CHAPTER 2. RUNOFF

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INTRODUCTION

The objectives of a study influence selection of a runoff measuring device. For example, if a water budget is studied in an area where runoff accounts for only about 5 percent of the water budget, 95 percent of the funds should not be spent on measurement of runoff. More funds should be directed to other parts of the hydrologic cycle. If the objective of a study, on the other hand, is to measure the effect of land treatment on the water supply derived from a watershed, a precalibrated runoff measuring device would be required. Less funds and time should be directed to other parts of the hydrologic cycle.

This chapter provides guidance on selecting devices for measuring runoff. It also provides guidance on types of recorders, field installation and maintenance of a station, and how to process data.

INSTALLATIONS

Selection of a device for measuring runoff depends on such factors as the peak runoff rate; distribution of runoff volume by categories of flow rate; absence or presence of sediment or woody trash, or both, in the flow; whether backwater submergence will affect flow through the device; icing conditions; foundation conditions; material availability; and economics.

The term "runoff" normally is used to distinguish surface flows from ground-water contributions to a streamflow. This distinction generally is derived through analysis of hydrographs since independent measurement is not feasible. The terms "runoff" and "streamflow" are used interchangeably here. They refer to all the flow, regardless of origin, that passes through the control at the point of measurement.

Selection of Gaging Station Control

In open channels, a control is a cross section or length of reach above which the water level is a stable index of the discharge rate. All sections have equal capacity to pass a flow. In natural streams the control may shift from one point to another with changes in stage. For use as a runoff station, a control must be selected, altered, or constructed to provide a stable head-discharge relationship.

Many conditions influence selection of a control for flow measurements. The ultimate objective is to provide a stable relationship between the depth of water and the rate of flow. Since the rate of flow equals the product of average velocity and the cross-sectional area, controls should be selected for stability of cross section and such factors as slope; configuration; channel roughness; and absence of tailwater, which affects velocity.

Quantitative evaluation of flow is easier if the flow passes from subcritical to supercritical around the control section. Precalibrated devices use this advantage. Natural controls that maintain critical flow at all stages are unusual. They are selected, therefore, to provide subcritical velocities at all depths since changes in depth are approximately equal to changes in specific energy. Measurement of flow at critical depth should be avoided since it presents so many difficulties. This sometimes can be accomplished by converting to subcritical flow through impoundment or manipulation of the channel gradient. The flow subsequently can pass through critical downstream of the point of head measurement.

For some purposes the control must be located so that gaged streamflow represents the entire flow from the watershed—none escapes beneath or around the control. Cutoff walls extending vertically in impervious strata and laterally in floodplains may be needed to prevent flow from bypassing the gaging station. Bypassing flow may be unimportant for some studies and at some locations.

Other considerations in selecting a control include capacity needed for major flows; silt, ice, and debris content of expected flows; and structural requirements such as footings and protection against frost heaving. Controls are classified herein as (1) precalibrated devices, (2) existing structures adapted to calibration, and (3) natural controls, such as cross sections or channel reaches with suitable hydraulic characteristics for calibration. Procedures for each class will be given.

A flow chart to assist in selecting a runoff measuring device is shown in figure 2.1. Although other criteria besides size may be used in selection, the use of size is convenient. Thus, the primary key to the flow chart is the maximum discharge rate to be selected. The flow chart provides paths based on the absence or presence of debris, type of flow in the approach channel, whether ponding is allowed, whether backwater problems are anticipated, and, finally, the type of stream and flow characteristics (perennial streams have relatively long, constant requirements for measuring base flow). Although a flow chart is a quick reference, the following material should be checked carefully to ensure selection of the best measuring device for the flow conditions encountered.

Precalibrated Devices for Measuring Runoff

Devices for measuring runoff usually are selected to meet specific needs at each location. Runoff should be measured as accurately as possible at low, medium, and high rates of discharge. Flow occurs on some watersheds only as the result of occasional storms of high intensity. Other watersheds have continuous flow.

Wherever possible, precalibrated devices should be used to gage the entire flow. For large watersheds, precalibrated devices large enough to gage high rates of flow may not be feasible. Instead, a precalibrated device may be needed to measure low flows and current-meter measurements may be needed to establish the high part of the stage-discharge relationship.

Weirs

A weir is a low dam or overflow structure built across an open channel. It has a specific size and shape with a unique free-flow, head-discharge relationship. The edge or surface over which the water flows is called the crest. Discharge rates are determined by measuring the vertical distance from the crest to the water surface in the pool upstream from the crest. When the water level in the downstream channel is sufficiently below the crest to allow free aeration of the nappe (the thin sheet of water falling over the crest), the flow is said to be free. If the nappe is partially under water, the weir is said to be submerged and the head-discharge relationship may not be unique. If submergence is permitted, special head-discharge computations may be necessary and accuracy of measuring the flow may be reduced. Do not install weirs for measuring runoff sites where concentrations of sediment exceed low values.

Many formulas and shapes and sizes of weirs are used to compute the discharge rate. Some commonly used weirs will be described here.

Sharp-Edged Weirs

Kindsvater and others (*22*) developed a method for computing rates of flow over rectangular, sharp-edged weirs. Their method includes both suppressed and unsuppressed free-discharge weirs with correction factors for approach velocity. These factors change with the ratio of crest length to approach channel width and with the ratio of head-to-crest height above the bottom of the approach channel or weir box.

V-notched Weirs

Triangular or V-notched weirs measure low discharges more accurately than horizontal weirs. The V-notch is usually a 90° opening with the sides of the notch inclined 45° with the vertical. The approach velocity can be neglected if the minimum distance from the weir to the channel banks is at least twice the head and if the minimum distance from the channel bottom to the crest is at least twice the head.

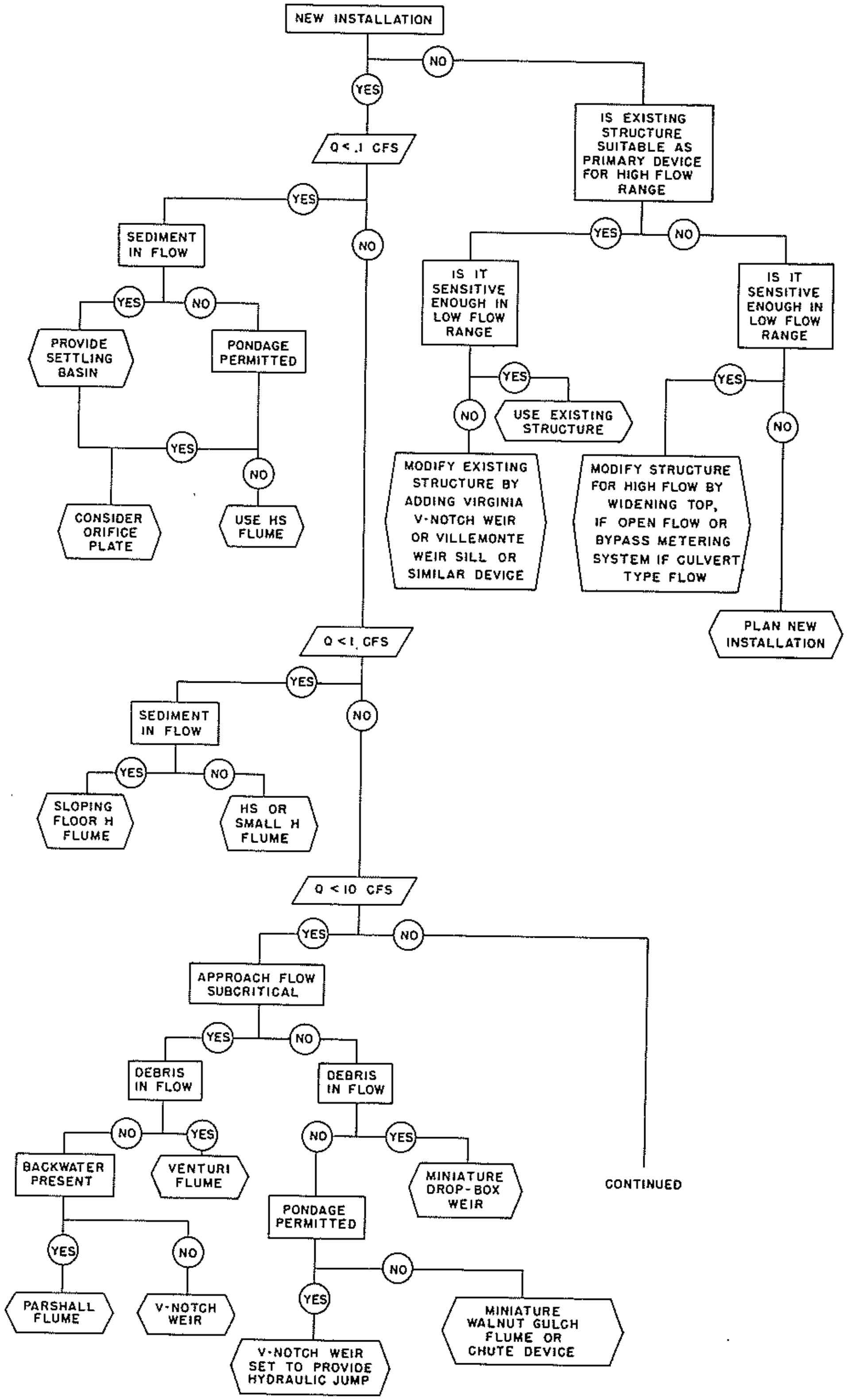

FIGURE 2.1.—Selection guide for measuring runoff.

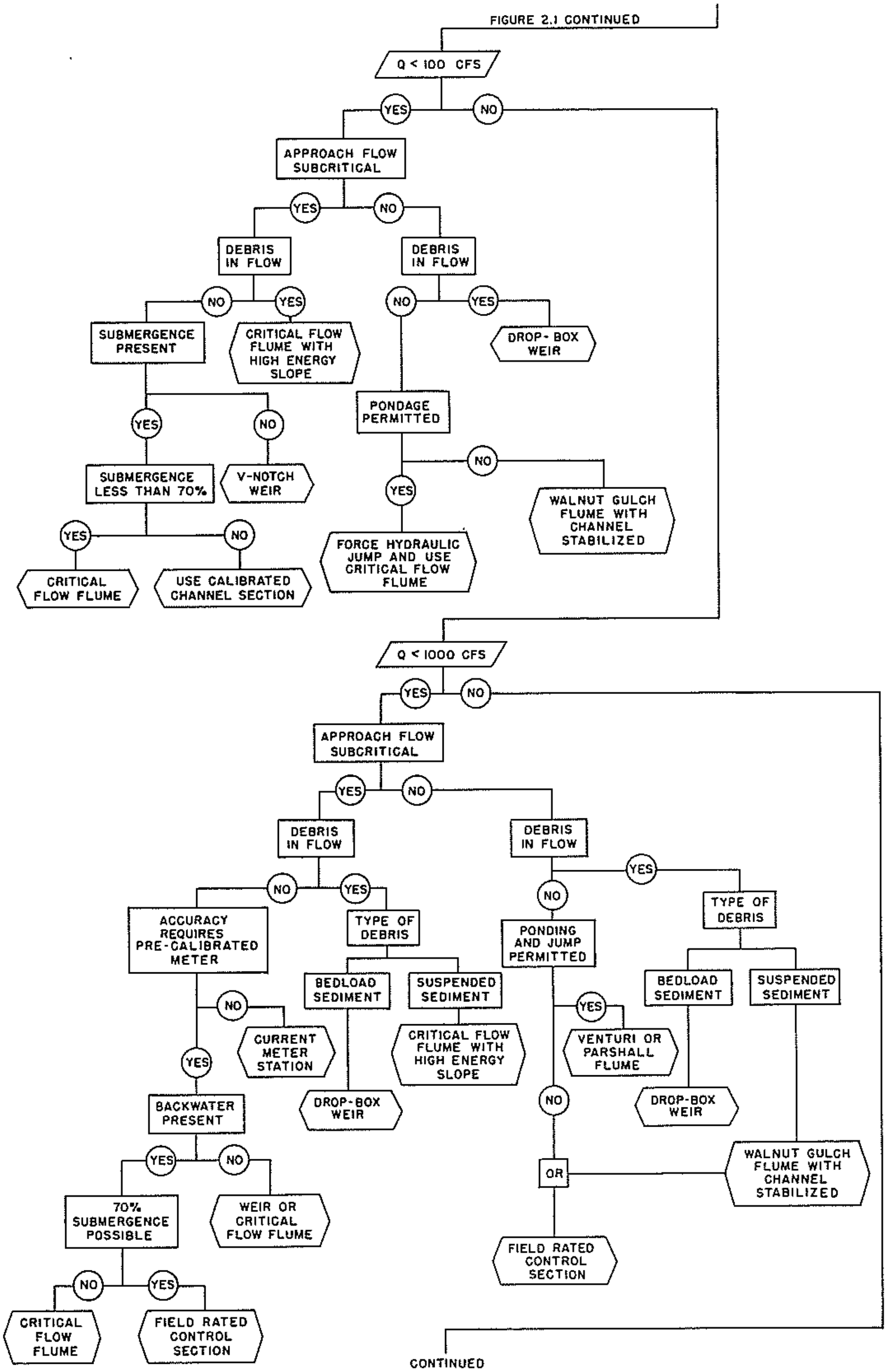

FIGURE 2.1.—Selection guide for measuring runoff.

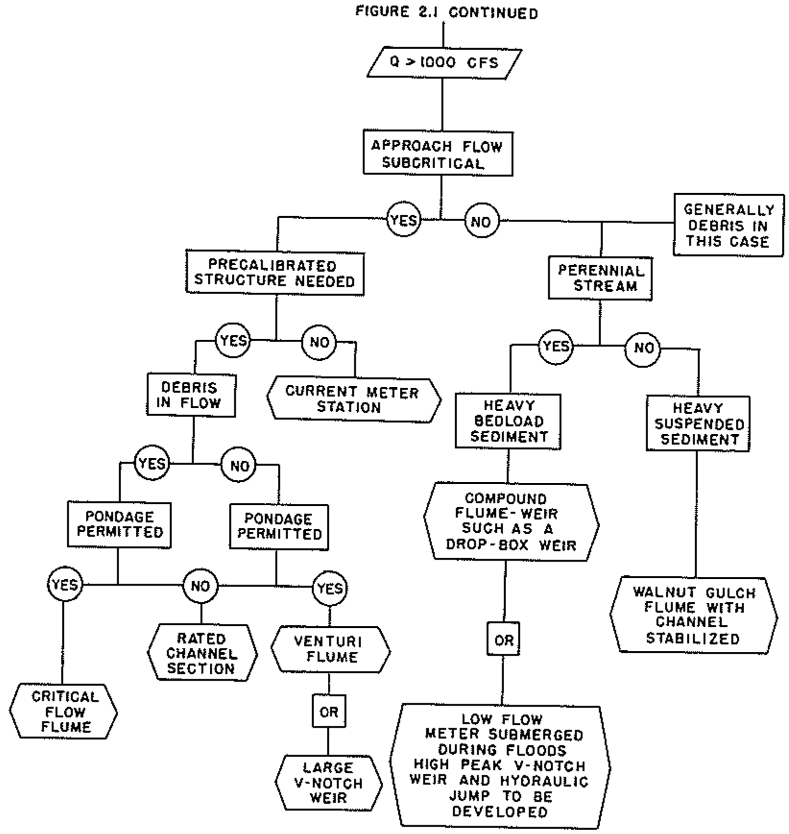

FIGURE 2.1.—Selection guide for measuring runoff.

Broad-Crested V-Notched Weirs

Sharp-crested V-notched weirs are difficult to maintain if they are used to obtain discharge records for long periods. They may become dull due to impact from ice or drift (*11*). Broad-crested V-notched weirs, over which ice and debris pass without damage, are desirable for measuring runoff over long periods. These are called triangular weirs.

Triangular weirs with 2-to-1, 3-to-1, 5-to-1, and 10-to-1 crests measure flows larger than 1,000 ft^3/s (7.9 l/s). A 3-to-1 triangular weir installation is shown in figure 2.2. The recorder for water stages is located on a stilling well 10 feet (3 m) upstream from the weir. A reasonably straight channel that is level for 50 feet (15 m) above the weir is essential for accuracy. The notch must be 6 inches (15.2 cm) above the bottom of the channel.

Cross-sectional dimensions are given in figure 2.3. A weir of this size needs a substantial apron of concrete for about 12 feet (3.7 m) downstream from the weir, 2 feet (0.61 m) below the notch, and 20 feet (61 m) across the channel. A 3-foot (0.91 m) (plus) end cutoff wall also is needed to prevent the weir from being undermined. The middle 10-foot (3 m) width of this apron is level, and the two 5-foot (1.5 m) sides are sloped slightly more than the weir crest.

Calibration of these weirs, as given in table 2.1, is affected by the approach velocity. The cross-sectional area of the approach channel 10 feet (3 m) upstream from the weir at the point of recording gage heights, is a measure of the approach velocity. Table 2.1 supplies discharge figures at each foot of head from 1 to 6 feet (0.3 to 1.8 m) for several areas of cross section. For example, the discharge for a 3-to-1 weir at 2-foot (0.61 m) head and a section area of 40 ft^2 (3.7 m^2) is 48.7 ft^3/s (380 cm^3/s). For an area of 62 ft^2 (5.76 m^2) the discharge is 47.5 ft^3/s (379 cm^3/s).

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FIGURE 2.2.—Installed triangular weir with 3-to-1 side slopes, recorder shelter, and stilling well.

The basic expression for discharge through a V-notch neglecting velocity of approach is:

$$Q = C' \tan \frac{\theta}{2} H^{2.5} \qquad (2\text{–}1)$$

$$= C\frac{8}{15}\sqrt{2g} \tan \frac{\theta}{2} H^{2.5}$$

where

Q = discharge (ft³/s);

C = coefficient correcting for energy loss and contraction of the jet;

C' = $\frac{8}{15}\sqrt{2g}\, C$;

θ = total angle of V-notch opening (for 90°, tan 0/2 – 1.0);

H = head above the lowest point of the V-notch (ft); and

g = gravitational acceleration (ft²/s).

Most engineers consider the experimental work of Cone (*11*) very reliable. His formula for the 90° V-notch weir is:

$$Q = 2.49\, H^{2.48} \qquad (2\text{–}2)$$

where

Q = discharge (ft³/s); and

H = head above lowest point of V-notch (ft).

Equation 2.2 can be generalized for values of 0 between 60° and 90° by adding the $\tan \frac{\theta}{2}$ term or

$$Q = 2.49 \tan \frac{\theta}{2} H^{2.48} \qquad (2\text{–}3)$$

for heads between 0.2 and 2.0 feet. The same equation in metric units would be:

$$Q_m = 1.342 \tan \frac{\theta}{2} H_m{}^{2.48} \qquad (2\text{–}4)$$

where

Q_m = discharge (cubic meters per second); and

H_m = head above the lowest point in the V-notch (meters).

Likewise, this equation is accurate (1 percent) for 0 between 60° and 90° with heads between 0.06 and 0.6 meter.

For angles smaller than 60°, the experimental results reported by Lenz (*24*) should be used. The water-discharge equation without any correction for temperature is:

$$Q_f = \left(2.395 + \frac{N}{H^e}\right) \tan \frac{\theta}{2} H^{2.5} \qquad (2\text{–}5)$$

For a water temperature of 70°:

$$N = 0.035 + 0.033 \left(\tan \frac{\theta}{2}\right)^{-0.8} \text{ and}$$

$$e = 0.2475 \left(\tan \frac{\theta}{2}\right)^{0.09} + 0.340 \left(\tan \frac{\theta}{2}\right)^{-0.035}$$

This formula is recommended for 0 angles between 28° and 90°. The value of N/H_f^e always should be equal to or less than 0.09. If the weir is installed with complete contraction, the approach velocity will be low and can be neglected in the head-discharge relationship.

The value of discharge, Q, equation 2–5, expressed in cubic meters per second would be:

$$Q_m = \left(1.322 + \frac{0.522\,N}{3.281^e\,H_m{}^e}\right) \tan \frac{\theta}{2} H_m{}^{2.5} \qquad (2\text{–}6)$$

where

H_m = head in meters;

N $= 0.035 + 0.033 \left(\tan\frac{\theta}{2}\right)^{-0.8}$; and

e $= 0.2475 \left(\tan\frac{\theta}{2}\right)^{0.09} + 0.340 \left(\tan\frac{\theta}{2}\right)^{0.035}$

The term $0.522\,N/(3.281^e H_m{}^e)$ always should have a value equal to or less than 0.05.

The stage-discharge relation is approximated by use of the equations in figure 2.3. Whenever practical, make current-meter measurements to check this calibration. Any large cutting or filling changes in the approach cross section may require a revision of the calibration.

Rating values for heads between 0.2 and 0.8 foot may be obtained by

$$Q = \left[2.51 + 0.0066 \tan \frac{\theta}{2} + \left[0.3292 + 0.5074/\tan \frac{\theta}{2}\right] \log_{10} H\right] \tan \frac{\theta}{2} H^{2.5} \qquad (2\text{–}7)$$

where

Q = discharge, cubic feet per second (ft³/s);

θ = Total angle of V notch ($\tan \frac{\theta}{2} = S$ in fig. 2.3);

and

H = head above the point of zero flow on the V-notch, feet.

This equation is based on full-scale tests with negligible approach velocity. These tests were run in the SEA hydraulic laboratory at Stillwater, Okla. Tan $\frac{\theta}{2}$ varied between 2 and 10. The same equation for discharge in cubic meters per second for H_m between 0.06 and 0.3 meter is:

$$Q_m = 1.4795 + 0.003644 \tan \frac{\theta}{2} + \frac{0.1445}{\tan \frac{\theta}{2}} + \left(0.18175 + \frac{0.28013}{\tan\frac{\theta}{2}}\right) \mathrm{Log}_{10} H_m \tan \frac{\theta}{2} H_m{}^{2.5} \qquad (2\text{–}8)$$

Drop-Box Weirs

The drop-box weir was modeled at Washington State University. Use of this weir to improve the accuracy of measuring water under heavy sediment load and varying conditions of approach was reported by Copp and Tinney

TABLE 2.1—*Discharge values corresponding to various cross-sectional areas of the channel of approach 10 feet upstream from center of crest for triangular weirs*[1]

2:1 TRIANGULAR WEIRS

1-foot head		2-foot head		3-foot head		4-foot head		5-foot head		6-foot head	
Area	Dis-charge	Area	Dis-charge	Area	Dis-charge	Area	Dis-charge	Area	Dis-charge	Area	Dis-charge
Ft^2	Ft^3/s	Ft^2	Ft^3/s	Ft^2	Ft^3/s	Ft^2	Ft^3/s	Ft^2	Ft^3/s	Ft^2	Ft^3/s
6.0	5.60	13.0	41.3	26.0	132	46	270	72	500	102	900
6.2	5.58	13.2	40.6	26.2	129	47	259	73	480	103	840
6.4	5.56	13.4	40.0	26.4	127	48	250	74	464	104	790
6.6	5.54	13.6	39.4	26.6	125	49	244	75	450	105	750
6.8	5.52	13.8	38.9	26.8	124	50	239	76	437	106	725
7.0	5.51	14.0	38.4	27.0	123	52	231	77	428	107	706
7.2	5.51	14.2	37.9	27.5	118	54	225	78	420	108	692
7.4	5.50	14.4	37.5	28.0	116	56	220	79	413	109	682
7.6	5.49	14.6	37.2	28.5	114	58	216	80	408	110	675
7.8	5.48	14.8	36.9	29.0	112	60	213	82	400	112	660
8.0	5.47	15.0	36.6	30.0	108	62	210	84	393	114	647
8.5	5.45	15.2	36.4	32.0	104	64	207	86	387	116	636
9.0	5.44	15.4	36.2	34.0	102	66	205	88	382	118	627
9.5	5.43	15.6	36.0	36.0	100	68	203	90	378	120	620
10.0	5.42	15.8	35.8	38.0	98.0	70	201	95	370	125	605
11.0	5.41	16.0	35.6	40.0	97.0	72	200	100	364	130	592
12.0	5.40	16.5	35.2	42.0	96.0	74	199	105	359	135	581
13.0	5.40	17.0	34.8	44.0	95.0	76	198	110	354	140	572
14.0	5.39	17.5	34.5	46.0	94.0	78	197	115	350	145	565
15.0	5.39	18.0	34.3	48.0	93.5	80	196	120	346	150	560
20.0	5.38	19.0	33.9	50.0	93.0	82	195	125	343	160	548
25.0	5.37	20.0	33.6	55.0	92.0	84	194	130	341	170	541
30.0	5.36	21.0	33.4	60.0	91.0	86	193	140	337	180	537
35.0	5.36	22.0	33.2	70.0	90.5	88	192	150	334	200	528
40.0	5.35	23.0	33.1	80.0	90.0	90	191	160	331	220	523
45.0	5.35	24.0	33.0	90.0	89.5	100	189	180	328	240	520
50.0	5.35	25.0	32.9	100	89.0	110	187	200	325	270	514
60.0	5.34	30.0	32.5	120	88.8	120	186	230	323	300	510
70.0	5.34	35.0	32.2	140	88.6	130	185	260	321	350	508
80.0	5.34	40.0	32.0	160	88.4	140	185	300	320	400	506
		50.0	31.8	180	88.2	170	184	325	320	450	505
		60.0	31.7	200	88.0	200	183	350	319	500	504
		80.0	31.6	220	88.0	250	183	375	319	550	503
		110.0	31.6	240	88.0	300	183	400	318	600	502
		150.0	31.6	260	88.0	350	183	450	318	650	501

3:1 TRIANGULAR WEIRS

1-foot head		2-foot head		3-foot head		4-foot head		5-foot head		6-foot head	
8.0	8.37	20.0	57.3	40.0	188	70.0	425	110	715	160	1100
8.5	8.30	22.0	54.4	41.0	180	72.0	388	115	655	162	1075
9.0	8.25	24.0	52.7	42.0	175	74.0	373	120	625	164	1050
9.5	8.22	26.0	51.5	44.0	167	76.0	360	125	605	166	1025
10.0	8.19	28.0	50.8	47.0	159	78.0	352	130	590	168	1005
11.0	8.13	30.0	50.2	50.0	154	80.0	345	135	576	170	990
12.0	8.08	32.0	49.8	52.0	152	82.0	339	140	564	172	980
13.0	8.04	34.0	49.4	54.0	150	84.0	334	145	556	174	970
14.0	8.02	36.0	49.1	56.0	148	86.0	329	150	549	176	961
15.0	8.00	38.0	48.9	58.0	147	88.0	325	155	543	178	952
20.0	7.95	40.0	48.7	60.0	146	90.0	321	160	537	180	945
25.0	7.93	42.0	48.5	62.0	145	92.0	318	165	532	185	930

TABLE 2.1—*Discharge values corresponding to various cross-sectional areas of the channel of approach 10 feet upstream from center of crest for triangular weirs*[1]—Continued

3:1 TRIANGULAR WEIRS—Con.

1-foot head		2-food head		3-foot head		4-foot head		5-foot head		6-foot head	
Area	Dis-charge	Area	Dis-charge	Area	Dis-charge	Area	Dis-charge	Area	Dis-charge	Area	Dis-charge
Ft²	*Ft³/s*	*Ft²*	*Ft³/s*	*Ft²*	*Ft³/s*	*Ft²*	*Ft³/s*	*Ft²*	*Ft³/s*	*Ft²*	*Ft³/s*
30.0	7.92	44.0	48.3	64.0	144	94.0	315	170	528	190	912
35.0	7.92	46.0	48.2	66.0	143	96.0	312	175	524	200	890
40.0	7.92	48.0	48.1	68.0	143	98.0	310	180	521	210	870
45.0	7.92	50.0	48.0	70.0	142	100	308	185	518	220	858
50.0	7.92	52.0	47.9	72.0	142	105	304	190	515	230	847
60.0	7.92	54.0	47.8	74.0	141	110	300	195	512	240	838
70.0	7.92	56.0	47.7	76.0	141	115	297	200	510	250	830
80.0	7.92	58.0	47.6	78.0	140	120	295	210	508	260	822
		60.0	47.5	80.0	139	125	293	220	506	270	815
		62.0	47.5	82.0	139	130	291	230	504	280	810
		64.0	47.4	84.0	139	135	289	240	502	290	806
		66.0	47.4	86.0	138	140	287	250	500	300	803
		68.0	47.4	88.0	138	145	286	260	498	320	797
		70.0	47.3	90.0	138	150	285	270	496	340	791
		72.0	47.3	95.0	137	160	284	280	494	360	786
		74.0	47.3	100	137	170	283	290	493	380	782
		76.0	47.2	110	136	180	282	300	492	400	780
		78.0	47.2	120	135	190	281	350	490	450	776
		80.0	47.1	130	134	200	280	400	488	500	773
		85.0	47.1	140	134	250	277	450	486	550	770
		90.0	47.0	150	133	300	276	500	485	600	767
		100	47.0	200	133	400	275	550	484	650	765
		150	47.0	250	132	500	275	600	484		
5:1 TRIANGULAR WEIRS											
15.0	13.7	30	98.0	60	315	116	590	170	1070	230	1880
18.0	13.6	31	96.5	61	306	118	580	180	1015	240	1680
21.0	13.5	32	95.0	62	300	120	572	190	975	250	1615
30.0	13.4	33	93.5	63	295	122	565	200	950	260	1570
50	13.2	34	92.2	64	291	124	559	210	928	270	1530
60	13.3	35	91.2	65	287	126	554	220	910	280	1495
70	13.3	36	90.3	66	283	128	550	230	898	290	1465
80	13.3	37	89.5	67	280	130	546	240	888	300	1440
90	13.3	38	88.9	68	277	132	542	250	880	310	1424
100	13.3	39	88.4	69	274	134	538	260	871	320	1410
		40	88.0	70	272	136	534	270	864	330	1398
		42	87.2	72	268	138	531	280	858	340	1388
		44	86.5	74	264	140	528	290	853	350	1380
		46	85.9	76	261	145	522	300	850	360	1373
		48	85.4	78	259	150	518	310	847	370	1366
		50	85.0	80	257	155	513	320	844	380	1359
		55	84.1	84	253	160	510	330	842	390	1353
		60	83.5	88	249	170	504	340	840	400	1347
		65	83.0	92	246	180	499	350	838	410	1341
		70	82.7	96	244	190	494	360	836	420	1336
		75	82.4	100	243	200	490	370	834	430	1331
		80	82.2	110	240	210	487	380	832	440	1326
		85	82.0	120	237	220	485	390	830	450	1322
		90	81.8	140	234	230	483	400	828	500	1310

TABLE 2.1—*Discharge values corresponding to various cross-sectional areas of the channel of approach 10 feet upstream from center of crest for triangular weirs*[1]—Continued

5:1 TRIANGULAR WEIRS—Con.

1-foot head		2-food head		3-foot head		4-foot head		5-foot head		6-foot head	
Area	Discharge	Area	Discharge	Area	Discharge	Area	Discharge	Area	Discharge	Area	Discharge
Ft²	*Ft³/s*	*Ft²*	*Ft³/s*	*Ft²*	*Ft³/s*	*Ft²*	*Ft³/s*	*Ft²*	*Ft³/s*	*Ft²*	*Ft³/s*
		95	81.6	160	232	240	481	450	826	550	1290
		100	81.4	180	231	250	480	500	824	600	1283
		110	81.2	200	230	260	479	550	822	650	1278
		120	81.0	220	229	280	477	600	820	700	1275
		140	80.8	250	228	300	475	650	818	750	1272
		170	80.6	300	227	400	470	700	816	800	1270
		200	80.5	400	226	600	465	800	812	900	1270
		300	80.5	500	226	700	465	1000	805	1000	1270

[1] Based on hydraulic laboratory tests made by the Soil Conservation Service at Cornell University, Ithaca, N.Y.

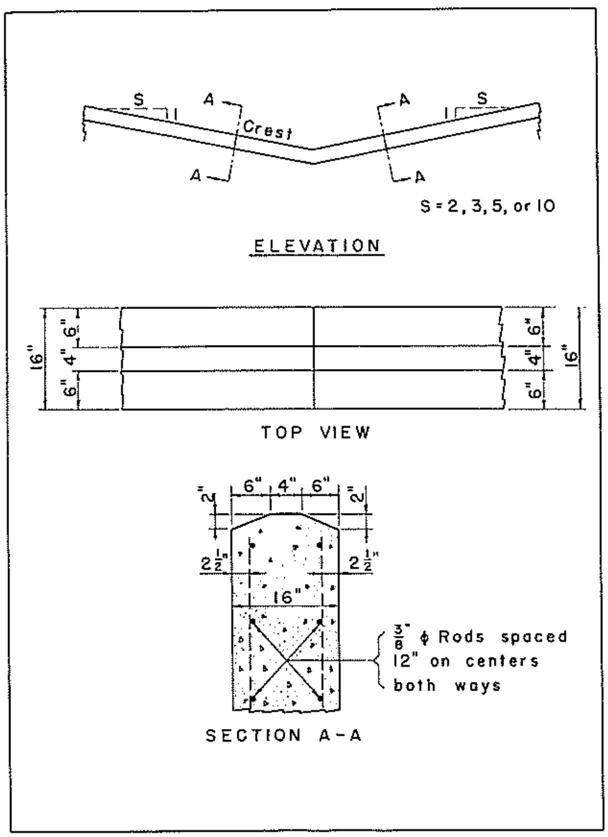


FIGURE 2.3.—Shape of a standard cross section of a triangular weir with a 16-inch (40.6 cm) crest.

(*13*). Johnson, Copp, and Tinney (*21*) reported on field experience with the drop-box weir in the Reynolds Creek Experimental Watershed. This weir is shown in figure 2.4.

The depth of the 1-to-1 V-notch weir was used to provide dimensionless proportion of the weir. The depth in the drop-box weir was 8 feet (2.4 m) or 1 D. The following dimensionless equation of discharge is recommended:

$$Q/D^{2.5} = 1.67\sqrt{g}\left(\frac{H_4}{D}\right)^{3.0} \qquad (2\text{–}9)$$

or:

$$Q = \frac{1.67\sqrt{g}H_4^{3.0}}{\sqrt{D}} \qquad (2\text{–}10)$$

where

Q = discharge, (ft³/s);
g = acceleration due to gravity (ft/s);
H_4 = head measured at gage 4 (fig. 2.3), feet; and
D = depth of 1-to-1 V-section, feet.

Equation 2–10 has a range of H_4/D values between 0.25 and 0.7. Precalibration tests are unnecessary for flow within this range. Field

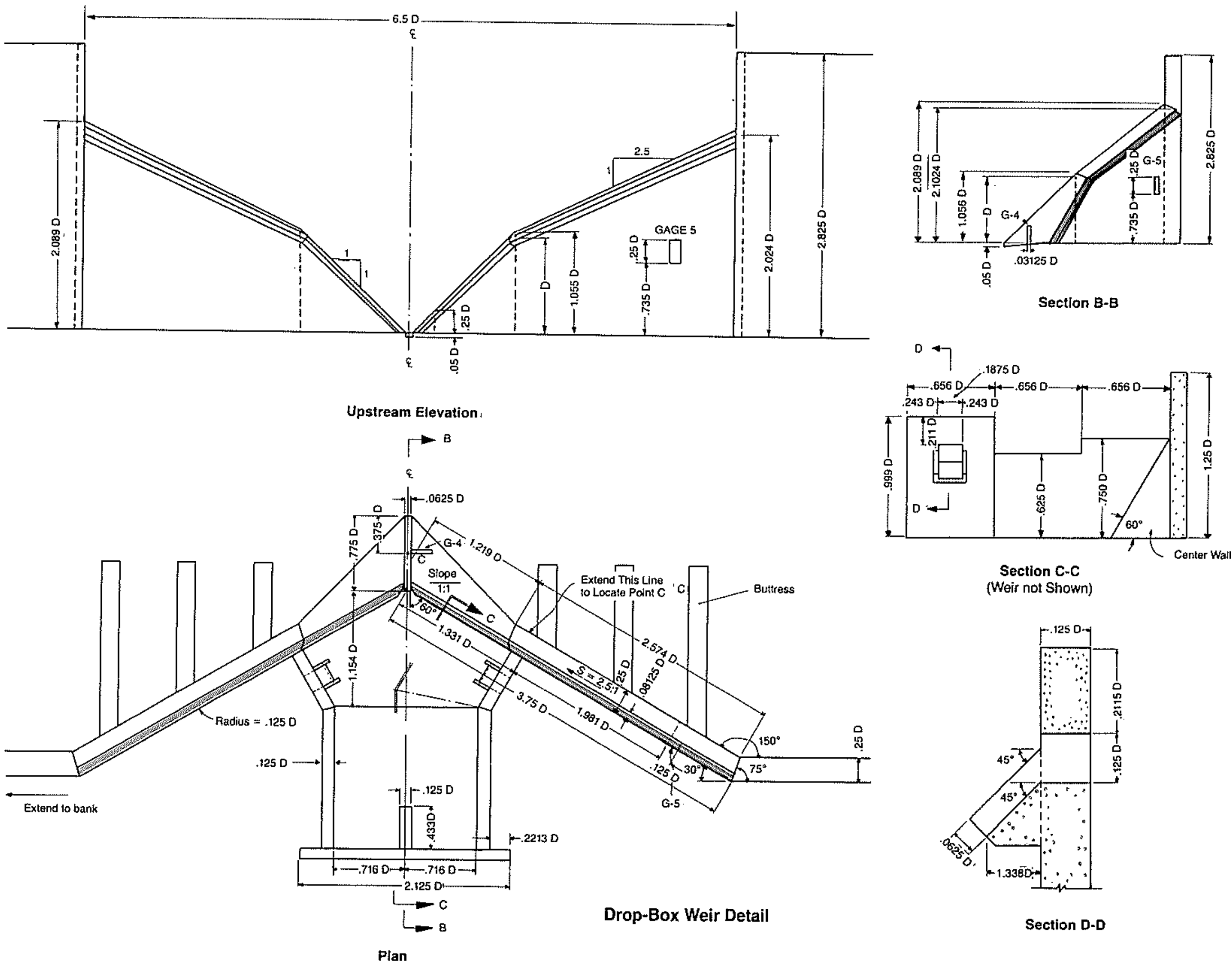


FIGURE 2.4.—Dimensions of the Salmon Creek weir of the Reynolds Creek Watershed, Idaho.

calibration tests are necessary for flows with H_4/D values less than 0.25.

Ratings above H/D value of 0.7 have the following limitations:

- Placement of the weir creates an approach angle (with the weir centerline) of 5° or less.
- The approach bed slope must be less than about 5 percent.
- Accuracy of the discharge rating must be within 5 percent.

The formula for gage 1 for H_4/D values between 0.7 and 1.3 is:

$$Q = 1.73\sqrt{g}\left(\frac{H_4}{D}\right)^{3.1} D^{2.5} = 1.73\sqrt{g}\,\frac{H_4^{3.1}}{0.6}. \quad (2\text{–}11)$$

The formula for gage 2 for H_5/D values between 0.8^D and 1.7 is:

$$Q = 1.20\sqrt{g}\left(\frac{H_5}{D}\right)^{1.9} D^{2.5} = 1.20\sqrt{g}\,H_5^{1.9}\,D^{0.6}. \quad (2\text{–}12)$$

Periodic maintenance will be required to keep the cage openings free from sediment and to keep the box sidewall orifice from becoming blocked with debris.

The estimated capacity of Salmon Creek was 7,000 ft³/s (19.8 m^3/s). No model data were reported, however, in this range of discharge. The value of D can be determined for designing by using the maximum discharge in the following formula:

$$D = \frac{[Q]^{0.4}}{6.82\sqrt{g}}.$$

This equation was obtained using the capacity of Salmon Creek, which is 7,000 ft³/s (19.8 m^3/s). The dimensions of the weir then may be determined using the Salmon Creek dimensions (fig. 2.3) divided by 8 feet (2.4 m) and multiplied by D.

Flumes

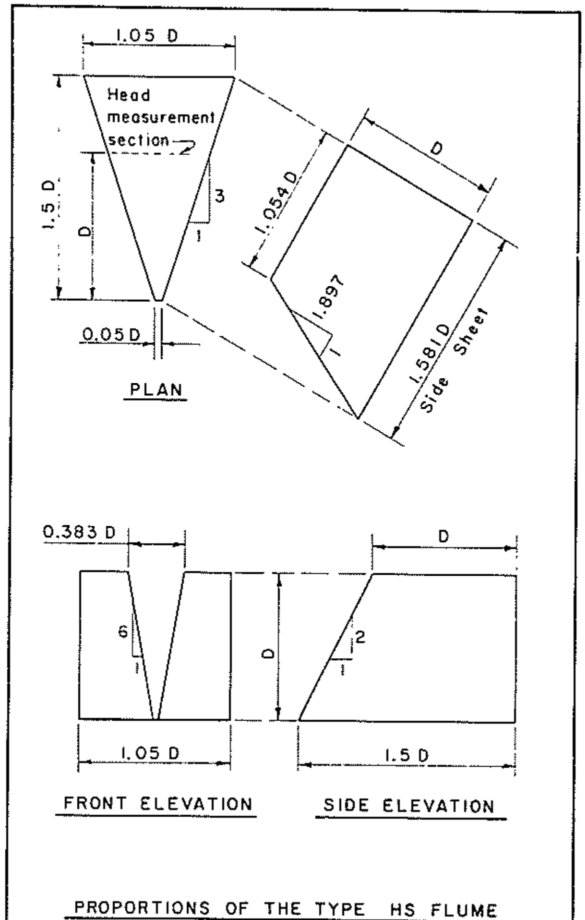
There are many types of precalibrated flumes. Most flumes use the principle of minimum energy or critical depth. Critical-depth flumes fall into two categories: (1) Flumes where critical depth occurs in region of curvilinear flow and (2) flumes for which the length dimension is such that critical flow occurs in a region where the flow lines are parallel or nearly parallel. Flumes that have little obstruction to the transport of these solids should be used to measure runoff where suspended and bedload solids are present in the flow. Where flow contains heavy suspended or bedload solids and velocity is reduced in the measuring section of the flume, material may deposit and interfere with performance of the flow. If reduction in velocity and deposit of sediment occur before the flow enters the flume, the flume will perform correctly.

Since *HS* flumes are designed to measure very small flows with a high degree of accuracy, construct them in strict accordance with the drawings and the following provisions. The slanting opening must be bound by straight edges and have precisely the dimensions shown on the drawings. The opening must lie in a plane with an inclination of the exact degree shown on the drawings. Prepare detailed drawings, using proportional dimensions shown in figure 2.5, with care taken to maintain the dimensional tolerance. Construction details are given with the discussion on *H* flumes.

Use the discharge equations for *HS* flumes in figures 2.6 and 2.7. These equations contain the basic dimensions that define the control section of the flume. Table 2.2 gives ratings for *HS* flumes of various sizes.

H Flumes:

Measure runoff from watersheds where the maximum runoff ranges from 0.3 to 30 ft³/s (0.009 to 0.85 m³/s). An *H* flume 0.5 foot deep will gage a maximum flow of 0.3 ft³/s (0.009 m³/s). Table 2.3 gives ratings for *H* flumes of various sizes. *H*-type flume dimensions and flow capacities are shown in figure 2.8. To measure flows with the required accuracy, construct the flume in strict accordance with the drawing and the following provisions of these specifications. The slanting opening must be bound by straight edges and have precisely the dimensions shown on the drawing.



APPROXIMATE	CAPACITIES
DEPTH–D Feet	CAPACITY C.f.s.
0.4	0.085
0.6	0.23
0.8	0.47
1.0	0.82

FIGURE 2.5.—Dimensions, capacities, and construction tolerances of the *HS* flume.

