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

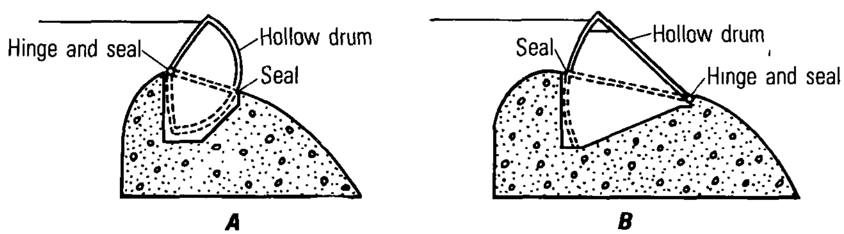


FIGURE 227.—Two types of drum gate.

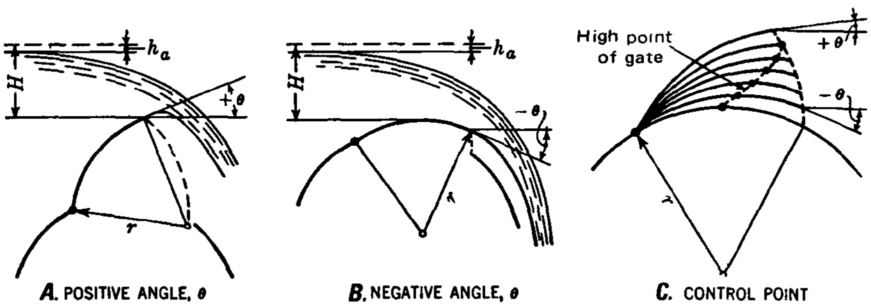


FIGURE 228.—Drum-gate positions. (After Bradley, 1953.)

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 (ft^3/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. 228A).

For negative values of Θ (fig. 228B), 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 228C, 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

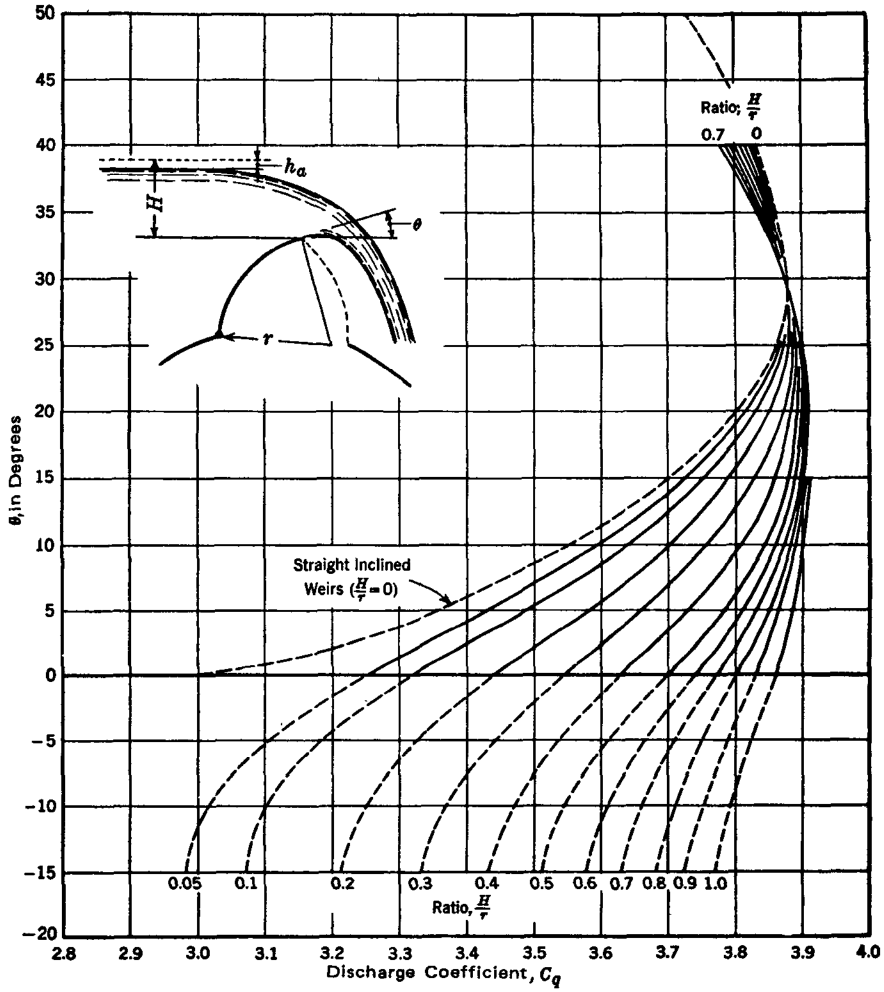


FIGURE 229.—General curves for the determination of discharge coefficients. (After Bradley, 1953.)

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

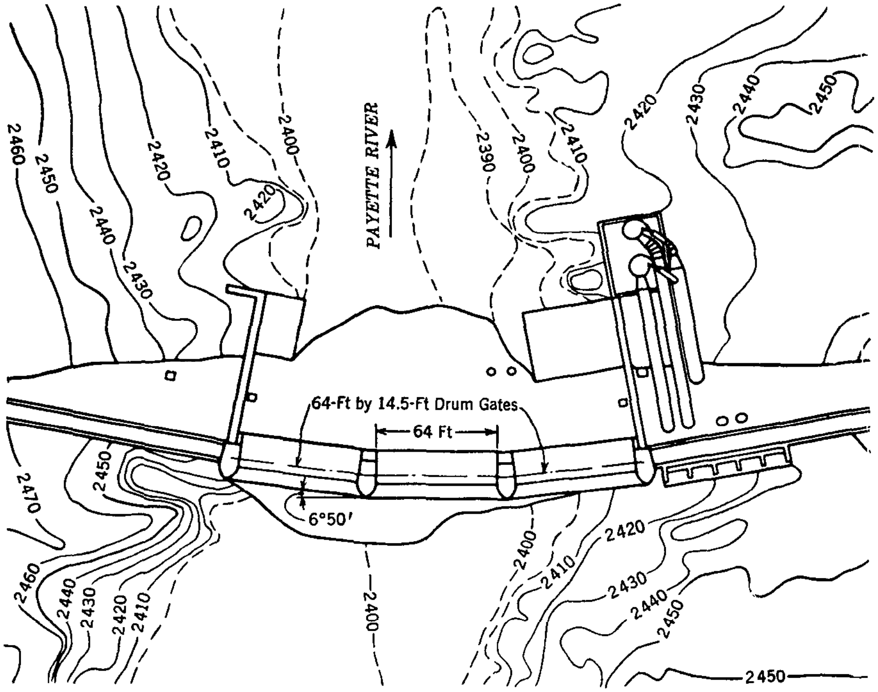


FIGURE 230.—Plan of Black Canyon Dam in Idaho. (After Bradley, 1953.)

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_0) of Black Canyon diversion dam was found to be 14.5 ft and the corresponding coefficient of discharge (C_0) 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

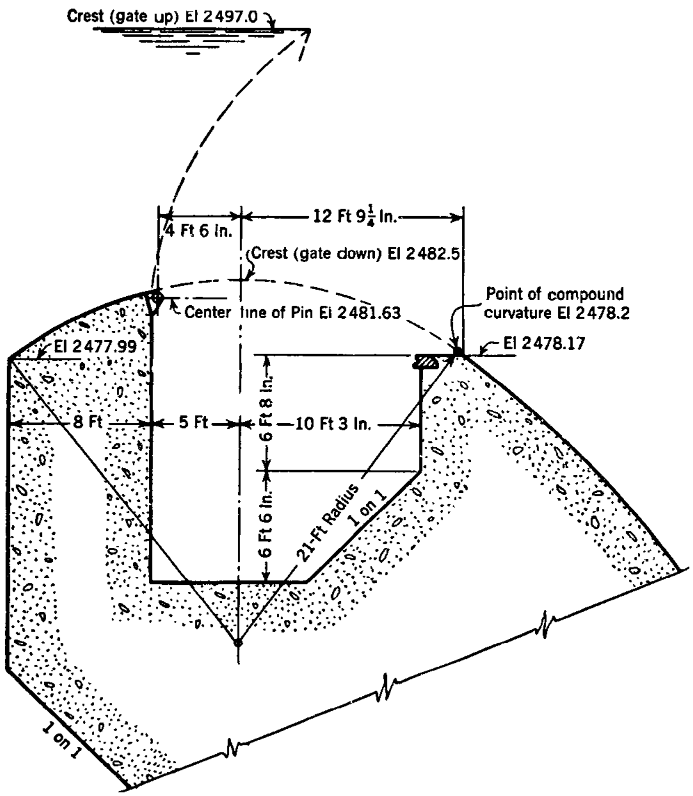


FIGURE 231.—Spillway crest detail, Black Canyon Dam, Idaho. (After Bradley, 1953.)

