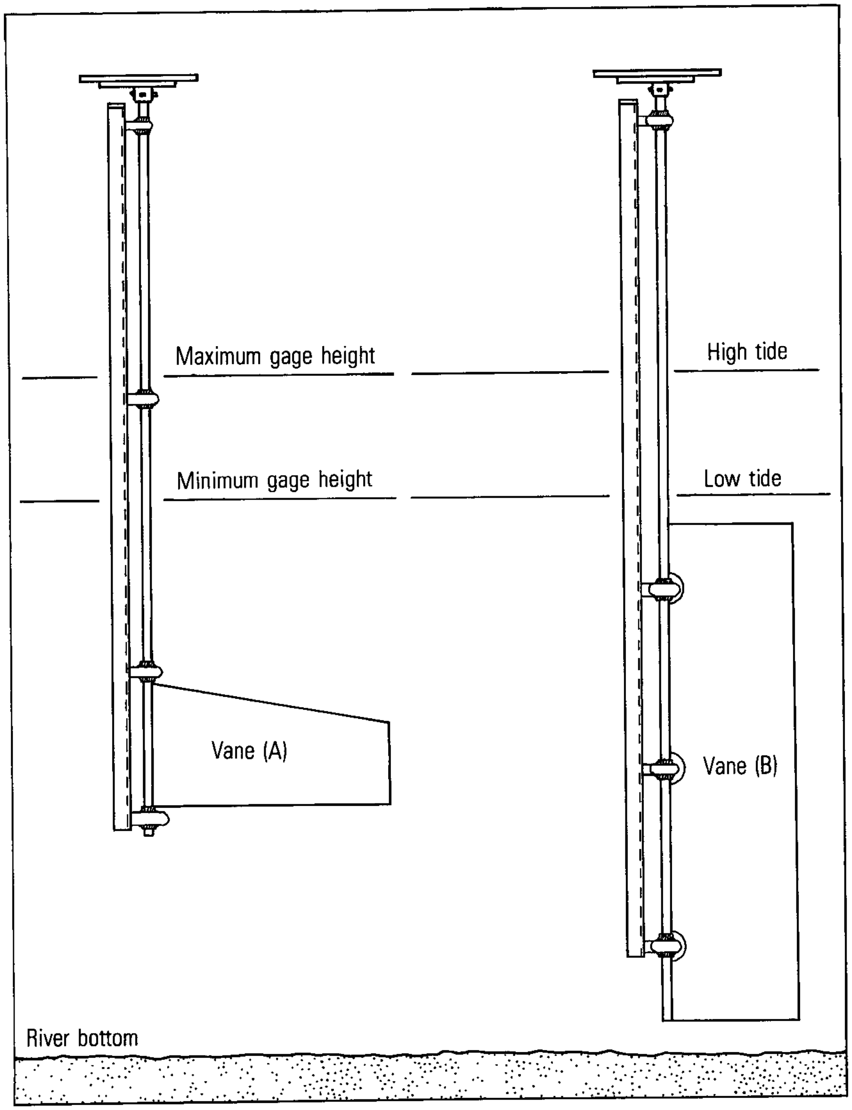


FIGURE 202.—Sketch of two types of vertical-axis deflection vanes.

recorder. A graphic recorder is shown in the system in figure 203. The vertical shaft also has an index plate fastened to it, and to the index plate is attached a counterweighted cable. When the velocity is zero, no lateral force is exerted on the vane and the counterweight will hold the vane in a position that is perpendicular to the direction of flow. A 15- to 20-pound counterweight is generally used with most vanes, but high velocities and (or) the use of a large vane may necessitate the use of a heavier counterweight in order to provide the counter-torque necessary to resist the rotary movement of the vane.

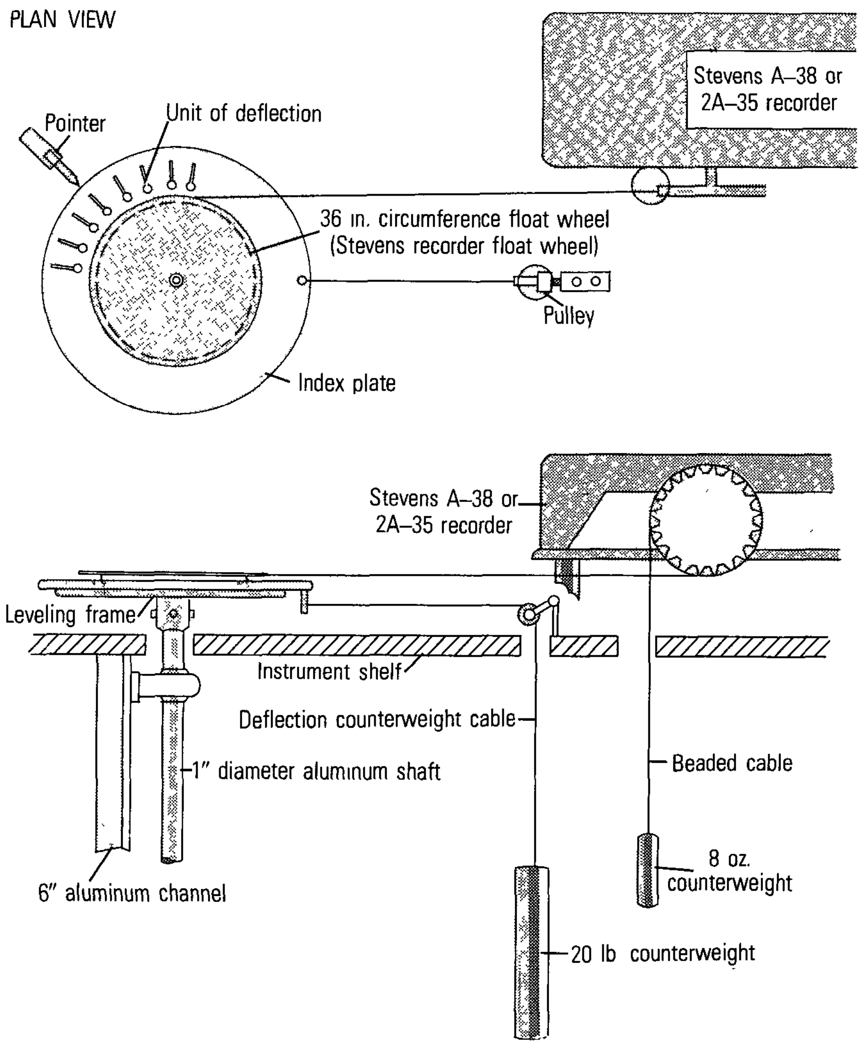


FIGURE 203.—Plan and front elevation views of a vertical-axis deflection meter attached to a graphic recorder.

A pointer for indicating the units of deflection on the index plate is attached to the instrument shelf. The index plate is calibrated by placing the recorder pen at zero position on the recorder and locking it there. The index plate is then scribed with a mark opposite the pointer. The index plate is rotated until the pen moves 1 inch on the recorder chart and another mark is scribed opposite the pointer. This process is repeated until marks for the full range of deflection have been scribed on the index plate and numbered. These units of deflection on the calibrated index plate are the reference marks for checking and resetting the recorder pen on future inspections of the deflection meter.

The vertical-axis deflection vane does have several drawbacks, the most serious of which is its tendency to collect floating debris which, in turn, affects the calibration of the vane. Another problem is the high degree of bearing friction resulting from the weight and bearing system of the vane assembly; the friction causes insensitivity at low velocities. In addition, removal of the vane for service and repair is difficult because of the weight involved. Furthermore, the projection of the vane assembly above the water surface makes it susceptible to damage by ice.

HORIZONTAL-AXIS DEFLECTION VANE

A recent development is the horizontal-axis or pendulum type deflection vane. This type is designed to overcome many of the difficulties mentioned in connection with the vertical-axis vane. For example, the pendulum vane can be installed with the mount totally submerged, thus reducing the possibility of collecting debris at or near the water surface where such debris is usually found. Its light weight and simplified bearing design greatly reduce the bearing friction, thus improving its low-velocity characteristics. Because no parts protrude from the water, there is little danger of damage by ice.

The pendulum-type vane consists of a flat triangular plate, suspended from above, that pivots about a horizontal axis located at the apex of the triangle (fig. 204). Interchangeable weights are available for attachment to the base of the triangular plate, thereby providing for optimum adjustment to the desired veolcity range. The location and design of the weights serve the additional purpose of reducing fluctuations caused by eddy shedding.

The force of the current acting on the horizontal-axis vane causes it to deflect. The angle formed by the vane itself and a small reference pendulum sealed within the pivot chamber is the angle of deflection. A potentiometer is positioned to generate an electrical signal that is proportional to the angle of deflection. The voltage that is generated is converted to a proportional shaft position for recording by a digital or graphic recorder.

It can be demonstrated that when the horizontal-axis vane is deflected by flowing water and the system is in mechanical equilibrium, the following relation exists between velocity of the water, angle of deflection, and the physical properties of the vane:

$$V^2 = \left(\frac{2WL_M}{\rho AL_A}\right)\left(\frac{\sin\Theta}{C_D\cos^2\Theta + C_L\sin^2\Theta}\right),$$

where V is horizontal velocity of the water,
W is weight of the pendulum in water,

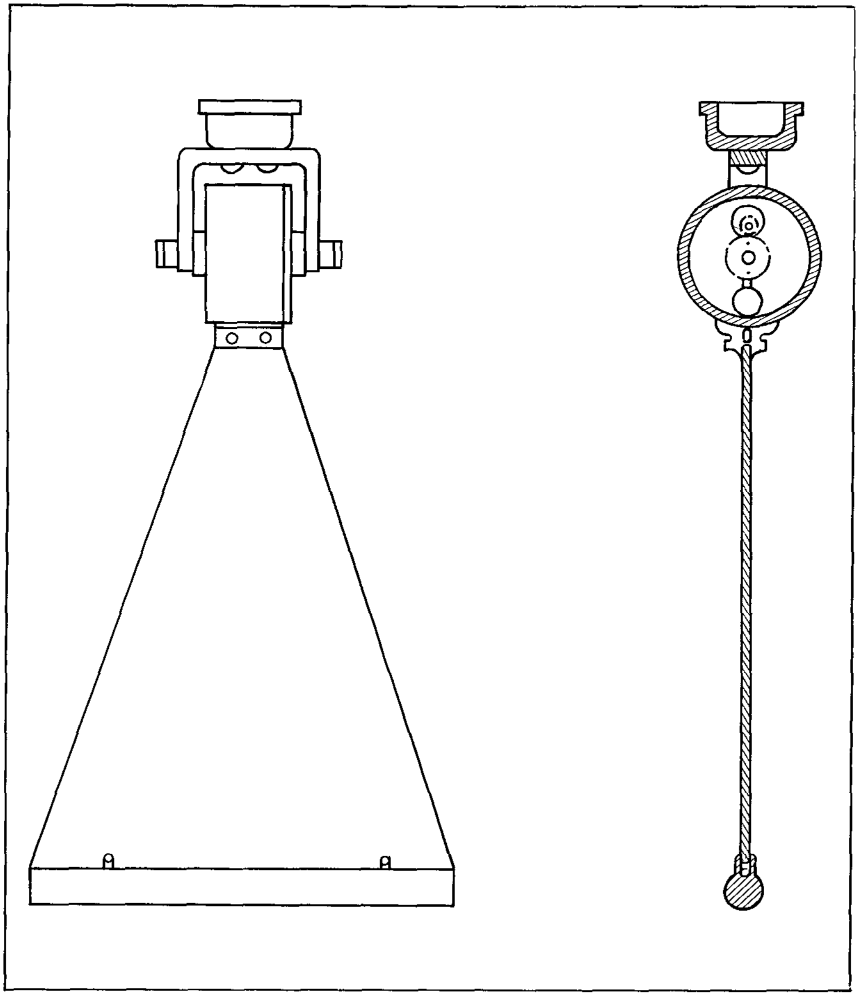


FIGURE 204.—Sketch of a pendulum-type deflection vane.

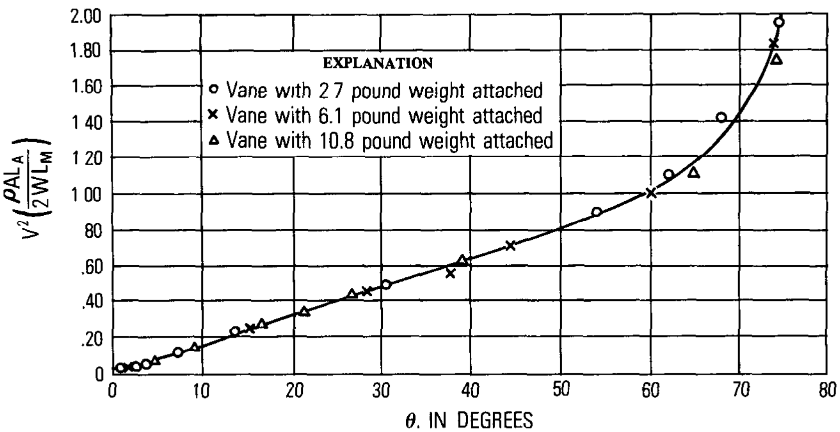


FIGURE 205.—Calibration curve for pendulum-type deflection vane.

ρ is the density of water,
A is the area of the vane,
L_M is the distance from the pivot point to the center of mass,
L_A is the distance from the pivot point to the center of the area,
Θ is the angle of deflection,
C_D is the coefficient of drag, and
C_L is the coefficient of lift.

Figure 205 is a graphical presentation of the above relation that can be used for selecting the weight needed for a given velocity range.

EXAMPLES OF STAGE-VELOCITY-DISCHARGE RELATIONS BASED ON DEFLECTION-METER OBSERVATIONS

Figure 206 shows a graphic-recorder chart for a gaging station in Florida where tidal flow reverses direction. The upper pen trace shows the stage at various times during the tide cycle for the period May 4–6, 1962. The lower pen trace shows the deflection units recorded during the same period. Zero flow is represented by a reading of four units on the deflection scale. Flow is in the seaward direction when the deflection is less than 4 units (hachured part of deflection graph in fig. 206); flow is in the inland direction when the deflection is greater than four units.

The rating curves shown in figure 207 were derived from discharge measurements. The units of deflection are indicative of velocity in a single vertical in the channel, having been obtained from a vertical-axis deflection meter equipped with vane B (fig. 202). The velocity curve shows the relation of deflection units to measured mean velocity in the channel; stage was not a factor in the relation because of the limited range (2 ft) in stage. For deflections of less than four units, velocity is negative, meaning that flow is in the seaward direction.

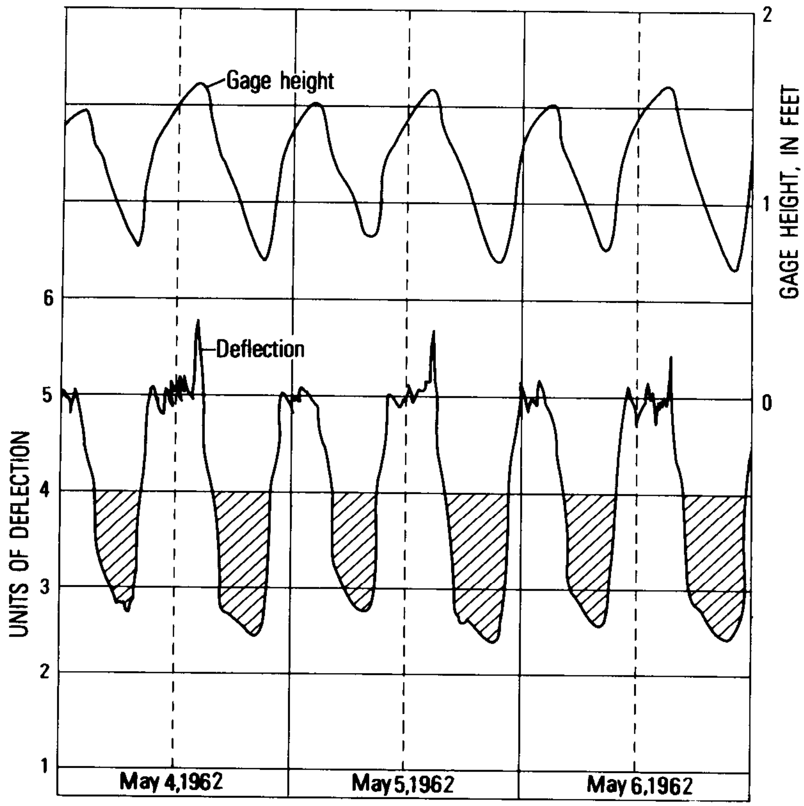


FIGURE 206.—Recorder chart for a deflection-meter gaging station on a tidal stream.

The stage at the time of discharge measurements was used to construct the area curve, which relates stage to cross-sectional area. Discharge is computed by multiplying area by mean velocity; negative values of discharge indicate seaward flow and positive values indicate inland flow.

Figure 208 shows the rating for a gaging station at the outlet of a large natural lake, immediately downstream from which are gates that regulate the flow for hydroelectric-power generation farther downstream. The deflection meter at the station is of the vertical-axis type and is equipped with vane A (fig. 202) to measure deflection at a "point" in the rectangular channel. Instead of deriving separate relations of stage versus cross-sectional area and deflection versus mean velocity, a single graphical relation, in the form of a family of curves, was derived for discharge versus stage and deflection. A preliminary study had shown that mean velocity was related to a combination of deflection and stage. The ratings for values of deflection other than

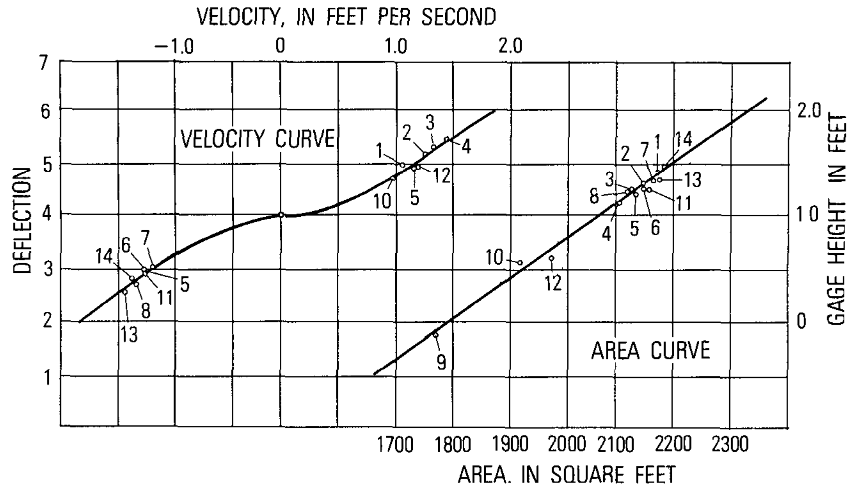


FIGURE 207.—Rating curves for a deflection-meter gaging station on a tidal stream.

those shown by the individual curves in figures 208 were obtained by interpolation between curves. Most of the 40 discharge measurements, which are shown by the small circles in figure 208, depart from the interpolated ratings by no more than 2 percent.

The use of separate relations for area and mean velocity is considered preferable to the use of a single compound relation for discharge, as was done in figure 208, because separate analysis of two components of discharge is simpler. Shifts in the discharge rating—that is, differences between measured and computed discharge—are also more easily analyzed when separate relations for area and mean velocity are prepared.

ACOUSTIC VELOCITY-METER METHOD

DESCRIPTION

Acoustic velocity meters are particularly advantageous in obtaining a continuous record of the discharge of large rivers in those situations where neither a simple stage-discharge relation nor a stage-fall-discharge relation can be applied satisfactorily. Those situations, as mentioned in the first section of this chapter, usually involve tidal flow or flow affected by hydroelectric-power generation, where the acceleration head in the equations of unsteady flow (p. 391) cannot be ignored. Acoustic velocity meters operate on the principle that the velocity of sound propagation through a fluid in motion is the algebraic sum of the fluid velocity and the acoustic propagation rate through the fluid. Thus acoustic pulses transmitted in the direction of flow will traverse a given path in shorter time than will acoustic pulses transmitted in opposition to the flow. The difference in transit

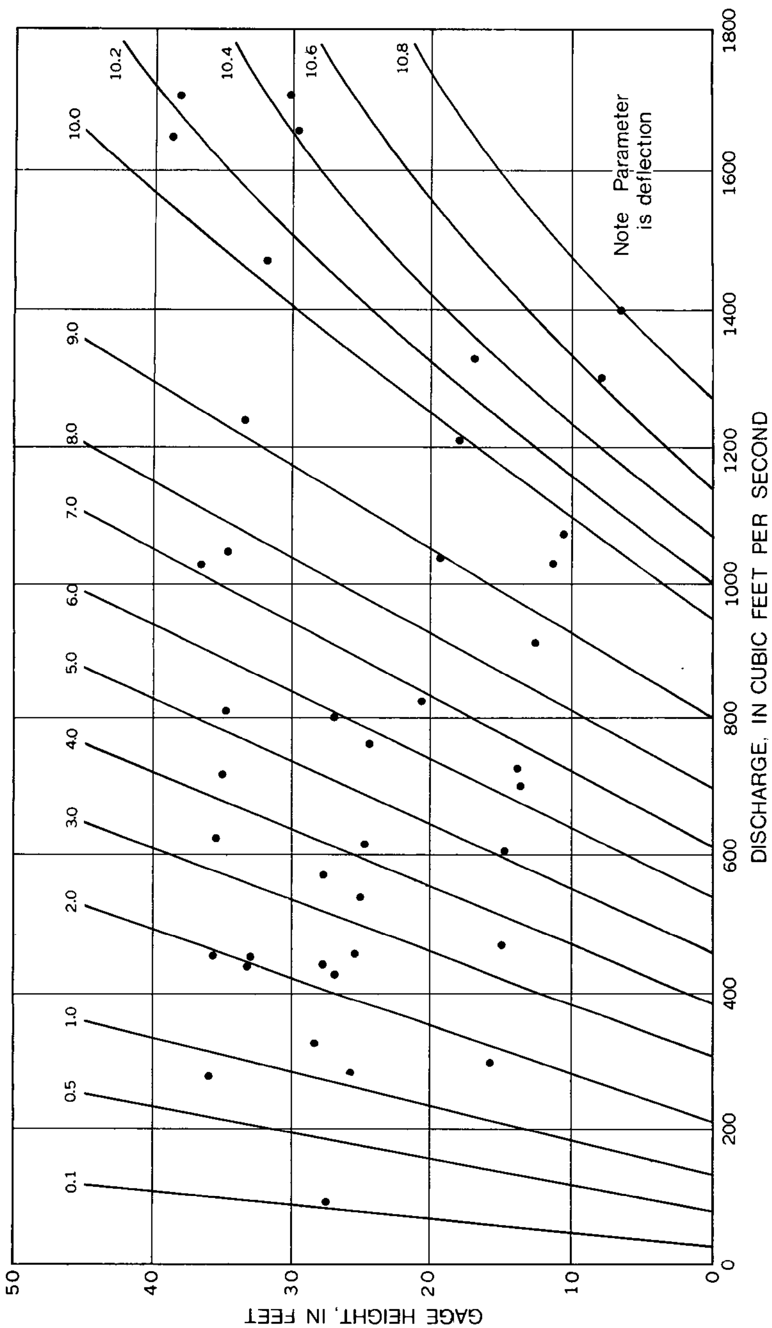


FIGURE 208.—Rating curves for a deflection-meter gaging station on Lake Winnipesaukee outlet at Lakeport, N.H.

times provides a measure of the line velocity—that is, the average value of the water velocity at the elevation of the acoustic path—and the line velocity is a satisfactory index of mean velocity in the channel. Because the transducers that transmit and receive the acoustic pulses are installed in the stream at a fixed elevation, the relation of line velocity to mean velocity varies with stage. The stage data required for the velocity relation are obtained from the stage recorder, which also provides an index of cross-sectional area.

Differences exist among the various acoustic-velocity metering systems that are commercially available, but the differences are not vital, and only one system will be briefly described. The major components of the acoustic monitoring system are two submerged transducers (fig. 209) and a console (fig. 210) housed on the streambank and electrically connected to both transducers. The two transducers, one on each side of the channel, are installed at the same elevation—an elevation that is below the lowest expected stage of the stream—on a diagonal path across the stream. The transducers convert electrical impulses generated in the console into sound pulses that travel through the water. They also convert the received sound pulses back into electrical signals. The console contains: the operating controls, the signal-generating and -receiving circuits (acoustic unit), the system clock that provides the basic timing pulses for the system and also furnishes the time-of-day readout, the digital processor (digital unit) that controls the transmission of acoustic pulses and performs the computations of the velocity index, and the velocity-index display. The velocity index is a measure of the line velocity. In the U.S.A., power for the system is usually furnished by a 110-volt alternating-current power supply.

Although acoustic-velocity meter systems are currently (1980) operational, the techniques and instrumentation are relatively new and are continually being improved. The cost of an acoustic-velocity meter installation is roughly 10 times that of a conventional gaging station. For that reason the acoustic-velocity method is limited to those sites where an accurate record of discharge is unattainable by the more conventional methods, but is of great value for water-management purposes.

THEORY

Measurement of the water velocity is possible because the velocity of a sound pulse in moving water is the algebraic sum of the acoustic propagation rate and the component of velocity parallel to the acoustic path. Reference is made to figure 211 in the following derivation of the mathematical relations of the system.

The traveltime of an acoustic pulse originating from a transducer at A and traveling in opposition to the flow of water along the path

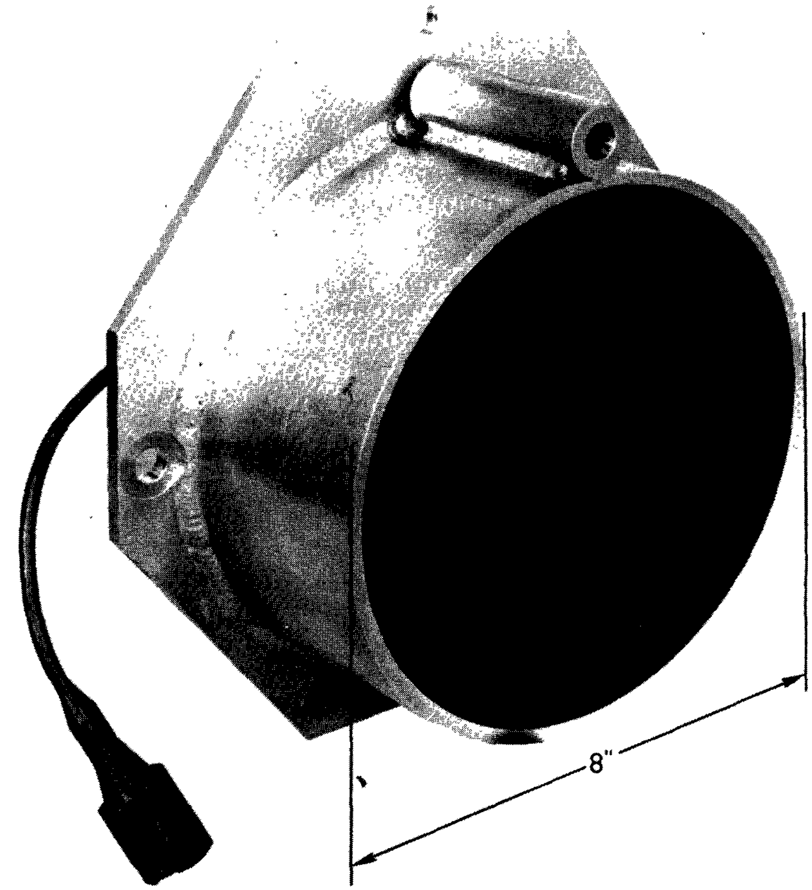


FIGURE 209.—Transducer.

A–C can be expressed as

$$T_{AC} = \frac{B}{c - V_p} , \tag{91}$$

where

c is the propagation rate of sound in still water,
B is the length of the acoustic path from A to C,
T_{AC} is traveltime from A to C, and
V_p is average component of water velocity parallel to the acoustic path.

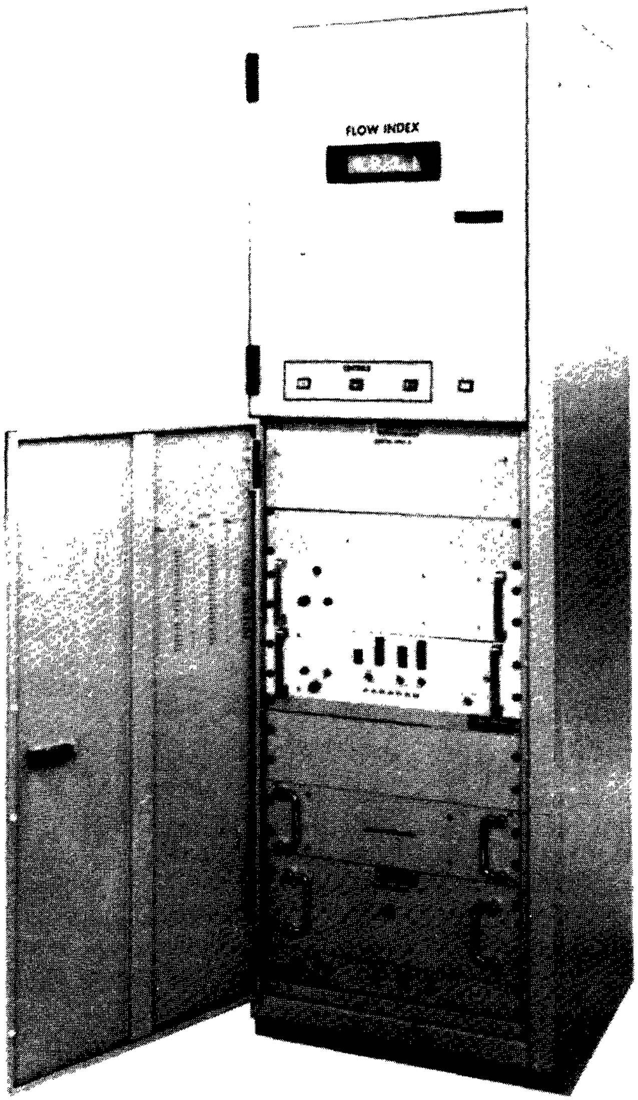


FIGURE 210.—Console.

Similarly, the traveltime for a pulse traveling with the current from C to A is

$$T_{CA} = \frac{B}{c + V_P}, \tag{92}$$

where T_{CA} is traveltime from C to A.

ΔT is the difference between T_{AC} and T_{CA}; therefore,

$$\Delta T = \frac{B}{c - V_P} - \frac{B}{c + V_P} = \frac{2BV_P}{c^2 - V_P^2} \quad ; \tag{93}$$

and since $V_P^2 <<< c^2$,

$$\Delta T \cong \frac{2BV_P}{c^2} \quad , \tag{94}$$

or

$$V_P \cong \frac{\Delta T c^2}{2B} \quad . \tag{95}$$

Both ΔT and c in equation 95 can be defined by measurement of the traveltimes of acoustic signals transmitted in each direction between transducers, c being computed by solving equations 91 and 92 simultaneously. The digital processor in the console can be scaled to produce a velocity index (I) that is equal to V_P. In some of the older systems used in the U.S.A. the velocity index was not scaled to equal V_P, but instead the velocity index was directly proportional to V_P, so that

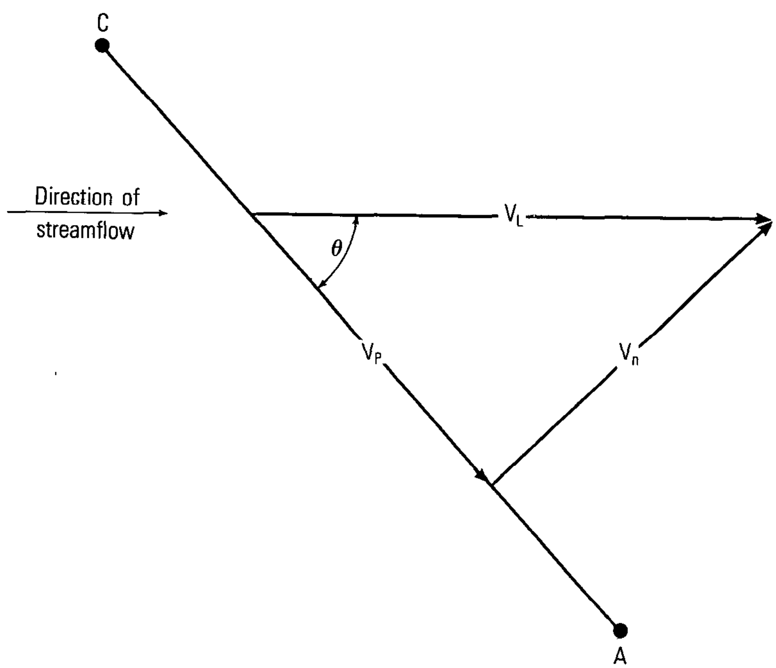


FIGURE 211.—Sketch to illustrate operating principles of the acoustic velocity meter.

$$V_P = C_1 I, \tag{96}$$

where C_1 is a constant of proportionality.
In the continuing discussion of "Theory", equation 96 will be used with the understanding that $C_1 = 1.00$ in some of the acoustic-velocity meter systems.

Figure 211 shows that

$$V_L = \frac{V_p}{\cos \Theta}, \tag{97}$$

where V_L is the average water velocity at the elevation of the acoustic path, and Θ is the acute angle between the streamline of flow and the acoustic path, AC.

By combining equations 96 and 97,

$$V_L = \left(\frac{C_1}{\cos\Theta}\right) I \tag{98}$$

Experimentation has shown V_L to be a stable index of $\overline{V}$, the mean velocity in the cross section at right angles to the streamlines of flow. The relation between V_L and $\overline{V}$ can be expected to vary with stage because V_L is a measure of the mean velocity along a line at a fixed elevation in the cross section. As the stage rises, the position of this line is moved downward in the cross section relative to the total depth, and resultant changes in the velocity distribution in the vertical column cause a change in the ratio between V_L and $\overline{V}$. Correlation of the ratio $V_L/\overline{V}$ with stage is accordingly necessary, and $\overline{V}$ can be expressed as follows:

$$\overline{V} = C_2 V_L, \tag{99}$$

where C_2 is a function of stage.

The basic equation for discharge (Q) is

$$Q = \overline{V} A, \tag{100}$$

where A is area of the cross section.

By substituting in equation 100, terms given in equations 98 and 99, the following equation is obtained:

$$Q = I \left(\frac{C_1 C_2}{\cos \Theta}\right) A. \tag{101}$$

When the symbol K is substituted for $\frac{(C_1 C_2)}{\cos \Theta}$ in equation 101, the result is

$$Q = KIA. \tag{102}$$

K varies with stage including, as it does, C_2 which is a function of stage.

To calibrate the system, discharge measurements are made to obtain measured values of A and $\overline{V}$. The measured values of A are correlated with stage to obtain a graphical stage-area relation. Measured values of $\overline{V}$ are divided by concurrent values of I, recorded by the console digital processor, to obtain concurrent values of K. Those values of K are correlated with stage to obtain an empirical graphical relation of K to stage. Such a relation is shown in figure 212.

To compute the discharge for any given value of I, the concurrent value of stage is first read. That value of stage is then used in the above grapnical relations to obtain the corresponding values of A and K. In a final step the values of K, I, and A are multiplied together, in accordance with equation 102, to obtain the required value of discharge.

Newer acoustic-meter velocity systems that have been designed provide a readout of discharge after the calibration coefficients have been determined. The additional calibration coefficients needed are provided by substituting mathematical relations of A to stage and K to stage, in place of the graphical relations discussed above. The computation of discharge is based on the following two assumptions:

1. The relation between area (A) and stage (H) is stable and can be adequately defined by the second-order equation,

$$A = C_1 + C_2H + C_3H^2, \tag{103}$$

where C_1, C_2, and C_3 are constants.

2. The ratio (K) between mean stream velocity ($\overline{V}$) and the velocity index (I), which is equal to, or linearly related to, the line velocity (V_p), can be defined by the second-order equation,

$$K = \overline{V}/I = C_4 + C_5H + C_6H^2, \tag{104}$$

where C_4, C_5, and C_6 are constants.

If sufficient data from discharge measurements are available, the "best" values of C in equations 103 and 104 can be computed from a least-squares solution of each of the equations. Usually, however, the C values in the two equations are obtained from the graphical relations of A versus H and K versus H. That is done by first selecting the coordinates of three significant points on one of the graphical relations, and then substituting those values in the appropriate equation—equation 103 when the area relation is used. The three resulting simultaneous equations are solved to produce the required C values. The process is then repeated, using equation 104 for the K relation. The six C values so obtained are then entered in the program for computing discharge. Discharge is computed as before, in

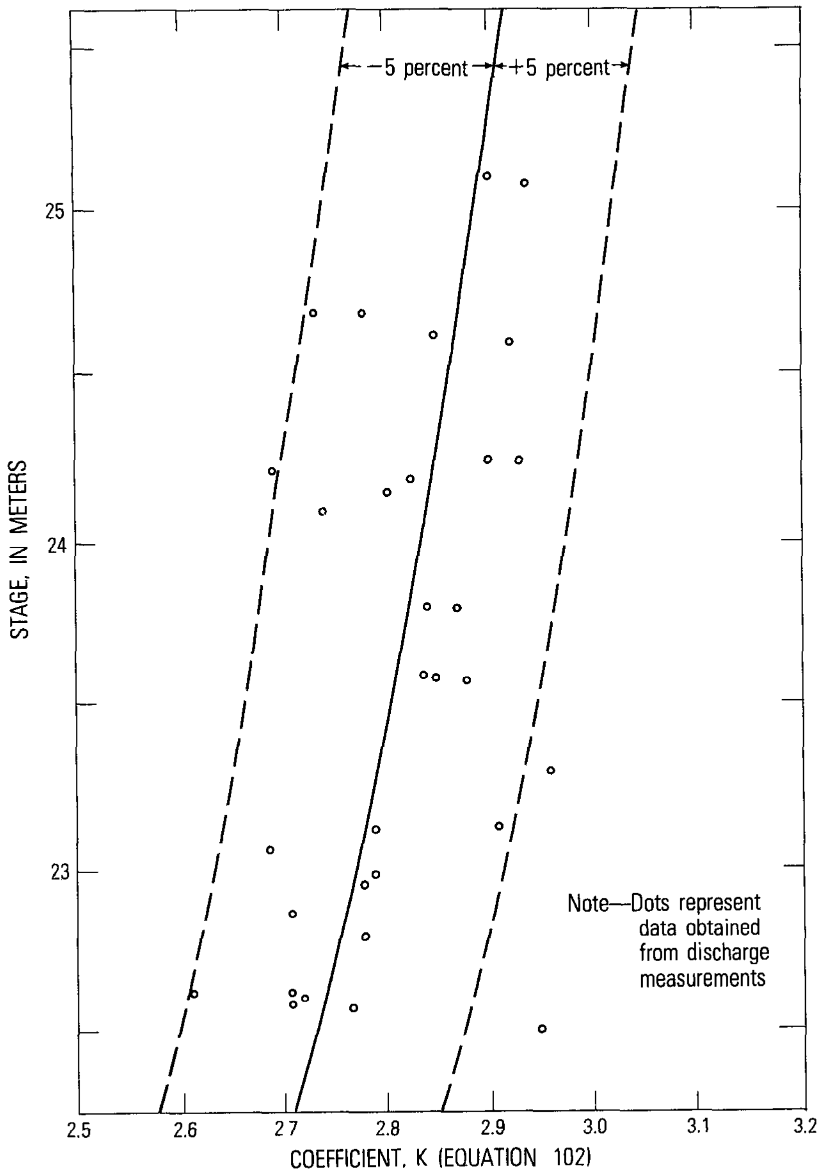


FIGURE 212.—Relation between stage and mean-velocity coefficient, K, for the acoustic-velocity meter (AVM) system, Columbia River at The Dalles, Oreg.

accordance with equation 102, except that the computations are performed in the console digital processor. The digital processor uses the C values, along with concurrent values of I and H, to make the required computations and provide a readout of K, I, A, H, and Q.

It would be simple, of course, to multiply I by the product of equations 103 and 104 and thereby obtain a single equation for Q. The result would be a fourth order equation of the form,

$$Q = I(k_1+k_2H+k_3H^2+k_4H^3+k_5H^4), \tag{105}$$

in which the k constants represented combinations of the C constants from equations 103 and 104. The "best" values of the constants could be obtained by a least–squares solution of equation 105, using measured values of Q and concurrent values of I and H. The use of equation 105 would simplify the computation of discharge, but the analyst would then lose much of his ability to analyze error sources in the calibration of shifts in the basic relations. It is therefore recommended that equations 103 and 104 be used rather than equation 105.

EFFECT OF TIDAL FLOW REVERSAL ON RELATION OF MEAN VELOCITY TO LINE VELOCITY

The value of C_2 in equation 99, $\overline{V} = C_2V_l$, varies only with stage in unidirectional flow. In streams where the direction of flow reverses in response to tide, the value of C_2 may vary not only with stage, but also with the four phases of the tide cycle. For such streams numerous discharge measurements, preferably by the moving-boat method (chap. 6), are required to evaluate C_2 for each of the tide phases. In using the moving-boat method of discharge measurement, it is necessary to determine a velocity coefficient for each individual discharge measurement and that is done by continuously defining the vertical-velocity distribution at several strategically located verticals that are representative of the main portion of streamflow. (See section in chapter 6 titled, "Adjustment of Mean Velocity and Total Discharge.")

The results of an evaluation of C_2 for a particular cross section in the Sacramento River in California are given in table 22 (Smith, 1969, p. 11–18). Column heading, $\overline{C}_2$, in table 22 refers to the mean values of C_2; column heading, s, refers to the standard deviations of C_2 values. Figure 213 is a plot of the data from columns headed, $\overline{V}$ and $\overline{C}_2$, in table 22.

ORIENTATION EFFECTS AT ACOUSTIC-VELOCITY METER INSTALLATIONS

EFFECT OF ACOUSTIC-PATH ORIENTATION ON ACCURACY OF COMPUTED LINE VELOCITY (V_l)

The basic accuracy or resolution of a given acoustic-velocity meter (AVM) system is controlled principally by the accuracy with which the arrival times of the acoustic pulses can be discriminated and by the accuracy of the timing circuitry used to measure elapsed times. A related factor that affects the accuracy of results obtained with a par-

TABLE 22.—*Variation of* C_2 *with tidal phase*

[$\overline{C}_2$ are mean values of C_2; s are standard deviations of C_2 values]

Velocity range (ft/s)	Increasing ebb			Decreasing ebb			Increasing flood			Decreasing flood		
	$\overline{V}$ (ft/s)	$\overline{C}_2$	s	$\overline{V}$ (ft/s)	$\overline{C}_2$	s	$\overline{V}$ (ft/s)	$\overline{C}_2$	s	$\overline{V}$ (ft/s)	$\overline{C}_2$	s
<1.00	0.90	0.961	(1)	0.84	1.030	0.031	0.87	1.005	0.030	----	----	----
1.00–1.40	1.16	.957	0.035	----	----	----	1.21	1.006	(1)	1.11	0.983	(1)
1.41–1.80	1.52	.965	(1)	1.41	1.040	(1)	1.75	.982	(1)	1.58	.997	0.022
1.81–2.20	1.96	.981	.011	2.02	1.019	.017	2.09	.970	(1)	2.08	.991	.020
2.21–2.60	2.44	.978	.014	2.46	1.019	.018	2.43	.985	.011	2.43	.992	.009
>2.60	2.75	.985	.009	2.76	.990	.014	2.92	.982	.007	2.84	.982	.009

[1]Not computed, only one or two values of C_2 available for determining $\overline{C}_2$

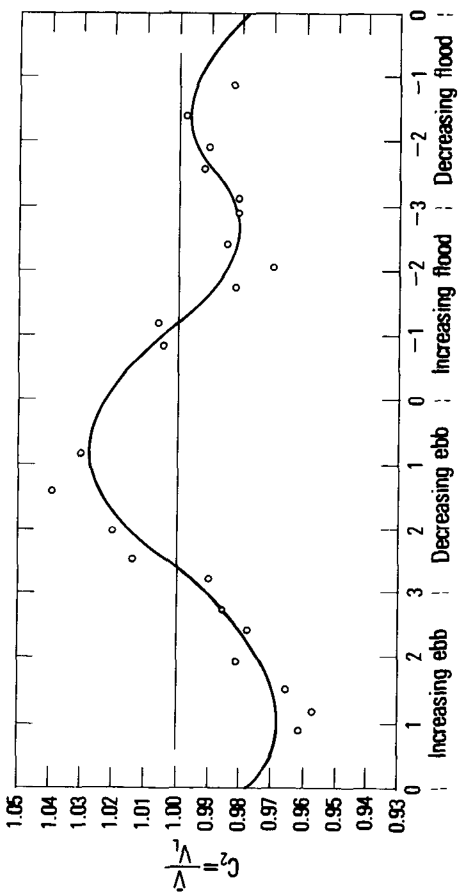


FIGURE 213.—Relation between C_2 and velocity and tide phase.

ticular *AVM* system is the orientation of the acoustic path with respect to the streamlines of flow. The effect of path orientation will now be examined for one of the *AVM* systems used in the U.S.A. It is assumed that no changes in the basic circuitry are made for operations over acoustic paths of various orientations, that the streamlines at all times and stages are parallel and their direction is invariant, and that acoustic performance is thoroughly reliable.

From figure 211

$$V_P = V_L \cos \Theta \tag{97a}$$

Insertion of the resolution error (R_e) for the system in equation 97a yields

$$V_P = V_L \cos \Theta \pm R_e$$

or,

$$V_L = \frac{V_P}{\cos \Theta} \pm \frac{R_e}{\cos \Theta}\,. \tag{106}$$

The last term in equation 106 represents the error (E) in computed values of V_L, meaning that

$$E = \pm \frac{R_e}{\cos \Theta} \tag{107}$$

In other words, for a given *AVM* system, the error in computed values of V_L decreases as Θ decreases.

According to the claim of the manufacturer of the *AVM* system under discussion, inaccuracy (E) attributable to the resolution error is ±0.05 ft/s when angle Θ is 45°. From equation 107, the implication is that the resolution error (R_e) equals ±0.05 cos Θ, or ±0.035 ft/s. Table 23 was computed from equation 107 using the above value of R_e. Because the error in computed values of V_L is independent of the magnitude of V_L, the greatest percentage errors in computed velocity occur at low velocities for any given orientation of the acoustic path.

EFFECT OF VARIATION IN STREAMLINE ORIENTATION

If an *AVM* system were located a short distance downstream from the confluence of two streams, as shown in figure 214, the direction of

TABLE 23.—*Error in computed* V_L, *attributable to resolution error, for various acoustic-path orientations, for a given* AVM *system*

Path-orientation angle Θ, in degrees	Error in V_L, in ft/s
30	±0.04
45	± .05
60	± .07
70	± .10
80	± .20

the streamlines of flow at the gage site could be expected to vary with the proportion of total discharge contributed by the tributary stream. When the tributary flow is low, the angle between streamlines and acoustic path is Θ, V_L is the velocity normal to the cross section whose area is A, and a value of V_p is recorded by the AVM;

$$V_L = \frac{V_p}{\cos \Theta} \quad , \tag{108}$$

and

$$Q_{AVM} = AV_L \tag{109}$$

If stage and discharge remain constant, but the proportion of flow from the tributary increases significantly, the angle Θ between the streamlines of flow and the acoustic path will increase by an increment ϕ, but V_L will remain constant because the discharge and stage remain constant. A value of V'_p will now be recorded by the AVM, where

$$V'_p = V' \cos(\Theta+\phi) \tag{110}$$

But,

$$V' = \frac{V_L}{\cos \phi} \tag{111}$$

Therefore,

$$V'_p = \frac{V_L \cos(\Theta+\phi)}{\cos \phi} \tag{112}$$

However the discharge has not changed. If the AVM system had been calibrated under conditions where, for the given discharge and given stage, the angle between streamlines and acoustic path was Θ, the AVM system will be unaware of the increase in angle from Θ to $(\Theta+\phi)$, and the discharge will be computed as

$$Q'_{AVM} = \frac{AV'_p}{\cos \Theta} = \frac{AV_L \cos(\Theta+\phi)}{\cos \Theta \cos \phi} \tag{113}$$

But the true AVM discharge (line velocity times area) is that shown by equation 109. Therefore the ratio between computed AVM discharge for the condition of the angle being $(\Theta+\phi)$ and the true AVM discharge is,

$$\frac{Q'_{AVM}}{Q_{AVM}} = \frac{\left(\dfrac{AV_L \cos(\Theta+\phi)}{\cos \Theta \cos \phi}\right)}{AV_L} \tag{114}$$

$$= \frac{\cos(\Theta+\phi)}{\cos \Theta \cos \phi}$$

$$= 1 - \tan\Theta \tan\phi \tag{115}$$

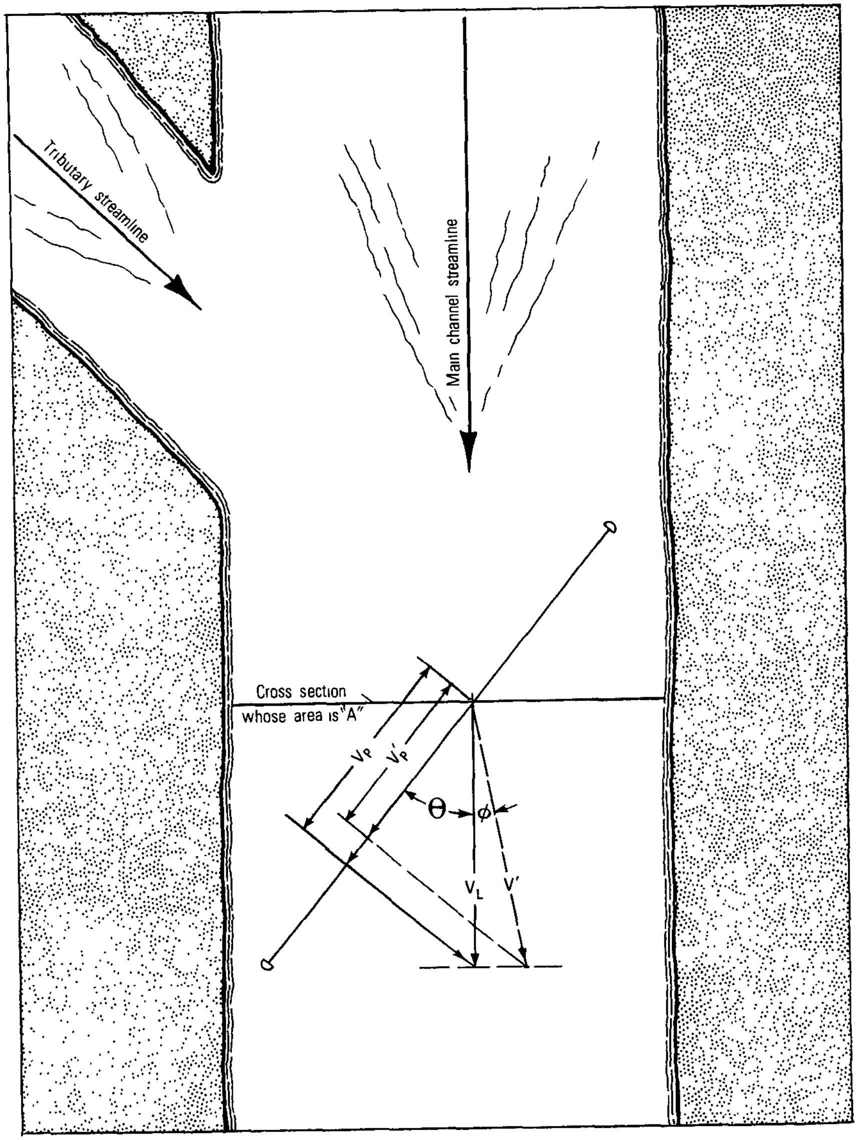


FIGURE 214.—Possible variation in streamline orientation.

Equation 115 is evaluated in table 24 for acoustic path orientations (Θ) ranging from 30° to 60°, and for streamline variations (ϕ) ranging from −4° to +4°.

As a general rule one should avoid installing an *AVM* system immediately downstream from the confluence of two streams. It is true

TABLE 24.—*Ratio of computed discharge to true discharge for various combinations of Θ and φ*

Θ	Ratios for the values of φ indicated below −4°	−3°	−2°	−1°	0°	+1°	+2°	+3°	+4°
30°	1.040	1.030	1.020	1.010	1.000	0.990	0.980	0.970	0.960
45°	1.070	1.052	1.035	1.017	1.000	.983	.965	.948	.930
50°	1.083	1.062	1.042	1.021	1.000	.979	.958	.938	.917
60°	1.121	1.091	1.060	1.030	1.000	.970	.940	.909	.879

that calibration of the system will be unaffected if, for each value of total discharge, there exists a particular ratio of tributary discharge to mainstream discharge. However, if that ratio is not constant for a given total discharge, error will be introduced in the calibration of the system, and therefore, in the computation of discharge.

FACTORS AFFECTING ACOUSTIC-SIGNAL PROPAGATION

In the installation of an *AVM* system, consideration must be given to the factors that affect the propagation of the acoustic signal through the water. Refraction or reflection of the acoustic beam away from the selected path or attenuation of the acoustic signal may result from:

1. temperature gradients in the stream,
2. boundary proximity,
3. air entrainment,
4. sediment concentration, and
5. aquatic vegetation.

TEMPERATURE GRADIENTS

Periodic loss of signal at some *AVM* installations where the transducers were relatively close to the water surface of a deep stream have led engineers to theorize that the development of even extremely small temperature gradients in the water column may cause refraction of the acoustic signal. In streams where mixing is poor, changes in solar radiation and air temperature could conceivably cause such gradients to develop. It has been reasoned that location of the acoustic path near mid-depth of the stream should minimize temperature gradients caused by variations in temperature or possibly by heat exchange between the water and channel perimeter.

BOUNDARY PROXIMITY

When the acoustic path is located near the water surface or near the streambed, part of the acoustic signal will be reflected from the boundary (air-water interface or streambed). The reflected component may arrive at the receiving transducer almost simultaneously with, but out of phase with, the primary pulse. In extreme cases, signals may be almost completely blanked out. This phenomenon is related to the ratio of path length to distance to a boundary and to the frequency of the transmitted signal.

The above considerations, combined with the possibility of the thermal effects discussed above, have led the designers of some *AVM* systems to develop the criteria curves for *AVM* site selection shown in figure 215. The curves indicate the performance that can probably be expected from some systems in a given channel geometry when the transducer elevation is set at mid-depth. The terms "excellent" and "acceptable" are relative, and their significance is dependent upon the reliability requirements at the site. The curves show that the depth of water required increases as the path length increases. For example, for a path length of 500 ft, excellent acoustic performance would be expected for depths greater than 18 ft and acceptable performance would be anticipated for depths between 10 and 18 ft, but for depths less than 10 ft, on-site investigation of the characteristics of acoustic transmission would be necessary. For a path length of 1,000 ft, these depth ranges change to 34 ft or more for excellent transmission and from 19 to 34 ft for acceptable transmission. On-site studies would be required for depths less than 19 ft. The curves in figure 215 should not be construed as providing all the information

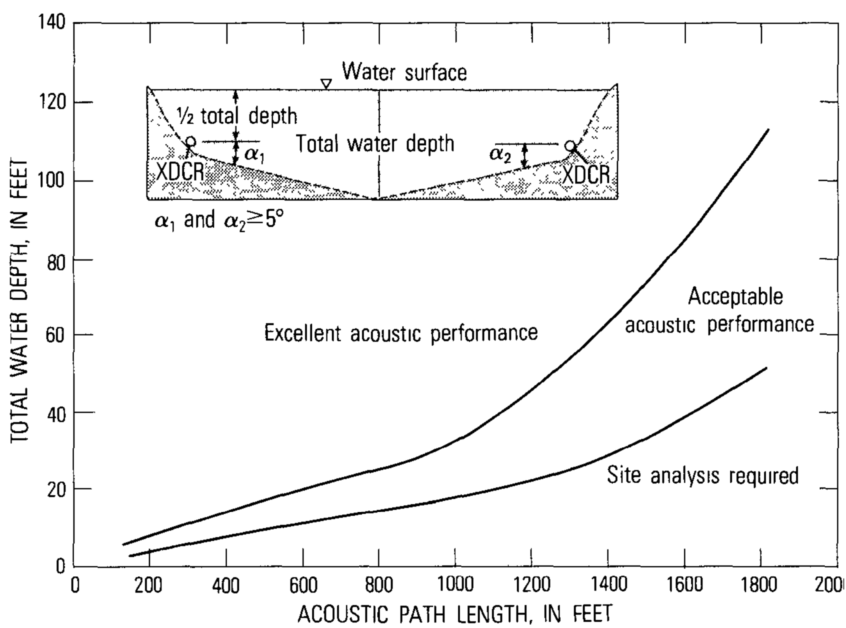


FIGURE 215.—Curves used as a preliminary guide for AVM site selection, based solely on consideration of channel geometry.

required for assessment of the potential for utilizing an *AVM* system. By these criteria, broad, shallow channels would seem to be questionable sites, but it is possible that further developments in transducer design and system characteristics may provide reliable performance in such channels also.

AIR ENTRAINMENT

Little quantitative information is available concerning the attenuation of acoustic signals by air bubbles entrained in the water, but the effect of air entrainment has been observed downstream from dams where the falling water becomes highly aerated. Bubbles formed at such sites may remain entrained in the water for a considerable distance downstream, and they absorb and reflect the acoustic signal much as fog absorbs and reflects a beam of light. The highly absorptive characteristics of water with entrained air precludes satisfactory operation of *AVM* systems, and locations close to spillways or other sources of air entrainment should consequently be avoided.

SEDIMENT CONCENTRATION

The degree of attenuation of signal strength caused by the reflection and scatter of the acoustic signals from sediment particles suspended in the stream has not been fully documented. The attenuation is influenced not only by suspended-sediment concentration, but also by the size of the sediment particles, water temperature, and length of the acoustic path. Equations given by Flammer (1962) for the evaluation of energy loss are:

$$E = E_0 10^{-0.1\alpha x}, \tag{116}$$

where

E = sound energy flux at a given point, if sediment is suspended in the transmitting fluid;

E_0 = sound-energy flux at the same point, if no sediment were present;

α = attenuation coefficient that is due to sediment alone, measured in decibels per inch; and

x = distance from the point of measurement to the sound source.

The attenuation coefficient α can be evaluated as

$$\alpha = C\left[\frac{K(\gamma-1)^2 S}{S^2+(\gamma+\tau)^2}+\frac{K^4 r^3}{6}\right]\frac{22.05}{2}, \tag{117}$$

where

C = concentration (1,000 mg/L = 0.001),

$K = 2\pi/\lambda$,

$\gamma = \rho_1/\rho_2$,

$S = [9/(4\beta r)]\,[1+1/(\beta r)]$,

$\tau = \tfrac{1}{2} + 9/(4\beta\tau)$, and

r=particle radius, in centimeters;

in which

λ=wave length of sound in water, in centimeters; ρ_1 and ρ_2=densities of particle and fluid, respectively;

$\beta=[\omega/2v]^{1/2}$

$\omega=2\pi f$;

v=kinematic viscosity of water, in stokes; and

f=frequency of sound wave.

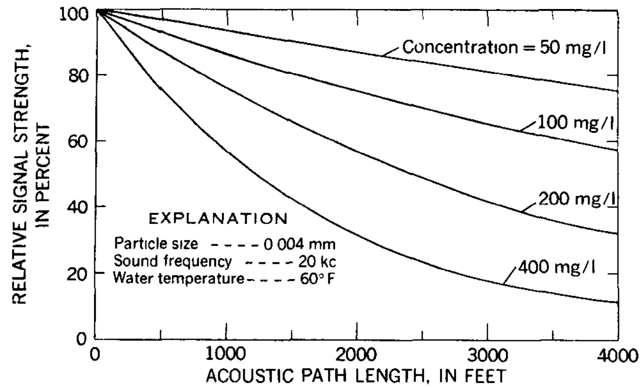
An example of the evaluation of equations 116 and 117 for an *AVM* site investigated in central California is shown in figure 216. Pertinent site and *AVM* characteristics were as follows:

Particle size—0.004 mm
Sediment concentration—20-100 mg/L
Water temperature—60°F(15.6°C)
Sonic-path length—4,000 ft (1219 m)
Sound frequency—20 kc

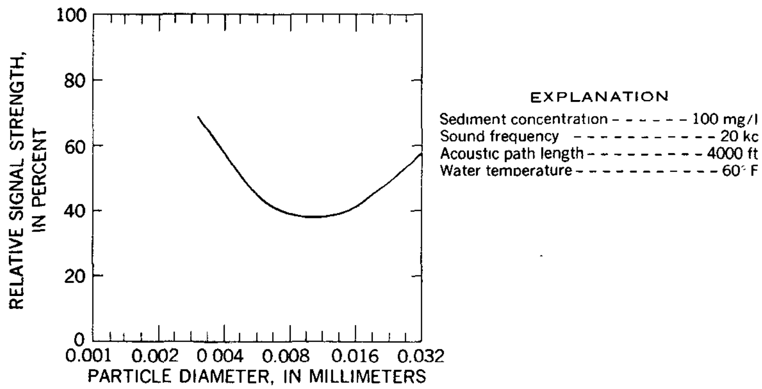
Figure 216*A* illustrates the general problem and shows the reduction in signal strength resulting from sediment concentrations ranging from 50 mg/L to 400 mg/L over acoustic paths as long as 4,000 ft. Figure 216*B* shows the signal loss for a given concentration and path length, as affected by particle size, and relates signal loss, for a path length of 4,000 ft, to sediment size when the sediment concentration is held constant at 100 mg/L. Figure 216*C* relates signal loss, for a path length of 4,000 ft, to sediment concentration when the sediment size is held constant at 0.004 mm. Figure 216*C* is of particular significance; it indicates that for the probable range in suspended-sediment concentrations at the site under consideration (20–100 mg/L), signal strength will vary from 90 to 56 percent of the levels possible in clear water. One of the requirements of an *AVM* designed for use at this site would be that no calibration changes should result from signal strength variations of that magnitude.