Construction specifications for the *H*-type rate-measuring flumes are:

- Prepare detailed drawings, using the proportional dimensions shown in figure 2.8.
- Use only new materials of the best commercial quality in constructing *H* flume. These materials must be free from defects.
- Use sheet metal of galvanized open-hearth iron or copper-bearing steel.
- Make all structural angles of high-grade structural steel and galvanize them. These angles must be straight, and the surface of the legs must be flat.
- Fabricate the flume by following the best commercial practice in all details of construction. Make all joints and seams watertight and strong.
- Cut all plate edges straight and sharp. Do not warp the plates or distort them by cutting.
- Make the vertical sides of the flume from one sheet. The bottom plate must not contain more than one joint, and no portion of this joint should lie within 12 inches (30.5 cm) of the outlet opening. Any necessary joint in the bottom plate must be transverse to the longitudinal axis of the flume and must be made so that the joint is substantially flush. Make all dimensions for which tolerances are not indicated on the drawings within 1/4 inch (0.64 cm) of those given on the drawings.
- Form the slanting outlet opening so that its dimensions are precisely those shown on the drawing. The slopes on this drawing must be rigidly adhered to. Edges of the opening must be straight and smooth.
- Clamp the plates rigidly in position and get the proper dimensions and slopes before making the final connections. Make the side plates perpendicular to the bottom of the flume. All cross sections of the flume must be symmetrical about the longitudinal axis. All plates must be flat and show no appreciable warp, dent, or other distortion. No projections should occur on the inside of the flume. All joints must be solid and watertight.
- Carry out all operations affecting the dimensions of the outlet opening and the straightness of its edges. Follow good machine shop practices in all operations. The completed flume should not have deep tool marks, dents, or other blemishes.
- Before using, inspect the flume to confirm its compliance with the plans and specifications.
- Flumes 4.5 feet (1.37 m) deep can be constructed in the field if the proportional dimensions in figure 2.8 are used. The flow capacity of this flume is 84 ft^3/s (2.35 m^3/s). Reinforced concrete floor resting on two reinforced concrete footings—one across the upper face of the flume and the other about 1 foot (30.5 cm) in from the downstream edge—provides a substantial base for the flume walls. These walls

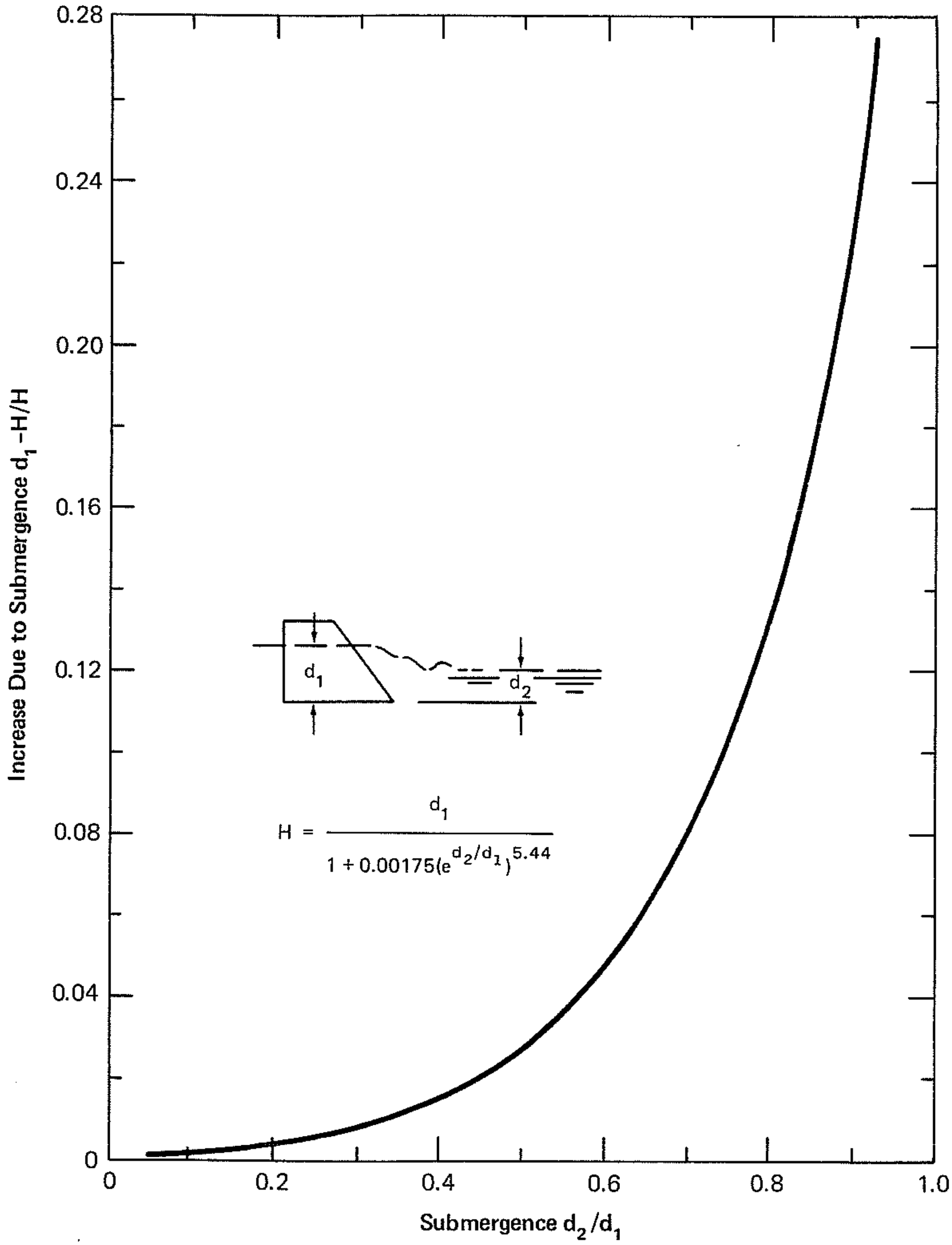
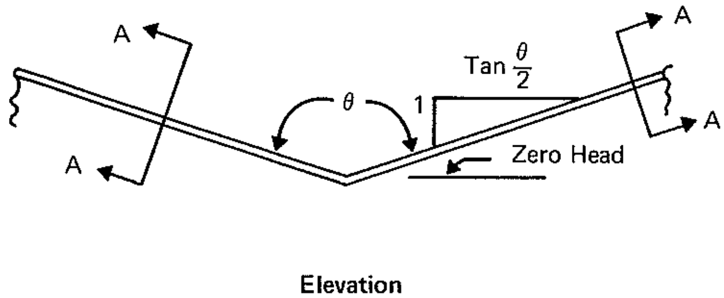
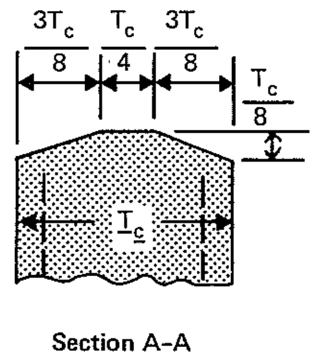


FIGURE 2.6.—Effect of submergence on the calibration of an *H* flume.





$$Q = \left(C_0 + C_1 \, \mathrm{Log}_{10} \frac{H}{T_c}\right) \sqrt{g} \tan\frac{\theta}{2} \left(H + \alpha \frac{v^2}{2g}\right)^{2.5}$$

α = 1.33

v = average velocity in cross section 10 feet (3.048 m) upstream from center of crest

H = head (water surface elevation – zero head) 10 feet (3.048 m) upstream from center of crest

T_c = crest thickness (16 in or 0.4064 m)

$\mathrm{Tan}\frac{\theta}{2}$	Range $\frac{H}{T_c}$	C_0	C_1
2	0.750 - 1.627	0.4798	0.06793
2	1.628 - 3.716	0.4906	0.01689
2	3.717 - 4.500	0.5142	-0.02450
3	0.750 - 2.379	0.4734	0.07075
3	2.380 - 3.755	0.4925	0.02002
3	3.756 - 4.500	0.5117	-0.01339
5	0.750 - 1.458	0.4766	0.07910
5	1.459 - 3.769	0.4879	0.01017
5	3.770 - 4.500	0.5669	-0.12692

FIGURE 2.7.—Discharge equations for triangular weirs.

may be either concrete or wood treated with preservative. In either case, angle iron forms the sloping edge of the flume. Sometimes the wood walls are lined with sheet metal. The wood should be 2-inch (5 cm) shiplap or tongue-and-grooved siding with watertight joints.

- *H* flumes using 1-on-8 sloping false floor are calibrated in table 2.3. Discharge equations for *H* flumes are listed in figure 2.6 and figure 2.7. These equations contain the basic dimensions that define the control section of the flume.

HL Flume:

The *HL* flume was designed to handle flow rates up to 117 ft³/s (3.28 m³/s). As shown in

TABLE 2.2—*Rating tables for HS flumes*[1]

[Discharge in cubic feet per second]

FLUME 0.4 FOOT DEEP

Head (feet)	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0	0	([2])	0.00016	0.00037	0.00064	0.00098	0.00141	0.00194	0.00258	0.00332
0.1	.00417	0.00509	.00608	.00717	.00837	.00968	.0111	.0126	.0143	.0161
0.2	.0179	.0200	.0221	.0244	.0268	.0293	.0320	.0348	.0378	.0409
0.3	.0441	.0475	.0511	.0548	.0586	.0626	.0668	.0711	.0756	.0803

FLUME 0.6 FOOT DEEP

Head (feet)	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0	0	([2])	0.00023	0.00053	0.00091	0.00138	0.00193	0.00259	0.00335	0.00421
0.1	.00517	0.00625	.00742	.00867	.0100	.0115	.0131	.0148	.0166	.0186
0.2	.0207	.0229	.0252	.0277	.0303	.0330	.0359	.0389	.0421	.0454
0.3	.0489	.0524	.0562	.0601	.0641	.0683	.0727	.0772	.0819	.0868
0.4	.0918	.0970	.102	.108	.114	.120	.126	.132	.138	.145
0.5	.152	.159	.166	.173	.181	.188	.196	.205	.213	.221

FLUME 0.8 FOOT DEEP

Head (feet)	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0	0	([2])	0.00030	0.00068	0.00116	0.00174	0.00242	0.00322	0.00412	0.00513
0.1	.00625	0.00750	.00884	.0103	.0118	.0135	.0153	.0172	.0193	.0214
0.2	.0237	.0262	.0287	.0314	.0343	.0373	.0404	.0437	.0471	.0506
0.3	.0543	.0582	.0622	.0664	.0708	.0752	.0799	.0847	.0897	.0949
0.4	.100	.106	.111	.117	.123	.129	.136	.142	.149	.156
0.5	.163	.170	.178	.186	.193	.202	.210	.218	.227	.236
0.6	.245	.254	.264	.273	.283	.293	.303	.314	.325	.336
0.7	.347	.358	.370	.381	.393	.406	.418	.431	.444	.457

FLUME 1.0 FOOT DEEP

Head (feet)	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0	0	([2])	0.00037	0.00083	0.00141	0.00209	0.00290	0.00384	0.00489	0.00606
0.1	.00736	0.00882	.0103	.0120	.0137	.0157	.0177	.0198	.0221	.0245
0.2	.0270	.0297	.0325	.0355	.0386	.0418	.0452	.0488	.0525	.0563
0.3	.0603	.0645	.0688	.0733	.0779	.0827	.0877	.0929	.0981	.104
0.4	.109	.115	.121	.127	.134	.140	.147	.154	.161	.168
0.5	.176	.183	.191	.199	.208	.216	.225	.233	.243	.252
0.6	.261	.271	.281	.291	.301	.312	.322	.333	.344	.355
0.7	.367	.379	.391	.403	.416	.428	.441	.454	.468	.481
0.8	.495	.509	.524	.538	.553	.568	.583	.599	.614	.630
0.9	.646	.663	.680	.697	.714	.731	.749	.767	.785	.803

[1] Rating derived from tests made by the U.S. Department of Agriculture's Soil Conservation Service at Washington, D.C., and Minneapolis, Minn. Table prepared April 1941.

[2] Trace.

TABLE 2.3—*Rating tables for H flume*[1]

[Discharge in cubic feet per second]

FLUME 0.5 FOOT DEEP

Head (ft)	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0	0	(2)	0.0004	0.0009	0.0016	0.0024	0.0035	0.0047	0.0063	0.0080
0.1	.0101	.0122	.0146	.0173	.0202	.0233	.0267	.0304	.0343	.0385
0.2	.0431	.0479	.0530	.0585	.0643	.0704	.0767	.0834	.0905	.0979
0.3	.1057	.1139	.1224	.1314	.1407	.1505	.1607	.1713	.1823	.1938
0.4	.205	.217	.230	.244	.257	.271	.285	.300	.315	.331

FLUME 0.75 FOOT DEEP

Head (ft)	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0	0	(2)	0.0006	0.0013	0.0022	0.0032	0.0046	0.0061	0.0080	0.0101
0.1	.0126	.0151	.0179	.0210	.0242	.0278	.0317	.0358	.0403	.0451
0.2	.0501	.0555	.0612	.0672	.0735	.0802	.0872	.0946	.1023	.1104
0.3	.119	.128	.137	.146	.156	.167	.177	.188	.199	.211
0.4	.224	.237	.250	.263	.277	.291	.306	.321	.337	.353
0.5	.370	.388	.406	.424	.443	.462	.482	.502	.523	.544
0.6	.566	.588	.611	.635	.659	.683	.708	.734	.760	.786
0.7	.813	.841	.869	.898	.927	.957	----------	----------	----------	----------

FLUME 1.0 FOOT DEEP

Head (ft)	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0	0	(2)	0.0007	0.0017	0.0027	0.0040	0.0056	0.0075	0.0097	0.0122
0.1	.0150	.0179	.0211	.0246	.0284	.0324	.0367	.0413	.0462	.0515
0.2	.0571	.0630	.0692	.0758	.0827	.0900	.0976	.1055	.1138	.1226
0.3	.132	.141	.151	.161	.172	.183	.194	.206	.218	.231
0.4	.244	.257	.271	.285	.300	.315	.331	.347	.364	.381
0.5	.398	.416	.434	.453	.472	.492	.512	.533	.554	.576
0.6	.598	.621	.644	.668	.692	.717	.743	.769	.796	.823
0.7	.851	.880	.909	.939	.969	1.000	1.031	1.063	1.096	1.129
0.8	1.16	1.20	1.23	1.27	1.30	1.34	1.38	1.41	1.45	1.49
0.9	1.53	1.57	1.61	1.66	1.70	1.74	1.78	1.83	1.87	1.92

FLUME 1.5 FEET DEEP

Head (ft)	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0	0	(2)	0.0011	0.0023	0.0039	0.0057	0.0078	0.0103	0.0131	0.0164
0.1	.0200	0.0237	.0276	.0319	.0365	.0414	.0467	.0523	.0582	.0645
0.2	.0711	.0780	.0854	.0931	.1011	.1095	.1183	.1275	.1371	.1470
0.3	.157	.168	.179	.191	.203	.215	.228	.241	.255	.269
0.4	.283	.298	.314	.330	.346	.363	.380	.398	.416	.435
0.5	.454	.473	.493	.514	.535	.557	.579	.601	.624	.648
0.6	.672	.697	.722	.747	.773	.800	.827	.855	.883	.912
0.7	.942	.972	1.002	1.033	1.065	1.097	1.130	1.163	1.197	1.231
0.8	1.27	1.30	1.34	1.38	1.41	1.45	1.49	1.53	1.57	1.61
0.9	1.65	1.69	1.73	1.78	1.82	1.86	1.91	1.95	2.00	2.05
1.0	2.09	2.14	2.19	2.24	2.30	2.35	2.40	2.45	2.50	2.56
1.1	2.61	2.67	2.73	2.78	2.84	2.90	2.96	3.02	3.08	3.14
1.2	3.20	3.27	3.33	3.39	3.46	3.52	3.59	3.66	3.73	3.80
1.3	3.87	3.94	4.01	4.08	4.15	4.22	4.30	4.37	4.45	4.52
1.4	4.60	4.68	4.76	4.84	4.92	5.00	5.08	5.16	5.24	5.33

See footnotes at end of table.

TABLE 2.3—*Rating tables for H flume*—Continued

[Discharge in cubic feet per second]

FLUME 2.0 FEET DEEP

Head (ft)	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0	0	([2])	0.0014	0.0031	0.0050	0.0073	0.0100	0.0130	0.0166	0.0205
0.1	.0248	0.0293	.0341	.0392	.0447	.0505	.0567	.0632	.0701	.0774
0.2	.0850	.0930	.1015	.1103	.1195	.1290	.1390	.1494	.1602	.1714
0.3	.183	.195	.207	.220	.234	.248	.262	.276	.291	.307
0.4	.323	.339	.356	.374	.392	.410	.429	.448	.468	.488
0.5	.509	.530	.552	.574	.597	.620	.644	.668	.693	.719
0.6	.745	.771	.798	.826	.854	.882	.911	.941	.971	1.002
0.7	1.03	1.07	1.10	1.13	1.16	1.20	1.23	1.27	1.30	1.34
0.8	1.38	1.42	1.46	1.49	1.53	1.57	1.62	1.66	1.70	1.74
0.2	1.78	1.83	1.87	1.92	1.96	2.01	2.06	2.10	2.15	2.20
1.0	2.25	2.30	2.35	2.40	2.45	2.51	2.56	2.62	2.67	2.73
1.1	2.78	2.84	2.90	2.96	3.02	3.08	3.14	3.20	3.26	3.32
1.2	3.38	3.45	3.51	3.58	3.65	3.71	3.78	3.85	3.92	3.99
1.3	4.06	4.13	4.20	4.28	4.35	4.43	4.50	4.58	4.66	4.74
1.4	4.82	4.90	4.98	5.06	5.14	5.23	5.31	5.40	5.48	5.57
1.5	5.65	5.74	5.83	5.92	6.01	6.11	6.20	6.29	6.38	6.48
1.6	6.58	6.67	6.77	6.87	6.97	7.07	7.17	7.27	7.37	7.47
1.7	7.58	7.68	7.79	7.90	8.00	8.11	8.22	8.33	8.44	8.56
1.8	8.67	8.78	8.90	9.01	9.13	9.24	9.36	9.48	9.60	9.72
1.9	9.85	9.97	10.09	10.21	10.34	10.47	10.60	10.72	10.85	10.98

FLUME 2.5 FEET DEEP

Head (ft)	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0	0	([2])	0.0018	0.0038	0.0061	0.0089	0.0121	0.0158	0.0200	0.0247
0.1	.0298	0.0350	.0406	.0465	.0528	.0595	.0666	.0741	.0820	.0903
0.2	.0990	.1081	.1176	.1275	.1379	.1486	.1597	.1713	.1834	.1960
0.3	.209	.222	.236	.250	.265	.280	.296	.312	.328	.345
0.4	.363	.381	.399	.418	.437	.457	.478	.499	.520	.542
0.5	.564	.587	.611	.635	.659	.684	.710	.736	.763	.790
0.6	.818	.846	.875	.904	.934	.965	.996	1.027	1.059	1.092
0.7	1.13	1.16	1.19	1.23	1.27	1.30	1.34	1.38	1.41	1.45
0.8	1.49	1.53	1.57	1.61	1.65	1.70	1.74	1.78	1.83	1.87
0.9	1.92	1.96	2.01	2.06	2.11	2.16	2.21	2.26	2.31	2.36
1.0	2.41	2.46	2.51	2.57	2.62	2.68	2.74	2.79	2.85	2.91
1.1	2.97	3.03	3.09	3.15	3.21	3.27	3.33	3.40	3.46	3.53
1.2	3.59	3.66	3.73	3.80	3.86	3.93	4.00	4.07	4.15	4.22
1.3	4.29	4.37	4.44	4.52	4.59	4.67	4.75	4.82	4.90	4.98
1.4	5.06	5.15	5.23	5.31	5.39	5.48	5.56	5.65	5.74	5.82
1.5	5.91	6.00	6.09	6.18	6.27	6.37	6.46	6.55	6.65	6.75
1.6	6.84	6.94	7.04	7.14	7.24	7.34	7.45	7.55	7.66	7.76
1.7	7.86	7.97	8.08	8.19	8.30	8.41	8.53	8.64	8.75	8.87
1.8	8.98	9.10	9.22	9.34	9.45	9.57	9.70	9.82	9.94	10.06
1.9	10.2	10.3	10.4	10.6	10.7	10.8	11.0	11.1	11.2	11.4

See footnotes at end of table.

TABLE 2.3—*Rating tables for H flume*—Continued

[Discharge in cubic feet per second]

FLUME 2.5 FEET DEEP—Con.

Head (ft)	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
2.0	11.5	11.6	11.8	11.9	12.0	12.2	12.3	12.5	12.6	12.7
2.1	12.9	13.0	13.2	13.3	13.5	13.6	13.8	13.9	14.1	14.2
2.2	14.4	14.5	14.7	14.8	15.0	15.1	15.3	15.5	15.6	15.8
2.3	16.0	16.1	16.3	16.4	16.6	16.8	17.0	17.1	17.3	17.5
2.4	17.6	17.8	18.0	18.2	18.3	18.5	18.7	19.1	19.1	19.2

FLUME 3.0 FEET DEEP

Head (ft)	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0	0	([2])	0.0021	0.0045	0.0073	0.0105	0.0143	0.0186	0.0234	0.0288
0.1	.0347	0.0407	.0471	.0538	.0610	.0686	.0766	.0851	.0939	.1032
0.2	.113	.123	.134	.145	.156	.168	.180	.193	.207	.220
0.3	.234	.249	.264	.280	.296	.312	.329	.347	.365	.383
0.4	.402	.421	.441	.462	.483	.504	.526	.549	.572	.596
0.5	.620	.644	.669	.695	.721	.748	.775	.803	.832	.861
0.6	.890	.920	.951	.982	1.014	1.047	1.080	1.113	1.147	1.182
0.7	1.22	1.25	1.29	1.33	1.36	1.40	1.44	1.48	1.52	1.56
0.8	1.60	1.65	1.69	1.73	1.78	1.82	1.86	1.91	1.96	2.00
0.9	2.05	2.10	2.15	2.20	2.25	2.30	2.35	2.41	2.46	2.51
1.0	2.57	2.62	2.68	2.73	2.79	2.85	2.91	2.97	3.03	3.09
1.1	3.15	3.21	3.27	3.34	3.40	3.46	3.53	3.60	3.66	3.73
1.2	3.80	3.87	3.94	4.01	4.08	4.15	4.23	4.30	4.37	4.45
1.3	4.53	4.60	4.68	4.76	4.84	4.92	5.00	5.08	5.16	5.24
1.4	5.33	5.41	5.50	5.58	5.67	5.76	5.84	5.93	6.02	6.11
1.5	6.20	6.30	6.39	6.48	6.58	6.67	6.77	6.87	6.96	7.06
1.6	7.16	7.26	7.36	7.47	7.57	7.67	7.78	7.88	7.99	8.10
1.7	8.20	8.31	8.42	8.53	8.64	8.75	8.87	8.98	9.10	9.21
1.8	9.33	9.45	9.56	9.68	9.80	9.92	10.05	10.17	10.29	10.41
1.9	10.5	10.7	10.8	10.9	11.0	11.2	11.3	11.4	11.6	11.7
2.0	11.9	12.0	12.1	12.3	12.4	12.6	12.7	12.8	13.0	13.1
2.1	13.3	13.4	13.6	13.7	13.9	14.0	14.2	14.3	14.5	14.6
2.2	14.8	14.9	15.1	15.3	15.4	15.6	15.7	15.9	16.1	16.2
2.3	16.4	16.6	16.7	16.9	17.1	17.2	17.4	17.6	17.8	17.9
2.4	18.1	18.3	18.5	18.7	18.8	19.0	19.2	19.4	19.6	19.8
2.5	19.9	20.1	20.3	20.5	20.7	20.9	21.1	21.3	21.5	21.7
2.6	21.9	22.1	22.3	22.5	22.7	22.9	23.1	23.3	23.5	23.7
2.7	23.9	24.1	24.3	24.5	24.7	24.9	25.2	25.4	25.6	25.8
2.8	26.0	26.2	26.5	26.7	26.9	27.1	27.4	27.6	27.8	28.0
2.9	28.3	28.5	28.7	28.9	29.2	29.4	29.7	29.9	30.1	30.4

See footnotes at end of table.

TABLE 2.3—*Rating tables for H flume*—Continued

[Discharge in cubic feet per second]

FLUME 4.5 FEET DEEP

Head (ft)	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0	0	(2)	0.0031	0.0066	0.0106	0.0154	0.0208	0.0269	0.0337	0.0413
0.1	.0496	0.0578	.0666	.0758	.0855	.0959	.1067	.1180	.1298	.1420
0.2	.155	.168	.182	.196	.211	.226	.242	.259	.276	.293
0.3	.311	.330	.349	.368	.388	.409	.430	.452	.474	.497
0.4	.520	.544	.569	.594	.620	.646	.673	.700	.728	.756
0.5	.785	.815	.845	.876	.907	.939	.972	1.005	1.039	1.073
0.6	1.11	1.14	1.18	1.122	1.25	1.29	1.33	1.38	1.41	1.45
0.7	1.49	1.53	1.58	1.62	1.66	1.71	1.75	1.80	1.84	1.89
0.8	1.94	1.99	2.04	2.09	2.14	2.19	2.24	2.29	2.35	2.40
0.9	2.45	2.51	2.56	2.62	2.68	2.74	2.79	2.85	2.91	2.98
1.0	3.04	3.10	3.16	3.22	3.29	3.35	3.42	3.49	3.55	3.62
1.1	3.69	3.76	3.83	3.90	3.97	4.04	4.12	4.19	4.27	4.34
1.2	4.42	4.50	4.58	4.65	4.73	4.81	4.89	4.98	5.06	5.14
1.3	5.22	5.31	5.39	5.48	5.57	5.66	5.74	5.83	5.92	6.02
1.4	6.11	6.20	6.29	6.39	6.48	6.58	6.68	6.77	6.87	6.97
1.5	7.07	7.17	7.27	7.37	7.48	7.59	7.69	7.80	7.90	8.01
1.6	8.12	8.23	8.34	8.45	8.56	8.68	8.79	8.90	9.02	9.14
1.7	9.25	9.37	9.49	9.61	9.73	9.85	9.98	10.10	10.22	10.35
1.8	10.5	10.6	10.7	10.8	11.0	11.1	11.2	11.4	11.5	11.6
1.9	11.8	11.9	12.0	12.2	12.3	12.5	12.6	12.8	12.9	13.0
2.0	13.2	13.3	13.5	13.6	13.7	13.9	14.1	14.2	14.4	14.5
2.1	14.7	14.8	15.0	15.2	15.3	15.5	15.6	15.8	15.9	16.1
2.2	16.3	16.4	16.6	16.8	16.9	17.1	17.3	17.4	17.6	17.8
2.3	18.0	18.1	18.3	18.5	18.7	18.8	19.0	19.2	19.4	19.6
2.4	19.7	19.9	20.1	20.3	20.5	20.7	20.9	21.0	21.2	21.4
2.5	21.6	21.8	22.0	22.2	22.4	22.6	22.8	23.0	23.2	23.4
2.6	23.6	23.8	24.0	24.2	24.4	24.6	24.9	25.1	25.3	25.5
2.7	25.7	25.9	26.1	26.4	26.6	26.8	27.0	27.2	27.4	27.7
2.8	27.9	28.1	28.4	28.6	28.8	29.0	29.3	29.5	29.7	30.0
2.9	30.2	30.4	30.7	30.9	31.2	31.4	31.7	31.9	32.2	32.4
3.0	32.7	32.9	33.2	33.4	33.7	33.9	34.2	34.4	34.7	35.0
3.1	35.2	35.5	35.8	36.0	36.3	36.6	36.8	37.1	37.4	37.7
3.2	37.9	38.2	38.5	38.8	39.0	39.3	39.6	39.9	40.2	40.5
3.3	40.8	41.2	41.3	41.6	41.9	42.2	42.5	42.8	43.1	43.4
3.4	43.7	44.0	44.3	44.6	44.9	45.2	45.5	45.8	46.1	46.4
3.5	46.8	47.1	47.4	47.7	48.0	48.3	48.6	49.0	49.3	49.6
3.6	49.9	50.3	50.6	50.9	51.2	51.6	51.9	52.2	52.6	52.9
3.7	53.2	53.6	53.9	54.3	54.6	54.9	55.3	55.6	56.0	56.3
3.8	56.7	57.0	57.4	57.7	58.1	58.4	58.8	59.2	59.5	59.9
3.9	60.2	60.6	61.0	61.3	61.7	62.1	62.4	62.8	63.2	63.6

See footnotes at end of table.

TABLE 2.3—*Rating tables for H flume*—Continued

[Discharge in cubic feet per second]

FLUME 4.5 FEET DEEP—Con.

Head (ft)	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
4.0	63.9	64.3	64.7	65.1	65.4	65.8	66.2	66.6	67.0	67.4
4.1	67.8	68.2	68.5	68.9	69.3	69.7	70.1	70.5	70.9	71.3
4.2	71.7	72.1	72.5	72.9	73.3	73.8	74.2	74.6	75.0	75.4
4.3	75.8	76.2	76.6	77.1	77.5	77.9	78.3	78.8	79.2	79.6
4.4	80.0	80.5	80.9	81.3	81.8	82.2	82.6	83.1	83.5	84.0

[1] Rating derived from tests made by the Soil Conservation Service at the Hydraulic Laboratory of the National Bureau of Standards using 1-on-8 sloping false floor.
[2] Trace.

figure 2.9, the *HL* flume may be built in several sizes. Only the 4-foot (1.2 m) *HL* flume is recommended because smaller flows are measured more accurately by *H* flumes. Construction is similar to that for a 4-foot (1.2 m) *H* flume. Calibration appears in table 2.4.

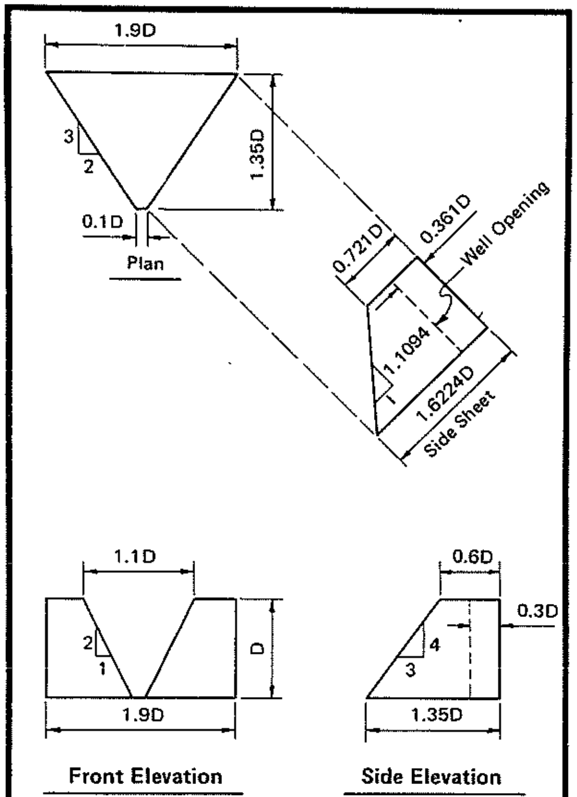
Discharge equations for *HL* flumes are listed in figures 2.6 and 2.7. These equations contain the basic dimensions that define the control section of the flume.

Venturi Flume:

Experiments on the Venturi flume were conducted in 1915 by Cone (*12*) at Colorado State University. This flume is characterized by a relatively long contracted section or throat. The floor of the throat is horizontal. Venturi flumes measure the discharge in an open channel by contracting the flow either by tapering the sidewalls of the flume or by changing the elevation of the floor of the flume, or both. These flumes require a measurement of depth at the entrance and at the constricted region. Later modifications using the properties of critical flow to define the flow properties in the throat under conditions of sufficient discharge required only a measurement of depth at the entrance. Critical-depth flumes usually are used in irrigation canals where the Froude number is less than 0.6 in the approach to the flume. Where sediment must be passed and the canal must be drained periodically, flat-bottomed flumes are more practical. For sediment movement, the velocities in the approach section of the flume should not be less than about 2.5 ft/s (0.76 m/s).

Ackers and Harrison (*1*) used critical-depth theory and proposed two methods of computing head loss in the throat to derive ratings for trapezoidal-throated flumes. Replogle (*35*) modified their boundary layer-thickness method to a boundary-drag-and-energy accounting method. He successfully predicted calibrations on a wide variety of throat and approach channel shapes including triangular, rectangular, and trapezoidal shapes, and combinations of these.

The head usually is measured in the entrance section to the flume (fig. 2.10). If we let *H* equal the maximum head in this section, the length of the entrance section and the throat section should be 2*H* or more. The throat section should not exceed about 20 times the minimum head of interest. Most experimenters agree that the convergence of each wall relative to the centerline in the transition section should not exceed 1:3. The throat should have a length of 2*H* to obtain parallel flow. The exit transition of 1:6 is recommended by Skogerboe and others (*39*). To avoid submergence, the downstream depth should not exceed two-thirds of the head for any range of flow. Deviation from these dimensions should be calibrated before use; otherwise, the flume should be constructed exactly the same as standard calibrated flume.



Proportions of the Type H Flume

Approximate Capacities	
Depth-D Feet	Capacity $FT.^3/s$
0.5	0.3+
0.75	1-
1.0	2
1.5	5+
2.0	11
2.5	19
3.0	30+

Note: For flumes less than 1 foot deep, the length of flume is made greater than 1.35D so that the float may be attached.

Flume Depth	Outlet Opening Dimension Tolerances		
	Bottom Width	Top Width	Depth
Feet	Inch	Inch	Inch
0.5	0.02	0.1	0.05
0.75	0.02	0.1	0.05
1.0	0.02	0.1	0.05
1.5	0.03	0.1	0.05
2.0	0.03	0.1	0.05
2.5	0.03	0.1	0.05
3.0	0.03	0.1	0.05
4.5	0.05	0.1	0.10

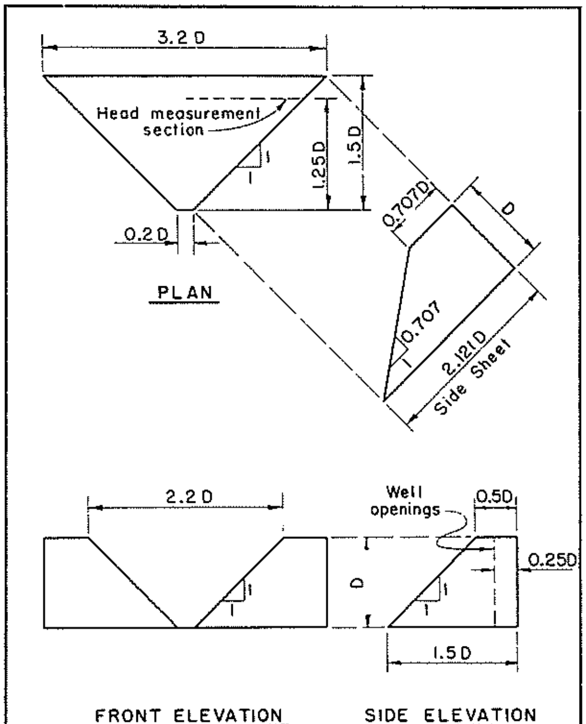
FIGURE 2.8.—*H* flume dimensions and capacities.

Parshall Flume:

This type of Venturi flume was developed in 1922 by the late Ralph L. Parshall in the hydraulic laboratory at Colorado State University. The main advantage of the Parshall flume is its relatively low loss of head. The head loss is only about a fourth of that needed to operate a weir having the same crest length. Parshall flumes are usually more expensive than weirs, however. Dimensions for various sizes are given in figures 2.11 and 2.12.

The discharge formula for small flumes was reported by Parshall (*29*) to be:

$$Q = 4\,WH_a^{\,1.522W^{0.026}} \qquad (2\text{–}14)$$



PROPORTIONS OF THE TYPE HL FLUME

APPROXIMATE CAPACITIES	
DEPTH-D Feet	CAPACITY C.f.s.
2.0	20
2.5	35
3.0	55
3.5	80
4.0	110

For this range in maximum flows it is better to use H-type flumes.

FIGURE 2.9.—*HL* flume dimensions and capacities.

TABLE 2.4.—*Rating tables for 4-foot HL flume*[1]

[Discharge in cubic feet per second]

Head (feet)	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0	0	([2])	0.005	0.012	0.020	0.029	0.039	0.050	0.062	0.075
0.1	.089	0.103	.119	.135	.152	.170	.190	.211	.232	.255
0.2	.278	.302	.327	.352	.378	.405	.434	.465	.497	.530
0.3	.565	.600	.635	.670	.705	.740	.780	.820	.860	.900
0.4	.940	.982	1.03	1.08	1.12	1.17	1.22	1.27	1.32	1.37
0.5	1.42	1.48	1.53	1.59	1.64	1.70	1.76	1.82	1.88	1.94
0.6	2.01	2.07	2.14	2.21	2.28	2.35	2.42	2.49	2.56	2.64
0.7	2.71	2.79	2.87	2.95	3.03	3.11	3.19	3.28	3.36	3.44
0.8	3.53	3.61	3.70	3.79	3.88	3.98	4.08	4.18	4.28	4.38
0.9	4.48	4.58	4.68	4.79	4.90	5.01	5.12	5.23	5.34	5.45
1.0	5.56	5.68	5.80	5.92	6.04	6.16	6.28	6.40	6.52	6.64
1.1	6.76	6.89	7.02	7.15	7.28	7.41	7.54	7.67	7.80	7.93
1.2	8.06	8.20	8.35	8.50	8.65	8.80	8.95	9.10	9.25	9.40
1.3	9.55	9.70	9.90	10.1	10.2	10.4	10.5	10.7	10.8	11.0
1.4	11.2	11.4	11.6	11.7	11.9	12.1	12.3	12.4	12.6	12.8
1.5	13.0	13.2	13.3	13.5	13.7	13.9	14.1	14.3	14.5	14.7
1.6	14.9	15.1	15.3	15.5	15.7	15.9	16.2	16.4	16.6	16.8
1.7	17.0	17.2	17.4	17.6	17.8	18.1	18.3	18.5	18.7	19.0
1.8	19.2	19.4	19.7	19.9	20.2	20.4	20.6	20.9	21.2	21.4
1.9	21.7	21.9	22.1	22.4	22.7	23.0	23.2	23.4	23.7	24.0
2.0	24.3	24.5	24.8	25.0	25.3	25.6	25.8	26.1	26.4	26.7
2.1	27.0	27.3	27.6	27.9	28.2	28.5	28.8	29.1	29.4	29.7
2.2	30.0	30.3	30.6	30.9	31.2	31.5	31.9	32.2	32.5	32.8
2.3	33.1	33.5	33.8	34.1	34.5	34.8	35.1	35.4	35.8	36.1
2.4	36.5	36.8	37.1	37.4	37.8	38.2	38.5	38.8	39.1	39.5
2.5	39.9	40.3	40.6	41.0	41.4	41.7	42.1	42.4	42.8	43.2
2.6	43.6	43.9	44.3	44.7	45.1	45.5	45.8	46.2	46.6	47.1
2.7	47.5	47.9	48.2	48.6	49.0	49.4	49.8	50.2	50.7	51.1
2.8	51.6	52.0	52.4	52.8	53.3	53.7	54.1	54.5	54.9	55.4
2.9	55.9	56.3	56.7	57.2	57.6	58.1	58.6	59.1	59.5	59.9
3.0	60.3	60.8	61.3	61.8	62.3	62.8	63.2	63.7	64.1	64.6
3.1	65.1	65.6	66.1	66.6	67.1	67.5	68.0	68.5	69.0	69.5
3.2	70.0	70.5	71.0	71.5	72.0	72.5	73.0	73.5	74.0	74.5
3.3	75.0	75.5	76.0	76.5	77.0	77.6	78.2	78.7	79.3	79.9
3.4	80.5	80.9	81.5	82.0	82.6	83.1	83.6	84.2	84.8	85.3
3.5	85.9	86.5	87.1	87.7	88.3	88.9	89.5	90.1	90.7	91.3
3.6	91.9	92.5	93.1	93.7	94.3	94.9	95.5	96.1	96.7	97.4
3.7	98.0	98.6	99.2	99.8	100	101	102	102	103	104
3.8	104	105	106	106	107	107	108	109	109	110
3.9	111	111	112	113	113	114	115	115	116	116
4.0	117	----------	----------	----------	----------	----------	----------	----------	----------	----------

[1] Rating derived from tests made at the National Bureau of Standards using flat floor.
[2] Trace.

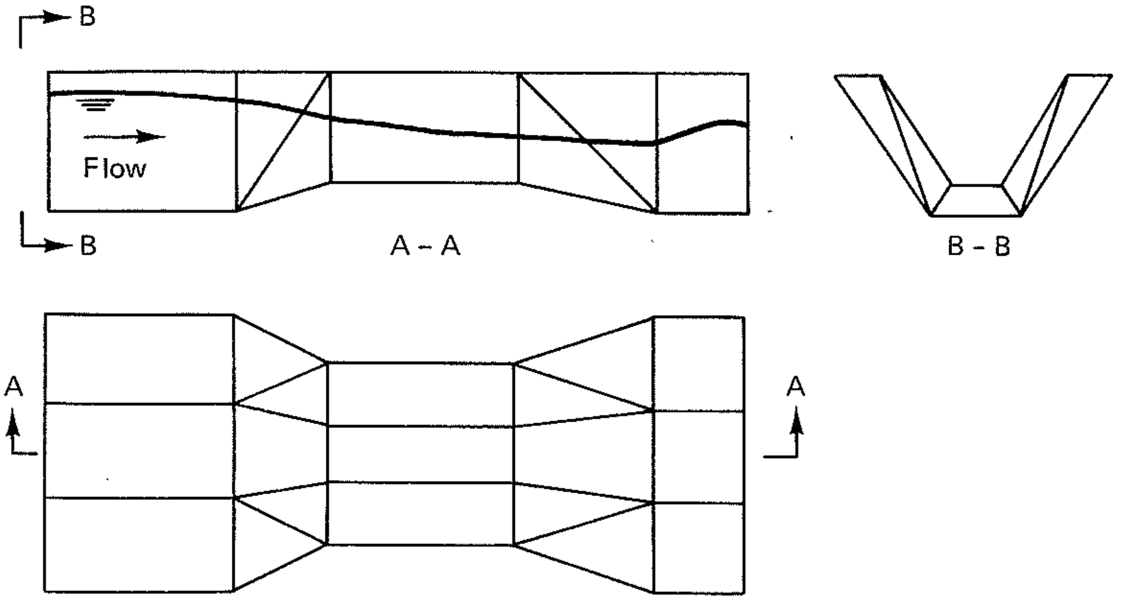


FIGURE 2.10.—Trapezoidal critical-depth flume.

where

Q = discharge, cubic feet per second;

W = throat width or length of crest (size of flume), feet; and

H_a = upper gage head [2/3(W/2+4) feet back from the crest], feet.

Equation 2–14 expressed in meters would be:

$$Q_m = 4\,[0.3048]^{2-1.57(W_m)^{0.026}}\,W_m H_{am}^{\;1.57(W_m)^{0.026}} \qquad (2\text{–}15)$$

where

Q_m = discharge (m^3/s);

W_m = throat width in meters;

H_{am} = upper gage head in meters [2/3($\frac{W}{2}$m+ 1.219) meters back from the crest].

Large flumes (10 to 40 ft, or 3 to 12 m) were modified by increasing the length and decreasing the side angle of the converging section. The general discharge formula for these flumes was reported by Parshall (*30*) to be:

$$Q = (3.6875\,W+2.5)\,H_a^{\;1.6} \qquad (2\text{–}16)$$

where

Q = discharge, cubic feet per second;

W = throat width (size of flume), feet; and

H_a = upper gage head (referenced to crest), feet.

Equation 2–16 expressed in meters would be:

$$Q_m = (2.29265\,W_m + 0.47376)\,H_{am}^{\;1.6} \qquad (2\text{–}17)$$

where

Q_m = discharge (m^3/s);

W_m = throat width in meters; and

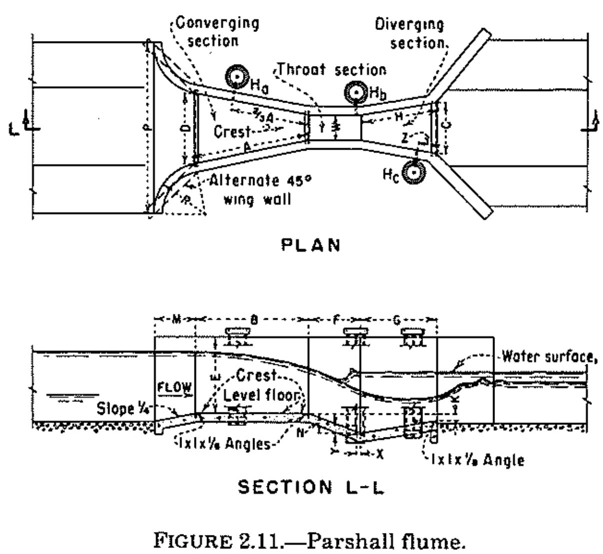


FIGURE 2.11.—Parshall flume.

H_{am} = upper gage in meters (referenced to crest).

Flow submergence occurs when the water in the diverging section of the flume rises to a level where it retards the flow in the converging section. The tailwater level is measured near the downstream end of the throat section (in fig. 2.11 it is labeled H_b). This zero reference, H_b, is the crest.

Skogerboe and others (*39*) developed the following equation for the Parshall flume:

$$Q = \frac{C_1 (H_2 - H_b)^{n_1}}{[-(\log H_b/H_a + M_2)]^{n2}} \qquad (2\text{–}18)$$

This equation is reasonably accurate (2 to 5 percent) for submergence values up to $(H_b/H_a) = 0.96$. Transition submergence, S_t, was defined as the value of submergence at which the change from free flow to submerged flow occurs. Values of submerged flow coefficients and exponents for Parshall flumes (C_1, H_a, H_b, n_1, C_2, n_2) are given in table 2.5. Submerged flow coefficients for use with meters are given in table 2.6.

Davis (*17*) used original data to develop the following comprehensive formula for the free outfall rating of a Parshall flume:

$$\frac{H_a}{W} + \frac{\left(\dfrac{Q}{g^{1/2}\,W^{5/2}}\right)^2}{Z\left(\dfrac{Ha}{W}\right)^2\left(1 - 0.4\dfrac{x_1}{W}\right)^2} = 1.351\left(\frac{Q}{g^{1/2}\,W^{5/2}}\right)^{0.645} \qquad (2\text{–}19)$$

where

H_a = upper gage head, feet;
W = throat width or length of crest, feet;
Q = discharge, cubic feet per second;
g = acceleration due to gravity, feet per second²; and
x_1 = horizontal distance parallel to the centerline between the crest and the point of observing the upper head, H_a, feet.

An electric computer can be used to solve this equation.

Davis (*17*) reported an error in equation 2–19 of less than 1 percent for the majority of points

	W		A		⅔A		B		C		D		E		F		G		H		K		M		N		P		R		X		Y		Z		FREE-FLOW CAPACITY MINIMUM	FREE-FLOW CAPACITY MAXIMUM
	FT.	IN.	FT.	IN.	FT.	IN.	FT.	IN.	FT.	IN.	FT.	IN.	FT.	IN.	FT.	IN.	FT.	IN.	FT.	IN.	FT.	IN.	FT.	IN.	FT.	IN.	FT.	IN.	FT.	IN.	FT.	IN.	FT.	IN.	FT.	IN.	SEC.-FT.	SEC.-FT.
[2]	0	1[1]	1	2 3/32	0	9 17/32	1	2	0	3 21/32	0	6 19/32	0	6 to 9	0	3	0	8	0	8 1/8	0	3/4	–	–	0	1 1/8		–	–		0	5/16	0	1/2	0	1/8	0.01	0.19
		2[1]	1	4 5/16		10 7/8	1	4		5 5/16		8 13/32		6 to 10		4 1/2		10		10 1/8		7/8		–		1 11/16		–	–			5/8		1		1/4	.02	.47
		3[1]	1	6 3/8	1	1/4	1	6		7		10 3/16	1 to 1½			6	1	0	1	5/32		1	–			2 1/4		–	–			1		1 1/2		1/2	.03	1.13
	0	6	2	7/16	1	4 5/16	2	0	1	3 1/2	1	3 5/8	2	0	1	0	2	0	–		0	3	1	0	0	4 1/2	2	11 1/2	1	4	0	2	0	3	–		.05	3.9
		9	2	10 5/8	1	11 1/8	2	10	1	3	1	10 5/8	2	6	1	0	1	6	–			3	1	0		4 1/2	3	6 1/2	1	4		2		3	–		.09	8.9
	1	0	4	6	3	0	4	4 7/8	2	0	2	9 1/4	3	0	2	0	3	0	–			3	1	3		9	4	10 3/4	1	8		2		3	–		.11	16.1
	1	6	4	9	3	2	4	7 7/8	2	6	3	4 3/8	3	0	2	0	3	0	–			3	1	3		9	5	6	1	8		2		3	–		.15	24.6
[3]	2	0	5	0	3	4	4	10 7/8	3	0	3	11 1/2	3	0	2	0	3	0	–			3	1	3		9	6	1	1	8		2		3	–		.42	33.1
	3	0	5	6	3	8	5	4 3/4	4	0	5	1 7/8	3	0	2	0	3	0	–			3	1	3		9	7	3 1/2	1	8		2		3	–		.61	50.4
	4	0	6	0	4	0	5	10 5/8	5	0	6	4 1/4	3	0	2	0	3	0	–			3	1	6		9	8	10 3/4	2	0		2		3	–		1.3	67.9
	5	0	6	6	4	4	6	4 1/2	6	0	7	6 5/8	3	0	2	0	3	0	–			3	1	6		9	10	1 1/4	2	0		2		3	–		1.6	85.6
	6	0	7	0	4	8	6	10 3/8	7	0	8	9	3	0	2	0	3	0	–			3	1	6		9	11	3 1/2	2	0		2		3	–		2.6	103.5
	7	0	7	6	5	0	7	4 1/4	8	0	9	11 3/8	3	0	2	0	3	0	–			3	1	6		9	12	6	2	0		2		3	–		3.0	121.4
	8	0	8	0	5	4	7	10 1/8	9	0	11	1 3/4	3	0	2	0	3	0	–			3	1	6		9	13	8 1/4	2	0		2		3	–		3.5	139.5
	10	0	–		6	0	14	0	12	0	15	7 1/4	4	0	3	0	6	0	–		0	6	–		1	1 1/2	–		–		0	9	1	0	–		6	200
	12	0	–		6	8	16	0	14	8	18	4 3/4	5	0	3	0	8	0	–			6	–		1	1 1/2	–		–			9	1	0	–		8	350
	15	0	–		7	8	25	0	18	4	25	0	6	0	4	0	10	0	–			9	–		1	6	–		–			9	1	0	–		8	600
[4]	20	0	–		9	4	25	0	24	0	30	0	7	0	6	0	12	0	–		1	0	–		2	3	–		–			9	1	0	–		10	1000
	25	0	–		11	0	25	0	29	4	35	0	7	0	6	0	13	0	–		1	0	–		2	3	–		–			9	1	0	–		15	1200
	30	0	–		12	8	26	0	34	8	40	4 3/4	7	0	6	0	14	0	–		1	0	–		2	3	–		–			9	1	0	–		15	1500
	40	0	–		16	0	27	0	45	4	50	9 1/2	7	0	6	0	16	0	–		1	0	–		2	3	–		–			9	1	0	–		20	2000
	50	0	–		19	4	27	0	56	8	60	9 1/2	7	0	6	0	20	0	–		1	0	–		2	3	–		–			9	1	0	–		25	3000

[1] Tolerance on throat width (w) ± 1/64 inch; tolerance on other dimensions ± 1/32 inch. Sidewalls of throat must be parallel and vertical.
[2] From Colorado State University Technical Bulletin No. 61
[3] From U S Department of Agriculture Soil Conservation Circular No 843.
[4] From Colorado State University Bulletin No. 426-A

FIGURE 2.12.—Parshall flume dimensions.