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,

$$Q = C_a bH^{3/2}. \tag{124}$$

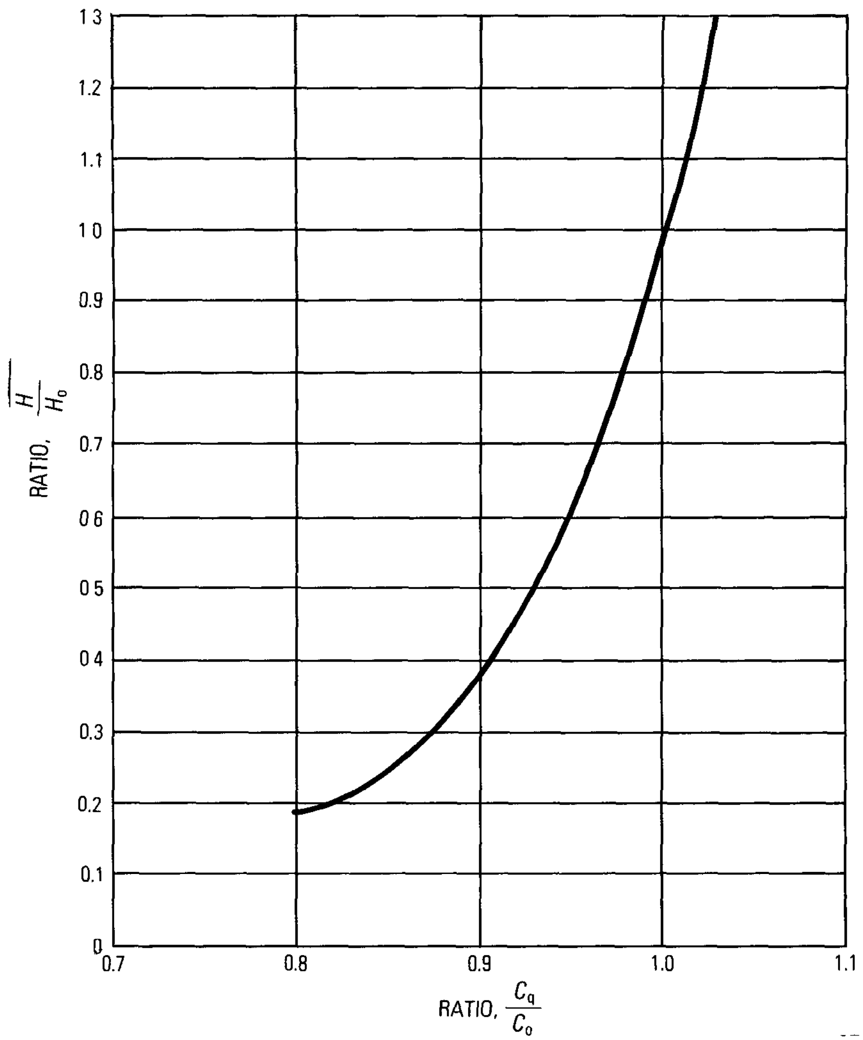


FIGURE 232.—Diagram for determining coefficients of discharge for heads other than the design head (After Bradley, 1953.)

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	Reservoir elevation, in ft	Ratio,[1] H/H_o	Ratio,[2] C_o/C_o	Coefficient, C_o	Q, in ft³/s [3]
(1)	(2)	(3)	(4)	(5)	(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: $Q = C_o b H^{3/2}$, 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

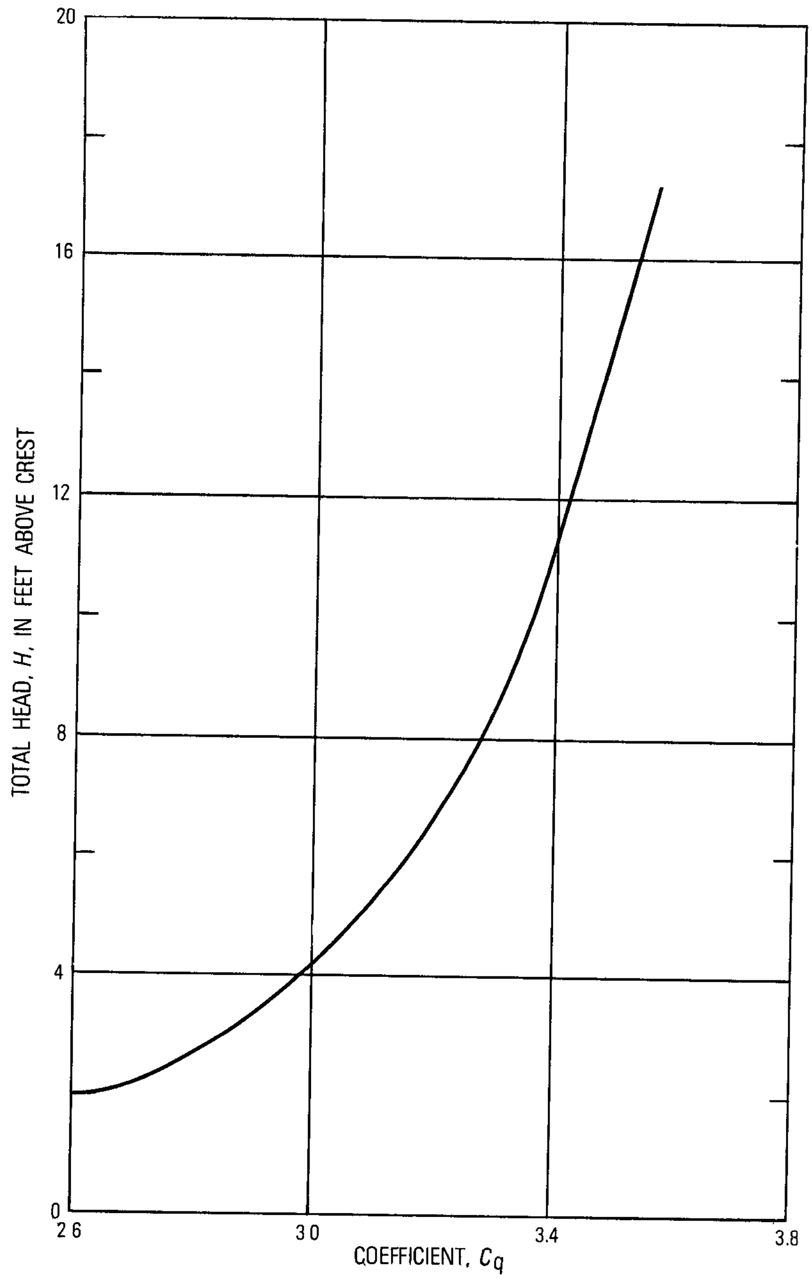
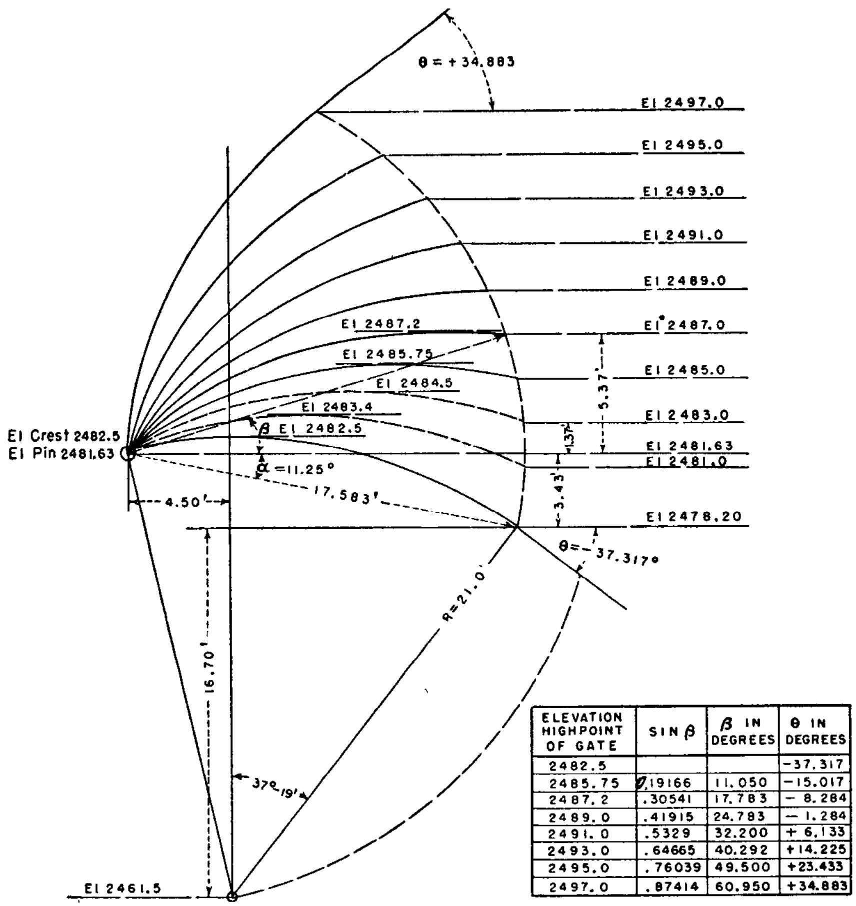