AQUATIC VEGETATION

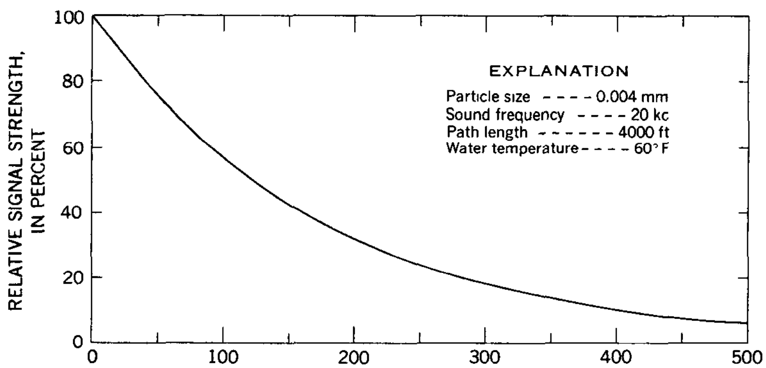
The effect of aquatic weeds in the acoustic path is variable, depending on the location and density of the weed growth. Dense growth may cause complete blockage of the signal. It has been found, in experiments in the United Kingdom, that the removal of only a small amount of weeds will increase the amplitude of the received signal. Further experimentation (Green and Ellis, 1974) has shown that weeds growing close to the transducer may actually cause the *AVM* system to overregister the velocity; the weeds reflect and scatter the wave train and the extra scattered signals are detected by the transducer. On the other hand, weeds in the midchannel result in a widely variable registration of velocity, in which the velocity is underestimated. In short, aquatic weeds in the acoustic path interfere with



A. Relation between signal strength, sediment concentration, and path length



B. Variation in signal strength with particle size



C. Relation between signal strength and sediment concentration

FIGURE 216.—Interrelation between signal strength, sediment concentration, particle size, and acoustic-path length.

the operation of an *AVM* system, but the quantitative results of experimentation with weed growths are not transferable from the experimental sites to other *AVM* sites.

It should also be noted that there has been no experimentation to relate attenuation caused by weed growth to sonic frequency. It appears probable that operation at a low frequency might reduce the attenuation; however, that would also reduce the basic accuracy of the *AVM* system.

SUMMARY OF CONSIDERATIONS FOR ACOUSTIC-VELOCITY METER INSTALLATIONS

The foregoing discussions of factors that influence *AVM* operation demonstrate that the interrelation of those factors must be considered in site selection of the acoustic path of an *AVM* system in a given stream. The most important consideration is to ensure reliable acoustic transmission and reception, and from that standpoint the acoustic path should be as short as possible to minimize acoustic refraction and attenuation losses. On the other hand, consideration of the hydraulic aspects of the system suggests use of a long path at a small angle of incidence (Θ in fig. 211) to the streamlines to achieve the best resolution of velocity and to reduce the effect of variations in streamline direction. These are opposing restraints on the system configuration, therefore compromise is often required. For most installations, the desired resolution can be attained by utilizing a path at the mid-depth position and at an angle of 45° to the streamlines. Narrow deep sections of a river are to be preferred over broad shallow sections, and locations influenced by tributary inflow should be avoided. On-site investigation of acoustic-propagation characteristics will be desirable at sites where the depth-to-path length criteria of figure 215 indicate possible problems.

Weed-covered sites and sites where air bubbles are entrained in the water should be avoided in selecting an acoustic path because of the likelihood of signal attenuation. For that same reason, the use of *AVM* systems may not be practical in streams that frequently carry large sediment loads.

ELECTROMAGNETIC VELOCITY-METER METHOD

GENERAL

The electromagnetic method of measuring velocity in stream-gaging operations will be discussed only briefly because it is still (1980) in the experimental stage. Experimental work in the U.S.A. has been largely in the use of the electromagnetic meter to obtain a continuous record of velocity at a point. The observed point velocities are then used as indexes of mean velocity in the stream, precisely as explained in earlier sections of this chapter, where the standard current meter and the deflection meter were the instruments used for continuously measuring point velocities. In several European countries, notably the United Kingdom, experimental work in electromagnetic stream-gaging has been largely in the use of an electromagnetic meter to obtain a continuous record of an index value of integrated mean velocity in the entire measurement cross section.

The operation of an electromagnetic velocity meter is based on the principle that an electromotive force, or voltage, is induced in an electrical conductor moving through a magnetic field. For a given field strength the magnitude of the induced voltage is proportional to the velocity of the conductor. In the electromagnetic velocity meter, the conductor is the flowing water whose velocity is to be measured. Although all devices for measuring water velocity electromagnetically are based on the above principle, the actual instrumentation for measuring point velocities differs greatly from that used for integrating the mean velocity in a cross section.

POINT-VELOCITY INDEX

INSTRUMENTATION

A variety of electromagnetic meters for measuring point velocity are available commercially. The meters differ in details of construction and performance, but essentially there are two general types.

One type of meter consists of the following elements: a nonmagnetic tube or pipe through which the water flows; two magnetic coils, one on each side of the pipe; electrodes in the walls of the pipe between the magnetic coils; and suitable electrical circuits to transform the induced voltage into a velocity indication on a meter dial. The other type of meter consists of a probe, or cylinder, containing an electromagnet internally and two pairs of external electrodes in contact with the water. Flow around the cylindrical probe intersects magnetic flux lines causing voltages to be generated that are detected by the electrodes. Electrical circuitry is provided to transform the induced voltage into a velocity indication on a meter dial.

For either type of meter, a source of electrical power is needed to activate the magnetic field and a transmitter is used to record the velocity signals on digital tape or to send the signals to desired stations. The meters used in the U.S.A. generally require an alternating-current source of 110 volts, but many are battery powered. The meters cause negligible head loss; accuracy claimed by the manufacturers is generally in the range of ±2 to ±3 percent or ±0.005 to ±0.007 ft/sec, whichever is larger. In other words, from a standpoint of percentage error, the higher velocities are more accurately measured than low velocities.

At the gage site the unattended electromagnetic velocity meter is securely anchored in a fixed position in the stream below the minimum expected stage. The considerations governing the precise location of the meter in the stream are identical with those discussed for the standard current meter when it is used to provide a point-velocity index (see section titled "Standard Current-Meter Method"). A recording stage-gage is operated in conjunction with the velocity meter. Velocity and stage are usually recorded on digital tape.

ANALYSIS OF POINT-VELOCITY DATA

The point-velocity data from the electromagnetic meter are analyzed in the same manner as discussed earlier in this chapter for the fixed standard current meter. Mean velocity for the measurement cross section, as obtained from discharge measurements, is correlated with concurrent stage and point velocity. Cross-sectional area is related to stage. The product of mean velocity and cross-sectional area gives the required discharge. Experimentation in the U.S.A. in the use of an unattended electromagnetic meter as a point-velocity index for gaging open-channel flow had lagged, primarily because of problems in suppressing electrical noise and in preventing the contamination of electrodes, but experimentation has recently been renewed. A description of a gaging-station operation in which point-velocity data are being obtained from an electromagnetic probe follows.

The gaging site on the Alabama River near Montgomery, Ala. is at a pool formed by a dam 43 miles downstream. The river is 600 ft wide and 40 ft deep, and the flow is largely controlled by the operation of hydroelectric-power dams upstream. The flow of the river is thus highly unsteady and in addition the water-surface slope varies because of operations at the downstream dam. The discharge of the river could not be related to stage or to stage and slope. Consequently, an electromagnetic meter was installed to provide point-index velocities.

The meter is of the portable probe type, is battery powered, and features solid-state electronics in a durable field housing. The form and size of the probe are shown in figure 217. The electromagnetic probe is mounted on a structure attached to the upstream end of a bridge pier in the center of the stream. The probe was positioned to sense the velocity at a point 6 ft upstream from the nose of the pier and 6 ft below the minimum stage of the water surface. The recorder and electronic package are installed in the gage house on the pier, about 35 feet above the mean high-water stage. The Geological Survey developed the electronics necessary to average the continuously generated velocity signal over 30-minute intervals and to record this average velocity on a digital recorder.

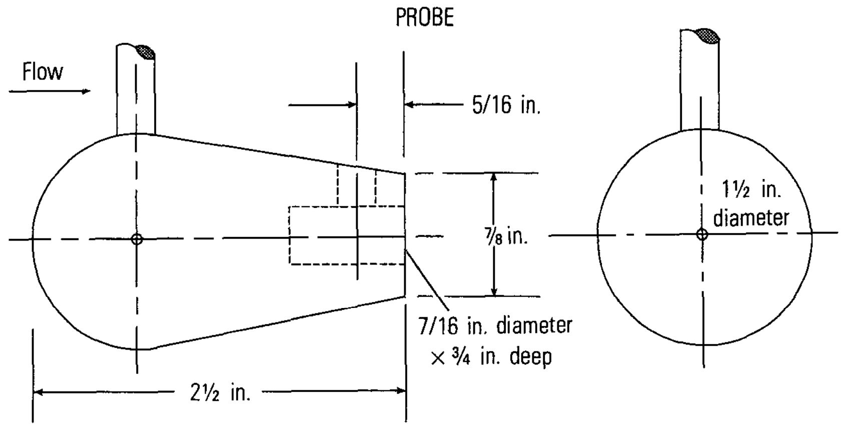


FIGURE 217.—Electromagnetic probe, model 201, Marsh-McBirney.

A series of current-meter discharge measurements was made to calibrate the relation of point velocity, as indicated by the probe, to the average velocity of the stream, as determined from the discharge measurements. Because of the unsteady flow, it was necessary that the discharge measurements be completed as quickly as possible. For that reason the measurements were made by defining the variation of velocity with time at a number of verticals in the stream-measurement cross section, as described in the section in chapter 5 titled, "Measurement Procedures During Rapidly Changing Stage—Case B. Small Streams." The relation between recorded point-index velocity and mean stream velocity determined from the discharge measurements is shown in figure 218. Although several of the plotted points scatter widely, the relation appears to be adequately defined over the range of velocity that was experienced. An attempt to improve the relation by the use of stage as an additional parameter, as in figure 201, was unsuccessful. A continuous record of discharge is computed at the gaging station by using the records of stage and point velocity, stage being an index of the cross-sectional area and point velocity an index of mean stream velocity.

Experience with the electromagnetic probe at the Alabama River gaging station has been very encouraging. The instrumentation appears to have wide application for gaging streams at sites where simpler rating methods such as stage-discharge or stage-slope-discharge are not adequate. The system has the sensitivity and accuracy required even at low velocities, is relatively inexpensive, has flexibility with regard to location because it is powered by dry-cell batteries, and can probably be used even at sites where the direction of flow reverses. The use of the system for gaging streams is consid-

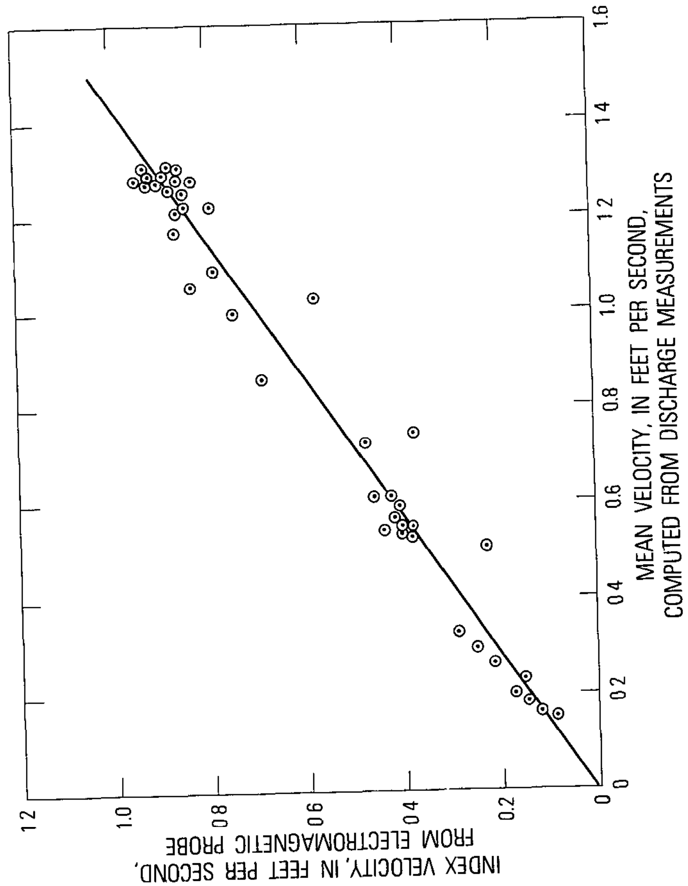


FIGURE 218.—Relation between point-index velocity and mean stream velocity for Alabama River near Montgomery, Ala.

ered to be in the experimental stage (1980), but it is hoped that further testing and development will result in the perfection of a reliable tool for gaging streams.

INTEGRATED-VELOCITY INDEX

THEORY

The discussions that follow have been extracted from British publications (Herschy and Newman, 1974; Newman, 1974; Plessey Radar, 1974).

If a conductor moves through a magnetic field, an electromotive force (voltage) is generated in the conductor. That principle can be applied to stream gaging. An electric current, flowing through a coil placed on a streambed at right angles to the flow, generates a magnetic field in the vertical direction. The flowing water is the conductor moving through the field, and the electromotive force (emf) generated in the water is at right angles to the flow. In accordance with Faraday's law of electromagnetic induction, the equation relating the length of the conductor moving in the magnetic field to the emf that is generated, is

$$E = HVb, \tag{118}$$

where

E is emf generated, in volts;
H is magnetic field intensity, in Tesla;
V is average velocity of the river water, in meters per second, and
b is river width, in meters.

In practice most streambeds will have some significant electrical conductivity that will allow electric currents to flow in the bed. The electric currents have the effect of attenuating the signal, predicted from equation 118, by a theoretically predictable factor called the conductivity-attenuation factor δ,

$$\delta = \frac{1}{1 + \left(\dfrac{b\sigma_0}{2h\sigma_1}\right)} \tag{119}$$

where

b is stream width,
h is stream depth,
σ_0 is streambed conductivity, and
σ_1 is river-water conductivity.

Equation 118 then becomes

$$E = HVb\delta \tag{120}$$

In an operational electromagnetic gaging station, the river and streambed conductivity should be continuously monitored and the output signal corrected accordingly.

When an electromagnetic gaging station uses an artificially produced magnetic field, that is, a magnetic field produced by a current-carrying coil, the field must, from practical considerations, be spatially limited. This means that electric currents flow in the areas outside the magnetic field, thereby reducing the output potential by a factor β, the end-shorting factor. That factor is a constant for a given coil size and configuration. Equation 120 now becomes

$$E = HVb\delta\beta. \tag{121}$$

For a given electromagnetic gaging station the magnetic-field intensity H, and the end-shorting factor β, are constants. The streambed resistivity attenuation factor, δ, is a function of the river-aspect ratio (the stage, if the river width is constant) and of the river-to-streambed conductivity ratio, σ_0/σ_1. Therefore, to insert the correct value of δ in equation 121 it is necessary to have measurements of the stage and the river-to-streambed conductivity ratio. The mean velocity of the river can then be computed. To obtain the discharge, the velocity is multiplied by the river cross-sectional area.

INSTRUMENTATION

An electromagnetic system for integrating stream velocity can be installed anywhere in a river or canal where the conductivity of the water is uniform but not necessarily constant. At present, installations have been confined to small streams. Measuring sections that are bounded by heavily reinforced concrete or by steel pilings are not suitable because of the relatively high electrical conductivity of those boundary elements. Although the signal-recovery techniques that are used make the system immune to ambient electrical noise, sites close to overhead or buried powerlines should be avoided if possible.

A description of one of the operational systems for integrating the stream velocity electromagnetically follows. In that system a large coil (fig. 219) is buried under the streambed and banks to a depth of about 0.5 m (1.5 ft, approx.). The trench in which the coil is laid roughly follows the contours of the bed and banks to minimize the effect of variation in the velocity profile. A magnetic field is produced by an electric current flowing through the coil.

Two signal probes placed in the magnetic field are fixed against the banks (fig. 219) or are driven vertically into the banks (fig. 220). The probes are used to detect the electromotive force induced in the moving water and to define precisely the cross section of the measurement area. The purpose of driving the signal probes vertically into the bank, as in figure 220, is to define a cross section whose area is rectangular. Such materials as aquatic vegetation and bed and bank sediments streamward from the probes are included in the size of the

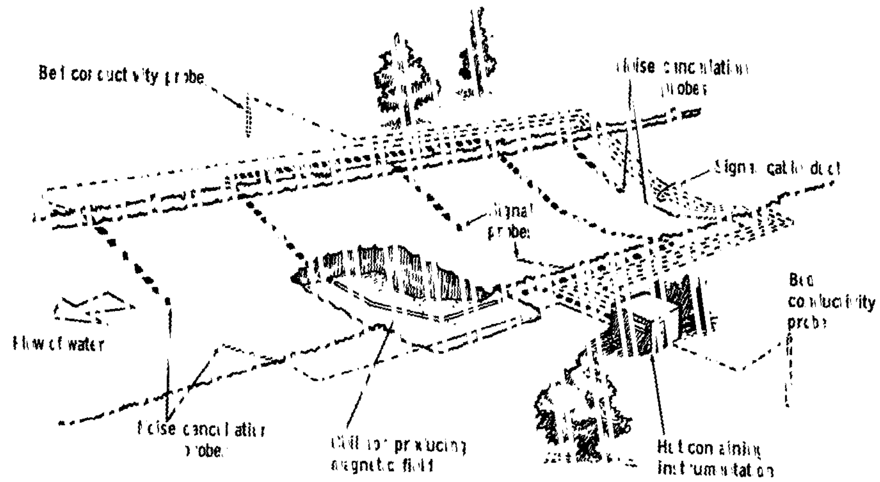


FIGURE 219.—Instrumentation for an electromagnetic stream-gaging station. (After Herschy and Newman, 1974.)

cross-sectional area the velocity of the extraneous material is zero and its effect on the system is therefore as innocuous as that of stagnant water.

Four noise-cancellation probes (fig. 219) outside the influence of the induced magnetic field are used to detect the ambient electrical noise which is later subtracted from the signal induced by the coil. These probes are mounted in a manner similar to the signal probes, with one pair upstream and one pair downstream from the measurement area.

Two bed-conductivity probes (fig. 219) are placed on each side of the stream, at a distance back from the bank that is approximately equal to the width of the stream. The probes and their cables may be buried sufficiently deep to avoid damage by agricultural work or other ground-disturbing activities. By the use of the bed-conductivity probes and the signal probes, the conductivity of the streambed is measured.

The power-supply unit provides a direct-current source that is connected to the coil through a switching unit. The switching unit, in response to timing pulses from the data-processor unit described below, reverses the direction of the current flowing through the coil at half-second intervals. That causes a synchronous change in the polarity of the electrical potential induced in the moving water by the magnetic field, with the result that the signal can be detected by the signal probes in the presence of electrical noise. At the signal-recovery unit, the probe voltage is amplified, and after filtering and conversion, a digital signal is provided to the data-processor unit.

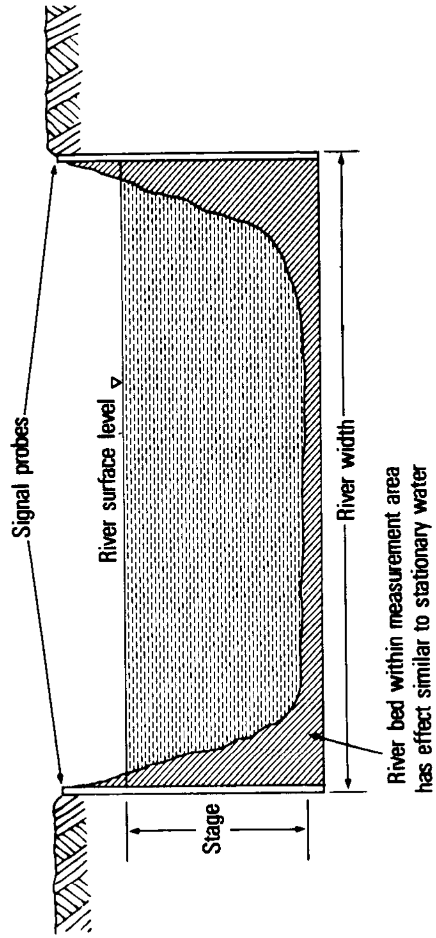


FIGURE 220.—Schematic diagram showing inclusion of bed and bank material in stream cross section. (After Herschy and Newman, 1974)

Additional instrumentation in the system includes a stage sensor and a power-operated pump that delivers continuous samples of water to a conventional conductivity sensor. The stage sensor, usually operating in a stilling well, provides a digital signal of stage to the data processor.

The block diagram in figure 221 shows the function of the data processor. In the data processor, the information from the probes is combined with that from the sensors for conductivities and stage. The latest information received is combined with similar prior information to provide a weighted average value. The weighted average value is then scaled, using preprogramed constants, to give an output of discharge in conventional units. The principles underlying the computation of discharge have been discussed in the subsection on theory of the integrated-velocity index. However, in the system described here, no separate computations of area and mean velocity are made. The two computations are easily combined because the cross-sectional area is a simple function of stage, the area bounded by the signal probes being a simple rectangle (fig. 220) or a trapezoid. The time constant in the process of averaging values is normally 15 minutes, which is also the time interval used in logging the data.

Information relating to discharge, stage, and water and streambed conductivities may be recorded locally on computer-compatible punched paper tape or on magnetic tape. Alternatively, the data may be transmitted to a control center over telephone lines or by a radio link. The transmission can be incorporated in a wider telemetry system for flood or pollution warning.

An initial field calibration, using discharge measurements, is required for the system. However, because the relation of electromagnetic output to discharge is linear, few discharge measurements are required to define the relation.

APPRAISAL OF METHOD

Studies to date (1980) indicate that the technique of electromagnetic stream gaging is feasible although there are still problems to be resolved. The method would probably have its principal use in gaging those streams that are not amenable to the more conventional methods of stream gaging—sand-channel streams with movable beds (see section in chapter 10 titled "Sand-Channel Streams,") and streams with profuse growths of aquatic weeds.

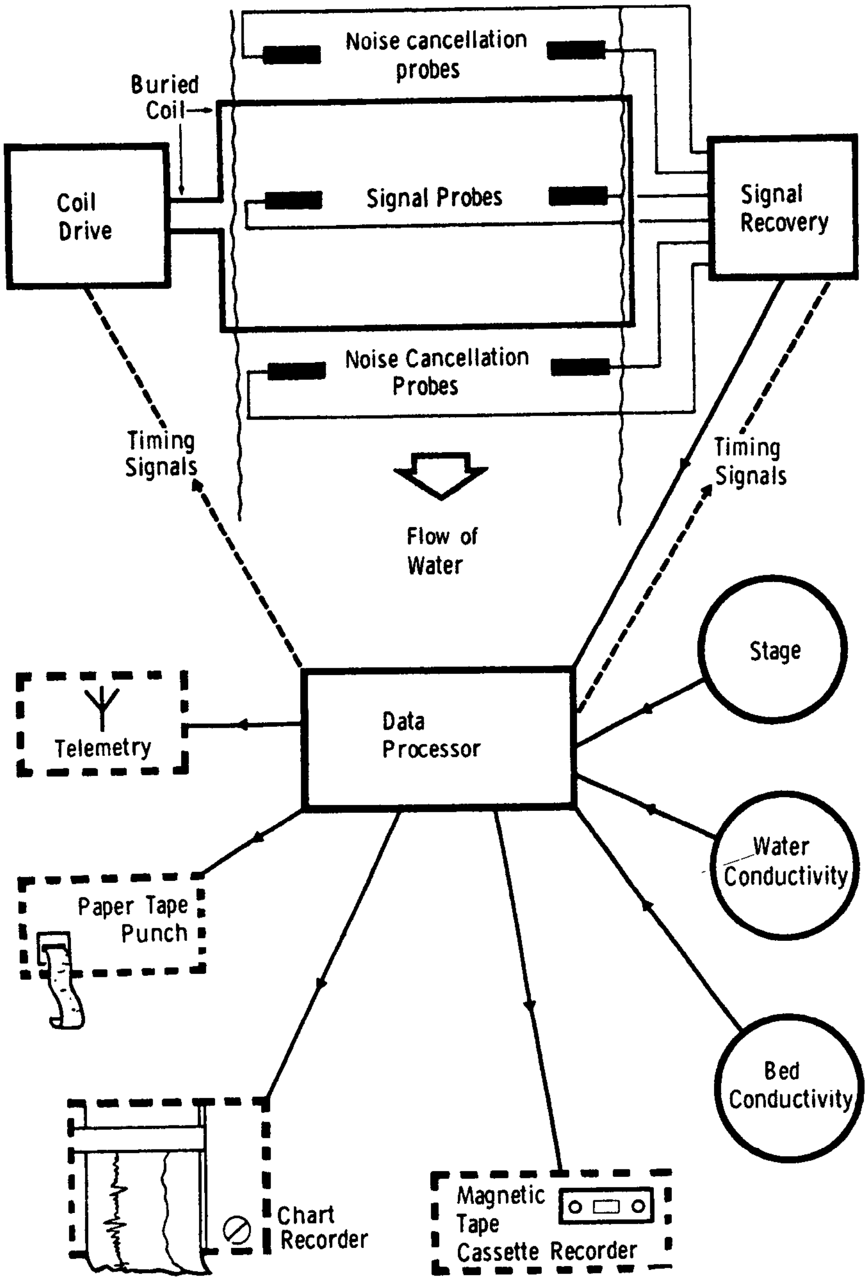


FIGURE 221.—Block diagram showing the function of the data processor. (After Plessey Radar, 1974. Reprinted by permission of the Plessey Company, Ltd.)

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CHAPTER 13—DISCHARGE RATINGS FOR TIDAL STREAMS

GENERAL

A discharge rating can be obtained for a tidal or tide-affected stream if a velocity index is used as a parameter in the rating along with stage. If the stream is quite small, a deflection meter is a satisfactory device for obtaining a continuous record of velocity; the electromagnetic method of stream-gaging is still in the experimental stage. For the larger streams the acoustic velocity meter is a satisfactory device for obtaining an index of velocity. (The instrumentation and methodologies involved in the use of deflection meters, electromagnetic velocity meters, and acoustic velocity meters were described in Chapter 12.) In the absence of such devices, there are two general approaches for obtaining a continuous discharge record—the theoretical approach involving evaluation of the equations of unsteady flow for a tide-affected reach of channel, and the empirical approach involving empirical relations whose effectiveness generally varies inversely with the degree of importance of the acceleration head (see section in chapter 11, titled "Theoretical Considerations"). The theoretical approach is much preferred.

Either approach requires a recording stage gage at each end of a long reach of channel. The two gages must be synchronized so that simultaneous stages at the two sites can be obtained from the stage records. Either approach requires discharge measurements for calibrating the discharge model; the moving-boat method of measuring discharge (chap. 6) is recommended for the larger streams.

EVALUATION OF UNSTEADY-FLOW EQUATIONS

It is beyond the scope of this manual to treat in detail the various methods of evaluating the equations of unsteady flow. Basic to all methods is the solution, by approximate step procedures, of the following pair of differential equations:

$$\frac{Q^2}{K^2} = -\frac{\partial H}{\partial x} - \frac{1}{g}\frac{\partial V}{\partial t} \tag{122}$$

$$\frac{\partial Q}{\partial x} = -B\frac{\partial h}{\partial t}, \tag{122a}$$

where Q is the discharge, K is the conveyance of the cross section, H is the total energy head, x the distance along the channel, g the acceleration of gravity, V the mean velocity, t the time, B the top width of the channel, and h is the water-surface elevation. Solution of the equations requires the use of a digital computer.

The four methods of equation evaluation that will be briefly discussed are:

1. power series,
2. method of characteristics,
3. implicit method, and
4. Fourier series.

For any of the four methods, it is necessary that a series of current-meter discharge measurements be made over several tidal cycles at either end of the reach. The measurements are needed to compute the resistance and conveyance properties of the reach and to serve as checks on the computed discharges. The series of measurements should include one series made at a time of low freshwater discharge and large tidal range, and another made when the freshwater discharge is high. These two series of measurements may be sufficient if the channel is stable with regard to scour and fill, but for an unstable alluvial channel, it is necessary that several additional series of discharge measurements be made. For meaningful results it is necessary that the channel either be stable (unchanging) or if subject to change, the channel changes must occur in the same way for each change in discharge during rises and subsequent recessions.

During a series of discharge measurements over a tidal cycle, enough measurements must be made to define the discharge hydrograph to an accuracy that will permit momentary discharges to be determined at 15-minute intervals for the duration of the cycle. The discharge measurements are commonly made at about hourly intervals and cover a few hours more than the duration of the tidal cycle.

The exact procedure used to measure the discharge in a tidal reach will vary with size of channel and flow conditions. For a small tide-affected stream, one or two field crews measuring continuously across the stream may be adequate for obtaining the data for an accurate definition of the discharge hydrograph. For a large tide-affected river, several measuring crews may be required, and it may be necessary to compute the hydrograph in the manner described for flash floods on small streams (see section in chapter 5 titled, "Measurement Procedures During Rapidly Changing Stage—Case B. Small Streams"). In that method a stage-mean velocity relation is first determined for each measurement vertical, and total stream discharge is then computed for selected stages. The preferred method of measuring discharge in a large tide-affected stream is, of course, the moving-boat method (chap. 6).

One of the basic assumptions of the methods to be presented is that the water in the tidal reach is substantially of homogeneous density, thereby eliminating the possibility of density currents. Therefore, consideration should be limited to those portions of tidal reaches that are affected by the propagation of long, low-amplitude, translatory waves but in which salinity intrusion does not cause a saline wedge to form or cause distinct stratification of flow.

An important consideration in the choice of a reach is the availability of a 110-volt power supply at both ends to drive the digital stage recorders synchronously. Although standby DC units, equipped with DC to AC inverters, are available to take over immediately in the event of a power failure, standby units should be used for periods no longer than absolutely necessary; the use of standby units for long periods of time invariably results in loss of synchronization. Any loss of synchronization between the two clocks, even fractions of a minute, may cause significant error in the computation of discharge. Recent (1976) tests indicate that battery-operated electronic clocks may have the required accuracy and reliability, and if so, the availability of a 110-volt power supply will no longer be a requirement.

POWER SERIES

In solving the equations of unsteady flow by use of a power series—commonly a Taylor series—finite differences are used. Velocity and stage and all orders of their derivatives are continuous functions with respect to x and t. The value of Δx, or length of reach to be used, is generally 3 to 7 miles—there is a theoretical maximum length that depends on tidal wave length (hours) and mean channel depth. The value of Δt to be used is usually 15 minutes.

In the power-series method, tidal flow is considered to be one-dimensional unsteady flow in a prismatic channel. However, a natural reach of channel usually differs greatly from an idealized tidal reach that has an unvarying prismatic cross-section and a constant bottom slope. Consequently, it is necessary to determine a mean cross-section that is representative of the reach and whose dimensions are variable with stage. From such a representation, the geometric parameters required for the discharge computations can be obtained. The number of cross sections to be surveyed in the field in order to compute a representative mean cross-section depends on the length and degree of uniformity of the tidal reach. Usually ten or more cross sections, somewhat evenly spaced in the reach, are required.

Local inflow or outflow (diversions) to or from the tidal reach is considered in the solution, but the quantity of such flow must be small in comparison with the flow in the main channel. The local inflow or outflow is often assumed to be constant throughout a complete tidal cycle and is considered to enter or leave the main channel uniformly along the entire length of reach; thus large concentrations of flow at a single point cannot be accommodated by the method.

A mathematical description of the mechanics of computing discharge in tidal reaches, by the use of a power series for solving the equations of unsteady flow, is beyond the scope of this manual. For such information the reader is referred to papers by Baltzer and Shen (1961, 1964) and by Davidian (1964).

METHOD OF CHARACTERISTICS

The method of characteristics is well adapted to the solution of partial differential equations in two variables, such as equations 122 and 122a. (All terms in those equations are related to velocity and (or) stage.) In the method, the basic partial differential equations are first transformed to characteristic equations and then to corresponding difference equations.

As in the power-series method, tidal flow is considered to be one-dimensional unsteady flow in a prismatic channel. In the finite-difference solution of the equations, computations are made at equal time intervals Δt (usually 15 minutes, or less) and at equal increments of distance x along the channel. The selected value of Δx must meet the criterion,

$$\Delta x \geqslant \Delta t\ (V + \sqrt{gd}), \tag{123}$$

where

V is mean velocity of flow at the starting cross section of increment Δx,

g is the acceleration of gravity, and

d is depth.

Both velocity and stage, as well as discharge, which is the product of velocity and area, can be obtained explicitly for each new step of Δt. Unlike the power-series method which provides the desired information for only one variable at a time—for example, the discharge at one end of the reach—the method of characteristics provides the desired information simultaneously for any selected points in the reach that lie at multiples of Δx from the end of the reach. However, it is possible to change the values of Δx and Δt during the computation, if that is desired, as long as the relation of Δt to Δx meets the criterion given in equation 123.

A single representative cross section is normally used for the entire reach, as in the power-series method. However, a multiple reach may also be used (Lai, 1967a). A long reach is divided into several subreaches, each with its own individual geometry and roughness coefficient. The basic method of characteristics is applied to each subreach, and additional boundary conditions are imposed at each junction between subreaches. Furthermore, the method of characteristics will accommodate the entrance of a large gaged tributary into the reach; the boundary conditions are then more involved, but there is no complication in principle.

The mathematical details of the method of characteristics are described by Lai (1965a, 1967a), Lister (1960), and Stoker (1953).

IMPLICIT METHOD

The implicit method is yet another finite-difference procedure for solving the basic partial differential equations for flows of homogeneous density in tidal reaches. The implicit method has one advantage over the method of characteristics in that the choice of Δt is less restricted; the stability of the solution is not limited by the criterion of equation 123.

The mathematical details of the implicit method are described by Lai (1965b, 1967b, 1968) and by O'Brien, Hyman, and Kaplan (1951).

FOURIER SERIES

The equations of unsteady flow have also been solved by a method of harmonics in which a Fourier series is used. The distinctive characteristic of a Fourier series is the periodicity of the trigonometric terms of which it is composed. The Fourier series lends itself well to the expression of periodic phenomena that are represented by linear differential functions; the equations of unsteady flow, however, are quasi-linear hyperbolic differential functions. Consequently, it is necessary to linearize the equation system when using a Fourier series. That distorts the equation system. An even more significant consideration is whether or not tidal flow can be described as a truly periodic phenomenon. The long translatory wave motion introduced into a tidal reach by the astronomical tide is periodic, but its periodicity is disturbed when the natural upland flow of a river system is superimposed on the tidal wave motion or when storm surges from the ocean occur.

Despite these drawbacks, Fourier series evaluation techniques for determining flow in tidal reaches have been developed (Dronkers, 1947, p. 127–137; Dronkers and Schönfeld, 1955, p.11–24; Schonfeld, 1951, p. 70–87 and 143–152). However this type of solution is the least suitable of the four methods that have been briefly described here for solving the differential equations of unsteady flow in tidal reaches.

EMPIRICAL METHODS

Four empirical methods of rating tidal reaches have been in use, all but one of which were developed before the use of digital computers became commonplace. Those techniques are:

1. method of cubatures,
2. rating-fall method,
3. tide-correction method, and
4. coaxial graphical-correlation method.

All the above methods have their shortcomings which are discussed, where appropriate, in the sections of the manual that follow.

METHOD OF CUBATURES

One of the oldest methods of computing discharge in tidal estuaries is the method of cubatures (Pillsbury, 1956). The method, still in use, is based on the equation of the conservation of mass;

Outflow at the study station = inflow ± change in storage.

The inflow term in the above equation is the freshwater discharge measured at a gaging station at or upstream from the head of tide—that is, a gaging station having a simple stage-discharge relation. The storage term refers to volume of water in the reach between the inflow gaging station and the study station on the estuary. Intermediate stage gages are usually needed for evaluating the storage term. The gages are spaced at such distances that no significant error is introduced in the computations by considering the water surfaces between gages as planes. That requirement ordinarily is met by stations some miles apart but suitably placed with regard to marked changes in the cross section of the waterway. The differences in the tidal ranges on the opposite shores of a wide estuary may usually be disregarded, but it may be necessary to establish tidal stations on any long tidal tributaries of the main waterway. For convenience in the computations, the tides at all stations should be reduced to the same horizontal datum, preferably taken low enough to make all stages positive.

If existing surveys do not afford reliable data on the areas of the water surfaces between the selected tidal stations, a survey to establish these surfaces is required. Usually such surface areas may be taken as increasing uniformly from low water to high water, but if there are any considerable tide flats that are exposed at the lower tidal stages, the area at the stage at which such flats are covered also should be found.

Freshwater inflow to the reach from tributary streams is estimated if the tributary flow is relatively small. If the tributary streams are large, they are gaged upstream from the head of tide to provide a continuous record of freshwater inflow, just as is done with the principal inflow stream.

A sample computation is shown in figure 222 for a 5.8 mile reach of the Delaware River between Trenton and Fieldsboro, N.J. This is the second reach in the estuary; the first reach extends upstream from

Time	Tidal stage: Upper station	Tidal stage: Lower station	Tidal stage: Mean (y)	Mean stage during time interval	Water-surface area (A) at mean stage	Change in stage (Δy) during time interval	Change in storage: In study reach $\Delta Q = A(\Delta y/\Delta t)$	Change in storage: In upstream reach	Change in storage: Total (upstream plus study reaches)	Outflow Q
(hr)	(ft)	(ft)	(ft)	(ft)	(1000's of ft²)	(ft)	(ft³/sec)	(ft³/sec)	(ft³/sec)	(ft³/sec)
1	2	3	4	5	6	7	8	9	10	11
0500	2.78	2.52	2.65							
				2.48	39,703	-0.35	-7,720	-500	-8,220	-20,420
0530	2.43	2.16	2.30							
				2.12	38,880	-.35	-7,560	-490	-8,050	-20,250
0600	2.09	1.81	1.95							
				1.80	38,206	-.31	-6,580	-430	-7,010	-19,210
0630	1.79	1.50	1.64							
				1.52	37,656	-.25	-5,230	-410	-5,640	-17,340
0700	1.51	1.27	1.39							
				1.59	37,800	.40	8,400	20	8,420	-3,780
0730	1.52	2.06	1.79							
				2.52	39,686	1.47	32,410	2,840	35,250	23,050
0800	3.48	3.04	3.26							
				3.77	42,247	1.02	23,940	1,350	25,290	13,090
0830	4.41	4.16	4.78							
				4.66	44,053	.76	18,600	1,090	19,690	7,490
0900	5.16	4.93	5.04							
				5.36	45,478	.64	16,170	920	17,090	4,890
0930	5.79	5.58	5.68							
				5.96	46,702	.55	14,270	770	15,040	2,840
1000	6.32	6.14	6.23							
				6.48	47,755	.49	13,000	690	13,690	1,490
1030	6.80	6.64	6.72							

FIGURE 222.—Sample computation of tide-affected discharge by method of cubatures, using 30-minute time intervals. (After Pillsbury, 1956.)

Trenton to the Delaware River gaging station that is upstream from head of tide. Total freshwater inflow to the second reach is −12,200 ft^3/s, flow in the downstream (ebb) direction being considered negative in the computations. That inflow consists of 12,000 ft^3/s for the Delaware River mainstream and 200 ft^3/s for tributary inflow. The time interval (Δt) used in the computations is 30 minutes, or 1,800 seconds. The computations in the table in figure 222 are largely self-explanatory. The figures in column 9 were obtained from similar computations (not shown) for the reach upstream from the study reach. Column 10 is the sum of columns 8 and 9; column 11, the outflow from the study reach, is the sum of the total storage change (column 10) and the total freshwater inflow (−12,200 ft^3/s).

Figure 223 is the discharge hydrograph obtained by first plotting the outflow histogram (values from column 11 of fig. 222), and then drawing a smooth hydrograph to give balance between areas above and below the horizontal bars of the histogram.

The method of cubatures is not only cumbersome, but the discharge figures obtained are only rough approximations of the true values

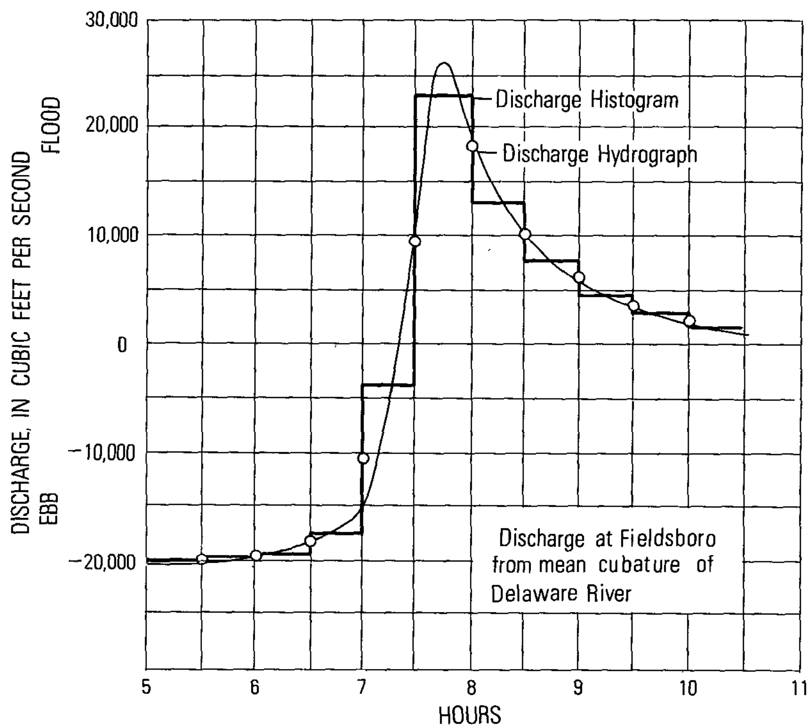

FIGURE 223.—Discharge hydrograph obtained for sample problem by method of cubatures. (After Pillsbury, 1956.)

because of the large errors inherent in computing the storage component of the continuity equation. When the method is used, the results, although approximate, should be checked for consistency—total computed outflow should approximate total inflow over some long-term period whose net change in storage is negligible.

RATING-FALL METHOD

Stage-fall-discharge relations have been used successfully for rating tide-affected streams where acceleration head is a minor factor. The rating-fall method that is discussed in detail in the section in chapter 11 titled, "Variable Slope Caused by Variable Backwater," is used for that purpose. Acceleration head is often a minor factor where the slope reach is located at the upper end of an estuary near the head of tide. Consequently, it is usually only at or near such locations that the rating-fall method can be used successfully.

TIDE-CORRECTION METHOD

The tide-correction method assumes that a direct proportionality exists between the cyclic range in stage observed at any two points within a tidal reach. On the basis of that assumption, a relation of mean discharge for a tidal cycle to mean stage for a tidal cycle is developed for the base-gage site. In calibrating that relation, the mean discharge for a tidal cycle, obtained by averaging several individual measurements made 1–2 hours apart throughout the cycle, is plotted against adjusted mean stage at the base gage. The adjustment applied to the mean stage at the base gage is determined from the difference, at the secondary gage, between observed mean stage and the stage that is presumed to exist under conditions of least tide fluctuation. That difference (D) is multiplied by the ratio of the stage range at the base gage to the stage range at the secondary gage; the product is the stage adjustment required at the base gage. In practice, the secondary stage observations are frequently made at a nearby ocean inlet. Mean sea level is assumed to represent the condition of least tidal fluctuation, and therefore, if all gages have their datums set to mean sea level, D is always equal to the mean stage for a tidal cycle at the secondary gage. Essentially the tide-correction method attempts to approximate the stage that would occur for a particular steady-flow discharge under a fixed backwater condition. An example of the tide-correction method (Parker and others, 1955) follows.

At Hialeah, Fla., the base gage is on the Miami Canal, 7.6 miles upstream from the ocean. A tide gage on the ocean is used as the secondary gage. Both gages have their datum at mean sea level. On a given date the following tidal-cycle data were obtained from the stage gages:

	Base gage	*Secondary gage*
Mean stage (ft)	2.64	1.61 = D
Maximum stage (ft)	3 18	2.74
Minimum stage (ft)	2.10	.48
Stage range (ft)	1.08	2.26

$$\text{Stage correction (base gage)} = D\left(\frac{\text{stage range at base gage}}{\text{stage range at secondary gage}}\right)$$
$$= 1.61\left(\frac{1.08}{2.26}\right)$$
$$= 0.77 \text{ ft}$$

The stage correction is always negative and therefore the stage at the base gage, to be applied to the mean discharge for the tidal cycle on that date, is

$$\text{Gage height} = 2.64 - 0.77 = 1.87 \text{ ft.}$$

The mean-cycle discharge, as determined from 20 sets of discharge measurements, was plotted against the actual mean-cycle gage height and also against the tide-corrected gage height, as indicated on figure 224. The rating curve shows the relation between the tide-corrected gage height and the mean tide-cycle discharge for the upper gage. The discharge, when plotted against actual mean-cycle gage height, shows a considerable scatter of the plotted points, but the discharge plotted against tide-corrected gage height shows a very

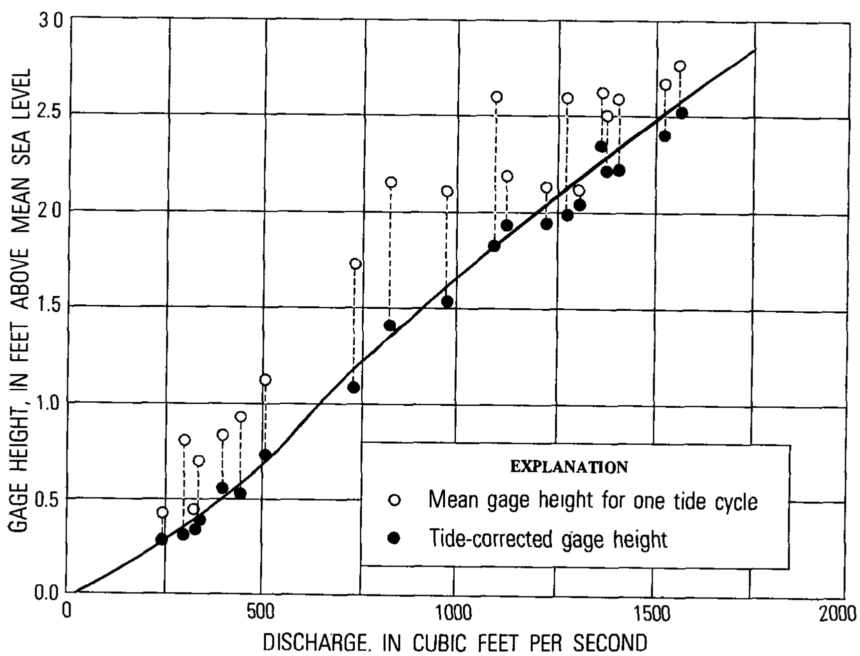

FIGURE 224.—Graph of relation between tide-corrected gage height and discharge for Miami Canal at Water Plant, Hialeah, Fla.

close agreement for the numerous measurements. The shape of the rating curve is characteristic of that for a stream having a large initial cross-sectional area at the point of zero flow.

The tide-correction method of rating a tide-affected stream may be used where reverse flows occur during a part of each tide cycle because the mean discharge for the cycle is the value used in the computation. It is also applicable to a reach of tidal waterway, on which both observation stations are upstream from the mouth of the waterway. Mean-cycle discharge obtained from the rating curve can be plotted against mean-cycle time on a hydrograph sheet, and after connecting the points by straight lines, the daily mean discharges can be determined.

The tide-correction method has been satisfactory, though cumbersome, for computing the daily discharge of tide-affected canals in Florida, but efforts to adopt the method for use elsewhere in the U.S.A. have generally been unsuccessful.

COAXIAL RATING-CURVE METHOD

The coaxial method of graphical correlation to determine discharge in a tidal reach was developed to fill the need for a simple method of making reasonably accurate "on-the-spot" determinations of streamflow. A method of this kind is required, for example, in the operation of a sewage plant discharging its effluent into a tide-affected stream. The method that was developed fills this need in that readings from a pair of stage gages can be used to determine momentary discharge directly from a set of rating curves.

The coaxial method is best described by an example. Coaxial rating curves were developed for the Sacramento River at Sacramento, Calif., on the basis of 302 discharge measurements made during the years 1957–60 (Rantz, 1963). Actually only 52 of the measurements were used to develop the curves; the remaining 250 discharge measurements were used to test the rating curves and refine them slightly. Measured discharges ranged from 4,060 ft^3/s (115 m^3/s) to 19,300 ft^3/s (547 m^3/s).

The streamflow-measurement section is at the site of the stage recorder in the city of Sacramento; the auxiliary stage recorder is 10.8 mi (17.4 km) downstream near the town of Freeport. Local inflow into the 10.8-mile reach of channel is negligible. The reach itself is located far enough upstream on the Sacramento River estuary so that no reversal of flow occurs. When upland flow (streamflow) into the estuary is less than about 30,000 ft^3/s (850 m^3/s), however, the discharge is affected by tidal action, and the flow in the reach is unsteady. The relative magnitude of the tidal effect in the reach increases with decrease in the upland flow and with increase in the range of elevation between high and low tides. The stages at Sacramento and

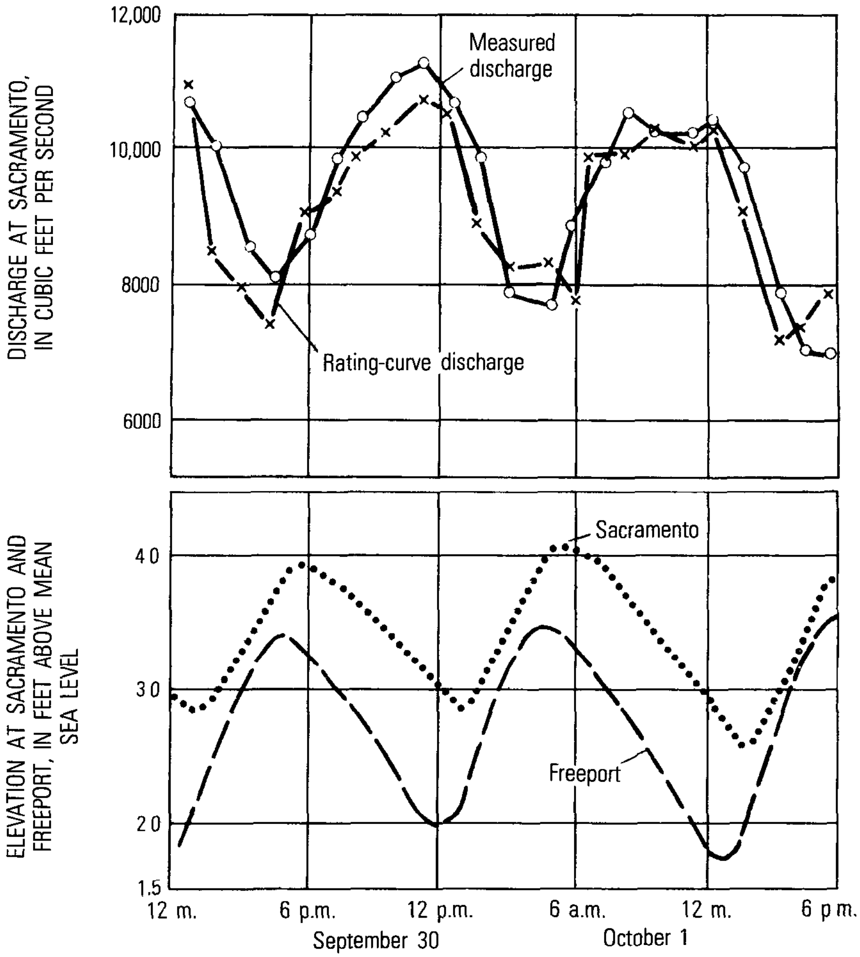

FIGURE 225.—Stage and discharge of the Sacramento River at Sacramento, Calif., Sept. 30 to Oct. 1, 1959.

Freeport during a 36-hour period and the fluctuation of discharge at Sacramento illustrate a typical low-flow condition (fig. 225). The upland flow above Sacramento was 9,300 ft/s (263 m^3/s). As a result of tidal effect, the discharge at Sacramento varied from 6,800 ft^3/s (193 m^3/s) to 11,300 ft^3/s (320 m^3/s).

The differential equations of unsteady flow were used to devise a graphical technique for determining discharge. The following parameters serve as indices of the terms that appear in these differential equations:

Dependent variable.—Measured discharge at Sacramento.

Independent variables.—(1) Stage at Sacramento, (2) fall in the

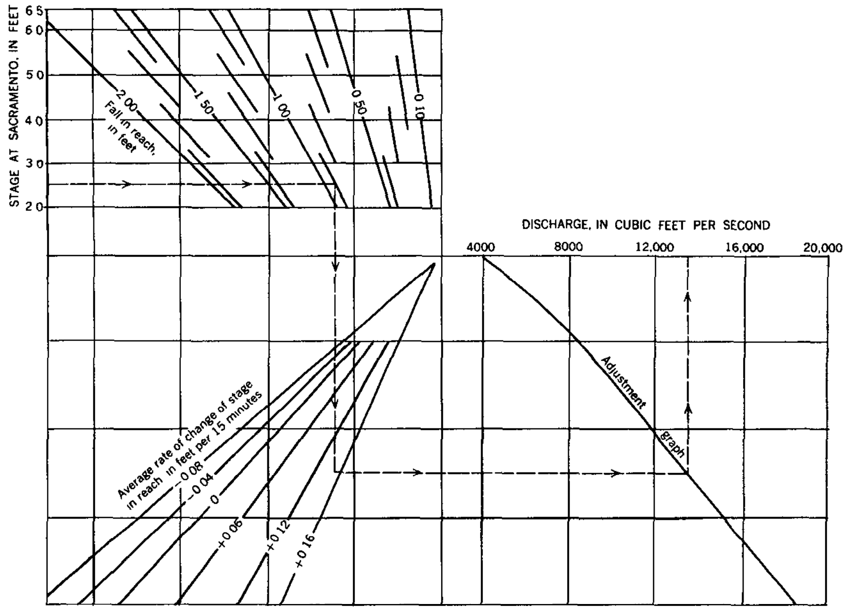

FIGURE 226.—Coaxial rating curves for the Sacramento River at Sacramento, Calif. (Dashed lines and arrows illustrate use of the curves.)

reach between Sacramento and Freeport, and (3) the algebraic average of the change in stage observed at Sacramento and Freeport during a 15-minute interval.

Because of the differential form of the equations of unsteady flow, there is no statistical model on which to base the relationship of these variables. A further complication arises from the fact that joint functions are involved, for interrelations among the independent variables affect the flow at Sacramento. The versatile statistical technique known as the coaxial method of graphical multiple correlation (Linsley and others, 1949, p. 650–655) was adopted for developing the rating curves for the Sacramento River.

The coaxial graphical correlation that was the end product of this study is shown in figure 226. In the interest of simplicity, only a few lines are shown in each family of curves. To use the graph, first, the curves in the upper left-hand group are entered with the stage at Sacramento and the fall in the reach; next, the curves in the lower left-hand group are entered with the average rate of change of stage in the reach; finally, the adjustment graph to the right is entered and the discharge is read. The adjustment graph was added to the correlation to introduce a necessary curvilinearity to the relationship. This curve may also serve another purpose—if the relation should change, as a result of channel dredging, for example, only the adjustment graph need be revised, thereby eliminating the laborious task of revising the two families of curves.

Of the 305 discharge measurements, 268, or 89 percent of them, checked the rating within ±10%; 286 measurements, or 95 percent of the measurements, checked the rating within ±15%. The rating was weakest during periods of low upland flow, but for only a few hours of any day during those periods. Those hours coincided with times when the acceleration head in the equations of unsteady flow was of relatively major importance.

The coaxial rating-curve method fulfills its purpose of being useful for making "on-the-spot" estimates of tidal flow at Sacramento, but the method is too cumbersome for use in computing a continuous record of discharge for a gaging station. Solution of the theoretical equations of unsteady flow (see section on "Evaluation of Unsteady-Flow Equations") is much better for the latter purpose.

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CHAPTER 14—DISCHARGE RATINGS FOR MISCELLANEOUS HYDRAULIC FACILITIES

INTRODUCTION

This chapter is a "catchall" for specialized problems in establishing discharge ratings for various hydraulic facilities, using techniques that are not specifically described in chapters 10–13. The hydraulic facilities are discussed under the following principal headings:

1. dams with movable gates,
2. navigation locks,
3. pressure conduits, and
4. urban storm drains.

DAMS WITH MOVABLE GATES

GENERAL

Dams are commonly equipped with movable gates for better control of pool stage and outflow. As a general rule the movable gates, as such, are not rated; instead, the channel downstream is rated by the most practicable method—simple stage-discharge relation (chap. 10), or stage-fall-discharge relation (chap. 11), or by use of a velocity index furnished, for example, by an acoustic velocity meter (chap. 12). However, in some situations none of those rating methods may be satisfactory. For example, consider a river controlled by a series of low navigation dams. In that situation, the river profile resembles a huge staircase—successive pools separated by dams. The movable dam crests negate the use of a simple stage-discharge relation; the slope of the water surface in the pools may be too flat for a stage-fall-discharge relation; and velocities may be too slow for accurate evaluation by an acoustic velocity meter. In that situation, the most practicable method of obtaining a continuous record of discharge is to calibrate the flow through or over the movable gates. If boat traffic is heavy and natural inflow is light, a significant part of the discharge may be the flow released through the navigation locks and the lockages must likewise be calibrated (see section on "Navigation Locks").

Calibration of the gates by discharge measurements during periods of light releases of water may be extremely difficult. If boat lockages are infrequent, standard current-meter measurements made downstream by boat, using a low-velocity meter, may be adequate. If boat lockages are frequent, the surges in discharge attributable to the lockages may cause unsteady and nonuniform flow conditions downstream; discharge measurements must then be made as rapidly as possible under conditions that are not conducive to accurate results. A rapid discharge measurement may be made by the moving-boat method (chap. 6) or by use of a bank of current meters operated from a bridge (see section in chapter 5 titled, "Networks of Current Meters"). If velocities are too slow for accurate measurement by either of those two methods, and if only small quantities of water are being released under the dam-crest gates, the best course of action might be to use the volumetric method for measuring flow over a dam crest that is described in the section in chapter 8 titled, "Volumetric Measurement." In using the volumetric method, the barge carrying the calibration tank is kept in place not only by lines operated from the banks, but also by an outboard motor on the barge to keep the barge from drifting downstream. The difficulty of measuring low flow under the conditions described above is apparent. At those times it may also be difficult to determine the actual head on the gates because lockages often cause longitudinal seiche-like waves to traverse the gage pool, and those waves travel back and forth over the length of the pool for a considerable period of time.

The flow at movable dam-crest gates may be placed in two general categories—weir flow over the gate or dam crest and orifice flow under the gate. Each of those types of flow may be either free or submerged, depending on the relative elevations of headwater, tailwater, and pertinent elements of the dam crest or gate. Listed below are the crest gates that will be discussed.

1. Drum gates
2. Radial or Tainter gates
3. Vertical lift gates
4. Roller gates
5. Movable dams
    a. Bear-trap gates
    b. Hinged-leaf gates
    c. Wickets
    d. Inflatable dams
6. Flashboards
7. Stop logs and needles

A gated dam usually has several gates along its crest. The gates are installed in bays that are separated by piers. All other conditions being equal, the discharge through a single gate, when adjacent gates are open, will be about 5 percent greater than the discharge through that same gate when adjacent gates are closed. The various types of gates should be calibrated by discharge measurements, but as an aid to shaping the calibration curves, experimental ratings where available are given in the text that follows.

Discharge measurements for the purpose of determining gate coefficients will almost always be made in the downstream channel and will include the flow for all the gates that are open. Furthermore, for given stages upstream and downstream from the gates, the gate coefficient will commonly vary with the gate position or opening. Consequently, if discharge is to be measured with more than one gate open, arrangements should be made, if possible, for all gates to be positioned identically. If the difference in the positioning of the gates are minor, and if the gate coefficient does not vary significantly with its positioning, a discharge measurement may be made; in the computation of the gate coefficient, an average gate position will be assumed for each of the bays carrying flow.

DRUM GATES

A drum gate consists of a segment of a cylinder which, in the open or lowered position, fits in a recess in the top of the spillway. When water is admitted to the recess, the hollow drum gate is forced upward to a closed position. One type of drum gate (fig. 227*A*) is a completely enclosed gate hinged at the upstream edge; buoyant forces aid in its lifting. That type of gate is adapted to automatic operation and also conforms closely to the shape of the ogee crest when lowered. A second type (fig. 227*B*) has no bottom plate and is raised by water pressure alone. Because of the large recess required by drum gates in the lowered position, they are not adapted to small dams.

With regard to its calibration, the drum gate resembles a thin-plate weir with a curved upstream face over the greater part of its travel. Given an adequate positioning indicator, the drum gate can serve as a satisfactory stream-gaging control. Its use for that purpose has been investigated by Bradley (1953), and the discussion that follows is taken almost verbatim from Bradley's paper dealing with a drum gate of the type shown in figure 227*A*.

When the drum gate simulates a thin-plate weir—that is,.when a line drawn tangent to the downstream lip of the gate makes a positive angle with the horizontal, as shown in figure 228*A*—four principal factors are involved. These factors are H, the total head above the high point of the gate; Θ, the angle between the horizontal and a line

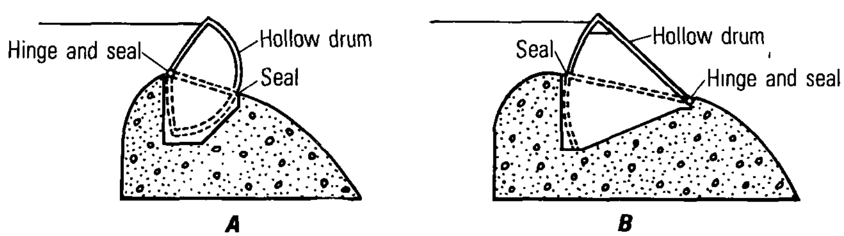


FIGURE 227.—Two types of drum gate.

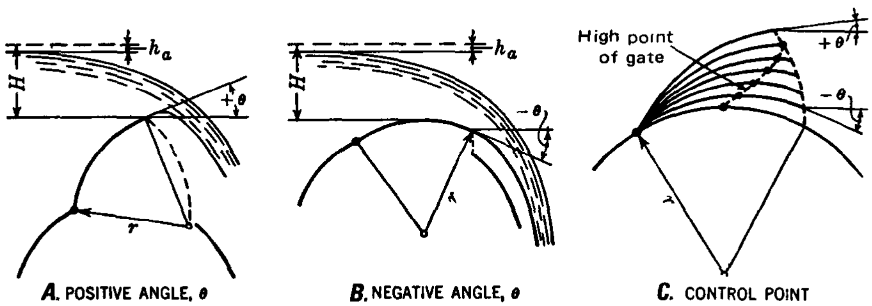


FIGURE 228.—Drum-gate positions. (After Bradley, 1953.)

drawn tangent to the downstream lip of the gate; r, the radius of the gate, or an equivalent radius if the shape of the gate is parabolic; and C_d, the coefficient of discharge in equation,

$$Q = C_d b H^{3/2}, \tag{124}$$

where

Q is discharge (ft³/s), and

b is length of the gate (ft) normal to the discharge.

The velocity in the approach section was not included as a variable because the drum-gate installations studied were on high dams where approach effects were negligible. It has been shown that when the approach depth measured below the high point of the gate is equal to or greater than twice the head on the gate, a further increase in the approach depth produces little change in the coefficient of discharge. Most drum-gate installations are on dams that meet the above depth criterion, particularly when the gate is in a raised position. Therefore, in the usual case of adequate approach depth, the four variables, H, Θ, r, and C_d completely define the flow over this type of gate when angle Θ is positive (fig. 228*A*).

For negative values of Θ (fig. 228*B*), the downstream lip of the gate no longer controls the flow. In that situation the control point shifts upstream to the vicinity of the high point of the gate for each setting, as illustrated in figure 228*C*, and flow conditions gradually approach those of the free crest as the gate is lowered. Although other factors enter the problem, similitude in the computation exists down to an angle of about $-15°$.