TABLE 2.5—*Submerged flow coefficients and exponents for Parshall flumes*

W	C_1	C_2	n_1	n_2	S_t
1 inch	0.299	0.00044	1.55	1.000	0.56
2 inches	.812	.0044	1.55	1.000	.61
3 inches	.915	.0044	1.55	1.000	.64
6 inches	1.66	.0044	1.58	1.080	.58
9 inches	2.51	.0044	1.53	1.060	.63
12 inches	3.11	.0044	1.52	1.080	.62
18 inches	4.42	.0044	1.54	1.115	.64
24 inches	5.94	.0044	1.55	1.140	.66
30 inches	7.22	.0044	1.555	1.150	.67
3 feet	8.60	.0044	1.56	1.160	.68
4 feet	11.16	.0044	1.57	1.185	.70
5 feet	13.55	.0044	1.58	1.265	.72
6 feet	15.85	.0044	1.59	1.230	.74
7 feet	16.15	.0044	1.60	1.250	.76
8 feet	20.40	.0044	1.60	1.260	.78
10 feet	24.79	.0044	1.59	1.275	.80
12 feet	29.34	.0044	1.59	1.275	.80
15 feet	36.17	.0044	1.59	1.275	.80
20 feet	47.56	.0044	1.59	1.275	.80
25 feet	58.95	.0044	1.59	1.275	.80
30 feet	70.34	.0044	1.59	1.275	.80
40 feet	93.11	.0044	1.59	1.275	.80
50 feet	115.89	.0044	1.59	1.275	.80

and less than 2 percent with few exceptions. Under normal operating conditions, however, the discharge can be determined within an accuracy of 2 to 5 percent.

Walnut Gulch Supercritical Flume:

A flume was developed in the late 1950's to measure the ephemeral, flashy, and sand-and-gravel-laden flows conveyed by the steep streams in the Walnut Gulch Watershed near Tombstone, Ariz. This flume was constructed on Walnut Gulch and was calibrated with model studies in the ARS Water Conservation Structures Laboratory, Stillwater, Okla. (*18,28*).

Walnut Gulch supercritical flumes (*40*) have a 15-foot (4.6 m), curved entrance approach to a 20-foot (6.1 m), straight section with a shallow V-shaped floor and sidewalls with a 1-on-1 slope. Figure 2.13 shows the design of a typical flume. The width, W, ranges between 5 and 120 feet. The cross slope, S_r, ranges between 7.5 and 15. The length of the straight portion should be about twice the maximum head. The curved approach has a cylindroid surface (coordinate origin shown in fig. 2.13) defined by the equation:

$$y = 0.03x + \frac{z - 0.03x^2}{0.00267x^2 + 1} \qquad (2\text{–}20)$$

where

x = horizontal coordinate positive in the upstream direction, feet;
y = vertical coordinate, feet; and
z = horizontal coordinate normal to centerline of the flume, feet.

A perspective view of this surface is shown in figure 2.14. The floor of the flumes has a slope of 0.03 in the downstream direction parallel to the centerline to insure movement of sediment

TABLE 2.6.—*Submerged flow coefficients (metric) and exponents for Parshall flumes*[1]

W_m (meters)	C_{tm}	C_2	n_1	n_2	S_t
0.025	0.172	0.0044	1.55	1.000	0.56
0.051	0.360	.0044	1.55	1.000	.61
0.076	0.535	.0044	1.55	1.000	.64
0.152	1.005	.0044	1.58	1.080	.55
0.228	1.432	.0044	1.53	1.060	.63
0.305	1.76	.0044	1.52	1.080	.62
0.457	2.56	.0044	1.54	1.115	.64
0.610	3.48	.0044	1.55	1.140	.66
0.762	4.26	.0044	1.555	1.150	.67
0.914	5.10	.0044	1.56	1.160	.68
1.219	6.66	.0044	1.57	1.185	.70
1.524	8.23	.0044	1.58	1.205	.72
1.829	9.74	.0044	1.59	1.230	.74
2.134	11.29	.0044	1.60	1.250	.76
2.438	12.68	.0044	1.60	1.260	.78
3.048	15.23	.0044	1.59	1.275	.80
3.658	18.03	.0044	1.59	1.275	.80
4.572	22.22	.0044	1.59	1.275	.80
6.096	29.22	.0044	1.59	1.275	.80
7.620	36.22	.0044	1.59	1.275	.80
9.144	43.22	.0044	1.59	1.275	.80
12.192	57.21	.0044	1.59	1.275	.80
15.240	71.20	.0044	1.59	1.275	.80

[1] Where $Q_m = \dfrac{C_{tm}(H_{um} - H_{bm})^{n_1}}{[-(Log_{10}H_{bm}/H_{um} + C_2)]^{n_2}}$

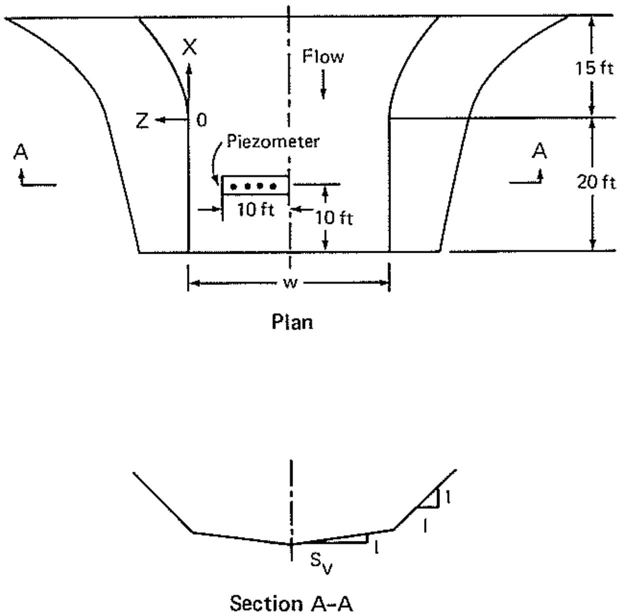


FIGURE 2.13.—Walnut Gulch supercritical measuring flume.

through the flumes. The flume invert is placed about 2 feet above the streambed to reduce backwater on the flume. The head is measured with piezometers at the midpoint of the straight portion of the flume. Velocities are supercritical at the head measuring section.

The approximate location of critical depth upstream from the piezometers is given by the following formula:

$$L = 0.88 + 14\,C_r \qquad (2\text{–}21)$$

where

- L = horizontal length between critical section and point of head measurement, feet; and
- C_r = contraction ratio (area of flow at point of head measurement divided by approach channel area) of flow.

Equation 2–21 is valid only for flows where the sidewalls are exerting control, that is, heads greater than the intercept of the floor with the 1-on-1 sidewalls. The contraction ratio, C_r, always should be less than 0.5.

A computed rating, h_p versus Q, may be obtained using the following equation of the energy relationship:

$$h_p = h_c + \frac{A_c}{2T_c} + 0.03L - \frac{Q_c^2}{2g\,A_p^2} \qquad (2\text{–}22)$$

where

- h_p = head above the flume zero, feet;
- h_c = depth of flow above V-shaped floor at the critical section, feet;
- A = cross-sectional area of flow, square feet;
- L = horizontal length between critical section and point of head measurement, feet;
- g = acceleration due to gravity;
- Q = discharge, cubic feet per second;
- T = top width of flow, feet;
- p = subscript relating to point of head measurement; and
- c = subscript relating to critical section.

The values of A_c and T_c are computed from a selected value of h_c. The discharge Q_c is calculated from the following equation:

$$Q_c = \left[\frac{gA_c^3}{T_c}\right]^{0.5} \qquad (2\text{–}23)$$

A value of L is assumed, and h_p is calculated from equation 2–22. A backwater computation is made from the point of head measurement to a point approximately 100 feet (30.48 m) upstream from the flume. This estimates the

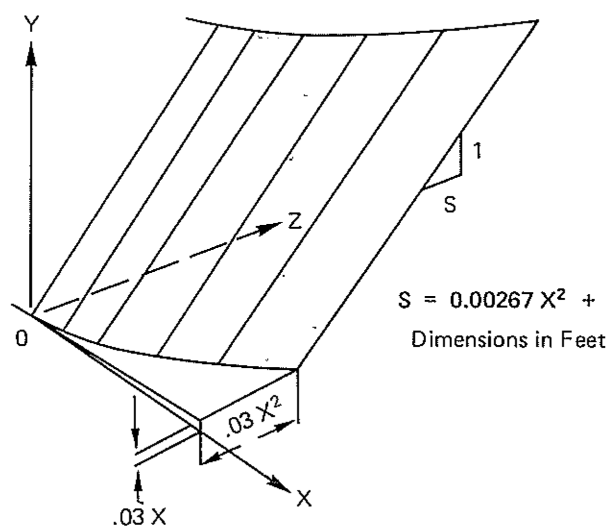


FIGURE 2.14.—The approach cylindroid surface.

water surface for determining the cross-sectional area of the approach channel. The contraction ratio, C_r, is calculated and used to compute the value of L in equation 2–21. If L disagrees with the initially assumed value of L, the calculation is repeated. This iteration is continued until a balance is obtained. The computed rating is valid for flows controlled by the 1-on-1 sidewalls of the straight portion of the flume. For flows within the floor of the V section (lower head range), field determinations of the rating should be made. These lower flows combine channel and flume control.

Observations in the prototype have shown that the thalweg in the approach section is unstable and varies with flow sequences. Therefore, the flow streamlines are not always parallel to the flume centerline and cause some irregular water surface profiles at the measuring section. Such eccentricity can be corrected by inserting porous training dikes in the approach channel. These dikes provide symmetrical flow (fig. 2.15).

The metric equivalent of the curved cylindroid surface defined by equation 2–22 is:

$$y_m = 0.03x_m + \frac{z-0.09842x_m^2}{0.0287x_m^2+1} \qquad (2\text{–}24)$$

where

y_m = vertical coordinate, meters;

x_m = horizontal coordinate positive in upstream direction, meters; and

z_m = horizontal coordinate normal to the centerline of the flume, meters.

Equation 2–21 expressed in meters would be:

$$L = 0.29+4.59\,C_r \qquad (2\text{–}25)$$

where

L = horizontal length between critical section and point of head measurements, meters; and

C_r = contraction ratio (area of flume divided by area of approach channel).

Existing Structures

Runoff can be measured by using hydraulic structures. These include culverts, conservation structures, reservoirs, spillways, and other works for water conveyance and control.

Laws governing flow through these structures are now fairly well understood because of hydraulic studies, both analytical and experimental, which have been made for guidance in design. Studies also have provided the coefficients needed to calculate discharge rates. By the use of these data, head-discharge relationships can be predicted for certain structures without the need for further calibration. Re-

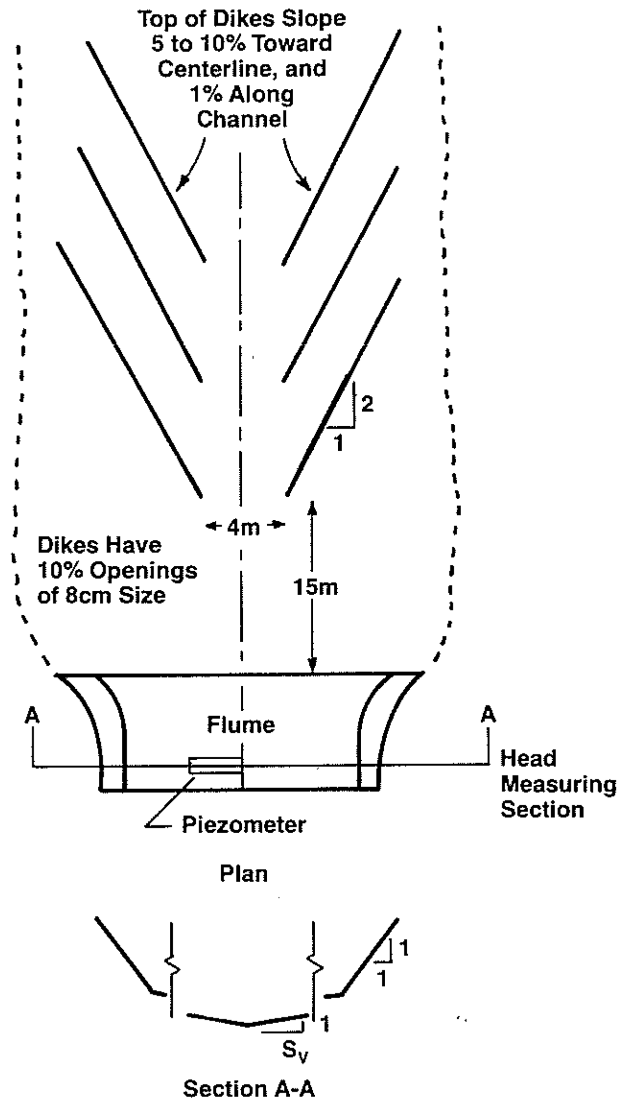


FIGURE 2.15.—Walnut Gulch supercritical measuring flume with porous dike training fences for ensuring symmetrical flow (*40*).

sults are available for other hydraulic studies made specifically to calibrate structures for runoff measurement. Principles in the studies guide the hydrologist, who may consider using an existing structure to measure runoff.

Accuracy of the discharge measurement can range from a rough estimate to a determination approaching the 5-percent level. Under good conditions where the field structure is similar in form to the model used in the hydraulic studies and comparable relative velocities exist in magnitude and direction, accuracy will be ± 10 percent. With calibration studies from model studies and from current-meter measurements in the field, the accuracy level can be raised to ± 5 percent. If a nonstandard condition exists—for instance, a high approach velocity—the discharge rate may be underestimated by 30 percent.

These accuracy estimates also apply to custom-built devices for measuring runoff, but standard approach conditions can be built into the installation. Where existing structures are used to achieve an inexpensive runnoff measuring station, nonstandard conditions are not corrected because of cost or inaccessibility of the approach.

Cost of the analysis should be considered carefully. It should not exceed the average annual cost of the study, which includes station installation, maintenance, operation, and data analysis. Installation of the station may be only a small part of the total cost, especially for a long study. Therefore, the use of an existing structure for the flowmeter may afford relatively small savings. If existing structures are used at the expense of data quality, the study will become expensive. Structures for runoff measurement may provide the only way to get the data at a site.

Sometimes the performance of a water control works is studied to evaluate the criteria used in its design. Part of the works—for example, a spillway—is used to measure the discharge rates. An alternative is to use a watershed that drains through an existing structure with satisfactory hydraulic properties. This watershed is found at certain places in a proposed hydrologic study. Use of this structure as a flowmeter is economical.

One flowmeter is the single-head control structure that includes weirs, spillways, culvert entrances, flume entrances, or free overfalls. Measurement of the elevation water surface upstream from this structure is sufficient to determine the flow rate through it. The hydraulic law operating for the structure can be the critical depth law, the orifice flow law, or the sluice gate law. Closed conduits flowing full and having free discharge also can be included in this category, or they can be in a category subject to friction control, which would be unsatisfactory for measurement of flow.

A head-loss structure also can be used to measure runoff. For this structure, two water surface deviations must be measured—one upstream and one immediately downstream. The difference between the two measurements is a function of the discharge rate. Head-loss structures include bridge opening, culverts under tailwater control, and closed conduit spillways with both the entrance and the exit submerged. They can be divided into two subgroups: (1) Where the head loss is caused largely by velocity head differences at the two measuring stations and (2) where the head loss is caused mainly by friction loss in the conduit. Structures in the first group may be better flowmeters than those in the second group because friction factors are easier to estimate and may change with time.

A third structure for measuring runoff is the reservoir. For this structure, an estimate of the runoff rate is the product of the rate of the reservoir water surface rise and the surface area of the reservoir at the corresponding instant. Outflow occurring at the same time must be added to the reservoir volume change used to obtain the true inflow rate of the reservoir. Although this estimate is no different than the pondage correction principle, the characteristic that establishes this category is a relatively small rate of outflow as compared to the rate of volume change. Although the spillway for the outflow is not a good flowmeter in itself, the rate of runoff can be estimated by gaging fluctuations of the water surface. Even a large percentage error in estimating the outflow rate—such as an error caused by accumulation of trash at the spillway inlet—may result in only a small percentage error in estimating the inflow rate.

Highway Box Culverts

Highway culverts can be used as runoff meters. They are usually simple and regular in form, and their hydraulic behavior is well understood because of several studies on flow through culverts (*8*). Six types of flow that can occur at a culvert are shown in figure 2.16. Types 1 and 5 are suitable for metering flow.

Discharge for type 1 can be calculated by the equation:

$$Q = CA\sqrt{2g\left(h_1 - z + \frac{V_1^2}{2g} - d_c - h_{f_{1.2}}\right)} \qquad (2\text{–}26)$$

where

- Q = discharge rate in cubic feet per second;
- C = coefficient of discharges;
- A = cross-sectional area of culvert (ft^2);
- g = acceleration of gravity (ft/s/s);
- h_1 = head depth of flow (ft);
- $h_{f_{1.2}}$ = head loss (ft) due to friction between sections 1 and 2;
- z = vertical distance between culvert outlet and entrance (ft);
- V_1 = velocity of section 1 (ft/s), a distance (1a) in feet upstream;
- d_c = critical depth of flow;
- D = height of culvert opening;
- W = width of culvert opening; and
- H = hydraulic head causing flow above culvert floor invert.

If $(h_1 - z)$ is replaced by H, V_1 and $h_{f_{1.2}}$ are disregarded because they are small, d_c is written as a function of H, and A is written as a function of W and H, the equation can be simplified to read:

$$Q = C^1 W H^{3/2}. \qquad (2\text{–}27)$$

This is the form in which the relationship between Q and H often is found. Indiscriminate use of equation 2–27 can lead to serious error. For example, if V_1 is not small, the flow will be underestimated. If $h_{f_{1.2}}$ is large and neglected, the discharge rate will be overestimated. Therefore, equation 2–27 should be used, rather than equation 2–26, to estimate the head-discharge relationship.

The coefficient of discharge is affected by geometry of the culvert entrance. Values suggested by Carter are given in figure 2.16. The discharge coefficient is determined by applying

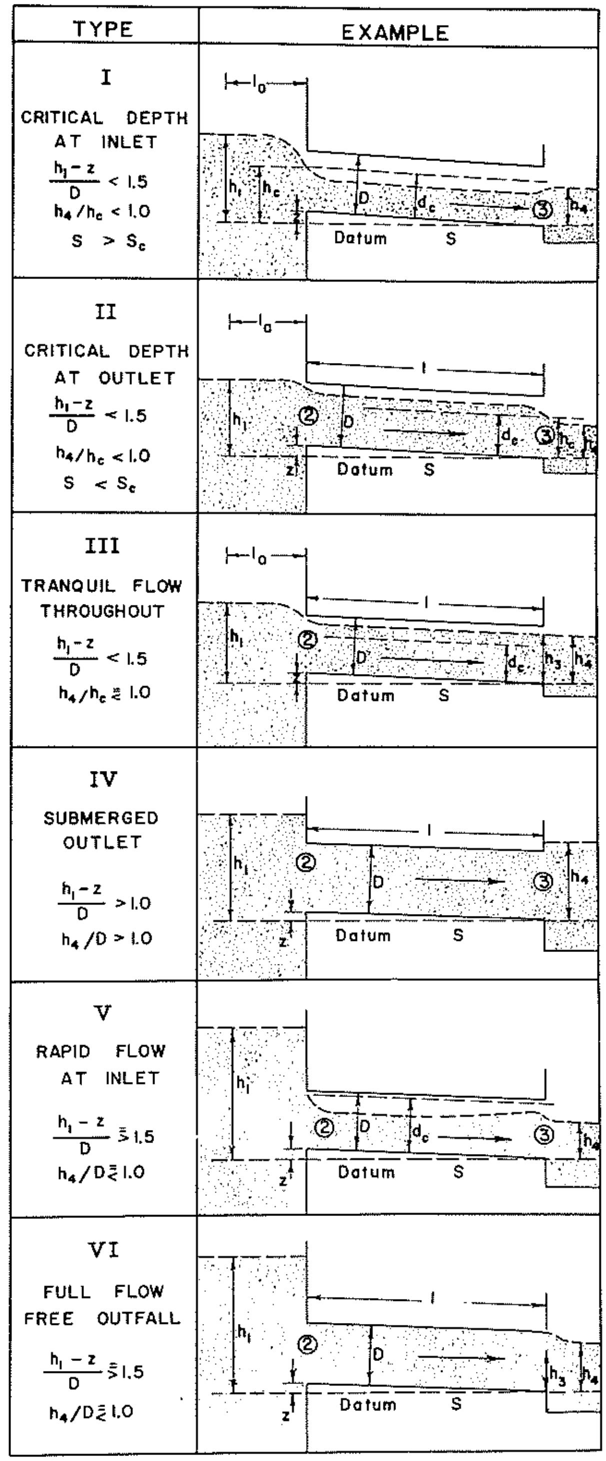


FIGURE 2.16.—Six possible types of flow at a culvert.

corrections to the base coefficient. Do not exceed 0.08 for the net coefficient value.

Use of values given in figure 2.17 will be illustrated by the following examples:

Given, a rectangular box culvert with wingwalls at a 60° angle with the roadway line. From figure 2.17, the base coefficient for box culverts is 0.95. The cosine of 60° equals 0.50. Therefore, C is determined by multiplying the base value of 0.95 by 1.215, the correction (k_e) for Cos θ = 0.50. Thus, C = 0.95×1.215 = 1.155. Since this value is higher than the maximum of 0.98, use 0.98.

Given, a concrete pipe culvert with a 24-inch diameter. This culvert is flush with vertical headwall but has a beveled-joint entrance. Depth of the groove is 2.4 inches (6.1 cm), or 0.1 D. The entrance diameter of the bevel is 27.8 inches (70.6 cm). Thus, t (thickness of culvert wall) is ½ (27.8−24)=1.9 inches (48 cm). The tangent of the entrance bevel is (2.4/1.9)−1.26, or the angle (θ) is about 52°. C will assume a head (h_i-z) equal to 1.2 feet (36.6 cm). each point on the rating curve. For example, assume a head (h_1-z) equal to 1.2 feet (36.6 cm). The ratio of head to diameter is (1.2 ft/2.0 ft)=0.6. Read the table to obtain 0.927 for the base coefficient. The correction factor for the entrance bevel is obtained from the appropriate table when entered with a t/D ratio of 1.9/2.4 × 0.079 and an angle of 52°. Interpolation between the 45° and 60° columns normally is necessary. The value of k_w (entrance-beveling correction) for 45° is 1.15, however. This indicates that the product of the base coefficient and k_w will exceed the maximum value of 0.98. Again, use 0.98.

If the pipe projects beyond the headwall, calculate the coefficient for a flush pipe and apply a correction factor for a projection condition. The projection correction factor (k_1) computed as length of culvert projection beyond embankment headwall (l_p) divided by depth of box (D) is:

$\frac{l_p}{D}$	K_1
0	1.0
0.1	.92
≧1.0	.90.

For type 5 flow, discharge may be computed from the equation:

$$Q=CA\sqrt{2g(h_1-z)}. \qquad (2\text{–}28)$$

Coefficients of discharge vary with the degree of rounding or beveling of the entrance (whether flush vertical wall or wingwalls) and type of entrance pipes projecting or mitered.

If the pipe projects beyond the headwall, use the correction factors (k_1) that were given for type 1 flow. If the end of the pipe is mitered flush with a sloping embankment, multiply coefficients (table 2.7) given for the square-ended pipe by 0.92.

Assume that a watershed with a culvert at its outflow point has been selected for a runoff study. Suitability of the culvert for flow measurement must be evaluated as follows:

- Does the culvert have a simple cross-section box or pipe with a symmetrical entrance? If so, it might be suitable without a special calibration. If not, calibration will be required, but the effort may be worthwhile.
- Does the culvert have a steep barrel? If the computed value of the term, $\frac{SD^{1/3}}{n^2}$, for a circular culvert falls to the right of the curve labeled S = Sc in figure 2.17, critical depth occurs at the culvert inlet. If the computed value of this term falls to the left of the curve, critical depth occurs at the outlet of the culvert. Figures 2.18 and 2.19 are used similarly for rectangular culverts. Mannings n values needed for this determination are:

Material	*Mannings n values*
Concrete:	
Very smooth	0.012
Ordinary field construction	.015
Badly spalled	.020
Corrugated metal	.024
Cement rubble	.020–.030
Cast iron	.013
Vitrified pipe	.013

- Does the culvert have free discharge at the outfall as evidenced by a drop or scour hole at the outlet? If so, it will be suitable if the

BASE COEFFICIENTS OF DISCHARGE

Pipe Culverts, Square Entrance, Flush with Vertical Headwall

$\frac{h_1 - z}{D}$	:	C
0.6	:	0.927
.7	:	.920
.8	:	.908
.9	:	.897
1.0	:	.884

$\frac{h_1 - z}{D}$	:	C
1.1	:	0.870
1.2	:	.856
1.3	:	.841
1.4	:	.826
1.5	:	.811

Pipe Culverts, Mitered, Flush with Sloping Embankment

$\frac{h_1 - z}{D}$	:	C
0.6	:	0.900
.7	:	.902
.8	:	.896
.9	:	.884
1.0	:	.868

$\frac{h_1 - z}{D}$	:	C
1.1	:	0.847
1.2	:	.823
1.3	:	.794
1.4	:	.764
1.5	:	.730

Box Culverts, Square Entrance, Flush with Vertical Headwall

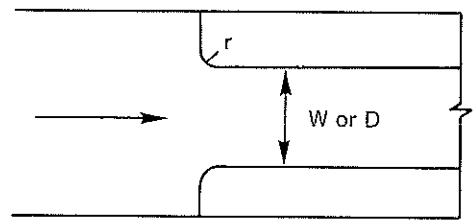
C = 0.95 for all heads

CORRECTIONS TO BASE COEFFICIENT

Correction for Entrance Rounding, k_r

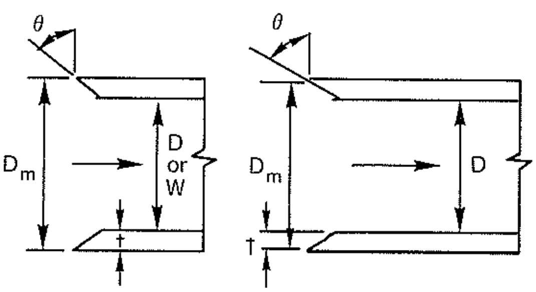
$\frac{r}{W}$ or $\frac{r}{D}$	:	k_r
0	:	1.00
.01	:	1.02
.02	:	1.04
.03	:	1.06
.04	:	1.08
.05	:	1.10
.06	:	1.12

$\frac{r}{W}$ or $\frac{r}{D}$	:	k_r
0.07	:	1.14
.08	:	1.155
.09	:	1.17
.10	:	1.18
.11	:	1.19
.12	:	1.195
.13	:	1.20



Correction for Entrance Beveling, k_w

$\frac{t}{W}$ or $\frac{t}{D}$	:	k_w $\theta = 30$	:	$\theta = 45$	:	$\theta = 60$
0	:	1.00	:	1.00	:	1.00
.02	:	1.03	:	1.065	:	1.09
.04	:	1.05	:	1.11	:	1.16
.06	:	1.07	:	1.135	:	
.08	:	1.08	:	1.15	:	
.10	:	1.09	:	1.15	:	



Correction for Wingwall Angle, k_e

Cos ϕ	:	k_e
1.00	:	1.00
.90	:	1.05
.80	:	1.095
.70	:	1.14
.60	:	1.18
.50	:	1.215

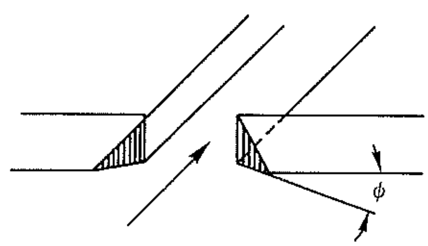


FIGURE 2.17.—Coefficients of discharge as affected by geometry of culvert entrance.

TABLE 2.7.—*Discharge coefficients for type 5 flow for culverts*

BOX OR PIPE CULVERTS SET FLUSH IN A VERTICAL HEADWALL

$(h_1-z)/D$	Value of C when r/W, t/W, r/D, or t/D equals—						
	0	0.2	0.04	0.06	0.08	0.10	0.14
1.4	0.44	0.46	0.49	0.50	0.50	0.51	0.51
1.5	.46	.49	.52	.53	.53	.54	.54
1.6	.47	.51	.54	.55	.55	.56	.56
1.7	.48	.52	.55	.57	.57	.57	.57
1.8	.49	.54	.57	.58	.58	.58	.58
1.9	.50	.55	.58	.59	.60	.60	.60
2.0	.51	.56	.59	.60	.61	.61	.62
2.5	.54	.59	.62	.64	.64	.65	.66
3.0	.55	.61	.64	.66	.67	.69	.70
3.5	.57	.62	.65	.67	.69	.70	.71
4.0	.58	.63	.66	.68	.70	.71	.72
5.0	.59	.64	.67	.69	.71	.72	.73

BOX CULVERTS WITH WINGWALLS

$(h_1-z)/D$	Value of C when the angle (θ) of wingwall is—				
	30°	45°	60°	75°	90°
1.3	0.44	0.44	0.43	0.42	0.39
1.4	.46	.46	.45	.43	.41
1.5	.47	.47	.46	.45	.42
1.6	.49	.49	.48	.46	.43
1.7	.50	.50	.48	.47	.44
1.8	.51	.51	.50	.48	.45
1.9	.52	.52	.51	.49	.46
2.0	.53	.53	.52	.49	.46
2.5	.56	.56	.54	.52	.49
3.0	.58	.58	.56	.54	.50
3.5	.60	.60	.58	.55	.52
4.0	.61	.61	.59	.56	.53
5.0	.62	.62	.60	.58	.54

culvert barrel is steep. If no definite evidence of free outfall exists and no deposition is present on the floor of the barrel, another test can be made before eliminating the culvert from consideration. A tail-water elevation discharge curve must be developed for the channel at the point of culvert discharge. This requires a survey of the downstream channel of approximately 1,000 feet (305 m). Obtain sufficient data to compute water surface profiles, starting at the downstream point and working up to the culvert. A water surface elevation must be assumed for the starting point. When the station of the culvert is reached, this elevation can be estimated accurately if the 1,000 feet (305 m) have been broken into enough subreaches for the computation. After the tail-water elevation-discharge curve has been developed, it must be compared with the curve of the water surface elevation versus discharge for critical depth (d_c) at the outlet. If the latter curve lies above the expected tail-water curve, type 1 flow can exist if the culvert barrel is steep.

- Does the culvert have sufficient capacity to pass the maximum floodflow? This can be determined by estimating the expected flow from the watershed and calculating the culvert capacity at the maximum possible head. If the

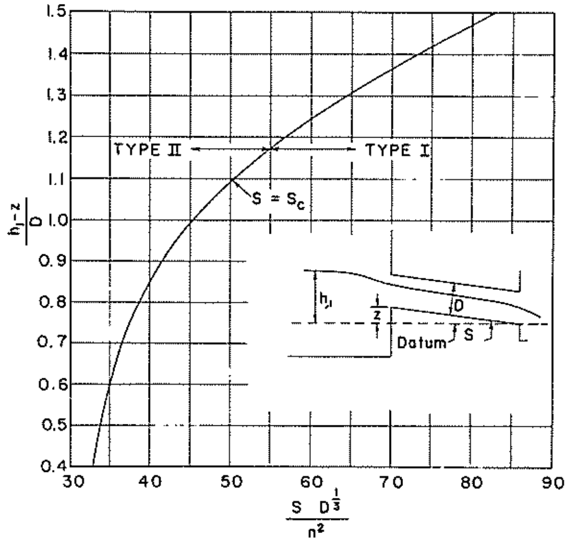


FIGURE 2.18.—Critical slope as a function of head for culverts of circular section with free outfall.

culvert does not have the capacity to pass maximum flow, other means of estimating the infrequent overflow should be considered.

- Are the approach conditions satisfactory? Steep approach channels or channel directions oblique to the centerline of the culvert are undesirable but do not necessarily disqualify the culvert. Calibration will be required for these cases.

If the culvert passes the preceding tests satisfactorily, location of the gage well should be selected. A position near the end of a wingwall is convenient, but the gage well should be kept out of the road ditch and out of the path of the approaching flow. If the approach is slightly nonsymmetrical, choose the dead-water side of the culvert entrance instead of the higher velocity flow side for the gage-well intake. Some difficulty may be experienced with sediment, but this may be as desirable as the effects of possible approach velocity on the water level of the gage well. With the position of the gage well fixed and the culvert dimensions measured, the stage-discharge curve can be computed.

The stage-discharge relationship is calculated from equations 2–26 and 2–28. These equations can be plotted on log-log paper to find the point of intersection. Flows below the intersection point will be type 1, and flows above will be type 5. The maximum head for culvert control may be governed by the low point in the road ditch at the divide rather than the low point in the road crossing the culvert.

Pondage corrections probably will be needed when culverts are used for runoff measurements. (Use the method described in the section on data processing, p. 56.)

Culvert Modification

Rectangular culverts are not good meters for low flows. A low flowmeter is required to accurately measure small flows. Before discussing these modifications, consider the need for such a device.

To gain sensitivity in the low-flow range, rectangular culverts are modified to confine low flows to a small opening and the low flowmeters are designed not to interfere with the maximum capacity of the culvert. Since the total available operating head remains the same, an increase in sensitivity obtained at low heads is gained at the expense of sensitivity in the high-head range. To design an effective device for measuring flow, characteristics of the flow must be studied. The meter then can be

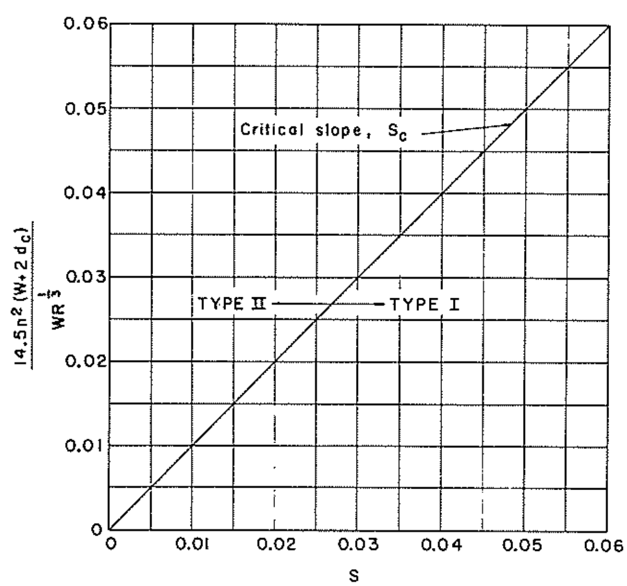


FIGURE 2.19.—Critical slope as a function of head for culverts of rectangular section with free outfall.

designed so that it will obtain the greatest accuracy in the range where most runoff occurs.

A cumulative distribution graph of percentage of total runoff against runoff rate is required for a rigorous solution. For example, if total runoff from a watershed can be estimated by measuring 95 percent of the runoff, the meter must be sensitive only down to the rate above which 95 percent of the flow occurs. Such detailed knowledge is unavailable, however, for an ungaged watershed. Obtain estimates from the flow-duration characteristics of streams in the area. If low flows are sustained for many days, a large percentage of the flow will come off at low rates and a low flowmeter will be required. In regions where streams are flashy and flows are ephemeral, low flow rates contribute only a small percentage of the total runoff. For these regions, a low flowmeter is unnecessary and may be undesirable. If a low flowmeter is installed, two arrangements are possible—one within the culvert or one outside the culvert.

The Villemonte weir sill is installed within the culvert. In this position it is out of the way and does not reduce the maximum flow capacity of the culvert. Another advantage is that it stays clean and free of debris. It has the disadvantage of having some minimum head below which a measurement is impossible. Construction within the culvert entails some difficulties, especially when flow is continuous.

The Villemonte weir sill is described in a bulletin by W. O. Ree and F. R. Crow (*33*). Dimensions of the design are given in terms of the culvert width (*W*) (fig. 2.20). The weir consists of a pair of converging sills placed on the floor of the culvert downstream from the entrance. The shape and placing of the sills leave a small trapezoidal opening between the sills through which the low flow must pass. The contraction is great enough to control the water level at the gage intake for all but the smallest trickles. As flow increases, control gradually passes to the culvert entrance. The transition is smooth and reversible, and the combination of measuring devices acts as a single unit.

Sills are placed in the culvert to measure low flows, but they also have a lower limit below which a measurement is impossible. The culvert apron is, in effect, a low dam interposed between the point of zero head (the floor at culvert throat) and the gage well. The relative height of this barrier depends on the culvert slope, and heads less than this height cannot be measured. Even after this height is exceeded, channel flow occurs between the gage well and the sill throat until backwater produces an effective control. An empirical expression derived from test data has provided the following relationship between culvert slope and minimum measurable flow:

$$\frac{Q}{W^{5/2}} = 0.000067\sqrt{g}\, S^{2.4} \qquad (2\text{–}29)$$

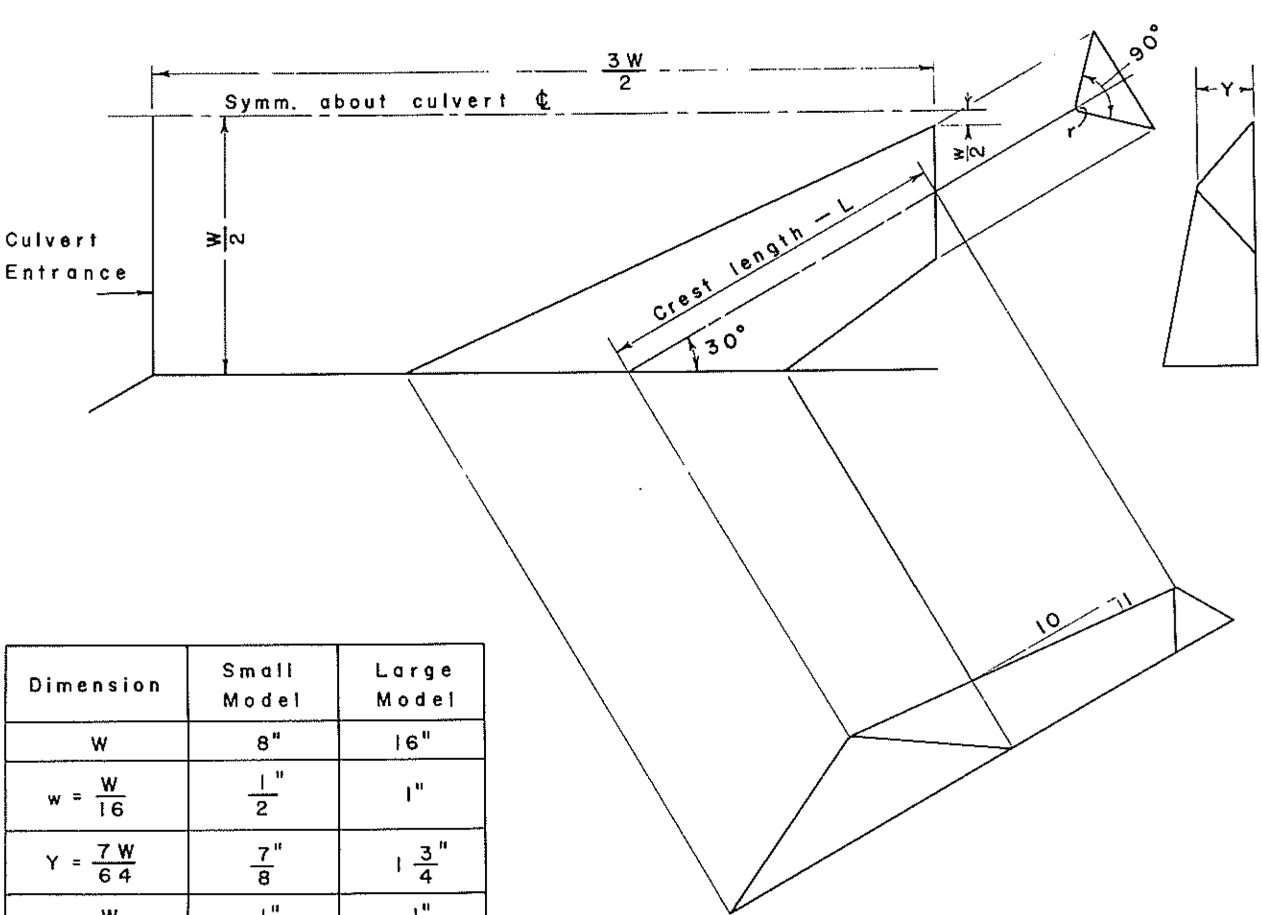
where

Q = discharge (ft^3/s);
W = width of culvert (ft);
s = slope (percent);
g = gravitational acceleration (ft/s^2).

The minimum measurable flow for the preceding 4- by 8-foot culvert with a 1.2 percent slope is 0.107 cubic foot per second. If flows less than this rate must be measured accurately, the head on the weir sill throat can be measured within the culvert a distance *W* downstream from the culvert entrance and a special rating can be used. Standardized ratio-type rating tables have been prepared for culvert floor slopes of 0, 1, 2, 4, 5, and 6 percent. The information is presented in figures 2.21 to 2.26 because these tables are rather extensive.

Weir sills should be installed carefully so that the standard rating table will apply. If the sills are made of concrete, they can be precast in a trough form. Layout dimensions for a trough form are shown in figure 2.27. A steel plate at the downstream end of each sill will protect and maintain the dimension of the throat.

The Virginia V-notch weir was devised for use in locations where unfavorable features of the Villemonte weir sill would be intolerable. The V-notch is placed on the culvert apron upstream from the culvert entrance. In this location flows can be measured down to zero head. Since the V-notch is outside of the culvert barrel, it is easier to construct than weir sills located inside culverts. A disadvantage of the Virginia V-Notch weir is possible reduction of culvert capacity. The weir also has the usual



Dimension	Small Model	Large Model
W	8"	16"
$w = \frac{W}{16}$	$\frac{1}{2}$"	1"
$Y = \frac{7W}{64}$	$\frac{7}{8}$"	$1\frac{3}{4}$"
$r = \frac{W}{32}$	$\frac{1}{4}$"	$\frac{1}{2}$"

FIGURE 2.20.—Dimensions of weir sills and their placement within the culvert.

ills of ordinary V-notch weirs associated with upstream deposition.

Capacity reduction was investigated first by the use of laboratory model studies. Figure 2.28 is reproduced from these studies to illustrate the findings concerning capacity reduction by weir sills of various heights and distances from the entrance of the culvert. In the practical range, however, reduction can be held within less than 5 percent.

Head-discharge characteristics of these V-notches have not been generalized except in the low flow range. When flow is confined to the V-notch opening, the standard rating will apply. The culvert is assumed to meet the criteria for entrance control so that free overfall exists at the weir in this range. When flow is increased further, control gradually passes to the culvert entrance. Throughout the range of flow the head-discharge relationship is reversible, and the combination provides a satisfactory flowmeter. Flow for entrance control can be the critical-depth type or orifice type, depending on whether the entrance is submerged. Flows in the upper range can be approximated from the hydraulic laws for the culvert opening. Calibration is still needed, however, to develop a satisfactory rating curve to cover the full range of operating head. Use of model studies, supplemented by field current meter or field volumetric measurements, is the best way to develop a rating curve.

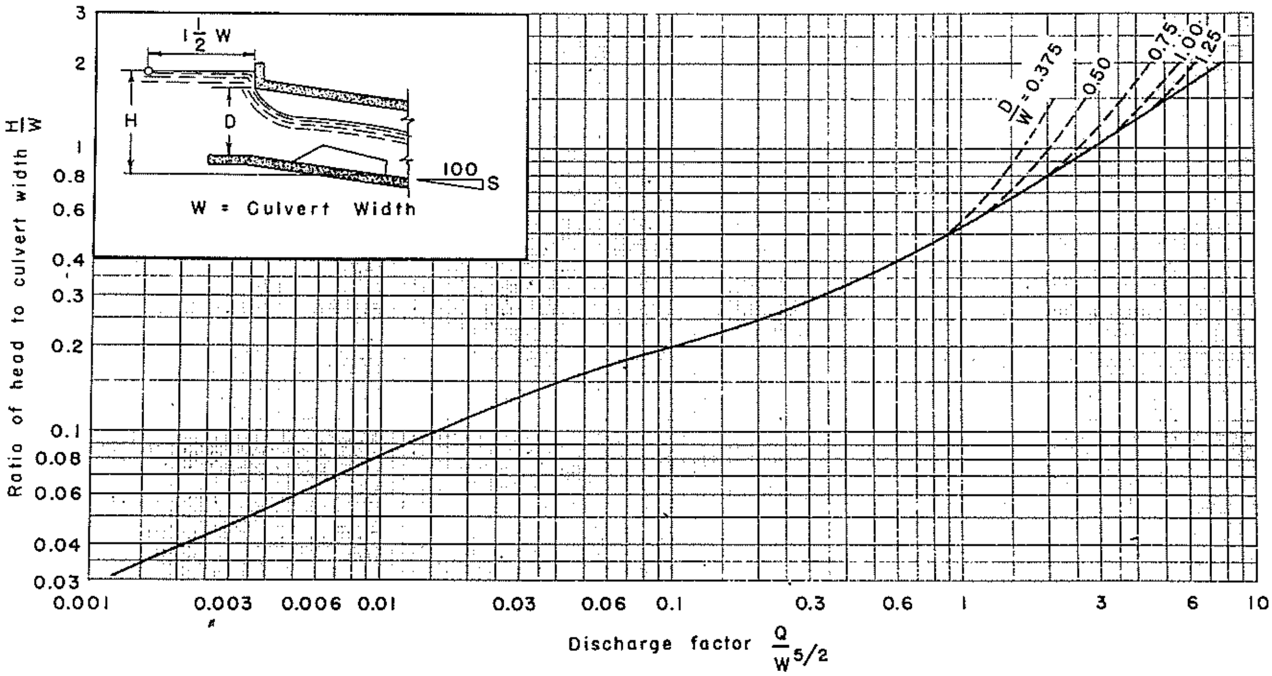


FIGURE 2.21.—Head-discharge relationship obtained from tests on short rectangular culverts equipped with weir sills and with a floor slope of 0 percent.