FIGURE 233.—Head-coefficient curve, Black Canyon Dam, Idaho. (After Bradley, 1953.)



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_a (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: $Q = C_a b H^{3/2}$.

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).

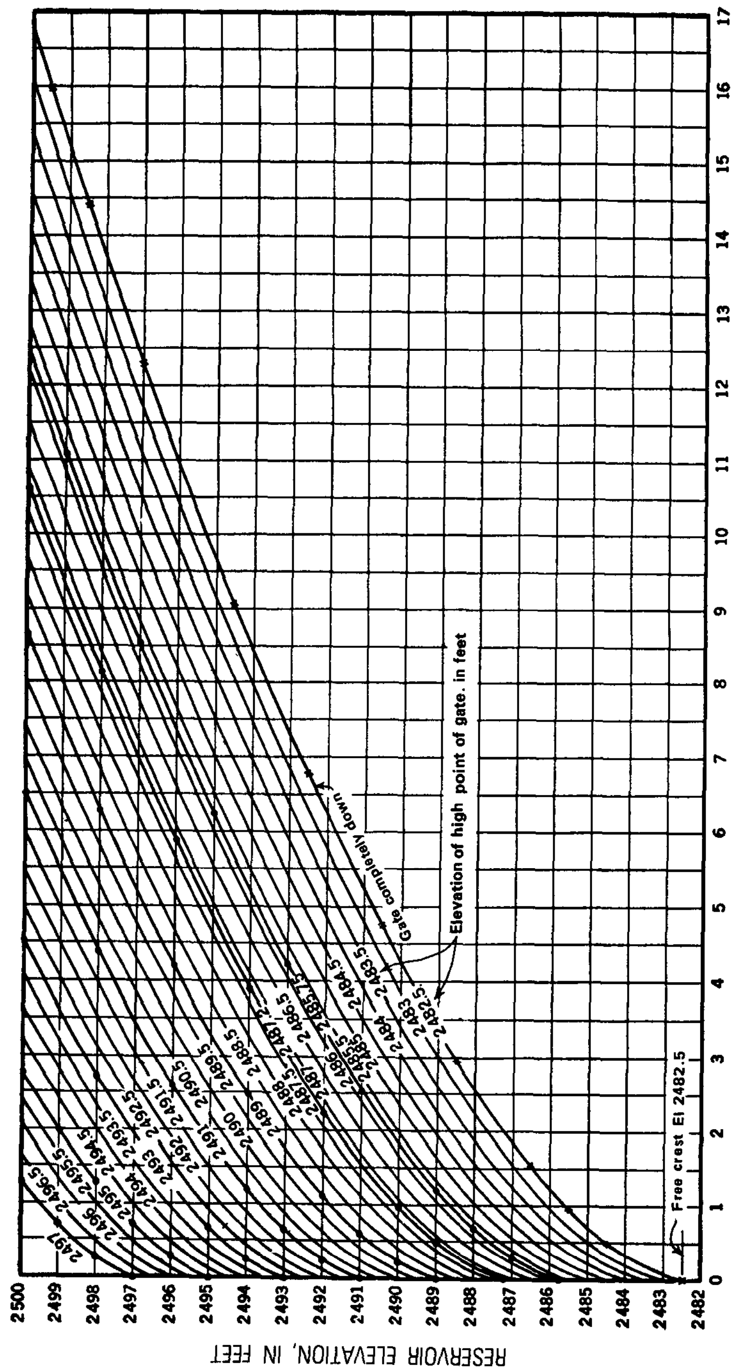


FIGURE 235.—Rating curves for drum-gate spillway of Black Canyon Dam, Idaho. (After Bradley, 1953.)

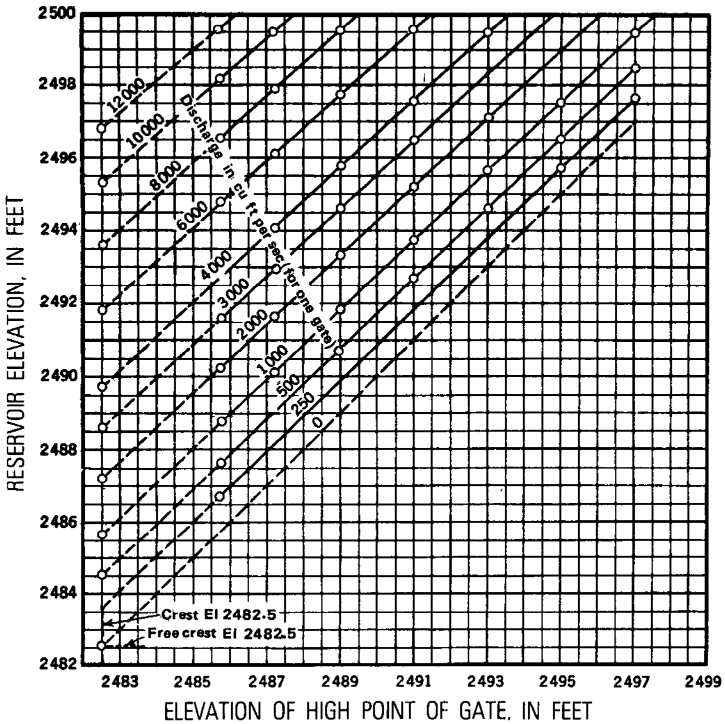


FIGURE 236.—Cross-plotting of values from initial rating curves, Black Canyon Dam, Idaho. (After Bradley, 1953.)

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

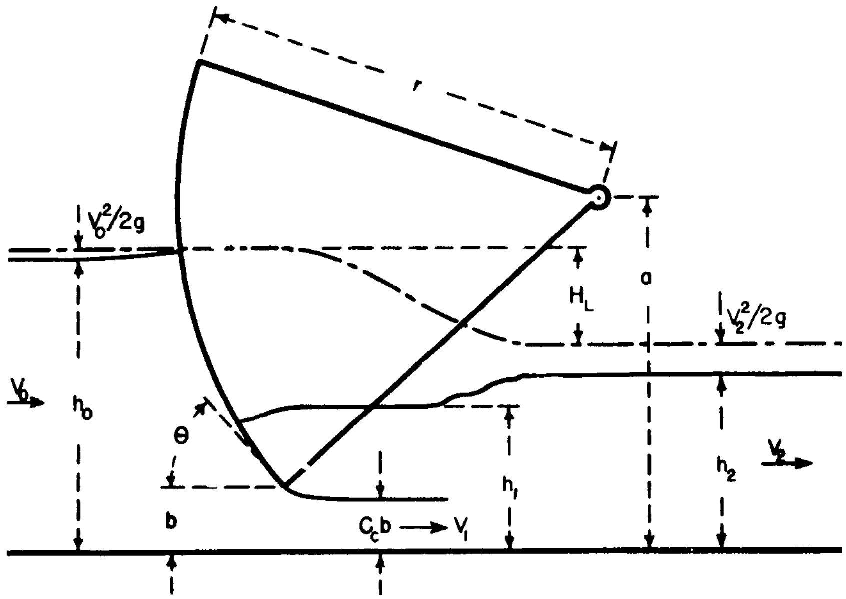


FIGURE 237.—Definition sketch of a radial gate on a horizontal surface. (After Toch, 1953.)