Experimentation with eleven drum gates produced the family of curves for C_d shown in figure 229. The discharge coefficients in the region between $\Theta = -15°$ and the gate completely down are determined by graphical interpolation, a method that will be explained in the example that follows. The effect of submergence of the drum gate on C_d was not investigated because drum gates are invariably used on high dams, and the probability of submergence is negligible. The data

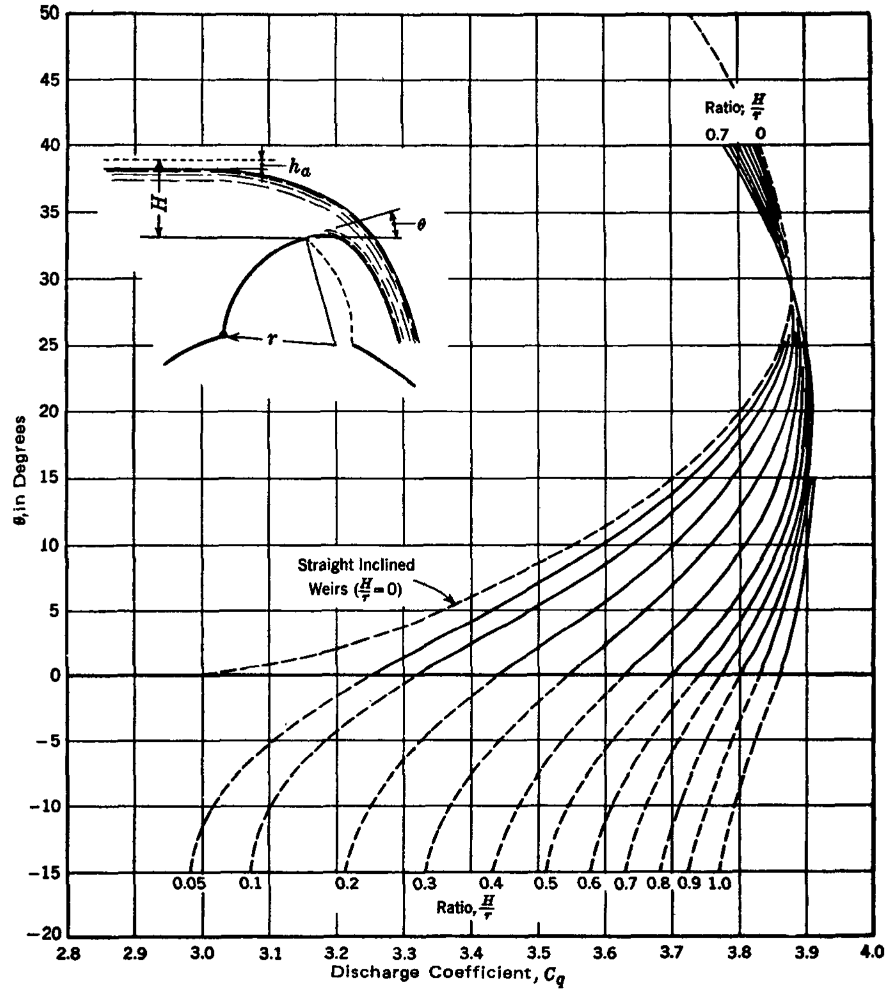


FIGURE 229.—General curves for the determination of discharge coefficients. (After Bradley, 1953.)

to be continuously recorded for computing discharge over rated drum gates are reservoir stage and the indication of drum-gate position for each gate.

The method of rating a drum gate on a round-crested weir will now be demonstrated using as an example the plan and spillway cross section of Black Canyon diversion dam in Idaho (figs. 230 and 231). The first step is the determination of the design head of the dam and the corresponding discharge coefficient for the free crest. That is done in accordance with the technique described under the heading "Nappe-fitting method" in the U.S.G.S. manual on computing peak

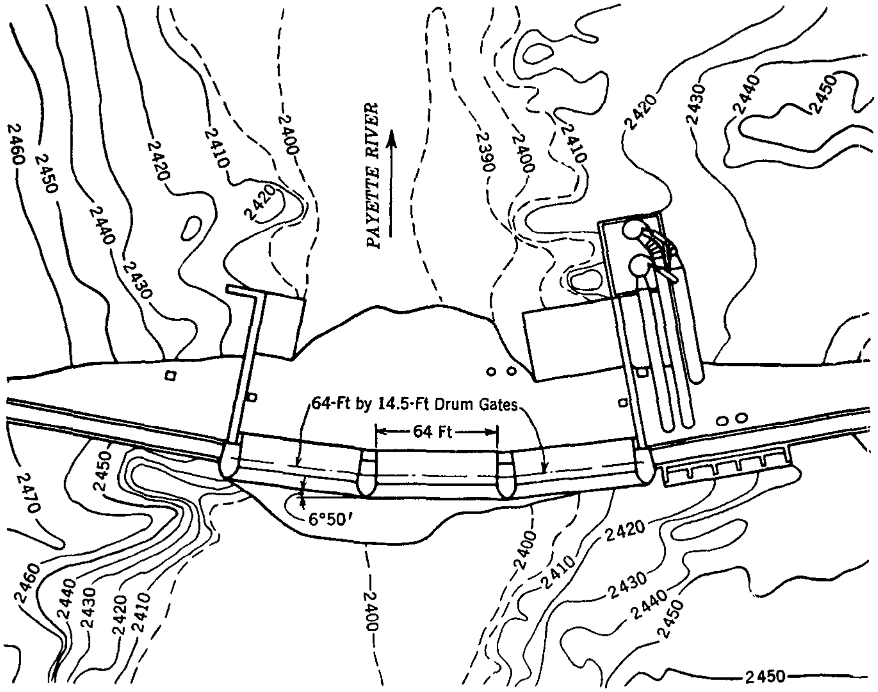


FIGURE 230.—Plan of Black Canyon Dam in Idaho. (After Bradley, 1953.)

discharge at dams (Hulsing, 1967, p. 13–23). If a discharge measurement has been made under the condition of flow over a free crest, the results of the measurement are used to check the value of design head and design-head coefficient, using the technique described under the heading "Index-measurement method" in the previously cited manual by Hulsing (1967, p. 23–24). The design head (H_o) of Black Canyon diversion dam was found to be 14.5 ft and the corresponding coefficient of discharge (C_o) was found to be 3.48.

With the coefficient of discharge known for free flow at the design head, the entire free-flow coefficient curve can be established by use of figure 232. The free-flow coefficient curve for the spillway of Black Canyon diversion dam (H_0 = 14.5 ft; C_0 = 3.48) is constructed by arbitrarily assuming several values of H/H_0 and reading the corresponding values of C/C_0 in figure 232. The method of computation is illustrated in table 25, and the head-coefficient curve for free flow (gate down) obtained in that manner is shown in figure 233.

Before considering the rating of the spillway with gates in raised positions, it is necessary to construct a diagram, such as that shown in figure 234, to relate gate elevation to the angle Θ for the Black

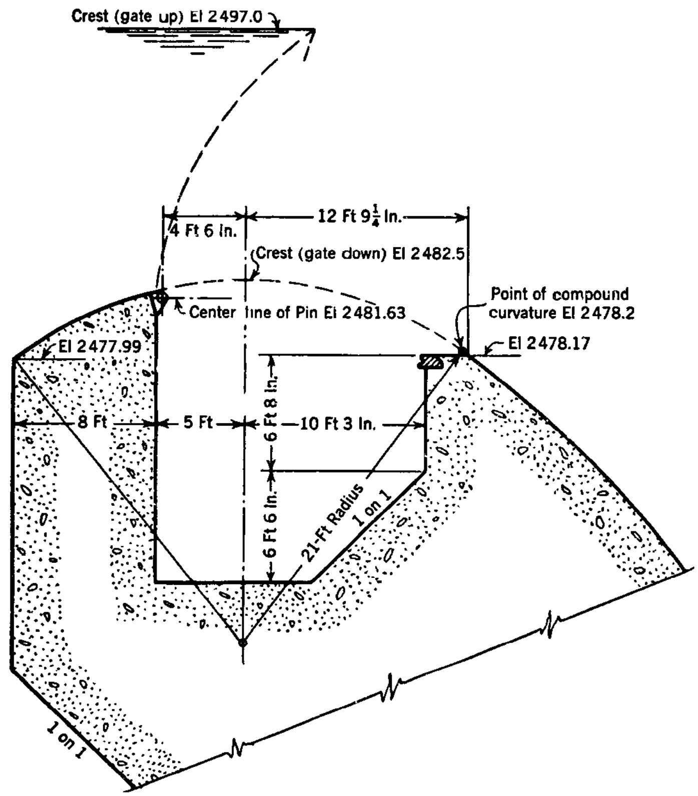


FIGURE 231.—Spillway crest detail, Black Canyon Dam, Idaho. (After Bradley, 1953.)

Canyon Dam gate. The tabulation in figure 234 shows the angle Θ for corresponding elevations of the downstream lip of the gate at intervals of 2 ft.

Beginning with the maximum positive angle of the gate, which is 34.883°, the computation may be started by choosing a representative number of reservoir elevations as indicated in column 2 of table 26. The difference between the reservoir elevation and the high point of the gate constitutes the total head on the gate, and values of head are recorded in column 3. Column 4 shows these same heads divided by the radius of the gate, which is 21.0 ft.

The discharge coefficients listed in column 5 (table 26) of the set of computations designated "A", are obtained by entering the curves in figure 229 with the values in column 4 for $\Theta = +34.883°$. The remainder of the procedure outlined in columns 6 and 7 of table 26, consists of computing the discharge for one gate from the equation,

$$Q = C_a bH^{3/2}. \quad (124)$$

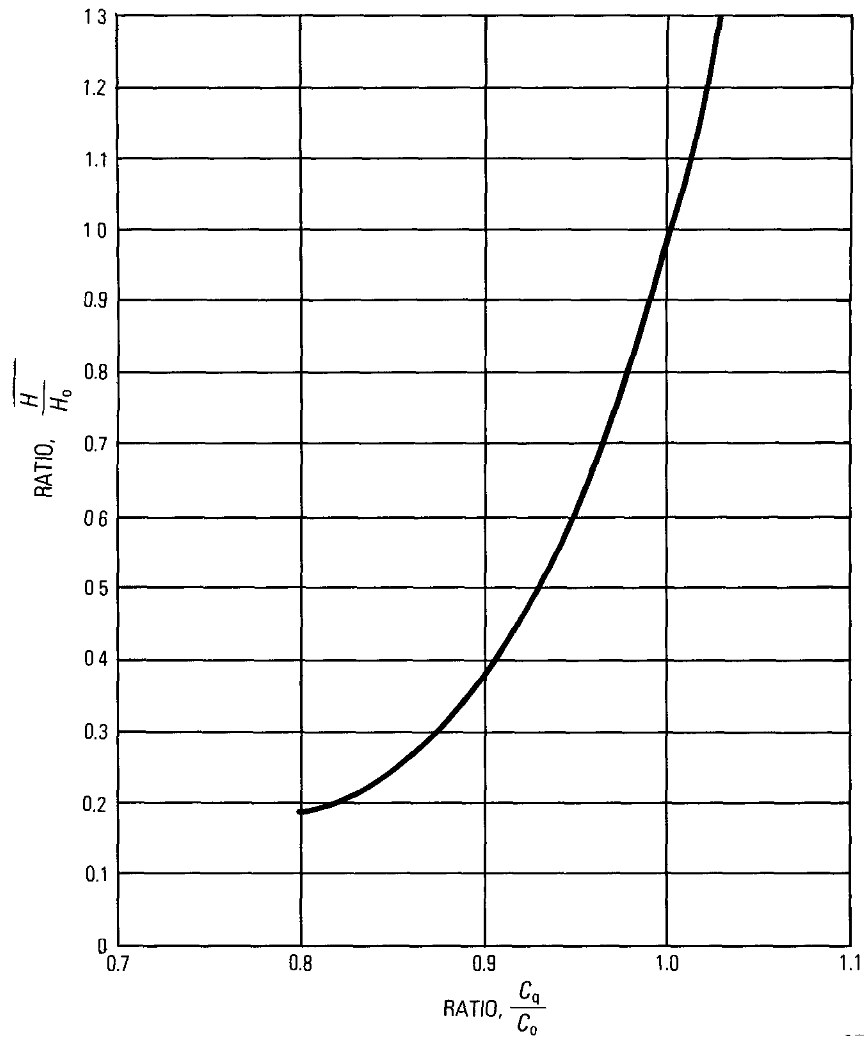


FIGURE 232.—Diagram for determining coefficients of discharge for heads other than the design head (After Bradley, 1953.)

A similar computation procedure is repeated for other positive angles of Θ, as in sets B, C, and D of table 26.

For positive values of angle Θ the high point of the gate is the downstream lip of the gate. As the angle Θ decreases to negative values, the high point of the gate is no longer the downstream lip. In determining the discharge for negative values of Θ between 0° and −15°, the procedure remains the same as was used for positive values

TABLE 25.—*Head and Discharge Computations for a Free Crest (Black Canyon Dam in Idaho)*

[After Bradley, 1953]

Total head, H, in ft (1)	Reservoir elevation, in ft (2)	Ratio,[1] H/H_o (3)	Ratio,[2] C_o/C_o (4)	Coefficient, C_o (5)	Q, in ft^3/s [3] (6)
17	2499.5	1.172	1.020	3.55	15,950
16	2498.5	1.104	1.012	3.52	14,420
14.5	2497.0	1.0	1.0	3.48	12,296
12	2494.5	0.827	0.980	3.41	9,072
10	2492.5	0.690	0 960	3.34	6,759
8	2490.5	0.552	0.940	3.27	4,736
6	2488.5	0.414	0.905	3.135	2,949
4	2486.5	0.276	0.850	2.957	1,514
3	2485.5	0.207	0.815	2.835	943
2	2484.5	0.138	0.760	2.642	478

[1] H_o = 14.5 ft. [2] C_o = 3.48. [3] The discharge for one gate: $Q = C_o bH^{3/2}$, in which b = 64.0 ft.

of Θ, but as mentioned above, the controlling difference between reservoir elevation and high point of the gate is no longer the head above the downstream lip. (See fig. 234.) Discharge computation for negative angles down to −15.017° are tabulated in sets E, F, and G of table 26.

The plotting of values of discharge, reservoir elevation, and gate elevation from table 26, results in the seven curves in figure 235 that bear the plotted points, shown by closed circles. An eighth curve, the extreme lower curve, which bears plotted points shown by X's, represents the discharge of the free crest with the gate completely down; the plotted points represent values obtained from table 25.

The discharge values shown in figure 235 are for one gate only. When more than one gate is in operation, the discharges from the separate gates may be totaled, providing the gates are each raised the same amount. The experimental models used in this study had from one to eleven gates operating, so that a reasonable allowance for pier effect on the discharge is already present in the results.

The intervals between the eight curves in figure 235 that are identified by plotted points are too great for rating purposes, particularly the gap between gate elevations 2485.75 and 2482.5 ft. That deficiency is remedied by cross-plotting the eight curves for various constant values of discharge as shown in figure 236. Fortunately the result is a straight-line variation for any constant value of discharge. The lines in figure 236 are not quite parallel, and there is no assurance that they will be straight for every drum gate. Nevertheless, this uncertainty will not detract appreciably from the accuracy obtained. Interpolated information from figure 236 is then utilized to construct the additional curves in figure 235. Figure 235 now shows the rating

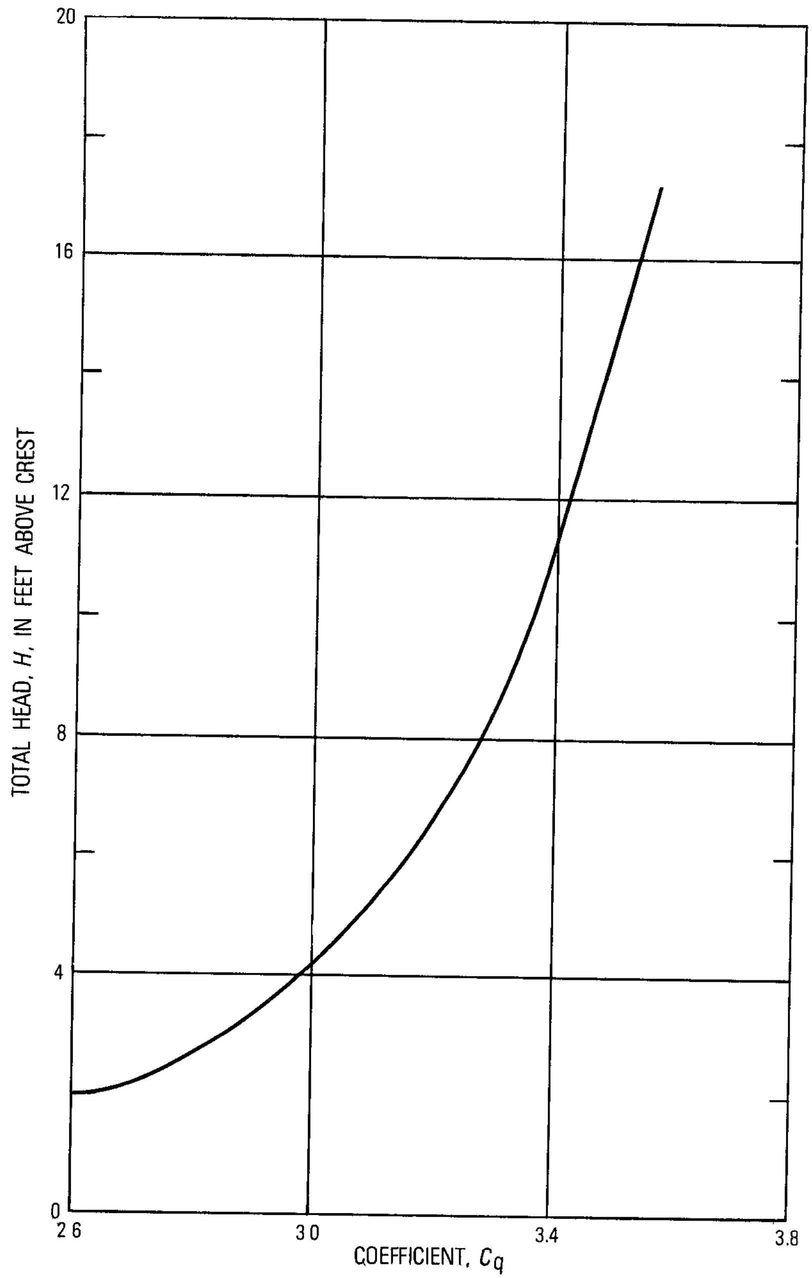
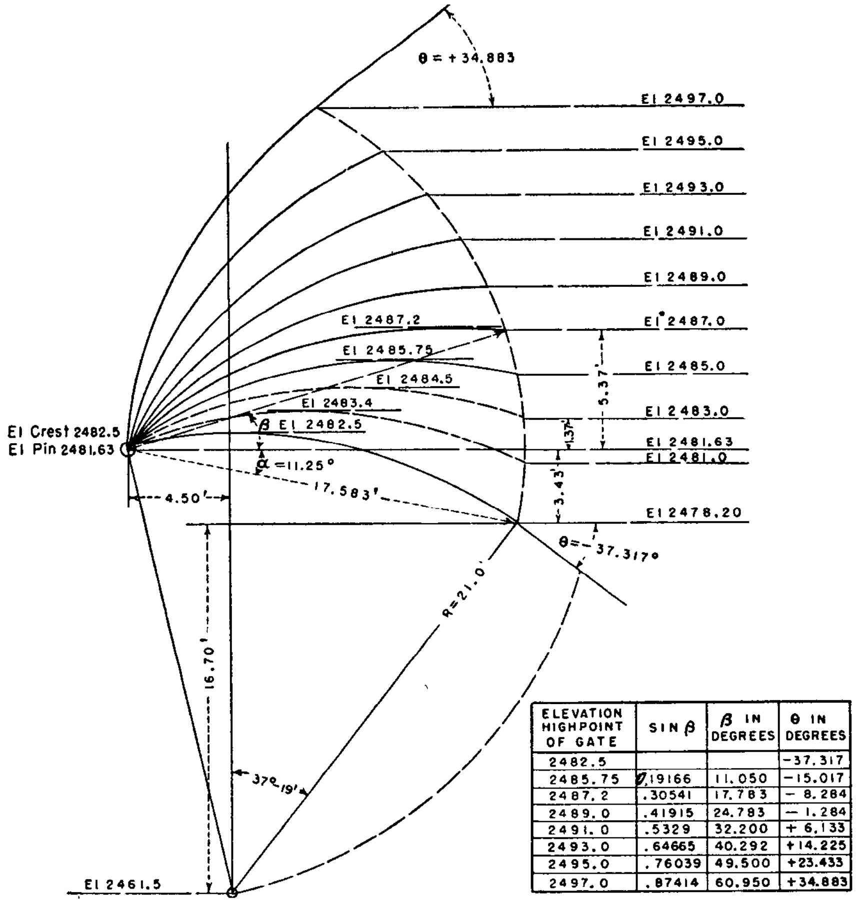


FIGURE 233.—Head-coefficient curve, Black Canyon Dam, Idaho. (After Bradley, 1953.)



ELEVATION HIGHPOINT OF GATE	SIN β	β IN DEGREES	θ IN DEGREES
2482.5			-37.317
2485.75	0.19166	11.050	-15.017
2487.2	.30541	17.783	- 8.284
2489.0	.41915	24.783	- 1.284
2491.0	.5329	32.200	+ 6.133
2493.0	.64665	40.292	+14.225
2495.0	.76039	49.500	+23.433
2497.0	.87414	60.950	+34.883

FIGURE 234.—Relation of gate elevation to angle θ. (After Bradley, 1953.)

for the Black Canyon Dam spillway for gate intervals of 0.5 ft. For intermediate values, straight-line interpolation is permissible.

RADIAL OR TAINTER GATES

The damming face of a radial or Tainter gate is essentially a segment of a hollow steel cylinder spanning between piers on the dam crest. The cylindrical segment is supported on a steel framework that pivots on trunnions embedded in the downstream part of the piers. The gate is raised or lowered by hoisting cables that are attached to each end of the gate; the cables lead to winches on a platform above the gate. In its closed position, the lower lip of the gate rests on the dam crest.

TABLE 26.—*Head and discharge computations for drum gates in raised positions*

[After Bradley, 1953]

Set (1)	Reservoir elevation, in ft (2)	H, in ft (3)	Ratio, $\frac{H}{r}$ (4)	Coefficients, C_0 (5)	$H^{3/2}$ (6)	Q, in ft³/s[b] (7)
	Gate Elevation 2497.0, θ = + 34.88°					
A	2498.0	1	0.048	3.86	1	247
	2499.0	2	0.095	3.86	2.828	699
	2500.0	3	0.143	3.86	5.196	1.283
	Gate Elevation 2495.0, θ = + 23.43°					
B	2496.0	1	0.048	3.85	1	246
	2497.0	2	0.095	3.86	2.828	698
	2498.0	3	0.143	3.87	5.196	1,281
	2499.0	4	0.190	3.87	8.00	1,979
	2500.0	5	0.238	3.88	11.18	2,770
	Gate Elevation 2493.0, θ = +14.22°					
C	2494.0	1	0.048	3.69	1	236
	2495.0	2	0.095	3.73	2.828	675
	2496.0	3	0.143	3.75	5.196	1,247
	2498.0	5	0.238	3.80	11.18	2,719
	2500.0	7	0.333	3.84	18.52	4,552
	Gate Elevation 2491.0, θ = + 6.13°					
D	2492.0	1	0.048	3.47	1	222
	2493.0	2	0.095	3.51	2.828	635
	2494.0	3	0.143	3.57	5.196	1,187
	2496.0	5	0.238	3.63	11.18	2,597
	2498.0	7	0.333	3.70	18.52	4,386
	2500.0	9	0.429	3.77	27.00	6,515
	Gate Elevation 2489.0, θ = − 1.28°					
E	2490.0	1	0.048	3.21	1	205
	2491.0	2	0.095	3.28	2.828	594
	2492.0	3	0.143	3.34	5.196	1,111
	2494.0	5	0.238	3.45	11.18	2,469
	2496.0	7	0.333	3.545	18.52	4,202
	2498.0	9	0.429	3.63	27.00	6,273
	2500.0	11	0.524	3.695	36.48	8,627
	Gate Elevation 2487.2, θ = − 8.28°					
F	2488.0	0.8	0.038	3.02	0.716	138
	2489.0	1.8	0.086	3.10	2.415	479
	2490.0	2.8	0.133	3.17	4.685	950
	2492.0	4.8	0.229	3.31	10.52	2,229
	2494.0	6.8	0.324	3.43	17.73	3,892
	2496.0	8.8	0.419	3.51	26.10	5,863
	2498.0	10.8	0.515	3.58	35.49	8,131
	2500.0	12.8	0.610	3.635	45.79	10,653
	Gate Elevation 2485.75, θ = − 15.02°					
G	2487.0	1.25	0.060	3.00	1.398	268
	2488.0	2.25	0.107	3.07	3.375	663
	2489.0	3.25	0.155	3.15	5.859	1,181
	2491.0	5.25	0.250	3.275	12.03	2,522
	2493.0	7.25	0.315	3.375	19.52	4,216
	2495.0	9.25	0.440	3.465	28.13	6,238
	2497.0	11.25	0.536	3.51	37.73	8.548
	2499.0	13.25	0.631	3.595	48.23	11,097

*H is the total head on the gate. [b]The discharge for one gate: $Q = C_0 b H^{3/2}$.

RADIAL GATES ON A HORIZONTAL SURFACE

Experimental work has been performed to determine discharge coefficients for radial gates that control flow along a horizontal surface (Toch, 1953). The results of those experiments are shown in figures 237 to 240. Figure 237 is a definition sketch for a radial gate on a horizontal surface. The discharge coefficient, C_d, is defined as

$$C_d = \frac{q}{b(2gh_0)^{1/2}} \tag{125}$$

where q is discharge per unit width of gate, g is acceleration of gravity, and h_0 and b are elements shown in the definition sketch (fig. 237).

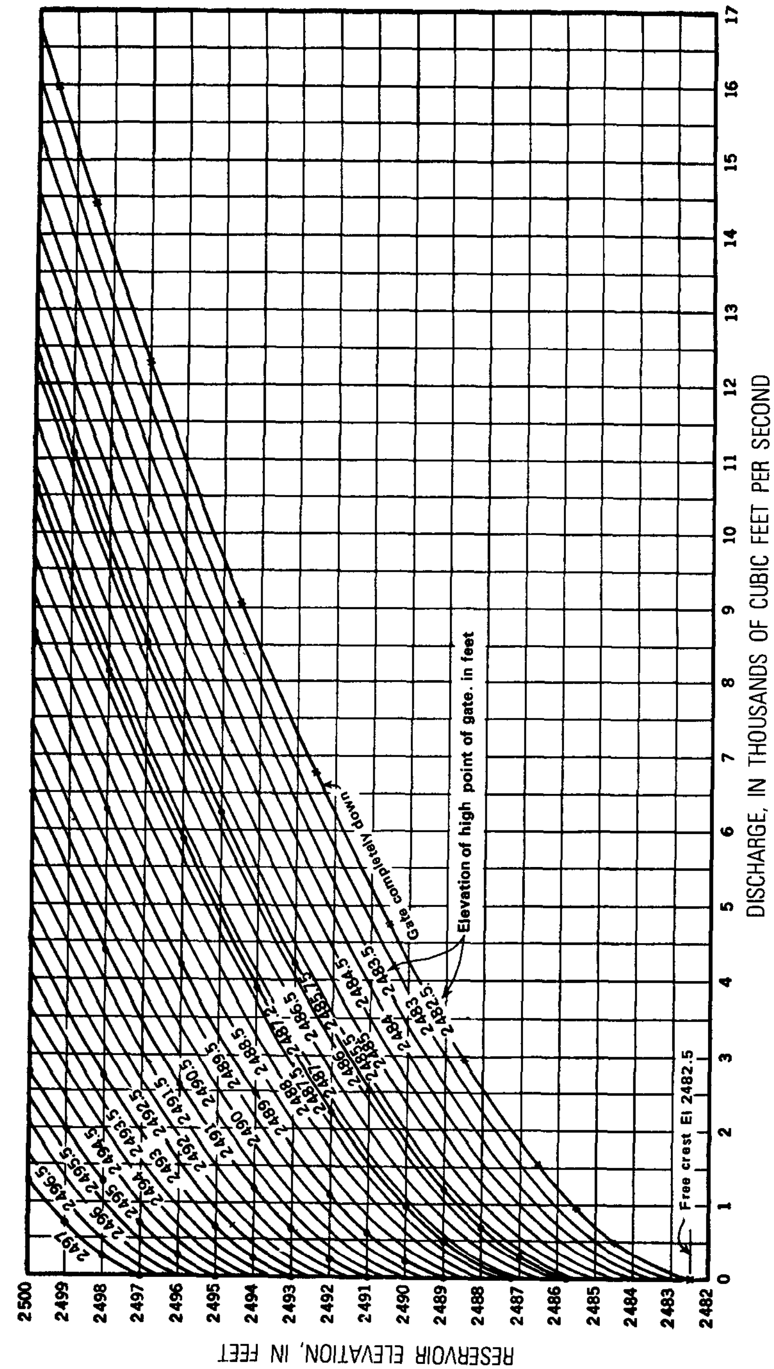


FIGURE 235.—Rating curves for drum-gate spillway of Black Canyon Dam, Idaho. (After Bradley, 1953.)

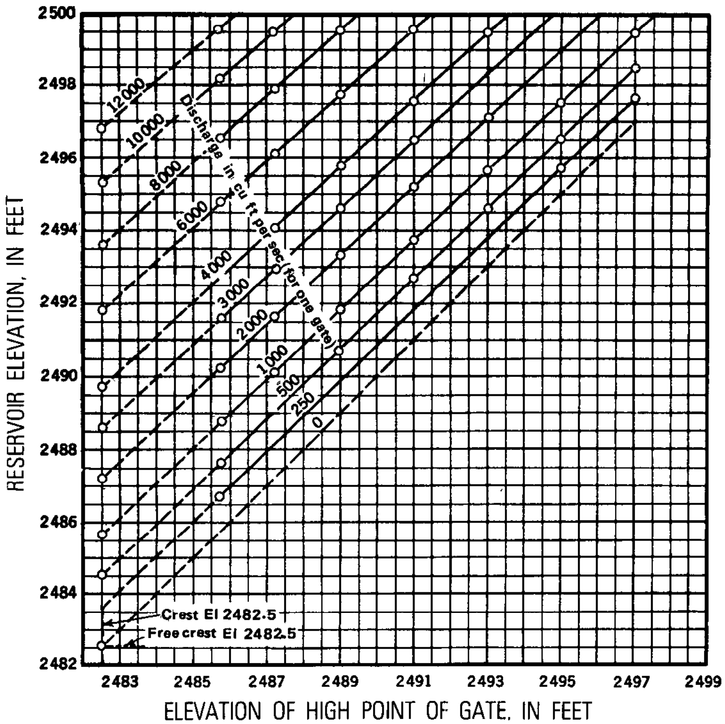


FIGURE 236.—Cross-plotting of values from initial rating curves, Black Canyon Dam, Idaho. (After Bradley, 1953.)

Figures 238 to 240 show values of C_d for three values of the ratio a/r, where a is trunnion elevation and r is gate radius. In the relations shown in the three figures, all pertinent elements have been made dimensionless by using gate radius, r, as a reference. Thus the relative headwater depth is h_0/r, the relative tailwater depth is h_2/r, the relative height of opening is b/r, and the relative trunnion height is a/r. Free efflux (flow) occurs when $h_2 < b$; submerged efflux occurs when $h_2 \geqslant b$. Each of the three graphs shows values of the coefficient of discharge for:

a. Free efflux for three values of b/r,
b. Submerged efflux for two values of b/r, when $h_2/r = 0.5$, and
c. Submerged efflux for three values of b/r, when $h_2/r = 0.7$.

RADIAL GATES ON A CURVED DAM CREST OR SILL

More commonly radial gates are used to control the flow over a curved dam crest or over a sill. The discharge coefficients determined for a radial gate on a horizontal surface cannot be transferred to a

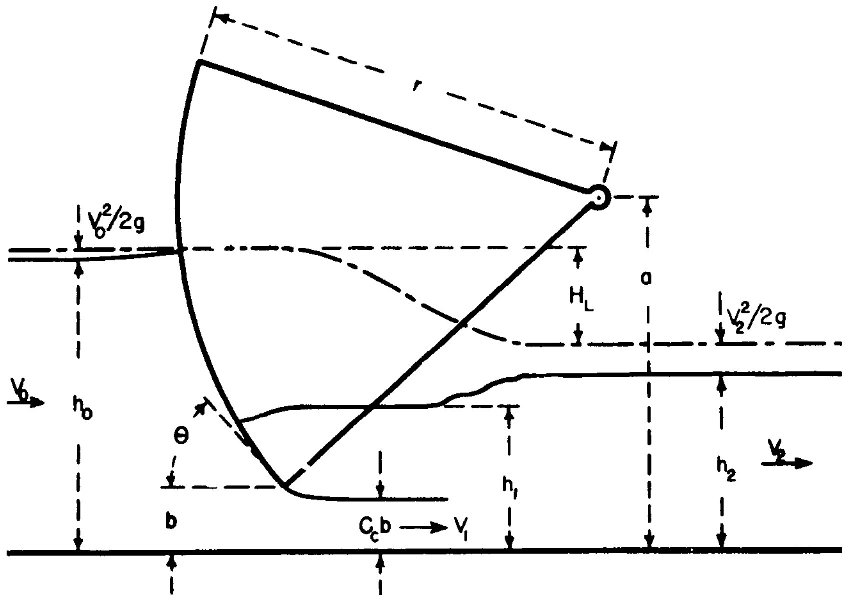


FIGURE 237.—Definition sketch of a radial gate on a horizontal surface. (After Toch, 1953.)

radial gate on a curved dam crest or sill because of differences in the pressure distribution. The flow under radial gates on a curved crest or sill is controlled by the geometry of three interrelated variables—the crest shape, the gate, and the gate setting. Major factors that influence the discharge relations are the position of the gate-seal point with respect to the highest point of the spillway crest and the curvature of the upstream face of the gate. Therefore, experimentally derived discharge coefficients for various prototype dams cannot be transferred to other installations unless the several variables involved are similar. Consequently, radial gates will invariably require rating by current-meter discharge measurements.

When radial gates control the flow over a sill or a curved dam crest, six flow regimes may occur, namely,

1. free orifice flow,
2. submerged orifice flow,
3. free weir flow,
4. submerged weir flow,
5. free flow over closed radial gate, and
6. submerged flow over closed radial gate.

Figure 241 is a definition sketch for the discussions that follow, all of which are concerned with only a single gate. As mentioned earlier in this discussion of movable gates, when discharge measurements for

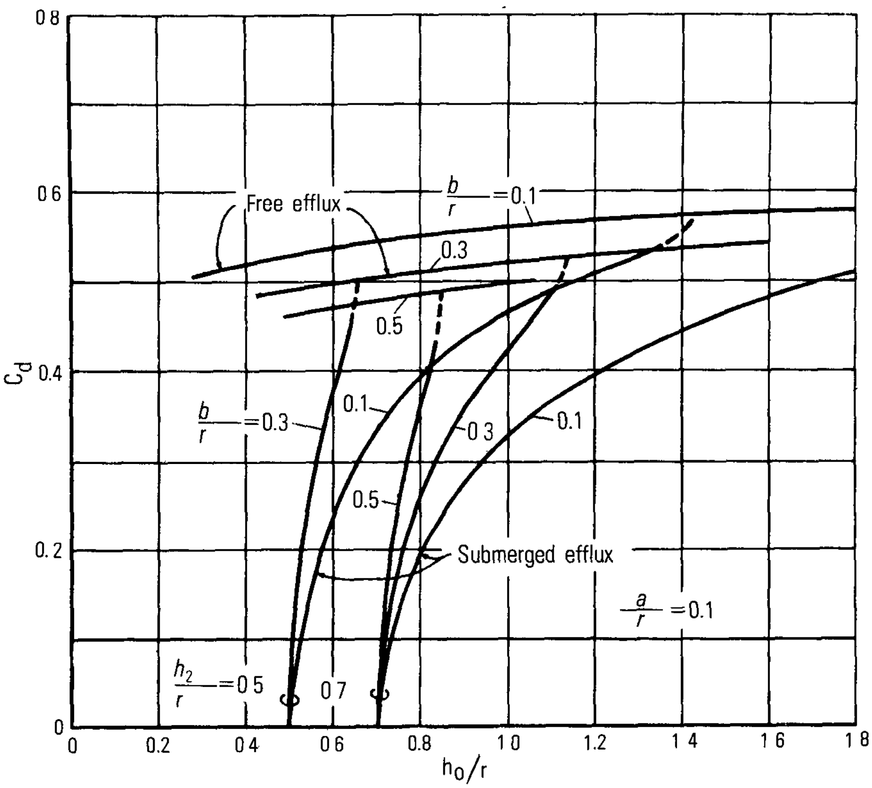


FIGURE 238.—Coefficient of discharge for free and submerged efflux, $a/r = 0.1$. (After Toch, 1953.)

calibration purposes are made with several gates open, it is highly desirable that all gate openings be identical, unless of course the gates are all raised sufficiently for their lower lips to be clear of the water. If gate openings are variable under the condition of orifice flow, it will be necessary to use an average gate opening in computing discharge coefficients for the gates from the measured discharge.

Free orifice flow.—Free orifice flow occurs when the lower lip of the raised gate is submerged by headwater but is above the elevation of tailwater. When the radial gate is on a sill, as in figure 241, free orifice flow occurs under the gate when h_g is less than $(2/3)h_1$, and h_3 is less than h_g. Discharge for that condition is computed from the equation,

$$Q = Ch_g b\ (2gh_1)^{1/2}, \qquad (126)$$

where

Q = discharge for one gate,
c = discharge coefficient,

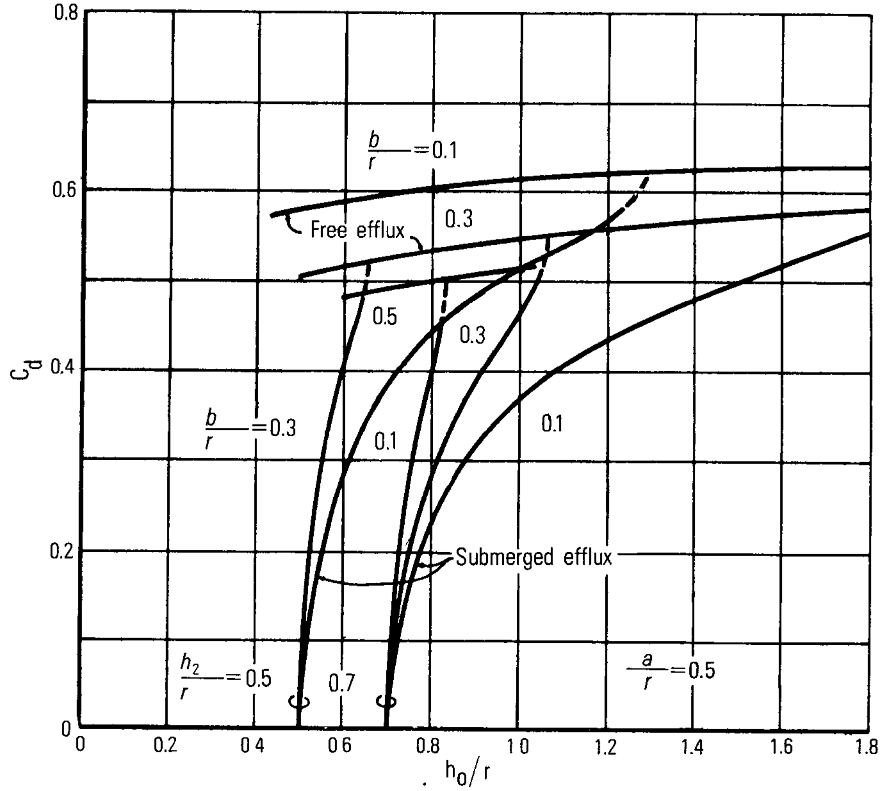


FIGURE 239.—Coefficient of discharge for free and submerged efflux, a/r = 0.5 (After Toch, 1953.)

b = lateral gate length (normal to flow), and
g = acceleration of gravity.

The remaining symbols in equation 126 are defined in figure 241. Values of C will vary inversely with h_g because the change in slope of the lower lip of the gate, as the gate is raised, progressively decreases the hydraulic efficiency of the orifice. There is also a tendency for C to increase with h_1, particularly at low stages, but that effect is usually minor compared to the effect of h_g. Consequently C can usually be related to h_g alone. In developing the relation, discharge measurements should be made throughout the expected range of h_g and h_1. Values of C are then plotted against h_g and the plotted points are fitted with a smooth curve. For convenience in later computations of discharge, the ordinates of the curve are put in tabular form.

The vertical gate opening, h_g, is computed from the following equation based on gate geometry and the position of the reference point at various gate settings:

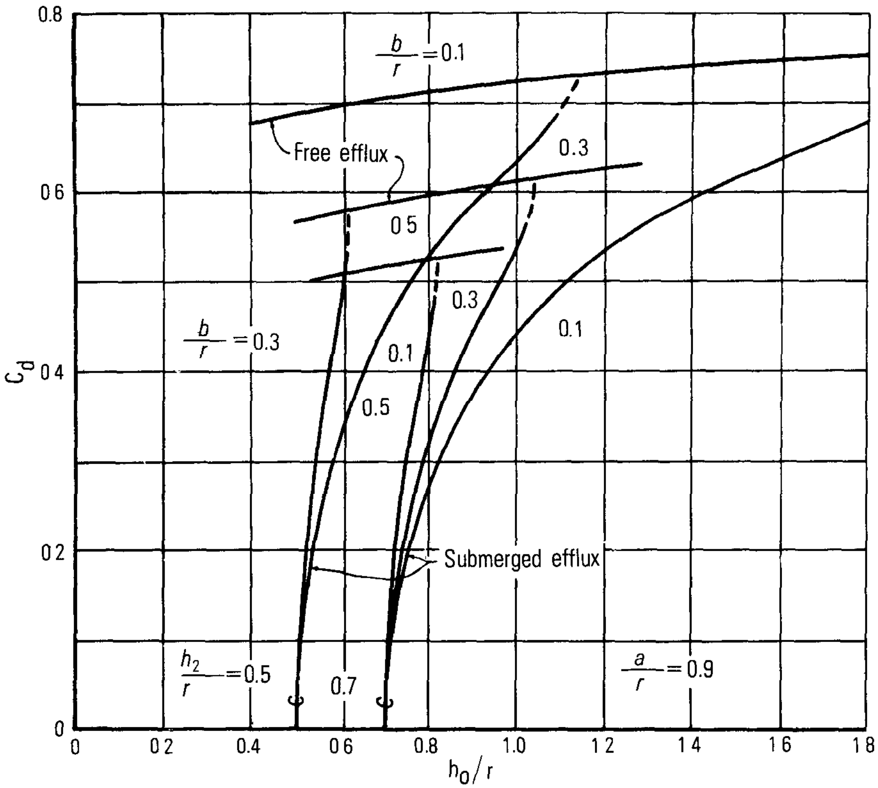


FIGURE 240.—Coefficient of discharge for free and submerged efflux, $a/r = 0.9$. (After Toch, 1953.)

$$h_a = R \cos\Theta\left(\frac{c-a}{r}\right) + a - R \sin\Theta \sqrt{1-\left(\frac{c-a}{r}\right)^2},$$

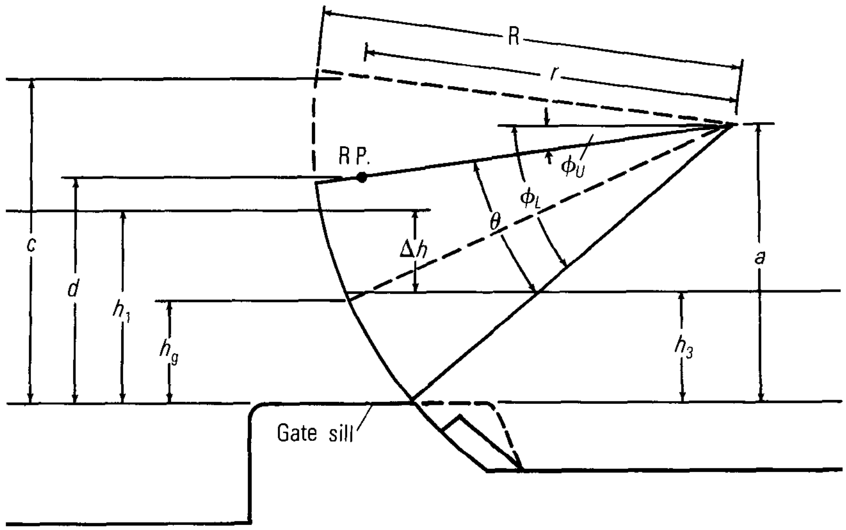
where

$$\Theta = \Phi_L - \Phi_L = \sin^{-1}\left(\frac{a}{R}\right) - \sin^{-1}\left(\frac{a-d}{r}\right).$$

Because C does not vary linearly with h_a it is highly desirable, and often necessary, that all gates be positioned identically during a discharge measurement to avoid the necessity of using an average value of h_a in the computation of C.

Submerged orifice flow.—Submerged orifice flow occurs when the lower lip of the raised gate is submerged by both headwater and tailwater. When the radial gate is on a sill, as in figure 241, submerged orifice flow occurs when h_3 is greater than h_a, and h_a is less than $(2/3)h_1$. The basic equation for computing discharge is

$$Q = C_{as} h_a b (2g\Delta h)^{1/2}, \tag{127}$$



Definitions of symbols used in sketch are:

a = elevation difference, trunnion centerline to sill;
c = elevation difference, gate reference point (R.P.) to sill,
d = elevation difference, gate R.P to sill with the gate in a closed position;
h_1 = static headwater referenced to gate sill;
h_3 = static tailwater referenced to gate sill;
h_g = vertical gate opening,
r = radius from trunnion centerline to gate R.P.,
R = radius from trunnion centerline to upstream face of a Tainter gate;
R P = reference point used as indicator of gate position,
$\Delta h = h_1 - h_3$ = static head loss through structure;
θ = included angle between radial lines from the trunnion centerline through the R P. and through the lower lip of the gate,
ϕ_L = the angle measured from horizontal to the radial line from the trunnion centerline through the lower lip of the gate with the gate in a closed position, and
ϕ_U = the angle measured from horizontal to the radial line from the trunnion centerline through the gate R.P

FIGURE 241.—Definition sketch of a radial or Tainter gate on a sill

where C_{gs} is the coefficient of discharge for a submerged gate. The remaining symbols in equation 127 are defined either in figure 241 or in the preceding discussion of equation 126. Values of C_{gs} are determined from discharge measurements, and in addition, values of h_3/h_g and h_3/h_1 are computed for each measurement. For calibration purposes it is desirable to have measurements that cover the range of 1 to 100 for the ratio h_3/h_g, with several in the range of 1 to 2. The value of C_{gs} is a function of h_g, h_1, and h_3, and the complexity of that function depends on the geometry of the hydraulic structure. The geometry may be such that all computed values of C_{gs} show little variation from a mean value, and when that occurs the mean value of C_{gs} is used in equation 127.

However, computed values of C_{gs} will often vary, particularly in the range of 1 to 2 for the ratio h_3/h_g. If that occurs, three relations involving C_{gs} are plotted graphically, and the one that best fits the plotted points is selected for use. The three relations are:

C_{gs} versus h_g,
C_{gs} versus h_3/h_1, and
C_{gs} versus h_3/h_g.

Quite often the last of the three relations will show the best fit. It will plot as a straight line on logarithmic graph paper and have the general equation,

$$C_{gs} = K(h_3/h_g)^B. \tag{128}$$

When equation 128 is substituted in equation 127, the result is

$$Q = K(h_3/h_g)^B h_g b(2g\Delta h)^{1/2}. \tag{129}$$

Ordinates of the relation indicated by equation 128 are put in tabular form for convenience in later computations of discharge. Because C_{gs} does not vary linearly with h_g, it is highly desirable, and often necessary, that all open gates be positioned identically during a discharge measurement to avoid the necessity of using an average value of h_g in the computation of C_{gs} from measured discharge.

Free weir flow.—Weir flow will occur when the lower lip of the gate is above the water surface. When the radial gate is on a sill, as in figure 241, weir flow will occur when h_g is greater than $(2/3)h_1$, because of drawdown of the water surface at the dam crest; the lower lip of the gate will then be above the water surface. Whether the weir flow is free or submerged will depend on the relative elevations of h_3 and h_1. Free weir flow will occur when the submergence ratio, h_3/h_1, is less than about 0.5–0.7, depending on the geometry of the weir crest. The discharge equation is,

$$Q = C_w b h_1^{3/2}, \tag{130}$$

where C_w is the coefficient of discharge for free weir flow. Values of C_w, which are dependent on the shape of the dam crest, are determined from discharge measurements, and the computed values are then plotted against h_1. Approach velocity head is usually negligible, but even where it is not, its effect is included in the variable coefficient, C_w. Measurements should be made at headwater (h_1) intervals of 1 to 2 feet throughout the expected headwater range to establish the functional relation between C_w and h_1. Information contained in a previously cited report by Hulsing (1967) will usually be helpful as a guide to the probable shape of that relation.

Submerged weir flow.—As mentioned above, weir flow is submerged when the submergence ratio h_3/h_1 is greater than about 0.5–0.7, depending on the geometry of the weir crest. The discharge equation for that condition is

$$Q = C_w C_{ws} b h_1^{3/2}, \tag{131}$$

where C_w is the coefficient previously determined from equation 130. Values of C_{ws}, which is a submergence coefficient, must be determined from discharge measurements and expressed as a function of h_3/h_1. Satisfactory definition of the functional relation will probably require 10–12 discharge measurements well distributed over the range of h_3/h_1. Information contained in the Hulsing report (1967) will often be helpful in the analysis. If the submergence is greater than 0.95 for much of the time, it may be advisable to attempt to develop a relation of discharge to tailwater stage for use during periods of excessive submergence.

Flow over closed radial gate.—At extremely high flows, the closed radial gate may be overtopped, at which time the discharge over the gate is computed from the general weir equation,

$$Q = Cbh^{3/2}, \tag{132}$$

where h is the head on the upper lip of the gate. The gate itself will act as a thin-plate weir. Values of the discharge coefficient C will vary primarily with the geometry of the gate and with h; the geometry of the dam crest or sill will have a lesser effect on the value of C. Discharge measurements will be required to define the rating for flow over the gate, both for unsubmerged flow (tailwater below the upper lip of the gate) and for submerged flow (tailwater above the upper lip of the gate).

Flow over a radial gate can also occur at low stages if the gate is of the submersible type. A submersible gate is designed to be lowered to allow flushing of upstream debris over the top of the gate. When so lowered, the bottom lip of the gate drops below the normal sill elevation. The upper surface of a submersible gate usually has an ogee or rounded crest.

Automated digital recording of elements for computing discharge.—To facilitate the computation of discharge, the Geological Survey has developed an automated digital system for the multiple recording of those elements that are required for discharge computation. The elements monitored are headwater, tailwater, and individual crest-gate positions. At navigation dams additional elements recorded include the number of lockages and, where supplemental hydroelectric power is produced, turbine pressure drops or commercial turbine monitor outputs. All of these elements are recorded on a digital recorder at preselected time intervals, usually hourly or bihourly. The recorded values of headwater, tailwater, and gate settings are the instantaneous values of those elements at the time of recording. The lockage count recorded is the number of lockages between the recordings. The turbine monitor integrates turbine pressure drops over the time interval between recordings.

All data at a site are recorded on paper punch tape in a preselected sequence by a master control console that queries the individual monitors in sequence. The punched tape is removed, usually once a month, and information from that tape is transferred to a magnetic tape. The magnetic tape is then used as input to a computer program for the computation of the streamflow record.

VERTICAL LIFT GATES

Vertical lift gates are simple rectangular gates of wood or steel spanning between piers on the dam crest. The gates move vertically in slots in the piers, and all but the smallest gates are mounted on rollers to reduce the friction caused by the hydrostatic force on the gate. The vertical lift gate, like the radial gate, must be hoisted at both ends, and the entire weight is suspended from the hoisting cables or chains (U.S. Army Corps of Engineers, 1952.) Piers must be extended to a considerable height above high water to provide guide slots for the gate in the fully raised position. To reduce the height of the piers required for operating large vertical lift gates, the large gates are often built in two horizontal sections, so that the upper section may be lifted and placed in another gate slot before raising the lower section. This design also reduces the load on the hoisting mechanism. Discharge may occur over either one or both sections of the gate or over the spillway crest. Discharge over the spillway crest may occur as weir flow if the gate is raised above the water surface, or as orifice flow if the raised gate does not clear the water surface.

The principles that govern the rating of radial gates likewise apply to vertical-lift gates. When the elevation of the lower edge of the raised gate is less than two-thirds of the upstream head, orifice flow occurs. The orifice flow is free if the tailwater is below the lower edge of the raised gate; the orifice flow is submerged if the tailwater is above the lower edge. General equations 126 and 127 apply to the discharge, and values of C and C_{gs} in those equations must be determined from discharge measurements.

If the elevation of the lower edge of a raised gate is greater than two-thirds of the upstream head, weir flow over the dam occurs. If the weir flow is free, equation 130 applies; if the elevation of the tailwater

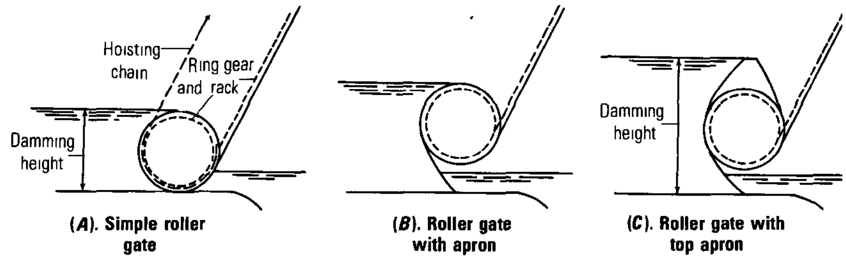


FIGURE 242.—Schematic sketches of roller gates. (U.S. Army Corps of Engineers, 1952.)

causes submergence effect, equation 131 applies. The coefficients in the two weir equations are primarily dependent on the shape of the weir crest. Values of the coefficients are determined from discharge measurements, but helpful information concerning them is found in a report by Hulsing (1967).

When a closed gate is overtopped by headwater, the upper edge of the gate acts as a weir and general equation 132 is applicable. The upper edge of a vertical-lift gate commonly has the shape of a modified horizontal broad-crested weir. Coefficients of discharge are determined from discharge measurements, but again, helpful information is to be found in the Hulsing report (1967).

ROLLER GATES

A roller (or rolling) gate (fig. 242) is a horizontal, internally braced, metal cylinder spanning between piers. Rings of gear teeth at the ends of the cylinder mesh with inclined metal racks supported by the piers, and when a pull is exerted on the hoisting cable or chain, the gate rolls up the rack (fig. 242*A*). The effective damming height of the cylinder can be increased by means of a projecting apron (fig. 242*B*) which rotates into contact with the dam crest as the gate rolls down the inclined racks (U.S. Army Corps of Engineers, 1952). A similar apron or rounded lip may be added to the top of the gate (fig. 242*C*).

As in the case of radial and vertical-lift gates, orifice flow will occur under partly raised rolling gates; weir flow over the dam will occur when the gates are raised sufficiently (⅔ or more of the headwater elevation) to be clear of the water surface, and weir flow over the gates will occur when the closed gates are overtopped by headwater. The principles of rating roller gates are similar to those discussed for radial gates and vertical-lift gates.

MOVABLE DAMS

A movable dam consists of a low concrete sill and a damming surface that can be raised above the water surface to maintain a desired

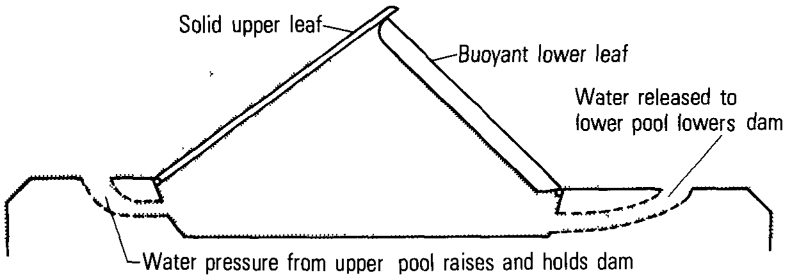


FIGURE 243.—Bear-trap gate. (U.S. Army Corps of Engineers, 1952.)

pool level, or lowered to the sill at higher discharges so as to offer no interference to the flow. The most commonly used gates or damming surfaces are bear-trap gates, hinged-leaf gates, wickets, and inflatable dams.

Bear-trap gate.—A bear-trap gate (fig. 243) consists of two leaves of timber or steel hinged and sealed to the dam or sill. When water is admitted to the space under the leaves, they are forced upward. The downstream leaf is hollow so that its buoyancy aids the lifting operation. When the dam is collapsed by the release of water from under the leaves, the leaves lie flat. (U.S. Army Corps of Engineers, 1952).

Hinged-leaf gate.—A hinged-leaf gate (fig. 244) is a rigid flat leaf hinged at bearings along its lower edge. In its raised position, the leaf slopes upward and downstream at an angle of between 20° and 30° from the vertical. When lowered, it lies approximately in a horizontal position. The position of the leaf is controlled by a mechanical hoist or by a counterweight device that causes the leaf to rise or fall automatically with slight incremental changes in headwater level.

Wickets.—A wicket is a shutter held in position against the water load by a metal prop (fig. 245*A*). It is not intended that water flow over the wicket at an appreciable depth, because the resultant water load will shift to a point above the prop and cause the wicket to overturn or vibrate violently (U.S. Army Corps of Engineers, 1952). The metal prop, hinged at midlength of the wicket, seats against a shoulder on a metal fixture ("hurter") embedded in the foundation. The wicket is raised by an upstream pull on a hoisting line attached to the bottom of the wicket. This causes the prop to fall into its seat after which the wicket is rotated into position against the sill (fig. 245*B*). The wicket is lowered by pulling upstream on a line attached to the top of the wicket; the base of the prop is pulled away from its seat and falls to one side into a groove in the hurter in which it can slide freely downstream. Wickets are raised and lowered by use of a boat operating on the upstream side of the dam. Figures 245*C* and

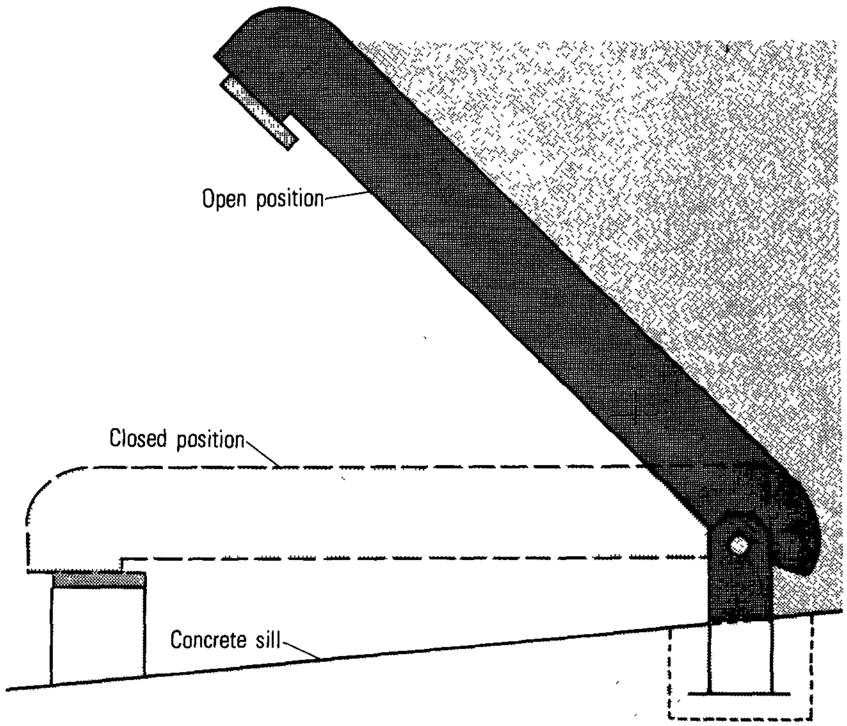


FIGURE 244.—Hinged-leaf gate.

245*D* show improved types of wickets. The Bebout wicket (fig. 245*D*) trips automatically to permit the passage of high flows.

Inflatable dams.—An inflatable dam, before activation, is a collapsed nylon-rubber bladder that occupies the full width of the stream and is attached to a concrete sill on the channel bottom. The dam is activated by pumping water into the bladder, thereby inflating it to form a barrier across the channel. The dam is deactivated by releasing water from inside the bladder. Inflatable dams are usually used on shallow streams to maintain a water level in the stream that is sufficiently high to submerge the intake of a diversion works. When the river stage is high, the dam is deflated. The inflation and deflation are often automatically controlled in response to the changing stage of the stream. Although it would probably be feasible to determine the rating for an inflatable dam by monitoring both the stream stage and the pressure within the dam bladder, inflatable dams have not been used as gaging-station controls. It is invariably simpler to operate a conventional gaging station on the stream either downstream from the inflatable dam or far enough upstream to be beyond the

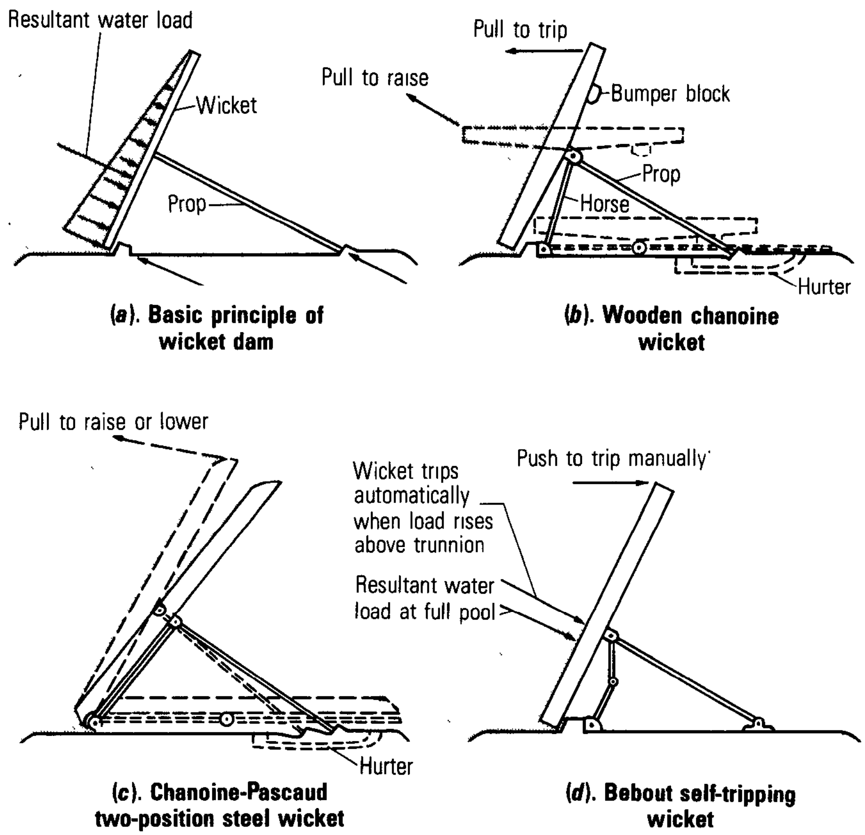


FIGURE 245.—Wickets. (U.S. Army Corp of Engineers, 1952.)

influence of backwater from the dam.