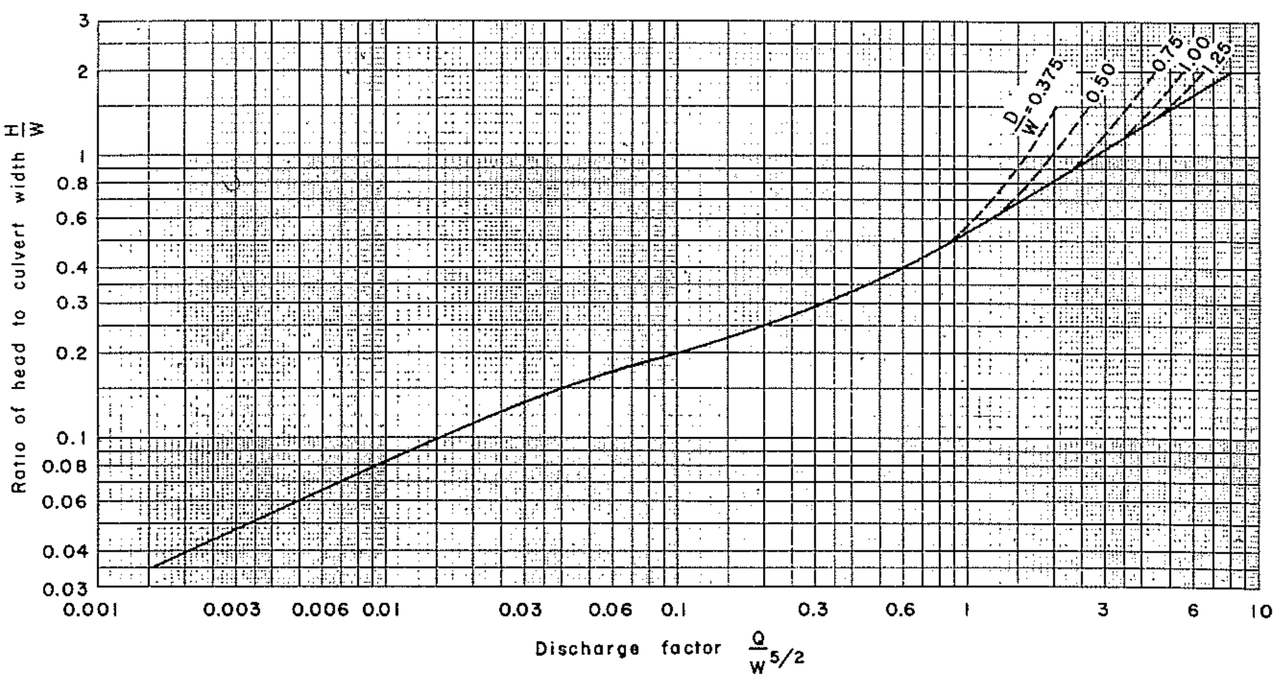


FIGURE 2.22.—Head-discharge relationships obtained from tests on short rectangular culverts equipped with weir sills and with a floor slope of 1 percent.

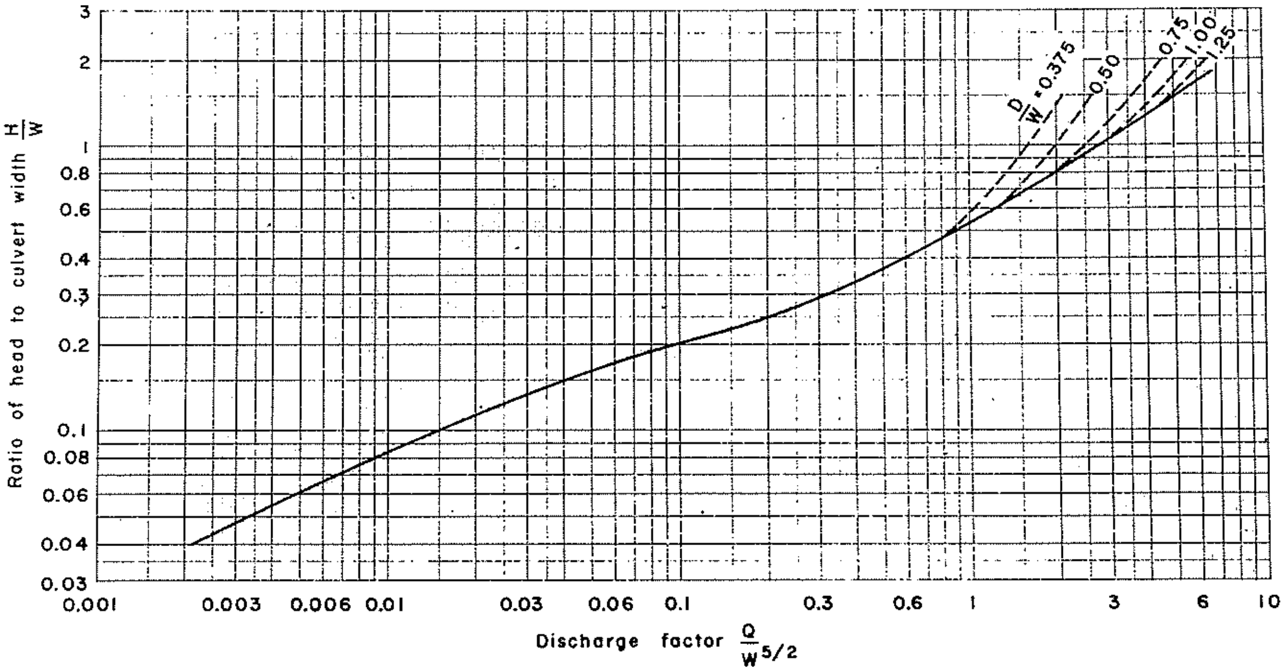


FIGURE 2.23.—Head-discharge relationships obtained from tests on short rectangular culverts equipped with weir sills and with a floor slope of 2 percent.

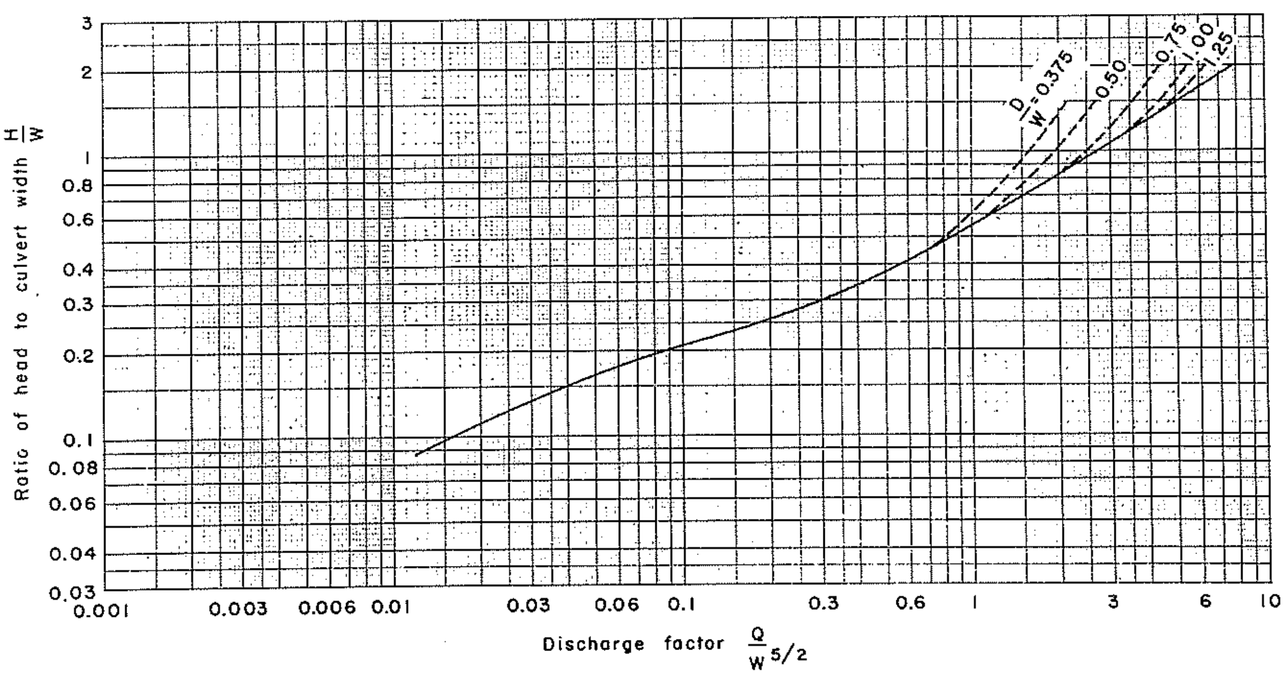


FIGURE 2.24.—Head-discharge relationships obtained from tests on short rectangular culverts equipped with weir sills and with a floor slope of 4 percent.

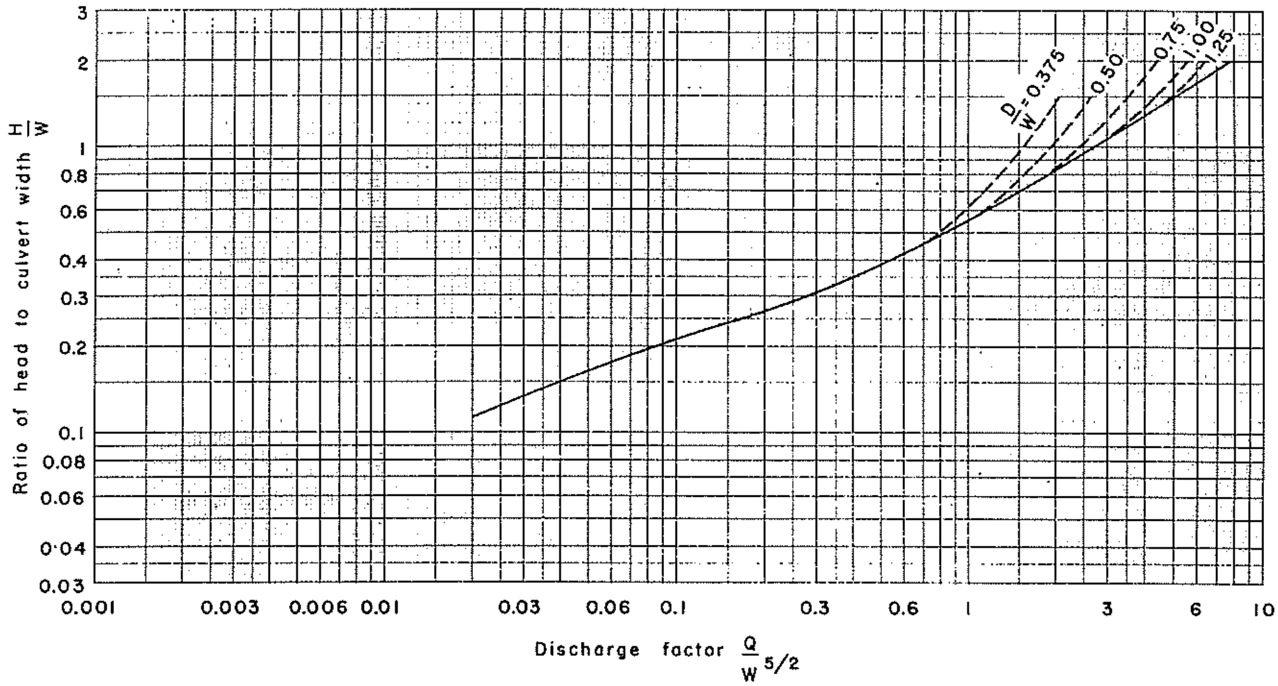


FIGURE 2.25.—Head-discharge relationships obtained from tests on short rectangular culverts equipped with weir sills and with a floor slope of 5 percent.

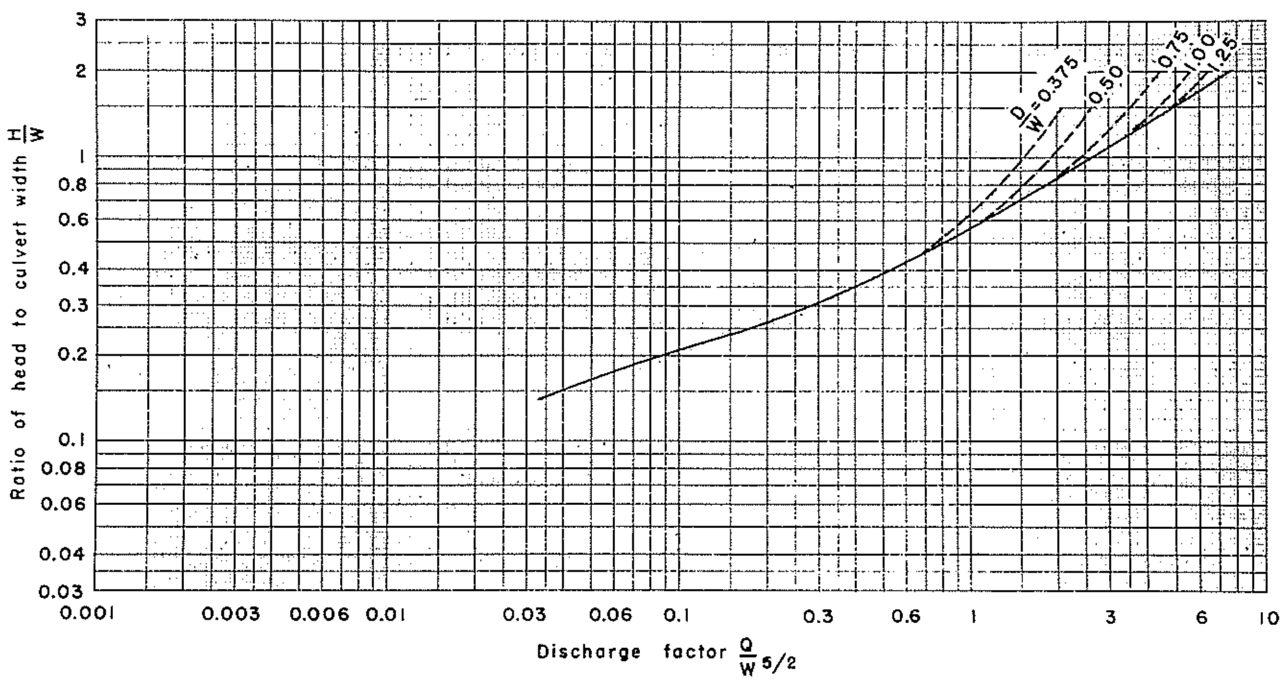


FIGURE 2.26.—Head-discharge relationships obtained from tests on short rectangular culverts equipped with weir sills and with a floor slope of 6 percent.

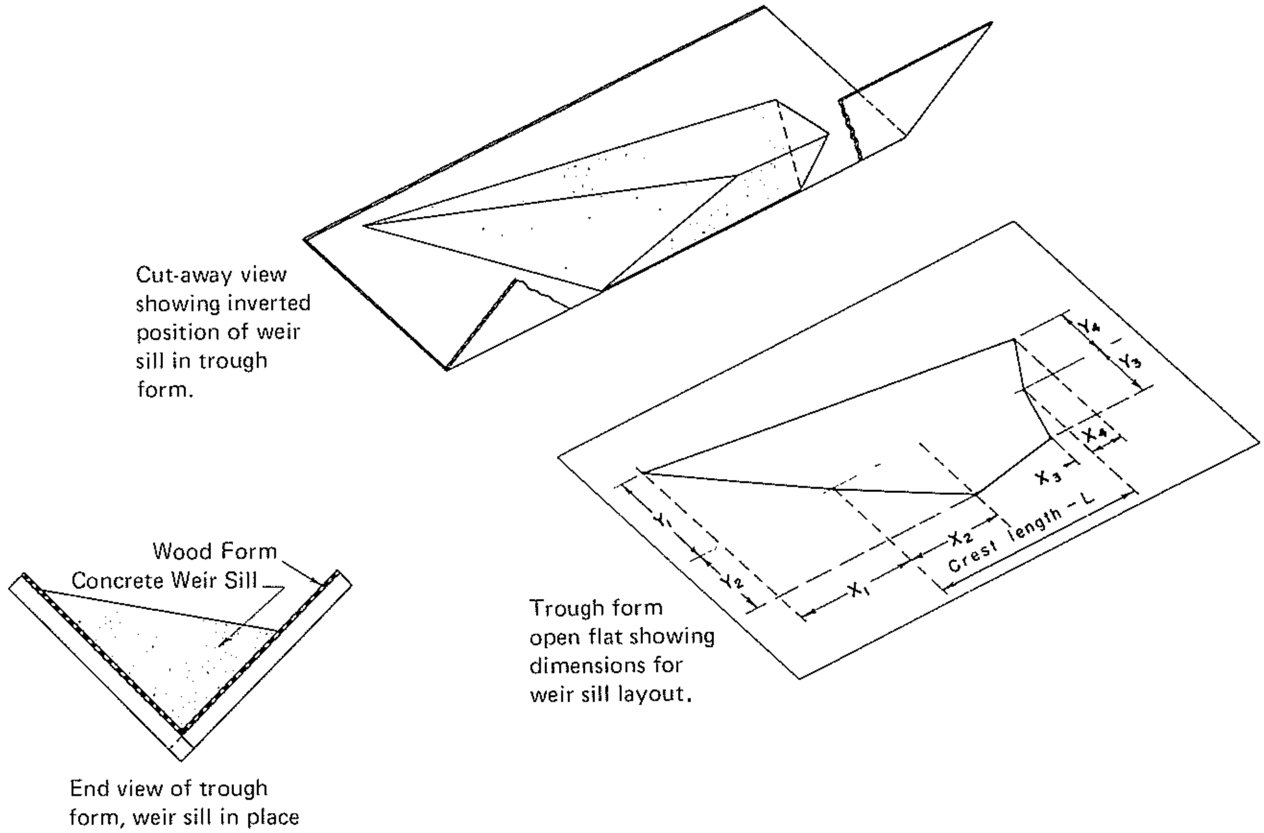


FIGURE 2.27.—Villemonte weir sill showing details of right-angle trough for casting concrete sills.

Soil Conservation

Many water control structures have been built as part of a soil and water conservation program. The original hydraulic studies for design guidance can be used to estimate the head-discharge relationships of those that have been constructed.

Closed Conduit Spillways

A closed conduit spillway is a tube through a dam. This definition includes simple culverts that have been discussed. Closed conduit spillways will be analyzed according to the suitability of various devices for measuring runoff. The devices considered will be those most often found in conservation areas.

Drop Inlet

A closed conduit with a drop inlet entrance can be used for runoff measurement, but only some structures under certain conditions can yield a fair measurement. A study of the hydraulics of the drop inlet will indicate structures that can be used. Blaisdell and Donnelly (*4*) pointed out that several types of flow are possible for a combination-type structure. Four possible controls are: (1) weir flow, (2) entrance orifice control, (3) short tube control, and (4) pipe flow.

Under some conditions a fifth control might be possible—that of orifice control at the juncture of the riser and the barrel. A structure that can exercise all these controls on the reservoir water level is unsuited for flow measurement. In figure 2.29, three discharge rates are possible for a given elevation of water surface. Since the discharge rate can fluctuate over this range, the action is unpredictable. A properly designed drop inlet will not exhibit this behavior; instead, it is proportioned so that the orifice and short tube controls are impossible. Either weir flow or pipe flow will occur, each in its own range. The head-discharge relationship will follow the same curve on either the rising or falling stage. This structure can be used to measure runoff. Thus, before a drop inlet can be considered for use as a flowmeter, its hydraulic behavior should be studied. The head-discharge relationship should be calculated for each control and should be plotted on a common graph, as was done in figure 2.29.

Weir flow.—This type of flow is estimated by the equation:

$$Q = CLH^{3/2}. \tag{2–30}$$

The value of C can range from about 2.5 to 5.7, varying with approach conditions, weir crest-form riser proportions, and head. Values of C should be based on those obtained from studies on similar structures. The crest length (L) is measured on the inside edge if the drop inlet of the weir is square edged. If it is rounded, the weir length is measured around the point of tangency on the crest.

Pipe flow.—When the conduit is flowing completely full, the head (H_1) causing the flow is the difference between the level of the headwater surface and the point where the hydraulic grade line pierces the plane of the conduit exit. Discharge is calculated by the equation:

$$Q = \frac{\pi D^2}{4}\sqrt{\frac{2gH_1}{K_e+K_o+f\frac{l}{D}+f_r\frac{l_r}{4R_r}\left[\frac{A}{A_r}\right]^2}} \tag{2–31}$$

where

D = diameter of circular barrel;
K_e = entrance coefficient;
K_o = exit loss coefficient;
f = Darcy-Weisbach friction coefficient for the barrel;
l = length of the barrel;
R_r = hydraulic radius of the drop inlet;
A = area of the barrel opening; and subscript r refers to the riser.

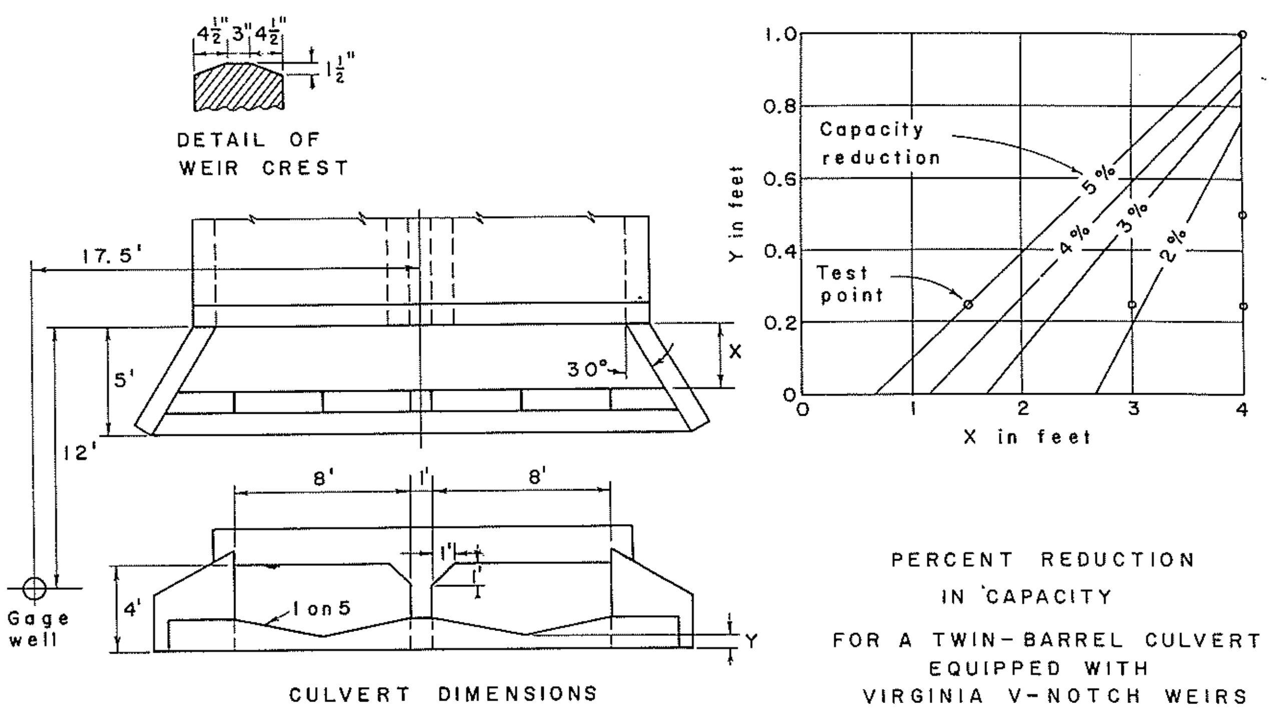


FIGURE 2.28.—Reduction of flow capacity caused by a pair of V-notch weirs on the apron of a twin-barrel culvert.

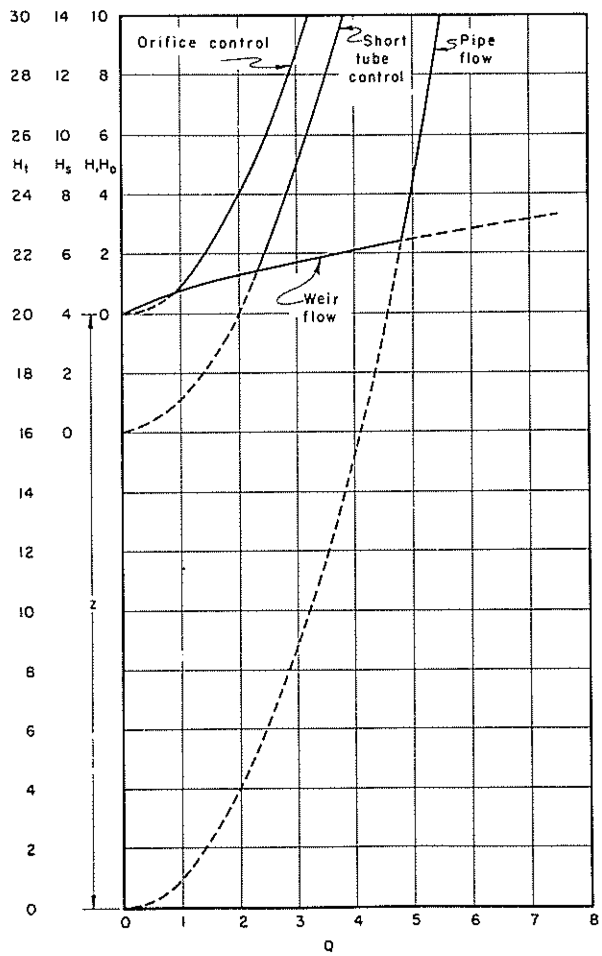


FIGURE 2.29.—Typical head discharge curves.

Values of the coefficients suitable for a preliminary and *rough estimate* of the head-discharge relationship are:

Coefficients		*Value*
$K_e =$		0.6
$K_o =$		1.0
$f =$ corrugated pipe[1]	18-inch diameter	.10
	24-inch diameter	.09
	36-inch diameter	.08
	54-inch diameter	.07
$f =$ concrete pipe		.02
$f_r =$ concrete riser		.02

[1] These values are for plain corrugated pipe. If the pipe has a paved invert the f values will be less when the pipe is new.

The head-discharge relationship for orifice (H_o) control at the entrance is calculated by the equation:

$$Q = C_O A H_O^{1/2}. \qquad (2\text{–}32)$$

Values of crest (C_O) suggested for rough estimating are square-edged crest = 4.0 and rounded crest = 6.0.

The equation for estimating the head-discharge relationship under short tube control is:

$$Q = C_8 A H_8^{1/2}. \qquad (2\text{–}33)$$

where H_8 is assumed to be the distance from water surface in the reservoir to the top of the barrel at the juncture with the riser.

For a well-rounded entrance to the barrel, the breakaway point may be a little higher than the line at the top of the barrel. Uncertainty of the short tube coefficient (C_8) makes the difference meaningless. The values of C_8 range from 1.9 for a short riser with a large flaring entrance section to 5.3 for a 6-inch (15.2 cm) vitrified clay tile with a bellmouthed entrance.

If hydraulic characteristics of the drop inlet are suitable runoff measuremens, the calculated rating curve should not be used to estimate runoff rates. Instead, the drop inlet should be calibrated by current meter. The major difficulty is accumulation of trash. Field current meter measurements on one structure have showed the discharge rate to range from 2 to 83 ft³/s (0.06 to 2.5 m³/s) for the same head, the variation being caused by accumulation of trash on the intake. If trash accumulates, the calculated rating curve should not be used to measure runoff.

Straight Headwall Inlet

The straight headwall inlet is essentially a culvert. Because of its length and high operating heads, it probably will flow full over much of its operating range. The principles used to analyze the drop inlet can be applied here.

As conduits are usually circular, the rating for the weir or critical depth flow through a circular opening can be shown best graphically. Three such curves are shown on figures 2.30 and 2.31. These are dimensionless and apply to

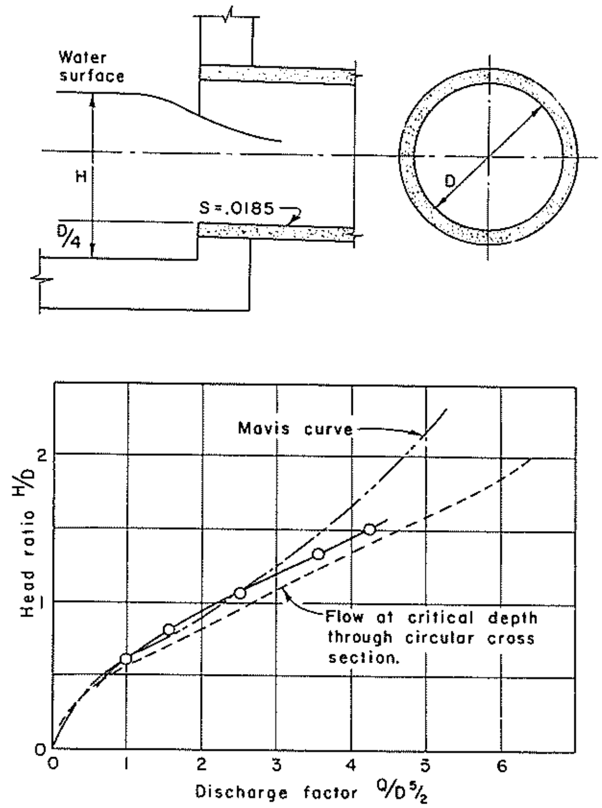


FIGURE 2.30.—Diagram of a circular opening and rating curve for weir flow or critical depth flow through the opening.

any diameter. The only requirement for these curves is that the pipe slope below be steep so that the entrance alone controls the rate of flow.

Pipe flow will be estimated the same as conduit flow with a drop inlet entrance except that the term for riser loss is omitted from equation 2–31. The values of entrance coefficients (K_e) also will be different: Suggested values for rounded entrance, 0.25; for square edge, 0.7; and for pipe groove, 0.3.

The discharge equation for orifice flow also will be the same as for the drop inlet. Orifice coefficien values (C_o) suggested for estimating are 4.8 for square edge and 6.0 for rounded edge. Current-meter calibration is recommended, particularly in the pipe flow range. Again, the possibility of trash fouling the intake must be considered.

Box Inlet Drop Spillway

Box inlet drop spillways, with entrances as shown in figure 2.32, have rather long weir lengths. As such, they are unsuitable for measuring small flow rates. Under some conditions, however, they are a satisfactory estimate of peak flows. For these conditions, information on calculating the flow rate can be found in reference (*3*). Three types of flume entrances are presented.

U-type flume entrance.—The *U*-type flume entrance is a box inlet with a sloping floor (fig. 2.33). The rating curve for inlet control can be approximated by the equation:

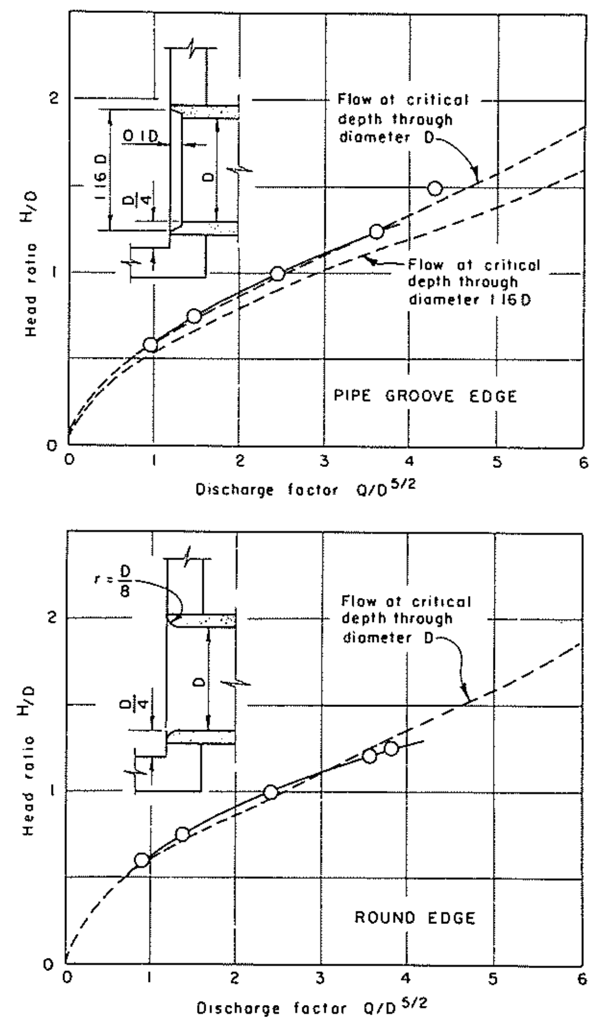


FIGURE 2.31.—Effect of grooved edge and of rounded edge on the rating curve of a circular opening.

$$\frac{Q}{W^{2.5}} = 0.95\sqrt{g}\left[\frac{B}{W}\right]^{0.225}\frac{L}{W}\left[\frac{H}{W}\right]^{1.73} \qquad (2\text{–}34)$$

where
- Q = discharge rate (ft³/s);
- W = width of the flume (ft);
- B = length of the box, (ft);
- L = total length of weir (ft);
- H = head on lip of box (ft); and
- g = acceleration of gravity (ft/s).

The inlet control ceases when the discharge reaches the value indicated by the following equation:

$$\frac{Q}{W^{2.5}} = 1.367\sqrt{g}\frac{D}{W} \qquad (2\text{–}35)$$

where D is depth of box, feet.

The rating curve for headwall opening control is:

$$\frac{Q}{W^{2.5}} = 0.547\sqrt{g}\left[\frac{H+D}{W}\right]^{1.6} \qquad (2\text{–}36)$$

Construct a rating curve for the flume entrance by plotting the inlet control and headwall control discharge curves on a single sheet. Intersection between the two should be smoothed by placing a short section of reverse curve between them. This should approximate the rating.

Wisconsin flume entrance.—The Wisconsin flume entrance has a vertical curve connecting the entrance apron with the floor of the flume. It was tested with circular entrance walls and 2-to-1 tapered entrance walls. Their arrangement is shown in figure 2.33.

The rating curve for either entrance of the Wisconsin flume is expressed by the equation:

$$\frac{Q}{W^{2.5}} = 0.617\sqrt{g}\,W^{0.94}H^{1.56}. \qquad (2\text{–}37)$$

2:1 flume entrance.—For the 2:1 flume entrance the apron floor intersects the floor of the flume at an angle. The design is shown on figure 2.33. The approximate discharge equation for this entrance is:

$$Q = 0.661\sqrt{g}\,W^{0.9}H^{1.6}. \qquad (2\text{–}38)$$

Test data are available on rectangular drop spillways with a trapezoidal channel approach. Proportions are shown in figure 2.34. The discharge equation for this structure, which contains an approach velocity term, is:

$$Q=CL\left[H+\frac{V^2}{2g}\right]^{3/2} \qquad (2\text{–}39)$$

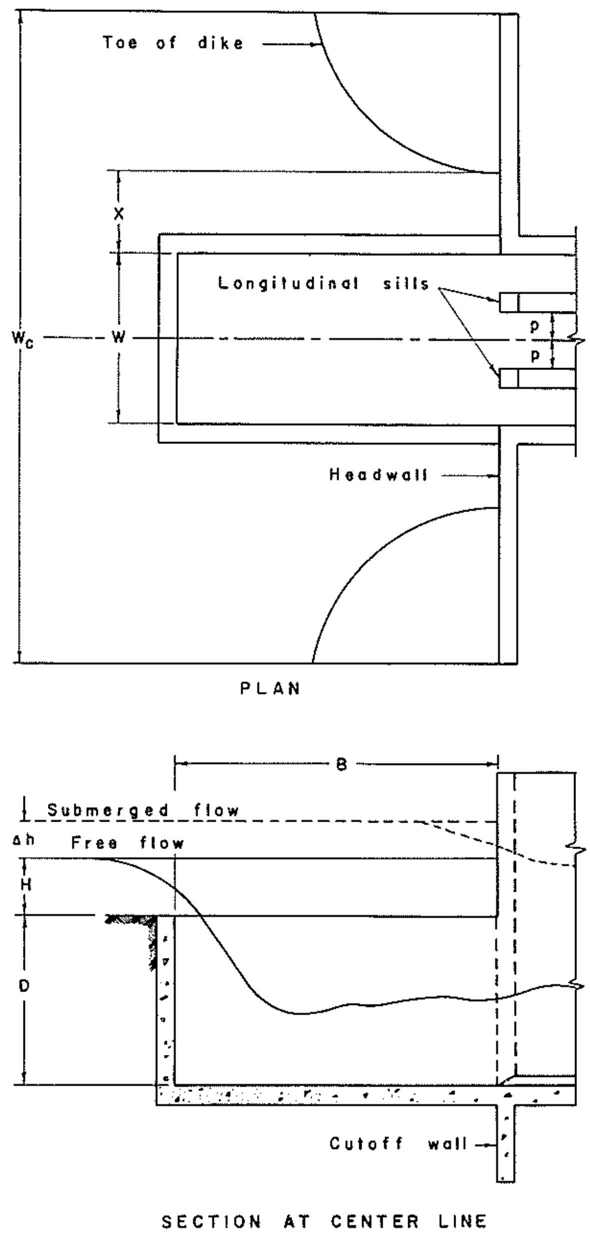


FIGURE 2.32.—Construction of a box-inlet drop spillway.

where

C = weir coefficient for flow;
L = total weir length;
H = hydraulic head on weir;
V = velocity of flow; and
g = acceleration of gravity.

The coefficient varies with the dimensional proportions of the notch opening and the values given in figure 2.32.

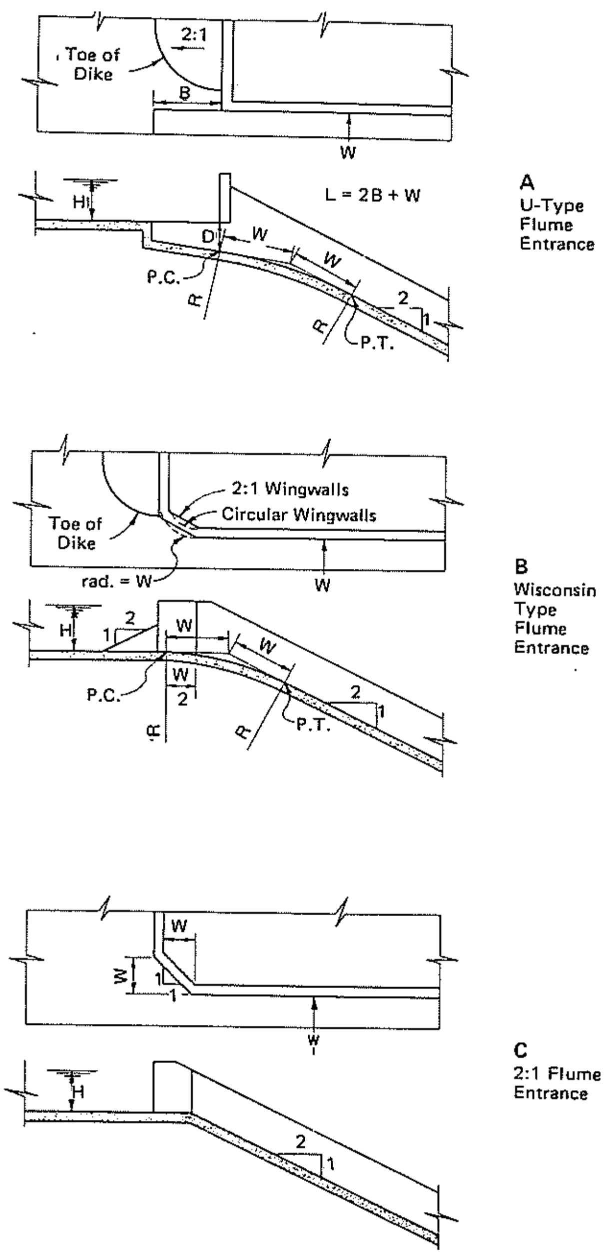


FIGURE 2.33.—Three types of box-inlet drop spillways with sloping floors.

Installation of *HS*, *H*, and *HL* Flumes

Whenever possible, installation of these flumes should be made with approach boxes depressed below the natural ground surface, as shown in figure 2.35. Where the watershed slope is small and the flow dispersed, gutters may be necessary to collect the runoff at the bottom of the slope and channel it into the approach box (fig. 2.36). These gutters and the approach box sometimes are covered to prevent complication of runoff record by rain falling thereon. The more common approach box is shown in figure 2.37. Here, the runoff has concentrated naturally into a small stream channel upstream from the flume.

Metal flumes are bolted to the concrete approach with gaskets to make a watertight joint. The concrete cutoff wall extends below the concrete approach at the upstream face of the flume to provide substantial support and to prevent seepage below the flume. A drain tile releases water from below the flume. Leveling bolts on the downstream supporting wall are used to fasten and adjust the flume to this level.

The flume floor must be level. If silting is a problem, a 1-on-8 sloping false floor can be set in to concentrate low flows and thereby reduce silting. The difference in calibration of a flume with a flat floor and that with a sloping false floor is less than 1 percent.

The stilling well for water stage recorder floats usually is made of sheet metal attached to the flume wall. Openings to the flume supply ready exchange of water between the flume and the stilling well. Support and shelter for the water stage recorders are fastened to the flume wall. Where freezing temperatures are expected, the well should be heated or drained automatically after each runoff period.

HS Flumes

Instructions for setting water level recorders for *HS* flumes are:

- See that the water level recorder is fastened securely to its support and that the support prevents movement of the recorder relative to the measuring device. Make sure

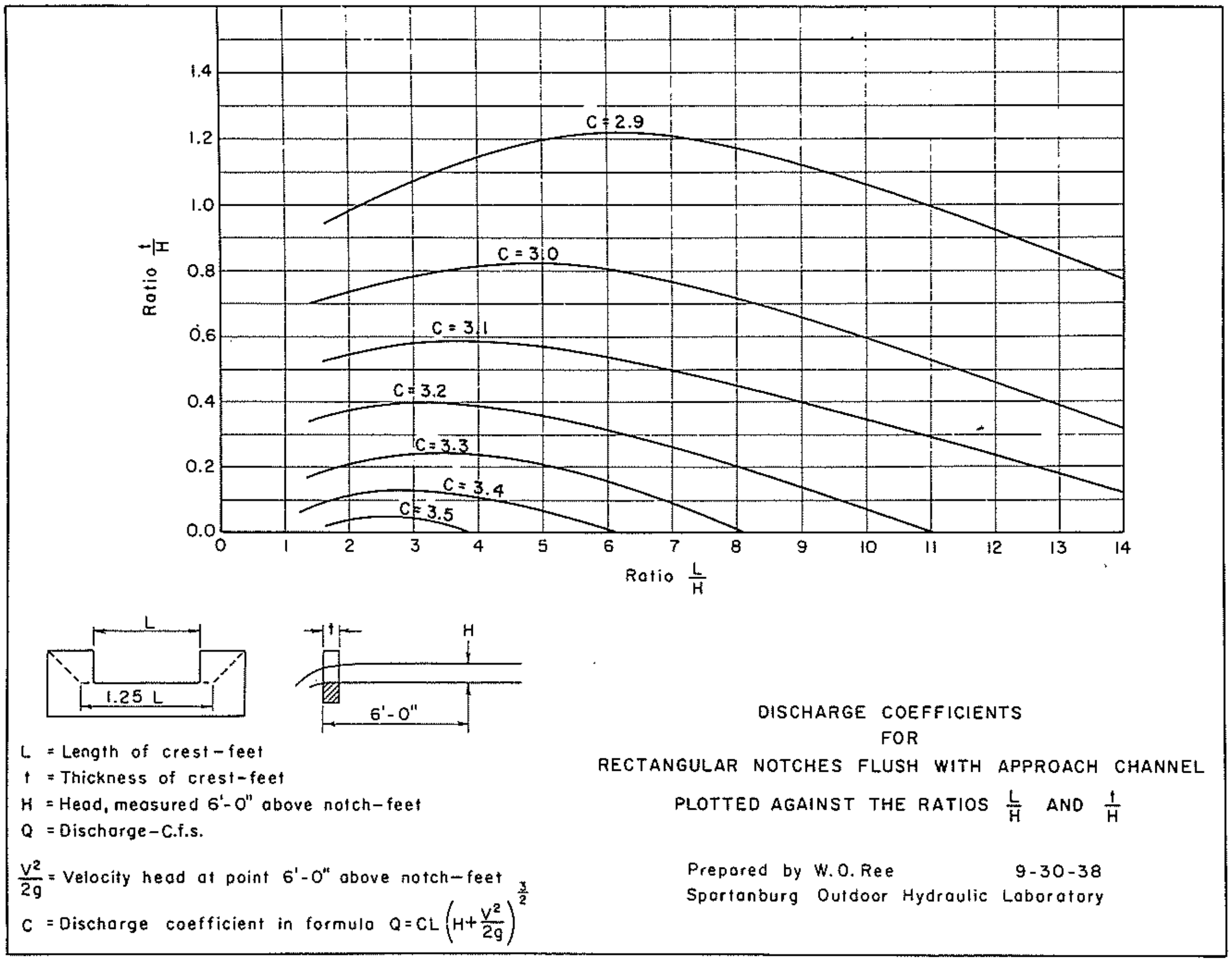


FIGURE 2.34.—Discharge coefficients for rectangular drop spillways with trapezoidal channel approach.

that the chart is mounted correctly on the chart drum.

• Mount a point gage vertically over the floor near the tip of the flume. This ordinarily will be done by a temporary point-gage support.

• Use modeling clay (plasticine) or a similar material to dam the outlet end of the measuring device. If necessary, dam the inlet end to prevent loss of water. To avoid the effects of surface tension, the nearest point of the dam at the elevation of water surface should be at least a half inch away from the point gage.

• Fill the flume and float well with water until a depth of ½ to 1 inch (1.3 to 2.5 cm) of water is obtained over the control section.

• Obtain point gage readings for the water surface and floor of the flume. Subtract the crest reading from the water surface reading.

• When point gage readings are made, the water level recorder will note the elevation of water surface on the chart. Subtract the difference between point gage readings from the chart reading to obtain the chart reading for zero head on the measuring device.

• Check the two preceding steps with a different amount of water in the flume.

H and *HL* Flumes

Instructions for setting the water level recorders for *H* and *HL* flumes are:

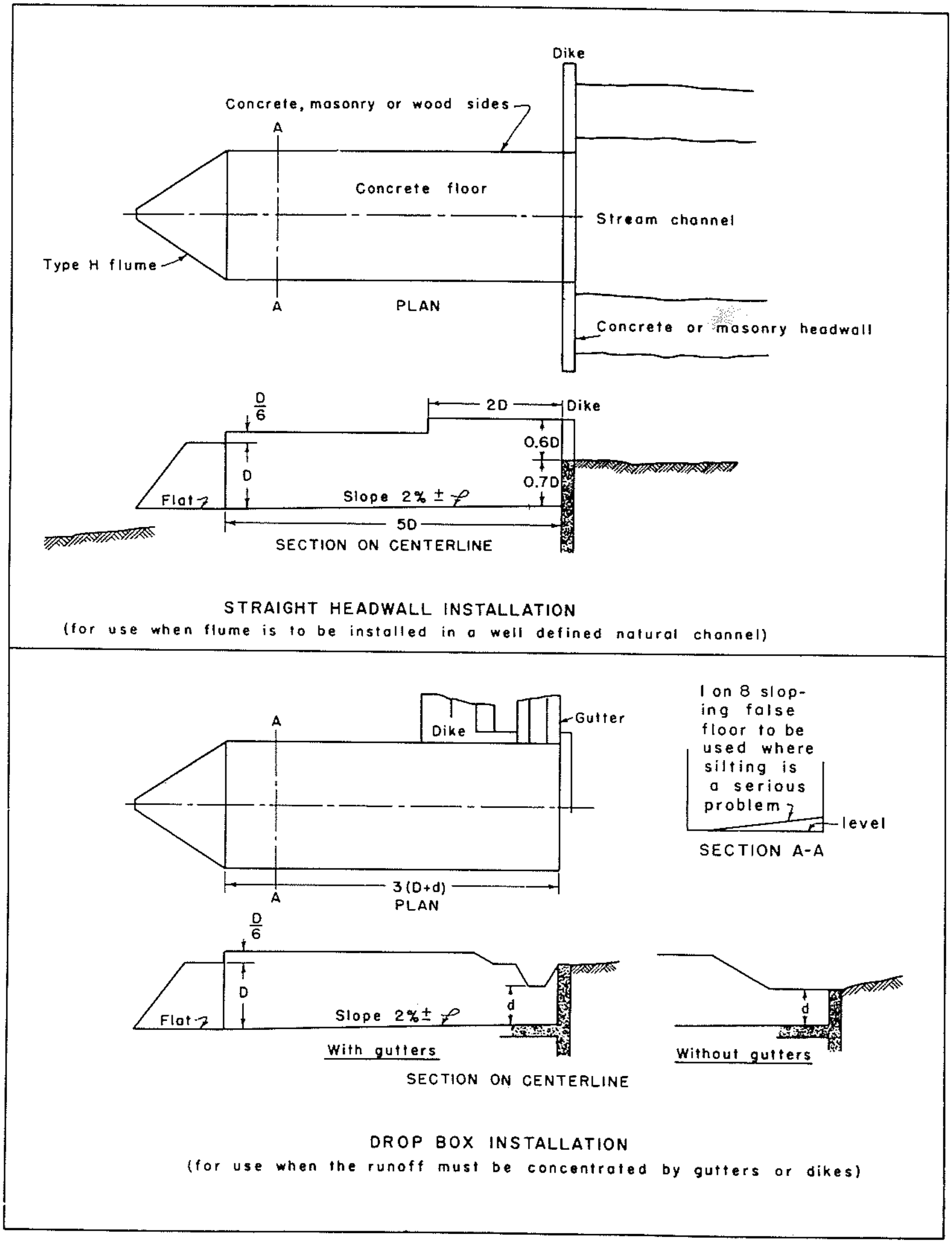


FIGURE 2.35.—Plans for straight headwall and drop box installations of *HS*, *H*, or *HL* flumes.