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

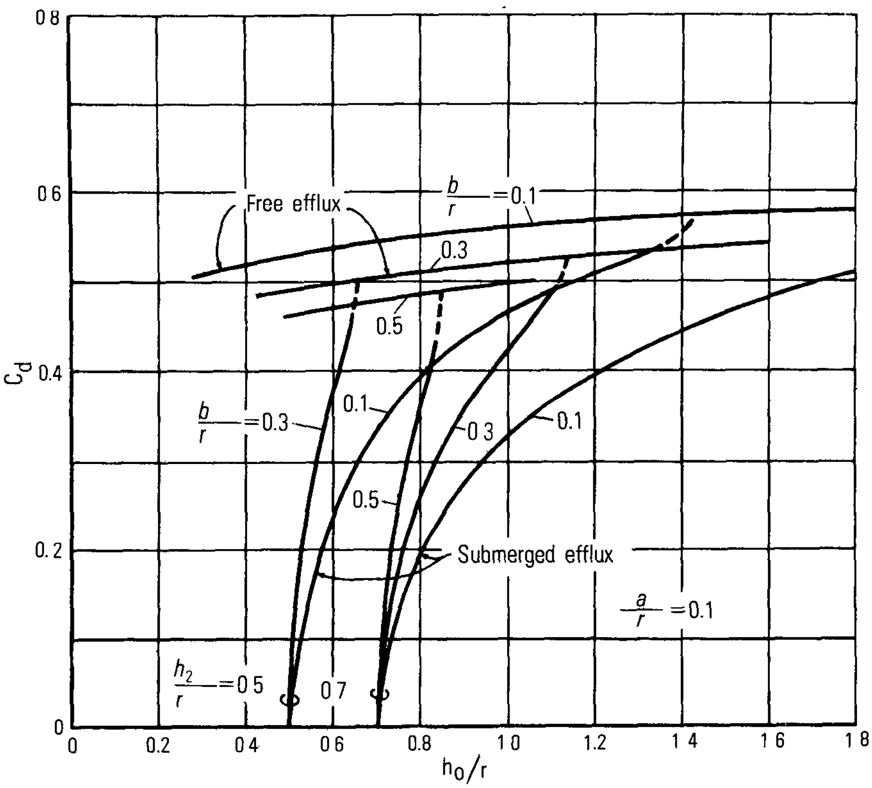


FIGURE 238.—Coefficient of discharge for free and submerged efflux, a/r = 0.1. (After Toch, 1953.)

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,

$$Q = Ch_g b\ (2gh_1)^{1/2}, \tag{126}$$

where

Q = discharge for one gate,
c = discharge coefficient,

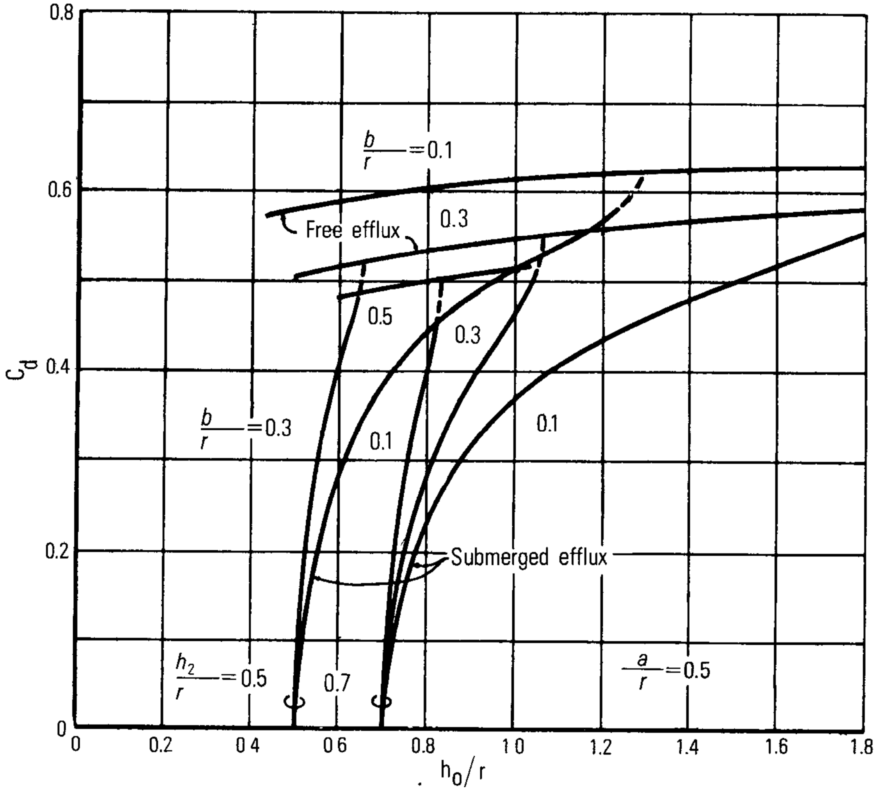


FIGURE 239.—Coefficient of discharge for free and submerged efflux, a/r = 0.5 (After Toch, 1953.)

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:

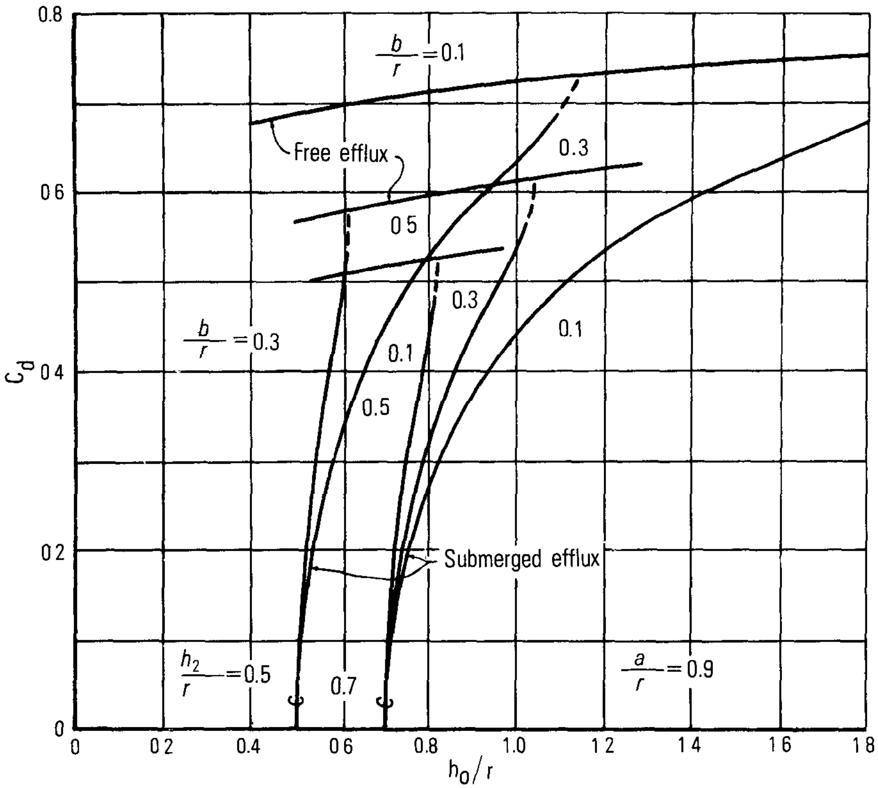


FIGURE 240.—Coefficient of discharge for free and submerged efflux, a/r = 0.9. (After Toch, 1953.)

$$h_g = R \cos\Theta\left(\frac{c-a}{r}\right) + a - R \sin\Theta \sqrt{1 - \left(\frac{c-a}{r}\right)^2},$$