Discharge characteristics.—The discharge characteristics of bear-trap gates, hinged-leaf gates, and wickets are similar. In their lowered position they act as broad-crested weirs that control the stage-discharge relation over a limited range of low-water stage. The stage at which they become submerged depends primarily on the height of the sill on which they rest. Their discharge ratings in the lowered position will resemble that for a highway embankment (Hulsing, 1967, p. 26–27) whose general equation is

$$Q = CbH^{3/2}, \tag{133}$$

where

Q is discharge,
C is the coefficient of discharge,
b is the width normal to the flow, and
H is the total head.

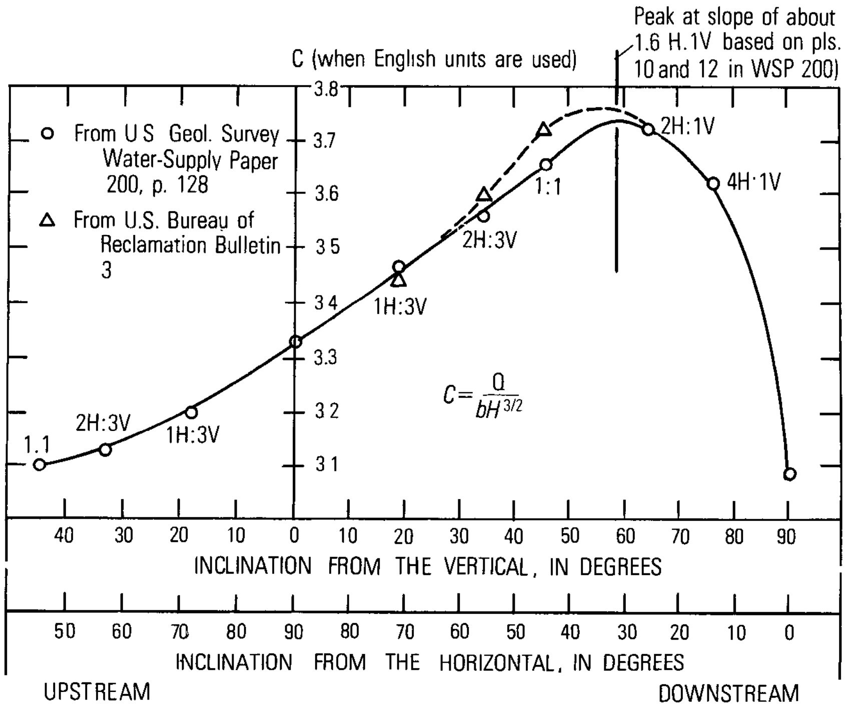


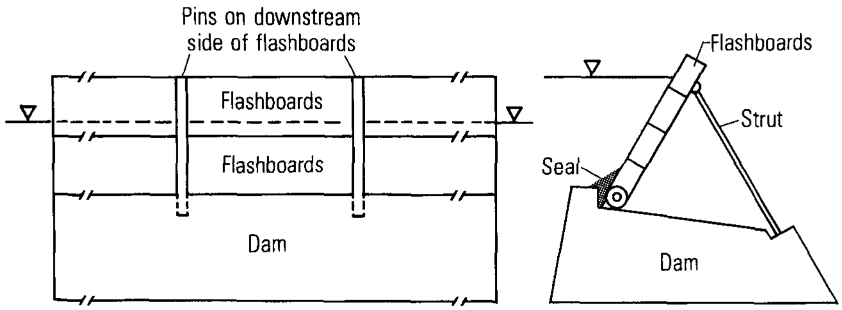
FIGURE 246.—Discharge coefficients for an inclined rectangular thin-plate weir.

The value of C will be dependent on the elevations of headwater and tailwater, the length of the crest in the direction of flow, and the geometry of the crest. For unsubmerged flow (tailwater $\leq$ 0.7 times headwater) C, when English units are used, can be expected to range from about 2.6 to 3.1, depending primarily on the ratio of static head (h) to length of sill in the direction of flow (L). For submerged flow, the free-flow value of C will be multiplied by a factor that ranges from almost zero to almost 1.00, depending on the degree of submergence.

When overtopped in their raised position by headwater, the three types of movable dam—bear-trap gate, hinged-leaf gate, and wickets—act as inclined thin-plate rectangular weirs. Figure 246 gives values of the discharge coefficient C in the general weir equation (eq. 133) for various angles of inclination of such weirs. If the upstream edge of the crest is rounded, the value of C may increase by 5–10 percent.

FLASHBOARDS

The usual flashboard installation consists of horizontal wooden panels supported by vertical pins placed on the crest of a spillway (fig. 247*A*). Such installations are temporary and are designed to fail



(A). Temporary type (Front view)

(B). Permanent type (End view)

FIGURE 247.—Flashboards.

when the water surface in the reservoir reaches a predetermined level. A common design uses steel pipe or rod set loosely in sockets in the crest of the dam and designed to bend and release the flashboards at the desired water level. Temporary flashboards of this type have been used in heights up to 4 or 5 ft. Because temporary flashboards are lost each time the supports fail, permanent flashboards are more economical for large installations. Permanent flashboards usually consist of horizontal wooden panels that can be raised or lowered from an overhead cableway or bridge. The bottom edge of the panels is placed in a seat or hinge on the spillway crest, and the panels are supported in the raised position by struts (fig. 247*B*) or by attaching the top edge of the panels to the bridge.

To rate the vertical flashboards shown on figure 247*A*, a value of $C = 3.33$ (English units) is usually used in the general weir equation,

$$Q = CbH^{3/2} \tag{133}$$

As for the permanent flashboards in figure 247*B*, when the flashboards are lowered, the value of C that should be used is that for the free dam crest (no flashboards). The value of C to use when the flashboards are raised and supported by struts is determined from figure 248, which shows C values for various angles of inclination. If the raised flashboards are supported in an inclined position by a bridge so that the top edge of the flashboards is flush with the upstream edge of the bridge floor, we have in effect a flat-crested rectangular weir with inclined upstream face. The bridge floor acts as the flat weir crest and the flashboards act as the inclined upstream face of the weir. Discharge is computed by the use of equation 133; the value of C to be used in that equation can be obtained from figure 151 (chap. 10).

STOP LOGS AND NEEDLES

Stop logs consist of horizontal timbers, similar to flashboards, spanning between vertically slotted piers on the dam crest. The timbers may be inserted into, or removed, from the vertical slots by hand or with a hoist. There is usually considerable leakage between the timbers and considerable time may be required for their removal if they become jammed in the slots. Stop logs are ordinarily used only for small installations where the cost of more elaborate devices is not warranted or in situations where the removal or replacement of the stop logs is expected only at infrequent intervals.

Needles consist of timbers standing on end with their lower ends resting in a keyway in the spillway and their upper ends supported against the upstream edge of a bridge floor. Needles are easier to remove than stop logs but are difficult to place in flowing water. Consequently, they are used mainly for emergency bulkheads that are installed during periods of low flow.

The simple crest shape of stop logs and needles makes it easy to determine the theoretical value of the discharge coefficient C in the general weir equation 133. (See report by Hulsing (1967) on computing discharge over dams.) However, it is usually futile to rate stop logs or needles theoretically because of the appreciable leakage between them.

NAVIGATION LOCKS

Navigation locks are required for boat traffic to overcome the difference between headwater and tailwater elevations at a dam. The boat enters the open gate of the lock; the lock is closed behind the boat; valves are used for filling or emptying the locks, as the case may be, to bring the water level in the lock to that of the pool ahead of the boat; the other lock gate is opened and the boat proceeds on its journey. Various lock-filling and lock-emptying systems have been devised as a compromise between two conflicting demands: (1) that the filling time be short so as not to delay traffic, and (2) that the disturbances in the lock chamber not cause stresses in mooring hawsers which might cause the boat or barges to break loose and thereby damage either the boat or lock structure.

The flow through navigation locks is computed as the total volume of water released during a finite time interval, usually 1 day. The volume of water discharged for any one lockage is the product of the plan or water-surface area of the lock and the difference between headwater and tailwater at the time of lockage. These volumes are summed for the day and divided by 86,400, which is the number of seconds in a day, to obtain the average lockage flow in cubic feet per second or cubic meters per second. Usually it will be sufficiently accurate to compute the daily average lockage discharge (Q_L) by use of the equation,

$$Q_L = \frac{N}{86{,}400} A(h_h - h_t), \tag{134}$$

where

N is the number of lockages in a day,
A is the plan or surface area of the lock,
h_h is the daily mean headwater elevation, and
h_t is the daily mean tailwater elevation.

If appreciable leakage through the lock occurs between boat lockages, the daily average leakage must be added to the daily average lockage discharge.

MEASUREMENT OF LEAKAGE THROUGH NAVIGATION LOCKS

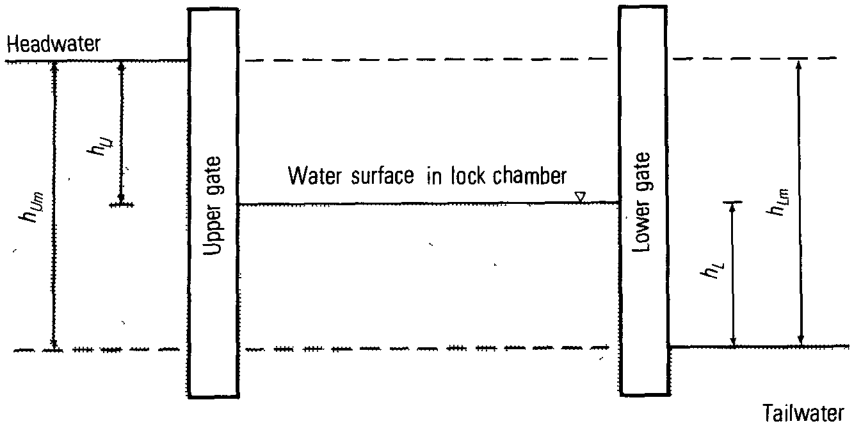
If the leakage through the closed lock gates is great, it can be measured in the forebay with a low-velocity current meter. The leakage will seldom be that great, however, and it usually will have to be computed by a volumetric method.

If, for considerable periods of time between lockages, the lockmaster keeps the valves and lower gates closed and the upper gates open, leakage will occur through the lower gates, and it is that leakage (Q_{Lm}) that must be determined. If, instead, it is the valves and upper gates that are kept closed and the lower gates that are kept open, leakage will occur through the upper gates, and it is that leakage ($Q_{t\,m}$) that must be determined. If all valves and gates are kept closed, it is the equilibrium leakage (Q_{Le}) through the lower gate that must be determined.

Instructions for determining Q_{Lm}, $Q_{t\,m}$, and Q_{Le} follow. Figure 248 is a definition sketch of a lock.

FIELD WORK

1. Close upper and lower lock gates and open the valve to fill the lock chamber. When the lock chamber is filled, close the valve and open one upper gate slightly.
2. Attach the zero end of a steel tape by a small staple to the middle of a long plank. Float the plank in a lock chamber against the lock wall after first setting a reference mark on top of the wall for use as an index for reading the tape. A portable electric-tape gage (see section in chapter 4 titled, "Electric-Tape Gage") is even more satisfactory for reading stages in the lock chamber.
3. Record gage heights in the upper pool and lower pool and the tape reading in the lock chamber.
4. Close the upper gate. Read the tape immediately after the gate is fully closed and seated, and start a stop watch. Thereafter, read the



Definitions

$h_{Um}=h_{Lm}$	Maximum head on upper or lower gates for given headwater and tailwater stages
h_U	Head on upper gate
h_L	Head on lower gate
Q_U	Leakage through upper gate produced by h_U
Q_{Um}	Leakage through upper gate produced by h_{Um}
Q_L	Leakage through lower gate produced by h_L
Q_{Lm}	Leakage through lower gate produced by h_{Lm}
Q_n	Rate of storage in lock with both gates closed$=Q_L-Q_U$. (When Q_n is negative, the water level rises in lock chamber, when Q_n *is positive, the water level falls in lock chamber.)*
h_{Ue}	Equilibrium head on upper gate when $Q_U=Q_L$
h_{Le}	Equilibrium head on lower gate when $Q_U=Q_L$
Q_{Le}	Leakage through lower gate produced by h_{Le}

$$h_U+h_L=h_{Um}=h_{Lm}$$
$$h_{Ue}+h_{Le}=h_{Um}=h_{Lm}$$

FIGURE 248.—Definition sketch of a lock.

tape and stop watch at intervals of about 0.5 ft as stage decreases in the chamber, or at 1-minute intervals, whichever comes first. Continue for about 10 minutes.

5. Empty the lock chamber by opening the lower gate, and then partly close the lower gate; that is, leave one lower gate slighty open.

6. Record gage heights in the upper pool and lower pool, and the tape reading in the lock chamber.

7. Close the lower gate. Read the tape immediately after the gate is fully closed and seated and start a stopwatch. Thereafter read the tape and stopwatch at intervals of about 0.5 foot as stage increases in the chamber, or at 1-minute intervals, whichever comes first. Continue for about 10 minutes.

8. Obtain dimensions of the lock chamber for use in computing

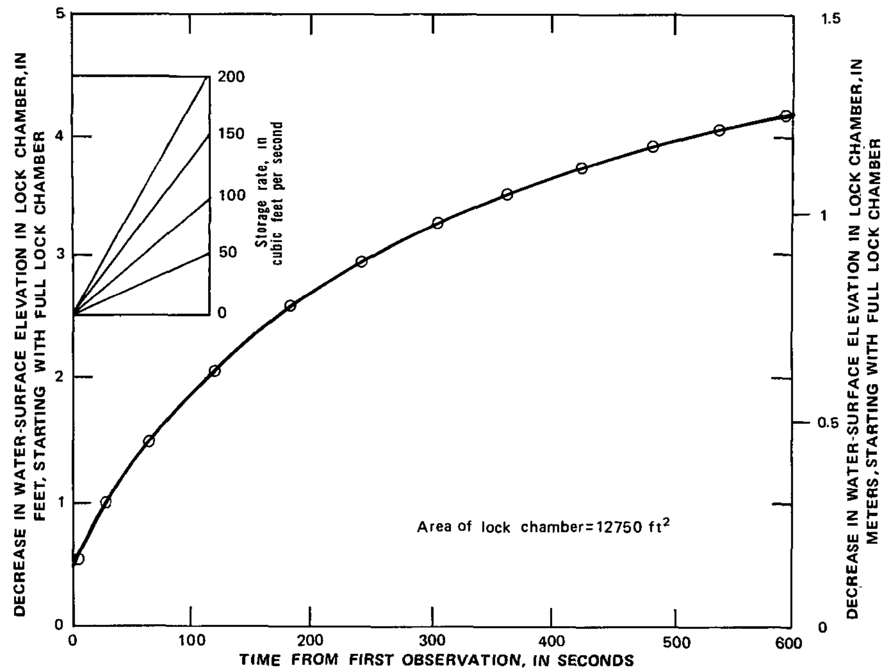


FIGURE 249.—Storage diagram starting with lock chamber full.

volumes of water involved in the leakage. That completes the field work.

COMPUTATIONS FOR Q_{Lm}

1. Use readings obtained when observations were started with a full lock chamber. Subtract initial tape reading (made with upper gate open slightly) from all tape readings.

2. Plot adjusted tape readings from step 1 against time in seconds. The first reading made after the upper gate was fully closed is plotted at zero seconds. Too much uncertainty usually exists as to when the gate actually seated to use the closure of the gate as the starting time for the graph. See figure 249. The plot should be made on a large sheet of graph paper.

3. Connect the plotted points with a smooth curve. A tangent to the curve at any value of the abscissa represents the rate of change of water-surface elevation at that instant. The rate of change multiplied by the surface area of the lock chamber gives the instantaneous rate of storage in the lock chamber; that is, the difference in rate of leakage out of the chamber through the lower gate and rate of leakage into the chamber through the upper gate.

At the instant the upper gate is closed, the leakage out of the chamber is at its maximum, Q_{Lm} (full head on the lower gate), and the leakage into the chamber is zero (zero head on the upper gate). As the stage in the chamber falls, the leakage out of the chamber decreases because of the decreased head on the lower gate, and leakage into the chamber increases because of the increased head on the upper gate. Eventually leakage into the lock would equal the leakage out of the lock (Q_{Le}), and the stage in the chamber would remain constant.

4. In order to obtain the rate of storage at any instant from the tangent of the curve showing the decrease in lock stage with time, construct a diagram showing the storage rate (Q_{s}) for various tangential slopes.

The method of constructing the diagram is demonstrated in figure 249. The area of the lock chamber is 12,750 ft^2. If the stage in the chamber dropped 2 feet, the change in volume would be 2 × 12,750 or 25,500 ft^3. If Q_{s} were 200 ft^3/s, the time required for a 2 ft drop would be 127.5 seconds. A vertical line is drawn at 127.5 seconds on figure 249 and a diagonal line having a drop of 2 ft is drawn between the abscissa values of 0 and 127.5 seconds. A tangent to the storage curve having a similar slope would have a Q_{s} value of 200 ft^3/s. Diagonals representing other values of Q_{s} are added as shown.

5. Select two points on the storage curve, one near the origin (0 seconds) and the other no more than 1 ft lower in stage. Draw tangents to those points and use the slopes of those tangents with the tangential rate diagram to obtain the two values of Q_{s}. To obtain the tangential slope at a point on the curve, use a pair of dividers to lay off short equal distances on the curve on each side of the selected point. A chord connecting the equidistance points will have a slope approximately equal to that of the tangent.

6. The two values of Q_{s} obtained in the preceding step will be used to compute Q_{Lm}. No further use will be made of the leakage curve, except that it has value for making a rough check on the basic assumption that will be made in the computations that follow. That assumption is that the leakage through a gate can be treated as though it all occurred at an orifice at the bottom of the gate. In other words,

$$\frac{Q_L}{Q_{Lm}} = \left(\frac{h_L}{h_{Lm}}\right)^{1/2} \quad \text{and} \quad \frac{Q_U}{Q_{Um}} = \left(\frac{h_U}{h_{Um}}\right)^{1/2}$$

or

$$Q_L = Q_{Lm}\left(\frac{h_L}{h_{Lm}}\right)^{1/2} \quad \text{and } Q_U = Q_{Um}\left(\frac{h_U}{h_{Um}}\right)^{1/2} \qquad (135)$$

7. From figure 248 and equation 135.

$Q_{s} = Q_L - Q_U$

or

$$Q_n = Q_{Lm}\left(\frac{h_L}{h_{Lm}}\right)^{1/2} - Q_{Um}\left(\frac{h_U}{h_{Um}}\right)^{1/2} \qquad (136)$$

For each of the two values of Q_n, all values in equation 136 are known, except for the values of Q_{Lm} and Q_{Um}. The known values can be substituted in equation 136 to give two simultaneous equations, which can then be solved for the desired value of Q_{Lm}.

8. In the preceding step, it would be a simple matter to solve for Q_{Um}, but we do not do so. Our basic assumption of orifice flow may not be strictly correct, and experience has shown that the desired value of Q_{Um} can be computed with much more accuracy by using the field data obtained when observations of leakage were started with an empty lock chamber.

9. Obtain values of leakage through the lower gate when the upper gate is open, for other values of total head. Use the following equation:

$$Q'_{Lm} = Q_{Lm}\left(\frac{h'_{Lm}}{h_{Lm}}\right)^{1/2} \qquad (137)$$

where Q_{Lm} and h_{Lm} are values obtained from a leakage test as described above, and Q'_{Lm} is the leakage through the lower gate corresponding to any other value of total head, h'_{Lm}.

10. Prepare a rating table of Q_{Lm} versus h_{Lm}.

COMPUTATIONS FOR Q_{Um}

1. Use readings obtained when observations were started with an empty lock chamber. Subtract initial tape reading (made with lower gate slightly open) from all tape readings.

2. Plot adjusted tape readings from step 1 against time in seconds.

3. Proceed with computations in a manner analogous to that used in the computation of Q_{Lm}.

4. Obtain Q_n for two points on the leakage curve, one near the origin (0 seconds) and the other no more than 1 ft higher in stage.

5. Use equation 136 to solve for the desired value of Q_{Um}.

6. Obtain values of leakage through the upper gate when the lower gate is open, for other values of total head. Use the following equation:

$$Q'_{Um} = Q_{Um}\left(\frac{h'_{Um}}{h_{Um}}\right)^{1/2}, \qquad (138)$$

where Q_{Um} and h_{Um} are values obtained from a leakage test as described above, and Q'_{Um} is the leakage through the upper gate corresponding to any other value of total head h'_{Um}.

7. Prepare a rating table of Q_{Um} versus h_{Um}.

COMPUTATIONS FOR Q_{Le}

1. Q_{Le} is the leakage through the lower gate when equilibrium exists; that is, the stage in the lock chamber is constant because $Q_U = Q_L$.

2. Starting with the equation, $Q_{Ue} = Q_{Le}$, it is a simple matter to transform the equation to

$$h_{Le} = h_{Lm} \Big/ \left[(Q^2_{Lm} / Q^2_{Um}) + 1\right] \tag{139}$$

All values on the right-hand side of equation 139 are known because in preceding steps Q_{Lm} and Q_{Um} had been computed. Solve for h_{Le}.

3. Obtain the desired value of Q_{Le} from the equation

$$Q_{Le} = Q_{Lm}\left(\frac{h_{Le}}{h_{Lm}}\right)^{1/2}. \tag{140}$$

4. Use the rating tables for Q_{Lm} and Q_{Um} with equations 139 and 140, to prepare a rating table of Q_{Le} versus h_{Lm}.

PRESSURE CONDUITS

GENERAL

In one respect, the gaging of a pressure conduit is simple in that the cross-sectional area is constant for all discharges. The calibration of the metering device offers difficulty, however, because the discharge measurements require special instrumentation unless they can be made by current meter in the forebay or afterbay of the conduit where open-channel conditions exist.

The following are the metering devices used for pressure conduits:

1. Mechanical meters
   a. Displacement meter
   b. Inferential meter
   c. Variable-area meter
2. Differential-head meters
   a. Constriction meters
      (1) Venturi meter
      (2) Flow nozzle
      (3) Orifice meter
   b. Bend meter
   c. Pressure differential in a reach of unaltered conduit
3. Electromagnetic velocity meter
4. Acoustic velocity meter
5. Laser flowmeter.

Changes in the rating of mechanical meters occur only as a result of wear on the moving parts of the meter. Changes in the rating of differential-head meters that are kept clean occur only as a result of changes in perimeter roughness of the conduit with time. The electromagnetic, acoustic, and laser velocity meters are complex electronic devices, and as such, they are subject to the occasional calibration drift that for various reasons affect such devices.

The various meters must be calibrated when first installed, and the calibration must be periodically checked thereafter. Methods of measuring discharge for that purpose include:

1. pitot-static tubes and pitometers,
2. salt-velocity method, and
3. Gibson method.

This section of the manual closes with a brief discussion of discharge ratings for turbines, pumps, gates, and valves, all of which are associated with pressure conduits.

METERING DEVICES FOR PRESSURE-CONDUIT FLOW

MECHANICAL METERS

Mechanical meters are widely used in water-distribution systems because of their low cost and small size, but they can only be used to measure a relatively narrow range of discharge. They are not suited for the measurement of very low flow rates because the liquid may pass the meter without moving the mechanical elements; they are seldom used to measure discharges greater than 10 ft^3/s (0.28 m^3/s) because of high head loss. A large variety of mechanical meters are commercially available, but only the three general types—displacement, inferential, and variable-area—will be described here (Howe, 1950, p. 210–212).

Displacement meters.—An elementary form of displacement meter consists of a single or multiple piston arrangement in which fluid passing through the meter moves a piston back and forth. The movement of the piston is readily registered upon a counting device calibrated in any desired units to give total volume of flow. Such meters can have a fairly large capacity and are accurate if no slippage occurs.

Another commonly used displacement meter is the disk meter which oscillates in a measuring chamber; for each oscillation a known volume of water passes the meter. The motion of the disk operates a gear train which in turn activates a counting mechanism, thereby furnishing a measure of the total volume of flow. When the disk is new, the meter is accurate to within 1 percent, but the meter may underregister significantly as the disk becomes worn.

Inferential meters.—Inferential meters are in effect small turbines and are called "inferential" because the rate of flow is inferred from the speed of rotation of the propeller. An essential element of such meters is a set of guide vanes which may be adjusted to change the calibration of the meter. However, the calibration may inadvertently change if the surface of the propellor blades becomes worn or coated. Although inferential meters normally register only volume of flow, equipment may be added to the meter to indicate instantaneous rate of discharge.

Variable-area meter.—The variable-area meter consists of a vertical tapered tube containing a small plunger "float." In some instruments the plunger is completely immersed in a transparent, graduated tube; in others, a stem projects through the end of the conical tube and traverses a scale. In both types the plunger rises as the rate of flow increases, thereby increasing the area around it. By calibration, the position of the plunger can be related to the rate of flow. These instruments are restricted to the measurement of rather small discharges and will not accommodate any great change in viscosity without recalibration. Accuracy within 1 percent is possible.

DIFFERENTIAL-HEAD METERS

The flow of fluid through a constriction in a pressure conduit results in a lowering of pressure at the constriction. The drop in piezometric head in the reach between the undisturbed flow and the constriction is a function of the flow rate. The venturi meter, flow nozzle, and orifice meter (fig. 250) are constriction meters that make use of this principle. The difference in piezometric head may be measured with a differential manometer or pressure gages. In order that such an installation may function properly, a straight length of pipe at least 10 diameters long should precede the meter. Straightening vanes may also be installed in the conduit just upstream from the meter to suppress disturbances in the flow.

Venturi meters.—Venturi meters (fig. 250*A*) are highly accurate and efficient flow meters; they have no moving parts, require little maintenance, and cause little head loss (U.S. Bureau of Reclamation, 1971). They operate on the principle that flow in a given closed-conduit system moves more rapidly through areas of small cross section (D_2 in fig. 250*A*) than through areas of large cross section (D_1 in fig. 250*A*). The total energy in the flow, consisting primarily of velocity head and pressure head, is virtually the same at D_1 and D_2 within the meter. Thus the pressure must decrease in the constricted throat, D_2, where the velocity is higher; and conversely the pressure must be greater at D_1, upstream from the throat, where the velocity is lower. This reduction in pressure from the meter entrance to the meter throat is directly related to the rate of flow passing through the meter and is the measurement used to determine flow rate. Tables or dia-

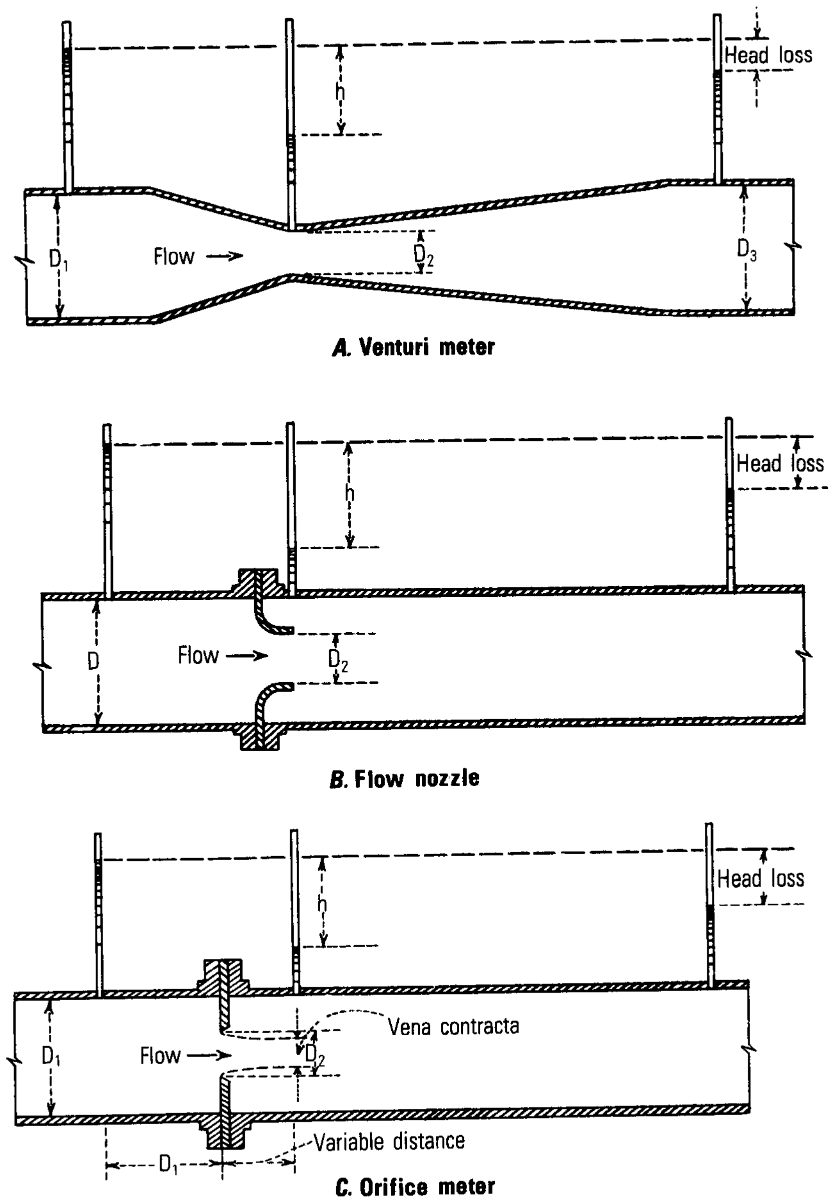


FIGURE 250.—Three types of constriction meter for pipe flow. (Courtesy of U.S. Bureau of Reclamation.)

grams of this head differential versus rate of flow may be prepared, and flow indicators or flow recorders may be used to display the differential or the rate of flow.

The relation of rate of flow, or discharge, to the head and dimensions of the meter is

$$Q = \frac{CA_2\sqrt{2gh}}{\sqrt{1 - r^4}}, \tag{141}$$

where

A_2 = cross-sectional area of the throat, in square feet,
h = difference in pressure head between upstream pressure-measurement section and the downstream pressure-measurement section, in feet,
g = 32.2 feet per second per second,
r = ratio of the throat diameter to pipe diameter = D_2/D_1, and
C = coefficient of discharge for the venturi meter.

The coefficient of discharge for the venturi meter varies with a Reynolds number that is based on the diameter and velocity at the throat and on the kinematic viscosity of the water; the kinematic viscosity of the water is in turn a function of the water temperature. The formula for computing the Reynolds number is

$$R = \frac{V_2 D_2}{\nu}, \tag{142}$$

where

R = Reynolds number (dimensionless),
V_2 = mean velocity in the throat (ft/s),
D_2 = throat diameter (ft), and
ν = kinematic viscosity (ft^2/s).

Table 27 gives values of kinematic viscosity corresponding to selected

TABLE 27.—*Values of kinematic viscosity corresponding to selected water temperatures*

[From American Society of Civil Engineers (1942, p 60)]

Water temperature (°F)	Kinematic viscosity ($\nu \times 10^{5}$) (ft^2/s)
32	1.931
40	1.664
50	1.410
60	1.217
70	1.059
80	.930
90	.826
100	.739
110	.667
120	.609
130	.558
140	.514
150	.476
160	.442
170	.413
180	.385
190	.362
200	.341
212	319

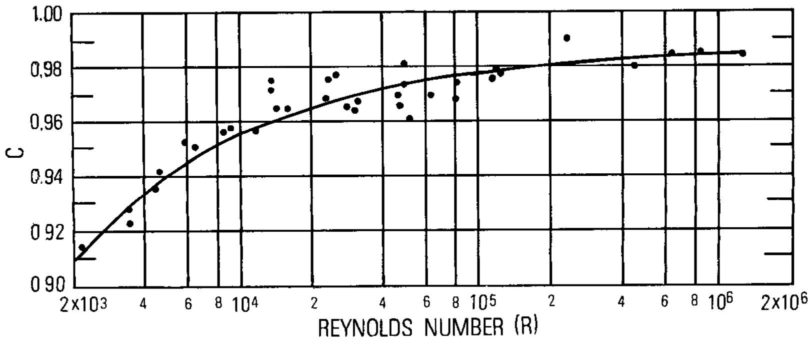


FIGURE 251.—Discharge coefficients for venturi meters as related to Reynolds number. (After Howe, 1950. Reprinted by permission of John Wiley & Sons, Inc.)

water temperatures. Figure 251 shows values of the discharge coefficient for venturi meters as related to the Reynolds number. Figure 251 is based on discharge data for meters having a diameter ratio (r) equal to 0.5, and although the discharge coefficient will vary slightly with the geometry of the venturi meter, the relation shown in the figure is considered accurate to within 1 percent for meters that are carefully maintained.

Flow nozzles.—Flow nozzles operate on the same basic principle as venturi meters. In effect, the flow nozzle is a venturi meter that has been simplified and shortened by omitting the long diffuser on the outlet side (fig. 250*B*). The streamlined entrance of the nozzle provides a straight cylindrical jet without contraction, so that the coefficient is similar, in considerable degree, to that for the venturi meter. In the flow nozzle, the jet is allowed to expand of its own accord, and the high degree of turbulence created downstream from the nozzle causes a greater loss of head than occurs in the venturi meter where the long diffuser suppresses turbulence.

The relation of rate of flow to the head and dimensions of the flow nozzle is

$$Q = \frac{CA_2\sqrt{2gh}}{\sqrt{1 - r^4}}, \tag{141a}$$

which is identical with equation (141) given above for the venturi meter. The symbols have the same meaning in both equations, except that C in equation 141a is the coefficient of discharge for the flow nozzle.

Specifications for the manufacture and installation of flow nozzles vary, and extensive research on the various types has resulted in the accumulation of a large body of data on discharge coefficients. Space limitations preclude detailed discussion of those coefficients, but it may be stated that for the more popular types of design and in the usual range of operation the coefficients generally range from 0.96 to 0.99; for any type of flow nozzle, the discharge coefficients increase with Reynolds number and tend to become constant at Reynolds numbers greater than 10^6. A recommended source of data on discharge coefficients is a report of the American Society of Mechanical Engineers (1937).

The upstream pressure connection for measuring head is frequently made through a hole in the wall of the conduit at a distance of about 1 pipe diameter upstream from the starting point of the flare of the nozzle. The pressure observed is that of the stream before it has begun to turn inward in response to the inlet curvature of the nozzle. The downstream pressure connection may be made through the pipe wall opposite the end of the nozzle throat.

Orifice meters.—A thin-plate orifice inserted across a pipeline can be used for measuring flow in much the same manner as a flow nozzle (fig. 250*C*). The upstream pressure connection is often located at a distance of about 1 pipe diameter upstream from the orifice plate. The pressure of the jet ranges from a minimum at the vena contracta—the smallest cross section of the jet—to a maximum at about 4 or 5 conduit diameters downstream from the orifice plate. The downstream pressure connection—the center connection shown in figure 250*C*—is usually made at the vena contracta to obtain a large pressure differential across the orifice. The location of the vena contracta may be determined from data provided in standard hydraulic handbooks.

The relation of rate of flow to the head and dimensions of the metering section is

$$Q = \frac{CA_2\sqrt{2gh}}{\sqrt{1 - r^4}}, \tag{141b}$$

which is identical with equations 141 and 141a except that C in equation 141b is the coefficient of discharge for the orifice meter.

For pressure taps located 1 pipe diameter upstream from the orifice plate and at the vena contracta, the coefficient of discharge ranges from 0.599 for an r value of 0.20 to 0.620 for an r of 0.71, when the Reynolds number exceeds 2×10^5. The principal disadvantage of orifice meters, as compared to venturi meters or flow nozzles, is their greater loss of head. On the other hand, they are inexpensive and are capable of producing accurate flow measurements.

Bend meters.—Another type of differential head meter is the bend meter, which utilizes the pressure difference between the inside and outside of a pipe bend. The meter is simple and inexpensive. An elbow already in the line may be used without causing added head loss. For best results a bend meter should be calibrated in place. The meter equation is

$$Q = CA\sqrt{2gh}\,, \tag{143}$$

where C is the coefficient of discharge, h is the difference in piezometric head between the outside and inside of the bend at the midsection, and A is the cross-sectional area of the pipe. For best results it is recommended that the lengths of straight pipe upstream and downstream from the bend be equal to at least 25 pipe diameters and 10 pipe diameters, respectively.

Lansford (1936) experimented with 90° bends, and he concluded that if calibration of a 90° bend is not feasible, results at moderate to high Reynolds numbers that are accurate to within 10 percent can be obtained from a simple formula for C, in which

$$C = \sqrt{r/(2D)}\,. \tag{143a}$$

In equation 143a, D is the pipe diameter and r is the centerline radius of the bend.

Pressure differential in a reach of unaltered conduit.—If a pressure-conduit system has high velocities and low pressures, it may not be practical to install a venturi meter in the line because cavitation will occur in the throat along with excessive vibration. In that situation the installation of a manometer between two piezometer taps in the conduit, several hundred feet apart, may be the most feasible method of metering the flow. One but preferably two discharge measurements would suffice to rate the manometer, and a third measurement could be made to check the rating equation which is,

$$Q = K\sqrt{\Delta h}\,, \tag{144}$$

where

Q = discharge,
K = a constant, and
Δh = head differential.

If two discharge measurements are used in the initial calibration, the two computed K values, which should agree closely, are averaged.

In the case of reaction turbines, the discharge may be metered by a manometer that measures the pressure drop in the scroll case. The scroll case of a reaction turbine has a decreasing diameter, being largest at its upstream end where it is joined to the penstock. A set of piezometer taps is installed at each end of the scroll case forming, in effect, a type of venturi section. Discharge is computed by use of equation 144, K being determined from discharge measurements, preferably made over the complete range of output, and simultaneous observations of the pressure drop. The calibration should remain constant as long as the turbine efficiency does not change.

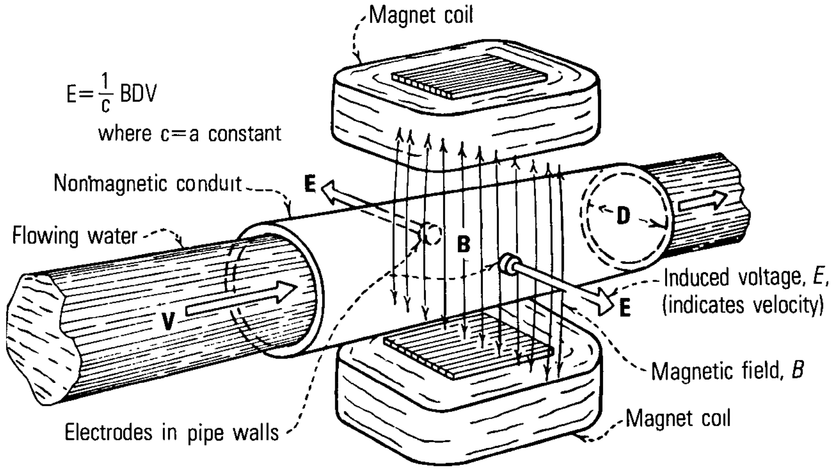


FIGURE 252.—Schematic view of one type of electromagnetic velocity meter. (Courtesy of U.S. Bureau of Reclamation.)

Summary.—Differential-head meters are very satisfactory metering devices as long as they are kept clean and the velocities in the conduit are high enough to give significant pressure differentials between the two piezometer taps.

ELECTROMAGNETIC VELOCITY METER

Electromagnetic velocity meters for measuring flow in pressure conduits are commercially available. The principle of the electromagnetic velocity meter was explained in the section in chapter 12 titled, "Electromagnetic Velocity-Meter Method"; but to repeat briefly, when a fluid which is an electric conductor moves across a magnetic field at 90°, as shown in figure 252, an electromotive force is produced in the fluid at right angles to both the flux of the magnetic field and the velocity of the fluid. The induced voltage is proportional to the average velocity of the fluid, V. If the pipe is a conductor, as it usually will be, an insulating liner must be installed in the metering section and the probes must contact the water. Two or more discharge measurements are required to calibrate the meter.

ACOUSTIC VELOCITY METER

Acoustic velocity meters for measuring flow in pressure conduits are commercially available. The principle of the acoustic velocity meter was explained in the section in Chapter 12 titled, "Acoustic Velocity-Meter Method," and will not be discussed further other than to state that better results are obtained with the transducers of the meter in direct contact with the fluid stream than are obtained with the transducers mounted on the outside of the conduit walls

(Schuster, 1975). The acoustic velocity meter should be calibrated by discharge measurements.

LASER FLOWMETER

Laser (light amplification by stimulated emission radiation) beams have been used for studying the turbulent characteristics of flowing liquids and for determining the velocity of fluid flow (Schuster, 1970). The Doppler principle, which involves a measurable shift in the frequency of the light rays under the influence of an external velocity imposed on the system, underlies the operation of the laser flowmeter. The flowing water scatters part of a beam of light (laser) directed through it. By comparing the frequencies of the scattered and unscattered rays, collected in receiving lenses on the opposite side of the stream, the velocity of the water (hence the discharge) can be calculated. In laboratory experiments, the instrument has measured fluid flows as slow as a fraction of an inch per second and as fast as 1,000 or more feet per second. The device is a valuable research tool, but it should also be considered a possible future device for measuring discharge in both open channels and pressure conduits.

DISCHARGE-MEASUREMENT METHODS FOR METER CALIBRATION

MEASUREMENT OF DISCHARGE BY PITOT-STATIC TUBES AND PITOMETERS

Pitot-static tubes and pitometers may be classed as differential-head meters, but they are seldom used for continuous-flow measurement. Instead, they are usually used for calibrating other metering devices in place and for intermittent measurements. Pitot tubes and pitometers indicate the velocity head at a point in the conduit cross section.

The operation of pitot-static tubes or pitometers is based on the principle that the increase in head at the mouth of a bent tube facing upstream is a measure of the velocity head of the oncoming flow. The most commonly used type of pitot-static tube (fig. 253*A*) consists of two separate parallel tubes, one for indicating total head, P_t (sum of static and velocity heads), and the other for indicating only static (pressure) head, P_s. Manometers are commonly used to measure these heads, the velocity head being the difference between the static head and the total head. A pressure transducer may also be used instead of the manometer for measuring the differential head. Where pitot-static tubes are used for continuous-flow measurement, oscillograph or digital recording of the electrical signal from the transducer provides a continuous record of the changes in head.

The general equation for pitot-static tubes and pitometers is

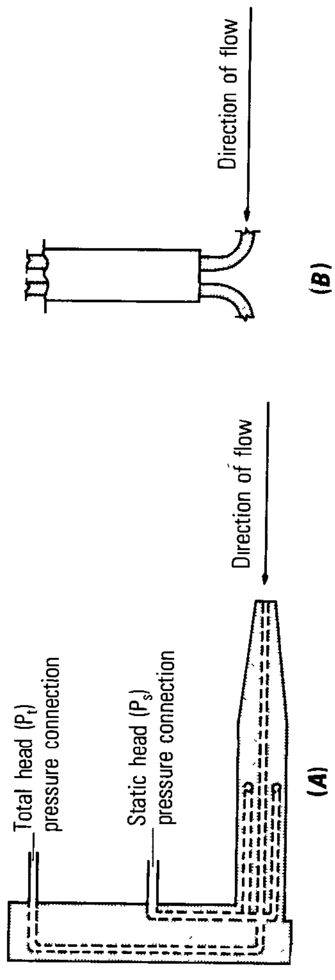


FIGURE 253.—Schematic drawing of (A) pitot-static tube and (B) Cole pitometer.

$$V = C_1\sqrt{2g\Delta h}, \tag{145}$$

where

V = velocity,

C_1 = coefficient,

g = acceleration of gravity, and

Δh = observed velocity-head differential.

The coefficient C_1 will vary with the dimensions and geometry of the meter, but the instruments are usually individually rated by the manufacturer in the manner that current meters are rated, and the value of C_1 is therefore known. For the pitot-static tube shown in figure 253*A* the value of C_1 usually ranges from 0.98 to 1.00.

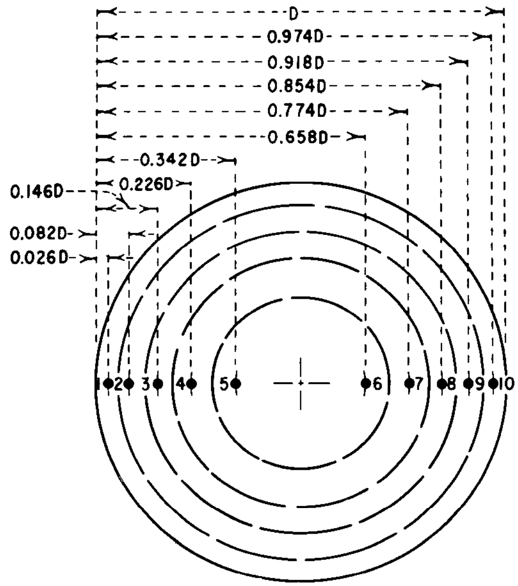
Another commonly used type of pitot device is the Cole pitometer (fig. 253*B*), which consists of two tubes headed in opposite directions. The tubes can be rotated so that the instrument may be inserted through a small bushing in a pipe. When in operating position, the downstream tube registers a negative pressure because its opening is in the wake of the instrument. The differential of the water columns is therefore considerably greater than $V^2/2g$. The value of C_1 in equation 145 usually ranges from 0.84 to 0.87.

Reinforced pitot tubes and pitometers have been used successfully in pipes up to five feet in diameter having flow velocities of 5–20 ft/s (U.S. Bureau of Reclamation, 1971). Even larger pipes can be traversed by pitometer by having access ports on both sides of the pipe and by probing to or past the conduit centerline from each side. The principal disadvantage encountered is that relatively large forces push on the tube when flow velocities are high, making positioning and securing of the instrument difficult. Dynamic instability may also occur, causing the tube to vibrate and produce erroneous readings. At moderate flow velocities the measurements are accurate.

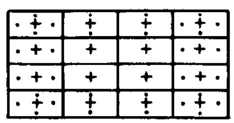
The most common pressure conduit is the circular pipe. For a constant rate of flow, the velocity varies from point to point across the stream, gradually increasing from the walls to the center of the pipe. The mean velocity is obtained by dividing the cross-sectional area of the pipe into a number of concentric equal-area rings and a central circle. The standard 10-point system is shown in figure 254*A*. More divisions may be used if large flow distortions or other unusual flow conditions exist. Observations are made at specific locations in these subareas (fig. 254*A*) and mean velocity is computed from the equation,

$$V_{\text{mean}} = C_1(\sqrt{2g})(\sqrt{\Delta h})_{\text{average}}. \tag{146}$$

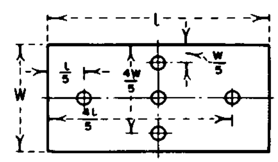
The mean velocity in rectangular ducts can be determined by first dividing the cross section into an even number—at least 16—of equal



(A). Ten-point system for circular conduits



(B). System for rectangular conduits, where at least 16 divisions must be used



(C). Additional points for data in areas around periphery of the rectangular conduit

FIGURE 254.—Locations for pitot-tube measurements in circular and rectangular conduits. (Reproduced from B.S. 1042, Flow Measurement (1943), by permission of the British Standards Institution.)

rectangles geometrically similar to the duct cross section, and then making a pitot-tube observation at the center of each subarea (fig. 254*B*). Additional readings should be taken in the areas along the periphery of the cross section in accordance with the diagram in figure 254*C*. Mean velocity is then computed from equation 146.

When using pitot-static tubes or pitometers, it must be remembered that at low velocities, head differentials are small and errors in reading head differentials will seriously affect the results. Also the openings in the tubes are small and foreign material in the water, such as sediment or trash, can plug the tubes.

MEASUREMENT OF DISCHARGE BY SALT-VELOCITY METHOD

Discharges in conduits flowing full may be determined from the known dimensions of the conduit and velocity observations made by the salt-velocity method. Basically, the method uses the increased conductivity of salt water as a means of timing the travel of a salt solution through a length of conduit. A concentrated solution of sodium chloride is suddenly injected into the conduit at an injection station. At two downstream stations, electrodes are connected to a recording ammeter. An increase in the recorded electric current occurs when the prism of water containing the salt passes the electrodes (fig. 255). The difference in time (t) between the centers of gravity of the recorded salt passage is obtained from the recorder chart as shown in figure 255. The discharge is equal to the volume of the conduit between the two electrodes—it is not necessary that the conduit be uniform—divided by time, t, in seconds.

The brine-injection system that is used is quite complex (figs. 256 and 257). A turbulence-creating device (turbulator) is also sometimes used to insure adequate mixing of the brine and water by the time the upstream electrode station is reached. The required equipment and techniques have been described in detail by Thomas and Dexter (1955).

MEASUREMENT OF DISCHARGE BY THE GIBSON METHOD

The Gibson method was developed for computing the discharge of a conduit or penstock controlled by a valve, turbine, or regulating device located at the downstream end. The pressure conduit must extend at least 25 feet, and preferably much more, upstream from the valve or regulating device, but the conduit need not be of uniform

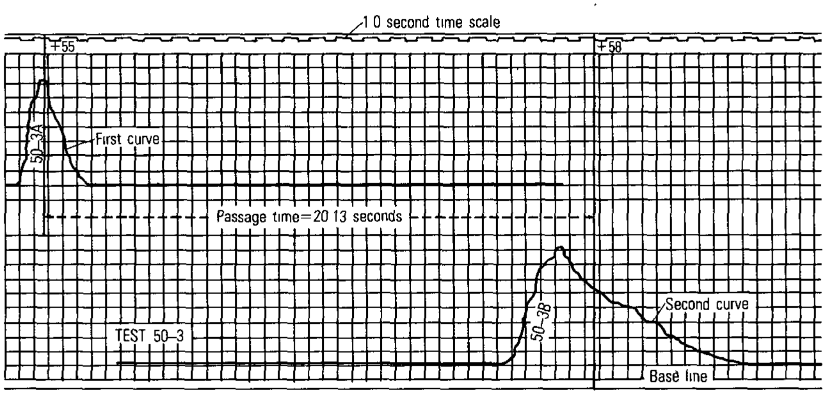


FIGURE 255.—Sample record of a salt cloud passing upstream and downstream electrodes in the salt-velocity method of measuring flows in pipelines. (Courtesy of U.S. Bureau of Reclamation.)

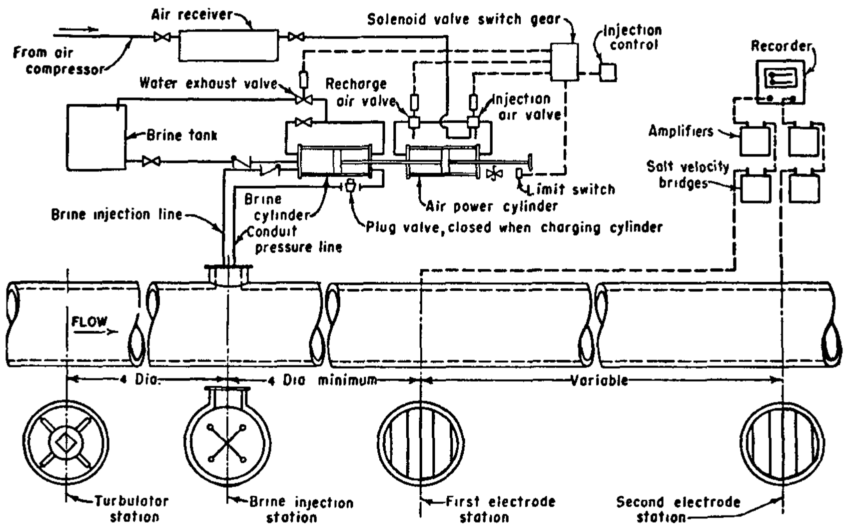


FIGURE 256.—General arrangement of salt-velocity equipment for pressure conduits. (Courtesy of U.S. Bureau of Reclamation.)

cross-sectional area. The underlying principle of the method is that the pressure rise that results from gradually shutting off the flow in a conduit is an indication of the original velocity of the water (Howe, 1950, p. 209–210).

The Gibson apparatus (fig. 258*A*) consists of: a mercury U-tube connected to the penstock just upstream from a gate; a light source behind the U-tube and a pendulum that swings in front of the box; and a narrow slit in the box directly behind the U-tube. Light shines through the U-tube and exposes a film on a rotating drum unless blocked by the pendulum or the mercury in the tube. During a test, the film therefore registers the fluctuation of the mercury column and the time intervals indicated by the pendulum (fig. 258*B*). The period of deceleration, T, terminates when the oscillations become symmetrical (point B, fig. 258*B*, where $t_1 = t_0$). An integration of the area $ABCA$ leads directly to the discharge through application of the equation,

$$Q = \left(\frac{\pi D^2}{4}\right)\left(\frac{g}{L}\right)(\text{area } ABCA), \tag{147}$$

in which Q is the discharge, D and L the diameter and length of conduit, and g the gravitational constant. The lower boundary of the area AC (practically a straight line) must be located by a trial-and-error process which is somewhat time-consuming but which nevertheless gives an accurate location of the line.

Equation 147 is applicable for a conduit of uniform cross section. If

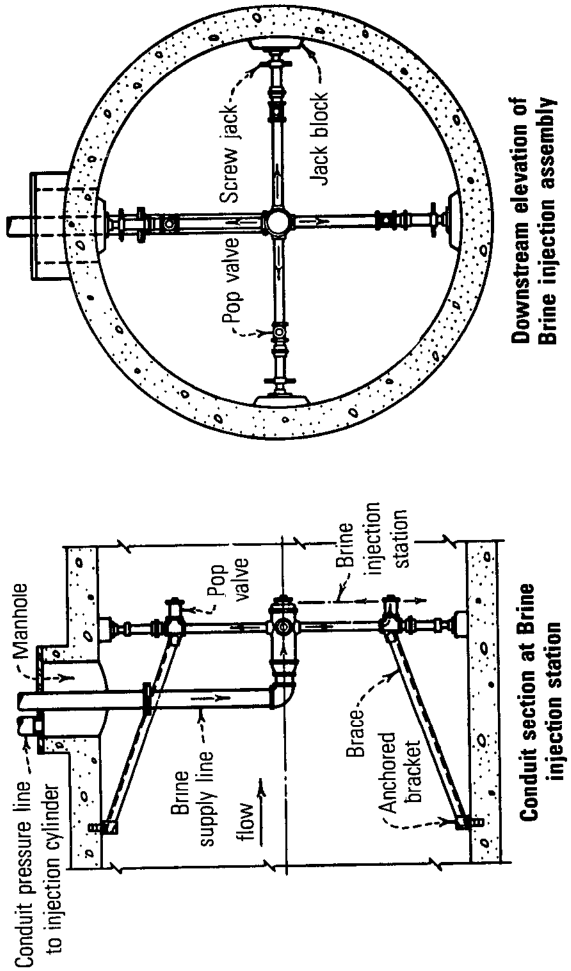


FIGURE 257.—Brine injection equipment in conduit. (Courtesy of U.S. Bureau of Reclamation.)

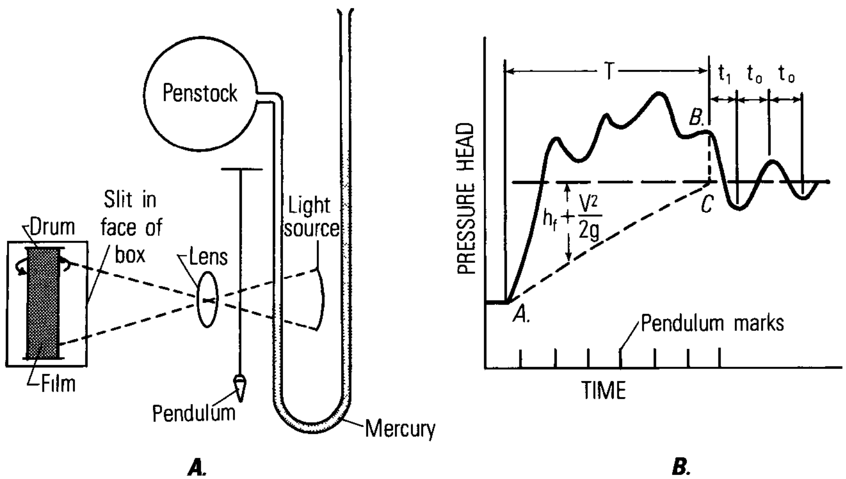


FIGURE 258.—Gibson apparatus and pressure-variation chart. (After Howe, 1950. Reprinted by permission of John Wiley & Sons, Inc.)

the conduit is not of uniform cross section throughout its length but is made up of a series of different sections of length l_1, l_2, l_3, etc. having cross-sectional areas a_1, a_2, a_3, etc., equation 147 must be modified. In that modification, we substitute the value $\Sigma(a/l)$ for the composite term $\left(\frac{\pi D^2}{4}\right)/L$; the value $\Sigma(a/l)$ is the sum of the quotients obtained by dividing the cross-sectional area of each conduit section by its respective length. The modified equation is therefore,

$$Q = g\left[\Sigma(a/l)\right]\left[\text{area } ABCA\right] \tag{147a}$$

It is generally agreed that the Gibson method is very accurate. As an application of the momentum principle, this might be expected. The personnel requirements are not great, since only one operator is required to run the instrument. Neither is cost of the equipment excessive. A series of tests consumes a considerable time, however, because of the necessity for alternately shutting down the flow and bringing it back to a steady rate. Nevertheless, it must be concluded that the Gibson method offers a fairly simple and accurate approach to certain measurement problems that might otherwise be difficult.

CALIBRATION OF TURBINES, PUMPS, GATES, AND VALVES

The calibration of a reaction turbine by the measurement of pressure drop in the scroll case was discussed at the end of the section titled, "Differential-Head Meters." However, in some hydraulic systems it may be desirable, or perhaps necessary, to consider the turbine, pumps, gates, or valves themselves as flowmeters for the system. To do that, it is required that the pertinent hydraulic element be calibrated. The calibration is often done in the laboratory using hydraulic models, but it is preferable that the hydraulic element be calibrated in place, or at least have its laboratory-derived calibration checked by field measurements of discharge. For field calibration, discharge measurements are made by one of the three methods discussed in the section on "Discharge-Measurement Methods for Meter Calibration," if they cannot be made by current meter in the forebay or afterbay of the system where open-channel conditions exist.

In the case of turbines or pumps, relations of discharge versus power are generally desired. They may be defined by observing the metered power output or input during periods when discharge measurements are made for various load conditions. Suitable curves or tables may be developed from these test data to show the discharge (Q) that occurs for specific types of operation. Curves or tables may also be prepared from model test data if the test data can be verified by a few discharge measurements. The calibration will change with time if there is a change in the efficiency of the turbine or pump resulting from long service or from other factors that cause deterioration.

If the range of operating conditions for a pump or turbine is narrow, the calibration is simplified. In such a situation—for example, where power input or output is metered—a simple relation of discharge versus power divided by head may be adequate. For a pump operated by an internal-combustion engine, where power was not metered but rotational speed was automatically recorded, the following calibration scheme has been used. For the most commonly used rotational speed, $(RPM)_r$, a base rating of discharge (Q_r) versus head was defined by current-meter discharge measurements. To obtain the discharge (Q_m) for other rotational speeds, $(RPM)_m$, an empirical adjustment relation of Q_m/Q_r versus $(RPM)_m/(RPM)_r$ was defined by the discharge measurements. (The method of defining the two relations is similar to that used in the constant-fall method of rating open-channel discharges, discussed in the section in chapter 11 titled, "Rating-Fall Constant." The use of head in the pump rating is analogous to the use of stage in the open-channel method; the use of rotational speed of the pump is analogous to the use of fall in the open-channel method.) After the two relations have been defined, to obtain the discharge (Q_m) for a given head and a given rotational speed, $(RPM)_m$, the ratio $(RPM)_m$ to $(RPM)_r$ is first computed. That ratio is then used in the adjustment relation to obtain the ratio Q_m/Q_r. The value Q_r is the discharge corresponding to the given head in the base rating. The desired discharge (Q_m) is then computed by multiplying Q_r by the ratio Q_m/Q_r.

For gates and valves, relations of discharge versus gate opening for various appropriate heads are desired. They may be defined by observing the gate or valve openings during periods when discharge measurements are made for various operating heads. Measurements made over the full range of gate openings and heads will provide the data for establishing the required curves or tables. Generally the relations are in the form of discharge (Q) for gate openings expressed as a percentage of full opening for pertinent operating heads. Curves or tables may also be prepared from model test data if the test data can be verified by a few discharge measurements. As with turbines and pumps, the calibrations for gates and valves are subject to change with time as wear or deterioration occurs.

URBAN STORM DRAINS

Quantitative studies of urban storm runoff have been handicapped by a lack of proper instrumentation for metering the flow in sewers. An ideal sewer flowmeter should have the following characteristics: (1) capability to operate under both open-channel and full-flow conditions, (2) a known accuracy throughout the range of measurement, (3) a minimum disturbance to the flow or reduction in pipe capacity, (4) a minimum requirement of field maintenance, (5) compatibility with real-time remote data-transmission, and (6) reasonable construction and installation costs.

Over the years many devices have been tested for use as sewer flowmeters. Wenzel (1968) has reviewed the methods and devices tested—weirs, depth measurement, depth and point-velocity measurements, dilution methods, and venturi flumes—and found that all have disadvantages of one kind or another. Of those devices, one of the most favorable was the flat-bottom venturi flume specifically designed for flow measurements in conduits by Palmer and Bowlus (1936). That flume has a throat of trapezoidal cross section, a flat bottom, and upstream and downstream side and bottom transitions. The flat bottom permits debris to flow smoothly through the throat and the transitions reduce the head loss substantially below that which would be caused by a weir, for example.

Wenzel (1968), in his study, concluded that further effort in designing some new modifications of a venturi flume offered the greatest promise of success in developing a more satisfactory flowmetering device for urban storm drains. Accordingly three new variations of a venturi section have been designed and laboratory tested in the U.S.A. The U.S.G.S. sewer flowmeter is now (1976) being field tested; the Wenzel asymmetrical and symmetrical sewer flowmeters are still awaiting installation in the field. The three types are briefly described below.