PN–5878

FIGURE 2.36.—Installed *H* flume with approach box.

- Form temporary watertight pool around intakes outside of stilling well.
- Raise water level in stilling well until it is 1 or 2 inches (2.5 or 5.0 cm) above lowest intake.
- Place water level recorder on floor of shelter or on shelf; install float, counterweight, and graduated float tape in position; install tape index pointer (I.P.); insert clock; place chart paper on clock; fill pen with ink and place it in position to record.
- Observe the record for about 5 minutes to see if the setup is watertight. If the water level drops during this period, find the leak and repair it.
- With surveyor's level, take backsight (B.S.) on crest of flume or notch of weir to get elevation of the height of instrument (H.I.). All rod readings are to 0.001 foot (0.03 cm).
- Attach plumb bob to steel tape graduated in 0.01 foot (0.3 cm). Set point of plumb bob at elevation of H.I. and read tape at horizontal index line (L) marked on shelter or any other convenient object over the pool. (Estimate tape reading to 0.001 ft (0.03 cm)).
- Lower plumb bob to water surface of pool and read tape at index line. Repeat this step for a check. Read tape index pointer on float tape immediately.
- Subtract difference of tape readings at (*L*) of the preceding two steps from height of instrument to get elevation of water surface.
- Lower the float by the difference between the two preceding readings (if the preceding reading is less than the reading preceding it) or raise the float (if the preceding reading is greater than the reading preceding it) with respect to the float tape. Minor adjustments up to 0.05 (1.5 cm) foot can be made by adjusting the index pointer.
- Check height of instrument by rod reading on flume crest.
- Set pen on chart to read water surface elevation obtained in the last reading.

PN–5879

FIGURE 2.37.—Installed *H* flume with approach box through a concentrating dike.

- Check the preceding eight steps with water at a different level.

Submergence Effect on H Flumes

Flumes should be installed with free outfall or no submergence, wherever possible. The free discharge head, H, can be computed with the terms shown in figure 2.6 by using the following equation:

$$H = \frac{d_1}{1 + 0.00175\,(ed_2\, d_1)^{5.44}} \qquad (2\text{–}40)$$

where
H = free flow head;
d_1 = actual head with submergence;
d_2 = tail-water depth above flume zero head; and
e = 2.71828 . . . base of natural logarithms

$$0.15 < d_2/d_1 < 0.90.$$

Submergence of 30 percent has less than 1 percent effect on the free discharge head (fig. 2.6). A 50-percent submergence has less than a 3-percent effect.

Rating Malformed Flumes and Weirs

The crests or controlling edges of a weir or H flume are the most important part of a measuring device. They should be formed carefully so that the dimensions are precise. The controlling edges should be straight and smooth. If the weir, as built, deviated appreciably from the design slope, a correction must be applied to the discharge values determined from the laboratory tests.

For triangular weirs, the slope term is $\tan\frac{\theta}{2}$, where θ is the total angle of the V-shaped crest. The angle θ is obtained from field measurement on each crest. One method of obtaining the crest measurements is to establish a horizontal control and a datum for elevation and obtain at least eight equally spaced points on each crest. These crests should be relatively straight. Therefore, an equation of a straight line can be obtained by using least-square methods for each crest and by using elevation and horizontal distance as the X and Y terms. Solving the two equations simultaneously for the common Y will yield the zero elevation for head. The $\tan\frac{\theta}{2}$ can be obtained by averaging the slope terms of the two equations1 The $\tan\frac{\theta}{2}$ term is then used in the discharge equations 2.3 to 2.8. Discharge equations for triangular weirs are listed in figure 2.7. More than eight points may be necessary if the crests are not smooth. The texture (concrete, steel, pebble) of the crest on triangular weirs is unimportant, however, as long as the shape is well defined.

H flumes may be corrected for small deviations from the design dimensions by measuring the control edges of the flume. Enough measurements should be taken to obtain the values of B_0 and B_1 in figures 2.38 and 2.39. The flume should be installed before these measurements are made to obtain the correct head, H, for each width of control obtained. The discharge rating then may be computed using the discharge equations in figure 2.38. These equations apply only where the velocities approaching the flume are uniform and tranquil. The sloping floor is a 1-on-8 false floor in the H flume to concentrate flows along the wall with the stilling well intake. The height of the false floor is 0.2D at the intersection of the wall and entrance to the flume. This floor slopes downward 1-on-8 along the wall and also at the entrance to the flume. If drop boxes or sloping channels in the direction of flow produce high velocities in the approach channels, the flume should be calibrated under these conditions. Discharge equations in metric form are listed in figure 2.39.

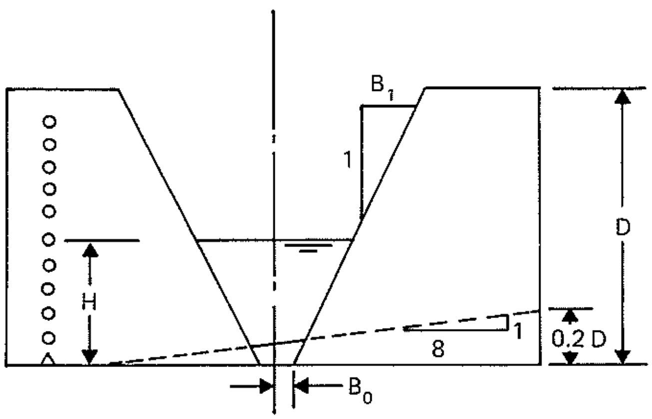
Preparation of Rating Tables

The laboratory staff conducting the calibration tests should prepare rating tables. Using discharge coefficients to prepare the tables can eliminate small errors constructing the prototype measuring device. This is accompanied by incorporating prototype measurements in the discharge equation. Where electronic computers are used in the runoff analysis, rating tables may not be required. The hydrographer ususally desires a rating table, however.

Wherever practical, current-meter or volumetric measurements should be made to check the calibration. Any large cutting or filling changes in the approach cross section may require a revision of the calibration. These changes may be especially large in sediment-laden streams where deposition may alter approach gradients and thereby alter the approach velocity. In such instances, a different measuring device should have been used.

Pondage Corrections

Installation of a flume or weir in a natural waterway may cause unnatural detention or retention of storm runoff in the pond formed



Front Elevation

Flume Type	Design Value	
	B_0	B_1
HS	0.025 D	0.1667
H	0.050 D	0.5000
HL	0.100 D	1.0000

Low Flow

$$Q = A_0 (2B_0 + B_1 H) H [H - 0.01]^{A_1}$$

Transition

$$Q = (K_0 B_0 + K_1 B_1 H) \sqrt{2g}\, H^{3/2}$$

Medium and High Flows

$$Q = [(E_0 + E_1 D) B_0 + (F_0 + F_1 D) B_1 (H + v^2/2g)] \sqrt{2g}\, (H + v^2/2g)^{3/2}$$

if $D > 1$ then $D = 1.0$ $\quad$ v = average velocity, head measuring section

Flume	Floor	Range H in Feet	A_0	A_1	E_0	E_1	F_0	F_1	K_0	K_1
HS	Flat	0.01 - 0.10	3.93	0.526						
HS	Flat	0.20 - D			0.861	0.112	0.479	-0.035		
H	Flat	0.01 - 0.10	3.14	0.486						
H	Flat	0.20 - D			0.612	0.209	0.409	-0.024		
H	Sloping	0.01 - 0.10	3.30	0.500						
H	Sloping	0.20 - D			0.630	0.174	0.400	-0.018		
HL	Flat	0.01 - 0.10	3.57*	0.522*						
HL	Flat	0.20 - D							0.7804*	0.3788*

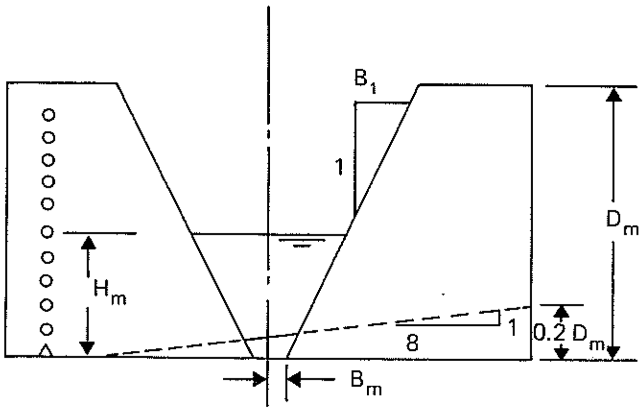
*Derived from rating table (4-foot HL)

Transition range, heads 0.1 to 0.2 feet

Values of K_0 and K_1 are determined by a simultaneous solution of the value of Q for H equal to 0.1 foot from low flow equation and value of Q for H equal to 0.2 feet.

FIGURE 2.38.—Discharge equations for *HS* flume, *H* flume, and *HL* flume, using flume dimensions.

by this installation. Both conditions may result if the pond is large in relation to the size of the watershed. The record for this interruption in the natural streamflow may need to be corrected to derive values of watershed runoff without the pond. Pondage correction can be determined for a few storms where the rate of change in depth through the control structure is large. This test will show the size of corrections, and the hydrographer can determine if



Front Elevation

Flume Type	Design Value	
	B_m	B_1
HS	0.025 D_m	0.1667
H	0.050 D_m	0.5000
HL	0.100 D_m	1.0000

Low Flow

$$Q_m = A_m\,(2B_m + B_1 H_m)\,H_m\,[H_m - 0.003]^{A\,1}$$

Transition

$$Q_m = (K_0 B_m + K_1 B_1 H_m)\,\sqrt{2g}\,H_m^{3/2}$$

Medium and High Flows

$$Q_m = [(E_0 + E_m D_m)\,B_m + (F_0 + F_m D_m)\,B_1\,(H_m + v^2/2g_m)]\,\sqrt{2g_m}\,(H_m + v^2/2g_m)^{3/2}$$

if $D_m > 0.3048$ then $D_m = 0.3048$ $\quad$ v = average velocity, head measuring section

Flume	Floor	Range H_m in Meters	A_m	A_1	E_0	E_m	F_0	F_m	K_0	K_1
HS	Flat	0.003 - 0.03	2.238	0.526						
HS	Flat	0.06 - D			0.861	0.367	0.479	-0.115		
H	Flat	0.003 - 0.03	1.705	0.486						
H	Flat	0.06 - D			0.612	0.686	0.409	-0.079		
H	Sloping	0.003 - 0.03	1.822	0.500						
H	Sloping	0.06 - D			0.630	0.571	0.400	-0.059		
HL	Flat	0.003 - 0.03	2.023*	0.522*						
HL	Flat	0.06 - D							0.7804*	0.3788*

*Derived from rating table (4-foot HL)

Transition range, heads 0.03 to 0.06 meters

Values of K_0 and K_1 are determined by a simultaneous solution of the value of Q_m for H equal to 0.03 meter from low flow equation and value of Q_m for H equal to 0.06 meter.

FIGURE 2.39.—Discharge equations for *HS*, *H*, and *HL* flumes using metric flume dimensions.

such corrections are worth the effort. Pondage corrections generally increase the peak rate and shift the peak time forward.

Some pondage corrections are ignored or neglected if only the total runoff volume is of interest to the watershed study. Therefore, peak rate and hydrograph are unimportant. If the total runoff for each day from midnight to midnight is to be calculated, the difference in amount stored at the beginning and end of the day must be considered.

Pondage corrections generally are made by using computerized runoff analysis. Procedures for calculating pondage corrections are described in the section on data reduction (p. 184).

A contour map of the area of the pond is used to compute pondage rates corresponding to rates of change in stage. The maximum elevation of the contour map should be the maximum head on the outflow structure. Use the areas obtained from the contour map of the pond to compute the volume of pond for various stages. A typical calculation and curves are shown in figure 2.40.

Natural Controls

Streamflow is gaged at an uncalibrated runoff-measuring station by measuring discharge rates (product of the flow velocities and channel cross-section areas) of various stages of flow. Measured discharge rates are plotted against stage to provide a means of converting continuous records of stage into runoff rate and amounts. The field-calibrated gaging station may be the most practical means of measurement where the quantity of water is large and operating conditions are unfavorable for precalibrated devices.

Low Water Controls

Natural low water controls may be found in streams at rock outcrops or ledges in the channel. The sensitivity of low water control will depend on its contraction in the channel and the amount of fall below the control. Critical depth usually will be found upstream of the brink for low flows.

As stage increases, the low water control may become ineffective as a flow control due to tail water. This usually occurs when submergence exceeds 0.90. Sometimes, however, the submergence value can be as high as 0.95 without affecting the flow control. The zero reference used in determining the submergence ratio may be difficult to obtain on a natural control. One method is to plot the free discharge measurements on logarithmic paper, using var-

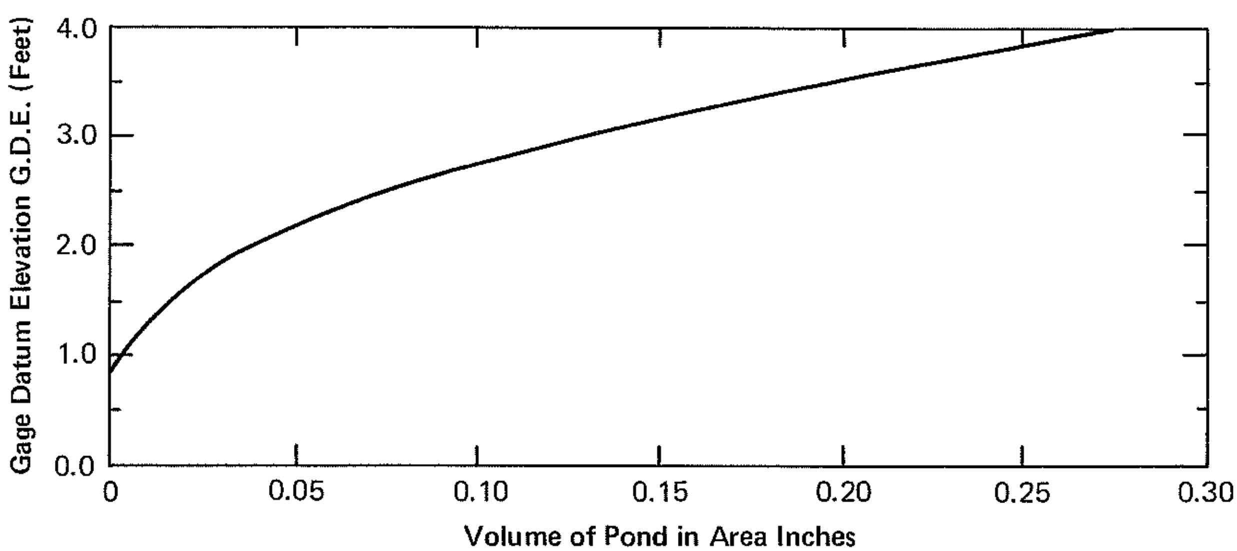


FIGURE 2.40.—Computations to obtain volume of pond for various stages of elevation and pondage volumes and curves, Fennimore, Wis.

ious elevation references for zero head. Then use the zero reference that will give 1.5 for the exponent of the head (slope of the line).

High Water Controls:

As the stage increases on a natural control, the control may shift from low water control to a bridge downstream or a bend in the channel, or even to channel control.

Bridge openings and contracted sections.— Bridge openings and similar contracted sections can be used to obtain estimates of peak flows. The U.S. Geological Survey devised a method for making such estimates (*2, 3, 6, 16, 19, 22, 25*). It is described for guidance in estimating of peak flows, but reference to the circular will be required to evaluate the coefficients needed for a solution.

This method is for computing an isolated peak discharge rate from the high watermarks it left behind. It does not provide a system for obtaining continuous measurement of flow rate through the bridge opening. The principles outlined can be adapted to obtain a continuous record, but the current-meter rating curve for a station with the bridge for a control probably would be better. The contracted-section method is satisfactory, and it is often the only way for estimating floods. This method is different from that for culverts in that the estimate is based on the relationship between the drop of the water surface level and the velocity change instead of critical flow. Thus, elevations of water surface are required at two points in the flow system rather than at one point as in the previously described methods.

Figure 2.41 shows a typical contracted section.

The two points of measurement are at sections 1 and 3. A discharge equation can be derived by writing the energy and continuity equations for the reach between these two sections. This equation is:

$$Q = Ca_3\sqrt{2g}\left[\Delta h + \alpha_1 \frac{V_1^2}{2g} - h_f\right] \qquad (2\text{–}41)$$

where

Q = discharge in cubic feet per second;

C = coefficient of discharge;

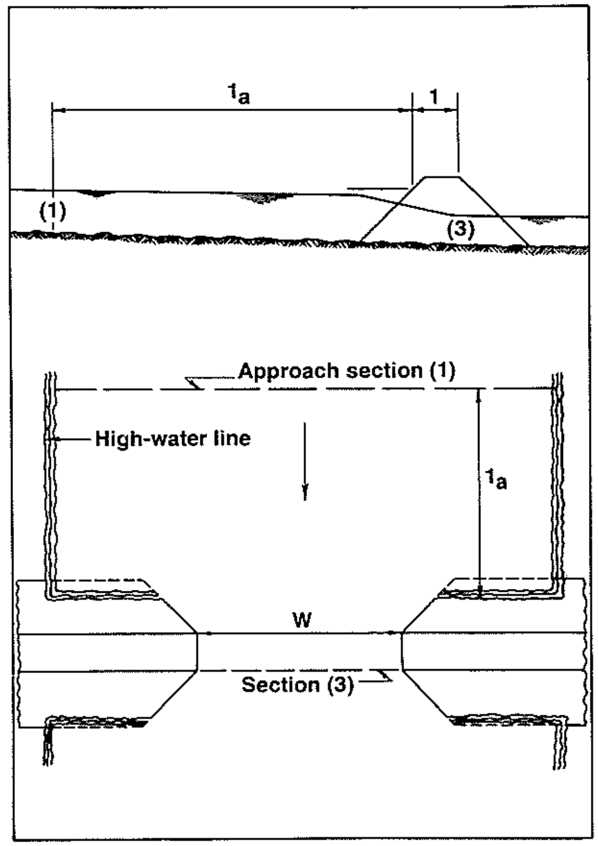


FIGURE 2.41.—Contracted section for estimating peak flows.

a_3 = gross area of section 3 in square feet (this is the minimum section parallel to the constriction between the abutments and is not necessarily at the downstream side of the bridge);

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

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

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

The fieldwork necessary for evaluating the terms in this equation is described in detail, but the office procedure is discussed in more general terms. If the fieldwork is done well and completely, the flow can be computed when the needed reference material is at hand.

Define and locate sections 1 and 3. Section 1 is located at least one bridge-opening width upstream. It must be at right angles to the direction of the stream at this point and is to extend across the entire valley. Section 1 may be omitted when the degree of contraction is large and the difference in roughness between sections 1 and 3 is not great. In this instance, the upstream water-surface elevation is obtained on the embankment at a distance from the center of the opening equal to one bridge-opening width. When the approach section has much vegetation, friction loss will be a large part of the total fall and must be calculated. If surveying of section 1 is unduly difficult because of this vegetation, estimate its cross section by measurements of the nearest accessible similar cross section, say at the edge of the right-of-way. The value of Manning's n (coefficient of roughness in Manning's formula) selected for the approach section should represent the upper half of the reach between section 1 and the bridge opening.

Section 3 is defined by a straight line parallel to the constriction that marks the minimum area between the two abutments of the bridge. The area, a_3, used in equation 2–41, is always the gross area of the section extending to the free-water surface. It does not exclude the area occupied by piles or piers in the plane of the section, nor does it include any submerged lower part of the bridge. Where large scour holes are under the bridge, the bridge geometry no longer forms the control and the available coefficients do not apply.

Use of a transit is recommended for a field survey. The following method is recommended:

- Locate and obtain the elevation of floodmarks near the constriction. Include marks along the channel edges and marks on the upstream and downstream faces of the embankments forming the constriction.
- Locate the river channel, bridge opening, and all features pertinent to the hydraulics of the site; in brief, locate a plan of the bridge and vicinity.
- Obtain ground elevation for cross sections at sections 1 and 3. If flow also occurred over the highway embankment, obtain elevations (a profile) of the centerline of the submerged portion of the highway. Obtain typical cross sections of the roadway.
- Survey and measure the bridge abutments and embankments so that plan and elevation drawings can be made. Identify all factors that influence the discharge coefficient. Physical features that affect the coefficient are (1) percentage of channel contraction; (2) ratio of abutment length to bridge opening width; (3) rounding of entrance corners; (4) wingwall angle; (5) wingwall length; (6) angularity of bridge with respect to flow line; (7) distance from free-water surface at section 3 to tops of right and left banks under the bridge; (8) embankment slope; (9) abutment slope; (10) shape of entrance when embankments and abutments slope; (11) eccentricity of bridge opening with respect to cross section of channel; (12) area of bridge piles or piers; and (13) degree of submergence of the bridge.
- Select values of Manning's n for sections 1 and 2. If n cannot be decided in the field, take sufficient data in notes and photographs for later study and comparison with channels of known n value.

Estimates of the flow can be made by using standard hydraulic computation procedures. The coefficient of discharge must be estimated, however, as it can range from 1.0 down to 0.50 or less for one type of bridge opening.

Bends in Channel:

Bends in a channel cause spiral flow or counterrotation currents that contribute to moving the bedload downstream and toward the inside of bends. Eddies develop about three-fourths of the way around the bend because of the spiral motion of the flow, causing deposition on the inside of the banks in the bend. The slack water produces some energy loss due to friction. Friction loss also occurs on the walls and bottom of the channel. All these factors produce a measure of control over the flow in the channel. Unless the banks in the bend are stabilized, the bend should not be used as a control.

Channel control.—Channel control is the main control for high flows in natural channels. Bridge openings and bends can produce local disturbances. The natural channel, however, will return to normal flow depth upstream and downstream of the disturbance. Since channel roughness controls the flow depth, changes in roughness will change the depth-discharge relationship. These changes include debris, vegetation, size of bed material, scour or filling of the channel, and bed forms in the channel.

The slope-area method can be used to obtain an approximate discharge. Discharge determined by this method is only approximate, however. It consists of using the hydraulic slope in a uniform reach of channel, and the average cross-sectional area of that reach, to give a discharge rate. The channel reach should be at least 300 feet (91 m), and preferably 1,000 feet (305 m), in length. The course should be straight and should be free of rapids, abrupt falls, and sudden contractions or expansions. Discharge may be computed from Manning's formula:

$$Q = \frac{1.486}{n} A\, R^{2/3}\, S^{1/2} \qquad (2\text{–}42)$$

where

Q = discharge, cubic feet per second;
A = area of cross section, square feet;
R = hydraulic radius (area/wetted perimeter), feet;
S = hydraulic slope (loss of energy/length of reach); and
n = roughness factor depending on character of the channel lining and depth of flow.

Slope and areas must be determined simultaneously if the water levels are changing. The terms $A\, R^{2/3}$ should be computed for each end of the reach and averaged for use in Manning's formula.

Shifting Control:

Control of flow in a natural channel with no natural falls or supercritical velocities depends on the loss of energy within the total system. Increase or decrease in energy loss will influence the discharge rate. Therefore, the control will shift, depending on roughness of the channel.

Effects of scour and fill.—Scour is the erosive action of running water in streams. This action excavates and carries away earth and solid rock material from the bed and banks. Fill or aggradation is the opposite of scour. Scour occurs where local velocities are large enough to remove and transport material. Turbulence level, sediment concentration, water temperature, and size of bed material are other factors that influence scour. Aggradation usually is caused by a reduction in stream velocity. Many changes can reduce velocity. Scour and fill change the area of a channel and the loss of energy in the channel. Thus, the control is changing continually. A current-meter rating is necessary at all times during a flow event.

Channel change above the control.—Changes in the channel above a control will affect the velocity of approach to the constricted area of the control. These changes may result from aggradation or degradation of the main channel or changes in vegetation along the banks. Therefore, the rating (depth-discharge relationship) will shift. The amount of the shift will depend on the relative change in the channel. Such changes also may affect precalibrated measuring devices and are a continuing problem when collecting accurate runoff records.

Return of overbank flow.—Constriction or bridge openings force the flow to pass through a limited area. This produces lateral flow from the overbank areas to the main channel both upstream and downstream of the constriction. Control of the flow is a combination of the whole system both upstream and downstream. Submergence of the bridge occurs because of backwater from the flow downstream. Control may shift from above the bridge, to downstream, and back again. The only measure of flow at such a location is the head loss through the bridge or constriction. Discharge determined by this method is only approximate.

Effect of ice.—Ice can shift the control for various reasons. If the stream is wide (200 ft or 61 m) and the ice is 1 to 2 feet (30.5 to 61 cm) thick, the ice cover is sufficiently flexible to rise and fall with the stream and the stream can flow as a closed conduit. The underside of the ice can become rippled due to the fluid turbulence when the ice is formed. If the stream is narrow, bridging or airspace can occur under the ice. In the spring and fall slush, ice can form on the surface and present problems for current metering. Ice jams in the spring can produce shifts in the depth-discharge relation and in the flow control position. Ice in streams causes a variable backwater that only can be determined by frequent discharge measurements during such periods.

Effect of vegetal and aquatic growth.—Aquatic growth increases the energy loss of a stream. This can produce changes in the approach velocity to a current-meter station, and it can shift the stage-discharge relationship seasonally. Increase in vegetal and aquatic growth also can shift the flow from the main channel to the overbank.

Other Factors in Selecting Gaging Station Locations

Field-calibrated gaging stations should be located in straight, uniform reaches of channel having smooth beds and banks of a permanent nature whenever possible. Their location should be far enough from sewage outfall, power stations, or other installations causing flow disturbances so that the relationship of gage height to discharge will not be affected. In many channels these conditions are difficult to locate, and care must be taken to obtain a satisfactory station. Daily current-meter measurements may be necessary where sand shifts occur.

Geology of the watershed must be considered in the location of a gaging station. Ground water flow and surface return flow can be important elements of the total hydrologic analysis of the flow. The use of rock formations for support of precalibrated runoff measuring devices and dissipation of the added energy created by these devices can save on installation costs and prevent failure.

Topographic features of the watershed should be considered in locating gaging stations. If ice is a problem, a measuring device may be located in a protected area that receives sunlight most of the time. Orographic features may determine the spacing of gaging stations within a watershed.

Instrumentation

The stage of a stream is the height of the water surface above an established datum. It usually is expressed in feet (meters) and hundredths of a foot (meters).

A control provides a stable cross section wherein the rate of flow can be determined if the stage is known. The frequency of flow determinations is dictated by the purposes and conditions of the study. Instruments are commercially available to provide continuous stage records. These instruments are particularly useful when a record of the entire hydrograph is needed on rapidly fluctuating streams. When less detail is needed or when flow fluctuation is gradual, periodic readings on a graduated staff may be adequate. When only maximum peak flow information is needed, a crest gage will protect and retain a high watermark for subsequent observation.

Water Stage Record

Water stage record is important in the runoff study. The rate of flow is determined by applying the depth discharge relationship to the stage reading. Therefore, reliability of the stage reading is important.

Nonrecording Gages

Two general types of nonrecording gages are (1) wire-weight gages, float, or point gages, which measure from a fixed point, and (2) staff or crest gages, which read stage directly.

Wire-weight gages.—Various wire-weight gages have been used for many years, usually as an independent reference of water levels outside the stilling well at gaging stations. The gage height indicator on the wire-weight gage is set to correspond with the recorder datum. When the bottom of the weight is at the water surface, water level is checked to assure accurate recording. Wire-weight gages are useful at bridges, weirs, and other structures over the water. These gages consist of mounting frame, turning handle, cable drum, water-level indicator, and weight wire or cable and weight (7). They can be attached securely to a wall or bridge rail for ease in lowering the weight and observing the water level.

Staff gages.—Staff gages are used as an easily visible reference of water level for stage recorder charts and for periodic observation of water levels where continuous records are not required. Staff gages should be porcelain, enameled iron, or stainless steel. They should be fastened securely to a vertical wall, column, or other well-anchored support. For best visibility, staff gages should be separated from the high-velocity area in the stream. They should be referenced carefully to a permanent elevation marker and the recorder datum for ease in checking the gage setting after frost and ice periods, which may damage or loosen the support. Reservoir staff gages often are fastened to an inclined timber, metal, or concrete surface. This surface must be graduated carefully and placed accurately to insure correct stage readings. The location and setting of stage gages for weirs, flumes, and gaging stations should follow recommendations in appropriate handbooks (*23, 41*).

Crest gages.—These gages are an inexpensive means of determining maximum stage when no observer is present. They indicate the maximum stage reached between visits of the observer and provide useful information for computing peak discharge of flood events. They are located with reference to bridges, culverts, spillways, and stable streambank sections for which stage-discharge relationships can be obtained.

A commonly used crest gage consists of a well-anchored 2-inch (5 cm) galvanized pipe with a perforated cap on the bottom and a fill cap or pipe cap with vent hole on top. A length of thin-wall conduit or a wood measuring stick is placed within the pipe, as shown in figure 2.42. Extreme caution should be taken when inserting a wood stick since rapid lowering may cause a false high water measure. A supply of powdered or granulated cork in the pipe rises with the water level and adheres to the measuring rod, thus recording maximum water level. A graduated rod simplifies reading of high water levels. The cap should be locked or fastened securely for protection against vandalism. Figure 2.43 shows a typical crest gage installation.

Float-Type Recording Gages

Many float-actuated water-level recorders are available from instrument manufacturers. The recorder and accessories best suited for a specific site depend on (1) the time interval between chart changes and clock servicing, (2) expected maximum rate of stage changes, (3) available funds for site instrumentation, (4) importance of continuous records, (5) accuracy required in flow computations, and (6) compatibility with other instrumentation in the total program. Drum-type recorders are much cheaper, require weekly servicing, and are less compatible with other hydrologic instrumentation. Therefore, more expensive strip-chart recorders usually are used at remote locations where continuous records are necessary.

The water-level recorder should be equipped with a graduated float tape and index pointer to enable the observer to check the recorder pen reading against the actual water level in the stilling well and on the outside staff gage. Careful calibration of the water-level recorder before field installation will prevent inaccurate data from being recorded. Proper calibration and adjustment assure that (1) pen reversals occur at the appropriate edge of the printed chart, (2) the pen arm is the proper length and applies proper pen-inking pressure, (3) binding in the bearings or misalinement in the mechanism does not occur, (4) the pen follows the chart lines on the full traverse, (5) no recorder parts are missing or damaged, and (6) the clock is timed properly.

Field installation should follow instructions provided with the recorder. Detailed planning of the field installation should provide:

- Manufacturer's installation instructions.
- Careful packing of the recorder and accessories for safe transport.
- Proper tools to complete the installation.
- Adequate personnel to establish a precise recorder level datum.
- Necessary equipment for a field calibration.

Drum-type recorders.—Vertical or horizontal drum-type recorders are used in runoff studies on small watersheds and plots where visits are scheduled to the site weekly or more frequently. These recorders may be driven by spring-wound, falling-weight, or battery-powered clock mechanisms and usually are designed with interchangeable gear ratios for time and gage scales. The time scale varies with the anticipated maximum rate of change in stage. A 6-hour drum rotation (1 min resolution) may be needed on most plot studies. Most recorders have reversing mechanisms that do not limit

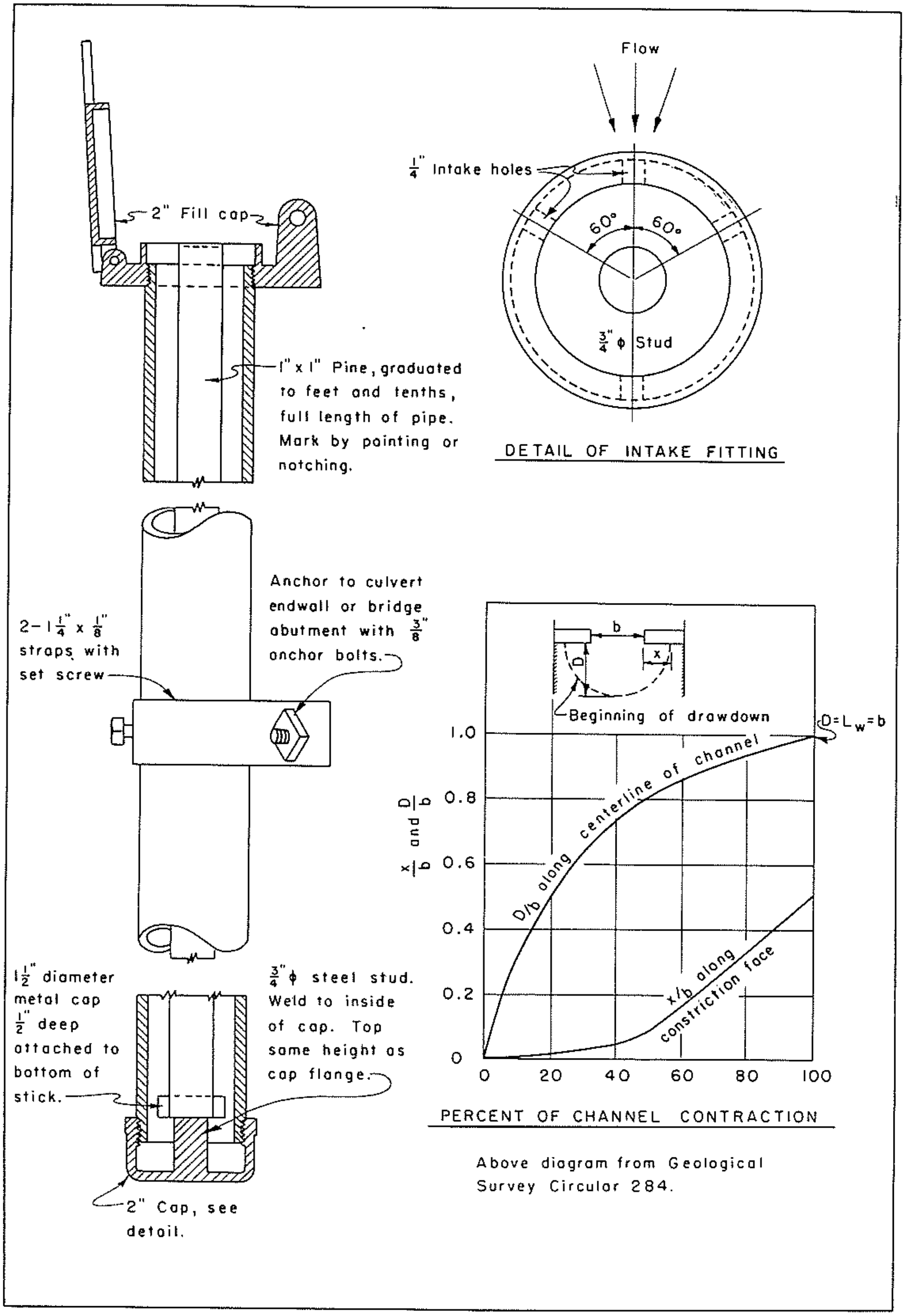


FIGURE 2.42.—Details of a maximum stage gage and its location.

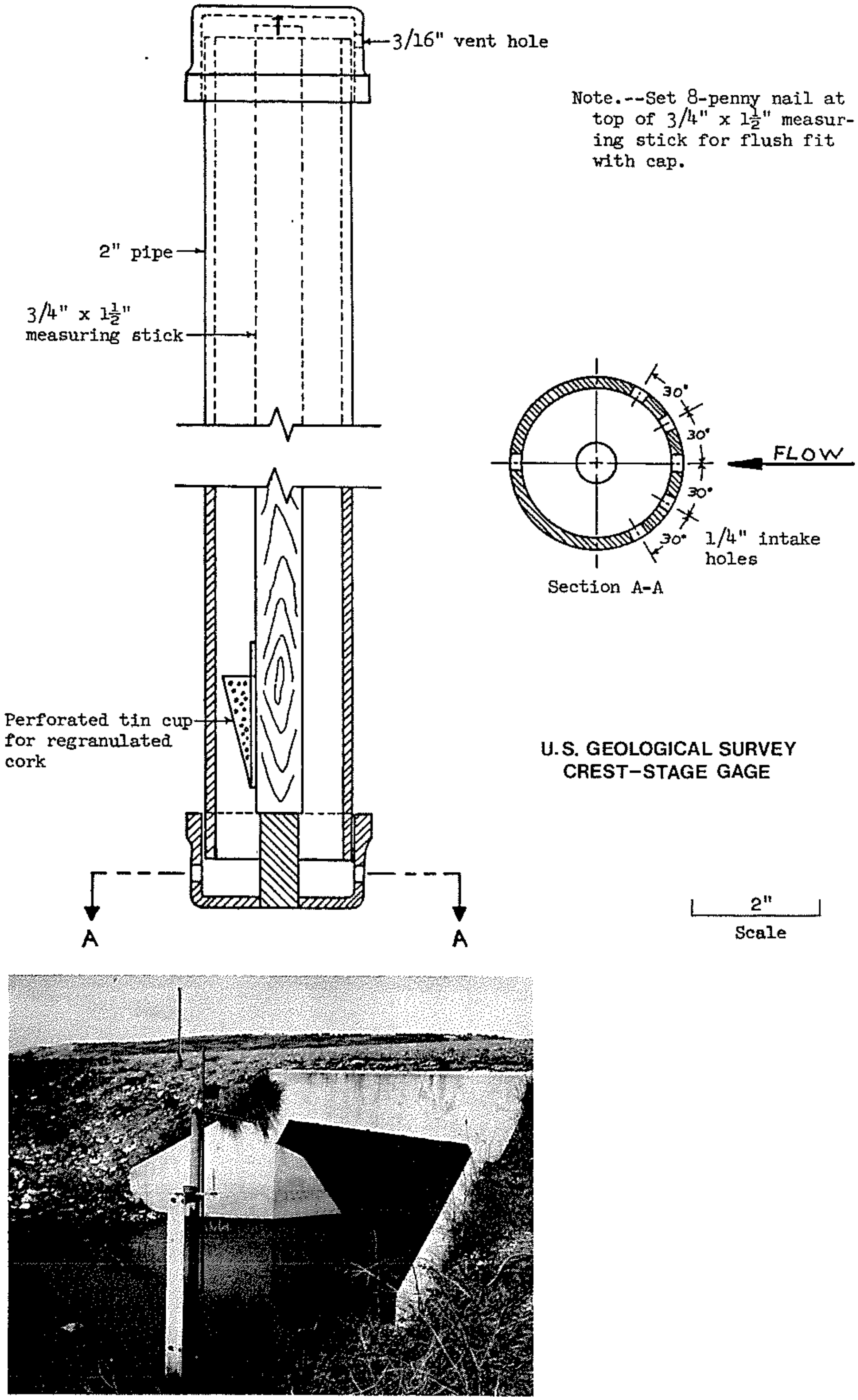


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FIGURE 2.43.—Typical crest gage installation at a highway bridge with detailed drawing.

the range in stage. Instruments have been adapted for self-starting in special studies (*15*).

Strip-chart recorders.—Most strip-chart recorders will operate unattended for a month or longer, depending on the length and speed of the chart. Clock and drive mechanisms usually are powered by a falling weight, a spring drive, or a battery. For unusual installations, chart speeds are available from about 1 inch (2.54 cm) to over 700 feet (213 m) per day. The range in stage may vary from a few inches to several hundred feet. The common A–35 recorder accommodates a strip chart 25 yards (23 m) long and usually is changed 1 to 12 times per year, depending on the chart speed selected.

The pen reverses at each chart margin to record a wide fluctuation in water level. Many floats, float tapes, float pulleys, time-scale gears, stage-scale gears, and driving mechanisms are available on order.

Catalogs describing strip-chart recorders are available at many Federal and State offices concerned with hydrologic measurements. Numerous publications explain suitable locations and successful installations (*7, 34*).

Digital punch recorders.—Digital punched tape recorders are battery-operated, paper-tape recording instruments that mechanically convert angular positions of a rotating shaft into a coded digital output. The primary actuating unit permits liquid level analog measurements to be digitally converted and recorded on punched paper tape.

The basic recorder (fig. 2.44) may be used in conjunction with many primary actuating devices for measuring and recording parameters. The liquid level recorder has been designed, however, to measure liquid level by a cable, drum, and float assembly similar to that used in drum and strip-chart recorders. The measurement source may be geared to the liquid level recorder to obtain a convenient ratio

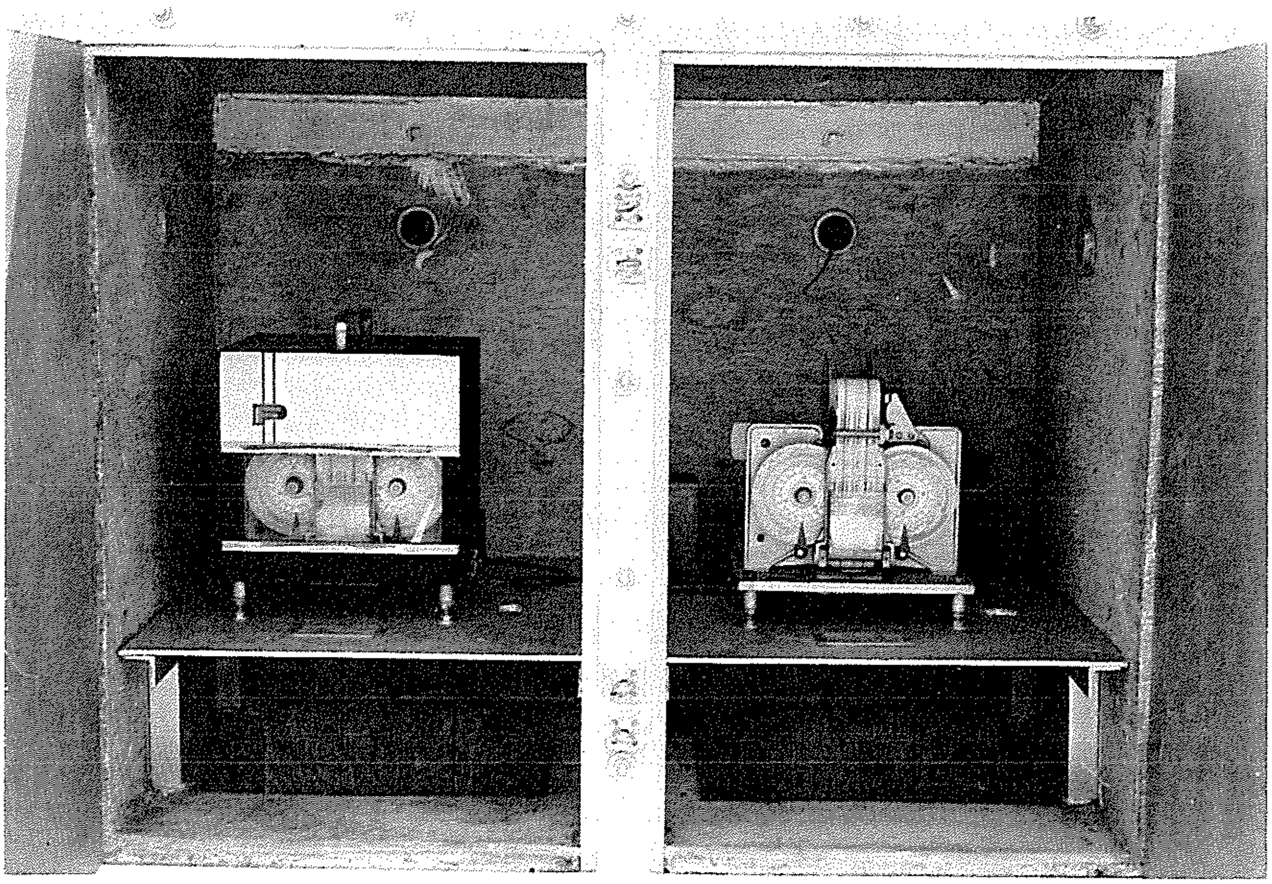

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FIGURE 2.44.—Digital punched tape recorders: *Left*, exterior; *right*, interior.

between relation of the recorder input shaft and rotation of the primary actuating unit shaft. For example, when used with a liquid level transducing system having a 1.0-foot (30.5 cm) circumference float wheel on the input shaft, each revolution of the input shaft will represent a change of 1 foot (30.5 cm) in liquid level; that is, the low-order code disk (fig. 2.45) will represent the hundreds and tens digits. The high-order code disk will represent the units and tens digits. This instrument may include a push button mounted on the front plate, which initiates a punch sequence upon demand, primarily for testing.

The punched-tape recorder is operated by two interrelated systems—the input gearing-count memory system and the punch programing cycle system. The input system receives the shaft motion from an external source and converts this motion to successive positions of memory code disks. By means of a clock mechanism, the punch programing system initiates the punch and reset operations required for recording measured values on a data recording tape.

The timer unit, consisting of a clock, cam, follower, and switch assembly, derives its operating power from an external $7^1/_2$-volt dry cell. The clock continuously rotates a cam arranged so that a switch is actuated every 5, 15, 30, or 60 minutes, or each 12 hours, as specified. The cam also is used for setting the time within the specified cycle.

An available optional electronic timer is recommended for improved accuracy and dependability. This timer consists of a precise quartz crystal oscillator whose frequency is counted by cascade flip-flops. The resultant count of the last stage is the time cycle. This cycle may be 5, 6, 15, 30, or 60 minutes, with the crystal excited by a $7^1/_2$-volt battery. Output is through a silicon-controlled rectifier. The timer has no moving parts and is unaffected by environmental changes.

Two recorders may be actuated by one timer equipped with a special slave unit to provide an almost simultaneous (about 3 seconds difference) punchout of two bits of data. This is especially valuable in applications that obtain synchronous stages upstream and downstream of weirs, flumes, or drop structures.