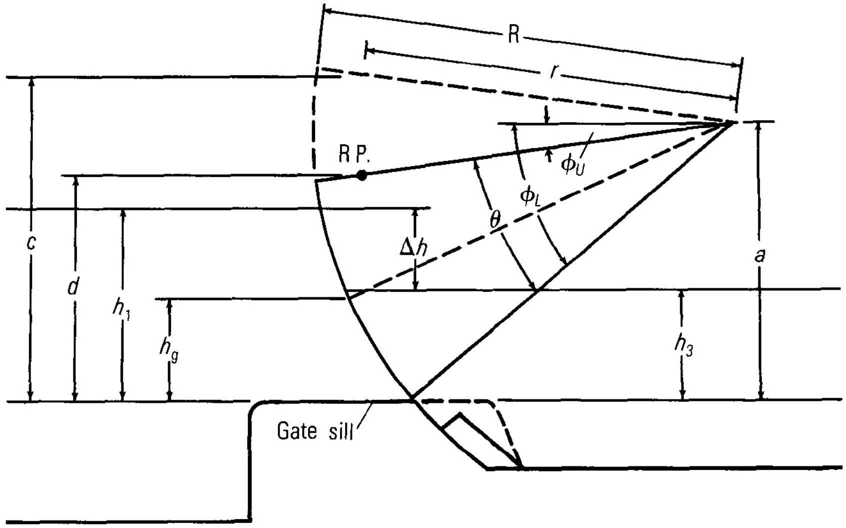
where

$$\Theta = \Phi_1 - \Phi_2 = \sin^{-1}\left(\frac{a}{R}\right) - \sin^{-1}\left(\frac{a-d}{r}\right).$$

Because C does not vary linearly with h_g 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_g 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_g, and h_g is less than $(2/3)h_1$. The basic equation for computing discharge is

$$Q = C_{gs} h_g b (2g\Delta h)^{1/2}, \tag{127}$$



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

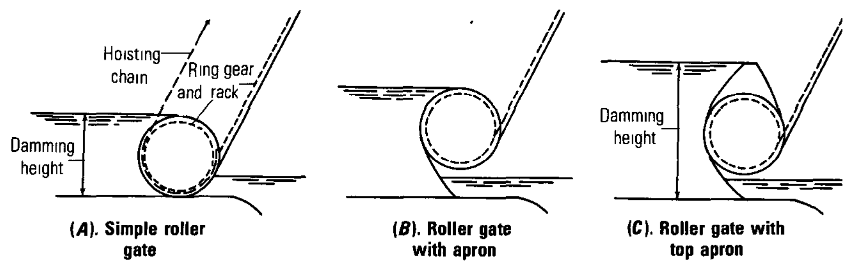


FIGURE 242.—Schematic sketches of roller gates. (U.S. Army Corps of Engineers, 1952.)

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

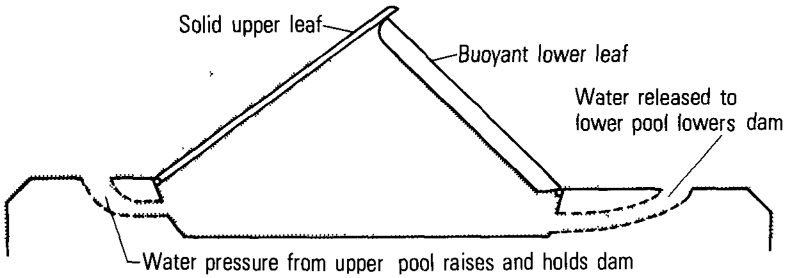


FIGURE 243.—Bear-trap gate. (U.S. Army Corps of Engineers, 1952.)

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

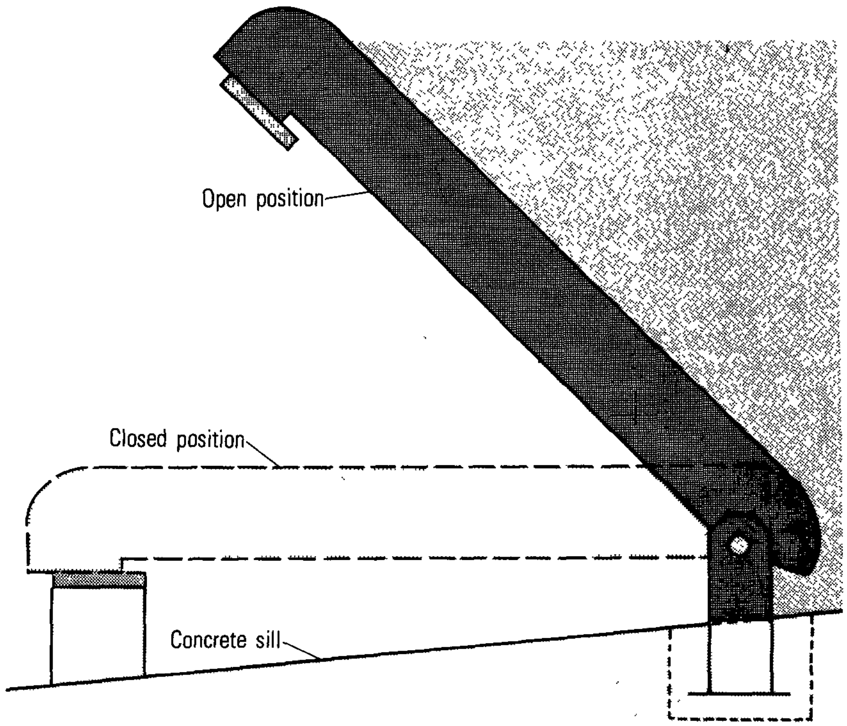


FIGURE 244.—Hinged-leaf gate.

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

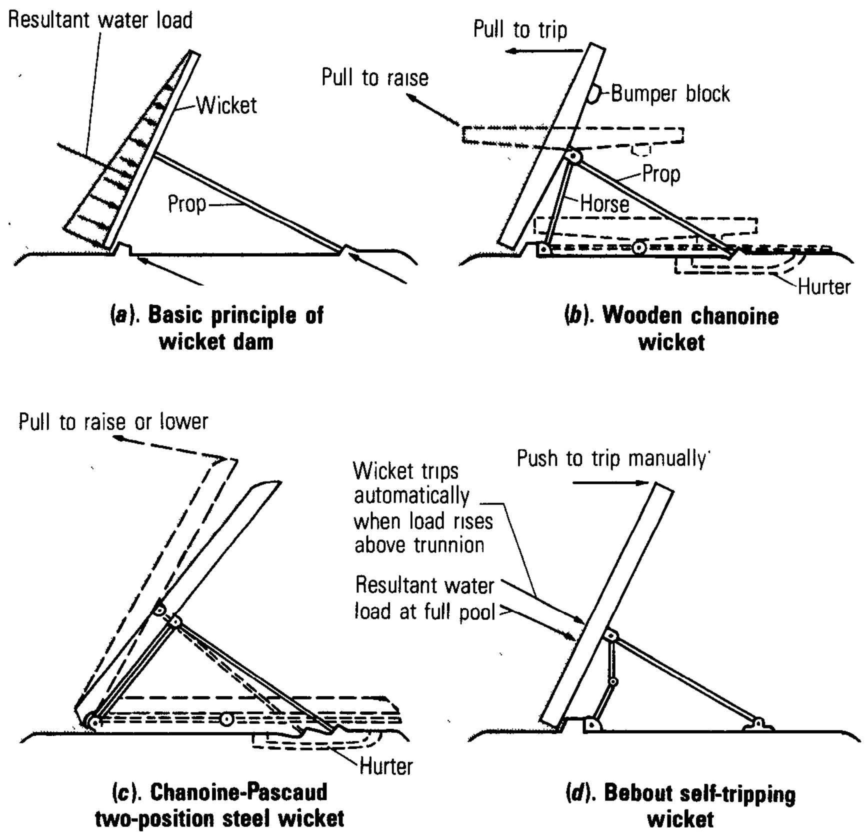


FIGURE 245.—Wickets. (U.S. Army Corp of Engineers, 1952.)

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.