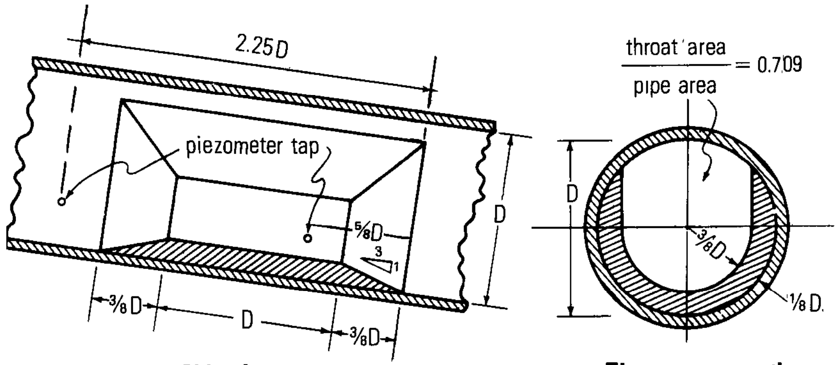


FIGURE 259.—Sketch of USGS flowmeter in a sewer.

U.S.G.S. sewer flowmeter.—The U.S.G.S. meter is a U-shaped constriction made to be inserted in a circular pipe (fig. 259). The symmetry of the design permits fabrication in two half sections for easy transportation and installation. Molds are available for fiberglass prototypes in standard pipe sizes from 24 to 60 in. (0.61 to 1.52 m).

The overall length from toe to heel is 1.75 pipe diameters. The throat length, equal to one pipe diameter, and the approach and getaway apron slopes of 1 on 3, resemble venturi meter specifications. The constriction, in fact, is a venturi flume for open channel flows; for pressure flows it may be considered to be a modified venturi meter.

For subcritical open-channel flows, the constriction dams up the flow, which then passes through critical depth as it spills through the throat. If the oncoming flow is supercritical, two conditions are possible: a hydraulic jump may be forced to form, which then spills through the throat and continues downstream as supercritical flow, or, on steeper slopes, the oncoming flow may remain supercritical throughout the entire constriction. As discharge increases, the water surface on the upstream side rises, touches the top of the pipe, and fills the upstream pipe, while the downstream side continues to flow part full. A discharge rating is available for each of these open-channel conditions.

Further increases in discharge trigger full-pipe conditions, which also are well rated. It is for these pressure-flow conditions that the question of head loss becomes of interest. Head loss, or backwater, is taken to be the increase in the upstream piezometric grade line caused by the presence of the constriction in the sewer line. For this constriction shape, the head loss is expressed as a function of the throat velocity head:

$$H_L = 0.04\frac{V^2_t}{2g}.$$

The constriction is considered to be self-cleaning. Inasmuch as sewers are generally laid to a self-scouring slope, any silting upstream from the constriction is expected to flush out on the next rise. The deposition of silt would have a negligible effect on the rating for small discharges, and no effect for high discharges.

The curved floor in the throat, parallel to the circumference of the pipe rather than being horizontal, retains some self-cleaning ability. It is a compromise between a V-notch base which would have great rating sensitivity for small discharges but a tendency to clog with small debris, and the other extreme, a horizontal floor as in a Palmer-Bowlus trapezoidal constriction. The floor thickness, one eighth of the pipe diameter, provides enough height to produce and maintain a stable hydraulic jump, and it also provides enough constriction (throat area is 0.709 of pipe area) to produce an adequate pressure drop for full-pipe flows. Yet, it is low enough to maintain open-channel flow for a larger range of discharge than would be maintained by a thicker constriction. By leaving the upper part of the pipe unconstricted, a quick transition from open-channel to full-pipe flow conditions is assured, and pressure build-up upstream from the constriction and head loss are minimized.

The pressures in the approach and in the throat of the constriction are measured remotely by pressure transducers. Dry nitrogen gas is bubbled at a constant rate through tubes to the two piezometer openings. The pressure at each opening is reflected to the head of the gas column where the transducer is located.

Data from the flowmeter are entered into the system and converted to two digital numbers proportional to the two pressures measured. The two transducer outputs are applied to a dual analog input amplifier that transforms them to analog voltage levels, which are then applied to analog-to-digital converters. Provisions are made so that one may compress, expand, or shift the range at the analog section.

The format under which data are recorded is dependent upon the conditions indicated by the system data inputs. The system logic inhibits data recordings during dry-weather, no-flow conditions. When flow begins in the sewer to be monitored by the system, the pressure at the approach tap will increase. During the period when this pressure exceeds a preset value, as indicated by the corresponding analog voltage exceeding a programed level, recordings will be continuous on a 1-minute cycle. The recordings are usually on on-site digital-punched paper tape, but variations have provided for analog recording as well as telemetry.

One or more recording precipitation gages and an automatic water sampler are included in the instrumentation for studying urban storm runoff.

It is desirable that the meter be calibrated in place by current-meter discharge measurements. However, as a guide to the probable meter rating and for use until field calibration is completed, the following laboratory discharge equations are presented. The coefficients shown are for use with English units.

A. *Pipe flowing full*

$$Q/D^{5/2} = 5.74\left(\frac{\Delta h}{D}\right)^{0.52}, \tag{148}$$

where

Q is discharge,

D is pipe diameter, and

Δh is the head differential between piezometer readings.

The constant 5.74 includes the constant for the acceleration of gravity. The exponent 0.52 fits the laboratory data better than the theorical exponent 0.5.

B. *Open-channel flow*

1. *Supercritical regime*

$$Q/D^{5/2} = 5.58\ (h_1/D)^{1.58}, \tag{149}$$

where h_1 is the depth above pipe invert at the upstream piezometer.

2. *Subcritical regime—slope of culvert <0.020*

a. *For $h_1/D \geqslant 0.30$*

$$Q/D^{5/2} = 2.85\,(h_1/D - 0.191)^{1.76} \tag{150}$$

b. *For $h_1/D < 0.30$*

$$Q/D^{5/2} = 1.15\,(h_1/D - 0.177)^{1.38} \tag{151}$$

3. *Subcritical regime—slope of culvert $\geqslant$ 0.020*

$$Q/aD^{5/2} = 1.07\ (h_1/D)^{2.71} \tag{152}$$

$$\text{where } a = 2.15 + \left[(9.49)(10)^{11}\,(\text{Slope} - 0.008)^{6.76}\right]. \tag{153}$$

C. *Transitional flow between open-channel flow and full-pipe flow*

$$Q/D^{5/2} = 2.6 \pm\left(\frac{|0.590 - h_2/D|}{0.164}\right)^{1/2}, \tag{154}$$

where h_2 is the depth above the flowmeter invert at the downstream piezometer.

Wenzel asymmetrical and symmetrical flowmeters.—A generalized drawing of the asymmetrical venturi section devised by Wenzel (1975) is shown in figure 260. The symmetrical venturi section differs from the asymmetrical type shown by having identical constrictions on either side of the vertical centerline of the pipe. The constriction

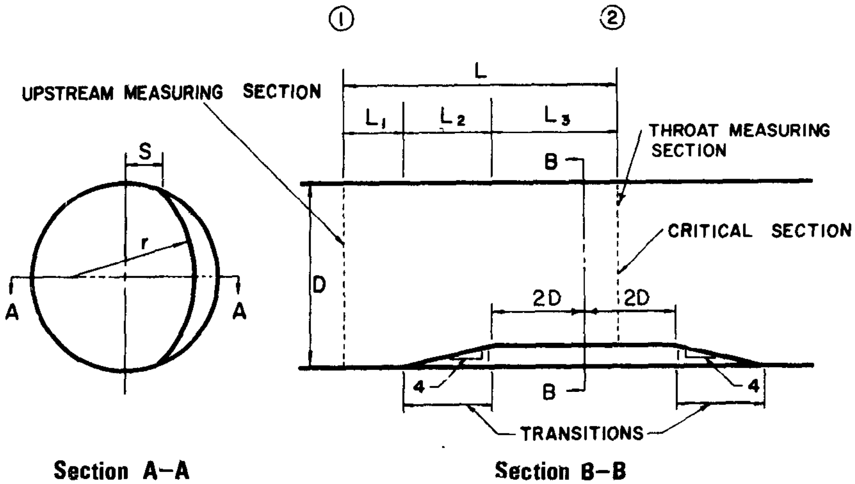


FIGURE 260.—Sketch of Wenzel asymmetrical flowmeter in a sewer. (After Wenzel, 1975.)

consists of a cylindrical section, whose radius is greater than that of the pipe, with entrance and exit transitions having a slope of 1 on 4. The cylindrical section intersects the pipe wall a distance S from the centerline, thereby maintaining the invert region free of obstruction so that self-cleaning is facilitated. In all laboratory tests, a constant value of 0.1 was maintained for S/D, but the ratio r/D was varied to provide various ratios of throat area to pipe area for testing. A throat length between $2.25D$ and $4.0D$ is recommended. The upstream piezometer tap is located approximately $D/3$ upstream from the beginning of the entrance transition; the downstream piezometer tap is located approximately at the center of the throat. As mentioned earlier, no information on the field performance of the Wenzel flowmeters is as yet available.

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CHAPTER 15—COMPUTATION OF DISCHARGE RECORDS

GENERAL

Streamflow records for each gaging station are computed and published annually. The 12-month period used, which is known as the water year, usually does not coincide with the calendar year. In the U.S.A. the water year runs from October 1 to September 30 and is designated by the calendar year of the last 9 months—for example, the 1975 water year runs from October 1, 1974 to September 30, 1975. The following considerations govern the choice of the 12 months that will constitute the water year. The 12-month record is essentially an inventory of the water supply. As with any inventory, it should be made when the stock on hand (available water resource) is at a minimum. That is the case in most of the U.S.A. on September 30, at which time the growing season is at an end. Not only are groundwater, soil-moisture, and surface storage at or near a minimum on that date as a result of heavy water use during the preceding summer, but the replenishing rains of autumn have not yet begun and streamflow is also near minimal. In short, the 12-month period to be used as the water year is determined by the climatic regime of the region.

A daily record of discharge, along with momentary values of peak discharge and minimum flow, is computed for the water year from the record of stage and the discharge rating for the gaging station. The type of stage recorder used determines whether the computations are performed manually or by an electronic computer. In either system, the engineer must study the data and prepare what is termed a station analysis before the actual computation of discharge is begun.

STATION ANALYSIS

A station analysis, which documents the results of the study of the data, is prepared for each station for each water year. The study includes the following items, all of which are needed as a preliminary to computing the discharge record.

1. A review of field surveys of gage datum and a determination of the datum corrections, if any, to be applied to stage observations or recordings during the year.
2. A listing and review of discharge-measurement notes
3. An analysis of the discharge rating and the determination of the rating (or shift) applicable during each period of the year.
4. The preparation of tables that express the discharge rating, using the rating curves derived in the above item 3.

Documentation of items in the station analysis is made as the various steps in the analysis and computation of the discharge record are completed. The station-analysis document is described in detail later in this chapter after all items in the analysis and computation of discharge have been discussed. Examples of the methods of analysis and computation are interspersed in the discussions of methodology for illustrative purposes.

DATUM CORRECTIONS

The datum of the gaging station is the elevation of the zero point of the base or reference gage, preferably referred to mean sea level. (For a discussion of reference and auxiliary gages see the section so-titled in chapter 4. The base gage or reference gage is the gage to which the recording instrument is set; at a nonrecording station it is the gage whose daily readings are recorded by the observer.) Levels are run periodically to all bench marks, reference marks, reference points, and gages at each station for the purpose of determining if any datum changes have occurred as a result of settlement or other movement of any of the gages or of the bubble orifice. If significant movement is indicated by the levels, the gage or bubble orifice is reset to its original datum.

Figure 261 is a typical set of level notes obtained in checking the datum of a recording stage-gage of the float-sensor type; the base gage is a vertical staff gage in the stilling well immediately below a reference point (RP1). Where a vertical staff gage consists of a number of standard USGS porcelain-enameled gage plates, each 3.4 ft long, the elevation of one of the central graduations on each plate should be checked. This is usually done by measuring to each plate with a steel tape whose zero end is held at a reference point of known elevation; the reference point, as mentioned, is established directly above the staff gage. The level notes in figure 261 for the inside staff gage (IG) show that the above procedure was followed.

The level notes are checked in the field for mathematical errors before the field party leaves the gaging station.

If a change in datum has occurred, it is necessary to determine the effective date of the change. In the absence of any evidence indicating the date when the datum change occurred, the change is assumed to have occurred gradually from the time the last levels were run, and the change is prorated with time. On the station-analysis document there would be entered the date(s) when levels were run, the period(s) and magnitude(s) of the datum correction(s) required, and the date and time when the original datum was restored to eliminate the need for corrections. If no datum corrections were required, as indicated, for example, in the level notes of figure 261, that fact would be entered in the station–analysis document.

File No. 10-2450

LEVEL NOTES
(First sheet, to be completed in the field)

Station Clear Creek near Utopia, Calif.

Date Oct 6, 1965 Party Patch ⊼ Sweet ϕ

Date level adjusted: Oct 6, 1965

Object of these levels:

To check gage datum

Conclusions

Elevations of RM's and RP checked within 0 002 ft.
Outside staff-gage datum checked within 0 01 ft.
Inside staff-gage datum checked within 0.003 ft

Changes made:
To inside staff gage, out staff gage, tape gage, or R.M.'s.
No changes required; none made.

Remarks

Data to be added to, or changed on, Station Description (9-197) as a result of these levels.
No changes or additions to be made.

Sheet No. 1 of 2 sheets. S.F. 2-45

9-276
(Rev. 7-67)

UNITED STATES
DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY
WATER RESOURCES DIVISION

Station Number 10-2450

LEVEL NOTES

Stream Clear Creek
Locality near Utopia, Calif
Party Patch ⊼ Sweet ϕ Date Oct 6, 1965

STATION	B S	HT INST	F S.	ELEVATION	REMARKS
RM 1				11.142	Given
RM 2				20.142	Given
RP 1				19.870	Given
RM 2	1.510	21 652		20 142	Bronze tablet, 40' DS
RP 1			1.781	19.871	Head of brass screw
RM 1	10.746	21.889	10.509	11 143	Bronze tablet, 20' DS
RP 1			2 017	19 872	
RM 2			1 746	20.143	
	Levels to outside staff				
RM 1	0 657	11 799		11 142	
OG			10.19	1 61	1.60 ft graduation
OG			3 79	8 01	8 00 ft. "
OG			-3 20	15.00	15 00 ft "
Levels to inside staff - measured by steel tape from RP					
RP 1	0.000	19.870		19 870	
IG			18 268	1 602	1 60 ft. graduation
IG			14 869	5 001	5.00 " "
IG			11.572	8.298	8.30 " "
IG			8.271	11 599	11.00 " "
IG			4.873	14.997	15.00 " "
W.S. in well			18.850	1 020	Gage read 1.02

No. 2 of 2 sheets Comp. by MAP Chk. by GS

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FIGURE 261.—Level notes for check of gage datum.

REVIEW OF DISCHARGE MEASUREMENTS

The first step in the review of discharge measurements is to check the mathematics of the measurements. It is usually considered expedient, however, to accept, without checking, the results of a discharge measurement made by an experienced hydrographer if the measurement checks the rating curve within ±5 percent and if the measured discharge does not exceed all previously measured discharges. The discharge measurements (fig. 42), including indirect determinations of discharge (chap. 9), are then arranged in chronological order and numbered consecutively. The measurements are next compared with the gage-height record to ensure that all discharge measurements are at hand—the inspection notes on the stage record should indicate whether or not a discharge measurement had been made—and also to check the gage heights shown on the measurement sheet. If a datum correction is applicable, it is applied to the mean gage height for the measurement.

The measurements are then tabulated on a special form (USGS form 9-207 in fig. 262). Most of the column headings in figure 262 are self-explanatory. Those on the right half of the table supply information that is helpful to the analyst in appraising the comparative accuracy of the discharge measurements, in case he should find it necessary to give more weight to one measurement than to another in developing the discharge rating. The hydrographer's field appraisal of the probable accuracy of his measurement is shown in the column headed "Meas. rated," where E is excellent, G is good, F is fair, and P is poor. For example, measurements nos. 31, 32, 34–35A, and 44 are rated "poor" because the depths were too shallow or the velocities too low to obtain reliable discharges. In addition, only a few sections (verticals) were used for measurement nos. 32, 34, and 44. The gage-height change during the time required for the measurement is also listed because a rapidly changing stage would adversely affect the adequacy of the measurement. The outside gage reading is listed to provide the analyst with information as to whether or not the gage-well intakes were functioning properly. (Small differences between the readings of the base gage and of the outside auxiliary gage are often the norm because of the difference in location between intakes and outside gage.) The two columns headed "Rating. . . ." are discussed in the section titled, "Rating-Curve Analysis."

The "Remarks" column is most important to the analyst. If a measurement was made by any means other than wading, the method and the sounding weight used are indicated. Measurements made from a bridge or cableway are directly comparable for studying changes in the measurement cross section because the same cross section is used for all discharge measurements. With regard to noting the

FORM 9-207
(Rev. 11-69)

UNITED STATES DEPARTMENT OF THE INTERIOR

GEOLOGICAL SURVEY (WATER RESOURCES DIVISION)
DISCHARGE MEASUREMENT SUMMARY SHEET

Station No 10-2450

Discharge measurements of Clear Creek near Utopia, Calif. during the year ending Sept 30, 1966.

No	Date 1965	Made by—	Width	Area	Mean velocity	Gage height	Discharge	Rating 3: Shift adj	Rating 3: Percent diff	Method	Number meas sections	Gage height change	Time	Meas rated	Time / Water temp °C	Out side gage	REMARKS
			Feet	Sq ft	Fps	Feet	Cfs	Feet				Feet	Hr				
30	Aug. 27	A L Barnes	10	5.76	0.72	2.56	4.18	—	+3	.6	20	0	.7	G	0840 / 14	2.57	Zero flow = 1.95 ± .05 Control clear.
31	Sept. 29	E C King H B Rend.	4.5	1.26	.65	2.18	.82	—	0	.6	16	0	.3	P	1050 / 10	2.18	Zero flow = 1.95 ± .05 Control clear.
32	1965-66 Oct. 9	A. N. Dunn	3.1	.33	.91	2.08 / 2.06	.30	-.02 / 0	0 / 0	.6	6	0	.4	P	1510 / 8	2.07	Removed tree limb from control. *F = 1.95
33	27	E.C.K.	8.7	4.17	.72	2.48	3.00	—	-2	.6	17	0	.5	G	1600 / 4	2.49	Zero flow = 2.00 ± .05 Control clear.
34	Nov. 20	S.K. Steen.	3.2	.35	.60	2.04	.21	—	0	.6	4	0	.2	P	1120 / 2	2.05	No ice in stream.
35	Dec. 16	E.C.K	15	16.1	.13	* 3.12	2.15	Ice -.75	—	.6	18	-.02	.6	P	0930 / 1	3.12	Complete ice cover * OG
35A	1966 Jan. 18	S K S	9.2	10.2	.32	* 3.26	1.76	Ice -.84	—	.6	15	0	.6	P	1430 / 1	3.26	Complete ice cover * OG
36	Feb. 25	R L B H.B.R.	10	6.05	.76	2.68	4.60	Rating 4 —	-1	.6	20	+.01	.5	G	1550 / 1	2.68	Control clear Intakes thawed.
37	Mar. 29	H.B.R.	3	channels		6.60	210	—	-1	.2-.8	26	0	.8	G	1310 / 3	6.62	No ice in stream Control clear.
38	Apr. 25	E.C.K.	52	344	2.38	10.90	820	—	+3	.2-.8	30	+.02	1.1	G	1000 / 4	10.92	Cable - 50 #
39	May 27	H B R.	—	—	—	32.51	9,310	—	0	Slope Area	—	—	—	F	—	* 32.61	* From HWM F = 0.68, n = 0.035
40	June 28	H B R.	31	75.9	1.41	5.26	107	—	+1	.2-.8	24	0	.9	G	1240 / 10	5.26	Control clear
41	July 26	E.C.K	18	20.5	1.00	3.45	20.5	—	-2	.6	21	0	.8	G	1420 / 14	3.46	Gravel bar below gage. Zero flow 2.05 ± .05
42	Aug. 25	A.N.D.	15	17.2	.89	3.22	15.3	—	+3	.6	22	+.01	.7	G	1350 / 16	3.24	Gravel bar moved out. Zero flow 2.00 ± .05
43	Sept. 28	S K S	12	8.76	.82	2.84	7.20	—	+2	.6	18	0	.5	G	0910 / 14	2.84	Control clear Zero flow 2.00 ± .05
44	1966-67 Oct 10	E.C.K	3.6	.65	.65	2.21	.42	—	-5	.6	7	0	.3	P	1740 / 13	2.22	Control clear zero flow 2.05 ± .05

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FIGURE 262.—List of discharge measurements.

sounding-weight size, the measured discharge tends to be greater than the true discharge if too light a weight is used in high-velocity flow because depth soundings tend to be erroneously high and the meter also tends to rise to a higher (and faster) level than intended when positioned at the desired depth for a velocity observation.

The condition of the control—whether clear, ice-covered, or debris-covered—is also noted in the "Remarks" column, along with the gage height of zero flow on the control at the time of low-flow measurements. (Zero flow equals gage-height minus depth of water over the lowest point on the control.) The stability of the rating is dependent on control conditions; the elevation of zero flow is highly important for extrapolating the low-water end of the rating.

In the case of an indirect discharge determination (no. 39), the gage-height of the outside high-water mark is noted in the "Remarks" column, along with the Froude number and roughness coefficient. The equation for computing the Froude number (F) is $F=V/\sqrt{gd}$, where V is mean velocity in the measurement section, g is the acceleration of gravity, and d is mean depth in the measurement section; d is computed by dividing the area of the measurement section by its width. A Froude number close to unity casts some doubt on the indirect determination because it indicates the probability of unstable flow conditions. As for the roughness coefficient, more reliability is generally attached to indirect determinations for smooth channels (low roughness coefficient) than to such measurements for rough channels (high roughness coefficient).

If the gaging station is on an intermittent stream—one that goes dry for periods during the year—the list of discharge measurements should also list chronologically the dates when the hydrographer actually observed that there was no flow in the stream.

STATION RATING—SIMPLE STAGE-DISCHARGE RELATION

The rating curve for a gaging station is a graphical depiction of the relation between stage and discharge. Additional parameters such as fall or velocity index may be required in the rating (see section titled, "Stage Rating—Three-Parameter Discharge Relation"), but this section of the manual deals only with simple stage-discharge relations. Each station rating curve presents individual problems based on the control characteristics for the station, a knowledge of which is a prerequisite for the rating analysis. The principles underlying simple stage-discharge relations were discussed in chapter 10; this section deals only with the mechanics of computing and preparing the station rating.

PLOTTING OF DISCHARGE MEASUREMENTS

Rating curves and discharge measurements should be plotted on logarithmic graph paper, and it is often advantageous to have an additional plot of the low-flow data on rectangular-coordinate graph paper so that the point of zero flow may be plotted. If a new station is being analyzed, the scales selected should be such as to accommodate the ranges of stage and discharge that are expected. If the station is not new, all measurements made since the analysis of the preceding year should be plotted on the prints of the last-used rating curve. Each plotted measurement is tagged with its identifying number, and if the "Remarks" column of the list of measurements indicates that a measurement was made under altered control conditions, that fact should be temporarily indicated alongside the measurement number. Measurements that are affected by ice (nos. 35 and 35A in fig. 262) are not plotted because they serve no purpose in defining the rating. (The use of ice-affected discharge measurements is discussed in the section titled "Rating-Curve Analysis" that follows.) The measurements listed in figure 262 are plotted on the logarithmic rating-curve sheet used during the preceding year (fig. 263). In actual practice, the rating-curve sheet that is used is large enough to accommodate both parts of the plot shown in figure 263. In figure 264, the low-water discharge measurements have been replotted on rectangular-coordinate graph paper that bears a copy of the last-used discharge rating. Logarithmic rating-curve sheets have been designed with a rectangular-coordinate scale in one corner, thereby permitting both logarithmic and rectangular plotting on the same sheet.

RATING-CURVE ANALYSIS

The principles involved in simple stage-discharge relations (chap. 10) are used in analyzing the rating. After reviewing and plotting the discharge measurements, the analyst must determine whether the last-used rating is applicable for part or all of the water year. To do that, he computes percentage departures of his measured discharges from the discharges for the measurement stages, as indicated by the last-used rating table (rating no. 3 on figs. 263 and 264). The percentages are tabulated on the list of discharge measurements (fig. 262). As long as the departures are random in sign (plus and minus) and within ±5 percent, the last-used rating is kept in effect. Aside from the two ice-affected measurements, nos. 35 and 35A, all measurements above a stage of 3.00 ft closely check rating no. 3. Sometime between measurements no. 35A (January 18) and no. 36 (February 25), the ice in Clear Creek went out. When the ice went out, it apparently moved bed material which built up the lower part of the low-water control by about 0.06 ft; the build-up is evident from the change in zero-flow elevation (see "Remarks" column of fig. 262) and from the plotting of the measurements on the low-water curves of figures 263 and 264. Inspection of the gage-height chart indicates that the ice

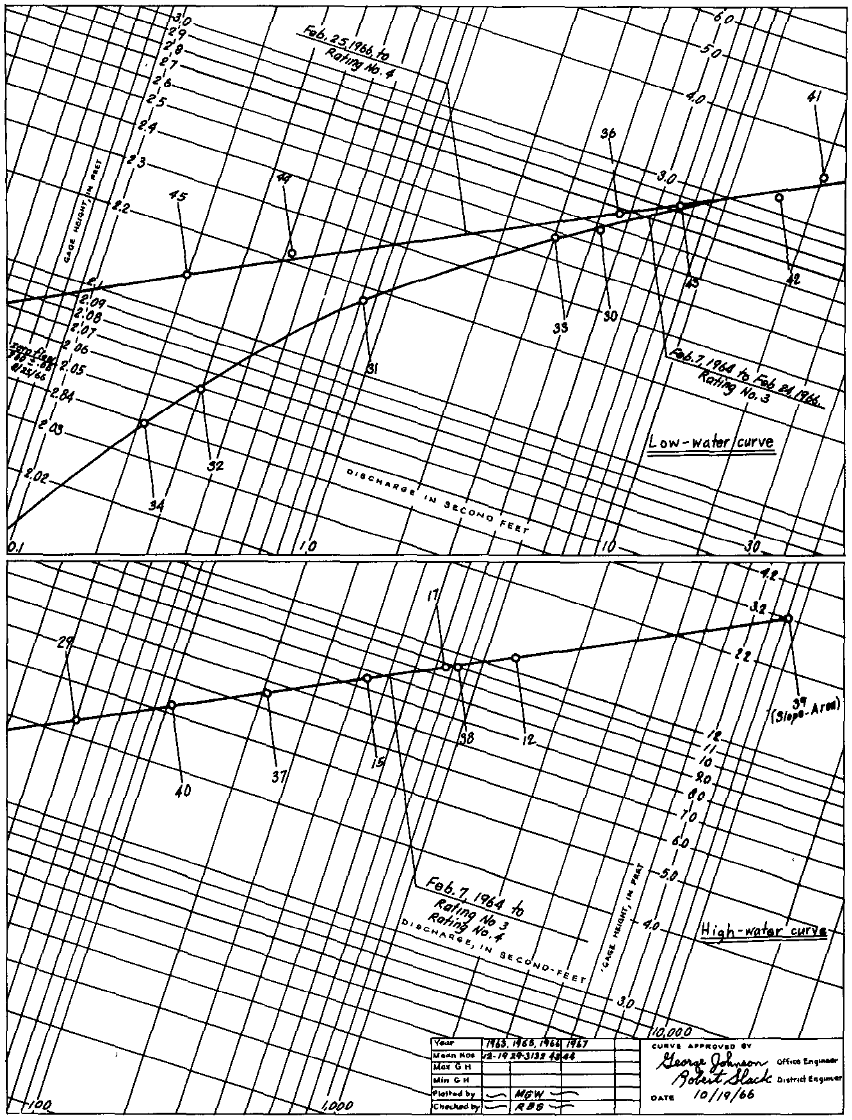


FIGURE 263.—Logarithmic plot of rating curve.

probably went out on a small rise in stage on February 24. Consequently a new rating curve (rating no. 4), based on measurements made after February 24, was developed for use starting February 25. Rating no. 4 is identical with rating no. 3 above a stage of 3.00 ft. One would expect the rating to change as a result of the major peak of May 27 (meas. no. 39), but no such change was evident from subsequent discharge measurements.

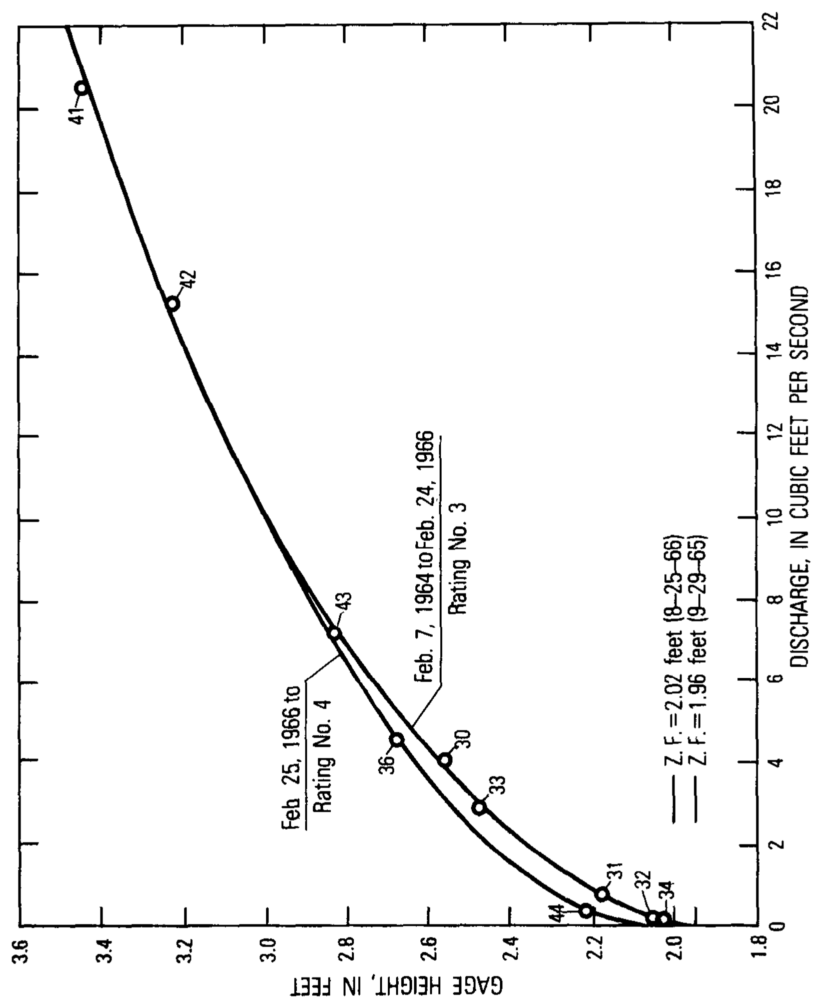


FIGURE 264.—Rectangular plot of low-water rating curve.

When discharge measurements depart from the rating curve by more than 5 percent, but the indicated change in rating is short-lived—less than a month or two—it is common practice not to establish a new rating curve, as such, for the short period. Instead, gage-height shifts (adjustments) are applied either to the rating in use prior to the period of shifting control or to a new rating, if one is later needed, that is established for use starting with the period of shifting control. (Shifts are discussed in detail in the section in chapter 10 titled, "Shifts in the Discharge Rating.") In our example for Clear Creek, aside from the period of ice effect shown by measurements nos. 35 and 35A, only one period of shifting control is in evidence. When the hydrographer visited the station on October 9, he found a heavy tree limb lodged on part of the control. He made his discharge measurement (no. 32) and then removed the limb. That is the proper sequence; had he removed the tree limb before the measurement, his results would be misleading unless he waited long enough for the surcharge storage to drain from the pool so that the stage and discharge became stabilized at the lower stage. That may take an hour or more, but if the measurement is made first, the drop in stage after removing the obstruction can be read later from the stage graph or punched tape. To get back to measurement no. 32, the stage dropped 0.02 ft after removal of the tree limb, and the measured discharge checked the rating curve at the lower gage height. The limb is believed to have lodged on the control on the recession following the minor rise of September 30. Consequently a shift of −0.02 ft is applied to all stages from October 1 to 1300h October 9 when the limb was removed. During that period 0.02 ft is subtracted from all recorded gage heights before obtaining the corresponding discharge from the rating table.

The period of rating shift that occurs as a result of ice effect is not classed as a period of shifting control because discharges are usually not computed by applying shifts to the gage-height record during an ice-affected period. The method of computing discharge for periods of ice effect is discussed in detail in chapter 10.

The basic rating curves to be used during the water year have now been defined and the next step is to transfer the ordinates of the rating curve to a rating table. That is done to refine the rating curve and to provide a more convenient way of obtaining the discharge corresponding to any given stage. The mechanics of preparing the rating table are described in the next section on "Rating Tables." At this point, we will assume that the rating table for rating no. 4 has been prepared, and the next task is to complete the forms that have been used up to now.

The first items to be considered are the two columns headed "Rating . . . ." in figure 262. A heavy line is drawn across the columns between the last measurement (no. 35A) for which rating no. 3 was used and the first measurement (no. 36) for which rating no. 4 was used, and above the latter measurement is inserted the heading "Rating 4." For measurement no. 32, the shift of −0.02 is inserted, as shown by the change in stage when the tree limb was removed from the control. Percentage differences are recomputed for measurements nos. 36–44, using discharges from rating table no. 4 as a base. The originally computed percentage differences for those measurements made at a stage greater than 3.00 ft will remain unchanged because the rating above the stage is unchanged. Shifts are computed and entered for the ice-affected measurements (nos. 35 and 35A), but no percentage differences are computed for ice-affected measurements because as mentioned earlier, shifting-control adjustments, as such, are not applied during the ice-affected periods. The shifts computed for ice-affected discharge measurements, therefore, are not an absolute requirement; they are shown solely for the purpose of giving the rating analyst a quick view of the magnitude of the backwater effect caused by ice. As an example of how shifts are computed, we consider measurement no. 35. The measured discharge of 2.15 ft^3/s corresponds to a gage height of 2.37 ft in rating table no. 3. The observed stage was 3.12 ft. The shift adjustment is −0.75 ft because that is the adjustment that must be applied to the observed stage (2.37 − 3.12) to obtain the stage corresponding to a discharge of 2.15 ft^3/s in rating table no. 3.

On figures 263 and 264 a closing date is added to rating curve no. 3. Rating curve no. 4 is replotted from the refined table for that rating—departures from the original plot of the rating should be very minor—and the new curve is tagged with its identifying number and the date on which it became effective.

To return to generalities about plotting discharge measurements and rating curves, the number of measurements and curves that have accumulated on a rating-curve sheet may in time be sufficient to clutter the sheet to the extent that the data are confusing. In that event a new rating curve should be drawn on a fresh sheet. Old high-water and extreme low-water measurements that are needed as supporting data for the new rating curve are transferred to the new curve sheet.

In the Clear Creek example that has been discussed, there was no need to extrapolate the rating curve. A slope-area determination of discharge had been made at the peak stage to define the high-water end of the curve, and current-meter discharge measurements defined the low-water end of the curve. Had extrapolation been required for either end of the curve, it would have been done by use of the methods discussed in the section in chapter 10 titled, "Extrapolation of Rating Curves."

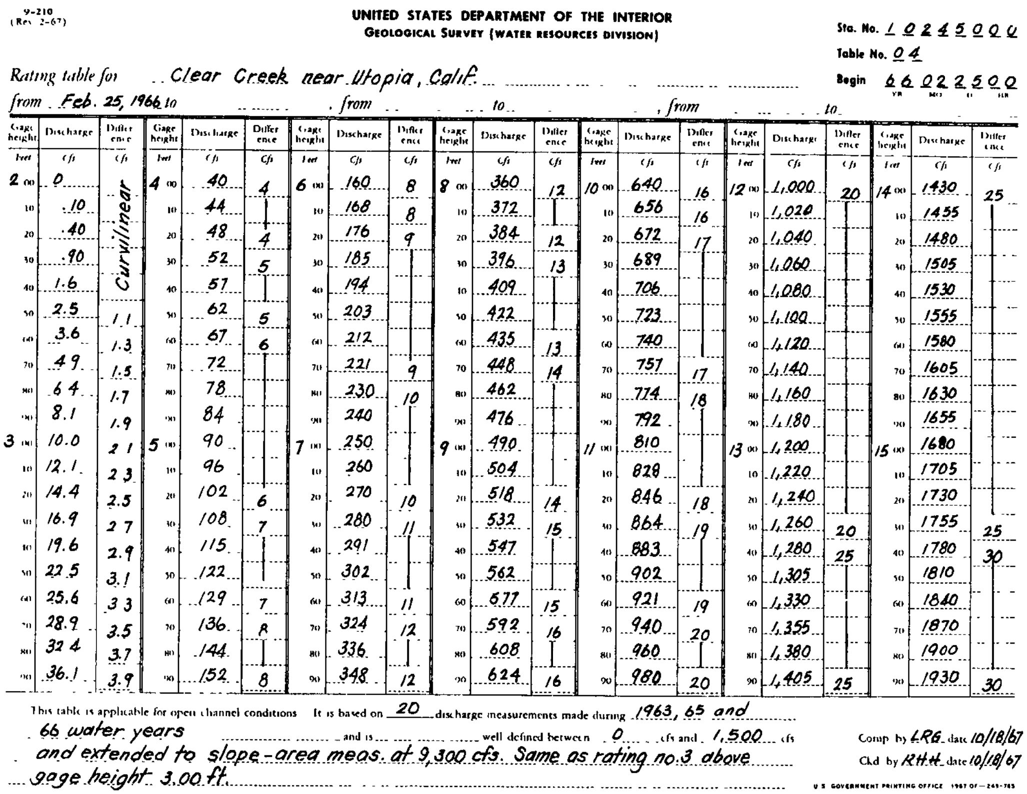
RATING TABLES

The rating table is a tabular expression of the information that is graphically presented by the rating curve. A part of rating table no. 4 for the Clear Creek example is given in figures 265 and 266.

In preparing the rating table from the rating curve, it is important to transfer to the table the identifying number of the rating and its starting date or period of application. Then starting with the low-water curve, the discharge is read and tabulated at intervals of 0.1 ft of stage on the standard rating-table form (fig. 265). On reaching the stage where the rating curve is no longer strongly curvilinear, the discharge may be tabulated at intervals of 0.5 ft of stage, and when the curve becomes more linear, the discharge is tabulated at intervals of 1.0 ft or more. For those parts of the rating that are truly linear on a logarithmic plot, the discharge may be computed from the equation of the rating (chap. 10). The blank spaces in the discharge column of the rating curve are then filled with values that are interpolated between the discharges that were entered in the table.

Differences in discharge for each 0.1 foot of gage height are then computed and entered in the appropriate column of the rating table (fig. 265). The differences should increase uniformly with stage, but this will seldom result from the discharges first entered from the rating curve. It will be necessary to adjust the differences so that they do vary uniformly, which in turn will necessitate a recomputation of the discharge figures, starting with the lowest value whose difference has been adjusted. The adjustment of the rating table must be done judiciously so that the recomputed discharges do not depart significantly from the original rating curve values, particularly in the vicinity of the plotted discharge measurements. Because the rating curve usually has changes in slope, the variation of the difference values can seldom be perfectly uniform. The aim of the smoothing process is to eliminate abrupt changes in the progression of differences, because those abrupt changes would indicate sharp bends in the rating curve. The differences should never decrease with increasing stage unless there is an actual reversal in the shape of the rating curve. Such reversals can only occur where some impeding effect on the discharge (increased backwater) comes into play; for example, where an arch bridge is the high-water control, the increase in waterway area with stage slows and finally ceases at the stages where the archway becomes submerged.

If difficulty is encountered in smoothing the progression of difference values while still adhering to the rating curve, it is helpful to compute second differences, that is, the differences between the dif-



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(Rev. 2-67)

UNITED STATES DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY (WATER RESOURCES DIVISION)

Sta. No. 1 0 2 4 5 0 0 0
Table No. 0 4
Begin 6 6 0 2 2 5 0 0 (YR MO D HR)

Rating table for Clear Creek near Utopia, Calif.
from Feb. 25, 1966 to , from to , from to

Gage height (Feet)	Discharge (Cfs)	Difference (Cfs)	Gage height (Feet)	Discharge (Cfs)	Difference (Cfs)	Gage height (Feet)	Discharge (Cfs)	Difference (Cfs)	Gage height (Feet)	Discharge (Cfs)	Difference (Cfs)	Gage height (Feet)	Discharge (Cfs)	Difference (Cfs)	Gage height (Feet)	Discharge (Cfs)	Difference (Cfs)	Gage height (Feet)	Discharge (Cfs)	Difference (Cfs)
2.00	0	Curvilinear	4.00	40	4	6.00	160	8	8.00	360	12	10.00	640	16	12.00	1,000	20	14.00	1430	25
10	.10		10	44		10	168	8	10	372		10	656	16	10	1,020		10	1455	
20	.40		20	48	4	20	176	9	20	384	12	20	672	17	20	1,040		20	1480	
30	.90		30	52	5	30	185		30	396	13	30	689		30	1,060		30	1505	
40	1.6		40	57		40	194		40	409		40	706		40	1,080		40	1530	
50	2.5	1.1	50	62	5	50	203		50	422		50	723		50	1,100		50	1555	
60	3.6	1.3	60	67	6	60	212		60	435	13	60	740		60	1,120		60	1580	
70	4.9	1.5	70	72		70	221	9	70	448	14	70	757	17	70	1,140		70	1605	
80	6.4	1.7	80	78		80	230	10	80	462		80	774	18	80	1,160		80	1630	
90	8.1	1.9	90	84		90	240		90	476		90	792		90	1,180		90	1655	
3.00	10.0	2.1	5.00	90		7.00	250		9.00	490		11.00	810		13.00	1,200		15.00	1680	
10	12.1	2.3	10	96		10	260		10	504		10	828		10	1,220		10	1705	
20	14.4	2.5	20	102	6	20	270	10	20	518	14	20	846	18	20	1,240		20	1730	
30	16.9	2.7	30	108	7	30	280	11	30	532	15	30	864	19	30	1,260	20	30	1755	25
40	19.6	2.9	40	115		40	291		40	547		40	883		40	1,280	25	40	1780	30
50	22.5	3.1	50	122		50	302		50	562		50	902		50	1,305		50	1810	
60	25.6	3.3	60	129	7	60	313	11	60	577	15	60	921	19	60	1,330		60	1840	
70	28.9	3.5	70	136	8	70	324	12	70	592	16	70	940	20	70	1,355		70	1870	
80	32.4	3.7	80	144		80	336		80	608		80	960		80	1,380		80	1900	
90	36.1	3.9	90	152	8	90	348	12	90	624	16	90	980	20	90	1,405	25	90	1930	30

This table is applicable for open channel conditions. It is based on 20 discharge measurements made during 1963, 65 and 66 water years and is well defined between 0 cfs and 1,500 cfs and extended to slope-area meas. at 9,300 cfs. Same as rating no. 3 above gage height 3.00 ft.

Comp by LRG date 10/18/67
Ckd by RHH date 10/18/67

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FIGURE 265.—Standard rating table.

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UNITED STATES DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY (WATER RESOURCES DIVISION)

Sta No 10245000
Table No 04
Begin 66 02 25 00 (YR MO D HR)

Rating table for Clear Creek near Utopia, Calif.
from Feb. 25, 1966 *to* ... *, from* ... *to* ... *, from* ... *to* ...

Gage height (Feet)	Discharge (Cfs)	Gage height (Feet)	Discharge (Cfs)	Gage height (Feet)	Discharge (Cfs)	Gage height (Feet)	Discharge (Cfs)	Gage height (Feet)	Discharge (Cfs)	Gage height (Feet)	Discharge (Cfs)	Gage height (Feet)	Discharge (Cfs)	Gage height (Feet)	Discharge (Cfs)	Gage height (Feet)	Discharge (Cfs)	Gage height (Feet)	Discharge (Cfs)
2 00	0	2 20	0.40	40	1.60	60		80		00		20		40		60		80	
01	0	21	.44	41	1.68	61		81		01		21		41		61		81	
02	0	22	.48	42	1.76	62		82		02		22		42		62		82	
03	.01	23	.52	43	1.84	63		83		03		23		43		63		83	
04	.02	24	.56	44	1.92	64		84		04		24		44		64		84	
05	.03	25	.60	45	2.00	65		85		05		25		45		65		85	
06	.04	26	.66	46	2.10	66		86		06		26		46		66		86	
07	.05	27	.72	47	2.20	67		87		07		27		47		67		87	
08	.06	28	.78	48	2.30	68		88		08		28		48		68		88	
09	.08	29	.84	49	2.40	69		89		09		29		49		69		89	
2 10	.10	30	.90	50	2.50	70		90		10		30		50		70		90	
11	.12	31	.96	51		71		91		11		31		51		71		91	
12	.14	32	1.02	52		72		92		12		32		52		72		92	
13	.16	33	1.08	53		73		93		13		33		53		73		93	
14	.19	34	1.15	54		74		94		14		34		54		74		94	
15	.22	35	1.22	55		75		95		15		35		55		75		95	
16	.25	36	1.29	56		76		96		16		36		56		76		96	
17	.28	37	1.36	57		77		97		17		37		57		77		97	
18	.32	38	1.44	58		78		98		18		38		58		78		98	
19	.36	39	1.52	59		79		99		19		39		59		79		99	
	.40		1.60																

This table is applicable for open-channel conditions. It is based on 20 discharge measurements made during 1963, 65 and 66 water years and is well defined between 0 cfs and 9,300 cfs

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Ckd by RHH date 10/18/67

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FIGURE 266.—Expanded rating table.

ferences per tenth of a foot of stage. The second differences are then adjusted so that they form a uniform progression; second differences usually change quite slowly. After adjusting the second differences, the first differences are recomputed and finally the discharges are recomputed. As an aid in smoothing the second differences, it is often helpful to plot second differences against stage and then fit a smooth curve to the plotted points. It is highly desirable that a smooth rating table be obtained, but too great an effort to attain the ultimate in smoothness is unwarranted.

To obtain discharges from the rating table for gage heights that are expressed in hundredths of a foot, the discharges are computed by linear interpolation between the values shown for tenths of a foot of stage. Where sharp curvature occurs at the low-water end of the rating curve, such interpolation may be too crude. In that case the discharge for each hundredth of a foot of stage is picked from a large-scale plot of the low-water rating curve, and the discharge values are transferred to an expanded rating table (fig. 266).

Each rating table should be complete within itself for the entire range of stage through which it will be used so that it will not be necessary to refer to some other table that may be identical in part. For example, rating no.4 for Clear Creek is identical with the preceding rating no. 3 at stages above 3.0 feet. Nevertheless, rating no. 4 is completed in figure 265 for all stages above 3.0 ft so that there will be no shuffling back and forth between rating table sheets when applying discharges to recorded stages. By having each rating table complete in itself, the probability of error is reduced. If, as in the case of rating no. 4, the rating is identical with some former rating for some particular range of stage, that fact should be noted at the bottom of the rating table. The blank spaces below the rating table should also be filled to indicate the data on which the rating is based, the range of discharge that has actually been measured by current meter, and the basis of rating-curve extrapolation. As mentioned earlier, the completed rating table is used as the basis for computing the percentage differences for discharge measurements in figure 262, and it is also used to replot the rating curves in final form in figures 263 and 264. As a general rule, no more than three significant figures are used for discharge in the rating table.

STATION RATING—THREE-PARAMETER DISCHARGE RELATION

When a station rating involves three parameters—stage, discharge, and a third parameter such as fall or velocity index—the instructions given in the preceding sections will require some amending. The list of discharge measurements (fig. 262) will require an additional column for the third parameter. The additional column can be provided by reducing the width of the "Remarks" column or by using the column normally reserved for outside gage height.

The general principles concerning the plotting of the discharge measurements and rating curves remain unchanged, but additional curves are required as shown, for example, in figures 190–195. The curves may be plotted on rectangular-coordinate graph paper, as shown in figures 190–195, but logarithmic graph paper may be preferable because then the principles of rating analysis are more easily followed. It may also be advantageous to use more than one sheet of graph paper for the curves to avoid clutter and attendant confusion in working with the graphs.

Because a 3-parameter discharge relation requires more than one relation curve—for example; a rating-fall curve, a fall-ratio curve, and a Q_r rating curve—more than one rating or relation table is required. The general principles discussed on the preceding pages for transferring curve ordinates to a table are applicable for any table.

COMPUTATION OF DISCHARGE RECORDS FOR A NONRECORDING GAGING STATION

The computation of discharge records for a nonrecording gaging station is identical with that for a recording station equipped with a graphic recorder, except for the early steps in computing the gage-height record. Consequently only those early steps will be discussed in this section of the manual. The remaining steps in the computation of the discharge record are discussed on those pages of this chapter that deal with stations equipped with graphic stage recorders.

COMPUTATION OF GAGE-HEIGHT RECORD

The first step in computing the record for a nonrecording gage is to compare the readings on the weekly gage cards mailed in by the observer with those he has entered in his quarterly book of gage height observations. (See introductory pages of the section in chapter 4 titled, "Nonrecording Stream-Gaging Stations.") The observer's readings should also be compared with readings made by the hydrographer on his regular visits. After reconciling any differences, the next step is to apply datum corrections, if any, to the observed gage heights. Both the corrections applied and the corrected gage-height values are entered in the book of gage observations (fig. 8). The corrected gage-heights are plotted at the appropriate time ordinates on fragments of unused recorder chart that are excess when a new roll of recorder paper is installed in a graphic stage-recorder. It is not necessary to plot gage heights for the long periods of gradually receding flows that follow stream rises. For the days during such periods, the daily mean gage heights are computed as the mean of the two observed readings for each day.

A stage hydrograph is sketched through the plotted gage heights, using the graphic stage record from a nearby recording gaging station as a guide to the probable shape of the stage hydrograph. Observed high-water marks, for each of which the gage-height has been determined, and crest-stage gage readings are used where available, to give the peak stage of major rises. (Crest-stage gages are discussed in the last section of chapter 4.) The result is a stage hydrograph which, from the standpoint of discharge-computation methodology, is equivalent to the stage record from a graphic recorder after the recorder chart has had time and gage-height corrections applied to it.

Consequently, the remaining steps in computing the discharge record are, in effect, continued on the pages that follow the discussion of time and gage-height corrections for graphic-recorder charts. As mentioned above, from that point on the computation procedures are identical for nonrecording and graphic stage-recorder stations.

COMPUTATION OF DISCHARGE RECORDS FOR A RECORDING STATION EQUIPPED WITH A GRAPHIC RECORDER

COMPUTATION OF GAGE-HEIGHT RECORD

At a station visit when the recorded segment of the gage-height chart is removed and a fresh segment of chart is started, the hydrographer makes note of all information that will be needed in computing daily gage heights. His notations are made both on the end of the recorded chart and on the beginning of the fresh segment of chart. Those notations include name of the station, date, readings on all gages and the time of those readings, the instrument stage ratio, and notes explaining any unusual appearance of the pen trace. In addition to making a pen "tick" at the point where the pen rests at the time of chart removal and again at the time the fresh segment of chart is started, the hydrographer also rotates the float wheel to indicate the pen-reversal points on the chart. If the float wheel of the recorder is equipped with a tape, the step method of checking pen reversal is used. (See fig. 267.) The step method is used in making gage-height corrections to the pen trace and is explained in the section on "Determination of Gage-Height Corrections."

DETERMINATION OF TIME CORRECTIONS

Before determining the time corrections to be applied to the gage-height record, the chart should be dated. Each day is numbered on

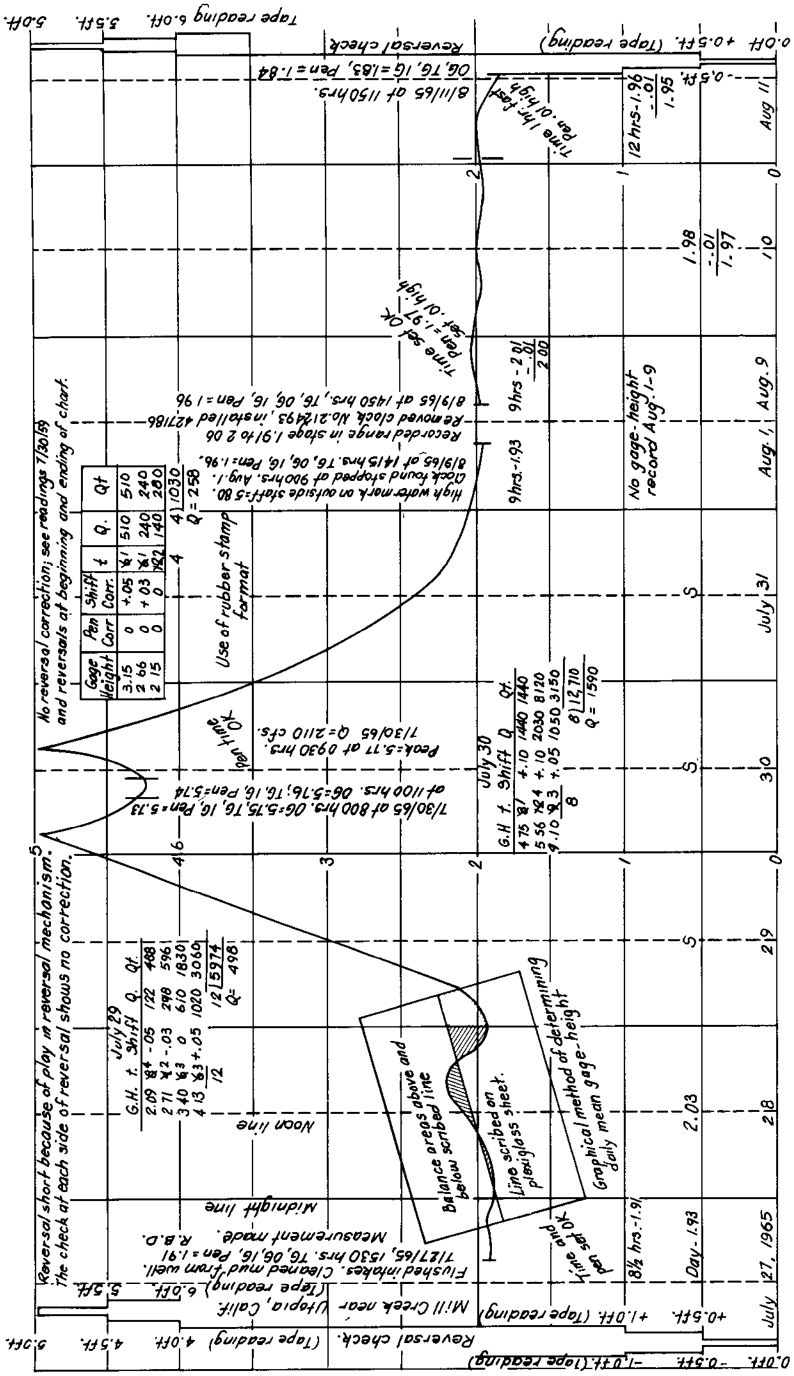


FIGURE 267.—Computation of daily mean gage height on graphic-recorder chart.

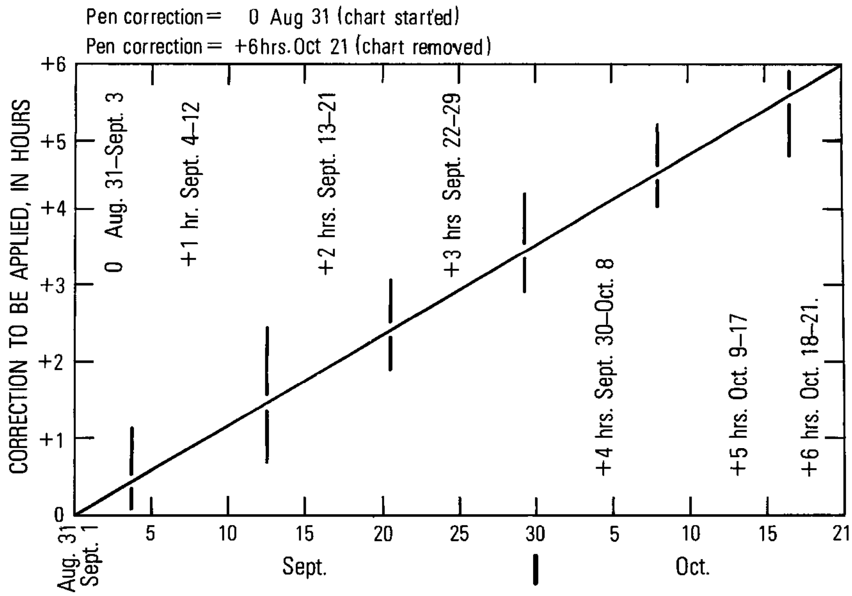


FIGURE 268.—Example of graphical interpolation to determine time corrections.

the lower base line at the noon line. The month is shown about every fifth day, and the year is shown about once a month.

The first step in computing time corrections for a segment of chart is to list the time corrections needed at each of the two or more days when the chart was field inspected. If the time correction at the end of the chart is large, the record should be inspected for evidence of large abrupt timing error—for example, clocks have been known to stop and then restart some hours later. If no abrupt timing errors are found, the time corrections are prorated by straight-line interpolation in which corrections are determined to the nearest hour. Figure 268 is an example of such an interpolation. The graph in figure 268, which is self-explanatory, would normally be drawn on the recorder chart near the beginning of the chart segment being studied. If the total time correction for the chart segment is small, the interpolated distribution of time corrections may be computed arithmetically without the use of a graph.

The computed time corrections are applied by changing the positions of the midnight lines for the affected days. Heavy vertical lines are drawn to indicate the new midnight lines, using care to ensure that the time adjustments are applied in the correct direction. It is advisable to make all interpretive notes, figures, and time corrections in colored pencil on the gage-height chart to differentiate them from the original notes.

DETERMINATION OF GAGE-HEIGHT CORRECTIONS

Gage-height corrections to the recorder trace are next determined. They are based on differences in readings of the recorder pen and the base gage, usually the inside staff, at station inspections. These corrections are also prorated with time unless there is evidence of abrupt instrumental error, such as would occur as a result of float-wheel slippage, or unless a systematic error with stage is shown to exist when the reversal points are checked by the step method at station inspections. An error in setting the pen at the start of a segment of strip chart will be carried throughout the length of that segment, but the original error may be increased or decreased by the above-mentioned errors. Gage-height corrections should be noted on the chart in such a manner that they can be easily applied to the gage-height values that are determined later.

Reversal errors, that is, errors that occur when the pen reverses direction at or near the upper or lower base lines, and systematic errors that vary with stage are usually caused by expansion or contraction of the chart, but they may also be caused by skewed travel of the chart. Reversal errors may also result from wear or maladjustment of the reversal mechanism of the stage recorder.

The step method of checking reversal points when changing the chart in the field provides a means of determining the gage-height corrections that vary with stage. The method requires that the recorder float wheel be equipped with a tape. The procedure used by the hydrographer is as follows:

1. Before removing the chart, raise the float tape to a value that is exactly 1 foot less than the foot mark at which the pen reverses; pull the chart forward a short distance to put an identifying "step" on the chart at that stage (fig. 267). Enter the tape reading on the chart.
2. Raise the float tape an additional half-foot and repeat the procedure.
3. Raise the float tape to the reversal point and repeat the procedure.
4. Repeat the above procedure, first with the tape reading 0.5 ft more than the foot mark at which the pen reverses, and again with the tape reading 1.00 ft more than the reversal foot mark.
5. Continue to raise the float tape and repeat steps 1 to 4 for the other base line reversal.
6. After the recorded segment of chart has been removed and the fresh segment of chart has been engaged, the pen is set to the correct gage height and steps 1 to 5 are repeated.

An example of the step method of checking reversal points is shown in figure 267. The step method in figure 267 actually indicates the need for a correction of +0.01 foot at a recorded stage of 4.99 ft and a correction of −0.01 foot at a recorded stage of 5.01 ft. In other words a true stage of 5.00 ft is recorded as 4.99 ft on one side of the reversal and 5.01 ft on the other side. However, the gage inspections at 5.73 ft and 5.74 ft indicate that no corrections are needed and none were applied.

As a final step, datum corrections (see section on "Datum Corrections"), if required, are noted for each affected day. The recorder chart is now ready for the determination of daily gage heights.

DETERMINATION OF DAILY MEAN GAGE HEIGHT

Daily mean gage heights are usually determined graphically by the use of a thin rectangular piece of clear plastic whose dimensions are approximately 2 by 4 inches; a centerline is scribed on the plastic parallel to the long edge. The plastic is placed over a 24-hour segment of the recorder chart with the scribed line approximately over the pen trace. The plastic is then maneuvered into a position where the areas bounded by the midnight lines and lying above the scribed line but below the pen trace are equal in size to the areas lying below the scribed line but above the pen trace. When the areas above and below the scribed line are so balanced, the gage height of the point at which the scribed line intersects the noon line is the uncorrected mean gage height for the day. An example of the graphical method of determining daily mean gage height is shown for July 28 in figure 267.

A gage-height correction and (or) a datum correction, if applicable, will have been entered on the chart at about the noon line and about 1½ inches above the base line. The uncorrected daily mean gage height determined by the graphical method is then entered above the correction(s), the required addition or subtraction is performed to obtain the corrected daily mean gage height, and the corrected value is written below the correction as shown for August 10 in figure 267.

SUBDIVISION OF DAILY GAGE HEIGHTS

When there is large variation in stage during the day, it is necessary to: subdivide the day into smaller increments of time, determine the mean gage height for each time increment, apply the corresponding discharge from the rating table to each incremental mean gage height, and compute a time-weighted mean discharge for the day. That procedure is necessary because the stage-discharge relation is curvilinear; consequently the discharge corresponding to the mean gage height for a segment of stage of large range will differ significantly from the true discharge, which is the discharge integrated over that range of stage. The allowable range of stage, for which the use of a mean gage height introduces no significant error in discharge, depends on the curvature of the stage-discharge relation; the more nearly linear the rating is, the larger the allowable range in stage.

The rule generally followed in the U.S.A. is to subdivide the gage-height graph for the day if the discharge corresponding to the daily mean gage height differs by 4 percent or more from the average of the two discharges corresponding to the maximum and minimum gage heights in the day. For any normal rating table, the average of the two discharges will be the larger figure. A simple method of computing a table of allowable range of stage for a rating is outlined below, using the rating table in figures 265 and 266 as an example.