At designated time intervals, the water level is punched on a paper tape (fig. 2.46). (A foil-back tape has been more satisfactory than regular paper tape.) Data are punched on tape

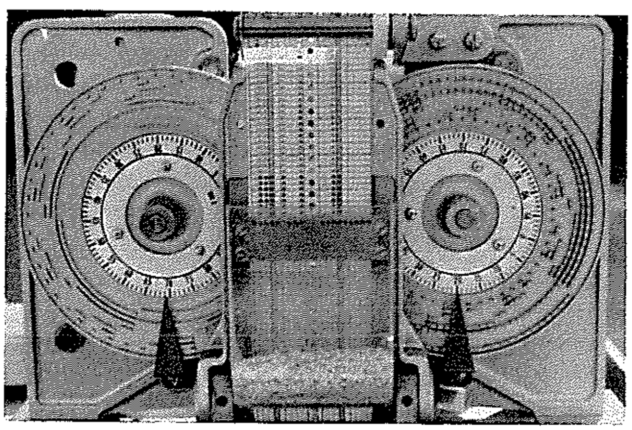

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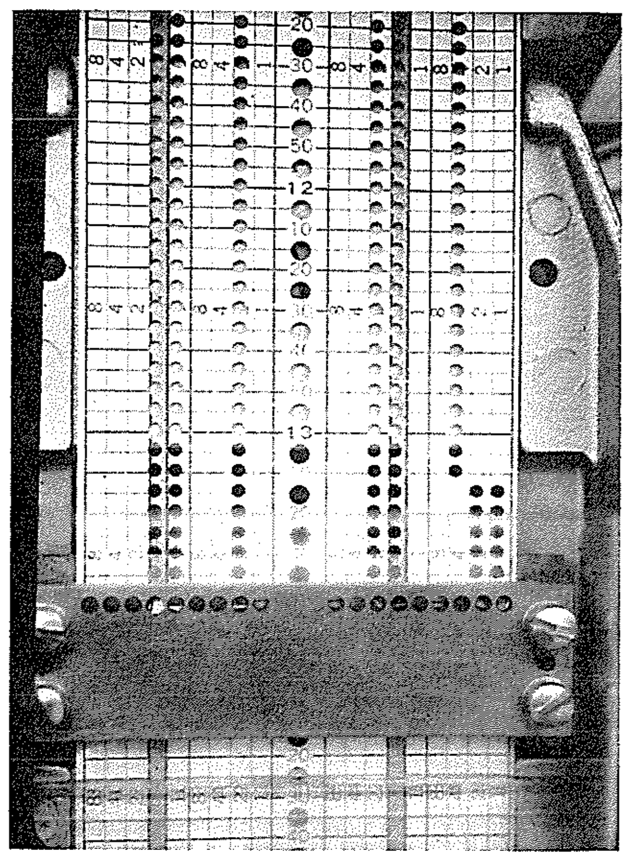
FIGURE 2.45.—Code-disks of a digital punch recorder.



PN-5883
FIGURE 2.46.—Paper tape punch unit.

in the binary-coded decimal system. Four binary digits (bits), 1, 2, 4, and 8, are added together to form a single digit. The recording tape used has time printed in the center, based on a 24-hour clock. After each punchout, the tape will move one-tenth of an inch and will position itself for the next punchout. The punched columns on the tape are printed so that actual values may be read by the observer at any time.

Special translators are available to convert punched-tape data into punched cards, or magnetic tape, for rapid computer processing. This process is discussed on page 184.

Punched-tape recorders can be equipped with optional equipment to present a parallel binary-coded decimal output in electrical form. This output will have the value that appears on the data recording tape for telemetering to remote equipment.

Bubble Gage Servo-Manometer

The U.S. Geological Survey developed the bubble gage servo-manometer shown in figure 2.47 to obtain more reliable records of surface water level of streams subject to large variation in bed elevation. The bubble gage is also useful on reservoirs or streams subject to large variation in water level. A standard bubble gage will record a 50-foot (15.2 m) range of water level. The initial cost of a bubble gage and its installation is often less than the cost of installing a gage well. A bubble gage can be moved to another gaging site with little expense, whereas movement of a gage well may be more expensive than installing a new gage well.

Sand point and orifice installation.—An excellent sand point for the orifice housing can be made from a 4-inch (10 cm) I.D. galvanized pipe about 20 feet long, as shown in figure 2.48. The orifice is placed in the side of the sand point near the top. While a vented cap on the sand point minimizes sediment entry, it permits nitrogen gas to escape so pressure will not build up in the sand point. Transfer of pressure from the stream to the sand point is provided by a series of lengthwise slots. These slots are covered with screen wire to hinder sediment entry.

When driven or jetted into the streambed, the 20-foot (6.1 m) length of sand point is sufficient to assure stability of the orifice. This length also provides ample storage for the sediment that enters the sand point during several runoff events. All steel in the assembly is galvanized to inhibit corrosion. The sand point may be installed by:

- Driving it into the streambed. A crane is required for this method.
- Jetting it down in one piece. A bridge or other structure to which block and tackle can be attached is needed for this operation. Separate tackle are required for the sand point and the jet pipe because the jet pipe must be able to move independently.
- Jetting it down in sections. The first section should be 10 feet (3 m) long, and additional sections should be 5 feet (1.5 m) or less. Couplings are needed for both the sand point and the jet pipe.

The jet pipe may be constructed by 1½- or 1¼-inch (3.8 or 3.2 cm) pipe. The pump used in jetting should have a capacity of 250 gallons per minute (950 l/min) at 15 pounds per square inch (10,547 kg/m) pressure.

The 4-inch (10.2 cm) sand point can be turned easily with a 36-inch (91.4 cm) chain wrench during installation. The saw-toothed edge at the bottom end of the sand point assists in cutting through tree limbs, other buried objects, and layers of gravel.

Elevation of the orifice is determined by the anticipated minimum elevation of the channel bed. The orifice must be set sufficiently low initially to measure the minimum anticipated stage because the pipe is difficult to lower or raise after the sand has settled around it. The vent assembly can be extended upward above the streambed, however, by simply removing the vent assembly and adding a 4-inch (10.2 cm) coupling and nipple, of the appropriate length, to the top of the sand point. The vent assembly performs best when all openings are above the streambed.

During low flows, a sand streambed meanders and may move away from the sand point, rendering the orifice inoperative. The low water channel may be stabilized laterally to prevent this meandering by installation of low water jetties that keep water flowing over the sand point. Jetties have been constructed by spurting 10- to 14-foot (3 to 4.2 m) lengths of angle

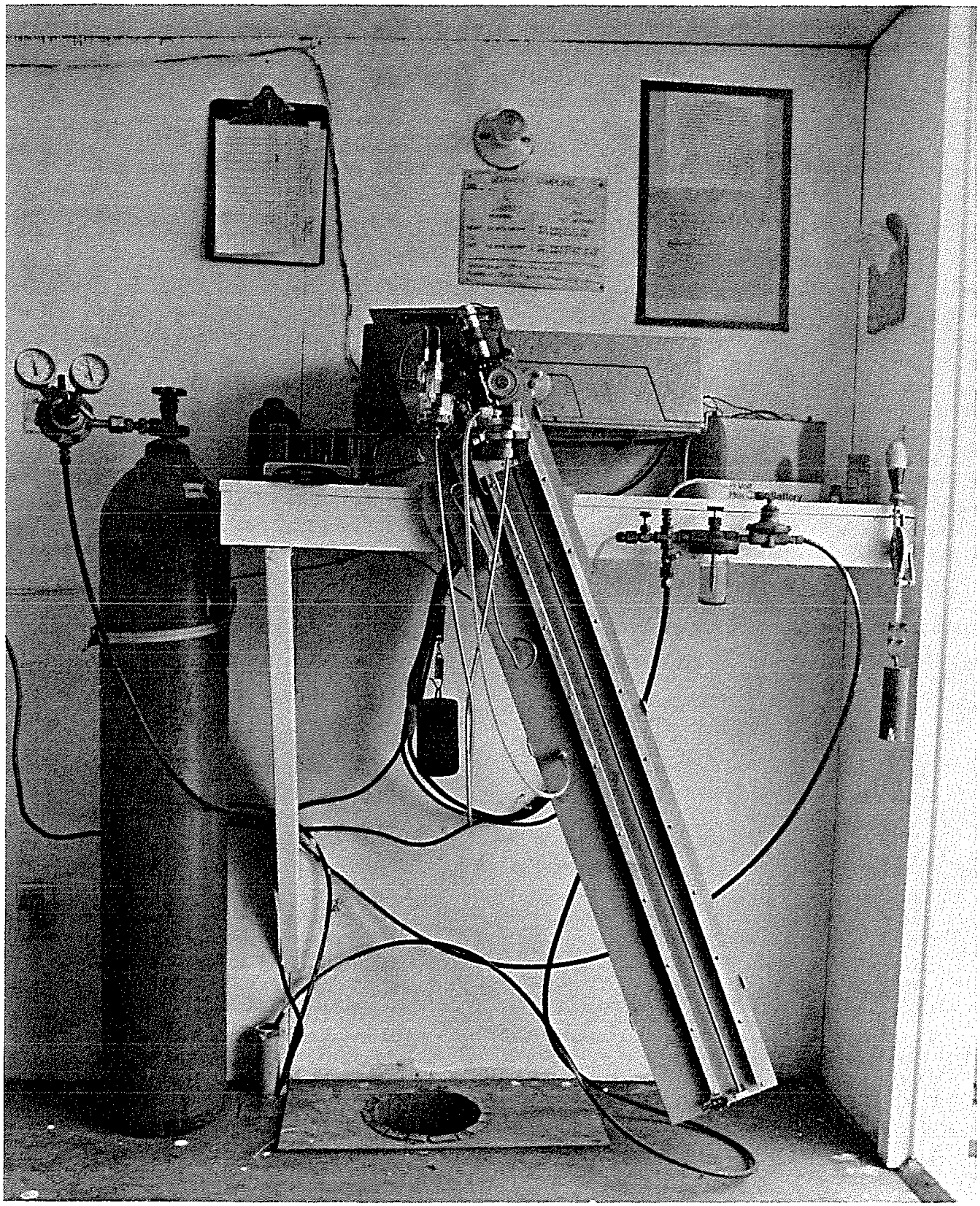
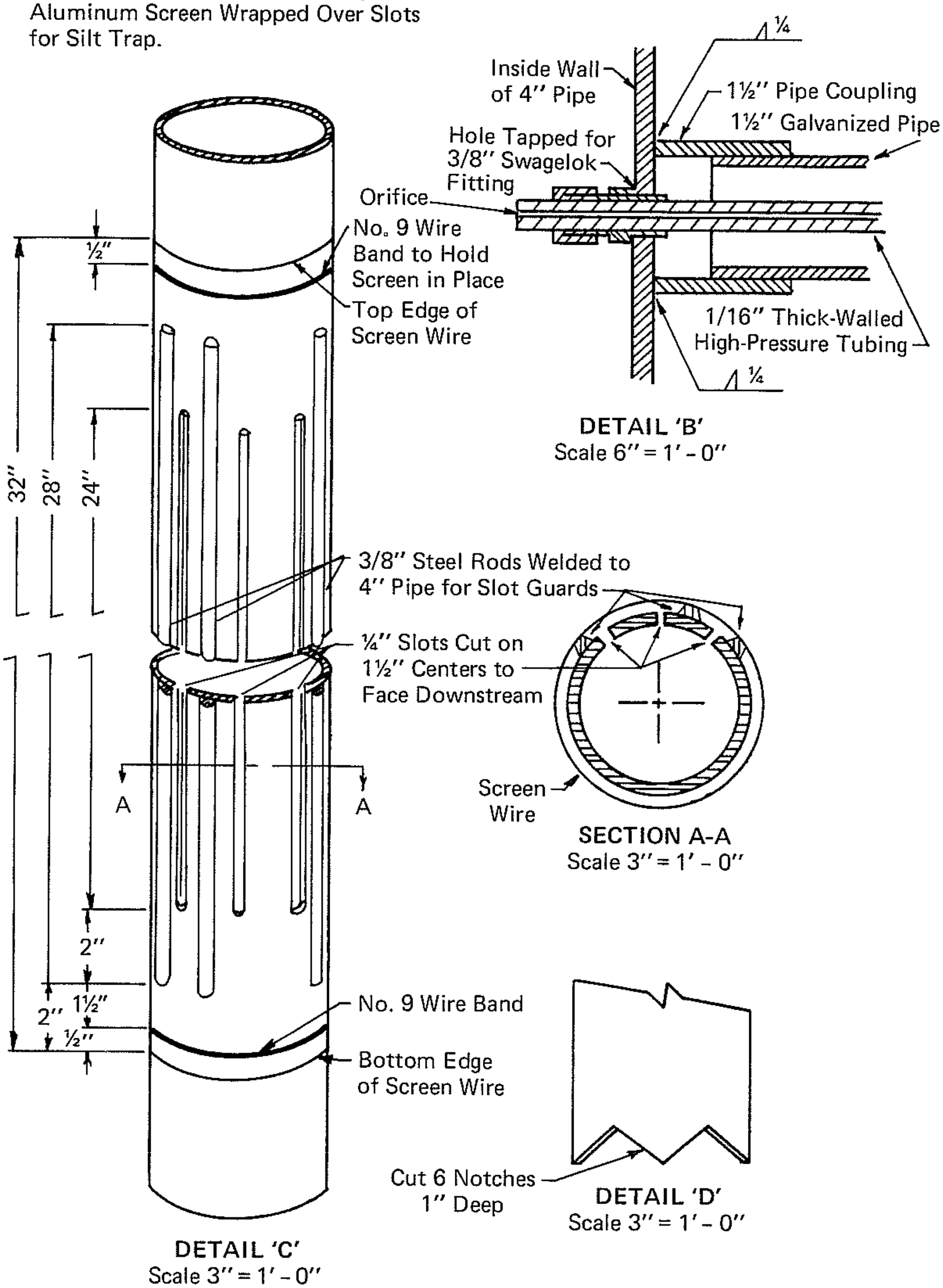


FIGURE 2.47.—Bubble gage servo-manometer water level sensor.

NOTE: 72" x 32" Piece of 16 Gage Mesh Aluminum Screen Wrapped Over Slots for Silt Trap.



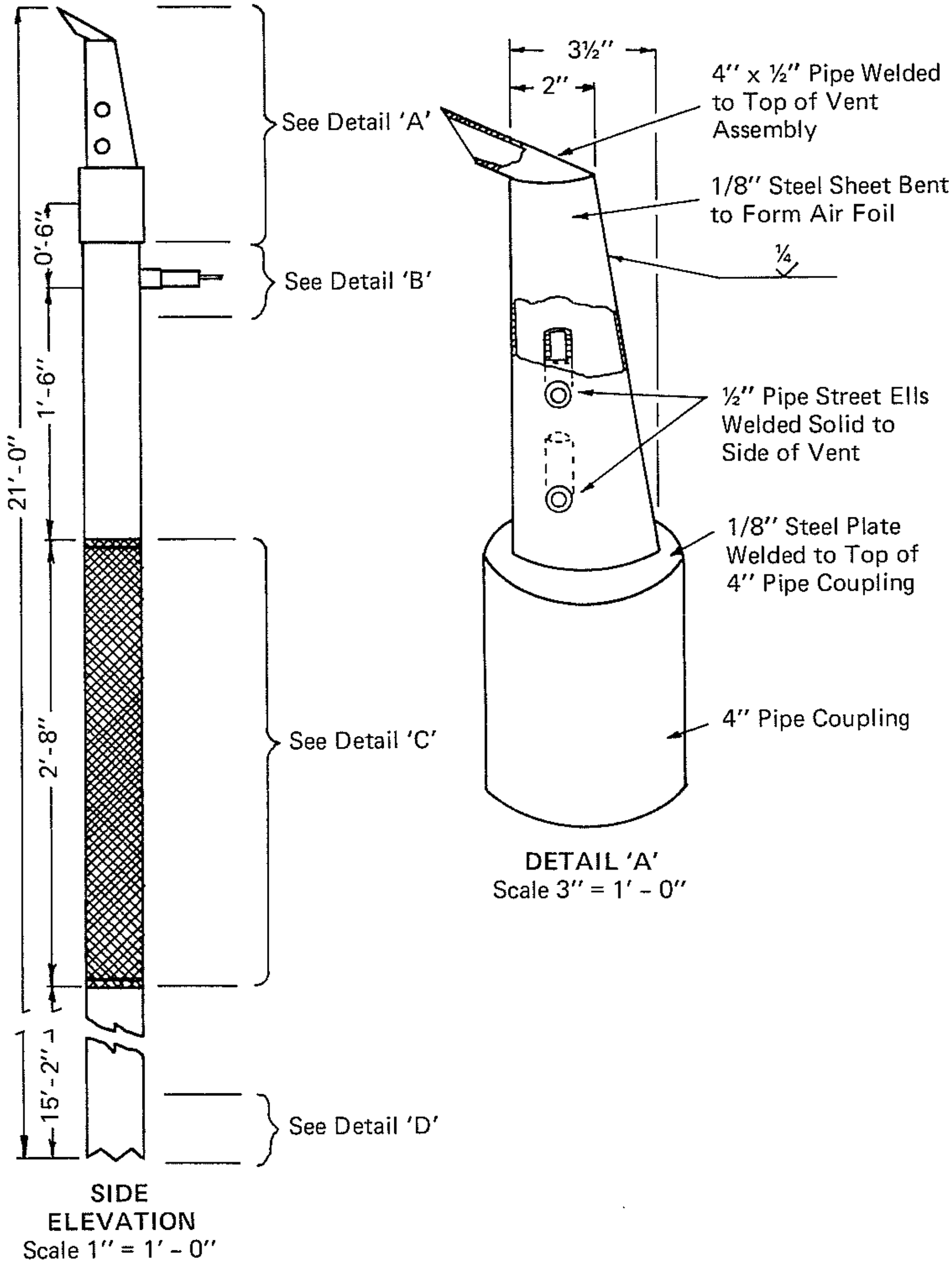


FIGURE 2.48.—Giant sand point with built-in silt trap and vent: *Left* (page 140), details C, B, and D; *Above*, detail A.

iron into the streambed and lacing them together with number 9 wire. Chicken wire of $1^1/_8$-inch mesh, attached to the angle iron projections, catches sufficient debris to deflect the low flow current. Tops of the low water jetties should not exceed 1 foot (30.5 cm) above the channel bottom.

Manometer installation.—Federal law requires that the nitrogen bottle be capped during handling to avoid accident. This bottle should be capped until it is fastened securely in the shelter. Although oil-pumped nitrogen is preferred, dry nitrogen is satisfactory.

Plastic tubing from gage to orifice should run downgrade continuously to avoid low spots where water or oil might collect and cause an erratic record. Splices in the line should be avoided. If a splice is necessary, it should be readily accessible for checking leaks. All threaded fittings in the purge system should be doped, except for the zytel fittings and connections between the nitrogen bottle and the constant pressure regulator.

Tube connections to the mercury reservoirs usually are not tightened at the factory. Tighten these connections with a small wrench before pouring the mercury into the float-switch reservoir. Slightly more mercury may be required than that which comes with the gage. Disengage the motor and turn the manometer reservoir shaft till the reservoir moves through the expected range of operation. Adjustment in the track may be required if the reservoir assembly does not move freely. A negator constant-torque spring drive or an electric drive is preferred to the weight drive because provision must be made for adequate vertical movement of the weight between servicings.

Electronic Transducers for Measurement of Water Level:

If alternating-current electricity is available at a measuring site, several transducers are available for measuring water elevation. These transducers have output that is compatible with high-speed digital computers. The field of electronic measurements is so dynamic that significant information for a handbook may become obsolete before the handbook becomes available.

At the Southwest Watershed Research Center in Tucson, Ariz., water depth has been measured successfully with pressure transducers located at the streambed or flume bed (must correct for water-sediment density changes); sonar transducers, which are suspended above the flow on a rigid support; and a potentiometer, which may be attached to the float wheel of a conventional float-tape-counterweight system in a stilling well. The EMF outputs from such transducers may be recorded on data loggers in digital form, using punched or magnetic tapes. They also may be recorded on a strip chart. The response time of such transducers is generally rapid, and the transducers provide better time resolution than spring-wound clock-type recorders. Although costs are decreasing for such equipment, they are still higher than the costs of mechanical equipment. Additional accuracy justifies the cost in many instances, however.

Stilling Well

Regardless of the type of runoff control used, the well should be located to one side of the waterway so it will not interfere with the flow pattern over the spillway. The well for a triangular concrete weir should be 10 feet (3.05 m) upstream from the centerline of the weir crest as the laboratory discharge calibrations are based on head measurements at that distance. If other structures are used for the control section, the well should be located far enough upstream to be out of the area of drawdown at the control. Tall wells required on detention reservoirs and ponds can be located near the spillway at the toe of the fill, or they can be dug into a side bank. Setting the well back in the bank will reduce the difficulty of supporting it and will allow for access by walkway rather than ladders. This location presents some problems in intake design, however. If the well is set at the toe of the slope and access to the instrument shelter is by ladder only, the station cannot be serviced during high stages, which may result in the loss of valuable records. Wells for metal flumes are fabricated as part of the flume.

The size of the well selected will depend on such factors as the required rigidity, height, type of material, necessity to get inside, and, possibly, protection from freezing. If ability to

TABLE 2.8—*Suggested pipe sizes for stilling wells*

Heads up to—	Diameter of corrugated pipe	Metal
	Inches	*Gage*
5 feet	18	14
5 to 10 feet	24	12
10 to 20 feet	30	12

get inside the well is unimportant, see table 2.8 for sizes of pipes that should provide the required stability. This table contemplates a length of intake pipe of only 15 feet (4.6 m). If actual installation requires a much longer pipe, the diameter or number of pipes will need to be increased. Although the minimum requirement is one pipe, two pipes are recommended because one may become plugged. They should be installed at slightly different elevations. When installing intake pipes, make provisions for flushing out the silt.

The well should be watertight below the lowest intake. All seams and rivets must be brazed or treated both inside and out. An asphalt or tar seal should be made on the inside at the junction of the pipe with the concrete base. Two access doors should be provided: One, near the top, to allow for adjusting the tape length and for changing gears without removing the recorder; the other, just above low water, to assist in checking the setting of the recorder and to permit removal of silt without entering the well. On low-head installations, one door with its bottom slightly above the lowest intake may be sufficient. Rusting of hinges on these access doors has caused considerable difficulty. Hinges of rust-resistant metal, such as brass or bronze, should be used. If no hinges are used, the door can be held in place by wingnuts on short bolts welded to the well.

When streams containing heavy sediment loads are measured, a reservoir must be provided beneath the intakes to store the sediment dropping through the intake slots. Experience in ephemeral streams with heavy bedloads has shown that small slots and a small stilling well provide the best record. A 3/4-inch (1.9 cm) pipe has been used successfully to hydraulically connect the intake to the stilling well. The intake system is then kept with water by providing flow from an auxillary reservoir with the intake water level held constant by a float valve. Air traps in the line between the stilling well and the intake must be eliminated to provide an accurate stage record. These small intakes have been inserted in larger systems and have eliminated problems connected with the intakes' silting full.

Where ladders are necessary, use 1/4- by 2-inch (0.64 by 5.04 cm) iron straps as rails. Space these straps 12 inches (30.5 cm) apart and use 3/4-inch (1.9 cm) round rods, 12 inches (30.5 cm) on centers, for the rungs. Bolt or weld the ladders to the well, using angle iron clips so the rungs are 3 inches from the pipe. Complete all fabrication work in the shop before taking the well to the field. After all cutting and welding are completed, give the well a coat of metal priming paint (especially those areas where galvanizing has been destroyed during fabrication), followed by a coat of aluminum.

Place the well on a concrete base that extends below the maximum penetration of frost. The top of this base need not be more than a few inches below the lowest intake. This will not provide much space for sediment storage, but removing small accumulations frequently is easier than removing the accumulation of several years at one time. Although the concrete base may need to extend 4 feet (1.2 m) below spillway elevation, corrugated pipe does not need to extend more than 12 inches (30.5 cm) into this base. The base should extend a minimum of 6 inches (15.2 cm) beyond the corrugated pipe. Figure 2.49 shows a typical stilling well that might be used with a triangular concrete weir or other low head installation.

Large streams often have a bridge pier in the stream at the gaging station. During a rise, the bed may scour several feet around the pier. A self-cleaning gage well can be made by installing a funnel-shaped bottom in the well and attaching the well to the pier at this elevation so that the bed will scour below the bottom of the well during a rise.

Intakes:

Intakes can be one or more galvanized pipes, a series of rectangular slots, or several 1-inch-diameter (2.54 cm) holes. A general guide to the size and number of intakes required is that their total cross-sectional area should be at least 1 percent of the cross-sectional area of the stilling well. Another guide is to limit the head loss in the intake for the expected maximum rate of change in stage to a given maximum amount, such as 0.02 foot (0.61 cm). For example, if the maximum rate of change in stage is assumed to be a half foot per minute, the minimum number and size of intakes for wells of different sizes are shown in table 2.9.

If an intake channel is dug between the pond and the well, it should have concrete sides and bottom and a perforated metal cover flush with the natural bank. This cover should be hinged so as to permit easy access for periodic removal of the accumulated silt. If 1- by 6-inch (2.54 by 15.2 cm) intake slots are used, only the minimum number need be provided above the line of possible maximum silt accumulation. Except for locations where storage is considerable below the spillway crest, the lowest intake slot would be only 0.03 to 0.05 foot (0.91 to 1.5 cm) below the spillway. If a considerable volume of storage occurs before discharge through the spillway, the lowest intake must be placed near the point of zero storage to get a complete record of rates and amounts of runoff.

Outside Staff Gage Supports:

Supports for the outside staff gage should be located the same distance upstream from the spillway crest as the intakes to the stilling well and at a point where they will not interfere with the flow pattern. They should be independent of, rather than attached to, the stilling well.

TABLE 2.9.—*Minimum intakes for stilling wells if maximum rate of change in stage is assumed as 0.5 foot per minute*

Diameter of well (inches)	Intake pipes[1]		1- by 6-inch slots	1-inch round holes
	Number	Diameter		
		Inches	*Number*	*Number*
15	1	3	1	5
18	1	3	1	7
24	1	3	1	11
30	2	3	2	15

[1] Intake pipe 15 ft. long.

A satisfactory support for permanent installations is a section of 6-inch (15.2 cm) channel iron imbedded about 18 inches (45.7 cm) into a concrete base and extended above the base to the maximum height desired. The concrete base should extend well below the maximum penetration of frost expected, and the top of the lowest concrete base (if more than one is required) should be a few inches below the lowest intake to the well. A piece of 2- by 6-inch (5.2 by 15.2 cm) cypress or other durable wood is bolted to that portion of the channel iron above the concrete base. An enameled staff gage section is fastened to the wood with brass screws when the instrument is installed.

Permanent Bench Marks:

Locate permanent bench marks of brass on and near the control or spillway and set in concrete bases in undisturbed ground. A concrete base set in fill material will settle. The base for the staff gage support also can serve as the location for one bench mark. Set another bench mark on the control or entirely independent of any part of the station. For independent installations, extend the concrete base below the maximum penetration of frost. To assure accuracy of records, check the setting of the instrument and staff gages a minimum of three times each year—once in the winter, once in the spring immediately after the frost is out of the ground, and once in midsummer.

Stilling Well Siphon System:

A siphon system has been used successfully as a reservoir stilling well intake. This system is often less expensive to install than conventional intake pipes. A sample installation is shown in figure 2.50. Maximum height of the siphon summit above the surface of the water is limited to the minimum atmospheric pressure expressed in feet of water at the installation site, less the vapor pressure in feet of water.

For a successful system, entrance of air into the siphon line must be minimized. This requires care in installation of all fittings, valves, and joints. Plastic tubing is not recommended for the siphon line. If no leaks exist in the system, a 600-in^3 (9,834 cm^3) trap will not fill

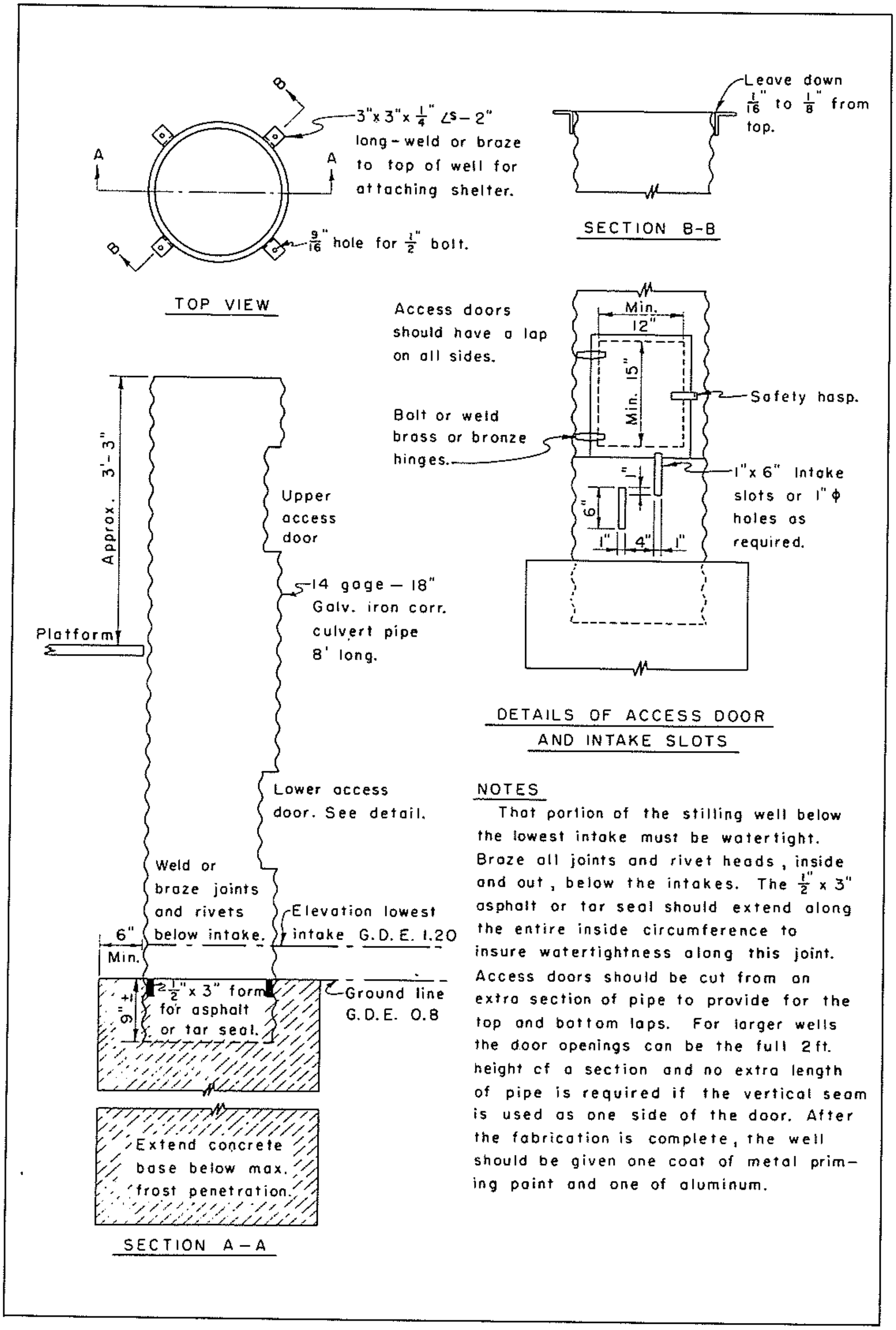


FIGURE 2.49.—Details of stilling well for low head installations.

with air in less than a year under normal fluctuations of water level.

Connection of the siphon line to a well point in the reservoir or stream is recommended to inhibit entrance of sediment into the system. A sufficient rate of water transfer must be provided, however, to prevent a significant lag in the gage height record. Relations between stilling well diameter and siphon line diameter for rates of change in gage height are the same as those for a conventional stilling well system.

Instrument Shelters:

Instrument shelters can vary from simple shelters, just large enough to cover the instrument and hinged to lift in the same direction as the instrument cover, to those commonly used on large streams that permit the observer to enter and remain inside with the door closed. If a simple shelter is used, the instrument cannot be serviced during periods of precipitation and charts and other supplies cannot be stored.

The most practical shelter for small watersheds is just large enough to cover the top of the wall and high enough to lift the recorder cover and change charts for checking the instrument even during severe weather. The door should be hinged at the top to provide protection for the observer when it is opened. Skirts can be made from pressboard or plywood and can be attached at right angles to the door near its edges, to afford additional protection to the observer in stormy weather. A double door also can be used, the upper two-thirds of the

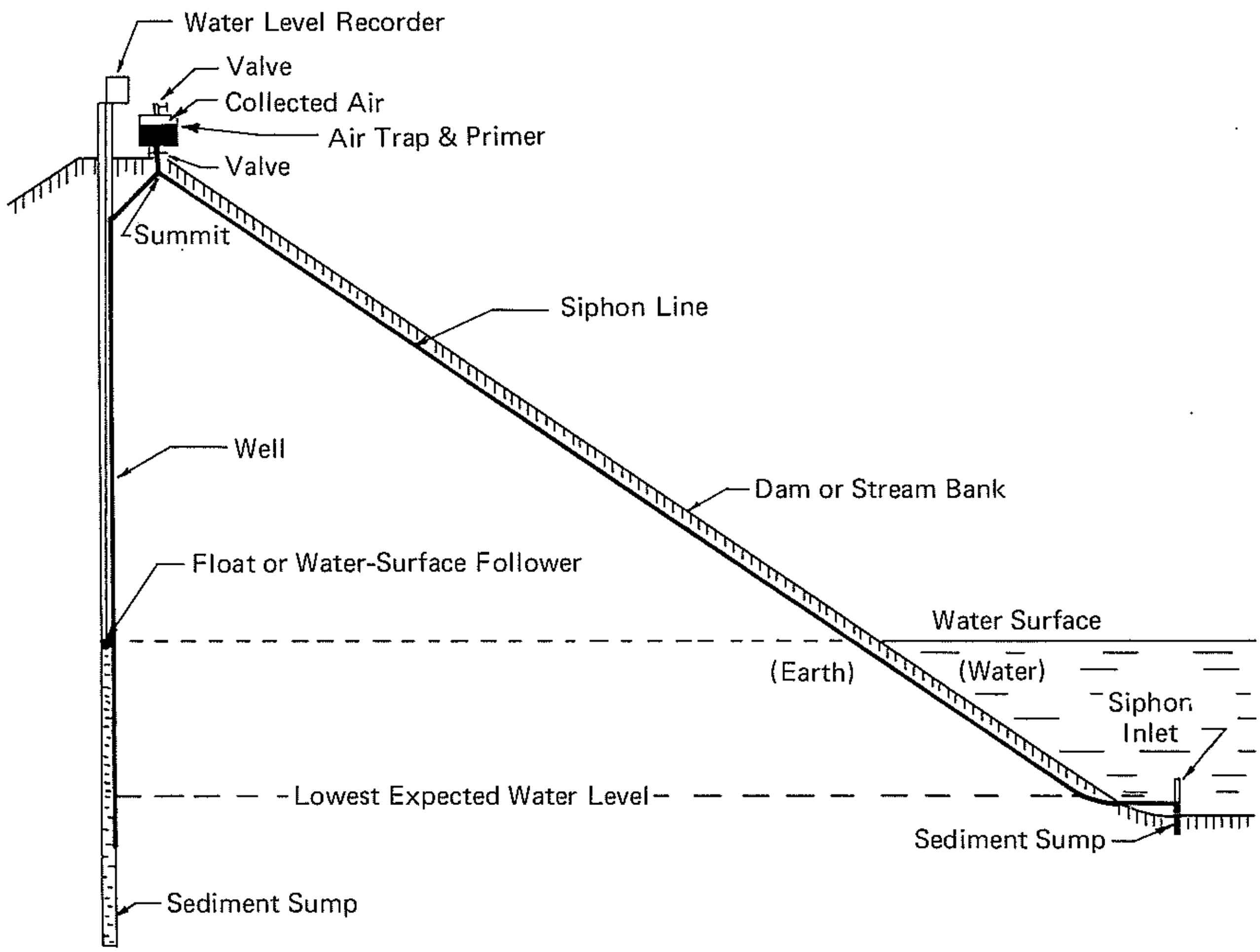


FIGURE 2.50.—Siphon system for measuring water levels.

opening being covered with a door hinged at the top and the lower third covered by a door hinged at the bottom to swing down horizontally for a small writing shelf. Any hasps or hinges should be placed so that they cannot be removed while the door is closed. A $^1/_8$- by $^3/_4$-inch (0.32 by 1.9 cm) strap iron with a small notch near one end should be hinged or pinned to each side of the door and should be run through a staple on each side of the door opening. Pulling the door out until the notch in the iron strap drops into the staple will hold the door in position while the observer checks the station.

The shelter house can be anchored to the well by bolting the floor at the four corners to the small angles welded to the top of the well. Another method is to bolt the framework to which the floor is nailed to the sides of the well at the four points of contact. The first method probably will be easier to assemble or dismantle. A hole with a 3-inch (7.6 cm.) diameter should be provided in the floor of the shelter directly below the clock so that gears, spindles, and clock spindle washers can be changed without removing the recorder. A shelter house that can be used with an 18-inch (45.7 cm) stilling well is shown in figure 2.51.

Large shelters.—A shelter that is 4 feet by 4 feet by 8 feet (1.2 by 1.2 by 2.4 m) is satisfactory for installation of a bubble gage. A shelter with walls fabricated from panels is shown in figure 2.52. A metal roof is desirable, but a plywood floor is adequate. This shelter contains adequate space for the bubble gage, strip chart recorder, and equipment for wading-discharge measurements. At locations where vandalism may occur, armor plate of rolled sheet metal $^1/_{10}$ inch (0.25 cm) thick may be desirable on the walls to protect expensive instrumentation.

Walkways, ladders, and observer's platform.—Where the well is short or set back in the bank, access to the shelter house can be provided by a walkway (fig. 2.53). Walkways should be at least 2 feet (61 cm) wide, should be above the maximum water stage expected, and should be entirely separate from the well or instrument shelter. The observer's platform and walkway should be approximately $3^1/_2$ feet (107 cm) below the shelter floor, have a minimum width equal to the width of the shelter, and extend far enough out from the well for the observer to open the door without getting off the platform. Ample railings should be provided where the walkway or platform is more than 3 feet (91 cm) above ground. Walkways, platforms, and railings should be constructed of good 2 by 4 and 2 by 6 lumber. Sufficient footings and bracing should be provided to give reasonable rigidity to the walkway and platform. Where the walkway is used with a concrete spillway, it can be anchored securely on one end to the spillway wingwall.

Where tall wells are used and set out from the bank, the access ladder and observer's platform should be anchored securely to the stilling well. The observer's platform and railings should be framed out of angle iron that is $1^1/_2$ by $1^1/_2$ by $1^1/_4$ inches (3.8 by 3.8 by 3.2 cm). The actual platform can be a section of subway grating.

Use of the walkways permits servicing most any time, whereas access ladders on a well can be used only during low water. This might result in the loss of valuable records.

Current-Meter Gaging Equipment:

Current meters for velocity measurement of open channel flow can be classified into vertical-axis and horizontal-axis meters. The vertical-axis or cup-type meter (fig. 2.54) generally is used in the United States for the following reasons:

- The meter will operate at low velocities, and its accuracy and consistency are equal to those of horizontal-axis meters in high velocities.
- The bearings operate in air pockets that largely eliminate entrance of sediment.
- Meter cups that become dented or slightly bent may be repaired in the field without seriously affecting the meter rating.
- The bucket wheel is relatively slow moving, and a single rotor serves for the entire range of velocities ordinarily found in stream gaging.

The horizontal-axis or propeller-type meter (fig. 2.55) is superior to the vertical-axis meter in measurement of flows containing large amounts of fine debris. Grass roots in the flow may immobilize the rotor of a cup-type meter or cause unreliable performance, whereas the

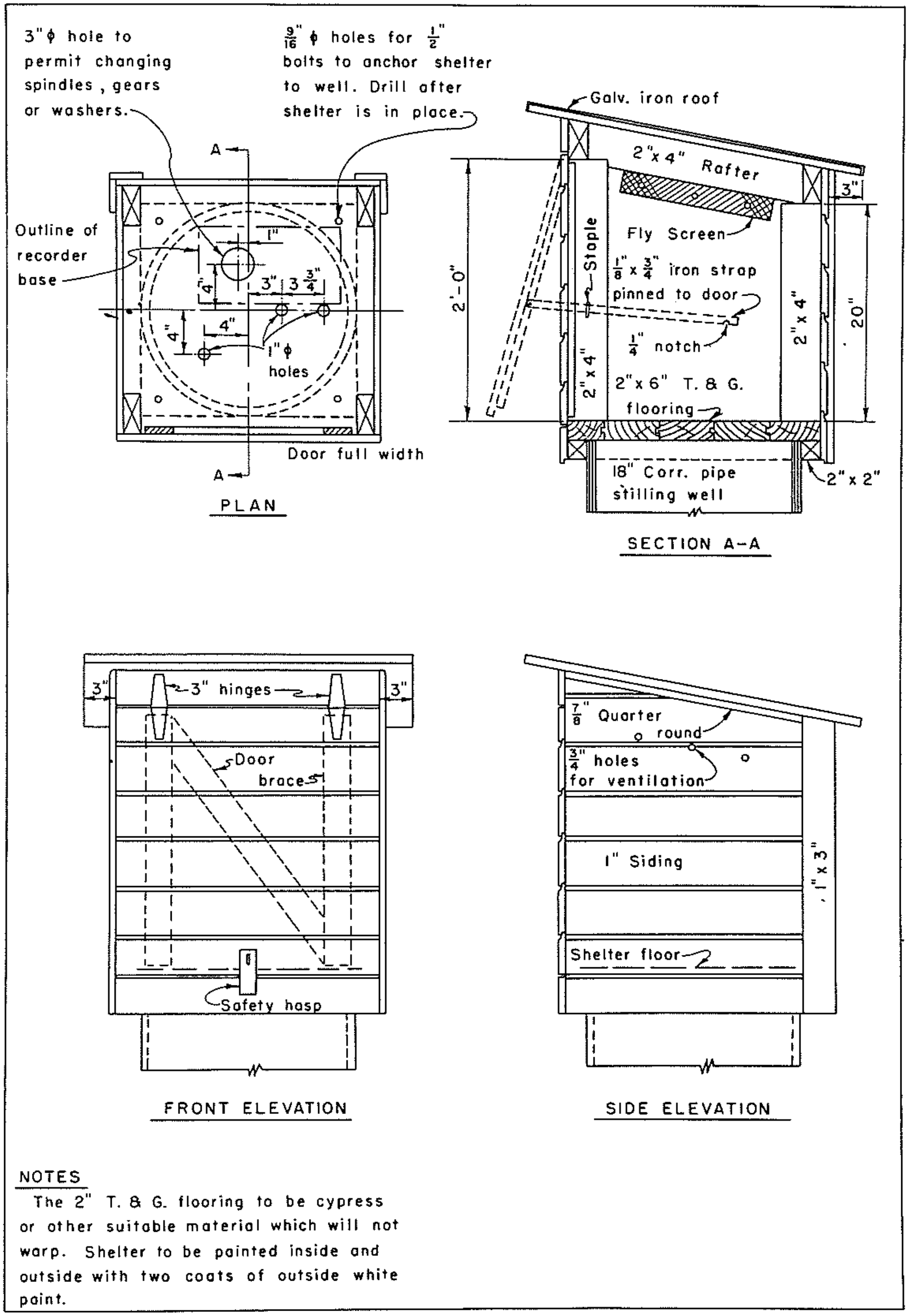
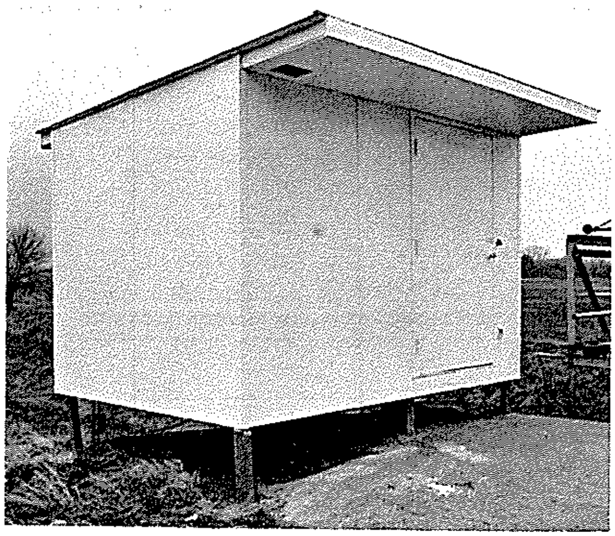


FIGURE 2.51.—Construction details of instrument shelter for 18-inch (45.7 cm) stilling well.



PN–5884

FIGURE 2.52.—Large shelter used for bubble gage installation.

same roots will not affect the performance of a horizontal-axis meter. Stream gagers generally agree that vertical-axis meters overregister and horizontal-axis meters underregister in turbulent flow.

Existing structures.—In deciding whether to use an existing bridge for stream gaging, consider the following:

- What are the hazards from traffic?
- Will great turbulence from piers or piling occur in the gaging section?
- Will piers or piling catch debris?
- Will low velocity or dead water occur in the gaging section?
- Must cable measurements be made through a bridge truss requiring frequent raising and lowering of the meter and weight?
- Does the gaging accuracy required justify the expense of a special bridge or cableway?

Traffic hazard sometimes can be solved by

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FIGURE 2.53.—Typical runoff measuring station.

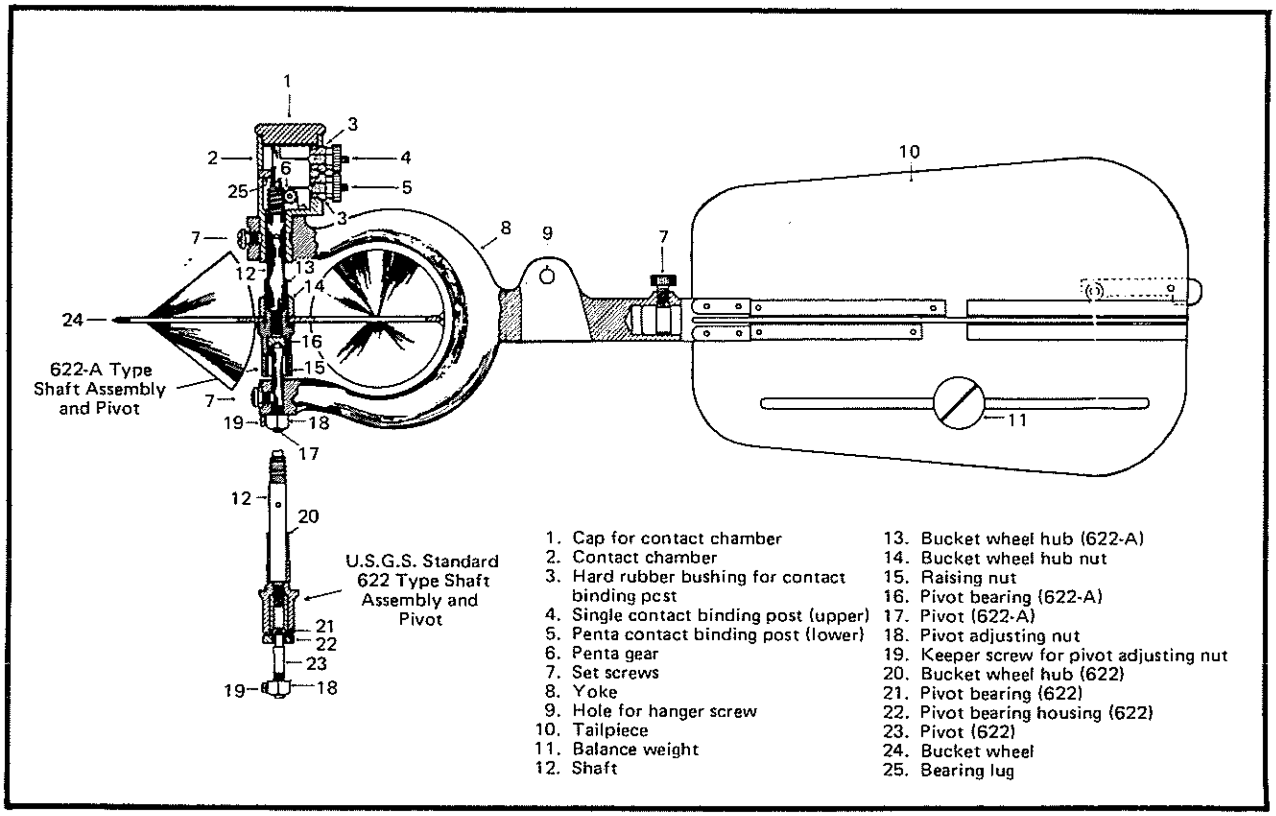

FIGURE 2.54.—Current meter and its parts.

constructing a catwalk on the outside of the bridge. This catwalk also makes the gaging section farther from turbulence around bridge piers. A suggested design is shown in figure 2.56. In designing gaging structures, the engineer is limited only by his imagination; strength of material; and, sometimes, availability of materials.