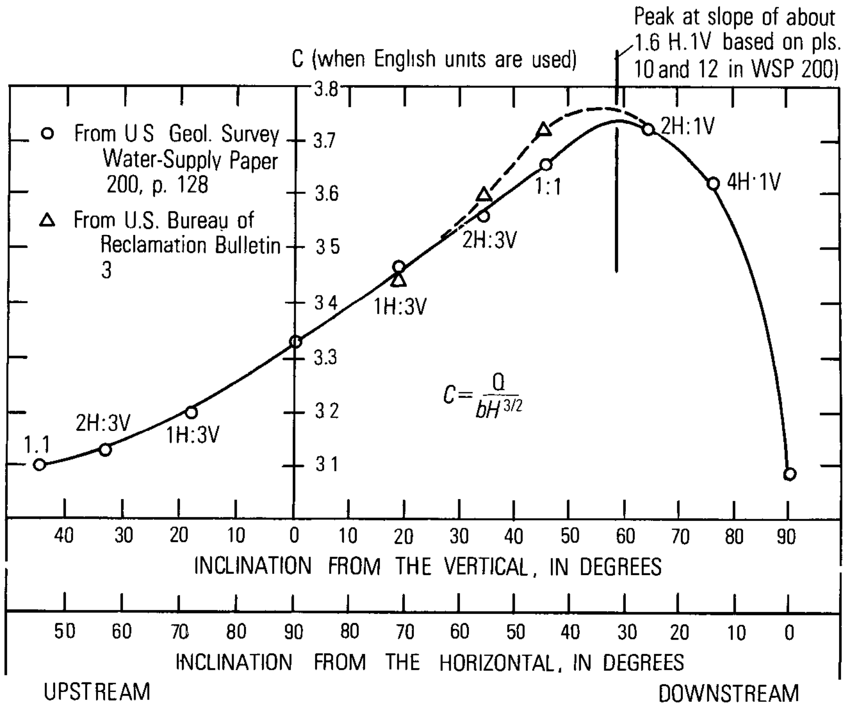


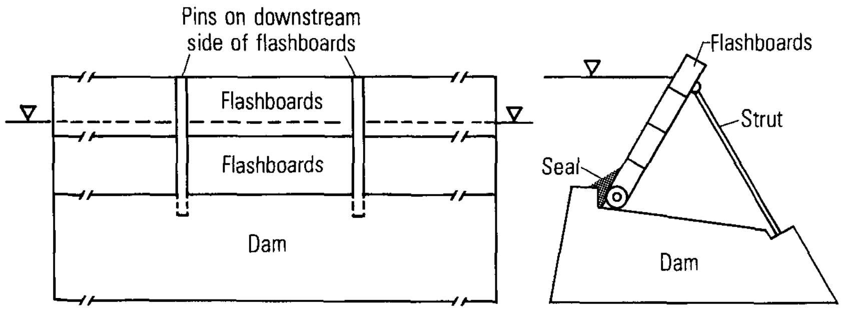
FIGURE 246.—Discharge coefficients for an inclined rectangular thin-plate weir.

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

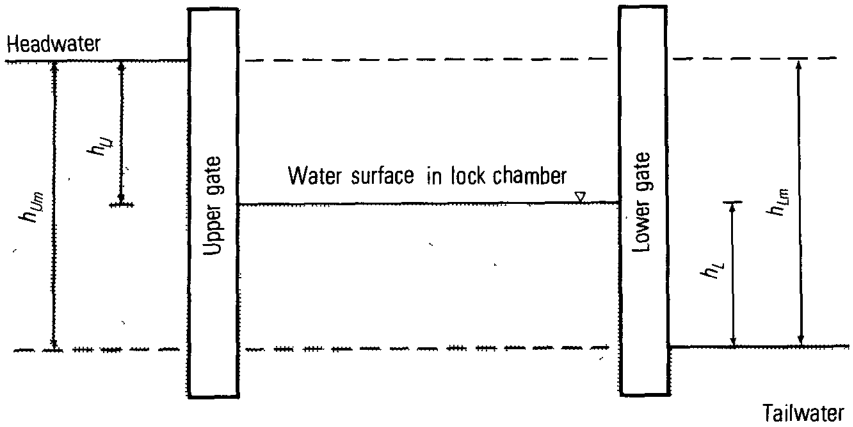
If the leakage through the closed lock gates is great, it can be measured in the forebay with a low-velocity current meter. The leakage will seldom be that great, however, and it usually will have to be computed by a volumetric method.

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 ($Q_{l\,m}$) 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}, $Q_{l\,m}$, 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

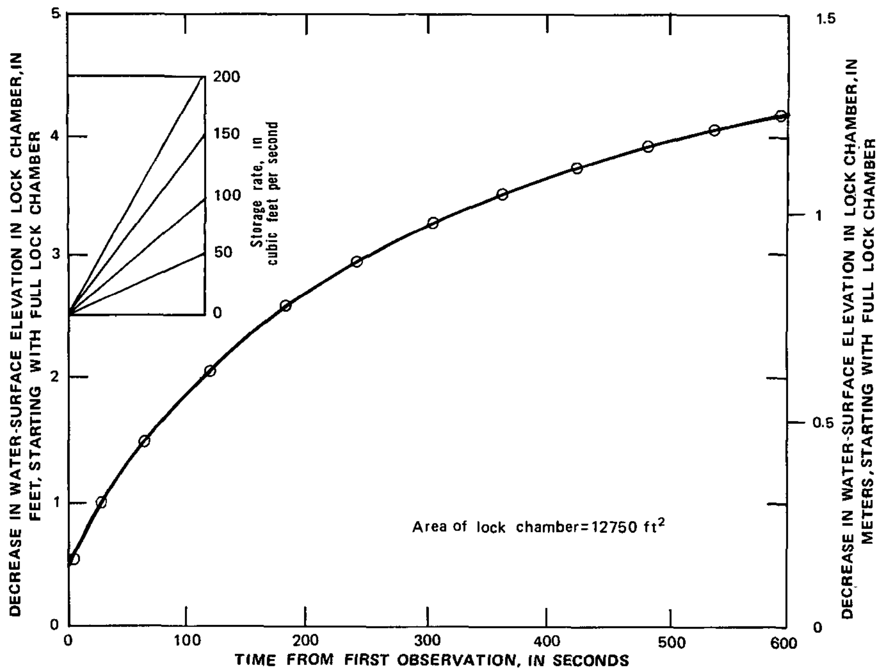


FIGURE 249.—Storage diagram starting with lock chamber full.

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_u) for various tangential slopes.

The method of constructing the diagram is demonstrated in figure 249. The area of the lock chamber is 12,750 ft². If the stage in the chamber dropped 2 feet, the change in volume would be 2 × 12,750 or 25,500 ft³. If Q_u were 200 ft³/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_u value of 200 ft³/s. Diagonals representing other values of Q_u 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_u. 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_u 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

$$Q_L = Q_{Lm}\left(\frac{h_L}{h_{Lm}}\right)^{1/2} \quad \text{and} \quad Q_U = Q_{Um}\left(\frac{h_U}{h_{Um}}\right)^{1/2} \qquad (135)$$

7. From figure 248 and equation 135.

$Q_u = Q_L - Q_U$

or
$$Q_n = Q_{Lm}\left(\frac{h_L}{h_{Lm}}\right)^{1/2} - Q_{Um}\left(\frac{h_U}{h_{Um}}\right)^{1/2} \tag{136}$$

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:

$$Q'_{Lm} = Q_{Lm}\left(\frac{h'_{Lm}}{h_{Lm}}\right)^{1/2} \tag{137}$$

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:

$$Q'_{Um} = Q_{Um}\left(\frac{h'_{Um}}{h_{Um}}\right)^{1/2}, \tag{138}$$

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-

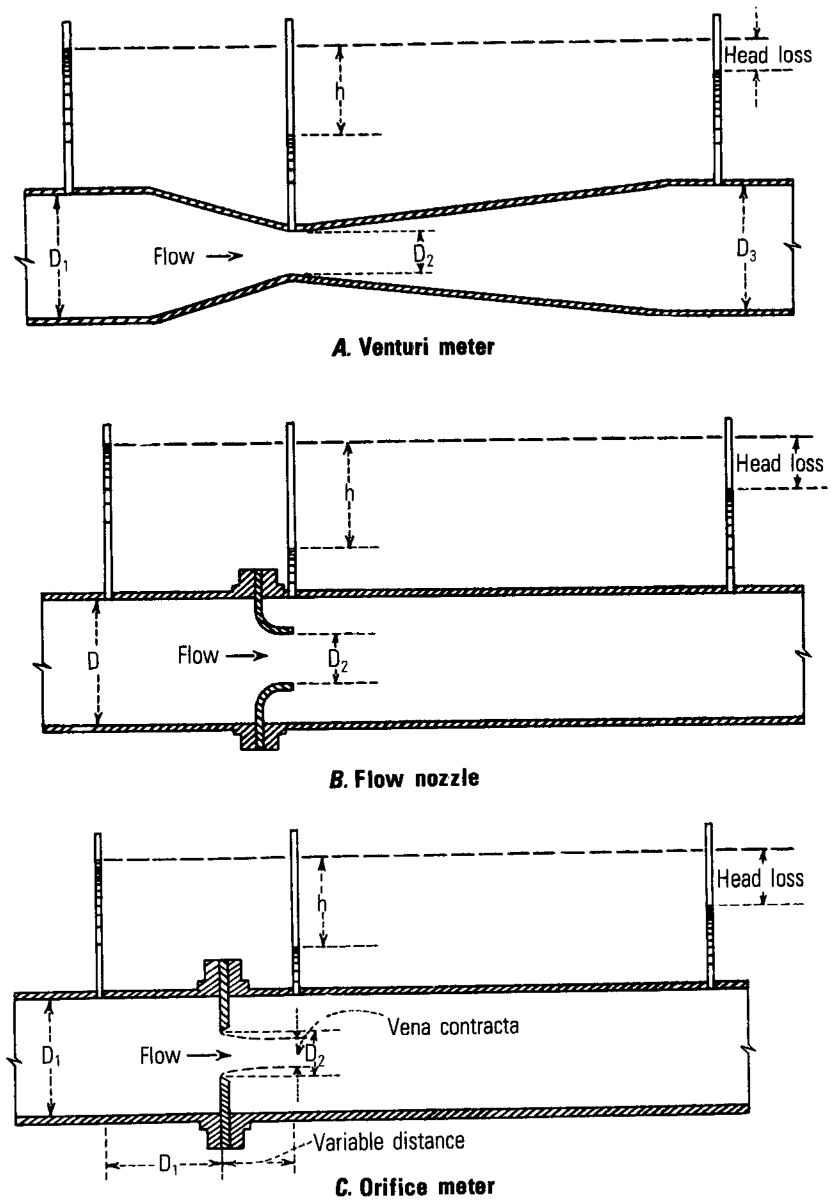


FIGURE 250.—Three types of constriction meter for pipe flow. (Courtesy of U.S. Bureau of Reclamation.)

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

$$R = \frac{V_2D_2}{\nu}, \tag{142}$$

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

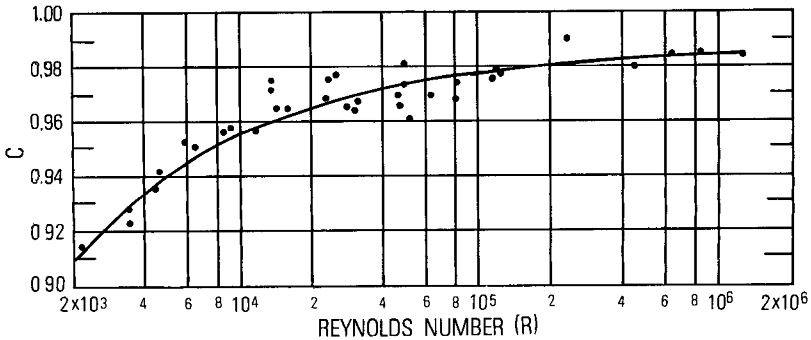


FIGURE 251.—Discharge coefficients for venturi meters as related to Reynolds number. (After Howe, 1950. Reprinted by permission of John Wiley & Sons, Inc.)

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.

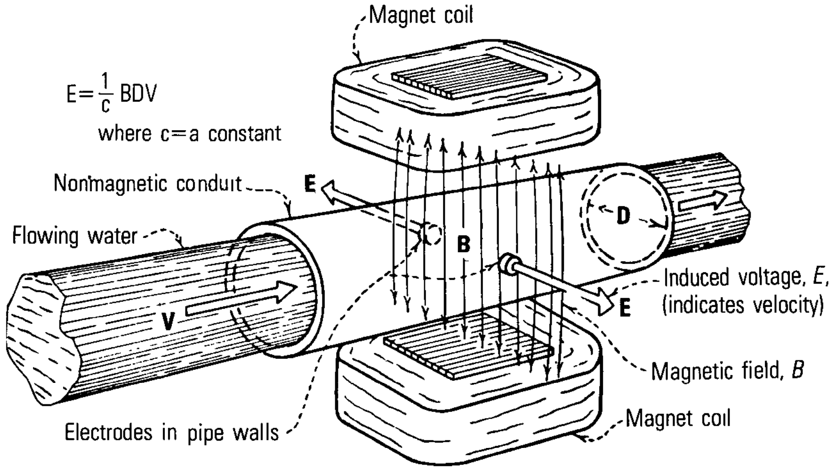


FIGURE 252.—Schematic view of one type of electromagnetic velocity meter. (Courtesy of U.S. Bureau of Reclamation.)

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

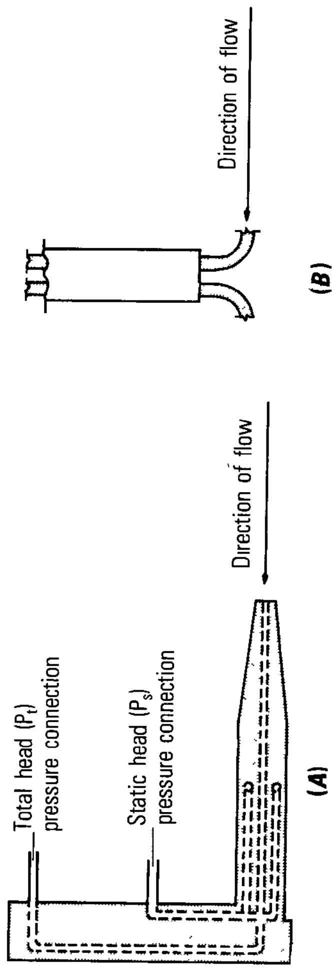


FIGURE 253.—Schematic drawing of (A) pitot-static tube and (B) Cole pitometer.

$$V = C_1\sqrt{2g\Delta h}, \tag{145}$$

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.

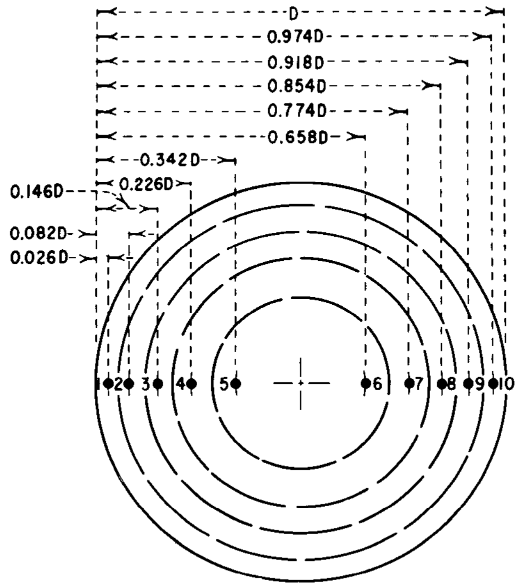
Another commonly used type of pitot device is the Cole pitometer (fig. 253*B*), which consists of two tubes headed in opposite directions. The tubes can be rotated so that the instrument may be inserted through a small bushing in a pipe. When in operating position, the downstream tube registers a negative 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.

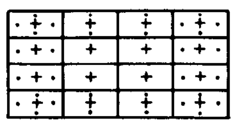
The most common pressure conduit is the circular pipe. For a constant rate of flow, the velocity varies from point to point across the stream, gradually increasing from the walls to the center of the pipe. The mean velocity is obtained by dividing the cross-sectional area of the pipe into a number of 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,

$$V_{\text{mean}} = C_1(\sqrt{2g})(\sqrt{\Delta h})_{\text{average}}. \tag{146}$$

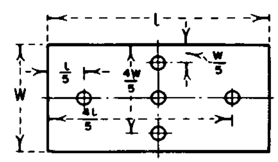
The mean velocity in rectangular ducts can be determined by first dividing the cross section into an even number—at least 16—of equal



(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

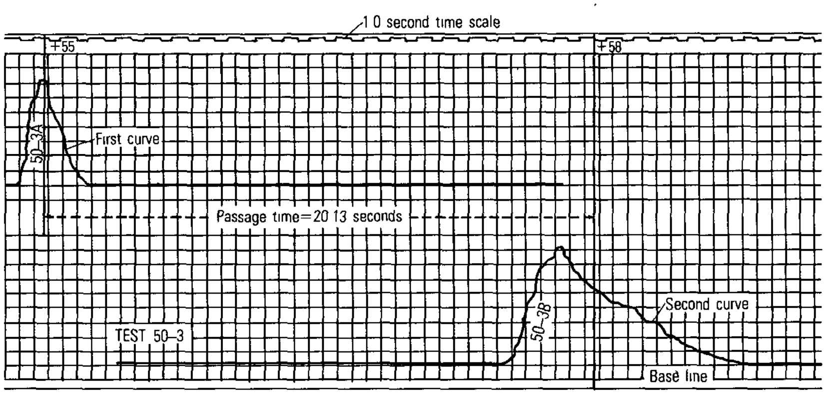


FIGURE 255.—Sample record of a salt cloud passing upstream and downstream electrodes in the salt-velocity method of measuring flows in pipelines. (Courtesy of U.S. Bureau of Reclamation.)