First, a gage height G is selected near the lower end of the rating. Because the allowable difference in discharge is 4 percent, the average of the two extreme discharges in the allowable range of stage is 1.04 Q_G where Q_G is the discharge from the rating table corresponding to gage height G. That means that 2.08 Q_G equals the sum of the two extreme discharges in the allowable range of stage. (A definition sketch is given in fig. 269.) The analyst using the rating table moves small equal distances in stage up and down from gage height G until he obtains a pair of stages whose discharges total 2.08 Q_G. The range,

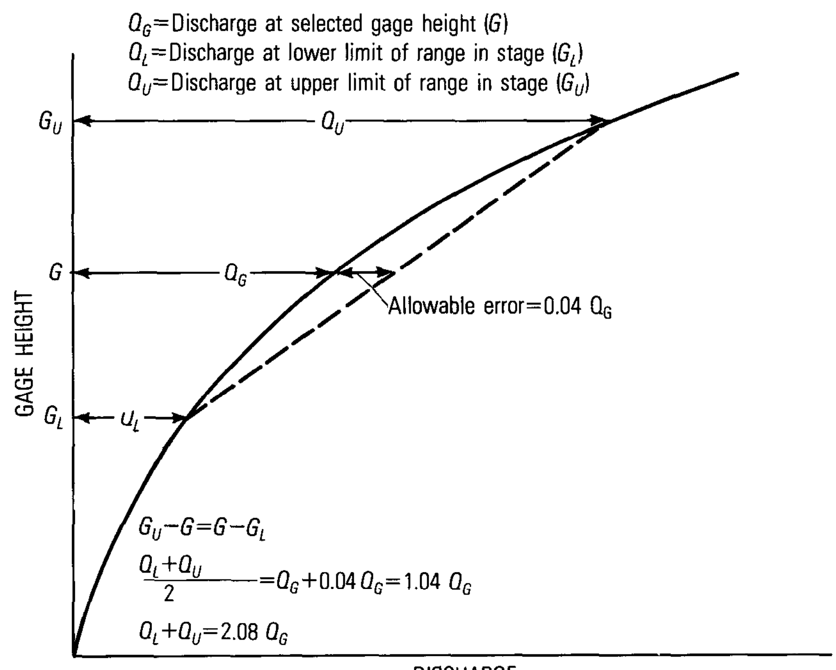


FIGURE 269.—Definition sketch illustrating computation of stage limits for application of discharge.

Mean gage height (ft)	Q (ft³/s)	Q x 2.08 (ft³/s)	Allowable limits of stage (ft)	Allowable range of stage (ft)
			Corresponding discharge (ft³/s)	
2.2	0 4	0.83	2.15–2.25 (0.22+0.60=0.82)	0.10
2.4	1.6	3.30	2.32–2.48 (1.02+2.30=3.32)	.16
2.8	6.4	13 3	2.65–2.95 (4 25+9.05=13.3)	.30
3.4	19.6	40.8	3.1–3.7 (12.1+28.9=41.0)	.6
4.0	40	83.2	3.6–4.4 (25.6+57.0=82.6)	.8
4.8	78	162	4.2–5.4 (48+115=163)	1.2
6.0	160	333	5.2–6.8 (122+203=325)	1.6
7.5	302	628	6.4–8.6 (194+435=629)	2.2
9.0	490	1020	7.6–10.4 (313+706=1019)	2.8

FIGURE 270.—Results of computation of allowable limits of stage for Rating No. 4, Clear Creek near Utopia, Calif.

in feet, between the pair of stages is the allowable range in stage for a mean gage height of G. The procedure just described is then used to obtain the allowable range in stage for other values of gage height. The results of such computations for the rating table in figures 265 and 266 are shown in figure 270. The information given by the table in figure 270 is reorganized to provide the table of allowable rise shown in figure 271, which is more convenient for use in subdividing days. For days that are subdivided it is not necessary to compute the daily mean gage height.

The table of allowable ranges for subdivision may require some revision for periods when shifting-control adjustments are used.

Gage height (ft)	Allowable rise (ft)	Gage height (ft)	Allowable rise (ft)
2.15	0.10	4.2	1.2
2 32	.16	5.2	1.6
2.65	.3	6.4	2.2
3.1	.6	7.6	2.8
3.6	.8		

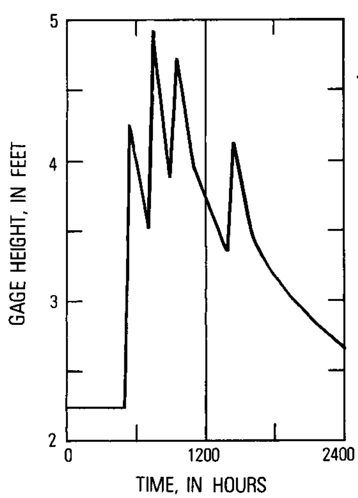
FIGURE 271.—Table of allowable rise for use with Rating No. 4, Clear Creek near Utopia, Calif.

However, revision will usually be necessary only in the unusual situation where the shifts to be applied are so extreme that they radically change the shape of the stage-discharge relation.

Either of two methods are used for computing discharge for subdivided days, and the procedure for subdividing the day varies with the method used. The first method is the increment-mean method. In that method the mean gage height is determined for each increment of the day by using the graphical process of balancing areas that was described earlier. Shifts, if appropriate, are applied to the mean gage heights, corresponding values of incremental mean discharge are obtained from the rating table, and a time-weighted daily mean discharge is computed. The time-weighting is done by first multiplying each incremental mean discharge by the number of hours in the increment, then adding the products, and finally dividing the sum of the products by 24 (number of hours in the day). The arithmetic is simplified if the increments of the day are all multiples of either 2, 3, 4, 6, 8, or 12 hours, because then the numerical values of the hours used can be reduced by factoring. For example, if the day had been subdivided into three increments of 6, 6, and 12 hours, those time periods could be expressed as multiples of 6. For weighting purposes, the hour values would be factored to give 1, 1, and 2, and the sum of the products would be divided by 4 rather than 24. (See subdivision for July 31 in fig. 267.)

The procedure for subdividing a day by the increment-mean method is as follows. The analyst starts at the lowest point of the pen trace and moves upward as far as the table of allowable rises will permit. That upper value of stage then becomes the starting point for the next increment of the day, whose upper limit is also determined from the table of allowable rises. The process is continued until the entire day has been subdivided. The ends of the time increments are adjusted to coincide with the nearest hour lines, but the adjustment should, if anything, decrease the range in stage for an increment from that indicated by the table of allowable rises. If feasible, the time increments are further adjusted to permit the factoring discussed in the preceding paragraph.

The second method of computing discharge for subdivided days is the point-intercept method. In that method, gage heights are noted along with the clock hour of occurrence, at the beginning of the day, the end of the day, and at all "breaks" in slope of the stage hydrograph during the 24 hours. It is important, however, not to permit the difference in stage between consecutive recorded gage heights to exceed values given by the table of allowable rises. If the stage difference for a time increment does exceed the allowable rise, one or more additional intermediate points on the hydrograph must be selected



JULY 1

(1) Time	(2) Corrected gate height	(3) Shift	(4) Time increase (hrs)	(5) Discharge (ft³/s)
0000	2 25	0	2½	24
0500	2.22	0	2¾	22
0530	4 28	0	½	464
0600	4.04	0	¾	392
0700	3.50	0	¾	250
0730	4.94	0	½	716
0800	4 55	0	¾	560
0900	3 87	0	¾	342
0930	4 77	0	½	648
1000	4.49	0	¾	536
1100	3.97	0	2	371
1400	3 34	0	1¾	213
1430	4.16	0	½	428
1500	3 88	0	¾	345
1600	3.48	0	2½	245
2000	2 94	0	4	133
2400	2 66	0	2	79
			24)	*5484 5
				229.0=Mean Q

*Summation of individual products of column 4 times column 5

FIGURE 272.—Sample computation of daily mean discharge for a subdivided day by point-intercept method.

for use. The end result is a tabulation such as that shown in the example in figure 272 where the gage heights are tabulated at the nonuniform hours associated with breaks in slope of the stage hydrograph.

Computation of the daily mean discharge by the point-intercept method is similar to that for the increment-mean method except for the manner of determining the number of hours (col. 4 of fig. 272) associated with each tabulated gage height. Each of the gage heights is assumed to represent the mean gage height for a time interval that extends from (a) the clock time midway to the preceding tabulated gage height to (b) the clock time midway to the following tabulated gage height. The discharges in column 5 of figure 272 correspond to the tabulated gage heights in column 2 after those gage heights have been adjusted for the shifts, if any, shown in column 3. The time-weighting of the discharge is then done by first multiplying each discharge (col. 5) by the correspnding number of incremental hours (col. 4). The individual products, which are not shown in figure 272, are then added, and finally the sum of the products is divided by 24 (number of hours in the day).

The advantage of the point-intercept method over the increment-mean method of computing daily mean discharge for subdivided days lies in the fact that the point-intercept method provides the data for reproducing the stage or discharge hydrograph for storm runoff. Consequently, the point-intercept method is always used in flood reports. Because daily mean discharges computed by the two methods will seldom agree exactly, it is best to use the point-intercept method, at least for major runoff events. Then if the major runoff event is made the subject of a later flood report, daily mean discharge in the flood report and in the routine annual streamflow report will agree. For complex flood events, such as that shown in figure 272, the point-intercept method will usually give somewhat more accurate daily mean discharges than will the increment-mean method, but only because more gage heights per day are usually used in the point-intercept method for such events. Subdivision is really a crude form of mathematical integration of the hydrograph. Mathematical integration gives the only truly accurate value of mean discharge, and the more points that are used in the subdivision, the more closely the subdivision will resemble integration. The difference in results between mathematical integration and subdivision rapidly dwindles to insignificance when sufficient points are used in the subdivision. Mechanical integrators, now largely superseded by digital recording and computation, are available to compute daily mean discharge for stations having large and frequent stage fluctuations, such as those that occur downstream from hydroelectric power plants.

COMPUTATION OF DAILY DISCHARGE

PREPARATION OF FORM FOR COMPUTING AND TABULATING DISCHARGE

The first step in the computation of daily discharge for a nonrecording station or a recording station equipped with a graphic recorder is to prepare a form, such as USGS form 9–192a which is shown in figure 273, to receive the computed values. The form in figure 273 provides columns for daily mean gage height and discharge for the 12 months in the water year, as well as spaces for monthly and annual summaries which will be discussed in the section on "Completion of the Discharge Form." The analyst fills in the blanks at the top of the form that supply general information such as name of station, drainage area, type of recorder, water-year date, numbers of the rating tables used, and so on. It is important that the form be prepared carefully because the data are copied from this form on to offset sheets used for publication of the data. In addition, prints of the form are often furnished to water users as preliminary data in advance of the published data.

Daily mean gage heights from the original water-stage recorder chart are copied in the columns headed "Gage height." In addition, the maximum and minimum gage heights that occurred during the year are listed in the spaces provided at the left margin. For those days that are subdivided for the computation of daily discharge, no

9-188-a. July 1937

Daily Gage Height, in Feet, and Discharge, in Second-Feet, of Clear **Creek**
Near Utopia, Calif. **for the Year Ending September 30, 19**66.
Drainage Area 160 **Square Miles. Water-Stage Recorder** Stevens A-35 **Ratio**

UNITED STATES DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY
WATER RESOURCES DIVISION

Clear Cr nr. Utopia, Calif.
1965-66

File Number Washington 10-2450 District

Used rating table Nos. 3, 4

Gage Read to ... **Once / Twice a Day by** ...

Gage heights used to half tenths between ... and ... feet hundredths below and tenths above these limits.

Day	October Gage height	October Discharge	November Gage height	November Discharge	December Gage height	December Discharge	January Gage height	January Discharge	February Gage height	February Discharge	March Gage height	March Discharge
1	2.35	1.8	2.56	3.8	2.53	3.3		2.0		2.0	2.80	6.4
2	2.36	1.9	2.54	3.7	2.55	3.6		2.0		2.1	2.78	6.1
3	2.36	1.9	2.55	3.8	2.54	3.4		1.8		2.2	2.81	6.6
4	2.38	2.1	2.56	4.0	2.56	3.6		1.0		2.3	2.84	7.1
5	2.37	2.0	2.54	3.7	2.58	3.8		1.6		2.4	2.82	6.7
6	2.39	2.1	2.53	3.6	2.95	3.6		2.2		2.5	2.86	7.4
7	2.40	2.2	2.55	3.8	3.10	3.4		2.4		2.6	2.89	7.9
8	2.41	2.3	2.54	3.7	3.18	3.2		2.4		2.7	2.91	8.3
9	2.42-13 2.40-11	2.4	2.57	4.1	3.16	2.6		2.2		2.8	2.94	8.9
10	2.41	2.5	2.55	3.8	3.17	2.5		2.2		2.9	2.95	9.0
11	2.40	2.4	2.56	4.0	3.15	2.4		2.2		3.1	2.97	9.4
12	2.42	2.6	2.58	4.2	3.19	2.5		2.1		3.2	2.99	9.8
13	2.45	2.9	2.59	4.3	3.21	2.6		2.0		2.8	3.00	10
14	2.43	2.6	2.62	4.6	3.18	2.5		2.0		2.9	3.03	11
15	2.46	2.9	2.60	4.5	3.20	2.4		2.0		3.0	3.05	11
16	2.49	3.2	2.45	2.9	3.15	2.2		1.9		3.1	3.08	12
17	2.48	3.1	2.30	1.6	3.18	2.3		1.8		3.2	3.16	14
18	2.51	3.4	2.20	.94	3.16	2.2		1.8		3.3	3.27	16
19	2.49	3.2	2.12	.54	3.17	2.1		1.8		3.4	3.40	20
20	2.48	3.1	2.05	.25	3.20	2.1		1.9		3.5	3.52	23
21	2.52	3.5	2.12	.54	3.22	2.0		2.0		3.5	3.61	26
22	2.53	3.6	2.21	1.0	3.26	2.1		2.0		3.5	3.83	34
23	2.51	3.3	2.30	1.6	3.30	2.0		2.0		3.7	4.15	46
24	2.54	3.6	2.40	2.4	3.40	2.0		2.2		15.	4.40	57
25	2.57	4.0	2.45	2.9	3.45	2.0		2.2	2.70	4.9	4.77	76
26	2.56	3.8	2.48	3.2		2.0		2.2	2.68	4.6	5.02	91
27	2.57	4.0	2.62	3.0		2.0		2.1	2.71	5.0	5.41	116
28	2.55	3.7	2.58	2.8		2.0		2.0	2.75	5.6	5.92	156
29	2.55	3.7	2.66	2.9		2.0		2.0	—	—	6.65	216
30	2.55	3.7	2.62	3.1		2.0		2.0			6.70	221
31	2.53	3.5				2.0		2.0			6.68	219
Total		91.0		89.27		78.4		62.0		101.8		1,466.6
Mean		2.94		2.98		2.53		2.00		3.51		47.3
Run-off in acre-feet		198		198		156		123		202		2,910
Maximum		4.0		4.6		3.8		2.4		15		221
Minimum		1.8		.25		2.0		1.0		2.0		6.1

Day	April Gage height	April Discharge	May Gage height	May Discharge	June Gage height	June Discharge	July Gage height	July Discharge	August Gage height	August Discharge	September Gage height	September Discharge
1	6.65	216	9.41	548	10.02	643	5.10	96	3.39	16	3.09	13
2	6.70	221	9.28	529	9.96	634	5.03	92	3.36	15		13
3	6.73	224	9.13	508	9.91	626	4.96	88	3.37	16		13
4	6.77	227	8.97	486	9.79	606	4.88	83	3.35	15		12
5	6.81	231	8.84	468	9.66	586	4.81	79	3.33	15		12
6	6.74	225	8.79	461	9.51	564	4.72	73	3.50	20		11
7	6.69	220	8.65	442	9.30	532	4.66	70	S	287		11
8	6.62	214	8.51	423	9.08	501	4.57	66	S	154		10
9	6.58	210	8.30	396	8.84	468	4.51	62	S	49		10
10	6.55	208	8.16	379	8.61	436	4.43	58	3.61	28		10
11	6.50	203	8.01	361	8.39	408	4.35	54	3.42	22	2.98	10
12	6.46	199	7.95	354	8.16	379	4.22	49	3.37	21	2.96	10
13	6.51	204	7.93	352	7.94	353	4.05	42	3.34	20	2.94	9.6
14	6.56	208	7.98	358	7.71	325	3.96	38	3.31	19	2.96	9.8
15	6.60	212	8.04	365	7.53	305	S	243	3.27	18	2.97	10
16	6.64	216	8.10	372	7.35	286	S	112	3.25	17	2.95	9.6
17	6.67	218	8.17	380	7.16	266	4.02	38	3.24	17	2.96	9.8
18	6.71	222	8.40	409	7.01	251	3.81	29	3.21	16	2.94	9.2
19	6.73	224	8.75	455	6.84	234	3.66	24	3.18	16	2.95	9.4
20	6.80	230	9.02	493	6.71	222	3.62	22		16	2.95	9.4
21	7.15	265	9.30	532	6.56	208	3.55	20		15	2.92	8.9
22	7.60	313	9.57	572	6.35	190	3.57	21	3.17	15	2.93	9.0
23	8.10	372	9.93	629	6.07	166	3.53	20	3.15	15	2.91	8.6
24	S	543	10.12	659	5.83	146	3.50	19	3.17	15	2.90	8.3
25	S	802	S	820	5.60	129	3.51	19	3.16	15	2.87	7.8
26	S	850	S	1,120	5.42	116	3.54	20	3.14	15	2.88	7.9
27	10.15	766	S	1,460	5.35	112	3.50	19	3.15	15	2.85	7.4
28	10.10	656	S	1,380	5.28	107	3.46	18	3.13	14	2.83	7.1
29	9.72	595	S	850	5.21	103	3.47	18	3.14	15	2.82	6.9
30	9.53	566	10.50	723	5.16	100	3.44	17	3.12	14	2.79	6.4
31			10.16	650			3.42	17	3.11	14		
Total		10,060		17,934		10,002		1,626		959		290.0
Mean		335		579		333		52.5		30.9		9.67
Run-off in acre-feet		19,950		35,570		19,840		3,230		1,900		575
Maximum		850		1,460		643		243		287		13
Minimum		199		352		100		17		14		6.4

September 1–10: Recorded range in stage was 2.97 to 3.09.

	Calendar Year 1965	Water Year 1966
Total	45,268.37	42,760.07
Mean	.124	117
Run-off in acre-feet	89,790	84,810
Maximum	1,520	1,460
Minimum	.25	.25

Peak discharge (base 800 cfs) -- Apr. 26 (1800 h) 1,080 cfs (12.40 ft); May 27 (1500 h) 9,310 cfs (32.51 ft); Aug. 7 (1830 h) 810 cfs (11.00 ft).

Max. Disch. 9,310 Sec-ft at 1500 h on May 27 (G H 32.51 ft) Max G H 32.51 ft at 1500 h on May 27
Min daily Disch. 0.25 Sec-ft on Nov. 20 (G H ft) Min G H ft on

Note.--Stage-discharge relation affected by ice Nov. 27 to Dec. 25.
No gage-height record Dec. 26 to Feb. 24, Aug. 20, 21, Sept. 2-10.
S- subdivided day; V- variable shift.

FIGURE 273.—Computation of daily discharge.

figures of daily mean gage height will be computed; for those days an uppercase letter "*S*" is entered in the gage-height columns. That symbol, as well as any others that are used, is explained by a footnote in the left margin; for example "*S*—subdivided day." For days of recorder malfunction, if the daily mean gage height is computed from a graph based on the observer's gage readings, the symbol "*g*" is added to the left of the gage-height value.

In a last step before applying discharges from the rating table to the gage heights, values of shifts to be applied are entered in columns constructed on the left side of the wide columns headed "Discharge" in figure 273. Little has been said about shifts in this chapter of the manual because they have been discussed in detail in the section in chapter 10 titled, "Shifts in the Discharge Rating." Shifts, it will be recalled, may vary with stage. If, during a subdivided day, shifts of varying magnitude are to be used because of the varying stage during the day, the symbol "v" is used in place of a numerical value in the shift column. The application of discharges to gage heights for subdivided days has been discussed in the section on "Subdivision of Daily Gage Heights." The reader is warned at this point that the shifts shown in figure 273 have no relation to the rating-curve analysis discussed in the section on "Rating-Curve Analysis." That analysis for Clear Creek indicated only a short period of shifting control in early October. Shifts have been scattered throughout figure 273 for the purpose of illustrating various conditions in applying discharge.

DETERMINATION OF DISCHARGE FROM THE GAGE-HEIGHT RECORD

Discharges are determined by applying the appropriate rating tables to the gage heights tabulated in figure 273. The rating analysis indicated a change in the rating after February 24, rating no. 3 being used up to and including that date and rating no. 4 thereafter. Consequently, before applying discharges a heavy horizontal line is drawn in the discharge column of figure 273 between February 24 and February 25 to warn the analyst of the change in rating on February 25. The daily mean discharges, in cubic feet per second, are entered in the discharge columns of figure 273. Daily discharges are shown to the nearest hundredth from 0.01 to 0.99 ft^3/s, to the nearest tenth from 1.0 to 9.99 ft^3/s, to the nearest unit from 10 to 999 ft^3/s, and to three significant figures above 1,000 ft^3/s. Where shifts are indicated, the amount of the shift is added algebraically to the tabulated gage height, and the discharge corresponding to the shift-adjusted gage height is determined from the appropriate rating table. It is important that there be no discontinuity between the discharge on the last day of the preceding water year and the first day of the current water year. That can easily occur if a new rating table is placed in effect on the first day of the current water year, or if shift adjustments to the gage height are used on either or both the first and last days of the two water years. Consequently the discharge for the last day of the preceding water year should be examined to ensure consistency.

To facilitate the determination of discharges from the rating table, it is advisable to expand the rating table to show the discharge for each one-hundredth of a foot of stage, as in figure 266, to cover the frequently occurring stages. For example, if the rating table were expanded to a stage of 7.0 ft, it would cover most of the gage heights tabulated in figure 273, thereby reducing the probability of error in mentally interpolating discharge values between the tenths of a foot of stage given in the standard rating table (fig. 265).

At this point all boxes for daily mean discharge in figure 273 will have been filled, except those opposite gage-height boxes that are blank for lack of record because of instrument malfunction, or those opposite gage-height boxes that carry the symbol "S" for subdivided day. The discharges for subdivided days are next computed. The method of computation was explained in the section on "Subdivision of Daily Gage Heights." The daily mean discharges are computed on the gage-height chart, as shown in figure 267, where the increment-mean method of computation was used. The computed discharges are then transferred to the discharge columns in figure 273.

ESTIMATION OF DAILY DISCHARGE FOR PERIODS OF INDETERMINATE STAGE-DISCHARGE RELATION

After the mean discharge has been computed for each day of the water year for which there is a gage-height record, a hydrograph of daily mean discharge is prepared on a form that has a logarithmic discharge scale. Discharge measurements are also plotted on the hydrograph sheet. The hydrograph is used for comparison with similar hydrographs of daily discharge for nearby stations as a test for consistency of the computed record. Obviously such comparison is only valid for streams whose daily flow is essentially natural, that is, not controlled significantly by the works of man. Hydrographic comparison usually brings to light any serious errors in the basic data computations and interpretations; it also provides a means of estimating discharge for days of no gage-height record and for days of indeterminate stage-discharge relation. A period of indeterminate stage-discharge relation does not refer to one in which the gage-height record is faulty; if the recorded gage-heights do not reflect the true stage of the stream, the period affected is considered to be one of no gage-height record. A period of indeterminate stage-discharge relation is one for which a satisfactory gage-height record is available, but one for which no stage-discharge relation can be determined. The most common situation of that kind occurs during an ice-affected period, and it may also occur during the passage of sand waves in an alluvial channel. Sometimes ephemeral backwater effect occurs when a channel is choked by debris for a few days, but in that situation the stage-discharge relation is not really indeterminate but is merely undefined because of the limited opportunity to define it by discharge measurements.

A period of indeterminate or undefined stage-discharge relation is indicated on the discharge tabulation form (fig. 273) by a heavy vertical line drawn between the gage-height and discharge columns. Such a line appears in November and December in figure 273 to indicate that the ice-affected discharges during those months bear no relation to the recorded stages. Where preliminary discharge values from the rating table have been entered for such days in figure 273 and are then shown by hydrographic comparison to be in error, they are replaced in figure 273 by the revised discharge figures.

Periods of ice effect.—The method of estimating discharge during periods of ice effect was discussed in detail in chapter 10 and will not be repeated here. Measurements nos. 35 and 35A (fig. 262) clearly indicated, by the magnitude of the backwater effect (shift values), that ice affected the stage-discharge relation.

Other periods of indeterminate stage-discharge relation.—For periods of indeterminate stage-discharge relation other than ice effect, discharges are estimated as though they occurred during periods of no gage-height record. Methods of treating periods of no gage-height record are described on the pages that follow; hydrographic comparison is one of those methods.

ESTIMATION OF DAILY DISCHARGE FOR PERIODS OF NO GAGE-HEIGHT RECORD

The analyst is often required to estimate discharge for periods of no gage-height record resulting from recorder malfunction, or a frozen well, or a plugged intake. Such periods are shown in figure 273 for periods December 26 to February 24, August 20–21, and September 2–10. The task of the analyst is greatly facilitated if the fieldman who finds the gage-height record incomplete makes an effort to collect as much supplementary information as possible. An attempt should be made to get the range in stage during the period of no gage-height record because that information indicates the limits of discharge within which any estimates made may vary. If the clock has stopped but the pen continues to function, the vertical line recorded on the chart will give the range in stage. Because of the possibility of the pen reversing during the period of no record, when the pen was scribing a vertical line, there may be some doubt as to the maximum gage-height reached during that period. If the tape gage is equipped with either a magnet or wire clip for indicating peak stage (see the section in chapter 10 titled, "Operation of a Recording Stream-Gaging Station"), the peak indicated by either of those devices should be noted. High-water marks should be sought both in the well and outside the gage structure. If the intakes have been plugged or the well frozen and a high stage had occurred during the period of no record, again an outside high-water mark should be sought. Local residents should be interviewed in an attempt to determine the time the peak occurred.

The previously mentioned annual hydrograph of daily mean discharge, with gaps left for periods of no gage-height record, along with the annual hydrograph of daily discharge for nearby stations, are prerequisites for estimating the discharges sought. Each of the station hydrographs should be plotted on a separate graph sheet, but the logarithmic discharge scales and time scales on the individual sheets should be identical. It is particularly helpful if one or more of the stations used is on the same stream as the station being studied. The hydrographs for uncontrolled streams in the same vicinity will usually have similar patterns of discharge.

In the discussion that follows, the procedure for estimating discharge for periods of no gage-height record is described under the following subheadings:

1. No gage-height record during a low- or medium-flow recession on an uncontrolled stream.
2. No gage-height record during periods of fluctuating discharge on an uncontrolled stream.
3. No gage-height record for a station on a hydroelectric powerplant canal.
4. No gage-height record for a station immediately downstream from a reservoir.
5. No gage-height record for a station on a controlled stream where the station is far downstream from the known controlled release.

CASE A. NO GAGE-HEIGHT RECORD DURING A LOW- OR MEDIUM-FLOW RECESSION ON AN UNCONTROLLED STREAM

If the vertical trace left by the inoperative recorder indicates no stages higher than that when the clock stopped nor any stages lower than that when the stoppage was discovered, there may well have been an unbroken recession from the time the clock first stopped. The hydrographs plotted for other nearby stations, particularly those on the stream being studied, should then be examined. If there is no evidence of anything but an unbroken recession, the discharge should be estimated by semilogarithmic interpolation. That is, the gap in the logarithmically plotted hydrograph for the station being studied should be filled by either a straight line or a smooth flat curve, depending on which best merges with the graph on either side of the dates of no gage-height record. The daily discharges that are estimated on the hydrograph are then transferred to the discharge-tabulation form with appropriate notation. (See record for September 2–10 in fig. 273.) If the period of no gage-height record involves only a few days, it is permissible to interpolate gage heights graphically on the recorder chart and then obtain the corresponding discharges from the rating table. That was actually done for August 20, 21 in figure 273.

CASE B. NO GAGE-HEIGHT RECORD DURING PERIODS OF FLUCTUATING DISCHARGE ON AN UNCONTROLLED STREAM

If a short period of recorder stoppage occurred near the peak of a stream rise, such as might occur if the float could not operate freely, knowing the peak stage of a stream makes it possible to sketch in the missing portion of gage-height record on the recorder chart. An even better estimate can be made on the recorder chart if the time of the peak is also known.

If long periods of no gage-height record are involved, the best method of making discharge estimates is by hydrographic comparison. A "light table" is used for the purpose in the manner described in the section in chapter 10 titled, "Hydrographic- and Climatic-Comparison Method." The logarithmic hydrograph of daily discharge for the study station is superposed on the logarithmic hydrograph for the reference station, and the date lines for the two sheets are matched. If the two stations are comparable, the two hydrographs should show similar runoff patterns. The study hydrograph is moved vertically until the hydrographs on either side of the period of no gage-height record match closely, making sure that the date lines match perfectly. An exception, to the perfect matching of date lines occurs, for example, where the two stations are on the same stream, but so distant from each other that the travel time between stations is approximately 24 hours. It would then be necessary to lag the hydrographs by a day. After matching the hydrographs, the missing portion of the study hydrograph is sketched by tracing the underlying reference hydrograph.

The hydrographic comparison also provides a simple means of comparing the runoff yield per square mile (unit yield) for the two stream basins. To make that comparison a short horizontal line, showing drainage-area size is marked on the logarithmic ordinate of each hydrograph. If, when the two hydrographs are matched vertically, the drainage-area lines also match, the two basins have equal unit yield. If the drainage area lines do not match, the basin whose drainage-area line is the lower of the two has the greater unit yield.

More often than not, it will be found that when the low-water part of the study hydrograph is matched with the low-water part of the reference hyrograph, the high-water parts of the two hydrographs do not match, and vice versa. When that occurs, the low-water parts of the two hydrographs are matched for sketching the low-water estimates, and the high-water parts of the two hydrographs are matched for sketching the high-water estimates of discharge. The discharge estimates for the medium-flow part of the study hydrograph is sketched while gradually sliding that hydrograph up or down, as required. Any discharge measurements made at the study station during the period of no gage-height record are especially valuable in positioning the two hydrographs, and unless it is known that the discharge measurement was made at a time of rapidly changing stage and is not representative of daily mean discharge, the sketched discharge on the study hydrograph should pass through the discharge measurement. If the range of stage for the period of no gage-height record is known, no estimated daily mean discharge should be smaller than the discharge corresponding to the minimum gage height for the period; no estimated daily mean discharge should equal or be greater than the discharge corresponding to the maximum gage height for the period, because the maximum daily discharge is seldom as great as the maximum momentary peak discharge. In figure 273 the daily mean discharges for the period of no gage-height record, December 26 to February 24, were estimated by hydrographic comparison with discharges for a nearby station.

It is desirable that hydrographic comparisons be made with more than a single reference station. The different comparisons will give estimates of daily discharge that differ from each other to some degree. In averaging the estimates, the greatest weight should be given to the results obtained from: reference hydrographs that show the closest fit with the study hydrograph; reference hydrographs on the same stream as the study station; and reference hydrographs for stations whose drainage areas approximate that of the study station.

If the period of no gage-height record involves a snowmelt period and the maximum stage is known, the maximum daily mean discharge can often be estimated fairly closely. Discharge has a diurnal fluctuation during snowmelt periods, and the ratio of maximum daily mean to maximum momentary discharge will vary with such factors as air temperature and date. However, examination of discharge records for the study station and for a snowmelt reference station may show how concurrent ratios vary at the two stations, and thereby give a strong clue to the ratio to be used to estimate maximum daily mean discharge during the period of no gage-height record.

On occasion, the station that has a period of no gage-height record may be located immediately upstream from a reservoir for the purpose of measuring inflow to the reservoir. If reliable records are available showing daily change in reservoir contents and daily spill and release from the reservoir, it is then a simple matter to compute the daily discharge (Q) at the gaging station from the formula:

$$Q = \text{Daily spill} + \text{daily release} \pm \text{daily change in reservoir contents.}$$

There may be times when record for a flood period is lacking and there is no nearby gaging station with which to compare runoff records. Under those circumstances, daily discharges for the flood period may be estimated from a model study of rainfall-runoff relations. It is beyond the scope of this manual to detail the development of such hydrologic models. A simpler task is to estimate the total volume of storm runoff from precipitation records. For general storms in the past at the study station, tabulate the total storm precipitation, its duration in days, and the total volume of storm runoff in inches or millimeters. Compute the value (infiltration index) that must be subtracted from each daily increment of precipitation during a storm to give the total volume of runoff from that storm. The infiltration index will vary with storms, but it can often be related to antecedent precipitation and month of the year. Apply the appropriate infiltration index to the storm precipitation during the period of no gage-height record to obtain the total volume of storm runoff during that period. This simple method provides only an approximate result; it should be used sparingly for general storms, and not at all for thunderstorms, which usually occur over limited areas.

CASE C. NO GAGE-HEIGHT RECORD FOR A STATION ON A HYDROELECTRIC-POWERPLANT CANAL

For a period of no gage-height record for a station on a powerplant canal, it is generally possible to use the powerplant record of daily kilowatt output to estimate reliably the daily mean discharges. That is done by means of a relation of daily discharge to daily power output that is developed for periods preceding and following the period of no gage-height record.

CASE D. NO GAGE-HEIGHT RECORD FOR A STATION IMMEDIATELY DOWNSTREAM FROM A RESERVOIR

Ratings are often available, or may be computed, for a reservoir spillway, gates, valves, and turbines (see the section in chapter 14 titled, "Pressure Conduits"). Ratings of those types will enable the engineer to estimate the discharge for a period of no gage-height record at a station immediately downstream from a reservoir.

Another method may be used if the reservoir itself is equipped with a stage gage so that a reliable record of daily change in reservoir contents is available. Daily changes in reservoir contents may be added algebraically to the daily mean discharge at the study station downstream from the reservoir to provide daily mean values of reservoir inflow during periods of record at the study station. An annual hydrograph of daily mean reservoir inflow is prepared and is compared with the hydrograph for a nearby natural-flow station. Using the technique described for Case B, the daily mean values of reservoir inflow are estimated for the period of no gage-height record at the study station. The known daily changes of reservoir contents are then subtracted algebraically from those estimated daily values of reservoir inflow to give the required daily discharge at the study station.

CASE E. NO GAGE-HEIGHT RECORD FOR A STATION ON A CONTROLLED STREAM WHERE THE STATION IS FAR DOWNSTREAM FROM THE KNOWN CONTROLLED RELEASE

Case E is a situation somewhat similar to Case D, except that the study station is so far downstream from the reservoir that tributary inflow between the reservoir and the study station cannot be ignored. Outflow from the reservoir cannot be compared directly with the discharge at the study station because the reservoir outflow is completely controlled and the discharge at the study station is partially controlled. The method of attacking the problem is to estimate daily tributary inflow during the period of no gage-height record at the study station, and then to add the estimated daily tributary inflow to the known upstream reservoir releases to obtain the required daily discharges at the study station. What is needed, therefore, is a means of estimating tributary inflow.

Daily releases from the reservoir are subtracted from the daily mean discharge at the study station to provide daily mean values of tributary inflow during periods of record at the study station. An annual hydrograph of daily mean tributary inflow is prepared and is compared with the hydrograph for a nearby natural-flow station. Using the technique described for Case B, the daily mean values of tributary inflow are estimated for the period of no gage-height record at the study station. As mentioned above, those estimated values of tributary inflow, when added to the concurrent reservoir releases, give the required discharges at the study station.

The above computational procedure may also be used for study reaches of channel that have diversions as well as tributary inflow, provided that the diverted discharges are measured. In that situation, the diversions must be subtracted from the reservoir releases. In other words, for reservoir release or outflow in the above description, we substitute reservoir outflow minus diverted flow.

COMPLETION OF THE DISCHARGE FORM

After all daily mean discharges have been entered on the discharge form (fig. 273), little is required to complete the form. Discharges from the appropriate rating table are entered in the left margin for the maximum and minimum stages of the water year that were previously recorded there. The summary discharge values at the bottom of figure 273 for each month, the water year, and the calendar year, are next computed. The mechanics of computing those total and average values are self-evident. The remaining entry in figure 273—peak discharges above a stated base—requires some explanation.

For stations whose high flows are not significantly regulated, peak discharges are shown for all peaks whose discharge equals or exceeds a chosen peak discharge, regardless of the number of peaks that occur in any given water year. A properly chosen base discharge is one that is exceeded, on the average, three times a year. The following suggestions are offered for selecting the base discharge:

1. For stations having records of more than 5 years, list the annual flood peaks, compute their recurrence intervals (R) in years by the formula, $R = (N + 1)/M$, and select as a base the discharge (rounded upward to two significant figures) whose value of R is 1.15 years. (In the formula, N is the number of years of record; M is the order number of the peak discharge after the peaks have been ranked in order of magnitude starting with 1 for the greatest peak.)
2. For stations having records of 5 years or less, select a base discharge, guided by judgment and by comparison with nearby stations having records of longer duration. The selected base can be modified as more data become available. It is, therefore, better to select a base discharge originally that is on the low side; if the base is later raised, it is a simple matter to drop originally selected peak discharges that do not exceed the new base value. If it is desirable later to lower the base discharge, it becomes necessary to search the earlier re-

corder charts for peak discharges that are smaller than the original base discharge but greater than the new base.

If two peak discharges that exceed the base discharge occur within 48 hours of each other, it is likely that the two peaks are not independent; only the larger of the two, or the earlier of the two if they are both equal, should be listed. If two adjacent peak discharges, both larger than the base, are separated by more than 48 hours, the lower of the two peaks is shown only if it is at least 1.33 times as large as the discharge of the trough between the adjacent peaks. For periods of diurnal peak discharges caused by snowmelt, only the highest peak that occurred during each distinct period of melting is shown regardless of the fact that other peaks may meet the criterion stated in the preceding sentence.

RECORD OF PROGRESS OF DISCHARGE COMPUTATIONS

Completion of the discharge form (fig. 273) marks the end of the actual computation of discharge for the water year. It is necessary, of course, that all computations be checked before the discharge figures are considered final. Furthermore, it is customary for the checker to initial and date any graphs or computation forms that he checks.

In the interest of efficiency it is advantageous to have a progress check list (fig. 274) attached to the folder in which the station computation forms are kept. The items on the check list are shown in the order in which they should be completed for maximum efficiency. Each item on the list has two boxes on the left margin. A checkmark is placed in the box at the extreme left when the item is completed; a checkmark is placed in the other box when the item has been checked. The supervisor of the discharge computations need only glance at the set of boxes to inform himself of the progress of the computations at a station.

STATION-ANALYSIS DOCUMENT

A complete analysis of data collected, procedures used in processing the data, and the logic upon which the computations were based must be recorded for each year of record to provide a basis for review and to serve as a reference in the event that questions arise about the records at some future date. Such a report is called the "Station Analysis." A record of any changes in: records collected, equipment, location, or other physical features should be included. The document should be written clearly and concisely and should contain sufficient information so that those who are totally unfamiliar with the station will be able to follow the reasoning used in computing the records. A

PROGRESS CHECK LIST
COMPUTATION OF GRAPHIC RECORDER RECORD

Station________________________ Water Year__________

Index Number________________________

Check work done. Complete in order. Initial when finished.

- [ ] Review chart for continuity, errors, peaks, faulty record.
- [ ] Check level notes. Apply datum corrections to chart and measurements.
- [ ] Check measurements, field notes, level notes for peak data. Enter on chart.
- [ ] Check mean gage heights of measurements. Compare with chart.
- [ ] List measurements and observations of no flow chronologically on 9-207 (fig. 262).
- [ ] Plot measurements on rating curve. Develop new curve and table, if necessary.
- [ ] Copy gage heights on 9-192 (fig. 273).
- [ ] Compute shifts, percentage differences on 9-207 (fig. 262).
- [ ] Enter shift corrections on 9-192 (fig. 273). "S" days on chart.
- [ ] Write station analysis.

Computed________________ Checked________________

....

- [ ] Apply discharge to 9-192 (fig. 273) and "S" days.

Computed________________ Checked________________

....

- [ ] Plot hydrograph. Enter measurements. Show drainage area size and discharge from rating tables.

....

- [ ] Estimate discharge for ice, missing, doubtful or backwater periods.
- [ ] Revise and complete daily discharges on 9-192 (fig. 273).
- [ ] Review and complete station analysis.

Computed________________ Checked________________

....

- [ ] Make monthly and yearly computations on 9-192 (fig. 273).
- [ ] Enter notes, maximum, minimum and peaks on 9-192 (fig. 273).
- [ ] Revise manuscript from previous year.

Computed________________ Checked________________

....

Final Review________________________________

FIGURE 274.—Form showing progress of computation of graphic-recorder record.

station analysis should be prepared for each station, including those for which records are furnished by other agencies.

The introductory paragraphs of the station analysis describe the equipment installed and the hydrologic characteristics of the drainage basin above the station. The remaining sections of the analysis outline the quality of the base data collected and the methods used to convert those data into the final discharge figures. The discussions are organized under the headings that follow.

(Station name and number)
STATION ANALYSIS
(WATER YEAR)

Equipment.—
Hydrologic conditions.—
Gage-height record.—
Datum corrections.—
Rating.—
Discharge.—
Special computations.—
Remarks.—
Recommendations.—

The following detailed discussion of each of the above items describes the type of information to be presented. As mentioned in the introductory pages of this chapter, documentation of that information is made as the various steps in the analysis and computation of the discharge record are completed.

STATION ANALYSIS

Equipment.—Provide a short statement that describes the equipment at the site. Designate the type of gage (float sensor or bubble-gage sensor); type of recorder; measurement facilities; artificial control, if any. Report any changes in equipment that may affect the accuracy of the record. Review the station description, revise it if necessary and include the statement, "Equipment conforms to station description dated. . . ."

Hydrologic conditions.—A brief description of the hydrologic characteristics of the basin should be carried forward in the station analysis from year to year. Review this paragraph and briefly describe any changes that might affect the runoff regime. These changes may result from fire (give date and percentage of basin area affected), or urban development (describe type and extent of development and give approximate dates), or from logging or road building operations. Usually several years elapse before the effects of these hydrologic changes become stabilized. Therefore, even if no changes occur in the current year, this paragraph should carry a statement referring to changes in the recent past such as: "No changes since the fire of August 21, 1961, which burned 6,000 acres of woodland;" or "No increase in urban development since September 1962."

Gage-height record.—Tabulate periods of faulty or no gage-height record and reasons for those problems. Discuss briefly any large instrument errors that affect the accuracy of the gage-height record. If portions of the gage-height record have been synthesized or adjusted on the basis of observers' readings and other data, this should be explained. Do *not* discuss in this paragraph how discharge was computed during periods of no gage-height record. That should be explained in the "Special Computations" paragraph.
Datum corrections.—Confusion frequently exists as to what should be included in this paragraph. Datum errors result from settlement of the base or reference gage to which the recording instrument is set or from movement of the bubble-gage orifice. Care should be taken, particularly with manometer and digital recorder combinations to differentiate between datum corrections and shift corrections. If datum corrections are necessary, the reasons should be explained and corrections listed in tabular form such as:

Period	*Correction applied*
Oct. 1–Jan. 15	+0.04
Jan. 16–Apr. 15	+0.05
Apr. 16–Aug. 3	+0.06
Aug. 4–Sept. 30	0

If applicable use a simple statement such as "None applied, last levels run on (date) ."
Rating.—Start this section with a description of the channel and the control, and provide sufficient detail to give anyone unfamiliar with the site a fairly good picture of the dominant features. Items discussed should include the size of the channel, composition of the bed (sand, gravel, boulders, or bedrock), location of the gage relative to the control, and the approximate elevation of any overflow areas.

Example: "The controlling reach of channel is sharply incised in the flood plain. Bed material is predominantly sand and gravel. The low-water control is generally a gravel riffle which moves up and down the channel in response to flood flows. At bankfull stage (about 21 feet), the channel is about 150 feet wide. At higher stages, it spreads out rapidly to a width of about 300 feet at a stage of 25 feet."

The remainder of the rating paragraph should be a chronological narrative of what occurred, hydraulically, during the year. Begin with a statement as to the number of measurements made and how they plot in relation to the rating curve in use at the end of the previous year. If new ratings are required, explain how this conclusion was reached and what caused the shift from one rating to the other. State exact time and date when rating changes were made. If ratings are modified during periods of significant flow by use of the shifting-control method, document these rating changes with shift tables or shift curves. These are rating changes too, and require the same explanations that a new table does. Because the reviewer does not always have access to the basic data, it is most important that the distribution of shifts be explained in detail, particularly if any unusual methods were used.

The statement "Shifts were distributed on basis of stage and (or) time" does not constitute a detailed explanation. The reviewer needs sufficient detail so that he can at least determine if a shift must be applied to the maximum and secondary peak stages and know its magnitude. For example, discharge measurements were obtained before and after a peak of 12.55 ft; the measurement preceding the peak shows a shift of −0.26 ft at gage height 2.56 ft, and the one following the peak shows a shift of + 0.06 ft at gage height 9.63 ft. One might reason that the rise scoured out the channel gradually, and the shift was zero at the peak. In the analysis, one might state "It was assumed that the shift of −0.26 ft indicated by measurement No. xx was gradually reduced during rise, and there was no shift at the peak; therefore, the shift between measurements No. xx and xxx was distributed on basis of stage." Or, one might have basis for this statement: "On the basis of shifts indicated by measurements No. xx and xxx and succeeding measurements, shift distribution was made on the assumption that the shift varied during the rise from −0.26 ft at gage height 2.50 ft to +0.06 ft at the peak and remained at +0.06 ft through the date of measurement No. xxx." Those two statements would indicate to the reviewer the shift needed for the peak stage and would give him a better idea of the distribution of shifts that was made. If a shift distribution were made on the basis of time, the statement "Shifts were distributed on the basis of time" is sufficient. However, if a peak discharge occurred during that shifting-control period, a statement should be added giving the shift used for the peak.

Discuss also the adequacy of the high-water rating. Is it defined to within 50 percent of the maximum discharge for the current year on the basis of measurements made during the year? (The 50 percent criterion is discussed early in the section in chapter 10 titled, "High-Flow Extrapolation.") If the extension has been made on the basis of older measurements or on the basis of a slope-area determination (chap. 9), give the date of those measurements or of the slope-area determination and state whether or not significant channel changes might have occurred within the intervening period.

Discharge.—This paragraph is a summary explaining how the stage records and rating data were combined to produce the discharge record. The information can best be presented in tabular form; an example for a station equipped with a graphic stage-recorder follows. (The table would be more complex for a station equipped with a digital stage-recorder; see page 599.)

Period	*Rating table used*	*Periods of shifting control*
Oct. 1 to Feb. 24	No. 3	Oct. 1–9, Oct. 14 to Nov. 14
Feb. 25 to Sept. 30	No. 4	July 15 to Sept. 1, Sept. 11–30

Special computations.—Describe the methods used for determining discharges during the periods of no gage-height record, ice effect, backwater, or other special conditions. Explain any unusual method for determining shifts. If daily discharges were estimated on the basis of hydrographic comparison with records for nearby stations, state the name of the stations used and how closely the station records compared. If weather records were used in the analysis, give the name or names of the weather stations used.

Remarks.—A statement should be made concerning the general accuracy of the daily records along with special accuracy statements regarding periods of ice effect, no gage-height record, high water, low water, backwater, shifting control, or other unusual conditions. A statement should be made here indicating that a hydrographic comparison was made. Identify station or stations used for comparison and state how well the hydrographs compared. Although the statement concerning hydrographic comparison duplicates some of the material given above under the heading "Special computations," the duplication is warranted because it will expedite the preparation of the "Remarks" paragraph of the manuscript station description. (See figs. 286*H* and 289.) It is helpful if all statements to be included in that manuscript paragraph can be drawn from material in the "Remarks" section of the station-analysis document. The "Remarks" section of the station-analysis document should also include any additional comments pertinent to the analysis of the record.

Recommendations.—A sample recommendation might read, "Flood schedule for next year should place high priority on high water measurements at this site. No measurements greater than 8,000 ft^3/s have been made since 1967. There have been several major peaks since that date."

(Authors) W. W. Smith (date)
A. R. Brown (date)

COMPUTATION OF DISCHARGE RECORDS WHEN A THREE-PARAMETER DISCHARGE RELATION IS USED

The section of this chapter titled, "Station Analysis," ended with a brief discussion of the preparation of the station rating when three parameters are involved—stage, discharge, and a third parameter such as fall or velocity index.

The first step that follows completion of the station rating is the computation of the gage-height record for the base-gage recorder, and for the auxiliary-gage recorder if fall is the third parameter. The daily mean gage heights are determined by the procedures explained for the graphic recorder in the section titled, "Computation of Gage–Height Record." Where subdivision of the day is required, the same time increments are used for both recorder charts. The daily mean fall, or mean fall for a time increment in a subdivided day, is computed by subtracting the downstream stage from the upstream stage. If a velocity index is the third parameter, as for example, where a deflection meter is used, the velocity-index record is used to determine daily mean values of the index or mean values for the time increments used in subdivided days. Gage-heights and velocity-index values are entered on a form similar to, but larger than, the form shown in figure 273. The expansion of the form is to accommodate an additional column each month for recording daily values of the third parameter; the additional column lies between the gage-height and discharge columns that are shown in figure 273.

The mechanics of computing discharge from stage and concurrent values of the third parameter were discussed in chapters 11 and 12. In chapter 11 slope (fall) is the third parameter; in chapter 12 a velocity index is the third parameter. Computed values of daily discharge are entered on the form bearing the daily values of stage and the third parameter. The daily discharges for periods of no record or of indeterminate discharge rating, such as ice-affected periods, are computed precisely as explained in a preceding section titled, "Computation of Daily Discharge;" hydrographic comparison is the principal method used. After all boxes for daily mean discharge on the discharge form are filled, the form is completed as shown in figure 273 and explained in the section on "Completion of the Discharge Form."

Throughout the computation procedure, a record of progress is kept, similar to that shown in figure 274 but modified to accommodate the additional steps needed to compute discharge when a 3-parameter discharge relation is used. A station-analysis document is prepared, similar to that described in the section immediately preceding this discussion of 3-parameter relations; the various items that are included are documented as corresponding steps in the analysis and computation of the discharge record are completed.

COMPUTATION OF DISCHARGE RECORDS FOR A RECORDING STATION EQUIPPED WITH A DIGITAL RECORDER

GENERAL

The fact that a gaging station is equipped with a digital stage-recorder does not affect the preparation of the station analysis (see the section on "Station Analysis"). Datum corrections are determined, discharge measurements are listed and reviewed, and graphical ratings are prepared and then converted to rating tables. The computations that follow the station analysis are similar to those described for a station equipped with a graphic stage-recorder, but instead of being performed manually they are performed by an electronic computer; the principal output forms are machine adaptations of the manual computation forms. The field offices generally send their input data to a central computer center where the computations are performed. The processing between field office and computer center may be accomplished by a combination of two or more of the following: mail, 16-channel paper-tape reader-transmitter, telephone line, and computer terminal.

The sequence and operation of an automated computing system is described in general terms in the last section of this chapter. It is not practicable to include a more detailed description of each step in the sequence because although the system of automated computation is well established, the particulars of each step are somewhat in a state of flux in response to continual improvement in storage and access procedures. Space limitations in this manual are also a factor in the treatment given to the subject. Additional pertinent information for the interested reader can be found in the following references that are listed at the end of this chapter: Carter and others, 1963; Edwards and others, 1974; WMO Technical Note No. 115, 1971 (contains a noteworthy bibliography).

The automated computation of discharge records from digital stage records is now (1980) more common in the U.S.A. than the manual computation of discharge records from graphic stage records. It may therefore seem incongruous to devote more space in this manual to manual computation than to automated computation. The two types of computation, however, are essentially similar, and a description of the manual method provides a far superior vehicle for explaining the computational technique.

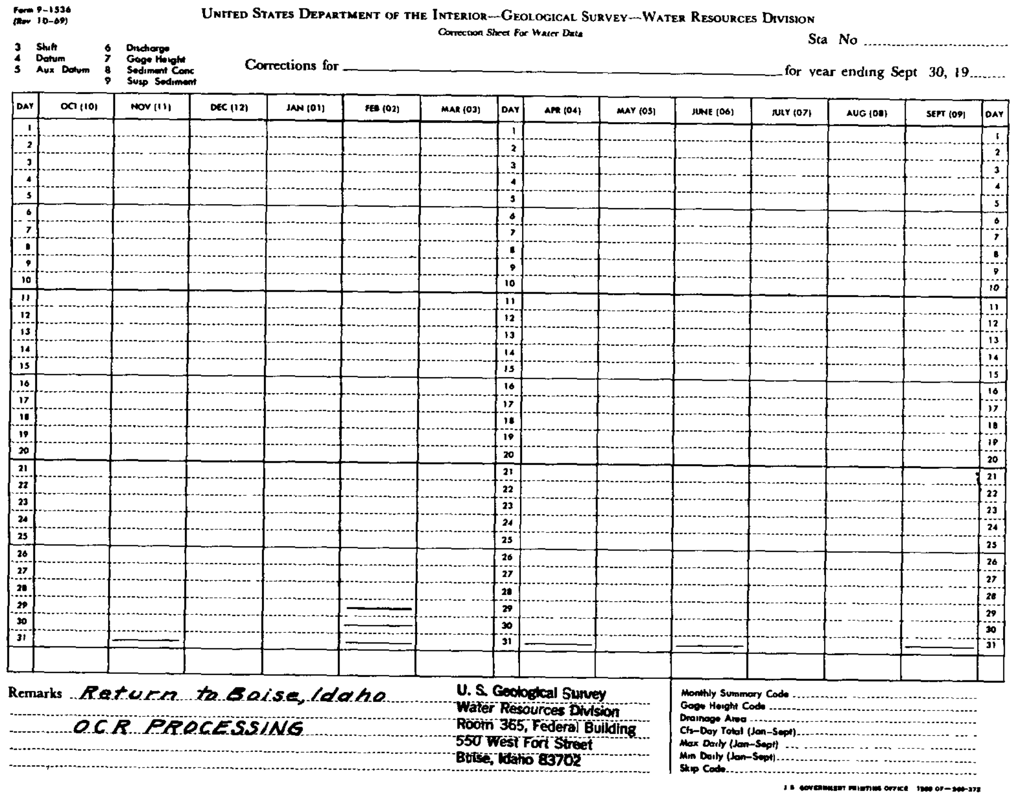
INPUT TO COMPUTER

The input to the computer for a routine gaging station consists of: (1) the digital record of stage, accompanied by a list of corrections, if needed, for instrumental error in recording time and (or) gage height (any necessary datum corrections are included with the gage-height corrections); and (2) the discharge ratings accompanied by a list of any necessary shift adjustments. (See USGS form 9-1536 in fig. 275.) Stations for which the stage-fall-discharge type of rating is applicable require that the digital-tape records of stage for both the primary and auxiliary gages be furnished to the computer. Also required are the stage-discharge relation and such supplementary information as the stage-fall relation and the relation of fall ratio to discharge ratio. For stations at which velocity index is a third parameter—for example, a station equipped with a deflection meter—input requirements include the digital stage record, the digital record of deflection units, the stage-area relation, and the relation of deflection units to mean velocity, along with any necessary shift adjustments to those two relations.

OUTPUT FROM COMPUTER

The principal output from the computer consists of two forms—the primary computation sheet and the print-out of daily discharge. The primary computation sheet presents the initial or preliminary discharge computations. Normally, the computation sheet is edited, discharges are corrected or revised where necessary, and the corrections are fed back to the computer before the print-out of daily discharge is produced. Computer-produced hydrographs of daily mean discharge may be obtained for both preliminary and final discharge values. The discharge hydrograph of daily mean discharge based on preliminary values of discharge is very helpful for correcting the preliminary values; the method used is that of hydrographic comparison with final records for a nearby station, as explained in the section titled, "Computation of Daily Discharge." (Hydrographic comparison of discharge records is discussed in the two subsections that deal with the estimation of daily discharge.)

The primary computation sheet for a routine gaging station includes a listing for each day of: the maximum, minimum, hourly, and mean gage heights; mean discharge; the gage height equivalent to the mean discharge; the shift adjustment; and the datum correction. Figure 276 is an example of a primary computation sheet for a routine gaging station. The primary computation sheet for a slope station, shown in figure 277, differs somewhat. Listed for each day are: the maximum, minimum, and mean gage heights, mean fall, and mean and hourly discharge. For a deflection-meter station, the primary computation sheet (fig. 278) lists for each day maximum,



Form 9-1536
(Rev 10-69)

UNITED STATES DEPARTMENT OF THE INTERIOR—GEOLOGICAL SURVEY—WATER RESOURCES DIVISION
Correction Sheet For Water Data

Sta No ....

3 Shift 6 Discharge
4 Datum 7 Gage Height
5 Aux Datum 8 Sediment Conc
9 Susp Sediment

Corrections for ______________________ for year ending Sept 30, 19........

DAY	OCT (10)	NOV (11)	DEC (12)	JAN (01)	FEB (02)	MAR (03)	DAY	APR (04)	MAY (05)	JUNE (06)	JULY (07)	AUG (08)	SEPT (09)	DAY
1							1							1
2							2							2
3							3							3
4							4							4
5							5							5
6							6							6
7							7							7
8							8							8
9							9							9
10							10							10
11							11							11
12							12							12
13							13							13
14							14							14
15							15							15
16							16							16
17							17							17
18							18							18
19							19							19
20							20							20
21							21							21
22							22							22
23							23							23
24							24							24
25							25							25
26							26							26
27							27							27
28							28							28
29							29							29
30							30							30
31							31							31

Remarks Return to Boise, Idaho
OCR PROCESSING

U. S. Geological Survey
Water Resources Division
Room 365, Federal Building
550 West Fort Street
Boise, Idaho 83702

Monthly Summary Code
Gage Height Code
Drainage Area
Cfs-Day Total (Jan-Sept)
Max Daily (Jan-Sept)
Min Daily (Jan-Sept)
Skip Code

U S GOVERNMENT PRINTING OFFICE 1969 OF—360-372

FIGURE 275.—Correction and update form for daily values of discharge.

UNITED STATES DEPARTMENT OF INTERIOR - GEOLOGICAL SURVEY - WATER RESOURCES DIVISION DIST 25
PRIMARY COMPUTATION OF GAGE HEIGHTS AND DISCHARGE
01103500 DATA PROCESSED 01-14-74
CHARLES RIVER AT CHARLES RIVER VILLAGE, MASS. USE RT 32

PROVISIONAL DATA FOR WATER YEAR ENDING SEPT. 30, 1973 TEST DIFF 0.2 PUNCH INT 15 MIN STORE PARM 00060, STATISTIC 00003
PARM 00065, STATISTIC 00003

DATE	MAX GH (TIME)	MIN GH (TIME)	MEAN GH	EQUIV GH	MEAN DISCH	SHIFT ADJ	AT HR	DATUM CORR	AT HR		STAGE, IN HUNDREDTHS OF FEET, AT INDICATED HOURS 1	2	3	4	5	6	7	8	9	10	11	12
9-18	1.07	0.92	0.98	.98	113					AM	93	93	93	93	93	93	93	93	92	93	93	93
	(1630)	(0715)								PM	96	99	104	106	107	106	105	104	104	103	102	102
9-19	1.02	1.00	1.00	1.01	122					AM	101	101	101	100	100	100	100	100	100	101	101	101
	(0015)	(0315)								PM	101	101	101	101	101	101	101	102	101	101	101	101
9-20	1.04	1.01	1.02	1.03	131					AM	101	102	102	102	102	103	103	103	103	103	103	103
	(1215)	(0015)								PM	104	104	104	104	104	104	104	103	104	104	104	104
9-21	1.04	1.00	1.02	1.02	127					AM	103	103	103	103	103	103	103	102	103	102	102	102
	(0015)	(2230)								PM	102	102	102	101	101	101	101	101	101	101	100	100
9-22	1.00	0.98	0.98	.99	116					AM	100	100	100	100	100	100	100	99	99	99	99	98
	(0015)	(1200)								PM	98	98	98	98	98	98	98	98	98	98	98	98
9-23	0.98	0.96	0.97	.97	110					AM	98	98	98	97	97	97	97	97	97	97	97	97
	(0100)	(1315)								PM	97	97	97	97	97	97	97	97	97	98	98	98
9-24	1.06	0.98	1.01	1.02	125					AM	98	99	99	99	99	100	100	100	101	101	101	101
	(2230)	(0015)								PM	101	101	102	102	103	103	104	105	105	105	106	106
9-25	1.14	1.06	1.09	1.10	155					AM	106	106	107	107	107	107	108	108	109	110	110	110
	(2030)	(0015)								PM	110	110	111	111	112	112	113	113	114	114	114	114
9-26	1.17	1.14	1.16	1.16	177					AM	115	115	115	115	116	116	116	116	117	117	117	117
	(0845)	(0015)								PM	117	117	117	117	117	117	117	117	117	117	117	117
9-27	1.17	1.14	1.16	1.16	175					AM	117	117	117	117	117	117	117	117	117	116	117	116
	(0015)	(1945)								PM	116	116	116	116	115	115	115	114	114	114	114	114
9-28	1.14	1.07	1.10	1.10	156					AM	113	113	113	113	113	112	112	112	112	111	111	111
	(0015)	(2015)								PM	110	110	109	109	108	108	108	108	107	107	107	107
9-29	1.07	1.00	1.03	1.04	133					AM	107	105	106	106	105	105	105	105	105	104	104	105
	(0015)	(2030)								PM	104	104	103	103	102	102	101	101	100	100	100	100
9-30	1.00	0.94	0.96	.97	109					AM	99	99	99	99	98	98	98	98	98	98	98	98
	(0015)	(1945)								PM	97	95	96	96	95	95	95	94	94	94	94	94
MONTH					116																	
PERIOD	1.17	0.83																				

RECORDER CLOCK RUNNING 3.1 MINUTES PER DAY FAST. RECORD ADJUSTED.

NOTE. SYMBOLS USED ABOVE HAVE THE FOLLOWING MEANINGS
A - SUCCESSIVE ADJUSTED PUNCH READINGS DIFFER BY MORE THAN THE SPECIFIED ALLOWABLE TEST DIFFERENCE
P - DAILY SUMMARY IS FOR AN INCOMPLETE DAY
R - ONE OR MORE INPUT VALUE IS OUTSIDE THE RANGE OF THE RATING IN USE
W - SHIFT IS A VALUE WEIGHTED BY DISCHARGE WHERE SHIFT VARYS WITH STAGE DURING THE DAY

FIGURE 276.—Primary computation sheet for routine gaging station.