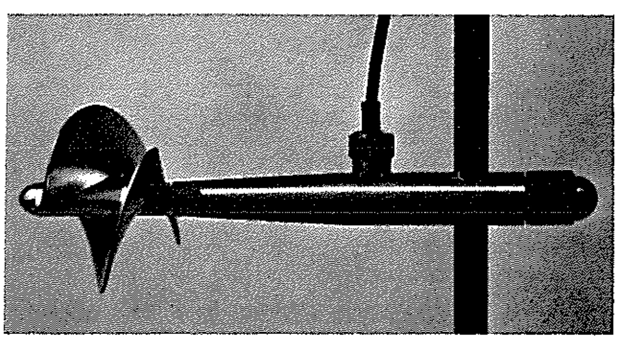

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FIGURE 2.55.—Propeller-type current meter.

Footbridges.—A clear span up to about 75 feet (23 m) is practical for a footbridge. Although more expensive than a cableway, a footbridge provides a convenient gaging structure from which more accurate measurements can be made. The length/width ratio for footbridges should be less than 12 to prevent swaying and possible failure. For long spans, trusses and deck support should be made of steel. A pressure-treated wood deck may be more desirable than steel grating because small items of gaging equipment may be lost by dropping through grating. Steel bridges spanning more than 40 feet (12.2 m) should have one end on rollers to provide free expansion and contraction. Pilings under abutments should extend at least a few feet below the streambed unless there is small chance of bank erosion or adequate protection against it around the abutments.

A steel footbridge with a clear span of 70 feet (21 m) is shown in figure 2.57. Truss connections can be made with gusset plates and bolts. Railings should be 36 to 44 inches (91.4 to 111.8 cm) above the deck.

Cableways.—Two types of cableways are used in measuring discharge. The passenger cableway has a standup or sitdown car in which the observer rides with the gaging equipment. The shore-operated cableway is handled from the shore with only the current meter assembly riding on the cable. The observer may stand or sit in an enclosure on the streambank.

Debris on the meter can be removed more quickly when using a gaging car rather than a shore-operated unit. For short spans and flow with little debris, however, the shore-operated cableway is less expensive and less hazardous to operate.

The design of passenger-type cableways is covered in reference (*32*). A modified cableway support is shown in figure 2.58.

An example of a shore-operated cableway, an elaborate gaging facility in Switzerland, is shown in figure 2.59. The head tower with the operating mechanism for the cableway and the tail tower on the opposite bank may be fixed installations, or the system can be portable with vehicles used as anchors on each side of the stream. A counter on the head tower reel positions the traveling block, and a counter on the reel raises and lowers the meter.

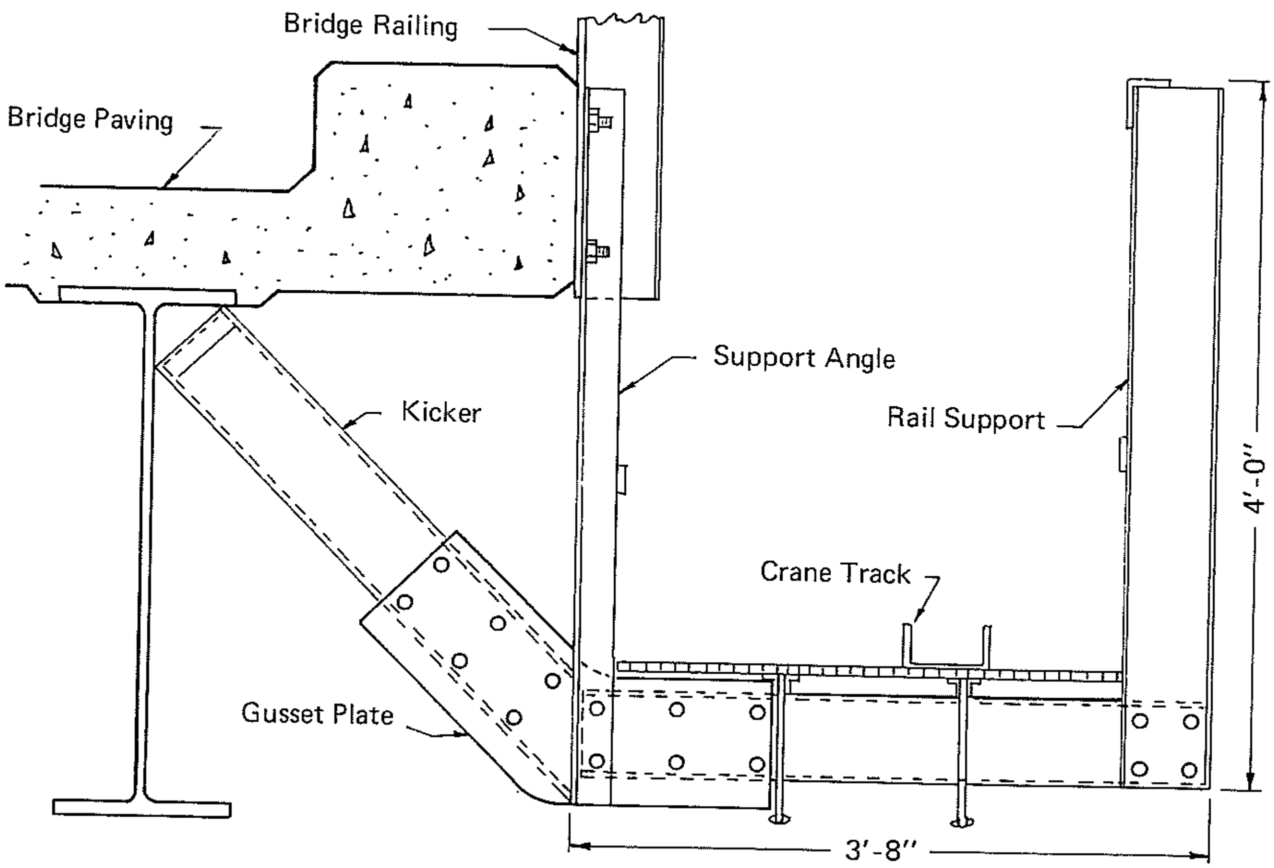

FIGURE 2.56.—Catwalk support.

PN–5887

FIGURE 2.57.—Footbridge.

FIELD OBSERVATIONS AND MAINTENANCE

Field observations are made (1) to record accurately and completely what is happening and (2) to promote proper notes and equipment operation in obtaining complete records of what will happen. Notes on instrument operation are vital to data tabulation, especially when appreciable lag occurs between obtaining the record and tabulating data. Notes on attending conditions are vital to data analysis and interpretation. Attending conditions not recorded on instruments must be noted personally. Records obtained with instruments not functioning properly may provide usable data if adequate notes are collected concerning the nature and cause of instrument failure. *Subject to specific instructions, an observer should adopt the philosophy that excess notes are merely excess material, whereas data not recorded are lost forever.*

Maintenance of equipment and runoff stations is mandatory for collecting good records. Improperly maintained equipment will not provide the accuracy which the equipment is capable of. Economy in equipment maintenance often can lead to excessive office work to explain what might have happened. Most equipment used in runoff observations is provided with recommended maintenance procedures from the manufacturer. Other suggestions for maintenance are provided in this section.

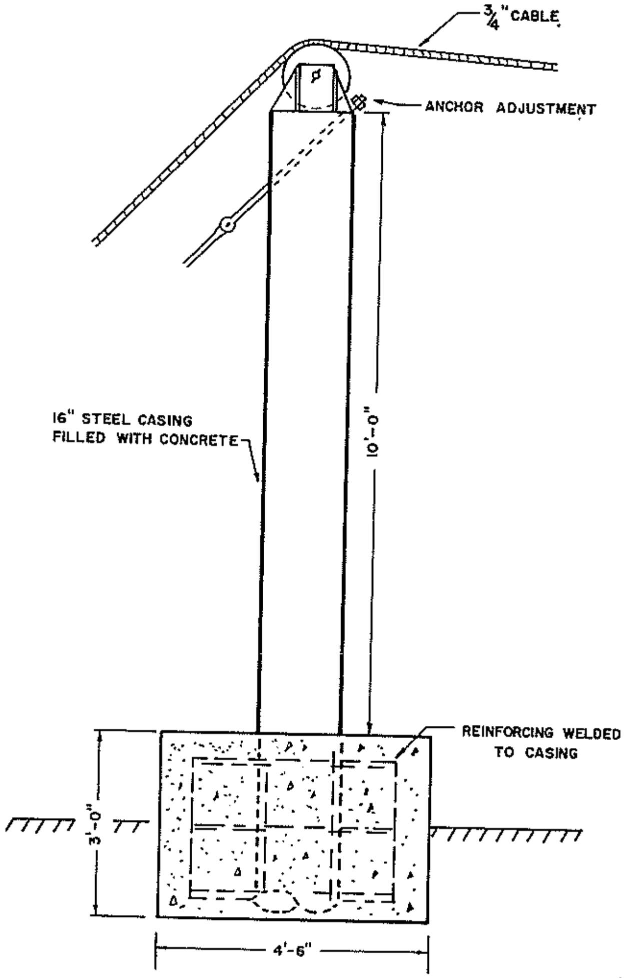

FIGURE 2.58.—Cableway support.

Instrument and Equipment Notation and Maintenance

Nonrecording gages

Safety is important when working near streams. Whether making observations or merely inspecting a runoff station, extreme dangers exist because of inclement weather such as lightning during thunderstorms, icy or wet conditions causing unstable footing, or rapid water in the stream due to high discharges. Observers must be aware of these problems and must be taught to operate as safely as possible (*4*).

Wire-Weight Gages:

Notation of wire-weight gage readings should be placed on recorder charts showing exact time read as well as stage reading. The observer should be identified by including his initials with the notes. If the reading is made in conjunction with discharge measurement, it should be shown with the recorder reading and time on the measurement note.

Staff Gages:

Nonrecording or staff gages are visited by the observer at regular intervals, their frequency being determined largely by the type of data desired and by accessibility of the gage.

The observer's notes for the nonrecording (staff) gage are shown in figure 2.60. General information at the top of these notes should include the name of the stream; distance and direction from the nearest city; exact location, section, town and range, mean sea level datum (m.s.l.) of the zero height, if available; month; and name of observer. The observer should visit the station at a regularly scheduled time, either morning or evening, and should note the date, time, stage, and other pertinent information under "Remarks." Additional observations on the rising and falling stages of floods should include the stage and approximate time of the high watermark (fig. 2.60).

The staff gage should be examined carefully after every major flow to ascertain physical damage requiring correction. Accumulated debris should be removed; and, in some instances, the face should be wiped clean to permit accurate reading. Periodic surveys are required to assure accuracy of gage datum.

Crest Gages (Maximum Stage Gages):

The crest gage normally is used to determine the highest stage during the year, but it can be serviced more frequently to give the maximum stage by storm, month, or season. Notes for several stations for any year can be shown on a single sheet, or a separate sheet can be used for each station to record the data for several years. Where several stations are shown on a single sheet (fig. 2.61) the heading should show only the year. Information on the location, drainage area, and type and size of control are entered in the office before going to the field.

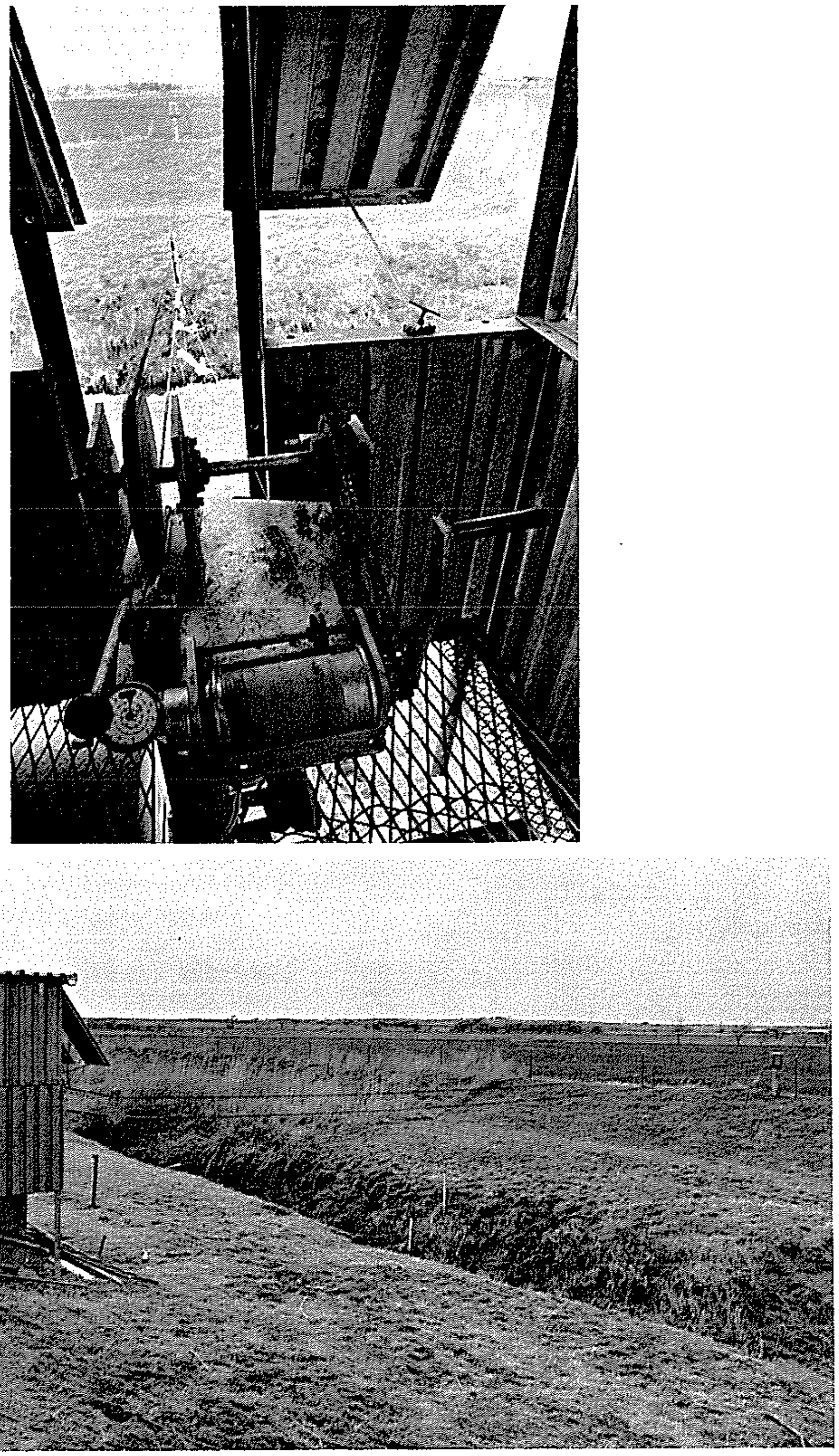

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FIGURE 2.59.—Interior (*top*) and exterior (*bottom*) of short-operated cableway.

Station: Fennimore Branch of Blue River
Location: 10 mi. NE of Fennimore, Wis. Sec. 36-T7N-R2W
M.S.L. Datum Zero of Gage = 797.0
Month: August, 1943 Observer: E. Johnson

Date	Time	Staff Gage Reading	Remarks	Date	Time	Staff Gage Reading	Remarks
1	7a	0.65		13	12 noon	2.05	
2	7a	0.67			3p	1.33	
3	7a	0.66			6p	1.12	
4	7a	0.65		14	7a	0.81	
5	7a	0.65		15	7a	0.74	
6	7a	0.65		16	7a	0.70	
7	7a	0.64		17	7a	0.69	
8	7a	0.64		18	7a	0.69	
9	7a	0.65		19	7a	0.68	
10	7a	0.64		20	7a	0.68	
11	7a	0.65		21	7a	0.68	
12	6a	1.72	Hard shower	22	7a	0.68	
	7a	4.53	about 5 a.m	23	7a	1.04	Rain during night
	8a	6.50	Maximum stage	24	7a	0.81	
	10a	4.25	6.65 (from high	25	7a	0.73	
	12 noon	2.73	water line on	26	7a	0.95	
	3p	1.64	gage) after 8 a.m	27	7a	0.77	
	6p	1.23		28	7a	0.71	
13	6a	1.50	Rain 2-6a	29	7a	0.69	
	7a	2.62	Maximum stage	30	7a	0.68	
	8a	3.73	3.90 before 8 a.m	31	7a	0.68	
	10a	3.02					

FIGURE 2.60.—Observer's notes of staff gage on Fennimore Branch of the Blue River.

Enter only the date of the observation, the maximum stage since the last inspection, and comments on condition of the station. When a separate sheet is used for each station (see fig. 2.62) the title should show the location, type and size of control, and drainage area in acres or square miles. Several observations for each year, shown on figure 2.62, are convenient where it is desirable to know whether the maximum annual peaks are the result of melting snow or summer thunderstorms. Steps in servicing the maximum-stage gages are:

- Upon arrival at the station, open the lock, lift the cover, and remove the stick.
- Note and record in the fieldbook (fig. 2.62) the date and maximum stage as shown by the line of powdered cork adhering to the stick.
- Wipe the powdered cork from the stick.
- Check to be sure that the holes in the bottom cap are open.
- Remove any accumulation of silt from the inside of the pipe. (Removal of the bottom cap may be necessary.)
- Add powdered cork to the metal cup, replace the stick, and lock the cover.
- Under "Remarks," note items that might affect the accuracy of record, such as excessive accumulation of silt.

If installed properly, maximum stage gages require relatively little maintenance. Visual inspection is necessary during routine readings to determine if physical damage has occurred from flow events or vandalism. Periodic surveys should be made to verify correct gage datums.

Maximum Stage Gage Records for Wisconsin (1957)

County	Location Sec-Tn-Rg	Dr. Area Acres	Control Type	Structure Size	Max. Stage Feet	Date of Obs.	Remarks
Buffalo	23-22N-13W	1120	drop inlet	twin 5x5	1.07	Nov.13	silt above inlet
	30-23N-13W	783	box inlet	~~8x12x19~~	1.56	" 13	silt level with crest
	8-23N-13W	2480	drop inlet	twin 6x6	2.13	" 13	silt gradient 1%
Iowa	34- 8N- 2E	350	box inlet		0.76	Nov.12	large storage below inlet
	34- 8N- 2E	485	drop inlet	twin 5x5	0.78	" 12	
LaCrosse	24-18N- 7W	534	drop inlet	twin 4x4	1.11	Nov.14	
	24-17N- 5W	585	box inlet	7.5x10.5x22	0.49	" 14	
	28-17N- 6W	221	flume	3.5 x 18	0.31	" 14	
	21-17N- 5W	11000	ogee spillway	6.5 x 60	2.65	" 14	old mill dam
Trempealeau	31-20N- 7W	690	drop inlet	~~twin 4x4~~	3.50	Nov.13	almost silted full
	1-19N-10W	1170	drop inlet	twin 5x5	2.20	" 13	silted above inlet
	34-20N-10W	385	drop inlet	twin 4x4	3.04	" 13	" " "
	6-19N- 9W	1530	box inlet	10x15x31	2.36	" 13	" " "

FIGURE 2.61.—Observer's notes on several maximum stage gages in Wisconsin.

Recorder Chart Notation and Maintenance

All recorders should be maintained according to the manufacturer's instructions. The observer should familiarize himself with instruction manuals provided by the manufacturer and should follow recommended maintenance and servicing procedures.

Drum Chart Recorders:

Placement of notes on charts will be simplified greatly by the use of rubber stamps (fig. 2.63). A series of stamps for runoff stations might include station designation; time zone (eastern, central, mountain, or Pacific, and standard or daylight saving); chart number; placement, inspection, and removal watch times (W.T.), and index pointer (I.P.); chart line and outside gage (O.G.); rising; falling; corrections—time and stage; notch (G.D.E.); and G.D.E. lowest intake. Charts should be numbered and dated to show that the record is continuous although no runoff occurred during the period covered by some of the charts. Use a 3.H pencil for all notes and proceed as follows:

For those charts covering periods during which no runoff occurred, show only chart number and dates in the upper righthand corner. No other notes are required as the main purpose of these charts is to show continuity of records.

For charts covering periods during which runoff occurred (fig. 2.63):

- Include the chart number and dates in the upper right-hand corner.

Maximum Stage Gage Record on Twin 4 x 4 Drop Inlet
in Sec. 34, T20N, R10W, Trempealeau County, Wisconsin
Drainage Area = 385 Acres (Steep-Fan shaped)

Date	Max. Stage Feet	Est. q c.f.s.	Remarks
Apr 15, 1949	0.43		Spring peak from melting snow
June 30,	0		
Aug. 2,	4.65	950	Flood of July 27, heavy damage to roads and
Nov. 15,	0		bridges from this storm
Apr. 15, 1950	0.50		
June 30,	1.33		
Nov. 15,	0.25		
Apr. 15,	0.65		
June 30,	0		
Nov. 15	1.23		

FIGURE 2.62.—Observer's notes for a season on a maximum stage gage in Wisconsin.

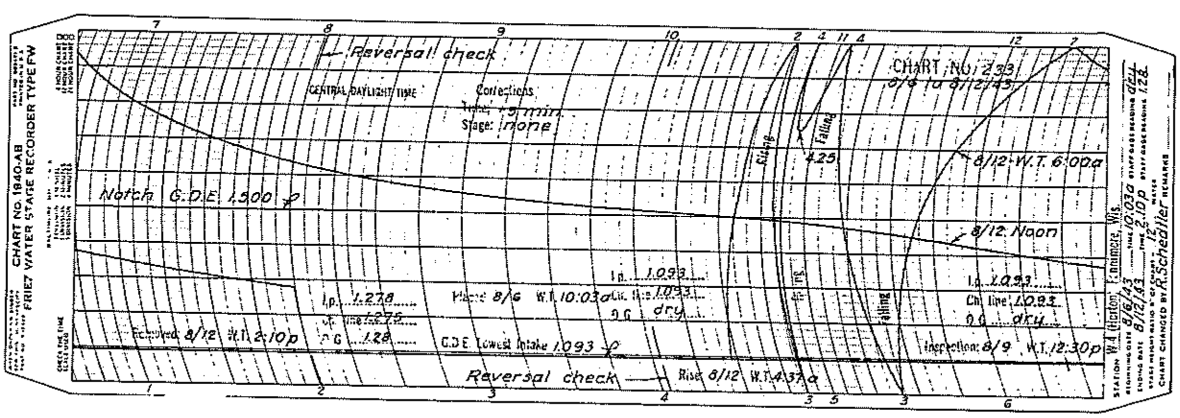


FIGURE 2.63.—Annotations of a stage chart for Fennimore, Wis.

- Show the time zone (eastern, central, mountain, or Pacific, and standard or daylight savings).
- Enter dates, watch time, index pointer, chart line, and outside gage reading at times of placement, inspection, and removal.
- Show the G.D.E. of the spillway crest and the lowest intake.
- Indicate the time scale and gage-height ratio used in the proper place on the marginal tabs.
- Check to see how placement and removal marks agree with the watch time. If they do not agree within 2 minutes, apply a time correction. To determine this correction, assume a straight line variation between placement or inspection and removal. For example, the chart time of figure 2.63 was correct at the time of placement, was 5 minutes slow at inspection, and was 10 minutes slow at removal. The total elapsed time between the last inspection and removal was 74 hours. The time from this inspection to the beginning of runoff was 64 hours. The correction to be applied would be $5 + \frac{64 \times 5}{74} = 9.3$ minutes. Since time data are tabulated only to the nearest minute, a time correction of plus 9 minutes would be applied.
- Write the correct times of rise, noons, midnights subsequent to the rise, and any other points necessary to simplify the compilation of records. (In locating these times, begin at the removal time and work backward.)
- Check for discrepancies between the chart line and index pointer and for failure of the pen to reverse at the edges of the printed portion of the chart. If the pen reverses below the limits of the engraved part of the chart about the same extent at both the upper and lower reversals, apply a constant correction to each traverse. This correction for the traverse upward across the chart is plus, whereas that for the downward traverse is minus. Where the lower reversal is correct and the upper reversal falls short, a graduated correction is correct theoretically. As tabulations are to be made only to the nearest 0.01 foot (0.3 cm), a graduated correction would not be feasible and a constant correction should be applied through a given range in stage. If the upper reversal falls 0.01 foot short, the entire correction should be applied only to the upper half of the chart. If the upper reversal is 0.02 foot (0.6 cm) short, a correction of 0.01 would be applied from 0.25 to 0.75, and 0.02 would be applied from 0.75 to 1.25. Note any correction that must be applied to the stage.
- Show the corrected stage values at all peaks, troughs, and reversals of the actual record. (For weirs, the number of reversals will be even at each edge of the chart provided that where the pen travels once across the chart for each foot change in stage, the stage at removal is in the same foot range as when the runoff began.
- Add notes to show when the stage is rising or falling.

Strip Chart Recorders:

A step-by-step procedure should be established and followed during each visit to a gaging station equipped with a strip chart recorder. Routine inspections, made weekly, require the following operations and chart notations:

- Identify sation on chart.
- Mark the chart by grasping the float pul·ey and rotating it in the direction required to raise the float a short distance. This causes the pen to make a short line normal to the time trace. Do not rotate the float pulley in the opposite direction (opposite to raising the float), which causes slack because the float tape may kink or slip around the float pulley wheel.
- Note the date and time directly on the chart. Use military time, preferably, or designate a.m. or p.m.
- Record gage heights as indicated by check gages, using initials to indicate type of gage (O.G., outside gage; I.G., inside gage; W.W.G. wire-weight gage; S.G., staff gage).
- Initial chart.
- Note pertinent information about the station or equipment condition on chart (repairs needed, repairs made, trash removed, intakes clogged, intakes cleared, clock stopped).
- Wind clock.
- Make sure pen is against the chart and an ample ink supply is in reservoir.
- Make sure adequate chart remains on the supply roll.

- After completing other work (such as making discharge measurements), inspect the instrument to make sure the procedure of winding does not cause the clock to stop or the paper to shift from its correct position.

The first six items will be repeated for every visit to the site if other operations, such as discharge measurements, sediment sampling, or repairs, are performed.

If necessary, replace the chart soon after the observer reaches the station so that the instrument operations can be checked after the work has been completed and the new chart has been running for some time. If trouble has developed, it can be corrected without loss of record.

Clocks and mechanisms are to be oiled on a regular schedule, usually annually or semiannually. Use lubricants specified by the manufacturer. Gaging station inspection sheet and chart removal instructions (for Leupold & Stevens, Model A–35 recorder) should be posted in recorder shelters. Figures 2.64 and 2.65 are examples of instruction sheets used in the field.

Digital Punch Recorders:

Follow the strip chart recorder routine when servicing digital punch recorders, omitting those steps not applicable. Since little space is available for writing on punched tape, a notebook should be placed in every recorder shelter. Comments other than station identification, date, time, gage height, and observer's initials will be recorded in this notebook. A satisfactory field checklist used by technicians is shown on figure 2.66.

Equipment Servicing

Equipment should be serviced on a specified schedule set up to maintain continuous operation. Timing will depend on the equipment used and the clock mechanism.

Flumes and Weirs:

The structure and upstream pond area must be kept free of weeds and trash, and sediment must be removed as it accumulates. The approach area and pool banks should be trimmed to maintain a clean approach channel.

The level of the crest should be checked periodically with reference to the elevation of the gage zero. The structure should be inspected for leakage. Leaks must be repaired, and the structure must be rechecked carefully to see that it is level and at the correct elevation.

The crest should be examined for nicks or dents that might reduce measurement accuracy. These imperfections should be corrected carefully. Imperfections that would change the shape of the weir opening should not be removed.

Flumes must be maintained in original condition and in accordance with design specifications. Levels and slopes of installation must be checked periodically and must be corrected if necessary.

Gage Wells:

If constructed properly, gage wells will require little servicing. The well, intake pipes,

GAGING STATION INSPECTIONS

Weekly

Routine inspection

1. Mark chart.
2. Note date.
3. Number tape (I.G.)
4. Note time (A—a.m., P—p.m.)
5. Initial.
6. Wind clock.
7. Put pen down.
8. Check ink supply in pen.

Monthly

Routine inspection plus station identification

1. Remove one month's chart record.

Quarterly

Routine inspection plus station identification

1. Remove chart.
2. Install new chart roll.

Yearly

Oil clock and mechanism (during April).

Cleanup Inspections

Routine inspection plus:

1. Check well for sandfill.
2. Check bar reading. Record on chart.
3. Wire weight or staff gage reading. Record on chart.
4. Replace sample bottles in storage box.

FIGURE 2.64.—Gaging station inspection sheet (Model A-35 recorder).

and silt trap (if used) should be free of silt. When the well or trap is cleared or the intake pipe is unclogged, the recorder stylus should be raised from the chart. Otherwise, surge in the well may cause excess ink on the chart to soften the paper, causing the pen to tear it. The gage should be read before and after cleaning, and gage heights and notes should be recorded on the chart.

After flow events, wells must be checked for debris accumulation that needs removal. In some channels, shifting sand may have isolated intakes, necessitating removal of a quantity of sand.

PROCEDURE FOR REMOVING STRIP CHART

1. Routine inspection.
2. Unlock screws on tape pulley.
3. Make pen reversal at both sides of chart.
4. Lift pen.
5. Pull lettered disk to right and turn counterclockwise to a stop.
6. Run chart forward till graph is beyond edge of writing plate.
7. Cut chart using writing plate as a guide.
8. Lift takeup roller out of bearings.
9. Remove flanged shaft and half tube clamp.
10. Replace takeup roller.

Procedure for replacing chart

1. Remove writing plate.
2. Lift out supply roll.
3. Remove knurled washer nut and remove old core.
4. Place new chart on supply cylinder—flush end toward flange.
5. Replace knurled nut.
6. Place supply roll on bearings with flange to the left.
7. Crease chart about $^3/_4$ inch from edge.
8. Feed chart between chart drum and friction roller.
9. Replace writing table.

(Start here monthly.)

10. Pull chart 1 inch beyond takeup.
11. Butt up left edge of chart square to flange on takeup roller.
12. Place half tube clamp over chart and takeup roller.
13. Turn takeup roller at least one turn—use knife to keep edges from doubling back under first turn of chart.
14. Make pen reversal—adjust pen if necessary.
15. *Lock screws on tape pulley.*
16. Mark point on chart where pen is to be set.
17. Set chart and pen to point.
18. Turn and push disk to left to restore friction drive.
19. Make routine inspection plus station identification.

FIGURE 2.65.—Chart removal instructions (Model A-35 recorder).

Bubble Gage System:

Preventive maintenance should be performed on the bubble gage as follows:

- Check tolerance between the jeweled bearings and float mast pivot. If the float does not operate properly, the pivot and bearings should be examined under a magnifying glass and should be replaced if worn or damaged. Avoid overtightening the bearings, which will damage the bearings and pivot.
- Remove corrosion accumulating on the contacts and float mast because it creates large steps in the stage record. Corrosion can be removed with crocus cloth or a small knife blade.
- Use a control with a time delay circuit because it will provide a better stage record and will prolong battery life. The time delay should be checked regularly, however, because resistors may need to be replaced.
- Wash the tubing from manometer to sand point or orifice with clean water followed with alcohol and then dried with a release of nitrogen once a year or as needed (*20*). A garden sprayer or a fire extinguisher can be adapted for this use.
- After each major flow event or a series of small events, remove the top (vent assembly) from the sand point and check the water depth in the sand point. Sufficient space within the sand point will prevent sand from covering the orifice during a major rise. Sand can be removed from the sand point with a portable pump and enough $1^1/_2$-inch (3.8 cm) hose to reach the bottom of the sand point.
- When cleaning the sand point, check the manometer and tubing for high pressure leaks. Plug the orifice and allow nitrogen to slowly increase the pressure in the system to an amount greater than would be expected during any flow event. Shut off the nitrogen supply. If the indicated stage begins to decrease, a leak is in the system. Leaks can be located by applying soapy water or children's bubble blowing solution at suspected leaky joints in the gas purge system.
- Replace the batteries when the motor speed becomes sluggish.

Instrument No. ____________________ Station No. ____________________

Date to be serviced					
Date actually serviced					
Synchronization					
Correct E.S.T.					
Tape time					
Time correction					
Tape Changed					
Off time (plus or minus)					
On time					
Tape Supply					
New tape installed					
Days remaining on tape					
Instrument Reading					
Punchout in line					
Punchout clean					
Dial reading					
Tape reading					
Float tape reading					
Staff gage reading					
Zero adjusted to T.B.M.					
Battery Supply					
Voltage					
Battery replaced					
Added 1½ volt battery					
Operation					
Contacts cleaned					
Gage movement free					
Intake flushed					
Debris in well					
Visible damage					
Repairs made					
V-notch clear					
Mechanical Malfunctions					
Broken leaf switch					
Defective timer					
Bent or broken punches					
Tape slipping					
Tape binding					
Others					
1.					
2.					
3.					
4.					
Gage Serviced By					

FIGURE 2.66.—Field checklist for digital punch recorder.

A suggested form for recording bubble gage maintenance is shown in figure 2.67.

Siphon Intake System:

One advantage of the siphon intake system for measuring water levels is its relatively trouble-free performance, which requires only a semiskilled observer. The air trap must be evacuated periodically, however. This trap also must be protected from freezing by housing, wrapping, burying, or heating. An alternative solution is filling the air trap with antifreeze. The antifreeze should have the same density as water, however, to avoid unbalancing the siphon. If the siphon inlet becomes covered with sediment, the inlet must be moved up or the sediment must be removed. The 4-inch (10.2 cm) pipe sand point previously described makes an excellent attachment to the siphon inlet. Sediment can be flushed from the sand point with a portable pump.

Stage Recorders

Several water level recorders are available from weather instrument manufacturers. Many recorders operate on the principle of a reversing mechanism for stage that permits the recording of an unlimited range in stage on a scale, which can be read accurately to the nearest 1/100 foot.

Several horizontal drum recorders are available, some with spring-wound clocks and other with weight-driven clocks. Most drum recorders have several time and stage-scale ratios available. All record on a selected time scale, others repeat, and still others record on a continuous roll. Those recording on a continuous roll of graph paper are well adapted for use in remote locations or places that are not readily accessible under all weather conditions. They will run as long as 60 days between windings, depending somewhat on the time scale used.

The FW–1 and FW–2 recorders are the simplest. Both have a spring-driven clock supported on a vertical spindle, and they can be equipped with several gears to make one revolution each 6, 12, 24, 48, 96, or 192 hours. The smallest time division for a 6-hour chart can be either 1 or 5 minutes, and for the 192-hour chart, 2 hours. These clocks will run 8 days on one winding regardless of the time scale used. Both recorders (FW–1 and FW–2) have reversing mechanisms and can record an unlimited range in stage on a scale of 5 inches (13 cm) of chart equal to 12 inches (30.5 cm) of stage. If greater refinement is required, as might be true for detention reservoirs, a small float wheel and special tape are available for older models and special gears are available for newer models. Each special equipment device will record the stage on a scale of 10 inches (25.4 cm) of chart to 12 inches (30.5 cm) of stage.

The chief difference between the FW–1 and FW–2 recorders is that the former uses a curvilinear chart and the latter uses a rectilinear chart. The time scale selected should vary with the anticipated maximum rate of change in stage, and the scale must be open enough to give the desired accuracy to the runoff computations. Thus, if accurate discharge computations require a change in stage not to exceed 0.2 foot (6.1 cm) between successive time intervals, the chart scale should be such that the maximum rate of rise during the minimum readable time interval will not exceed this amount. A rough guide is to use a 6-hour chart for areas up to 300 acres (121.4 ha) a 12-hour chart for areas from 300 to 2,500 acres (121.4 to 1,012 ha), and a 24-hour chart for larger areas. It may be advantageous to use the time scale on the water level recorder for the recording rain gages. Where several subwatersheds are being gaged, the addition of one 192-hour recorder will reduce the work of figuring faster time-scale records.

Regardless of the type of water level recorder selected, it should be equipped with a graduated float tape that passes over the float wheel and has the ends attached by ring connectors to the float and to a counterweight. If no tape index pointer is furnished as part of the instrument, one should be attached either to the instrument case or to the floor of the shelter house. The graduated tape and index pointer enable the observer to check the pen reading against the water level in the well and that shown on the outside staff gage. Failure of these to show reasonable agreement indicates errors and need for adjustment in the station setting.

Bubble Gage Log

rr —remove record
B—change batteries
T—change tanks
R—new supply roll
D—discharge measurement

Station:

Date	Time		Gage height		Pressure Psi	Bubbles per min	Party	Remarks
	Watch	Pen	Outside counter	pen	Cylinder feed			

FIGURE 2.67.—Bubble gage maintenance form.

Calibration of FW–1 Recorder Before Installation:

The instructions given here are for the FW–1 recorder. They can be adapted to other types of installation also.

FW–1 recorders (fig. 2.68) are calibrated at the factory, and, when in proper adjustment, the following relationships exist:

- The total pen travel between upper and lower reversals is about 5.00 inches (12.7 cm), and both reversals occur at or near the edge of the printed portion of the chart.
- Cam follower pin *C* is horizontal.
- The pen arm is parallel to the cam follower arm.
- The pen arm is tilted inward just enough so the pen will not fall away from the chart near the upper reversal. (Note: A slight change in tilt of the pen arm will cause a shift in position of the reversals).
- The clock spindle washer is of the proper thickness to place the centerline of the chart and the pen shaft in the same horizontal plane.

When establishing a new or exchange recorder, its calibration should be checked in the office or laboratory. Periodic checks or calibrations of established instruments are made more often in the field. The procedure for calibration in the office or laboratory is:

- Purchase a calibration wheel from an instrument manufacturer or prepare one similar to that shown in figure 2.69, using 24-gage sheet metal.
- Cut out a circular piece about 15 inches (38.1 cm) in diameter, and from the center of this, strike off a circle using a radius of 7.2 inches (18.3 cm).
- Set dividers at 4.45 inches (10.3 cm) and mark off 10 spaces along the arc of this circle. These 10 points should be equidistant; if not, reset the dividers to obtain 10 equal spaces.
- Punch or drill 3/32-inch (0.24 cm) holes at each of the 10 points.
- Drill an 11/32-inch (0.08 cm) hole in the center.
- Cut away most of the material to leave a center hub, six to eight spokes, and the outside ring.
- Place the recorder on a table or stand so that the float wheel just clears the edge.
- Loosen screws *E* and remove the knurled brass disk and the float wheel.
- Place the calibration wheel on the shaft, replace the float wheel, replace and tighten the knurled brass disk, and tighten screws *E*.
- Mark a line on the edge of the table or stand, directly behind the outside rim of the calibration wheel, for use as a reference point. The edge of the table top or stand also can be used for this purpose.
- Make sure the gear on the clock matches that on the recorder spindle.
- Fill in the data on the end tabs of a 1940 AB chart with india ink. Draw a horizontal line, about 2 inches (5.08 cm) long, near the right end of the chart, 5.05 inches (12.8 cm) above the lowest graduation. This represents the expanded width of the chart under conditions of high humidity (fig. 2.70).
- Wrap the chart around the clock cylinder; be sure that the chart fits snugly and smoothly and rests on the bottom flange throughout its circumference. (Note: In using the AB chart, which has a margin at both ends, the right margin is folded under, then placed over the left end with the chart clip running down through the fold.)
- Place the clock in the recorder by grasping it with the winding key and lowering it gently over the spindle until the gears are meshed. Be sure the clock rotates freely and does not bind at any point. Binding may indicate improper position of the clock mechanism with respect to the spindle, or it may indicate a bent spindle. To correct binding, remove the clock, loosen the three screws on the base, shift the gear away from the center, and tighten the screws. Further binding probably is caused by a bent spindle, which should be straightened or replaced.
- Rotate the float wheel to see that the pen arm is tilted inward just enough to prevent its falling away from the chart near the upper reversal.
- Place a small quantity of ink in the pen and check to see that the point gives a clear fine line. Check adjustment of the recorder as follows (for designated parts, see fig. 2.69).
- Note that the pen arm and the cam follower arm, which holds the counterweight *M* and *C* (fig. 2.68), are parallel.

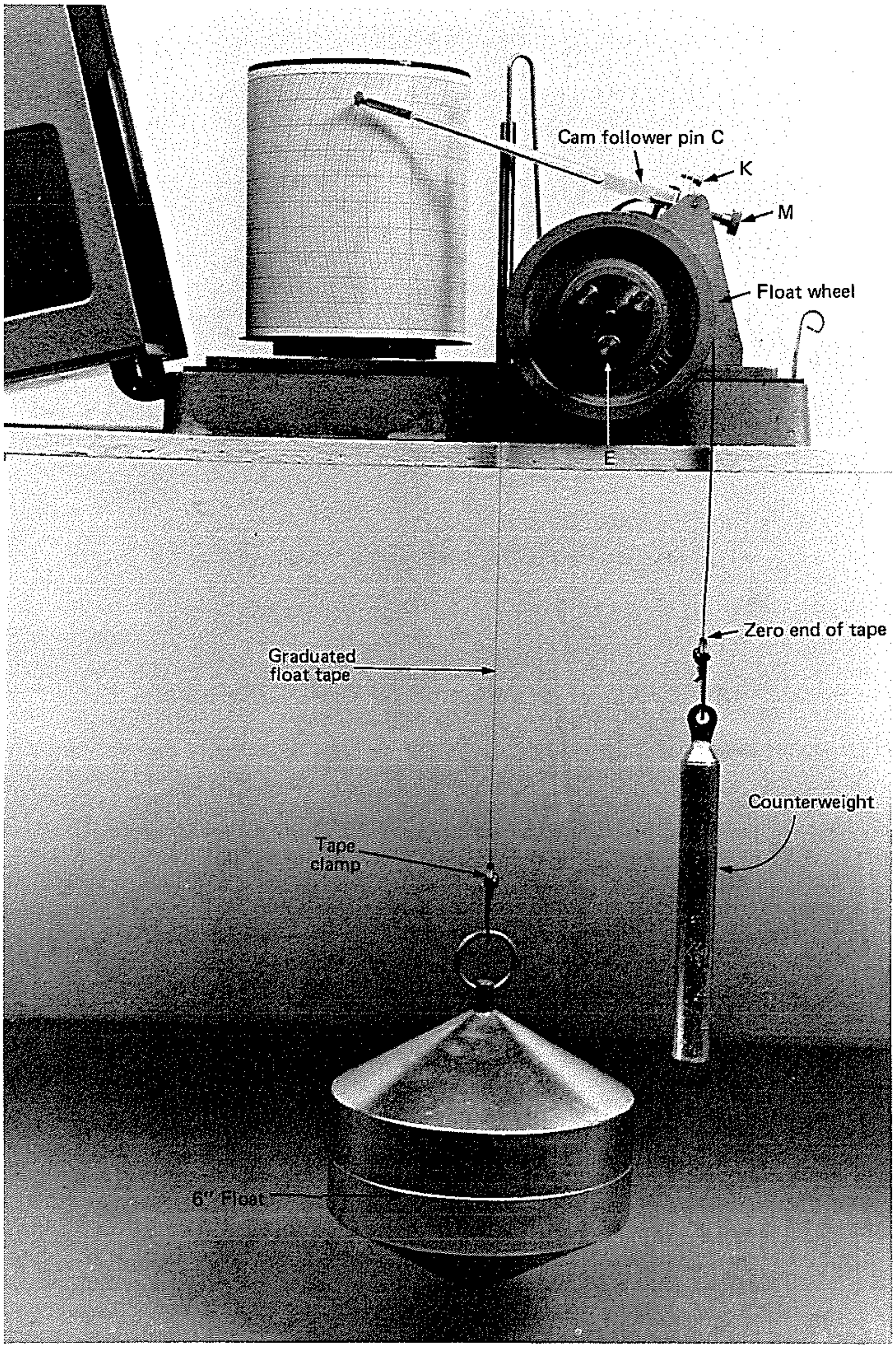


FIGURE 2.68.—Water level recorder with float and counterweight.

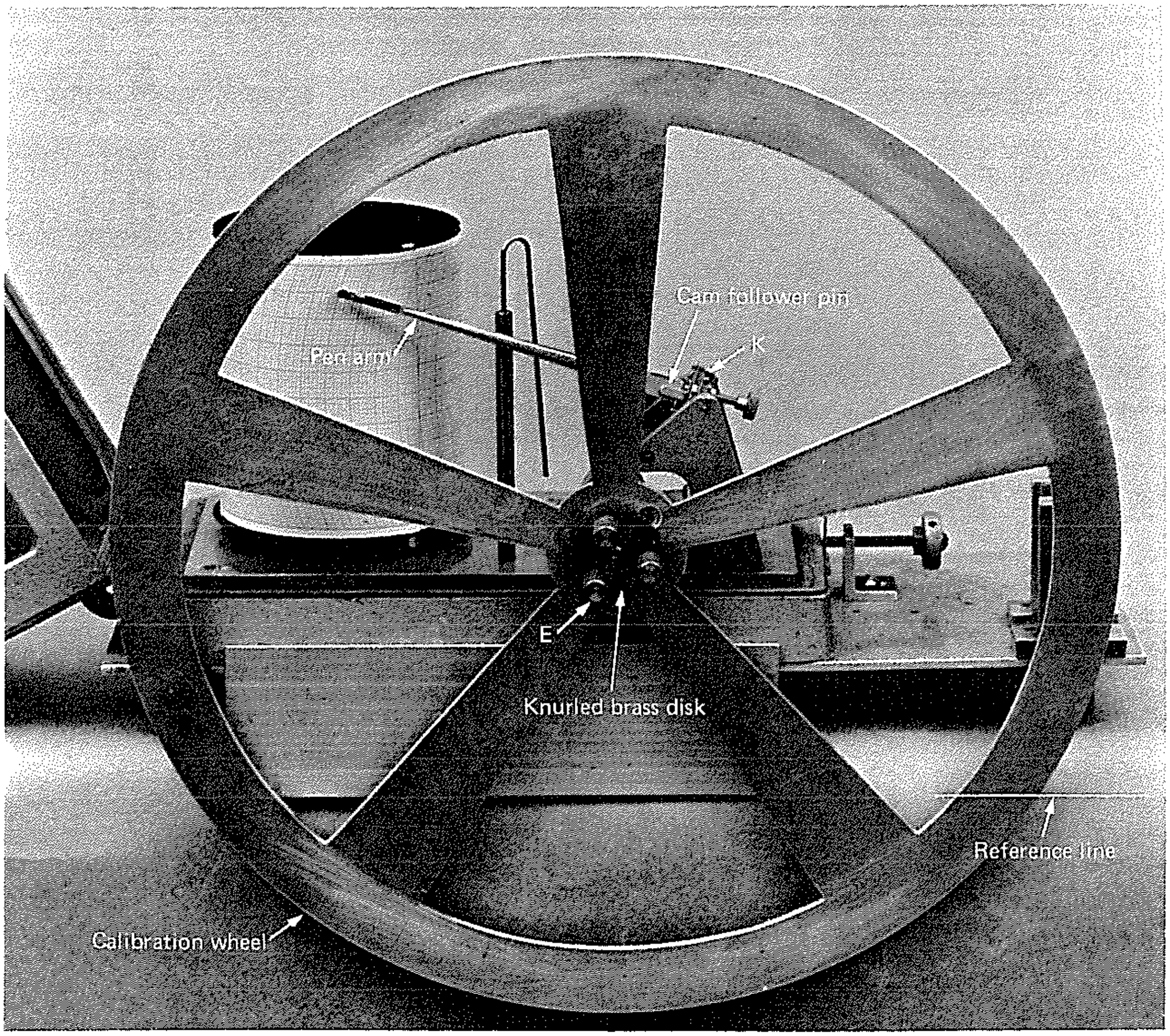


FIGURE 2.69.—Water level recorder with calibration wheel.