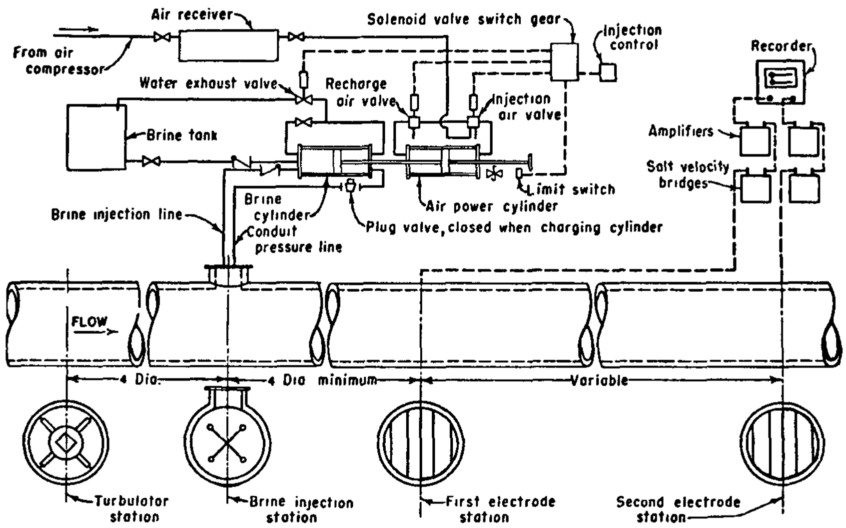


FIGURE 256.—General arrangement of salt-velocity equipment for pressure conduits. (Courtesy of U.S. Bureau of Reclamation.)

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

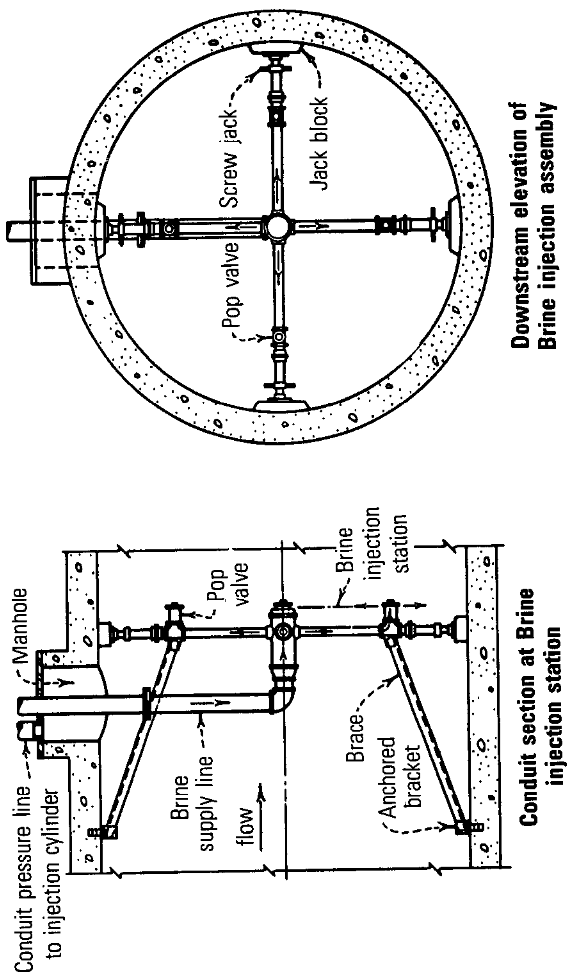


FIGURE 257.—Brine injection equipment in conduit. (Courtesy of U.S. Bureau of Reclamation.)

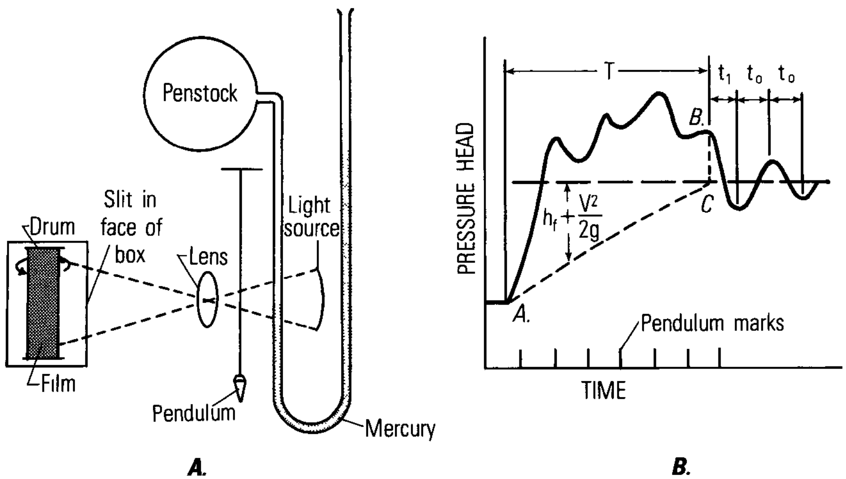


FIGURE 258.—Gibson apparatus and pressure-variation chart. (After Howe, 1950. Reprinted by permission of John Wiley & Sons, Inc.)

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,

$$Q = g\left[\Sigma(a/l)\right]\left[\text{area } ABCA\right] \qquad (147a)$$

It is generally agreed that the Gibson method is very accurate. As an application of the momentum principle, this might be éxpected. 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.

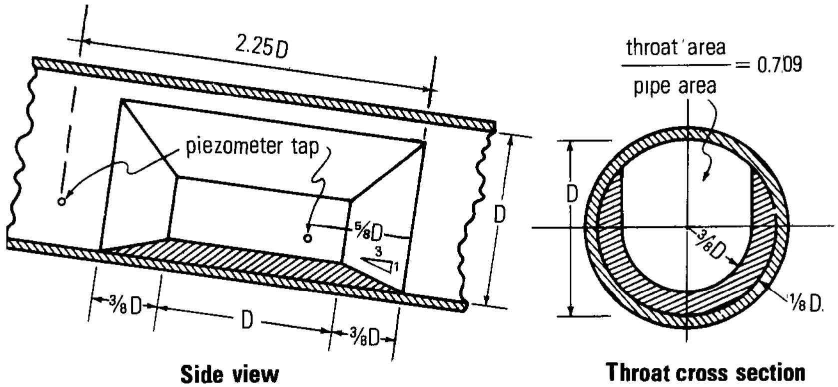


FIGURE 259.—Sketch of USGS flowmeter in a sewer.

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

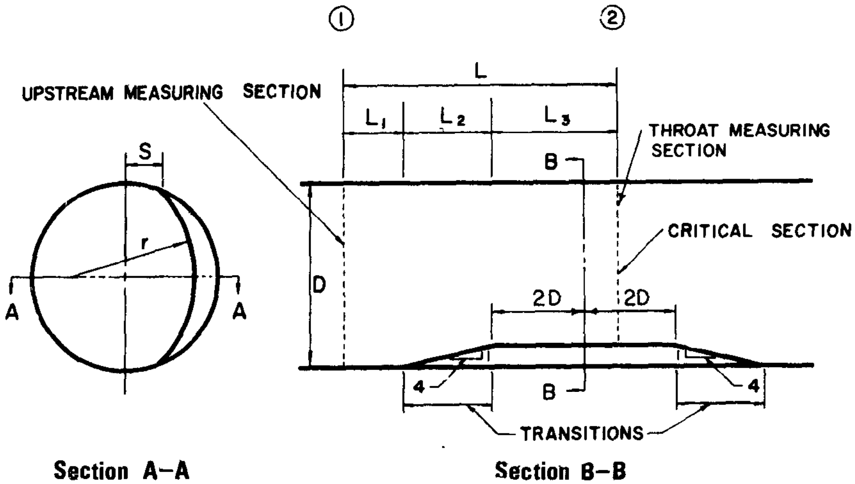


FIGURE 260.—Sketch of Wenzel asymmetrical flowmeter in a sewer. (After Wenzel, 1975.)

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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