UNITED STATES DEPARTMENT OF INTERIOR - GEOLOGICAL SURVEY - WATER RESOURCES DIVISION DIST 36
PRIMARY COMPUTATION OF DISCHARGE WHERE SLOPE OF THE WATER SURFACE IS A FACTOR
DATA PROCESSED 02-11-74

04246500
ONEIDA RIVER AT CAUGHDENOY NY DISCH RT 3 FALL RT 2 RATIO RT 1

PROVISIONAL DATA FOR WATER YEAR ENDING SEPT. 30, 1974
BASE GAGE IS UPSTREAM DATUM DIFFERENCE BETWEEN GAGES 0.00 TEST DIFF 0.1 STORE PARM 00060, STATISTIC 00003
SLOPE IS NOT A FACTOR BELOW OR ABOVE PUNCH INTERVAL 60 MIN

S	DATE	MAX GH (TIME)	MIN GH (TIME)	MAX DISCH (TIME)	MEAN GH	MEAN FALL	MEAN DISCH	1	2	3	4	5	6	7	8	9	10	11	12
								DISCHARGE AT INDICATED HOURS(1ST LINE A.M., 2ND LINE P.M.)											
	12-26	8.22	7.98	4560	8.06	3.08	4310	4180	4100	4230	4230	4260	4250	4260	4260	4220	4220	4230	4280
		(2300)	(0100)	(2300)				4310	4310	4330	4310	4340	4330	4390	4410	4420	4500	4560	4450
B	12-27*	8.62	8.19	5220	8.40	3.42	4850	4520	4550	4660	4640	4710	4630	4710	4630	4820	4740	4870	4750
		(2400)	(0100)	(2400)				4930	4830	4970	4930	5050	5000	5080	5080	5150	5140	5200	5220
	12-28	9.00	8.60	5860	8.82	3.84	5550	5190	5240	5300	5400	5350	5460	5390	5420	5390	5460	5520	5590
		(2400)	(0100)	(2400)				5620	5610	5640	5660	5690	5740	5740	5780	5790	5810	5840	5860
	12-29	9.17	9.00	6150	9.09	4.11	6010	5880	5910	5860	5940	5930	5940	5940	5960	6000	5980	6030	6000
		(2100)	(0100)	(2100)				6950	6010	6050	6060	6080	6060	6130	6100	6150	6100	6130	6050
	12-30	9.23	9.05	6250	9.17	4.28	6150	6110	6080	6000	5940	6050	6080	6230	6080	6210	6130	6160	6100
		(1700)	(0400)	(1700)				6160	6110	6230	6150	6250	6200	6230	6210	6210	6210	6230	6250
	12-31	9.27	9.21	6320	9.24	4.23	6260	6270	6250	6270	6230	6280	6230	6300	6280	6320	6270	6300	6270
		(0900)	(2100)	(0900)				6300	6250	6270	6250	6280	6270	6250	6270	6210	6270	6210	6210
	MONTH						4927												
	1-01	9.24	9.11	6270	9.19	4.16	6170	6230	6230	6250	6270	6200	6210	6200	6200	6230	6210	6210	6200
		(0400)	(1400)	(0400)				6160	6050	6180	6100	6160	6080	6180	6080	6150	6100	6130	6100
	1-02	9.15	9.07	6110	9.12	4.10	6060	6100	6110	6100	6110	6100	6100	6100	6100	6100	6080	6060	6100
		(0200)	(2300)	(0200)				6060	6050	6050	6050	6050	6030	6010	6010	6030	6010	5980	6030
	1-03	9.08	8.97	6000	9.03	4.02	5920	5980	6000	5960	6000	5940	5980	5920	5980	5930	5950	5890	5940
		(0200)	(2200)	(0200)				5890	5910	5860	5890	5890	5840	5880	5830	5880	5810	5880	5830
	1-04	8.99	8.93	5840	8.95	3.91	5780	5840	5790	5810	5780	5760	5760	5790	5760	5740			
		(0100)	(0900)	(0100)															

PERIOD MAX GH 9.27 (0900) 12-31 MIN GH 7.76 (0700) 12-21 MAX DISCH 6320 (0900) 12-31 MIN DISCH 1470.00 ON 11-16

NOTE.- SYMBOLS USED ABOVE HAVE THE FOLLOWING MEANINGS
A - SUCCESSIVE RECORDED GAGE HEIGHTS AT AUXILIARY GAGE DIFFER BY MORE THAN THE SPECIFIED ALLOWABLE TEST DIFFERENCE
B - SUCCESSIVE RECORDED GAGE HEIGHTS AT BASE GAGE DIFFER BY MORE THAN THE SPECIFIED ALLOWABLE TEST DIFFERENCE
R - ONE OR MORE INPUT VALUE IS OUTSIDE THE RANGE OF THE RATING TABLE
* - OBSERVED FALL IS ZERO OR NEGATIVE FOR ONE OR MORE SETS OF OBSERVATIONS

FIGURE 277.—Primary computation sheet for slope station.

minimum, and mean gage heights; maximum, minimum, hourly, and mean discharges; maximum and minimum velocities; volume and direction of flow (for a tidal stream whose flow reverses direction); and shift adjustments to the area and velocity relations.

The printout of daily discharge is virtually the same for all types of gaging stations. In addition to daily mean discharges, the printout includes monthly and yearly summaries in the same format that is used for publication (fig. 279). Besides being published, the figures on the printout are stored on a magnetic tape or disk. If, for some reason, it is found necessary to revise the computed records at some later date, corrections are made on the stored tape or disk.

SEQUENCE OF OPERATION OF AN AUTOMATED COMPUTING SYSTEM

The sequence of operation of the automated computing system used by the Geological Survey is as follows:

1. River stage is punched on 16-channel tape by the digital recorder in the gage house. When a segment of the tape is started by the hydrographer, he leaves a fresh inspection form (USGS form 9–176D in fig. 280) in the instrument shelter. On that form he fills out the box headed "Started by".
2. Tape is removed by field personnel at intervals of 30 to 60 days. Upon removal, the tape is checked for continuity and quality of record, and appropriate notes concerning identity of the station and quality of the record are made on the tape. The boxes headed "Removed by" and "Battery voltage" on inspection form 9–176D (fig. 280) are also filled out by the hydrographer, and the form accompanies the segment of 16-channel tape to the field office. If the hydrographer merely inspects the recorder without removing the punched segment of tape, he fills out the box headed "Insp'd by" on the inspection form and leaves the inspection form in the instrument shelter.
3. The tape, rating table, datum correction, and table of shifts are forwarded from the field office to the Automatic Data Processing Unit. Ratings may be submitted in one of three alternate forms. Discharge may be tabulated for each 0.01 foot of gage height for the part where curvilinear expansion between tenths of feet is necessary; it may be tabulated for each 0.1 foot; or, preferably, it may be defined by a series of coordinate values at the ends of straight-line segments on a logarithmic plot of the rating curve. The entry of ratings directly from the logarithmic plot eliminates the preparation of a rating table in the usual form. Shift adjustments are prorated with time to give a shift for each day between the days for which values of shift are submitted. A new rating may be put in use at any

UNITED STATES DEPARTMENT OF THE INTERIOR - GEOLOGICAL SURVEY - WATER RESOURCES DIVISION DIST 21
COMPUTATION OF DISCHARGE AT DEFLECTION METER STATION
03438192 DATA PROCESSED 02-26-74
BARKLEY-KENTUCKY CANAL NEAR GRAND RIVERS, KY. STANDARD DATUM CORR 0.00 FEET

PROVISIONAL DATA FOR WATER YEAR ENDING SEPT. 30, 1974 PUNCH INTERVAL 15 MIN STORE PARM 00060, STATISTIC 00003

DATE	GAGE HEIGHT IN FEET	DISCHARGE IN CFS	TIME	VELOCITY IN FT/SEC	VOLUMES IN MCF		HOURLY DISCHARGE (6 HOURS PER LINE, BEGINNING AT 1 A.M.) 1	2	3	4	5	6
1-23	MAX 62.87	MAX 35000.00	AT 0600	MAX 2.90	DOWN 1299.45	AM	10500.00	9840.00	8730.00	16200.00	24800.00	35000.00
	MIN 62.29	MIN 729.00	AT 0130	MIN .06	UP	AM	26100.00	2420.00	15908.00	7280.00	12200.00	6440.00
	MEAN 62.54	MEAN 15000.00				PM	11800.00	3170.00	21900.00	1590.00	18400.00	5640.00
	(DATUM 0.00)	(SHIFT TO AREA	0.00)	(SHIFT TO VEL	0.00)	PM	22500.00	21300.00	22000.00	11300.00	22600.00	21100.00
1-24	MAX 63.17	MAX 23400.00	AT 0030	MAX 1.90	DOWN 1115.41	AM	19100.00	6530.00	13000.00	21700.00	21700.00	10300.00
	MIN 62.83	MIN 1620.00	AT 1430	MIN .13	UP	AM	17200.00	12400.00	22800.00	17200.00	15600.00	18200.00
	MEAN 62.99	MEAN 13000.00				PM	11100.00	11100.00	2490.00	8960.00	10700.00	8560.00
	(DATUM 0.00)	(SHIFT TO AREA	0.00)	(SHIFT TO VEL	0.00)	PM	7470.00	10700.00	6510.00	9720.00	9350.00	4990.00
1-25	MAX 63.44	MAX 12500.00	AT 0545	MAX 1.00	DOWN 370.17	AM	6620.00	5000.00	12100.00	8370.00	12100.00	9010.00
	MIN 63.16	MIN	AT 0930	MIN	UP	AM	3250.00	8640.00	4130.00	2510.00		4140.00
	MEAN 63.31	MEAN 4310.00				PM	10300.00	4150.00	3270.00	3270.00	4150.00	2520.00
	(DATUM 0.00)	(SHIFT TO AREA	0.00)	(SHIFT TO VEL	0.00)	PM	735.00	756.00			1640.00	756.00
1-26	MAX 63.84	MAX 31900.00	AT 1545	MAX 2.50	DOW 475.20	AM	756.00	7550.00	1640.00	758.00		758.00
	MIN 63.40	MIN	AT 0030	MIN	UP	AM	1640.00	758.00	759.00	760.00	760.00	760.00
	MEAN 63.61	MEAN 5500.00				PM	6720.00	8520.00	14800.00	28000.00	13400.00	14100.00
	(DATUM 0.00)	(SHIFT TO AREA	0.00)	(SHIFT TO VEL	0.00)	PM	15100.00	5110.00	4200.00	1660.00	3320.00	1660.00
1-27	MAX 63.78	MAX 10500.00	AT 2145	MAX 1.47	DOWN 578.32	AM	6750.00		2550.00	2550.00	764.00	764.00
	MIN 63.39	MIN	AT 0015	MIN	UP	AM	764.00	763.00	764.00	1660.00	763.00	10900.00
	MEAN 63.62	MEAN 6640.00				PM	762.00	762.00	2540.00	9510.00	15700.00	10900.00
	(DATUM 0.00)	(SHIFT TO AREA	0.00)	(SHIFT TO VEL	0.00)	PM	18100.00	16300.00	12600.00	9070.00	17200.00	13600.00
1-28	MAX 63.41	MAX 28700.00	AT 1100	MAX 2.29	DOWN 971.55	AM	16500.00	17500.00	16800.00	10200.00	12500.00	10100.00
	MIN 62.97	MIN 4090.00	AT 2300	MIN .33	UP	AM	18800.00	13600.00	28100.00	17400.00	28700.00	16400.00
	MEAN 63.20	MEAN 11300.00				PM	13500.00	10100.00	6600.00	5730.00	4110.00	4100.00
	(DATUM 0.00)	(SHIFT TO AREA	0.00)	(SHIFT TO VEL	0.00)	PM	4100.00	4100.00	4090.00	4090.00	4090.00	5700.00
1-29	MAX 62.97	MAX 20700.00	AT 1445	MAX 1.68	DOWN 819.42	AM	5640.00	5690.00	5590.00	5690.00	4940.00	4940.00
	MIN 62.80	MIN 4940.00	AT 0600	MIN .40	UP	AM	5540.00	5580.00	5670.00	5670.00	5570.00	5670.00
	MEAN 62.87	MEAN 9440.80				PM	8500.00	16200.00	11400.00	17100.00	15900.00	9960.00
	(DATUM 0.00)	(SHIFT TO AREA	0.00)	(SHIFT TO VEL	0.00)	PM	12000.00	11700.00	6510.00	15900.00	13900.00	15900.00
1-30	MAX 63.37	MAX 20600.00	AT 1215	MAX 1.66	DOWN [illegible]	AM	15540.00	15300.00	15300.00	14900.00	10600.00	8510.00
	MIN 62.84	MIN 5680.00	AT 0900	MIN .46	UP	AM	6810.00	8270.00	5680.00	18900.00	16200.00	10600.00
	MEAN 63.03	MEAN 10600.00				PM	11400.00	6580.00	5600.00	8350.00	6620.00	7500.00
	(DATUM 0.00)	(SHIFT TO AREA	0.00)	(SHIFT TO VEL	0.00)	PM	6520.00	5760.00	6760.00	5780.00	5780.00	8420.00
PERIOD	MAX 64.16	MAX 60973.47		MAX 5.43								
	MIN 57.28	MIN		MIN								

FIGURE 278.—Primary computation sheet for deflection-meter station.

PROCESS DATE IS 02-27-7

UNITED STATES DEPARTMENT OF INTERIOR - GEOLOGICAL SURVEY

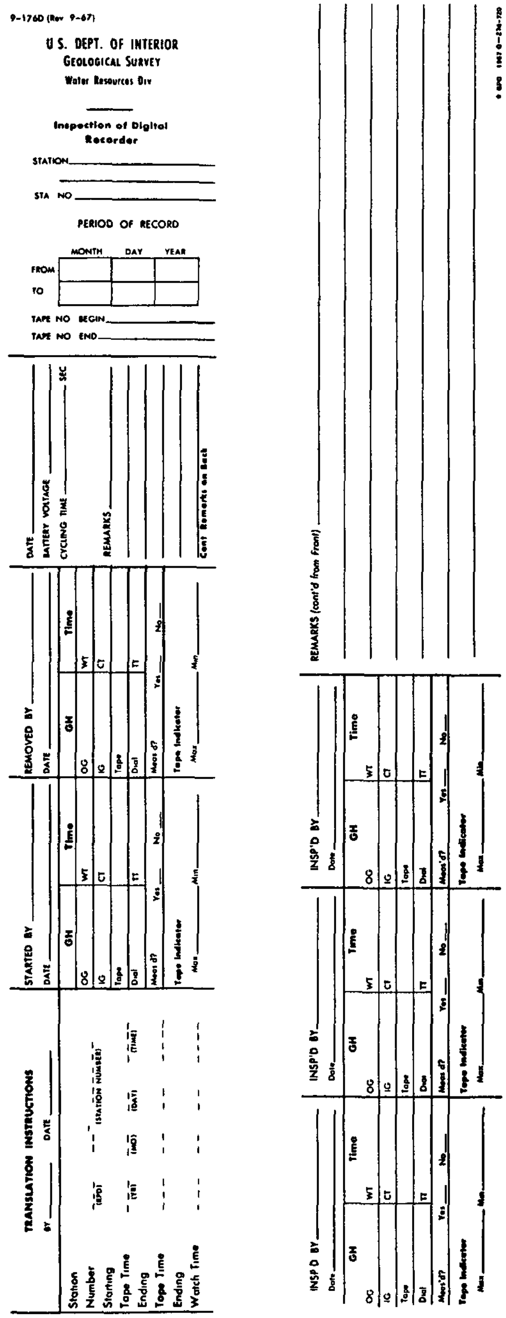
STATION NUMBER 11183600 WALNUT CREEK AT CONCORD, CALIF. STREAM SOURCE AGENCY USGS
LATITUDE 375643 LONGITUDE 1220255 DRAINAGE AREA 85.10 DATUM 35.44 STATE 06 COUNTY 013

DISCHARGE, IN CUBIC FEET PER SECOND, WATER YEAR OCTOBER 1974 TO SEPTEMBER 1975
MEAN VALUES

DAY	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP
1	12	16	9.2	9.7	116	27	45	30	18	13	12	9.8
2	12	17	23	10	170	32	40	29	19	12	13	9.8
3	11	9.2	98	9.7	121	25	44	31	20	13	13	9.9
4	9.7	17	44	10	170	24	184	30	21	13	13	11
5	9.7	8.6	14	10	32	38	272	26	19	14	13	11
6	9.2	8.1	12	86	23	42	130	26	19	13	11	11
7	9.2	20	11	23	61	177	170	24	18	14	13	10
8	9.7	18	10	90	55	127	354	24	18	13	13	11
9	9.7	10	11	19	293	44	110	23	17	13	13	11
10	9.2	17	10	15	201	133	81	23	17	13	12	10
11	10	9.7	11	14	48	52	71	22	16	13	13	11
12	10	10	11	12	71	35	65	23	16	12	13	9.3
13	9.2	9.7	11	12	574	439	59	23	16	13	13	9.3
14	9.2	8.1	11	12	84	157	57	22	15	12	13	10
15	9.7	8.1	10	12	45	142	71	23	14	17	13	9.6
16	10	8.1	11	12	34	227	59	23	17	14	12	9.9
17	10	8.1	10	12	27	81	50	21	16	14	12	10
18	27	8.6	9.2	12	25	63	45	22	15	14	15	9.6
19	11	8.6	9.2	12	47	55	44	22	14	13	14	9.5
20	14	8.1	9.2	12	48	50	42	19	14	13	13	12
21	19	32	9.2	12	25	1370	42	18	14	13	12	12
22	11	14	9.2	13	23	486	39	19	16	14	12	10
23	10	9.7	9.2	12	21	145	36	19	15	13	12	9.5
24	10	8.6	9.2	13	21	119	71	19	16	14	11	9.6
25	11	12	9.2	13	21	733	81	18	16	14	11	9.0
26	10	9.2	9.2	12	22	145	36	17	14	14	11	8.3
27	15	9.2	79	10	24	90	31	18	14	12	11	7.8
28	69	9.2	88	12	25	70	36	18	13	12	11	7.9
29	19	9.2	17	12	---	60	39	19	13	12	11	8.0
30	14	9.2	12	11	---	55	31	20	13	12	10	7.8
31	30	---	10	139	---	50	---	20	---	12	10	---
TOTAL	438.5	350.3	606.0	663.4	2427	5293	2435	691	483	408	381	294.6
MEAN	14.1	11.7	19.5	21.4	86.7	171	81.2	22.3	16.1	13.2	12.3	9.82
MAX	69	32	98	139	574	1370	354	31	21	17	15	12
MIN	9.2	8.1	9.2	9.7	21	24	31	17	13	12	10	7.8
AC-FT	870	695	1200	1320	4810	10500	4830	1370	958	809	756	584

CAL YR 1974 TOTAL 17259.8 MEAN 47.3 MAX 1450 MIN 8.1 AC-FT 34230
WTR YR 1975 TOTAL 14470.8 MEAN 39.6 MAX 1370 MIN 7.8 AC-FT 28700

FIGURE 279.—Printout of daily discharge.



9-176D (Rev 9-67)

U.S. DEPT. OF INTERIOR
GEOLOGICAL SURVEY
Water Resources Div

Inspection of Digital Recorder

STATION ______

STA NO ______

PERIOD OF RECORD

	MONTH	DAY	YEAR
FROM			
TO			

TAPE NO BEGIN ______
TAPE NO END ______

TRANSLATION INSTRUCTIONS
BY ______ DATE ______

Station
Number (EPD) (STATION NUMBER)
Starting Tape Time (YR) (MO) (DAY) (TIME)
Ending Tape Time
Ending Watch Time

STARTED BY ______ DATE ______
OG | GH | WT | Time
IG | CT
Tape
Dial | TT
Meas'd? Yes ___ No ___
Tape Indicator Max ___ Min ___

REMOVED BY ______ DATE ______
OG | GH | WT | Time
IG | CT
Tape
Dial | TT
Meas'd? Yes ___ No ___
Tape Indicator Max ___ Min ___

DATE ______
BATTERY VOLTAGE ______
CYCLING TIME ______ SEC
REMARKS ______
Cont' Remarks on Back

INSP'D BY ______ Date ______
OG | GH | WT | Time
IG | CT
Tape
Dial | TT
Meas'd? Yes ___ No ___
Tape Indicator Max ___ Min ___

INSP'D BY ______ Date ______
OG | GH | WT | Time
IG | CT
Tape
Dial | TT
Meas'd? Yes ___ No ___
Tape Indicator Max ___ Min ___

INSP'D BY ______ Date ______
OG | GH | WT | Time
IG | CT
Tape
Dial | TT
Meas'd? Yes ___ No ___
Tape Indicator Max ___ Min ___

REMARKS (cont'd from Front) ______

FIGURE 280.—Digital-recorder inspection form.

time during a day, and any shift applicable to the old rating on the same day will be dropped when the new rating takes effect.

The refinement considered in ratings for the initial run through the computer depends upon the complexity of the rating problem and the completeness of the data available. Sometimes final ratings can be prepared at the outset, at other times the output from the first run will be needed to complete the analysis. In the latter situation, only base ratings and approximate shift corrections are supplied.

Data from the 16-channel tape are translated by the central processing unit onto magnetic tape. The information on ratings is manually punched on cards. The magnetic tape and punch cards comprise the input to the digital computer. The rating table is stored on magnetic disk or tape at the computer center after the initial run.

4. The computer converts each instantaneous reading of river stage into a discharge value. Both daily mean discharge and daily mean gage height are computed as an average of instantaneous values. An equivalent daily mean gage height (the gage height corresponding to the daily mean discharge) is computed for each day so that recomputation, if necessary, can be made at a later date without reference to the individual items of the original base data. Daily mean values of gage height, discharge, and equivalent gage height are stored on a magnetic tape or disk. The printed output from the first computer pass consists of two items; a primary computation sheet, which is standard, and a daily discharge sheet, which is optional.

   The primary computation sheet (fig. 276) gives for each day the maximum, minimum, and mean gage heights, equivalent mean gage height, the datum and shift corrections applied, and the daily mean discharge. In addition, hourly gage heights and the time when maximums occurred are printed out.

   The printout of daily discharges (fig. 279), which is suitable for outside distribution, lists daily mean discharges for the period from the beginning of the water year to the end of the record being computed.

5. The field offices use the primary computation sheet in quality checks of the original and computed data, in further analysis of the stage-discharge relation, and in selecting instantaneous peak discharges to be published. Daily discharges from this sheet can be plotted for comparison with adjacent streams, and the usual studies can be made for periods of ice effect, no gage-height record, or backwater from various sources. Estimates can be made for all anomalous periods, and ratings can be revised, if

necessary, so that daily discharge can be recomputed from the effective mean gage heights on the second pass through the computer. The information necessary for revision or recomputation is forwarded to the Automatic Data Processing Unit.

6. The final tabulation is the same as figure 279 except that it is complete for the year and is produced on the second pass (update) through the computer. Where rating changes have been made as a result of the quality control analysis or where individual discharge figures have been estimated, the recomputation will involve substituting the estimated figures on the magnetic storage record, recomputing other discharge figures from revised ratings and the equivalent daily mean gage height, and printing out of the final discharge figures. (A printout from the subprogram for updating the primary computation sheet is shown in figure 281.) The printout from the final computer update is for the complete year. The format of the output is suitable for direct offset reproduction. The data on this form are also stored on magnetic tape for permanent storage.
7. A tabulation of daily mean gage heights may also be printed out during the second computer pass for stations designated by the field offices. That tabulation is prepared only for those stations for which there is a specific need.
8. The documentation file in the field offices consists of the original measurement notes, the 16-channel tapes, a station analysis, a list of discharge measurements, a rating curve, the primary computation sheet, a table of daily mean discharges from the final computer run, and possibly a rating table.

A record of progress of the discharge computations is kept in the field office on a check list such as that shown in figure 282. That form or perhaps a more detailed one, such as figure 283, is especially necessary because of the complication caused by records being shuttled back and forth between the field office and the computer center.

It is also necessary that a station-analysis document be prepared in the field office, as described for the graphic recorder in the section titled, "Station-Analysis Document." In that description it was mentioned that the "Discharge" paragraph showing the ratings used during the water year would be more complex for a digital-recorder station than for a graphic-recorder station. For a digital-recorder station, it is necessary to explain the origin of figures shown on the primary computation sheet as well as those on the final print-out. Documentation received on updating computer runs should therefore be referred to in the "Discharge" paragraph. A sample table of ratings used for a digital-recorder station having a somewhat complicated rating problem follows:

```
            UNITED STATES DEPARTMENT OF INTERIOR - GEOLOGICAL SURVEY - WATER RESOURCES DIVISION               DIST 17
                             UPDATE BY SUBSTITUTION OR RECOMPUTATION OF DAILY VALUES
03337000                                   DATA PROCESSED 02-11-74
BONEYARD CREEK AT URBANA, ILL
PROVISIONAL DATA FOR WATER YEAR ENDING SEPT. 30, 1973                                  STORE PARM 00065, STATISTIC 00003

                              DAILY GAGE HEIGHTS SUBSTITUTED(EQUIV GH SET THE SAME)
                                         9-20-73 TO  9-22-73
                                         9-25-73 TO  9-27-73

                                     STAGE-SHIFT VARIATION DIAGRAM
                                  HIGH POINT    2.00 SHIFT AT GH  5.00
                                  BASE          1.00 SHIFT AT GH  2.00
                                  LOW POINT     0.10 SHIFT AT GH  0.20

                                       SHIFT ADJUSTMENTS
                                         ADJ     DATE
                                         .09  09-13-73  AT GH   .55 PROJECTED TO   0.33 AT BASE GH
                                        -.02  10-01-73  AT GH   .65 PROJECTED TO  -0.06 AT BASE GH

                                     DAILY DISCHARGE RECOMPUTED
                                          USE RT 07
                                      09-18-73 TO 09-30-73
                                     (SHIFTS VARIED WITH STAGE)
```

FIGURE 281.—Printout from subprogram for updating primary computation sheet.

PROGRESS CHECK LIST
COMPUTATION OF DIGITAL RECORDER RECORD

Station______________________________ Water Year________.

Index Number____________________

1 2 3 4 Check work done. Complete in order. Initial when finished.

Examine and prepare tapes for transmittal.
List measurements on 9-207 (fig. 262).
Plot measurements on rating curve. Develop new curve and table, if needed.
Compute shift corrections, percentage difference on 9-207 (fig. 262).
Enter shift or datum corrections on 9-1536 (fig. 275).
Write preliminary rating analysis.

Computed______________ Checked______________

....

Tape transmitted.
Ratings transmitted.
9-1536 transmitted (fig. 275)

....

Inspect primary computation sheet.
Check measurements and field notes for peak data. Enter on PC sheets.
Revise shifts and recompute daily discharges on primary computation sheet.
Plot hydrograph.
Estimate discharge for ice, missing, doubtful or backwater periods.
Complete daily discharges monthly totals on primary computation sheets.
Complete station analysis.

Computed______________ Checked______________

....

Transmit updating corrections.

....

Enter notes, maximum, minimum and peaks on 9-211m (fig. 279).
Revise manuscript from previous year.

Computed______________ Checked______________

FIGURE 282.—Form showing progress of computation of digital-recorder record (sample 1).

"*Discharge.*—Computed as follows:

Ratings Used

Period	*Primary*	*Final*	*Update*
Oct. 1 to Jan. 4	No. 4	No change	None
Jan. 5 to Jan. 20	———	Special	May 12, 1973
Jan. 21 to Jan. 29	No. 4	No. 5	May 12, 1973
Jan. 30 to Feb 20	No. 4	Special	May 12, 1973
Feb. 21 to Aug. 10	No. 5	No. 5	None
Aug. 11 to Sept. 30	———	Special	Oct. 20, 1973"

All other instructions on the preparation of the station-analysis documents (p. 580–585) are applicable for a digital-recorder station.

SELECTED REFERENCES

Carter, R. W., and Davidian, Jacob, 1968, General procedure for gaging streams: U.S. Geol. Survey Techniques Water Resources Inv., book 3, chap. A6, p. 12–13.

```
WRD-ID-11
Jan 76

                    19__ water year A.D.P. check list

      Station No.____________
      Station Name__________________________________________________________

      List of measurements:
              Gage-heights . . . . . . . . . . . checked (_____)
              Original list of measurements. . checked (_____)
              List of measurements . . . . . . checked (_____)
              Shifts O.K. as submitted . . . . yes ( ) no ( )
              Shifts . . . . . . . . . . . . . . updated (_____ checked (_____)

      Rating curve:
              Measurements . . . . . . . . . . . plotted (_____) checked (_____)
              New curve needed . . . . . . . . . yes ( ) no ( )
              Measurements . . . . . . . . . . . plotted (_____) checked (_____)
              Curve. . . . . . . . . . . . . . . drawn (_____) checked (_____)
              Rating table . . . . . . . . . . . computed (_____) checked (_____)

      Primary sheets:
              Record complete. . . . . . . . . . yes ( ) no ( )
              Primary shifts O.K. . . . . . . . yes ( ) no ( )
              Primary datum corr. O.K. . . . . yes ( ) no ( )
              Correct rating in use. . . . . . yes ( ) no ( )
              Rating changed during year . . . yes ( ) no ( )
              Missing record . . . . . . . . . . estimated (_____) checked (_____)
              Ice period . . . . . . . . . . . . estimated (_____) checked (_____)
              Shift. . . . . . . . . . . . . . . update (_____) checked (_____)
              Datum. . . . . . . . . . . . . . . update (_____) checked (_____)
              Discharge. . . . . . . . . . . . . update (_____) checked (_____)
              Re-update. . . . . . . . . . . . . yes ( ) checked (_____)
              Re-update. . . . . . . . . . . . . yes ( ) checked (_____)

      Station analysis:
              Written by_______________ checked by_____________
              Reviewed by_______________

      Discharge table:
              Two copies . . . . . . . . . . . . yes ( ) no ( )
              Left margin attached . . . . . . yes ( ) no ( )
              Extremes . . . . . . . . . . . . . computed (_____) checked (_____)
              Supplemental peaks  . . . . . . (_____) checked (_____)
              Footnotes. . . . . . . . . . . . . (_____) checked (_____)
              Table annotated. . . . . . . . . (_____) checked (_____)

      Manuscript:
              Mean flow. . . . . . . . . . . . . computed (_____) checked (_____)
              Sheet updated with current data. (_____) checked (_____)
              Historical data changed. . . . . yes ( ) no ( )
              Footnotes. . . . . . . . . . . . . updated (_____) checked (_____)
              Skeleton rating. . . . . . . . . (_____) checked (_____)
```

FIGURE 283.—Form showing progress of computation of digital-recorder record (sample 2).

Carter, R. W., and others, 1963, Automation of streamflow records: U.S. Geol. Survey Circular 474, 18 p.

Corbett, D. M., and others, 1943, Stream-gaging procedure: U.S. Geol. Survey Water-Supply Paper 888, 245 p.

Edwards, M. D., and others, 1974, National water data storage and retrieval system; processing digital recorder records: U.S. Geol. Survey open-file report, 139 p.

World Meteorological Organization, 1971, Machine processing of hydrometeorological data: WMO–no. 275, Technical Note no. 115, 79 p.

CHAPTER 16—PRESENTATION AND PUBLICATION OF STREAM-GAGING DATA

GENERAL

After the computations of the discharge records for a water year are completed, the records are reviewed by designated engineering personnel and are prepared for publication. The publication process in the U.S.A. usually involves photo-offset printing, and copy must therefore be put in final form for photographing. From the photographic copy, a plate is made for use in the offset printing process.

FORMAT

The published annual report consists of an introductory text, stream-gaging and reservoir station records, tabulations of discharge at partial-record stations and at miscellaneous sites, and an index. The publication format used by the Geological Survey is illustrated in the example pages in figures 284–303 at the end of this chapter. The items that are included in the annual publication are listed in figure 284, which is an example of the table of contents of the report.

In general, most of the figures are self-explanatory, but some require additional explanation. The 9 pages of figure 286 include the 12 items in the table of contents (fig. 284) that start with "Introduction" and end with "Selected references." The 12 items are shown as part of a single figure because they constitute the introductory text that is printed on continuing pages; that is, each item is not started on a fresh page. The map in figure 287 is optional; if the map scale required to show the State or region on a single page is so small that the stations plot in a confusing clutter, the map may be omitted in the annual discharge report. However, any summary reports that cover a period of years of record for the stations should include a map of suitable scale that is folded and placed in a pocket attached to the back cover of the report. The graph in figure 288 is associated with the section titled "Hydrologic Conditions," near the end of the introductory text.

Figures 289–294 show samples of streamflow and reservoir tabulations for the water year that would appear in the main body of the annual report. Figure 289 is a sample page for a routine gaging station. Figure 290 is a sample page for a gaging station whose flow is regulated by a reservoir. Because the flow is controlled, no tabulation is made of supplementary peak discharges (those greater than a given base discharge). In the monthly and annual summaries at the bottom of figure 290, additional figures are given for the mean discharge adjusted for change in reservoir contents. Figure 291 is a sample page for a reservoir showing daily contents along with a monthly tabulation of change in contents. Daily contents are published only for major reservoirs. More commonly only the month end contents and the monthly change in contents are published, as in figure 292. Where the river basin contains several large reservoirs for which only month end contents and monthly change in contents are to be published, a table, such as that shown in figure 293*A* is published for the entire group. A table of that kind would usually be the last table for the river basin. Figure 293*B* is a continuation sheet for a group of such reservoirs. If all the reservoirs in the basin were relatively small, the data for the group of reservoirs would be abridged to take the form shown in figure 294.

In figure 295, tables *A* and *B* illustrate the way in which the records would be published if the gaging station were originally established a short time before October 1, the starting date of the water year. Table *A* is for a station that was established on Sept. 10. The data for the last 20 days in September would be published with the data for the complete year that followed. The short table shown as Table *A* would precede the daily table for the complete year. Table *B* is for a station that was established on August 1. The short table for August and September would precede the daily table for the complete year. Table *C* in figure 295 is a sample of the daily table for a station on an ephemeral stream that has few days of flow during the water year.

Figure 296 shows a sample "Revisions" paragraph for a gaging station whose past records require extensive revision. The revisions paragraph is always the last paragraph of the station description, as in figure 292. (The symbols used in the revisions paragraph in figure 292 are explained in figure 286*F*.)

If a highly developed river basin has a system of storage and diversion facilities that is too complex to be adequately described in the "Remarks" paragraph of the individual gaging stations, a schematic diagram is provided showing the locations of the reservoirs and canals with respect to the gaging stations. Such a diagram is found in figure 297; the diagram usually precedes the first discharge record for the basin.

Figures 298 and 299 show sample discharge records for partial-record stations. Figure 298 lists low-flow discharge measurements at sites where one or more such measurements are systematically made each year. Figure 299 lists peak discharges for the year, and occasionally one or more smaller peak discharges at sites equipped with a crest-stage gage (see last section in chapter 4). The discharges corresponding to observed peak stages are obtained from a rating table based on indirect determinations of discharge, such as slope-area determinations (chap. 9). Figure 300 shows the results of discharge measurements made at miscellaneous sites for special studies of various types. Miscellaneous sites are sites other than those where complete records or partial records are obtained each water year.

Figures 301 and 302 show the results of discharge measurements made at miscellaneous sites for two types of studies that are common enough to be identified by a general title. Figure 301 gives the results of a seepage investigation where base flow is measured at intervals in a reach of stream channel; the contribution of intervening tributary flow and the depletion of flow in intervening diversion canals are also measured. The purpose of the study is to investigate water gains and losses resulting from seepage through the streambed and banks. Figure 302 shows the results of low-flow discharge measurements made at miscellaneous sites during a drought period for the purpose of appraising the regional availability of surface flow during periods of critically low runoff.

The last section of the annual discharge report is an alphabetical index; figure 303 is a sample of the first page of such an index. Entries are made in the index for each station or measurement site for which figures of discharge or reservoir storage are given. For each station equipped with a continuous-recording gage, the entry is made under both the stream name and the place name. In addition, entries in the index are made for each section of the introductory text, for each of the terms listed under "Definitions of terms and abbreviations," for each illustration, and for each station plotted on the graph of hydrologic conditions (fig. 288).

In the past, basic groundwater and water-quality data were published under separate covers. At present (1980) the reports incorporate, in a single volume, those data with the surface-water discharge information that was described on the preceding pages. A discussion of ground-water and water-quality data is, however, beyond the scope of this manual.

SELECTED REFERENCE

Hodges, E. B., Ham, C. B., and Anderson, B. A., 1973, Preparation of surface-water data reports: U.S. Geol. Survey Surface-Water Techniques, book 9, chap. 1, 145 p.

CONTENTS

Page

ILLUSTRATIONS

III

FIGURE 284.—Table of contents for annual published report.

GAGING STATIONS, IN DOWNSTREAM ORDER,
FOR WHICH RECORDS ARE PUBLISHED

Page

OHIO RIVER BASIN
OHIO RIVER:
GREAT MIAMI RIVER BASIN
Great Miami River:
Whitewater River near Alpine . . . .
East Fork Whitewater River at Richmond . . . .

* * * * * * *

Ohio River at Evansville . . . .
WABASH RIVER BASIN
Wabash River near New Corydon . . . .
Wabash River at Bluffton . . . .
Wabash River at Huntington . . . .
Little River near Huntington . . . .
Salamonie River at Portland . . . .

* * * * * * *

Tippecanoe River at Oswego . . . .
Indian Creek:
Little Indian Creek near Royal Center . . . .
Big Monon Creek near Francesville . . . .
Tippecanoe River near Monticello . . . .

* * * * * * *

ST. LAWRENCE RIVER BASIN
STREAMS TRIBUTARY TO LAKE MICHIGAN
Little Calumet River (western portion, head of Calumet River):
Hart ditch at Munster . . . .
Little Calumet River at Munster . . . .
Thorn Creek at Thornton, Ill . . . .

* * * * * * *

STREAMS TRIBUTARY TO LAKE ERIE
St. Joseph River (head of Maumee River) near Newville . . . .
St. Joseph River at Cedarville . . . .
Cedar Creek at Auburn . . . .

* * * * * * *

UPPER MISSISSIPPI RIVER BASIN
MISSISSIPPI RIVER:
ILLINOIS RIVER BASIN
Kankakee River (head of Illinois River) near North Liberty . . . .

* * * * * * *

FIGURE 285.—List of surface-water stations.

WATER RESOURCES DATA FOR INDIANA. 19XX

PART 1. SURFACE-WATER RECORDS

INTRODUCTION

Surface-water records for the 19XX water year for Indiana, including records of streamflow or reservoir storage at gaging stations, partial-record stations, and miscellaneous sites, are given in this report and their locations shown in figures ___, ___. Records for a few pertinent gaging stations in bordering States also are included. The records were collected and computed by the Water Resources Division of the U.S. Geological Survey under the direction of M. D. Hale, district chief. These data represent that portion of the National Water Data System collected by the U.S. Geological Survey and cooperating State and Federal agencies in Indiana.

Through September 30, 1960, the records of discharge and stage of streams and canals and contents and stage of lakes or reservoirs were published in an annual series of U. S. Geological Survey water-supply papers entitled "Surface Water Supply of the United States."

Beginning with the 1961 water year, surface-water records have been released by the Geological Survey in annual reports on a State-boundary basis. Distribution of these reports is limited; they are designed primarily for rapid release of data shortly after the end of the water year to meet local needs. The discharge and reservoir storage records for 1961-65 also will be published in a Geological Survey water-supply paper series entitled "Surface Water Supply of the United States 1961-65."

COOPERATION

The U.S. Geological Survey and organizations of the State of Indiana have had cooperative agreements for the systematic collection of surface-water records since 1930. Organizations that supplied data are acknowledged in station descriptions. Organizations that assisted in collecting data through cooperative agreement with the Survey are:

State Department of Natural Resources, J. E. Mitchell, director, through Bureau of Water and Mineral Resources, W. J. Andrews, deputy director.

State Highway Commission, R. F. Whitehead, chairman, M. L. Hayes, executive director, and F. L. Ashbaucher, chief engineer.

State Board of Health, A. C. Offutt, commissioner, and B. A. Pool, director and chief engineer.

Assistance in the form of funds or services was given by the Corps of Engineers, U. S. Army, in collecting records for 67 gaging stations published in this report.

FIGURE 286A.—Introductory text.

WATER RESOURCES DATA FOR INDIANA, 19XX

The following organizations aided in collecting records:

The city of Indianapolis, through its Board of Public Works and Sanitation and its Flood Control Board; cities of Anderson, Bloomington, Muncie, North Vernon, Richmond, and Jasper; Indianapolis Water Co.; Indianapolis Power and Light Co.; Public Service Co. of Indiana; * * *.

DEFINITION OF TERMS

Definition of terms related to streamflow and other hydrologic data, as used in this report, are defined as follows:

Acre-foot (AC-FT, acre-ft) is the quantity of water required to cover 1 acre to a depth of 1 foot and is equivalent to 43,560 cubic feet or 325,851 gallons.

Cfs-day is the volume of water represented by a flow of 1 cubic foot per second for 24 hours. It is equivalent to 86,400 cubic feet, 1.9835 acre-feet, or 646,317 gallons, and represents a runoff of 0.0372 inch from 1 square mile.

Contents is the volume of water in a reservoir or lake. Unless otherwise indicated, volume is computed on the basis of a level pool and does not include bank storage.

Control designates a feature downstream from the gage that determines the stage-discharge relation at the gage. This feature may be a natural constriction of the channel, an artificial structure, or a uniform cross section over a long reach of the channel.

Cubic feet per second per square mile (CFSM) is the average number of cubic feet of water flowing per second from each square mile of area drained, assuming that the runoff is distributed uniformly in time and area.

Cubic foot per second (cfs) is the rate of discharge representing a volume of 1 cubic foot passing a given point during 1 second, and is equivalent to 7.48 gallons per second or 448.8 gallons per minute.

Discharge is the volume of water (or more broadly, total fluids), that passes a given point within a given period of time.

Drainage area of a stream at a specified location is that area, measured in a horizontal plane, enclosed by a topographic divide from which direct surface runoff from precipitation normally drains by gravity into the stream above the specified point. Figures of drainage area given herein include all closed basins, or noncontributing areas, within the area unless otherwise noted.

Gage height (G.H.) is the water-surface elevation referred to some arbitrary gage datum. Gage height is often used interchangeably with the more general term "stage," although gage height is more appropriate when used with a reading on a gage.

Gaging station is a particular site on a stream, canal, lake, or reservoir where systematic observations of gage height or discharge are obtained. When used in connection with a discharge record, the term is applied only to those gaging stations where a continuous record of discharge is obtained.

FIGURE 286B.—Introductory text—Continued.

WATER RESOURCES DATA FOR INDIANA, 19XX

Partial-record station is a particular site where limited streamflow data are collected systematically over a period of years for use in hydrologic analyses.

Runoff in inches (IN.) shows the depth to which the drainage area would be covered if all the runoff for a given time period were uniformly distributed on it.

Stage-discharge relation is the relation between gage height and the amount of water flowing in a channel, expressed as volume per unit of time.

WRD is used as an abbreviation for "Water-Resources Data" in the summary REVISIONS paragraph to refer to previously published State annual basic-data reports.

WSP is used as an abbreviation for "Water-Supply Paper" in references to previously published reports.

SPECIAL NETWORKS AND PROGRAMS

Hydrologic bench-mark station is one that provides hydrologic data for a basin in which the hydrologic regimen will likely be governed solely by natural conditions. Data collected at a bench-mark station may be used to separate effects of natural from man-made changes in other basins which have been developed and in which the physiography, climate, and geology are similar to those in the undeveloped bench-mark basin.

International Hydrological Decade (IHD) River Stations provide a general index of runoff and materials in the water balance (discharge of water, and dissolved and transported solids) of the world. In the United States, IHD Stations provide indices of runoff and of the general distribution of water in the principal river basins of the conterminous United States and Alaska.

DOWNSTREAM ORDER AND STATION NUMBERS

Records are listed in a downstream direction along the main stream, and stations on tributaries are listed between stations on the main stream in the order in which those tributaries enter the main stream. Stations on tributaries entering above all mainstream stations are listed before the first mainstream station. Stations on tributaries to tributaries are listed in a similar manner. In the list of gaging stations in the front of this report the rank of tributaries is indicated by indention, each indention representing one rank.

As an added means of identification, each gaging station and partial-record station has been assigned a station number. These are in the same downstream order used in this report. In assigning station numbers, no distinction is made between partial-record stations and continuous-record gaging stations; therefore, the station number for a partial-record station indicates downstream order position in a list made up of both types of stations. Gaps are left in the numbers to allow for new stations that may be established; hence the numbers are not consecutive. The complete 8-digit number for each station, such as 03-3355.00, includes the part number "03" and a 6-digit station number. In this report, the nonessential zeros are not shown. For example, the complete number 03-3355.00 would appear as 3-3355, just to the left of the station name. In this report,

FIGURE 286C.—Introductory text—Continued.

WATER RESOURCES DATA FOR INDIANA, 19XX

the records are listed in downstream order by parts. All records for a drainage basin encompassing more than one State could be arranged in downstream order by assembling pages from the various State reports by station number to include all records in the basin.

EXPLANATION OF SURFACE-WATER DATA

Collection and Computation of Data

The base data collected at gaging stations consists of records of stage and measurements of discharge of streams or canals, and stage, surface area, and contents of lakes or reservoirs. In addition, observations of factors affecting the stage-discharge relation or the stage-capacity relation, weather records, and other information are used to supplement base data in determining the daily flow or volume of water in storage. Records of stage are obtained from a water-stage recorder that gives a continuous graph of the fluctuations (for digital recorders, a tape punched at 15-, 30-, or 60-minute intervals) or from direct readings on a nonrecording gage. Measurements of discharge are made with a current meter, using the general methods adopted by the Geological Survey on the basis of experience in stream gaging since 1888. These methods are described in standard textbooks on the measurement of stream discharge. (See also SELECTED REFERENCES.) Surface areas of lakes or reservoirs are determined from instrument surveys using standard methods. The configuration of the reservoir bottom is determined by sounding at many points.

For a stream-gaging station rating tables giving the discharge for any stage are prepared from stage-discharge relation curves defined by discharge measurements. If extensions to the rating curves are necessary to define the extremes of discharge, they are made on the basis of indirect measurements of peak discharge (such as slope-area or contracted-opening measurements, computation of flow over dams or weirs), velocity-area studies, and logarithmic plotting. The application of the daily mean gage heights to the rating table gives the daily mean discharge, from which the monthly and the yearly mean discharge.are computed. If the stage-discharge relation is subject to change because of frequent or continual change in the physical features that form the control, the daily mean discharge is determined by the shifting-control method, in which correction factors based on individual discharge measurements and notes by engineers and observers are used in applying the gage heights to the rating tables. If the stage-discharge relation for a station is temporarily changed by the presence of aquatic growth or debris on the control, the daily mean discharge is computed by what is basically the shifting-control method.

At some stream-gaging stations the stage-discharge relation is affected by backwater from reservoirs, tributary streams, or other sources. This necessitates the use of the slope method in which the slope or fall in a reach of the stream is a factor in determining discharge. Information required for determining the slope or fall is obtained by means of an auxiliary gage set at some distance from the base gage. At some stations the stage-discharge relation is affected by changing stage; at these stations the rate of change in stage is used as a factor in determining discharge.

At some stream-gaging stations the stage-discharge relation is affected by ice in the winter, and it becomes impossible to compute the discharge in the usual manner. Discharge for periods of ice effect is computed on the basis of the gage-height record and

FIGURE 286*D*.—Introductory text—Continued.

WATER RESOURCES DATA FOR INDIANA, 19XX

occasional winter discharge measurements, consideration being given to the available information on temperature and precipitation, notes by gage observers and hydrologists, and comparable records of discharge for other stations in the same or nearby basins.

For a lake or reservoir station, capacity tables giving the contents for any stage are prepared from stage-area relation curves defined by surveys. Discharge over spillways is computed from a stage-discharge relation curve defined by discharge measurements. The application of the stage to the capacity table gives the contents, from which the daily, monthly, or yearly change in contents is computed.

If the stage-capacity curve is subject to changes because of deposition of sediment in the reservoir, periodic resurveys of the reservoir are necessary to define new stage-capacity curves. During the period between reservoir surveys the computed contents may be increasingly in error due to the gradual accumulation of sediment.

For some gaging stations there are periods when no gage-height record is obtained or the recorded gage height is so faulty that it cannot be used to compute daily discharge or contents. This happens when the recorder stops or otherwise fails to operate properly, intakes are plugged, the float is frozen in the well, or for various other reasons. For such periods the daily discharges are estimated on the basis of recorded range in stage, adjoining good record, discharge measurements, weather records, and comparison with other station records from the same or nearby basins. Likewise daily contents may be estimated on the basis of operator's log, adjoining good record, inflow-outflow studies, and other information.

The data in this report generally comprise a description of the station and tabulations of basic data. For gaging stations on streams or canals a table showing the daily discharge and monthly and yearly discharge is given. For gaging stations on lakes and reservoirs a monthly summary table of stage and contents or a table showing the daily contents is given. Tables of daily mean gage heights are included for some streamflow stations and for some reservoir stations. Records are published for the water year, which begins on October 1 and ends on September 30. A calendar for the 19XX water year is shown on the reverse side of the front cover to facilitate finding the day of the week for any date.

The description of the gaging station gives the location, drainage area, period of record, type and history of gages, average discharge, extremes of discharge or contents, and general remarks. The location of the gaging station and the drainage area are obtained from the most accurate maps available. River mileage, given under "LOCATION" for some stations, is that determined and used by the Corps of Engineers or other agencies. Periods for which there are published records for the present station or for stations generally equivalent to the present one are given under "PERIOD OF RECORD." The type of gage currently in use, the datum of the present gage above mean sea level, and a condensed history of the types, locations, and datums of previous gages used during the period of record are given under "GAGE." In references to datum of gage, the phrase "mean sea level" denotes "Sea Level Datum of 1929" as used by the Topographic Division of the Geological Survey, unless otherwise qualified. The average discharge for the number of years indicated is given under "AVERAGE DISCHARGE"; it is not given for stations having fewer than 5 complete years of record or for stations where changes in water development during the period of record cause the figure to have little significance. In addition, the median of yearly mean discharges is given for stream-gaging stations having 10 or more complete years of record if the median differs from the average by more than 10 percent. The maximum discharge (or contents) and the maximum gage height, the minimum discharge if there is little or no regulation (or the minimum contents), and the

FIGURE 286*E*.—Introductory text—Continued.

WATER RESOURCES DATA FOR INDIANA, 19XX

minimum gage height if it is significant are given under "EXTREMES." The minimum daily discharge is given if there is extensive regulation (also the minimum discharge and gage height if they are abnormally low). In the first paragraph headed "Current year:" the data given are for the complete current water year unless otherwise specified. In the second paragraph under "EXTREMES" headed "Period of record:" the data given are for the period of record given in the PERIOD OF RECORD paragraph. Reliable information concerning major floods that occurred outside the period of record is given in the third or last paragraph under "EXTREMES." Unless otherwise qualified, the maximum discharge (or contents) corresponds to the crest stage obtained by use of a water-stage recorder (graphic or digital), a crest-stage gage, or a nonrecording gage read at the time of the crest. If the maximum gage height did not occur at the same time as the maximum discharge or contents, it is given separately. Information pertaining to the accuracy of the discharge records, to conditions that affect the natural flow at the gaging station, and availability of Water Quality records, is given under "REMARKS"; for reservoir stations information on the dam forming the reservoir, the capacity, outlet works and spillway, and purpose and use of the reservoir, is also given under "REMARKS."

Previously published records of some stations have been found to be in error on the basis of data or information later obtained. Revisions of such records are usually published along with the current records in one of the annual or compilation reports. In order to make it easier to find such revised records, a paragraph headed "REVISIONS (WATER YEARS)" has been added to the description of all stations for which revised records have been published. Listed therein are all the reports in which revisions have been published, each followed by the water years for which figures are revised in that report. In listing the water years only one number is given; for instance, 1933 stands for the water year October 1, 1932, to September 30, 1933. If no daily, monthly, or annual figures of discharge were revised, that fact is brought out by notations after the year dates as follows: "(M)" means that only the instantaneous maximum discharge was revised; "(m)" that only the instantaneous minimum was revised; and "(P)" that only peak discharges were revised. If the drainage area has been revised, the report in which the revised figure was first published is given. It should be noted that for all stations for which cubic feet per second per square mile and runoff in inches are published, a revision of the drainage area necessitates corresponding revision of all figures based on the drainage area. Revised figures of cubic feet per second per square mile and runoff in inches resulting from a revision of the drainage area only are usually not published in the annual series of reports.

Skeleton rating tables are published for stream-gaging stations where they serve a useful purpose and the dates of applicability can be easily identified.

Skeleton capacity tables are published for all reservoirs for which records of contents are published on a daily basis.

The daily tables for stream-gaging stations give the discharge corresponding to the daily mean gage height unless there are large or rapid changes in the discharge during a day. For days having large or rapid changes, discharge for the day is computed by averaging the mean discharge for several parts of a day. For digital recorders, the daily mean discharge is always the average of the discharges at each punched reading. For stations equipped with nonrecording gages, the daily discharge corresponds to once-daily readings of the gage or to the mean of twice-daily readings; but for periods of rapidly changing stage the discharge is determined from a gage-height graph based on gage readings.

FIGURE 286F.—Introductory text—Continued.

WATER RESOURCES DATA FOR INDIANA, 19XX

The daily tables for reservoir stations give the contents corresponding to the water-surface elevation at a given time, usually at 2400 each day. For some reservoirs the elevation at a given time is given in the daily table.

The monthly summary is given below the daily table. For stream-gaging stations the line headed "TOTAL" gives the sum of the daily figures; it is the total cubic feet per second per day for the month. The line headed "MEAN" gives the average flow in cubic feet per second during the month. The lines headed "MAX" and "MIN" give the maximum and minimum daily discharges, respectively, for the month. Discharge for the month also may be expressed in cubic feet per second per square mile (line headed "CFSM"), or in inches (line headed "IN.") or in acre-feet (line headed "AC-FT"). Figures of cubic feet per second per square mile and runoff in inches are omitted if there is extensive regulation or diversion, if the drainage area includes large noncontributing areas, or if the average rainfall on the drainage basin is usually less than 20 inches.

For reservoir stations the monthly summary gives the elevation (or gage height) at the end of the month and the change in contents during the month. If elevation or gage height is given in the daily table, the monthly summary gives the contents at the end of the month, rather than the elevation or gage height. For some reservoirs a tabulation of monthly evaporation from the water surface also is included.

In the yearly summary below the monthly summary, the figures of maximum are the maximum daily discharges for the calendar and water years; likewise, the minimums in this summary are the minimum daily discharges.

For reservoir stations the yearly summary gives the change in contents for the calendar year and for the water year. For some reservoirs the yearly evaporation also is included.

Peak discharges and their times of occurrence and corresponding gage heights for many stations are listed below the yearly summary. All independent peaks above the selected base are given. The base discharge, which is given in parentheses, is selected so that an average of about three peaks a year can be presented. Peak discharges are not published for any canals, ditches, drains, or for any stream for which the peaks are subject to substantial control by man. Time of day is expressed in 24-hour local standard time; for example, 12:30 a.m. is 0030 and 1:30 p.m. is 1330.

In a general footnote, introduced by the word "NOTE" certain periods are indicated for which the discharge is computed or estimated by special methods because of no gage-height record, backwater from various sources, or other unusual conditions. Periods of no gage-height record are indicated if the period is continuous for a month or more or includes the maximum discharge for the year. Periods of backwater from an unusual source, of indefinite stage-discharge relation, or of any other unusual condition at the gage are indicated only if they are a month or more in length and the accuracy of the records is affected. Days on which the stage-discharge relation is affected by ice are not indicated. The methods used in computing discharge for various unusual conditions have been explained in preceding paragraphs. Footnotes to reservoir tables may be used to explain the use of new capacity tables or for other special conditions.

FIGURE 286G.—Introductory text—Continued.

WATER RESOURCES DATA FOR INDIANA, 19XX

Accuracy of Data

The accuracy of discharge data depends primarily on (1) the stability of the stage-discharge relation or, if the control is unstable, the frequency of discharge measurements, and (2) the accuracy of observations of stage, measurements of discharge, and interpretation of records.

The station description under "REMARKS" states the degree of accuracy of the records. "Excellent" means that about 95 percent of the daily discharges is within 5 percent; "good" within 10 percent; and "fair" within 15 percent. "Poor" means that daily discharges have less than "fair" accuracy.

Figures of daily mean discharge in this report are shown to the nearest hundredth of a cubic foot per second for discharges of less than 1 cfs; to tenths between 1.0 and 10 cfs; to whole numbers between 10 and 1,000 cfs; and to 3 significant figures above 1,000 cfs. The number of significant figures used is based solely on the magnitude of the figure. The same rounding rules apply to discharge figures listed for partial-record stations and miscellaneous sites.

Discharge at many stations, as indicated by the monthly mean, may not reflect natural runoff due to the effects of diversion, consumptive use, regulation, evaporation, or other factors. For such stations, discharge in cubic feet per second per square mile and runoff in inches are not published unless satisfactory adjustments can be made for such effects. Evaporation from a reservoir is not included in the adjustments for changes in reservoir contents, unless it is so stated. Even at those stations where adjustments are made, large errors in computed runoff may occur if adjustments or unadjusted losses (consumptive use, evaporation, seepage, etc.) are large in comparison with the observed discharge.

Publications

Each volume of the 1960 series of U.S. Geological Survey water-supply papers entitled "Surface Water Supply of the United States" contains a listing of the numbers of all water-supply papers in which records of surface-water data were published for the area covered by the individual volumes. Each volume also contains a list of water-supply papers that give detailed information on major floods for the area. A new series of water-supply papers containing surface-water records for the 5-year period October 1, 1960, to September 30, 1965, also will include lists of annual and special reports published as water-supply papers.

Records through September 1950 for the area covered by this report have been compiled and published in Water-Supply Papers 1305(3A), 1307(4), and 1308(5); records for October 1950 to September 1960 have been compiled and published in Water-Supply Papers 1725(3A), 1727(4), and 1728(5). These reports contain summaries of monthly and annual discharge and monthend storage for all previously published records, as well as some records not contained in the annual series of water-supply papers. All records were reexamined and revised where warranted. Estimates of discharge were made to fill short gaps whenever practical. The yearly summary table for each gaging station lists the numbers of the water-supply papers in which daily records were published for that station.

Special reports on major floods or droughts or of other hydrologic studies for the area have been issued in publications other than water-supply papers. Information relative to these reports may be obtained from the district office.

FIGURE 286*H*.—Introductory text—Continued.

WATER RESOURCES DATA FOR INDIANA, 19XX

Other Data Available

Data collected at partial-record stations and at miscellaneous sites are given in three tables at the end of the surface-water records in this report. The first is a table of discharge measurements at low-flow partial-record stations, the second is a table of annual maximum stage and discharge at crest-stage stations, and the third is a table of discharge measurements at miscellaneous sites.

More detailed information than that published for most of the gaging stations, such as discharge measurements, gage-height records, and rating tables, is on file in the district office. Many gaging-station records in (State) through (1966) have been analyzed to give several statistical summaries: (1) the number of days in each year that the daily discharge was between selected limits (duration tables); (2) the lowest mean discharge for selected numbers of consecutive days in each year; and (3) the highest mean discharge for selected numbers of consecutive days in each year.

At or near some gaging stations, water-quality records also are collected. Data are obtained on the chemical quality of the stream water, on water temperature, on suspended-sediment concentration, and on the particle-size distribution of suspended sediment and bed material. These data are given in Part 2 of this report. Under the "REMARKS" paragraph of the gaging-station description, reference is made to water-quality records collected on a regular basis.

HYDROLOGIC CONDITIONS

Precipitation was scattered throughout the year by area and time. Heavy rains the first half of December caused minor flooding in the Wabash and Maumee River basins. Lack of late summer showers left the central and southern parts 3 to 9 inches below average rainfall.

Deficient streamflow in October was relieved in the south by mid-November and in the north by the end of the month. Excessive to near excessive streamflow existed in the first part of December with near record streamflow in the upper Wabash River and Maumee River basins. Near normal streamflow existed from January to May with generally bankfull stages in March and May. Deficient * * *. (to be completed)

SELECTED REFERENCES

Carter, R. W., and Davidian, Jacob, 1968, General procedure for gaging streams: U. S. Geol. Survey Techniques Water-Resources Inv., book 3, chap. A6, 13 p.

Corbett, D. M., and others, 1943, Stream-gaging procedure, a manual describing methods and practices of the Geological Survey: U. S. Geol. Survey Water-Supply Paper 888, 245 p.

Langbein, W. B., and Iseri, K. T., 1960, General introduction and hydrologic definitions: U. S. Geol. Survey Water-Supply Paper 1541-A, 29 p.

FIGURE 286 *I*.—Introductory text—Continued.

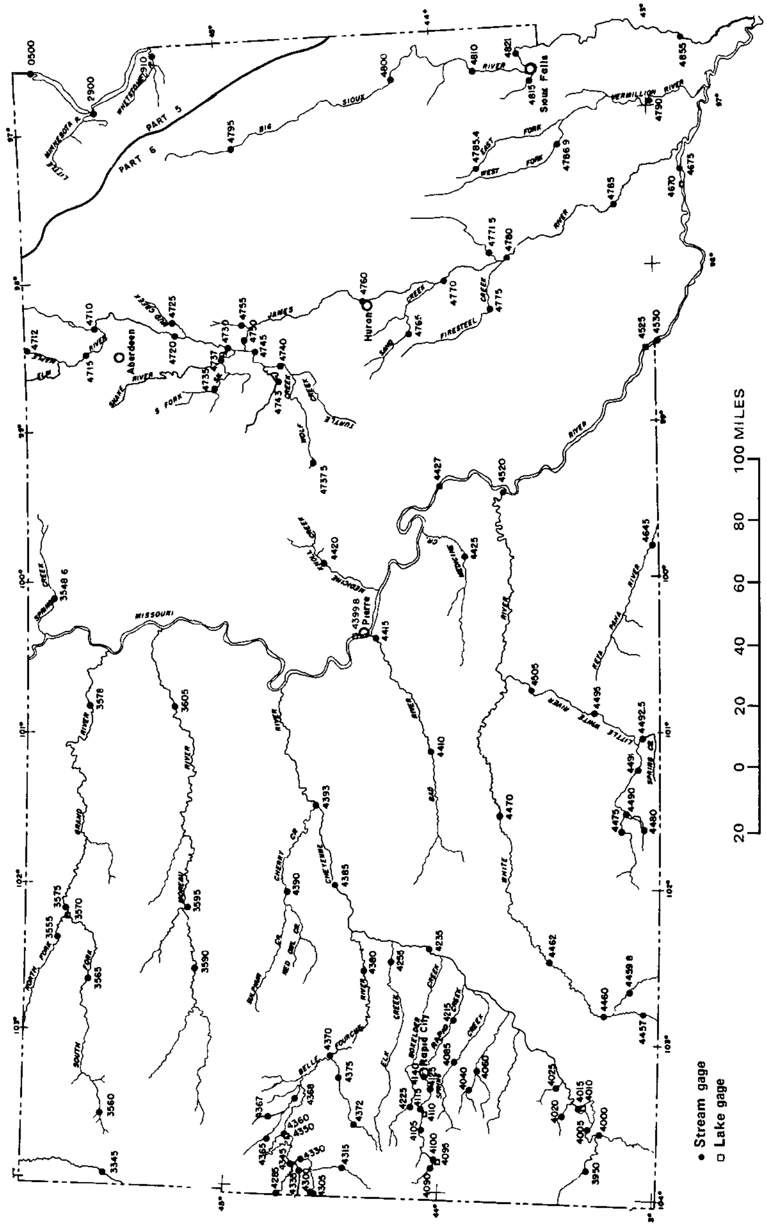


FIGURE 287.—Map of gaging-station locations.

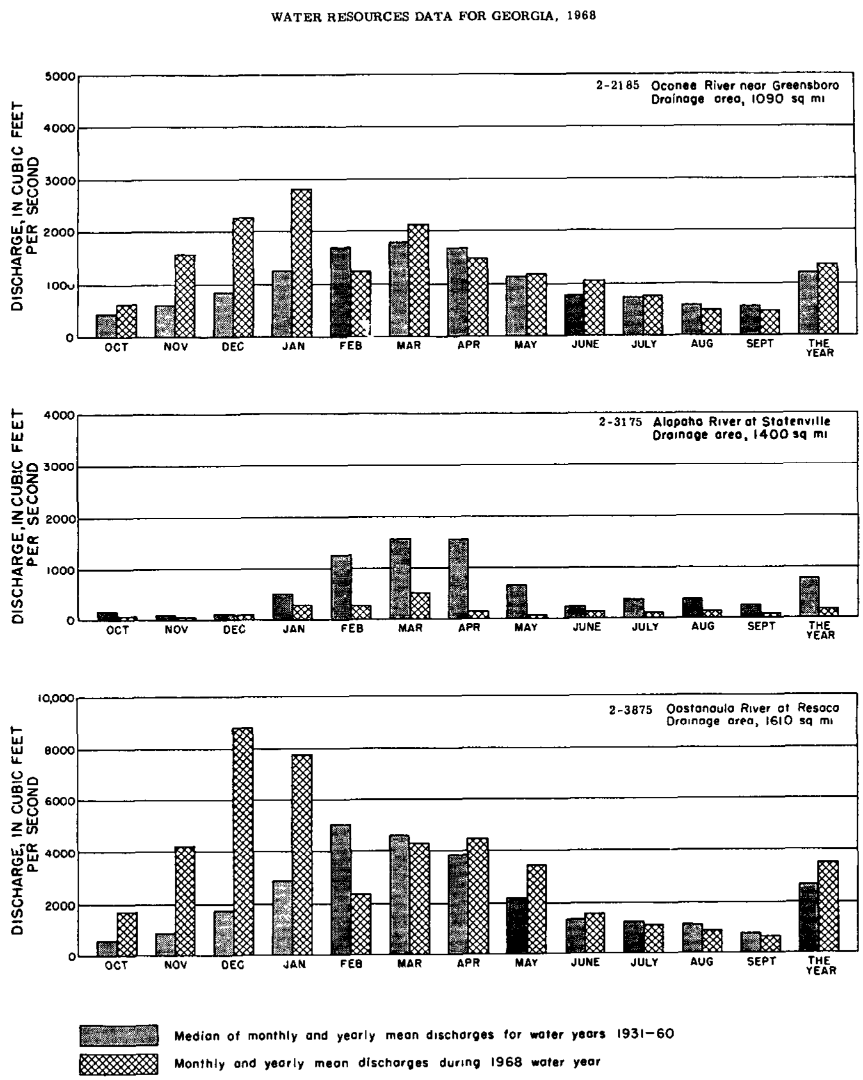


FIGURE 288.—Bar graph of hydrologic conditions.

SKUNK RIVER BASIN

5-4710. South Skunk River below Squaw Creek, near Ames, Iowa

LOCATION.--Lat 42°00'31", long 93°35'38", in NE¼NW¼ sec.13, T.83 N., R.24 W., Story County, on right bank 15 ft downstream from county highway bridge, 0.2 mile downstream from Squaw Creek, 0.3 mile upstream from bridge on U.S. Highway 30, 2 miles southeast of Ames, and at mile 222.6.

DRAINAGE AREA.--556 sq mi.

PERIOD OF RECORD.--October 1952 to current year. Prior to October 1966, published as Skunk River below Squaw Creek, near Ames.

GAGE.--Water-stage recorder and concrete control. Datum of gage is 867.10 ft above mean sea level.

AVERAGE DISCHARGE.--16 years, 226 cfs (5.52 inches per year, 163,600 acre-ft per year).

EXTREMES.--Current year: Maximum discharge, 7,310 cfs June 25 (gage height, 12.07 ft); no flow for many days.
Period of record: Maximum discharge, 9,260 cfs Mar. 30, 1960 (gage height, 13.20 ft); no flow for many days during 1953-57, 1964-65, 1967-68.
Flood of May 19, 1944, reached a stage of 13 ft, from floodmarks (discharge, 10,000 cfs).

REMARKS.--Records good except those for winter periods and period of no gage-height record, which are poor.

DISCHARGE, IN CFS, WATER YEAR OCTOBER 1967 TO SEPTEMBER 1968

DAY	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP
1	16	17	2.3	0	10	0	52	68	48	847	64	13
2	17	12	2.5	0	8.6	0	52	62	40	592	56	11
3	19	9.8	2.7	0	7.4	0	25	58	35	449	50	18
4	18	7.2	2.9	0	6.2	0	29	52	26	352	50	118
5	17	6.2	3.1	0	6.2	0	17	48	22	284	52	83
6	1.5	5.2	3.2	0	7.2	0	12	44	19	252	48	58
7	2.4	36	6.2	0	39	0	11	48	17	271	44	56
8	5.4	3.6	57	0	6.2	7.8	9.2	48	16	216	95	40
9	5.2	3.2	6.7	0	2.2	76	7.8	44	26	182	103	28
10	4.8	3.6	78	0	2.0	62	78	40	94	135	92	23
11	4.3	39	7.2	0	18	52	6.7	38	493	114	70	19
12	4.0	36	7.8	0	16	52	5.7	36	129	101	60	16
13	4.6	3.6	4.3	0	.58	38	6.7	38	52	98	54	14
14	4.0	32	3.6	0	.22	26	23	48	519	90	46	11
15	3.4	3.2	3.2	0	0	19	11	44	272	79	42	9.2
16	28	2.8	28	0	0	18	9.2	46	111	75	38	8.5
17	2.4	2.8	3.6	0	0	19	11	46	60	77	33	17
18	2.1	2.6	3.2	0	0	17	26	42	48	353	40	14
19	1.8	2.8	2.8	0	0	18	30	44	42	424	35	14
20	1.6	28	3.6	0	0	15	70	42	36	241	28	12
21	2.2	2.8	20	0	0	13	62	38	31	173	24	10
22	2.2	2.6	1.0	0	0	92	60	36	26	128	19	9.2
23	2.2	2.0	.54	0	0	62	233	36	52	181	17	8.5
24	5.7	2.6	.28	0	0	6.2	480	33	735	158	14	7.8
25	6.7	2.8	.15	0	0	5.7	324	36	5.910	107	15	6.2
26	6.7	2.6	0	0	0	5.2	216	52	3.600	90	21	5.7
27	6.2	2.5	0	0	0	6.2	153	52	2.000	157	16	4.7
28	5.7	24	0	2.0	0	57	107	65	1.330	161	13	4.7
29	10	2.3	0	18	0	5.7	85	92	1.980	103	12	47
30	12	2.2	0	7.2	------	57	75	54	1.440	83	15	38
31	15	------	0	6.7	------	14	------	54	------	72	15	------
TOTAL	133.6	127.5	89.17	33.9	64.10	502.6	2,123.5	1,484	19,209	6,645	1,281	724.5
MEAN	4.31	4.25	2.88	1.09	2.21	16.2	70.8	47.9	640	214	41.3	24.2
MAX	15	17	7.8	18	10	76	480	92	5,910	847	103	118
MIN	1.5	2.0	0	0	0	0	5.2	33	16	72	12	4.7
CFSM	.008	.008	.005	.002	.004	.03	.13	.09	1.15	.38	.07	.04
IN.	.009	.009	.006	.002	.004	.03	.14	.10	1.28	.44	.09	.05
AC-FT	265	253	177	67	127	997	4,210	2,940	38,100	13,180	2,540	1,440

CAL YR 1967: TOTAL 47,968.01 MEAN 131 MAX 4,070 MIN 0 CFSM .24 IN. 3.21 AC-FT 95,140
WTR YR 1968: TOTAL 32,417.87 MEAN 88.6 MAX 5,910 MIN 0 CFSM .16 IN. 2.17 AC-FT 64,300

PEAK DISCHARGE (BASE, 2,500 CFS)

NOTE.--No gage-height record Oct. 6 to Nov. 20.

DATE	TIME	G.H.	DISCHARGE	DATE	TIME	G.H.	DISCHARGE
6-25	1700	12.07	7,310	6-29	1500	6.71	2,550

FIGURE 289.—Daily discharge record.

MOBILE RIVER BASIN

2-3940. Etowah River at Allatoona Dam, above Cartersville, Ga.

LOCATION.--Lat 34°09'48", long 84°44'30", Bartow County, on right bank 0.8 mile downstream from Allatoona Dam, 2.0 miles upstream from Nashville, Chattahooga and St. Louis Railway bridge, and 3.0 miles east of Cartersville.

DRAINAGE AREA.--1,110 sq mi, approximately.

PERIOD OF RECORD.--September 1938 to current year. Prior to October 1949, published as Etowah River above Cartersville.

GAGE.--Water-stage recorder. Datum of gage is 686.92 ft above sea level (levels by Corps of Engineers). Prior to Dec. 19, 1938, nonrecording gage at same site and datum.

AVERAGE DISCHARGE.--30 years, 1,800 cfs (22.02 inches per year), adjusted for storage since 1950.

EXTREMES.--Current year: Maximum discharge, 9,340 cfs Jan. 18 (gage height, 7.65 ft); minimum daily, 213 cfs Jan. 27, 28, Feb. 3, 4.
Period of record: Maximum discharge, 40,400 cfs Jan. 8, 1946 (gage height, 20.8 ft), from rating curve extended above 26,000 cfs; minimum daily, 152 cfs Oct. 15, 1966.

REMARKS.--Flow regulated by Allatoona Reservoir since December 1949 (See sta 2-3935.)

COOPERATION.--Gage-height record, 19 discharge measurements, and computations of daily discharge furnished by Corps of Engineers; records reviewed by Geological Survey.

DISCHARGE, IN CFS, WATER YEAR OCTOBER 1967 TO SEPTEMBER 1968

DAY	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP
1	220	3,630	5,430	5,200	5,190	855	1,510	3,820	227	2,130	992	289
2	2,650	3,580	280	5,240	5,230	227	1,160	3,820	227	2,140	939	289
3	2,650	3,570	220	5,250	213	234	1,500	3,820	2,540	2,140	256	950
4	2,650	264	5,300	5,250	213	1,480	1,500	272	2,540	234	248	950
5	2,650	264	5,490	5,240	1,860	1,470	1,500	272	2,560	2,140	1,600	950
6	2,650	5,330	5,540	220	1,840	1,480	3,810	2,440	2,550	234	1,600	950
7	234	3,020	5,570	220	1,840	1,480	272	2,440	2,550	227	1,630	280
8	234	2,850	5,570	4,980	1,840	1,480	5,120	2,440	234	1,220	1,630	289
9	1,390	2,860	234	4,980	1,840	234	5,180	2,440	234	1,220	1,630	1,680
10	1,390	2,910	227	2,590	220	234	5,180	2,440	1,780	1,220	272	1,660
11	1,390	2,590	5,180	1,340	220	1,270	5,230	256	2,910	1,220	280	1,660
12	1,390	1,120	5,540	1,340	2,430	1,270	5,230	264	2,910	1,220	1,470	1,660
13	1,390	5,280	5,580	1,510	2,430	2,710	272	1,480	2,910	234	1,480	1,660
14	248	5,310	5,600	1,500	2,490	2,710	272	1,490	2,910	234	1,480	289
15	248	5,330	5,580	5,600	2,460	2,710	4,040	1,490	241	1,510	1,480	289
16	1,780	5,380	220	5,940	2,460	5,170	4,040	5,460	234	1,510	1,480	1,680
17	1,780	5,400	220	7,140	220	1,860	4,040	5,460	1,710	1,510	280	1,700
18	1,780	241	5,020	7,370	220	5,820	4,040	248	1,700	1,510	280	1,660
19	1,820	234	5,050	7,530	1,290	5,530	4,040	241	1,700	1,510	1,470	1,660
20	1,860	4,580	5,050	5,390	1,290	5,530	280	3,890	1,710	241	1,480	1,660
21	256	4,620	5,560	1,890	1,290	5,530	280	3,890	1,700	241	1,470	289
22	256	4,640	1,500	5,730	1,290	5,590	1,030	3,910	227	1,640	1,780	289
23	2,190	241	794	5,750	1,290	256	1,030	3,890	227	1,640	1,470	1,780
24	2,190	4,620	1,690	5,780	227	248	1,030	3,890	1,030	1,640	280	1,810
25	2,210	241	2,380	5,780	220	4,900	1,030	241	1,030	1,640	280	1,810
26	2,210	234	4,970	5,780	842	4,930	1,030	248	1,030	1,640	879	1,810
27	2,190	5,280	5,610	213	842	4,930	280	929	1,030	248	867	1,810
28	256	5,340	5,610	213	849	4,930	280	2,710	1,030	248	574	272
29	248	5,360	5,610	5,100	855	5,010	1,680	2,730	227	992	867	272
30	3,580	5,400	4,880	5,120	------	256	2,820	2,730	227	932	886	1,610
31	3,600	------	1,750	5,140	------	256	------	2,730	------	932	289	------
TOTAL	49,590	99,719	117,255	130,326	43,501	80,590	68,706	72,381	42,135	35,397	31,619	33,957
MEAN	1,600	3,324	3,782	4,204	1,500	2,600	2,290	2,335	1,405	1,142	1,020	1,132
MAX	3,600	5,400	5,610	7,530	5,230	5,820	5,230	5,460	2,910	2,140	1,780	1,810
MIN	220	234	220	213	213	227	272	241	227	227	248	272
MEAN†	1,197	2,524	3,327	4,362	2,169	3,317	3,198	2,366	1,415	1,079	680	840
CFSM†	1.08	2.27	3.00	3.93	1.95	2.99	2.88	2.13	1.27	.97	.61	.76
IN.†	1.24	2.53	3.46	4.53	2.10	3.45	3.21	2.46	1.42	1.12	.70	.85

CAL YR 1967: TOTAL 799,710 MAX 7,930 MIN 192 MEAN 2,191 MEAN† 2,230 CFSM† 2.01 IN.† 27.28
WTR YR 1968: TOTAL 805,176 MAX 7,530 MIN 213 MEAN 2,200 MEAN† 2,209 CFSM† 1.99 IN.† 27.09

† Adjusted for change in contents in Allatoona Reservoir.

FIGURE 290.—Daily discharge record (adjusted).

ARKANSAS RIVER BASIN

7-2590. Blue Mountain Reservoir near Waveland, Ark.

LOCATION.--Lat 35°06'06", long 93°39'02", in NE¼ sec.15, T 5 N., R 25 W., Yell County, at control tower for outlet works of dam on Petit Jean Creek, 1.3 miles southwest of Waveland

DRAINAGE AREA.--488 sq mi.

PERIOD OF RECORD.--March 1947 to current year.

GAGE --Water-stage recorder Datum of gage is at mean sea level (levels by Corps of Engineers)

EXTREMES --Current year. Maximum contents, 130,940 acre-ft May 23 (elevation, 405 37 ft); minimum, 2,150 acre-ft Oct 2, 3 (elevation, 370.68 ft).
Period of record: Maximum contents, 298,560 acre-ft May 26, 19XX (elevation, 422.54 ft), including 40,560 acre-ft of uncontrolled storage above spillway crest; minimum since initial filling to conservation pool level, 2,150 acre-ft Oct. 2, 3, 19XX (elevation, 370.68 ft).

REMARKS.--Reservoir is formed by earthfill dam, storage began Mar 13, 1947. Total capacity, 258,000 acre-ft at elevation 419.0 ft (crest of uncontrolled spillway), including 25,000 acre-ft at elevation 384.0 ft (conservation pool level), and 1,940 acre-ft of dead storage at elevation 370.00 ft (invert of gate sills of outlet tunnel). Under normal operating conditions reservoir water surface will be maintained at approximately conservation pool level for purposes of conservation and recreation, storage above this level is used for flood control. Figures given herein represent total contents.

COOPERATION.--Records furnished by Corps of Engineers.

Capacity table (elevation, in feet, and contents, in acre-feet)

370	1,940	390	44,810
371	2,250	395	66,370
372	2,920	400	93,620
375	5,680	405	128,140
380	14,400	406	135,700
385	27,700		

CONTENTS, IN ACRE-FEET, AT 2400, WATER YEAR OCTOBER 19XX TO SEPTEMBER 19XX

DAY	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP
1	2,160	2,380	2,460	15,000	3,250	26,810	29,480	29,940	104,570	28,620	26,230	23,220
2	2,150	2,380	2,450	12,440	3,250	26,810	29,320	29,910	101,410	28,470	25,930	23,110
3	2,920	2,450	2,430	10,020	3,310	26,750	29,200	29,850	97,420	28,250	25,650	23,050
4	5,580	13,190	2,420	7,340	5,050	26,660	29,320	29,970	92,740	28,560	25,560	23,020
5	6,400	15,780	2,420	5,950	8,780	26,320	29,510	33,560	88,610	28,680	25,440	22,970
6	5,750	14,140	2,420	5,120	11,620	25,860	29,720	51,800	84,600	28,430	25,280	22,890
7	4,580	11,340	2,420	4,680	13,300	25,310	29,910	57,390	80,830	28,130	25,220	22,830
8	3,450	8,740	2,440	4,740	14,730	25,310	30,030	57,780	77,100	27,910	25,160	22,750
9	3,080	6,400	2,450	5,160	16,140	26,110	29,910	56,340	72,680	27,790	25,100	22,800
10	2,850	4,940	2,580	5,420	17,020	26,600	29,850	53,850	68,350	27,610	25,040	22,720
11	2,580	3,680	5,310	5,420	17,680	26,690	29,850	51,040	64,010	27,490	24,950	22,670
12	2,420	3,050	8,290	5,310	18,230	26,320	29,750	47,680	59,750	27,360	24,790	22,480
13	2,850	2,750	9,290	5,450	18,710	26,050	29,720	44,190	55,940	27,700	24,730	22,370
14	2,990	2,580	8,430	9,620	19,090	28,190	29,720	40,860	51,840	27,790	24,590	22,200
15	2,880	2,620	6,760	21,070	19,760	34,630	29,720	37,420	48,000	27,580	24,500	22,170
16	2,650	2,750	8,170	11,670	20,310	37,090	29,820	34,060	44,550	27,330	24,450	22,060
17	2,480	2,780	16,280	18,230	21,900	36,920	29,910	31,680	41,550	27,090	24,340	22,010
18	2,400	2,780	26,690	21,160	23,960	36,110	29,750	32,920	38,660	27,090	24,370	21,950
19	2,340	2,780	30,490	20,640	25,250	34,940	29,540	56,470	35,810	26,940	24,310	21,870
20	2,360	2,740	29,750	18,660	25,890	33,460	29,690	100,700	33,390	26,840	24,310	21,870
21	2,380	2,670	27,700	16,140	26,480	31,810	29,380	127,160	31,850	27,030	24,340	22,310
22	2,380	2,550	25,130	13,450	26,780	30,490	29,290	130,180	30,800	27,090	24,260	22,280
23	2,360	2,480	22,450	11,330	27,120	29,880	29,320	130,410	30,000	28,560	24,090	22,170
24	2,340	2,410	19,210	9,420	27,000	29,450	29,410	128,520	29,600	32,350	23,980	22,230
25	2,320	2,340	16,210	7,770	26,540	30,120	29,410	126,260	29,230	33,120	23,820	22,370
26	2,320	2,320	12,990	5,900	25,890	30,940	29,480	124,520	29,690	32,750	23,740	22,170
27	2,320	2,330	19,210	4,840	25,310	31,240	29,480	121,350	29,850	31,440	23,630	21,950
28	2,320	2,440	23,270	3,900	25,930	31,110	29,510	118,060	29,540	30,000	23,540	21,680
29	2,320	2,480	22,830	3,410	26,900	31,170	29,820	114,810	29,200	28,560	23,460	21,430
30	2,320	2,480	20,800	3,280	------	30,280	29,850	111,360	29,170	27,640	23,380	21,240
31	2,370	------	17,950	3,250	------	29,820	------	107,630	------	26,940	23,300	------
(†)	371.18	371.35	381.49	372.50	384.74	385 69	385.70	402.16	385.48	384.75	383.51	382.76
(‡)	+210	+110	+15,470	-14,700	+23,650	+2,920	+30	+77,780	-78,460	-2,230	-3,640	-2,060
MAX	6,400	15,780	30,490	21,160	27,120	37,090	30,030	130,410	104,570	33,120	26,230	23,220
MIN	2,150	2,320	2,420	3,250	3,250	25,310	29,200	29,850	29,170	26,840	23,300	21,240

CAL YR 19XX....... . . .. ‡ -7,240
WTR YR 19XX. . . .. ‡ +19,080

† Elevation, in feet, at end of month.
‡ Change in contents, in acre-feet.

FIGURE 291.—Daily reservoir record.

MISSISSIPPI RIVER MAIN STEM

5-2010. Winnibigoshish Lake near Deer River, Minn.

LOCATION.--Lat 47°25'42", long 94°03'00", in XX¼ sec.25, T 146 N., R.27 W., Itasca County, at dam on Mississippi River, 1 mile northwest of Little Winnibigoshish Lake and 14 miles northwest of town of Deer River.

DRAINAGE AREA --1,442 sq mi

PERIOD OF RECORD.--April 1884 to current year (monthend contents only). Prior to October 1941 monthend contents published in WSP 1308. Published as Winnibigoshish Reservoir near Deer River October 1941 to September 1956.

GAGE --Water-stage recorder. Datum of gage is 1,289.47 ft above mean sea level. Prior to July 8, 1949, nonrecording gage at same site and datum

EXTREMES.--Current year: Maximum contents, 561,880 acre-ft July 29 (gage height, 9.95 ft); minimum, 466,570 acre-ft Apr. 1 (gage height, 8.50 ft).
Period of record· Maximum contents observed, 996,500 acre-ft July 30, 1905 (gage height, 14.45 ft), minimum observed, 33,680 acre-ft below zero of capacity table Oct. 20, 1931 (gage height, -0.69 ft).

REMARKS --Reservoir is formed by concrete and timber dam controlling Winnibigoshish Lake and several other natural lakes; dam completed and storage began in 1884. Capacity, 967,930 acre-ft between gage height 0 00 and 14.2 ft (maximum operating level), of which 653,580 acre-ft is usable storage above 6.00 ft (minimum allowable level). Figures given herein represent contents above gage height 0.00 ft. Water is used to benefit navigation on Mississippi River below Minneapolis.

COOPERATION.--Records furnished by Corps of Engineers in cfs-days and converted to acre-feet by Geological Survey.

REVISIONS (WATER YEARS).--WSP 1308: 1905(M).

MONTHEND GAGE HEIGHT AND CONTENTS AT 2400, WATER YEAR OCTOBER 19XX TO SEPTEMBER 19XX

Date	Gage height (feet)	Contents (acre-feet)	Change in contents (acre-feet)
Sept. 30	9.63	540,740	-
Oct. 31	9.51	532,880	-7,860
Nov. 30	9 31	519,750	-13,130
Dec. 31	9.28	517,780	-1,970
CAL YR 19XX	-	-	+106,510
Jan. 31	9.22	513,840	-3,940
Feb. 29	8.94	495,460	-18,380
Mar. 31	8.51	467,230	-28,230
Apr. 30	9 05	502,680	+35,450
May 31	9.37	523,690	+21,010
June 30	9.63	540,740	+17,050
July 31	9.80	551,900	+11,160
Aug. 31	9.21	513,190	-38,710
Sept 30	8.82	487,580	-25,610
WTR YR 19XX	-	-	-53,160

FIGURE 292.—Monthly reservoir record.

YAKIMA RIVER BASIN

Reservoirs in Yakima River basin

12-4740. KEECHELUS LAKE.--Lat 47°19'XX", long 121°20'XX", Kittitas County, at dam on Yakima River at outlet of Keechelus Lake, 3.X miles northwest of Martin and 10 miles northwest of Easton. Drainage area, 55.8 sq mi. Period of record, January 1906 to current year. *Nonrecording gage read twice daily. Datum of gage is at mean sea level* (Bureau of Reclamation bench mark). *Extremes for* current year: Maximum contents observed, 159,930 acre-ft June 17 (elevation, 2,517.84 ft); minimum observed, 33,560 acre-ft Sept. 30 (elevation, 2,450.97 ft). Extremes for period of record: Maximum contents observed, 160,570 acre-ft May 16, 1925 (elevation, 2,518.09 ft); minimum observed, 448 acre-ft Sept. 6, 12, 13, 1906 (original crib dam); minimum elevation observed, 2,428.30 ft Sept. 20, 1926.

Reservoir is formed on natural lake by earth- and gravel-fill dam completed in 1917; storage above crib dam began Jan. 12, 1906; above present dam Aug. *19, 1914. Initial filling of present reservoir June 15, 1920. Usable capacity,* 157,800 acre-ft between elevation 2,425 00 (invert of gate sill) and 2,517.00 ft (spillway crest). Dead storage below elevation 2,425.00 ft, XXX acre-ft. Spillway raised 2 ft Sept. 12, 1952. Figures given herein represent usable contents. Water is used for irrigation.

12-4755. KACHESS LAKE.-- * * *. (Format similar to the above)

12-4785. CLE ELUM LAKE.-- * * *. (Format similar to the above)

12-4875. BUMPING LAKE.--Lat 46°52'XX", long 121°18'XX", in SW¼ sec.23, T.16 N., R.12 E. (unsurveyed), Yakima County, at dam on Bumping River at outlet of Bumping Lake, 12 miles upstream from American River and 19 miles west of Nile. Drainage area, 68.6 sq mi. Period of record, June to July 1906, April 1909 to current year. Nonrecording gage read twice daily. Datum of gage is at mean sea level (Bureau of Reclamation bench mark). Extremes for current year: Maximum contents observed, 36,160 acre-ft June 16 (elevation, 3,427.85 ft); minimum observed, 4,980 acre-ft Sept. 30 (elevation, 3,396.67 ft). Extremes for period of record: Maximum contents observed, 39,840 acre-ft June 21, 22, 1925 (elevation, 3,430.55 ft); minimum observed, 1,130 acre-ft Feb. 5-9, 1949 (elevation, 3,390.80 ft).

Reservoir is formed on natural lake by earthfill dam completed in 1910; storage began Nov 3, 1910. Usable capacity, 33,700 acre-ft between elevation 3,389.00 (invert of gate sill) and 3,426.00 ft (spillway crest). No dead storage Figures given herein represent usable contents Water is used for irrigation.

MONTHEND ELEVATION AND CONTENTS AT 2400, WATER YEAR OCTOBER 19XX TO SEPTEMBER 19XX

Date	Elevation (feet)	Contents (acre-feet)	Change in contents (acre-feet)	Elevation (feet)	Contents (acre-feet)	Change in contents (acre-feet)
	12-4740. Keechelus Lake			12-4755. Kachess Lake		
Sept. 30	2,486.95	90,120	-	2,246.60	172,150	-
Oct. 31	2,493.45	103,090	+12,970	2,252 57	197,330	+25,180
Nov. 30	2,505.50	129,580	+26,490	2,259.10	225,930	+28,600
Dec 31	2,495.52	107,420	-22,160	2,252.02	194,970	-30,960
CAL YR 19XX	-	-	-15,540	-	-	+5,890
Jan. 31	2,482.77	82,300	-25,120	2,248.11	178,420	-16,550
Feb. 29	2,488.84	93,800	+11,500	2,250.63	189,030	+10,610
Mar. 31	2,496 23	108,920	+15,120	2,254.11	203,980	+14,950
Apr. 30	2,509.21	138,380	+29,460	2,260.20	230,860	+26,880
May 31	2,514.95	152,560	+14,180	2,260.39	231,710	+850
June 30	2,517.38	158,750	+6,190	2,261.88	238,440	+6,730
July 31	2,502.09	121,750	-37,000	2,250.93	190,310	-48,130
Aug. 31	2,473.38	66,130	-55,620	2,242 39	155,110	-35,200
Sept. 30	2,450 79	33,320	-32,810	2,237.37	135,660	-19,450
WTR YR 19XX	-	-	-56,800	-	-	-36,490
	12-4785. Cle Elum Lake			12-4875. Bumping Lake		
Sept. 30	2,181.55	192,540	-	3,404.42	10,700	-
Oct. 31	2,199.96	262,020	+69,480	3,400.70	7,820	-2,880
Nov. 30	2,226.81	375,470	+113,450	3,412.50	18,060	+10,240
Dec. 31	2,217 13	332,920	-42,550	3,403.55	10,000	-8,060
CAL YR 19XX	-	-	+102,930	-	-	-4,110
Jan. 31	2,202.74	273,140	-59,780	3,397.77	5,730	-4,270
Feb. 29	2,207.24	291,440	+18,300	3,400.18	7,440	+1,710
Mar. 31	2,215.62	326,460	+35,020	3,408.68	14,370	+6,930
Apr. 30	2,229.42	387,310	+60,850	3,413.32	18,900	+4,530
May 31	2,233.11	404,330	+17,020	3,423.87	30,980	+12,080
June 30	2,237.95	427,130	+22,800	3,427.31	35,430	+4,450
July 31	2,216.17	328,800	-98,330	3,410.90	16,470	-18,960
Aug. 31	2,187.53	214,330	-114,470	3,403.30	9,810	-6,660
Sept. 30	2,158.33	115,360	-98,970	3,396.60	4,930	-4,880
WTR YR 19XX	-	-	-77,180	-	-	-5,770

FIGURE 293A.—Group reservoir records (large reservoirs).

RIO GRANDE BASIN

Reservoirs in Rio Grande basin--Continued

MONTHEND ELEVATION OR GAGE HEIGHT AND CONTENTS, WATER YEAR OCTOBER 19XX TO SEPTEMBER 19XX

Date	Gage height (feet)	Contents (acre-feet)	Change in contents (acre-feet)	Elevation or gage height (feet)	Contents (acre-feet)	Change in contents (acre-feet)	Elevation or gage height (feet)	Contents (acre-feet)	Change in contents (acre-feet)
	8-3165. Nichols Reservoir†			8-3285. Jemez Canyon Reservoir†			8-3414. Bluewater Lake‡		
Sept. 30	142.3	168	-	0	0	-	7,371.2	5,510	-
Oct. 31	139.0	129	-39	5,143.2	753	+753	7,370.4	5,170	-340
Nov. 30	143.1	178	+49	0	0	-753	7,370.0	5,000	-170
Dec. 31	143.8	187	+9	0	0	0	7,369.7	4,880	-120
CAL YR 19XX	-	-	-349	-	-	0	-	-	-2,500
Jan 31	-	a228	+41	0	0	0	7,369.5	4,800	-80
Feb. 29	150.0	279	+51	0	0	0	7,369.4	4,760	-40
Mar. 31	157.7	438	+159	5,146.40	1,090	+1,090	7,381.4	12,020	+7,260
Apr. 30	167.5	701	+263	5,142.70	480	-610	7,381.0	11,700	-320
May 31	167.5	701	0	5,128.80	0	-480	7,380.4	11,240	-460
June 30	165 1	628	-73	0	0	0	7,379.5	10,560	-680
July 31	159.4	477	-151	0	0	0	7,378.5	9,820	-740
Aug. 31	154.2	360	-117	0	0	0	7,377.5	9,120	-700
Sept. 30	139.0	129	-231	0	0	0	7,376.7	8,580	-540
WTR YR 19XX	-	-	-39	-	-	0	-	-	+3,070
	8-3605. Elephant Butte Res.††			8-3620. Caballo Reservoir‡			8-3840. Alamogordo Reservoir‡		
Sept. 30	4,334.46	532,800	-	4,140.42	41,690	-	4,269.90	100,600	-
Oct. 31	4,333.30	517,500	-15,300	4,144.95	58,170	+16,480	4,269.80	100,200	-400
Nov. 30	4,335.70	549,300	+31,800	4,145.20	59,200	+1,030	4,270.40	102,600	+2,400
Dec. 31	4,338.40	586,400	+37,100	4,145.55	60,640	+1,440	4,271.90	108,600	+6,000
CAL YR 19XX	-	-	-402,400	-	-	-85,360	-	-	-15,100
Jan 31	4,338.50	587,800	+1,400	4,152.53	92,290	+31,650	4,273 20	114,100	+5,500
Feb. 29	4,338 50	587,800	0	4,157 71	125,850	+33,560	4,274.15	118,200	+4,100
Mar. 31	4,336.07	554,300	-33,500	4,152.24	92,340	-33,510	4,269.15	97,710	-20,490
Apr. 30	4,338 98	594,500	+40,200	4,156.60	118,400	+26,060	4,271.10	105,400	+7,690
May 31	4,336.90	565,700	-28,800	4,159.16	135,700	+17,300	4,264.10	79,540	-25,860
June 30	4,335.53	547,100	-18,600	4,157.49	124,300	-11,400	4,267.70	92,270	+12,730
July 31	4,330.10	476,400	-70,700	4,152.30	92,670	-31,630	4,274.90	121,600	+29,330
Aug. 31	4,324.61	410,100	-66,300	4,136.24	29,260	-63,410	4,274.65	120,500	-1,100
Sept. 30	4,322.42	385,100	-25,000	4,123 48	4,900	-24,360	4,273.95	117,400	-3,100
WTR YR 19XX	-	-	-147,700	-	-	-36,790	-	-	+16,800
	8-4005. Lake McMillan‡			8-4038. Lake Avalon‡			8-4100. Red Bluff Reservoir††		
Sept. 30	19 10	9,620	-	15.70	1,560	-	2,807.9	63,150	-
Oct. 31	18.30	7,330	-2,290	16.30	1,920	+360	-	a65,100	+1,950
Nov. 30	18.75	8,580	+1,250	17.40	2,620	+700	-	a69,100	+4,000
Dec 31	19.50	10,850	+2,270	18.20	3,200	+580	2,809.7	70,300	+1,200
CAL YR 19XX	-	-	-26,890	-	-	-2,060	-	-	-35,900
Jan 31	19 75	11,650	+800	15.75	1,590	-1,610	2,810.6	73,900	+3,600
Feb. 29	19.65	11,330	-320	16.10	1,800	+210	2,810.8	74,700	+800
Mar. 31	22.30	20,940	+9,610	14.75	1,070	-730	2,811.0	75,500	+800
Apr. 30	17.60	5,510	-15,430	15.40	1,400	+330	2,807.4	61,400	-14,100
May 31	19.75	11,650	+6,140	15.55	1,480	+80	2,805.5	54,750	-6,650
June 30	21.05	16,150	+4,500	14.75	1,070	-410	2,804.3	50,900	-3,850
July 31	25.95	38,580	+22,430	19.15	3,960	+2,890	2,815.4	96,000	+45,100
Aug. 31	24.15	29,140	-9,440	16.00	1,740	-2,220	-	a93,000	-3,000
Sept. 30	21.15	16,520	-12,620	15.70	1,560	-180	-	a85,400	-7,600
WTR YR 19XX	-	-	+6,900	-	-	0	-	-	+22,250

† Elevation or gage height at 2400.
‡ Elevation or gage height at 0800.
†† Daily mean gage height.
a Interpolated.

FIGURE 293 *B*. Group reservoir records (large reservoirs)—Continued.

ANDROSCOGGIN RIVER BASIN

Reservoirs in Androscoggin River basin

1-0505. RANGELEY LAKE on Rangeley Stream, at Oquossoc, Maine, used for power and log driving, has usable capacity of 1,339,200,000 cu ft in top 4 ft of lake (top of flashboards). Gage-height record furnished by Union Water Power Co.

1-0510. MOOSELOOKMEGUNTIC LAKE at Upper Dam, in Richardson Township, Maine, used for power and log driving, has usable capacity of 8,370,000,000 cu ft between gage heights 8.3 and 20.5 ft. Gage-height record furnished by Union Water Power Co.

1-0515. UPPER AND LOWER RICHARDSON LAKES on Rapid River, at Middle Dam, Maine, used for power and log driving, has usable capacity of 5,691,500,000 cu ft between gage heights 3.0 and 20.5 ft. Gage-height record furnished by Union Water Power Co.

1-0520. AZISCOHOS LAKE on Magalloway River, in Lincoln Township, 3 miles east of village of Wilsons Mills, Maine, completed in 1911 for power, has usable capacity of 9,593,000,000 cu ft between elevations 1,490.0 and 1,535.0 ft. Elevation record furnished by Union Water Power Co.

1-0530. UMBAGOG LAKE on Androscoggin River, at Errol Dam, 0.8 mile northeast of Errol, N.H., used for power and log driving, has usable capacity of 3,080,160,000 cu ft between gage heights 5.5 and 15.0 ft. Gage-height record furnished by Union Water Power C

1-0560. GULF ISLAND POND on Androscoggin River, 3 miles upstream from Lewiston, Maine, completed in 1928 for power, has capacity of 1,100,000,000 cu ft in top 10 ft of pond below elevation 262 ft. Elevation record furnished by Central Maine Power Co.

1-0565. LAKE AUBURN on outlet stream to Androscoggin River, at East Auburn, Maine, used for storing water supply of Auburn and Lewiston, has usable capacity of 580,000,000 cu ft between elevations 254.7 and 260.7 ft. .Elevation record furnished by Auburn Water District.

1-0575. PENNESSEEWASSEE LAKE on short outlet stream to Little Androscoggin River, at Norway, Maine, used for recreation, has usable capacity of 192,000,000 cu ft between gage heights 95.0 and 100.0 ft. Gage-height record furnished by Town of Norway.

1-0580. THOMPSON LAKE on short outlet stream to Little Androscoggin River, at Oxford, Maine, used for power and process water, has usable capacity of 950,000,000 cu ft between gage heights 95.0 and 100.0 ft. Gage-height record furnished by Robinson Manufacturing Co.

MONTHEND CONTENTS, IN MILLIONS OF CUBIC FEET, WATER YEAR OCTOBER 1967 TO SEPTEMBER 1968

Date	Rangeley Lake†	Mooselookmeguntic Lake†	Upper and Lower Richardson Lakes†	Aziscohos Lake†	Umbagog Lake†
Sept. 30, 1967	754	4,470	4,047	6,492	1,434
Oct. 31	837	3,076	3,862	6,926	1,668
Nov. 30	194	1,266	4,084	6,576	1,848
Dec. 31	84	534	4,047	6,688	1,920
Jan. 31, 1968	0	124	3,233	5,098	1,506
Feb. 29	0	0	2,540	3,938	850
Mar. 31	265	1,033	2,339	4,410	1,794
Apr. 30	1,366	6,474	4,918	8,830	3,138
May 31	1,366	8,154	5,691	9,702	2,300
June 30	1,366	8,370	5,691	9,872	3,119
July 31	1,185	7,724	5,350	9,253	1,958
Aug. 31	920	5,799	4,269	7,587	1,884
Sept. 30	656	4,368	3,270	6,394	1,578

Date	Gulf Island Pond‡	Lake Auburn†	Pennesseewassee Lake††	Thompson Lake‡‡
Sept. 30, 1967	2,316	386	98	1,913
Oct. 31	2,416	408	107	1,799
Nov. 30	2,195	386	111	1,742
Dec. 31	2,296	408	116	1,780
Jan. 31, 1968	2,199	430	111	1,723
Feb. 29	2,080	430	111	1,685
Mar. 31	2,500	604	111	1,970
Apr. 30	2,495	664	116	2,065
May 31	2,487	640	98	2,046
June 30	2,497	640	78	2,046
July 31	2,199	556	59	1,932
Aug. 31	2,211	474	66	1,818
Sept. 30	2,370	419	74	1,742

† Contents at 0700 on first day of following month.
‡ Contents at 2400.
†† Contents as of last day of month, by interpolation.
‡‡ Contents at 0800.

FIGURE 294.—Group reservoir records (small reservoirs).

"A"

DISCHARGE, IN CUBIC FEET PER SECOND, 19XX

Sept 10	4.6	Sept. 21	5.3
11	4.4	22	6.5
12	4.6	23	6.5
13	4.7	24	6.7
14	4.9	25	6.8
15	4.7	26	7.1
16	4.4	27	10
17	4.4	28	11
18	4.4	29	11
19	4.4	30	11
20	4.7		

[Daily table for complete year follows.]

"B"

DISCHARGE, IN CUBIC FEET PER SECOND, 19XX

DAY	AUG	SEP	DAY	AUG	SEP	DAY	AUG	SEP	DAY	AUG	SEP	DAY	AUG	SEP
1	6.9	5.4	7	9.3	5.4	13	6.9	4.1	19	6.4	3.1	25	4.9	3.4
2	6.4	4.9	8	12	5.4	14	6.4	3.8	20	5.9	3.8	26	4.1	3.8
3	6.4	4.9	9	12	6.4	15	8.6	3.4	21	5.9	3.4	27	4.5	3.8
4	6.4	4.9	10	10	4.5	16	11	2.8	22	6.4	3.4	28	4.5	3.8
5	7.4	4.9	11	17	4.1	17	6.9	2.3	23	6.4	3.1	29	4.5	3.8
6	9.3	4.9	12	8.0	4.1	18	6.9	2.8	24	5.4	3.4	30	4.5	3.4
												31	5.4	
Total													226.6	121.2
Max													17	6.4
Min													4 1	2.3
Mean													7.31	4.04
Runoff in acre-feet													449	240

PEAK DISCHARGE (BASE, 50 CFS).--No peak above base.

[Daily table for complete year follows.]

"C"

DISCHARGE, IN CUBIC FEET PER SECOND, FEBRUARY 1961 TO SEPTEMBER 1965

July 3, 1961	1.9	Nov. 7, 1963	0
July 30, 1961	1.5	Nov. 21, 1963	2
Aug. 18, 1961	3.4	Aug. 12, 1964	
Nov. 21, 1961	6	Sept. 14, 1964	1
Aug. 17, 1963	8	Aug. 17, 1965	
Oct. 19, 1963	.3		

Month	Cfs-days	Maximum	Minimum	Mean	Runoff acre-fe
July 1961	3.4	1.9	0	0.11	6.7
August	3.4	3.4	0	.11	6.7
November	.6	.6	0	.02	1.2
WTR YR 1962	.6	.6	0	.002	1.2
CAL YR 1962	0	0	0	0	0
August 1963	.8	.8	0	.03	1.6
WTR YR 1963	.8	.8	0	.002	1.6
October 1963	.3	.3	0	.01	.6
November	2.9	2.6	0	.10	5.8
CAL YR 1963	4.0	2.6	0	.01	8 0
August 1964	.5	.5	0	.02	1 0
September	1.0	1.0	0	.03	2.0
WTR YR 1964	4.7	2.6	0	.01	9.4
CAL YR 1964	1.5	1.0	0	.004	3.0
August 1965	.1	.1	0	.003	.2
WTR YR 1965	.1	.1	0	.0003	.2

PEAK DISCHARGE (BASE, 20 CFS).--July 3, 1961 (2130) 113 cfs (3.36 ft); July 30, 1961 (1500) 90 cfs (3.02 ft); Aug. 18, 1961 (2000) 179 cfs (4.28 ft); Nov. 21, 1961 (1030) 39 cfs (2.16 ft); Aug. 17, 1963 (1030) 46 cfs (2.28 ft); Oct. 19, 1963 (0700) 25 cfs (1.93 ft); Nov. 7, 1963 (1430) 24 cfs (1.91 ft); Nov. 21, 1963 (1300) 61 cfs (2.53 ft); Sept. 14, 1964 (about 2000) 74 cfs (2.75 ft).

NOTE.--Flow occurred only on days listed above.

FIGURE 295.—Discharge tables for short periods.

REVISIONS (WATER YEARS) --WSP 1277: 1939-42. Revised figures of discharge, in cubic feet per second, for the water years 1963-65, superseding those published in WRD XXXX, 1963-65, are given herewith

Date	Discharge	Date	Discharge	Date	Discharge
1962		1963-Con.		1965	
Dec. 28	137	July 22	157	Feb. 15	164
29	137	29	505	16	239
30	267			22	464
31	122	1964		26	228
		Mar. 6	276	Mar. 3	408
1963		7	258	6	997
Mar. 11	137	23	184	20	552
*	*	* *	*	*	*
20	323	Apr. 12	206	June 18	676

Month	Cfs-days	Maximum	Minimum	Mean	Per square mile	Runoff in inches
December 1962.......	1,676	267	13	54.1	1.45	1.67
CAL YR 1962.......	X,XXX.X	XXX	X.X	XX.X	.XX	XX.XX
March 1963..........	3,137	891	16	101	2 71	3.12
* *	*	*		*	*	*
CAL YR 1965.... ..	14,970 6	1,150	5 9	41 0	1.10	14.95

FIGURE 296.—Revisions of published records.

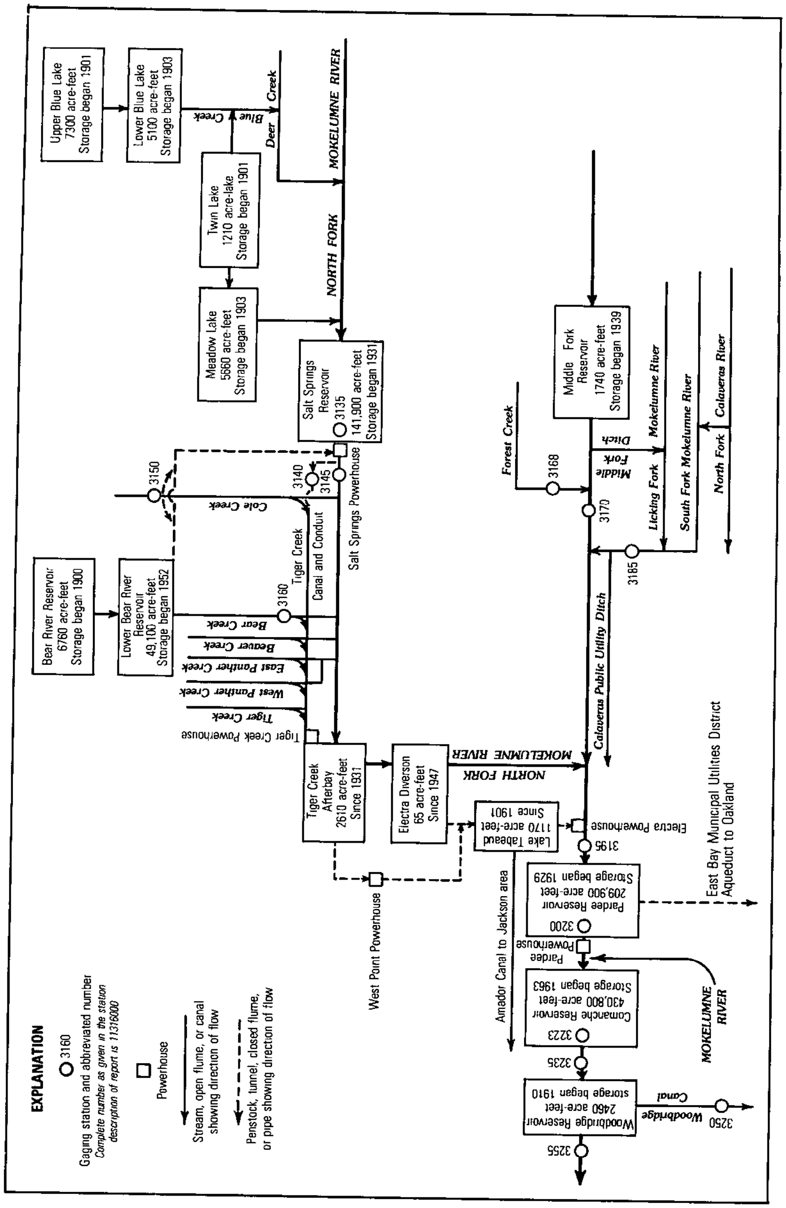


FIGURE 297.—Schematic diagram showing reservoirs, canals, and gaging stations.

DISCHARGE AT PARTIAL-RECORD STATIONS AND MISCELLANEOUS SITES

As the number of streams on which streamflow information is likely to be desired far exceeds the number of stream-gaging stations feasible to operage at one time, the Geological Survey collects limited streamflow data at sites other than stream-gaging stations. When limited streamflow data are collected on a systematic basis over a period of years for use in hydrologic analyses, the site at which the data are collected is called a partial-record station. Data collected at these partial-record stations are usable in low-flow or floodflow analyses, depending on the type of data collected. In addition, discharge measurements are made at other sites not included in the partial-record program. These measurements are generally made in times of drought or flood to give better areal coverage to those events. Those measurements and others collected for some special reason are called measurements at miscellaneous sites.

Records collected at partial-record stations are presented in two tables. The first is a table of discharge measurements at low-flow partial-record stations, and the second is a table of annual maximum stage and discharge at crest-stage stations. Discharge measurements made at miscellaneous sites for both low flow and high flow are given in a third table.

Low-flow partial-record stations

Measurements of streamflow in the area covered by this report made at low-flow partial-record stations are given in the following table. Most of these measurements were made during periods of base flow when streamflow is primarily from ground-water storage. These measurements, when correlated with the simultaneous discharge of a nearby stream where continuous records are available, will give a picture of the low-flow potentiality of a stream. The column headed "Period of record" shows the water years in which measurements were made at the same, or practically the same, site.

Discharge measurements made at low-flow partial-record stations during water year 1968

Station No.	*Station name*	*Location*	Drainage *area* (sq mi)	Period *of* record	Measurements	
					Date	Discharg (cfs)
		Savannah River basin				
2-1804	Tiger Creek at Lakemont, Ga.	Lat 34°47'03", long 83°24'58", Rabun County, at county highway bridge, at Lakemont.	a26	1951, 1959-60, 1962-68	7- 2-68 8-28-68	34 17
*2-1913	Broad River above Carlton, Ga.	Lat 34°04'37", long 83°00'22", Madison County, at State Highway 72, 2.8 miles northeast of Carlton.	760	1897-1913‡, 1943, 1953-54, 1959, 1962-68	7-22-68 9-13-68	532 396
*	*	* * *		*		*
2-1975.3	Sweetwater Creek near Bonesville, Ga.	Lat 33°26'18", long 82°27'00", McDuffie County, at State Highway 10, 0.8 mile northwest of Bonesville.	7.46	1953, 1955, 1959-61, 1963, 1965-68	8-30-68	.30

* Also a crest-stage partial-record station.
‡ Operated as a continuous-record gaging station.
a Approximately.

FIGURE 298.—Low-flow partial records.

DISCHARGE AT PARTIAL-RECORD STATIONS AND MISCELLANEOUS SITES

The following table contains annual maximum discharge for crest-stage stations. A crest-stage is a device which will register the peak stage occurring between inspections of the gage. A stage-discharge relation for each gage is developed from discharge measurements made by indirect measurements of peak flow or by current meter. The date of the maximum discharge is not always certain but is usually determined by comparison with nearby continuous-record stations, weather records, or local inquiry. Only the maximum discharge for each water year is given. Information on some lower floods may have been obtained, and discharge measurements may have been made for purposes of establishing the stage-discharge relation, but these are not published herein. The years given in the period of record represent water years for which the annual maximum has been determined.

Annual maximum discharge at crest-stage partial-record stations during water year 1968

Station No.	Station name	Location	Drainage area (sq mi)	Period of record	Annual maximum		
					Date	Gage height (feet)	Dis-charge (cfs)
		Virgin River basin					
9-4151	Pulsipher Wash near Mesquite, Nev.	Lat 36°48'04", long 114°06'35", in $NW\frac{1}{4}SW\frac{1}{4}$ sec.18, T.13 S., R.71 E., Clark County, at culvert on U.S. Highway 91, 2.4 miles west of Mesquite.	4.58	1963-68	11-21-67	a5.85	b150
*	*	* *	*		*		*
		Walker Lake basin					
10-2952	West Walker River at Leavitt Meadows, near Coleville, Calif.	Lat 38°19'XX", long 119°33'XX", in $NW\frac{1}{4}NW\frac{1}{4}$ sec.34, T.6 N., R.22 E., Mono County, at Leavitt Meadows Lodge, 16 miles south of Coleville.	73.0	1946-64, 1967-68	7- -67 6- -68	5.95 4.71	c1,600 940
*	*	* *	*		*		*
		Owyhee River basin					
13-1759	Reed Creek near Owyhee, Nev.	Lat 41°53'46", long 116°03'39", in $SW\frac{1}{4}SE\frac{1}{4}$ sec.7, T.46 N., R.53 E., Elko County, at culvert on State Highway 11-A, 3.X miles southeast of Owyhee.	6.51	1962-68	2-20-68	2.58	d5
13-1766	Taylor Canyon tributary near Tuscarora, Nev.	Lat 41°14'XX", long 116°02'XX", in $S\frac{1}{2}$ sec.29, T.39 N., R.53 E., Elko County, at culvert on State Highway 11, 11 miles southeast of Tuscarora.	e1.2	1967-68	1967 2-20-68	(f) 2.99	d7 (†)

See footnotes at end of table, p.________.

FIGURE 299.—Crest-stage partial records.

DISCHARGE AT PARTIAL-RECORD STATIONS AND MISCELLANEOUS SITES

Discharge measurements at miscellaneous sites

Measurements of streamflow at points other than gaging stations are given in the following table. Those that are measurements base flow are designated by an asterisk (*), measurements of peak flow by a dagger (†)

Discharge measurements made at miscellaneous sites during water year 19XX

Stream	Tributary to	Location	Drainage area (sq mi)	Measured previously (water years)	Measurements: Date	Measurements: Discharge (cfs)
Part 6A		Yellowstone River basin				
Little Wind River a/	Wind River	Lat 42°57'37", long 108°29'54", in NE¼SE¼ sec 22, T 1 S , R 3 E , Fremont County, at bridge on county road, 1.2 miles upstream from Popo Agie River, 0.5 mile west of Arapahoe, Wyo	b650	1966-67	10-26-XX 2- 1-XX 3-30-XX 6-23-XX 8- 1-XX	137 91 53 †3,760 *7 8
Bighorn River	Yellowstone River	Lat 44°12'XX", long 107°55'XX", in sec.28, T 49 N , R 92 W , Big Horn County, at bridge on county road, 0 2 mile west of Rairden, 1.2 miles downstream from Fivemile Creek, 5.8 miles southeast of Manderson, Wyo.	11,020	1949-53‡, 1955-56‡	10- 3-XX 2-11-XX 5-17-XX	*437 *c980 7,430
Do	.. do	Lat * * *.	-	-	6- 7-XX	†d17,300
Part 9		Green River basin				
Bitter Creek	Green River	Lat 41°31'25", long 109°25'41", in NE¼SE¼ sec.24, T.18 N , R 107 W , Sweetwater County, 1.7 miles east of town of Green River, Wyo., and 1.9 miles upstream from mouth.	-	1958, 1966-67	10- 6-XX 3-19-XX 7-13-XX	*5.6 15 * 91
Part 10		Bear River basin				
Bear River	Great Salt Lake	Lat 41°19'XX", long 111°01'XX", in NE¼ sec.1, T 15 N , R.121 W , Uinta County, at bridge on county road, 3.5 miles northwest of Evanston, Wyo.	715	1913-56‡	9-12-XX	*8 3
Part 13		Salt River basin				
Stump Creek	Salt River	Lat 42°47'00", long 111°03'10", in NE¼SW¼ sec 27, T.7 S., R 46 E , Lincoln County, at bridge on county road, 1.2 miles upstream from Wyoming-Idaho State line, 3.3 miles west of Auburn, Wyo.	102	1946-49‡, 1962-65	11- 3-XX 1-16-XX 5-21-XX 8-28-XX	*24 *11 †375 *26

‡ Operated as a continuous-record gaging station
a Formerly published as Popo Agie River.
b Approximately.
c Not previously published.
d Revised

FIGURE 300.—Discharge measurements at miscellaneous sites.

RIO GRANDE BASIN

Pecos River seepage investigations—Acme to Artesia, N Mex.

Two series of discharge measurements were made during the 19XX water year, on Jan. 20 and Mar. 31, on the Pecos River and tributaries and diversions in New Mexico, to study channel gains and losses. The reach is 81 6 miles in length and extends from the gaging station Pecos River near Acme (8-3860) to the gaging station Pecos River near Artesia (8-3965). The measurements were made during periods of constant base flow of the streams, for 10 days before the investigations no measurable precipitation had fallen Tributary flow was considered a contribution and not a gain; diversion was considered a deduction and not a loss Indicated gains or losses may be substantially in error as affected by small inaccuracies in open-channel measurements Records of chemical analyses and water temperatures obtained at the time of this investigation are published in Part 2 of this report. Previous seepage investigations of this reach were made at least once each water year 1953-60, 1962-66

Pecos River mile	Stream	Location	Discharge, in cubic feet per second: Jan 20, 19XX Meas. disch.	Gain or loss	Mar. 31, 19XX Meas disch.	Gain or loss
94 0	Pecos River	Gaging station near Acme (8-3860).	4.83	-	40 9	-
89 1	.. do ..	SE¼SW¼SE¼ sec.27, T.9 S , R 25 E., above Bitter Lakes.	2 33	-2 50	28 3	-12 6
78 4	. do. .	NE¼NE¼NE¼ sec.33, T 10 S , R 25 E , at mouth of Bitter Creek..	4.25	+1 92	33 8	+5.5
78 4	Bitter Creek	NE¼NE¼NE¼ sec 33, T 10 S , R 25 E , at mouth....	2 96	-	5 76	-
74.7	Pecos River	SE¼SE¼NE¼ sec.9, T.11 S., R.25 E., above Rio Hondo..	8.79	+1 58	43 8	+4.2
74 6	Rio Hondo	NE¼NE¼SE¼ sec 9, T.11 S., R 25 E., at mouth	7 14	-	7.23	-
74 5	Pecos River	SE¼NE¼SE¼ sec 9, T.11 S , R 25 E., below Rio Hondo.	15.8	-.13	49.4	-1.6
	*	* * * *	*		*	
46 7	Pecos River	SE¼NE¼NE¼ sec 12, T.14 S., R 26 E., at Hagerman bridge.	31 8	+9 7	70 9	+15.4
46 5	Diversion	* * *	9 42	-	15.7	-
44 2	Pecos River	SW¼SW¼SE¼ sec 13, T.14 S , R 26 E....	22 5	+ 1	58 6	+3 4
	*	* * * *	*		*	
30 6	Pecos River	Gaging station near Lake Arthur (8-3955)	25.7	+3 05	54.5	-2.9
	*	* * * *	*		*	
12 4	Pecos River	Gaging station near Artesia (8-3965).	34.0	+2 78	54.1	-4.2
		Overall net gain or loss		+14.8		+1.4

[NOTE.--In the above example many lines have been omitted as indicated by asterisks, but the figures of "gain or loss" are based on complete data.]

FIGURE 301.—Seepage investigation.

STREAMS TRIBUTARY TO LAKE ST. CLAIR

North Branch Clinton River basin low-flow investigations

Two series of base-flow discharge measurements were made in the North Branch Clinton River basin as part of a comprehensive program now being carried on in cooperation with the Macomb County Board of Supervisors' and the Macomb County Road Commission to investigate the surface water resources of the county The first series was made on June 9, as soon as conditions were suitable for high base flow The second series was made on Aug. 18, under conditions of low base flow. The data collected in these series of measurements, along with that already collected and to be collected in the future, will provide the basis for determining the base-flow yields of various parts of the basin.

Weather records at Mount Clemens near the southern part of the area and at Romeo near the west-central part show that no precipitation occurred for five days prior to June 9 and four days prior to Aug. 18. Therefore, the measurements are considered to represent base flow

The measurements on each stream are listed in order proceeding downstream, and each tributary is inserted in the order in which it enters the main stream. Drainage areas shown were determined from recent U.S Geological Survey topographic maps of a scale of 1·24,000 and contour interval of 5 to 10 ft Previous series of measurements were made in water years 1959-64

Discharge measurements of North Branch Clinton River and tributaries near Mount Clemens, Mich

Stream	Location	Drainage area (sq mi)	Discharge, in cubic feet per second			
			Measured discharge	Cfs per square mile	Measured discharge	Cfs per square mile
			June 9, 19XX		Aug. 18, 19XX	
North Branch Clinton River.	NW$\frac{1}{4}$ sec 27, T 6 N , R 12 E , at State Highway 53, in Almont	9 56	2.42	0 253	1.19	0.124
Do	NW$\frac{1}{4}$ sec.1, T 5 N , R.12 E., at Macomb-Lapeer County line, 2.8 miles southeast of Almont	17.1	3 34	195	1 26	.074
Apel drain	NW$\frac{1}{4}$ sec.13, T 5 N , R.12 E , at McKay Rd., 0 5 mile above mouth and 3 8 miles north of Romeo	4 04	93	.230	.48	.119
Newland drain . .	SW$\frac{1}{4}$ sec 19, T 5 N , R 13 E , at mouth, 2.8 miles northeast of Romeo.	9 39	2.11	.225	60	.064
Mahaffy drain...	NW$\frac{1}{4}$ sec.25, T 5 N , R 12 E , at Mack Rd. (34-Mile), 0.6 mile above mouth and 2 1 miles northeast of Romeo	2.64	.52	197	05	019
North Branch Clinton River	SW$\frac{1}{4}$ sec.30, T 5 N , R 13 E , at 33-Mile Rd , 2 2 miles northeast of Romeo	49 7	11.0	.221	4 00	.080
East Pond Creek....	SW$\frac{1}{4}$ sec 7, T-5 N., R 12 E., at Dewey Rd (36-Mile), 3.8 miles northeast of Lakeville.	9.49	1.87	197	.06	.006
Do	SW$\frac{1}{4}$ sec.29, T 5 N , R 12 E , at Schooley Rd. (33-Mile), 3.1 miles northwest of Romeo.	14 2	4 24	.299	1.47	.104
Do...	Gaging station near Romeo (4-1641).	21.8	8.13	373	3.10	142
Do	NW$\frac{1}{4}$ sec.6, T 4 N , R.13 E , at Powell Rd , 0.5 mile above mouth and 1.6 miles east of Romeo	24.5	8.52	348	3.43	.140
North Branch Clinton River	NW$\frac{1}{4}$ sec 16, T 4 N , R 13 E , at 30-Mile Rd , 2 0 miles northwest of Ray Center.	80 5	18 2	.226	5 77	.072
Coon Creek.	NE$\frac{1}{4}$ sec.21, T 5 N., R 13 E , at Armada Center Rd., 2 1 miles west of Armada	3.98	75	188	33	083
Do	SW$\frac{1}{4}$ sec.1, T 4 N , R 13 E , at North Rd , 3.4 miles south of Armada.	10 0	1.35	135	.34	.034
Do	SW$\frac{1}{4}$ sec 25, T 4 N , R 13 E , at North Rd , 1.4 miles north of Meade.	15.2	1 26	083	05	003
Tupper Brook	NW$\frac{1}{4}$ sec 11, T.4 N , R 13 E , at 31-Mile Rd., 3.7 miles south of Armada	4.21	0	0	0	0
Do	NW$\frac{1}{4}$ sec.23, T 4 N., R.13 E , at 29-Mile Rd , at Ray Center.	8 62	0	0	0	0
East Branch Coon Creek .	SE$\frac{1}{4}$ sec 2, T 5 N , R.13 E , at Pratt Rd , 3 0 miles north of Armada.	8.34	50	.060	.04	005
Do	Gaging station at Armada (4-1643)	13.0	.71	.055	06	005
*	* * *	*		*	*	
Middle Branch Clinton River.	Private Claim 546, T 2 N., R.13 E., at Heydereich Rd., near mouth, just above Miller drain, 1 0 mile west of Mount Clemens	74 7	8.88	119	2 71	.036

FIGURE 302.—Low-flow investigation.

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FIGURE 303.–Index for annual published report (only first page).

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