- See that the cam follower pin *C* is horizontal by sighting over the shaft that carries the pen arm and the cam follower arm.
- To see how close the pen follows the time line, place the pen on the chart, and rotate the float wheel through the upper and lower reversals. Deviation from the time line between the upper and lower reversals indicates either a bent spindle or improper thickness of clock spindle washer. Remove the clock, rotate the spindle 90°, and repeat the check. Failure of these pen traces to agree for positions of the spindle indicates that the spindle is bent and should be straightened or replaced before continuing with the check. If the arc described by the pen is the same for two or more positions of the spindle but still does not follow a time line, it indicates improper thickness of washer between the base and the gear. When the arc falls to the right of the time line at the top of

the chart (fig. 2.70), the clock spindle washer is too thick. When the arc falls to the left, the washer is too thin. Change the washer as indicated and check until agreement with the time line is good. The proper thickness of the washer for some older recorders is about 0.140 inch (0.36 cm).

• Rotate the float wheel to bring the pen within a half inch (1.3 cm) of the lower edge of the chart. Place the pen on the chart and continue rotating the float wheel until the pen reverses. If this lower reversal occurs above or below the lowest graduation, bring pen to this line by adjusting screw *K*. (The function of screw *K* is to make the cam follower arm parallel to the pen arm.) Remove the pen from the chart.

• Hold the calibration wheel so that one of the 3/32-inch (0.24 cm) holes is on the reference line. Loosen screws *E* and turn the knurled brass disk until the pen is at the lower reversal. Tighten screws *E*. Place the pen on the chart and rotate the clock to mark a line about 1/4 inch (0.6 cm) long.

• Check for ascending and descending symmetry: Rotate the calibration wheel in a clockwise direction and, while holding each hole at the reference line, rotate the clock to make short lines showing the actual chart reading

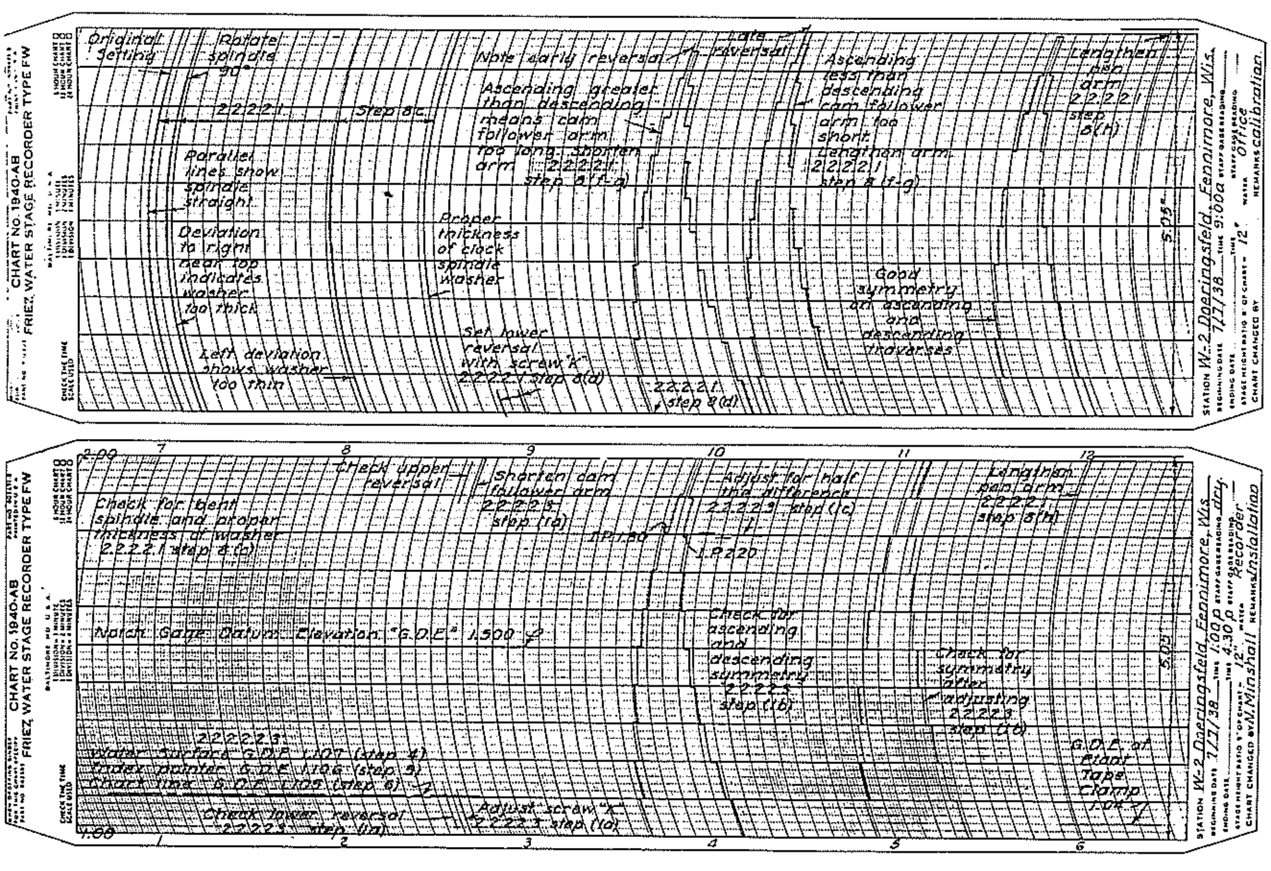


FIGURE 2.70.—AB calibration charts for FW-1 recorder.

for each 0.1 foot (2.5 cm) until the pen has made one complete traverse up and down the chart width.

- If these short lines on the ascending traverse are higher than those on the descending traverse and the reversal occurs too early, shorten the cam follower arm. This is done by loosening the setscrew and the nut on the side toward the cam and tightening the nut on the counterweight side. If lines on the ascending traverse are lower than those on the descending traverse, lengthen the cam follower arm. After each such adjustment, check to make sure the cam follower pin C is horizontal.
- Repeat the preceding four steps until best symmetry is obtained, then tighten the setscrew. Good agreement between the ascending and descending traverses is more important than having reversals occur at the exact stage.
- Lengthen or shorten the effective length of the pen arm so the upper reversal occurs at the 5.05-inch (12.8 cm) line. The effective length can be increased by adding thin, noncorrosive metal bands to the pen arm, back of the pen. If the effective length must be decreased, file back the pen-arm shoulder against which the pen is set. *Never* try to change the length of the pen arm by bending it. The alternate calibration method, which follows, is probably more satisfactory than filing the pen arm. Changing the length of the pen arm will change the position of the lower reversal, which must be adjusted by screw K before checking the upper reversal.

Calibration of FW–1 Recorder in the Field:

If an FW–1 recorder has not been calibrated according to recommendations in the previous section, field calibration can be used to adjust the instrument. Although field calibration is not as accurate as office calibration, it usually will leave the instrument reading within 0.005 foot (0.15 cm) of the correct value at all stages, which is sufficiently accurate for this type of work. The procedure for field calibration is:

- Set dividers at 4.45 inches (18.3 cm) and mark off 10 spaces along the arc of this circle. These 10 points should be equidistant; if not, reset the dividers to obtain 10 equal spaces.
- Punch or drill 3/32-inch (0.24 cm) holes at each of the 10 points.
- Drill an 11/32-inch (0.08cm) hole in the center.
- Cut away most of the material to leave a center hub, six to eight spokes, and the outside ring.
- Place the recorder on a table or stand so that the float wheel just clears the edge.
- Loosen screws E and remove the knurled brass disk and the float wheel.
- Place the calibration wheel on the shaft, replace the float wheel, replace and tighten the knurled brass disk, and tighten screws E.
- Mark a line, on the edge of the table or stand, directly behind the outside rim of the calibration wheel, for use as a reference point. The edge of the table top or stand also can be used for this purpose.
- Make sure the gear on the clock matches that on the recorder spindle.
- Fill in the data on the end tabs of a 1940 AB chart with india ink. Draw a horizontal line, about 2 inches (5.08 cm) long, near the right end of the chart, 5.05 inches (12.8 cm) above the lowest graduation. This represents the expanded width of the chart under conditions of high humidity (fig. 2.70).
- Wrap the chart around the clock cylinder; be sure that the chart fits snugly and smoothly and rests on the bottom flange throughout its circumference. (Note: In using the AB chart, which has a margin at both ends, the right margin is folded under and then placed over the left end with the chart clip running down through the fold.)
- Place the clock in the recorder by grasping it with the winding key and lowering it gently over the spindle until the gears are meshed. Be sure the clock rotates freely and does not bind at any point. Binding may indicate improper position of the clock mechanism with respect to the spindle, or it may indicate a bent spindle. To correct binding, remove the clock, loosen the three screws on the base, shift the gear away from the center, and tighten the screws. Further binding probably is caused by a bent spindle, which should be straightened or replaced.
- Rotate the float wheel to see that the pen arm is tilted inward just enough to prevent its falling away from the chart near the upper reversal.

• Place a small quantity of ink in the pen and check to see that the point gives a clear fine line. Check adjustment of the recorder as follows (for designated parts, see fig. 2.69).

• Note that the pen arm and the cam follower arm, which holds the counterweight *M* and *C* (fig. 2.68), are parallel.

• See that the cam follower pin *C* is horizontal by sighting over the shaft that carries the pen arm and the cam follower arm.

• To see how close the pen follows the time line, place the pen on the chart and rotate the float wheel through the upper and lower reversals. Deviation from the time line between the upper and lower reversals indicates either a bent spindle or improper thickness of clock spindle washer. Remove the clock, rotate the spindle 90°, and repeat the check. Failure of these pen traces to agree for positions of the spindle indicates that the spindle is bent and should be straightened or replaced before continuing with the check. If the arc described by the pen is the same for two or more positions of the spindle but still does not follow a time line, it indicates improper thickness of washer between the base and the gear. When the arc falls to the right of the time line at the top of the chart (fig. 2.70), the clock spindle washer is too thick. When the arc falls to the left, the washer is too thin. Change the washer as indicated and check until agreement with the time line is good. The proper thickness of the washer for some older recorders is about 0.140 inch (0.36 cm).

• Rotate the float wheel to bring the pen within 1 inch (2.54 cm) of the lower edge of the chart. Place the pen on the chart and continue rotating the float wheel until the pen reverses. If this lower reversal occurs above or below the lowest graduation, bring it to this line by adjusting screw *K*. Continue rotating the float wheel to make the upper reversal, which should occur near the 5-inch (12.2 cm) line. If the distance between upper and lower reversals does not come within 0.02 inch (0.05 cm) of the 5-inch (12.7 cm) line, change the length of the cam follower arm. If the traverse is less than 5 inches (12.7 cm), shorten the cam follower arm. If more than 5 inches (12.7 cm), lengthen the arm.

• Check for ascending and descending symmetry. Rotate the float wheel in a clockwise direction, stopping at each 0.1 foot (3.0 cm) on the tape index pointer. Rotate the clock to make a series of short lines to show the actual chart reading for each 0.1 foot (3.0 cm) until the pen has made one complete traverse up and down across the chart.

• If the lines on the ascending and descending sides do not agree, select the point of maximum discrepancy. Hold the float wheel firmly, loosen screws *E*, turn the knurled brass disk to correct for half the difference, and tighten screws *E*. Repeat until the two lines agree and repeat check for symmetry.

• Lengthen or shorten the effective length of the pen arm so the upper reversal occurs at the 5.05-inch (12.8 cm) line. The effective length can be increased by adding thin, noncorrosive metal bands to the pen arm, back of the pen. If the effective length must be decreased, file back the pen-arm shoulder against which the pen is set. *Never* try to change the length of the pen arm by bending it. The alternate calibration method, which follows, is probably more satisfactory than filing the pen arm. Changing the length of the pen arm will change the position of the lower reversal, which must be adjusted by screw *K* before checking the upper reversal.

• Place a tape clamp across the counterweight end of the tape about 1 inch (2.54 cm) below the bottom of the floor of the shelter house. This will prevent the float from dropping to a reading more than 1 inch below the bottom of the lowest intake. Recheck the tape index pointer and chart line readings, and make further adjustments, if necessary. Remove most of the excess tape at the float. Rotate the float wheel in a counterclockwise direction until the tape clamp touches the bottom of the floor of the shelter house. Place the pen on the chart and rotate the clock to mark a line about a half inch (1.3 cm) long. Remove the pen from the chart. Remove the chart from the clock cylinder and blot. Upon returning to the office, complete entries on the chart with india ink (fig. 2.70) and retain the chart and notes in the office file.

Current Meter Maintenance:

Operation of a current meter will be affected by the way in which it is used. While the design, material, and construction of the meter also contribute to its successful operation, they will not prevent errors caused by improper care of the instrument. Each fieldman should keep his meter in the proper condition, but he should not make repairs that may affect the rating of the meter. He should check the following items before and after each measurement.

True shaft alinement.—The shaft of any meter may become bent if the meter is subjected to a sharp blow or if the bucket wheel is lifted improperly. When lifting the cups from the pivot point by the bucket-raising nut, always hold the cups stationary and turn the bucket-raising nut. Spinning the rotor while the bucket-raising nut is held stationary creates momentum that, when suddenly checked, may spring either the yoke or the shaft. The shaft assembly does not need to be dismantled to see if the shaft is in alinement. With the contact chamber in place, the cap removed, and the meter held in a vertical position, turn the bucket wheel slowly. Watch the metal frame to which the inner edges of the cups are fastened and see if the shaft is bent. Observe movement of the shaft inside the contact chamber. A wobble not caused by a worn bearing lug in the chamber indicates that the shaft is bent. As a final test, dismantle the shaft and roll it on a flat surface. Repair any meter found with a bent shaft.

Proper Condition of Pivot Point and Pivot Bearing:

After every discharge measurement, thoroughly clean both the pivot and pivot bearing and remove all water, oil, dirt, or abrasive substances that may be present. Examine the pivot with a magnifying glass to see whether the point is fractured, worn, flat, or rough at the apex. Discard any pivot that is fractured or has a rough point. To examine conveniently and to clean the pivot bearing, carefully remove the contact chamber, so as not to mar the penta gear, and tilt the shaft assembly to one side. When the pivot bearing becomes coated with an oily substance that cannot be removed with a cloth, a blunt, soft wooden pin, turned several times within the bearing, usually will clean it. If the bearing shows rust, use a few drops of oil. Examine the pivot bearing and the pivot point for possible fracture, pits, and roughness, but do not remove the bearings from the housing. Do not pack the meter or transport it with the pivot bearing resting on the pivot point. Use a raising nut (*15*) to raise the cup wheel off the pivot when not in use.

Cleaning, Oiling, and Inserting New Parts:

In addition to the pivot bearing just discussed, every current meter has bearing surfaces above the bucket wheel. These include cylindrical bearings, mesh bearings, and thrust bearings. Examine these bearings and oil and clean them after each discharge measurement. If a new part must be inserted in the field, always test the meter afterward for proper adjustment. Take additional precautions in replacing the contact chamber cap as all caps do not screw into the chamber to the same fixed depth. During high velocities the caps may become worn by the head of the shaft riding on the cap, and occasionally the cap must be refaced when the meter is being repaired. Before a new cap is tightened into the contact chamber, lower the pivot or release the cups by lowering the bucket-raising nut because the head of the shaft may be brought to bear against the cap with sufficient pressure to bend or throw the shaft out of alinement. For lubrication, use the highest grade of oil as tests have shown that inferior oils do not eliminate the corrosion of several parts.

Standard requirements for operation.—The so-called "spin test" is the common method for determining the condition of the meter. Correct interpretation of the results from the test will determine the condition of the meter. When a meter is subjected to a spin test, place it so that the shaft is in a vertical position and the cups are protected against all air movements. As the rotating cups near the stopping point, observe them carefully to note whether they stop abruptly or gradually. Velocities more than 0.6 ft/s (18 cm/s) give sufficient excess motive power to overcome any appreciable effect of mechanical resistance in the meter. Therefore, the principal value of the spin test for velocities above 0.6 ft/s is to indicate whether the meter is operating freely. Any meter that spins 1½ minutes is satisfactory for measuring all velocities except those that are very low. A meter that spins 1 minute will measure velocities above 1 ft/s without appreciable error.

As the velocity decreases, motive power also decreases. For velocities less than 0.6 ft/s (18 cm/s) the difference between motive power and mechanical resistance of the meter decreases. Therefore, a meter should be well adjusted and should spin at least 2 minutes when velocities less than 0.6 ft/s (18 cm/s) are measured.

Check any meter not meeting the required spin test for pivot and bearings. Replace damaged wheel shafts and other parts.

The National Bureau of Standards in Gaithersburg, Md., calibrates the current meter at a small cost. The rating curve supplied by the National Bureau of Standards is used to prepare the rating table. Meters commonly are rated on a 0.5-inch (1.27 cm) round rod. Coefficients are available for adjusting the rod rating to standard cable suspension ratings with 15- or 30-pound (6.8 to 13.5 kg) lead weights.

Miscellaneous Gaging Equipment

Miscellaneous gaging equipment required may vary appreciably, depending on the type of measurement to be made—wading, cableway, bridge, boat, or through ice. A checklist of required items should be prepared by the engineer. All items should be inspected after each use, and repairs, adjustments, or replacements should be indicated immediately (*41*).

Discharge Measurement

Discharge measurements usually are computed in cubic feet per second and are plotted against gage height to define the station rating. At a shifting control station, measurements should be made frequently enough to achieve the desired accuracy in the runoff record. Weekly measurements with velocity observations at 15 verticals are preferable to biweekly measurements with 30 verticals if the station control is subject to considerable scour or fill. The reverse may be more desirable in field rating a man-made control.

All equipment on the inventory form shown in figure 2.71 is not needed to make a discharge measurement. The form has been useful, however, as a checklist for items that may be required. A semiannual inventory of quantity and condition of equipment is recommended.

Selecting the Measuring Station

The cross section for a discharge measurement should meet the following requirements, if possible:

- The section should be accessible.
- The section should be near the stage recorder and perpendicular to the direction of streamflow.
- There should be a straight and uniform channel for a distance upstream equal to at least five times the width of the stream and for a distance downstream equal to twice the width of the stream.
- No more than 15 percent of the flow should have a velocity less than 0.5 ft/s (15.2 cm/s), and the maximum velocity should not exceed 4 ft/s (1.2 m/s).
- No large overflow section should occur at flood stage. In some streams, a gaging section that is always free of turbulence cannot be found. Methods of gaging other than the velocity-area method may be more accurate for these streams. If piers and piling cause considerable turbulence under an existing bridge, the need for accurate discharge measurements should be weighed against the expense of spanning the stream with a special bridge or cableway.

Accuracy of the Price meter is poor in flow depths less than 0.5 foot (15.2 cm). If more than 15 percent of the flow is less than 0.5 foot (15.2 cm) deep, a Pygmy meter should be used in lieu of the Price meter. If a large amount of flow is less than 0.2 foot (6.1 cm) deep, an artificial section should be created by building a small dike out into the flow. This dike should be located downstream at a distance that depends on the new flow width.

Extreme caution must be used when entering a flowing stream to ensure against drowning caused by depths greater than anticipated or velocities too great to maintain footing. Excellent safety precautions are life preservers, which are mandatory when current meter read-

INVENTORY OF
DISCHARGE MEASUREMENT EQUIPMENT

Quantity	Item	Quantity	Item
________	Crane (USGS Type A)	________	Meter (Pygmy)
________	Crane (Special)	________	Meter (Price)
________	Reel (Type A)	________	Meter (Other)
________	Reel (Heavy Duty)	________	Headsets
________	Handline	________	Batteries (Wading)
________	Rule (six ft. aluminum) (graduated feet, tenths and hundredths)	________	Batteries (Crane)
		________	Wading rod (4 sections, base plate and slide) or top-setting rod
________	Weights (100 lb.)		
________	Weights (50 lb.)	________	Hangar Bars
________	Weights (30 lb.)	________	Stop Watch
________	Weights (15 lb.)	________	Stop Watch Holder
________	Signs (Men Working)	________	Pins (Connector)
________	Cable Car Puller	________	Pins (Weight)
________	Key for Car Lock	________	Screw (Meter)
________	Life Jacket	________	Bearings (Price, extra)
________	Tape (300 ft.)	________	Notebooks and Forms
________	Tape (100 ft.)	________	Pencils
________	Tape (50 ft.)	________	Tape (plastic)
________	Tagline	________	Pliers
________	Waders (Usable)	________	Screwdrivers
________	Lights (Usable—12 volt)	________	Cable Cutters
________	Lights (Usable—flash)	________	Wrenches
________	Lights (Usable—lantern)	________	Container of Clear Water
________	Lights (Flashing signal)	________	Rags
		________	Oil

COMMENTS:

____________________	____________________	____________________
Date of Inventory	Storage Location	Signature

FIGURE 2.71.—Inventory form of equipment used in discharge measurement with a current meter.

ings are obtained, and a two-man team with only one man in the flow.

Rod Suspension Measurement

The procedure for making streamflow measurements using the rod suspension method is as follows:

- Stretch the tape across the stream (fig. 2.72) as closely as possible at a right angle (90°) to the flow lines. Record location of measuring site (fig. 2.73).
- Observe the width of the water surface and divide this number by 20 to determine the position of the points of depth and velocity observations across the stream. Twenty observations will reduce the error in total gaging caused by inaccuracies of the single observations. If the stream width is 20 feet (6.1 m), make single observations at 1-foot (0.3 m) intervals. If this width is about 15 feet (4.7 m), 1-foot (0.3 m) intervals should be used along the banks when the velocity is low, and 0.5-foot (15.2 cm) intervals should be used in the stream section if velocity is high (fig. 2.72). The number of observations is not absolute. Conditions may warrant 15 or more observations across the stream. If the gage height changes rapidly, 15 observations or less may be necessary to com-

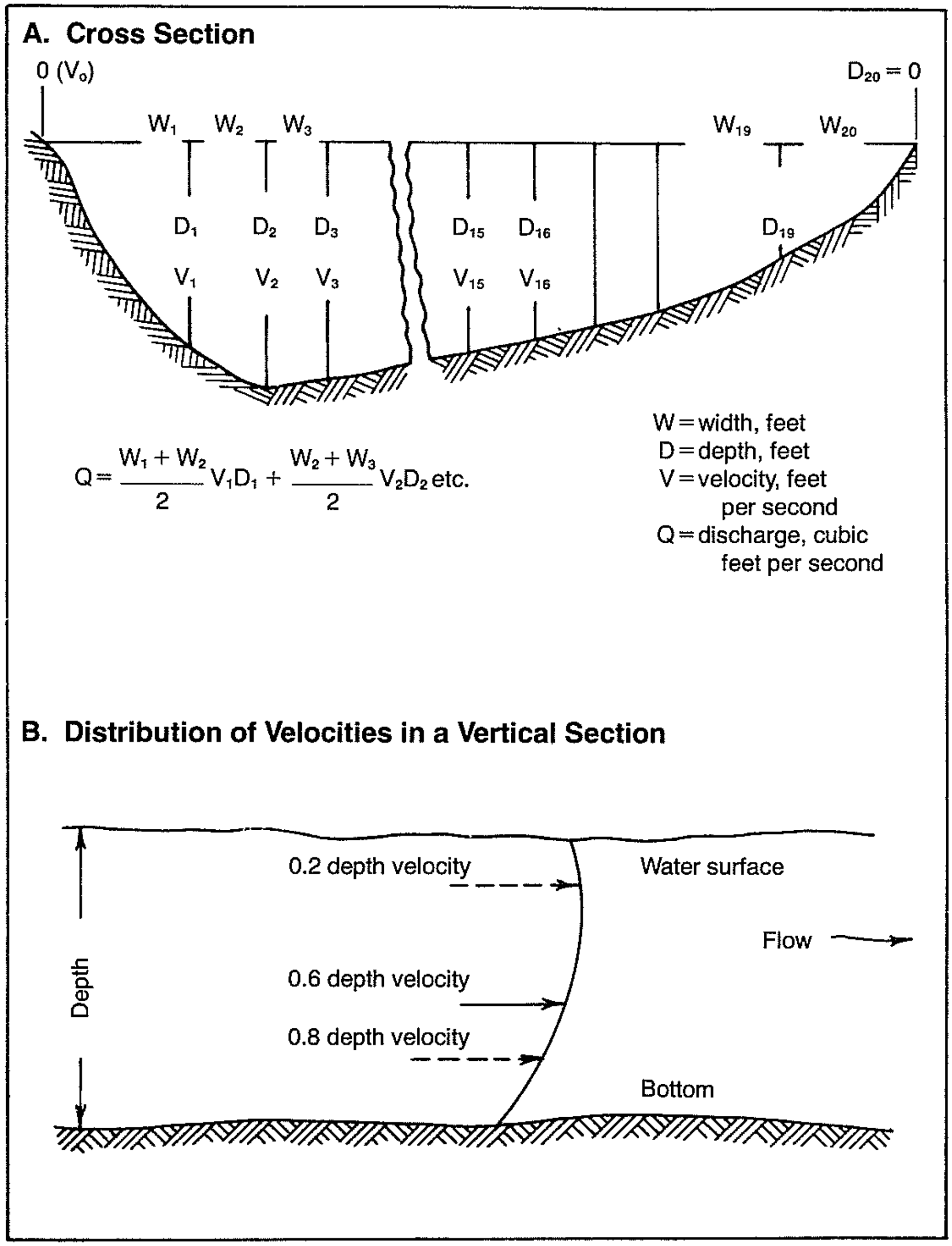


FIGURE 2.72.—Measurements needed to record channel cross section and vertical velocity curve.

Meas. No. 12

Comp. by W.H.P.

DISCHARGE MEASUREMENT NOTES Checked by C.M.B.

Little Mill Creek #95, Coshocton, Ohio

Date Jan. 6, 1956 Party L. W. Smith

Width 20.0 Area 32.3 Vel. 0.97 G. H. 1.24 Disch. 31.2

Method 2-8 No. secs. 20 G. H. change -0.1 in 1 hrs. Susp. Rod

Method coef. .... Hor. angle coef. Var. Susp. coef. 1.0 Meter No. SCS-13

GAGE READINGS				
Time	Sta.	Recorder	Inside	Outside
8:25A	—	1.32	1.32	1.32
8:30	Start			
9:00	19	1.25		
9:35	End			
9:40	—	1.20	1.20	1.20
Weighted M. G. H.		1.24	1.24	1.24
G. H. correction		0	0	0
Correct M. G. H.		1.24	1.24	1.24

Date rated 3-22-40 Used rating for rod — susp. Meter — ft. above bottom of wt. Tags checked —

Spin before meas. 3-0 after 3-0

Meas. plots ......% diff. from .... rating

Wading, cable, ice, boat, upstr., downstr., side bridge 50 feet, mile, above, below gage, and ....

Check-bar, chain found — changed to — at —

Correct —

Levels obtained —

Measurement rated excellent (2%), good (5%), fair (8%), poor (over 8%), based on following conditions: Cross section ....

Flow .... Weather Good

Other .... Air ......°F@ ......

Gage .... Water ......°F@ ......

.... Record removed .... Intake flushed No

Observer ....

Control ....

Remarks ....

G. H. of zero flow 0.00 ft.

FIGURE 2.73.—Data sheet for discharge measurement of stream.

plete the measurement before the stage changes too much—0.5 foot (15.2 cm) or more is too much change for some stages.

- Assemble the meter equipment. Check electric current and spin of the bucket wheel for friction tests. Check stopwatch. For low velocities, the meter should spin at least 90 seconds in no wind, after being given a good start. For high velocities, a 30-second spin should be satisfactory.
- Record current-meter number and type of measurement—rod or weight suspension from cable (fig. 2.73).
- Read the record time and gage height of water surface at recorder gage. (See notes, fig. 2.74). Mark this time on the recorder chart by rotating the float wheel or clock.
- Begin measurement by recording on the second sheet of notes (fig. 2.74) the tape reading (distance from initial point at each station or point of velocity observation across the stream) and depth of water on wading rod, both to nearest 0.1 foot (2.5 cm) at edge of water. LEW = left edge of water; REW = right edge of water looking downstream. If velocity of water at edge cannot be measured, estimate it as zero or as a percentage of the next measurement from initial point.
- Move across the stream and record distance, depth of water, and velocity of flow (revolutions per second of current meter) at each selected point of observation. Measure velocity as follows:
  - For depths less than 1.5 feet (45.7 cm), set the center of the current-meter bucket wheel at 0.6 of the depth and record 0.6 in the column "Observation Depth." If depth is 0.9 foot (27.4 cm), set the meter at $0.6\times0.9 = 0.54$ or 0.5 foot (15.2 cm) below the water surface. In streams of less than 0.5 foot (15.2 cm) depth, the standard-sized current meter does not measure velocity correctly. The Pigmy current meter often is used in shallow depths. Make corrections, however, to Pigmy meter velocities for depths of 0.3 foot (9.1 cm) or less. Avoid shallow depths wherever possible. Where shallow depth cannot be avoided, refer to reference (*31*) for correction coefficients.
  - For depths 1.5 feet (45.7 cm) or greater, make two observations of velocity at each station—0.2 and 0.8 of the depth—and record these two figures in the column "Observation Depth." If the depth of water at a station is 1.8 feet (54.9 cm), set the current meter at $0.2\times1.8 = 0.36$ foot and $0.8\times1.8 = 1.44$ feet (43.9 cm) below the water surface. Record velocity (revolutions per second) readings for both depths on their respective lines. For example, in figure 2.74 at station 18 (distance from initial point), the velocity at 0.2 of the depth was 10 revolutions in 90 seconds; that at the 0.8 depth was 10 revolutions in 25 seconds.
  - If ice or aquatic growth disturbs the normal distribution of velocity in the vertical section, make additional velocity measurements in the vertical section to establish the true mean for the section.
  - Hold the rod still, in a vertical position,

Little Mill Cr. #95 River at — Coshocton, O. 1-6-56

Angle coefficient	Dist. from initial point	Width	Depth	Observation depth	Revolutions	Time in seconds	Velocity: At point	Velocity: Mean in vertical	Adjusted for hor. angle or	Area	Discharge
	Meas. 50 ft. below gage Meter SCS-13, Rod										
	8:25 A.M. O.G. 1.25, I.G. 1.25, Pen 1.25										
	L.E.W. at 8:30 A.M.										
	12	0.5	0.6	(est. 2/3 sta. 13				0.111		0.3	0.033
	13	1	.8	.6	3	50		0.167		.8	.134
	14	1	.8	.6	5	22		.548		.8	.438
.99	15	1	1.1	.6	10	34		.701	.694	1.1	.764
.97	16	1	1.4	.6	10	28		.844	.818	1.4	1.145
.95	17	1	1.6	.2	10	22	1.07	.930	.884	1.6	1.415
o				.8	10	30	.790				
.98	18	1	1.8	.2	10	20	1.17	1.056	1.035	1.8	1.862
				.8	10	25	.942				
	19	1	2.2	.2	10	22	1.07	.988		2.2	2.173
	9:00 A.M.			.8	10	26	.907				7.964
	20	1	2.2	.2	15	24	1.45	1.285		2.2	2.828
				.8	10	21	1.12				
	21	1	2.3	.2	15	21	1.66	1.415		2.3	3.254
				.8	10	20	1.17				
	22	1	2.5	.2	15	24	1.45	1.196		2.5	2.990
				.8	10	25	.942				
	23	1	2.5	.2	15	26	1.34	1.141		2.5	2.854
				.8	10	25	.942				
	24	1	2.3	.2	10	20	1.14	1.023		2.3	2.342
				.8	10	26	.907				
	25	1	2.1	.2	10	21	1.12	1.013		2.1	2.128
				.8	10	26	.907				
							continued—				
	26	1	2.0	.2	10	22	1.07	.972		2.0	1.944
				.8	10	27	.874				
	27	1	1.8	.2	10	25	.942	.853		1.8	1.535
				.8	10	31	.765				
	28	1	1.6	.2	10	26	.907	.804		1.6	1.286
				.8	10	34	.701				
	29	1	1.4	.6	10	31		.765		1.4	1.071
	30	1	1.0	.6	10	30		.790		1.0	.790
	31	1	.4	.6	5	25		.486		.4	.194
	32	.5	.4	.6	5	50		.258		.2	.052
	R.E.W. @ 9:35 A.M.										
	9:40 A.M. O.G. = 1.20, I.G. = 1.20 pen = 1.20										
o								Totals		32.3	31.23

FIGURE 2.74.—Field notes and discharge computations.

with the meter facing upstream. Position of observer and placement of meter are important in wading measurements. Research is needed to determine the effect of the observer's position on the accuracy of meter registration. Stand with your body streamlined to the flow as far from the meter as possible and at an angle of 45° with the tape and meter. This reduces the registered velocity less than 4 percent. If the stream is no more than two arms lengths wide, stay out of the water during velocity observations.

- Inspect the meter frequently and keep it free from debris. Listen to the rhythm of the electric contacts. If it changes greatly, check the meter.
- Count the revolutions of the meter bucket wheel as indicated by electric clicks on the earphone while observing the time on the stopwatch. If measurement is not needed quickly, count the revolutions for not less than 40 seconds, nor more than 70 seconds, stopping at a revolution count of 3, 5, 7, 10, 15, 20, 25, 30, 50, 60, and so forth. When flow stage is changing rapidly, cut the time in half—20 to 35 seconds—to finish the measurement of discharge without too much change in stage.
- Record the angle of flow correction

where waterflow is not at 90° to the tape across the stream. Use the dot on the right margin and the numbers (angle coefficients) on the left margin of figure 2.74 to estimate the angle corrections. Hold this form in a horizontal plane with the dot next to you and the column lines parallel to the tape stretched across the stream. Watch the flow current and visually trace its direction across the form to get its angle. If the angle is about 90°, there is no correction. Note that the angle correction recorded for station 18 was 0.98 (fig. 2.74).

- Record the time when about halfway across the stream. (See station 19, 9:00 a.m., on fig. 2.74.) If stage is changing rapidly, record the time when one-third and two-thirds the way across the stream. This will help establish the weighted mean gage height for the entire measurement.
- Record edge of water and time at finish of measurement. Immediately observe water level at gage and mark the chart for later reference, if necessary.
- Make supplementary notes at the end of the form (fig. 2.74), which should be completed in the field as soon as possible after the discharge measurement.

Cable Suspension Measurement:

Streamflow measurements are made from a bridge or a cable car only when depth and velocity of the stream are unsafe for wading. *As a rule of thumb, it is unsafe when produce of the depth and velocity exceeds 10.* The procedure for making measurements from a bridge or cable car is the same as the procedure for wading measurements, except for the following:

- Measure bridge from the upstream side. Floating debris can be better observed and avoided in this way.
- Fasten streamline lead weight to the hanger in one of the bottom holes of the hanger bar. Suspend current meter in the hole marked for the size of weight being used as follows:

  15, C = 0.5 foot (0.15 m);
  30, C = 0.5 foot (0.15 m);
  50, C = 0.9 foot (0.27 m);
  75, C = 1.0 foot (0.30 m);
  100, C = 1.0 foot (0.30m); and
  150, C = 1.0 foot (0.30 m).

The first figure is the weight in pounds. The last figure is the vertical distance from the bottom of the weight to the center plane of the bucket wheel. This value must be used throughout the measurement. It is termed the depth constant of the weight below the meter. If you do not remember the value of C, the distance can be measured with a tape or rule to the nearest $^1/_{10}$ foot (3 cm). Selection of the weight size depends on the stream velocity and the equipment available for handling the weight.

- Zero the depth indicator with the center of the meter cups at the water surface. Lower the weight till it rests gently on the streambed. The depth is then the indicator reading plus the C-value. Velocity observations normally are made at 0.2 and 0.8 depth except in shallow water where the observation may be made at 0.6, 0.5, or 0.2 depth only. The weight should be large enough to prevent the line from drifting downstream more than 7° from the vertical. If this is impractical, make a depth correction depending on the size of the vertical angle. A 15-pound or 30-pound (6,810 g or 13,620 g) weight can be lifted by hand. Heavier weights require a hand-or power-operated reel. Where a rubber-covered hand cable (without reel) is used, the operator sets the meter cup centerline at the water surface; clamps a tape to the cable at a reference point such as the top of a handrail; lowers the weight to the stream bottom, allowing the clamped tape to follow; and reads the tape at the reference point. The tape reading plus the C-value is the water depth. Both air-line and wet-line correction tables are given in reference *41*. Corrections also are given for placing the meter at 0.2 and 0.8 depth.

- Use the form illustrated in figure 2.74 during the preceding measurement. If the gage height changes rapidly, however, additional gage height readings are required to obtain an accurate mean-weighted gage height for the measurement. A gage height reading should be recorded at about every 5 percent change in maximum flow depth. Inside readings and chart gage height readings should be adjusted to the outside gage reading unless accuracy of the outside reading is questionable because of wind or wave action.

Section Rating Method:

This method is recommended for making flow measurements when the gage height may change more than 10 percent of the maximum flow depth during a discharge measurement. Select 10 to 20 verticals in the measuring section so that 5 to 10 percent of the flow will be between each pair of adjacent verticals. During storm runoff, observe the velocity at these verticals enough times to adequately define a rating curve for each portion or section of the cross section represented by each vertical. Section rating curves may be combined to make up a composite rating curve for the storm or the station if the curve has a stable control.

Low Flow Estimates:

Accuracy of the low flow runoff record at shifting control stations can be improved with little additional labor by estimating flows less than a few cubic feet per second each time the observer visits the station and does not make a discharge measurement. The area of a cross section of the flow may be determined by multiplying the estimated flow width by the estimated average depth of flow. The surface velocity of the flow may be estimated by timing the movement of a floating object over a course that is a few feet long. The surface velocity should be multiplied by a factor between 0.6 and 0.8 depending on the distribution of velocity across the stream. The discharge rate is the area times the average velocity. Since the accuracy of this measurement is not comparable to the accuracy of measurement with a current meter, this method should be used sparingly.

Safety:

The greatest hazard in measuring streamflow is probably inability to swim and to work free from equipment and clothing when footing is lost while wading or when the gaging structure fails. All streamflow observers should be able to swim and should wear a lifejacket at all times. When wading measurements are being made, a tag line should be installed at the gaging section for the hydrographer to grab if he falls in the water.

The second greatest hazard to the streamflow observer may be caused by traffic on bridges where he often is required to work. Warning signals and signs can help if they are approved by local traffic officials.

The observer should carry a well-stocked first aid kit in his vehicle and should be familiar with the kit's contents and standard first aid practices. It is best to work in pairs when gaging with one person on the shore in the event of an accident. Protection from lightning, such as grounding the cableway or bridge, must be provided.

Gage Datum Maintenance

The ability to determine the quantity of water flowing through a calibrated gaging station depends on several types of information, one of which is the depth of water flowing through the control section. Instruments described in the instrumentation section (p.) are used to determine elevation of the water surface continuously or at a predetermined point in time. Flow depths are equivalent to the differences in elevations of the water surface and the lowest point of control over which the water can flow. Elevation of zero head through the control is referred to as gage datum elevation (GDE).

A permanent bench mark (BM) should be established for convenient use in checking and maintaining the proper relationship between the GDE and the water level recorder or gage. Elevation of the GDE (referenced to the BM) must be determined. For some flow controls it can be determined with an engineer's level backsight on the BM and a foresight on the low point of the flow control. For other controls, such as formed concrete weirs, the GDE of zero head should be computed by a method such as the application of least squares, based on a detailed and accurate survey of the flow control.

Recording water level gages have characteristics typical of most automatic equipment. Quality of the output (records of water surface elevation) varies directly with the amount and quality of equipment maintenance. The types of required maintenance depend to some extent on the type of instrument used. Some common situations that produce erroneous records of water surface elevation are:

- Debris may accumulate in or around the stilling well intake or both;
- Debris or aquatic growth may accumulate near or on the flow control;
- Recorder float (if used) may develop a leak or acquire debris, causing the float to lose buoyancy;
- Float tape or cable may jump a cog or slip;
- Recorder may get out of adjustment;
- Insects such as mud daubers may build nests on float tapes, restricting vertical movement.

For most flow controls, the nonlinear relationship between the flow depth and the quantity of water flowing produces a situation where small errors in flow depth measurements will result in large errors in computed streamflow. Procedures for checking recorder accuracy are required, therefore, for all types of instruments. These procedures vary with the instrument, but they should include techniques for quick visual comparisons of the actual and recorded elevations of water surface and for determining the water surface elevations, or depths of flow, to the nearest 0.001 foot (0.03 cm). Staff gages are used for quick visual comparison of elevations of water surface. Some techniques for accurately determining flow depths were discussed in the previous section. The procedure for checking gage datums follows:

For stations where streamflow is continuous, the water surface GDE can be checked by a backsight with an engineer's level, on the FM, and a foresight on the water surface at a point outside the stilling well, which represents the same elevation as the water surface inside the stilling well.

The conventional level rod can be improved for use in obtaining water surface readings by attaching a sharp point to the lower end. Additional improvement can be made by adding a jig to the lower end of the rod. This jig will help support the weight of the rod in the soft material usually found in or near streambeds. It also will allow the rodman to lower or raise the rod under a steady condition (see fig. 2.75).

Checking the GDE by use of the engineer's level requires at least two individuals. In lieu of this procedure, a system usually can be established that will enable one person working alone to make frequent GDE checks. This alternate system requires a conventional point gage and a small amount of initiative (fig. 2.76).

For stations where flow is not continuous and maintenance checks are made during periods of no flow, water must be transported to fill the gage well to the lowest opening and a point-gage mount must be installed inside the well. Set up the level so you can read the point gage when it is held on the crest as a rod and when it is mounted in the well. Use the point gage as a level rod and take backsight readings, *B.S.*, on the crest (be aware that the point gage is graduated upside down), using the vernier for an accurate reading to the nearest 0.001 foot (0.03 cm). Fix the point gage in the mount inside the well. Sight the point gage with the level and move it until a mark, say 1.000, is on the line of sight. This is the foresight, *F.S.* Simultaneously obtain the point gage reading at the index of the vernier. This is point gage reading 1, *PR* 1. Take a point gage reading on the water surface in the well. This is point gage reading 2, *PR* 2. Calculate the head represented by the water level in the well as follows:

$$\text{Head} = PR\,2 - PR\,1 + F.S. - B.S. \qquad (2\text{–}43)$$

A gage datum inspection form is shown in figure 2.77.

Runoff Into Farm Ponds and Reservoirs

A detailed topographical survey must be made of any pond or reservoir used for measurement of runoff. From this survey, the volume versus the water depth relations for the pond is determined. The pond may have to be resurveyed at regular intervals depending on the rate of which the volume of the pond is altered by deposition of sediment. For the common pond encountered on small watersheds, the survey should have sufficient detail to prepare at least a 1-foot (30.5 cm) contour map of the pond. Consult any good surveying reference for instructions on how to make the topographic survey.

Ponds with water stage recorders must be serviced and maintained regularly. If the time record of depth is to be differentiated for record of inflow rate, the most expanded time scales of the recorder must be used (that is, 1 division = 5 min or less). This necessitates changing the

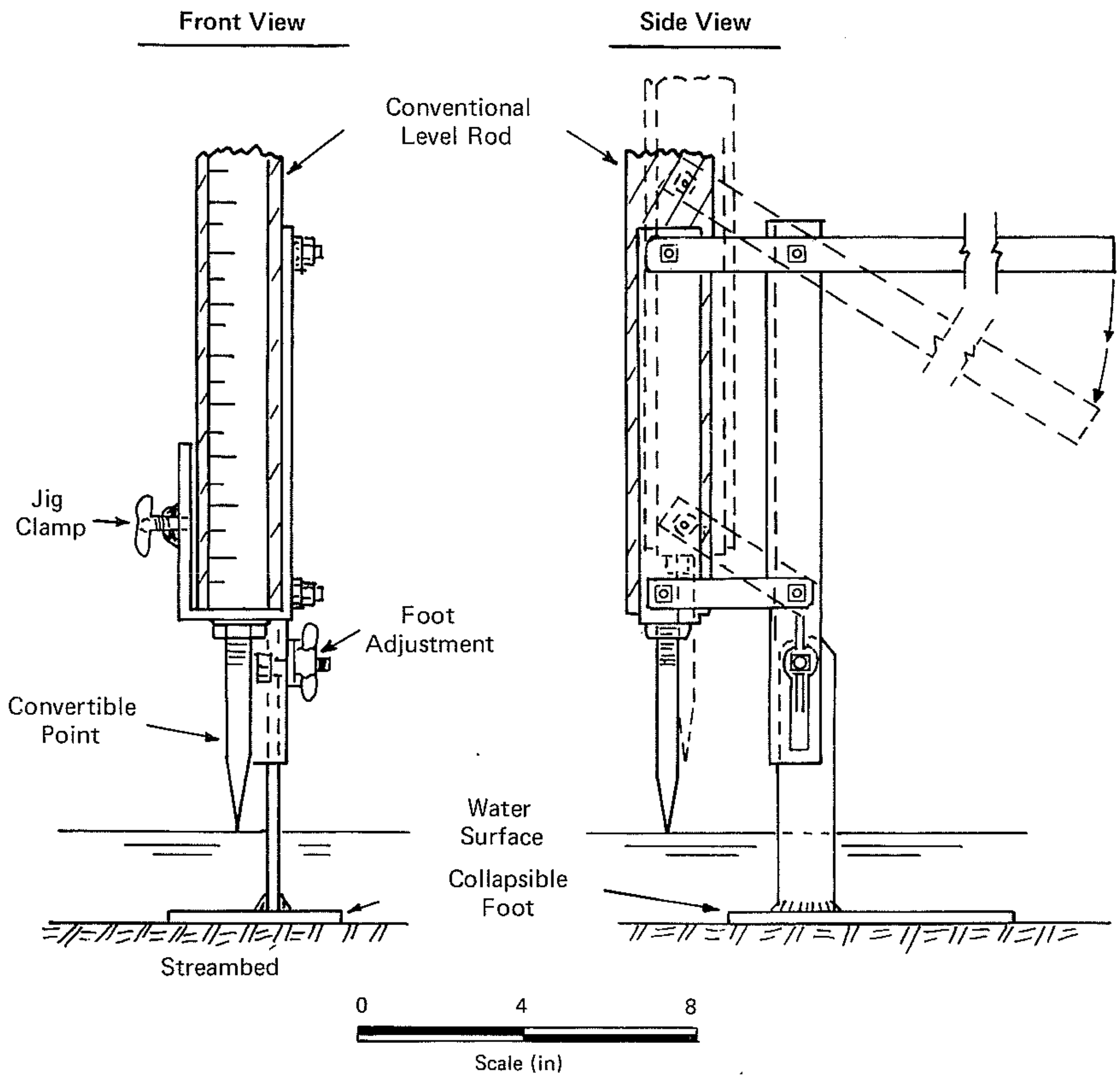


FIGURE 2.75.—Attachment for converting the conventional level rod into a convenient point gage for use in determining water surface gage datum.

chart at least once a week, winding the clock, and inking the pen. If feasible, change charts after any inflow into the pond. By doing so, the charts have a recording of one event with points of reference (beginning, end, and checks on the water level) easily determined.

Each time a chart is changed, all information to orient the record in space and time must be noted. Usual items of identification are:

- Watershed location name or number.
- Pond name or number.
- Date and time chart is put on recorder.
- Date and time chart is removed from recorder.

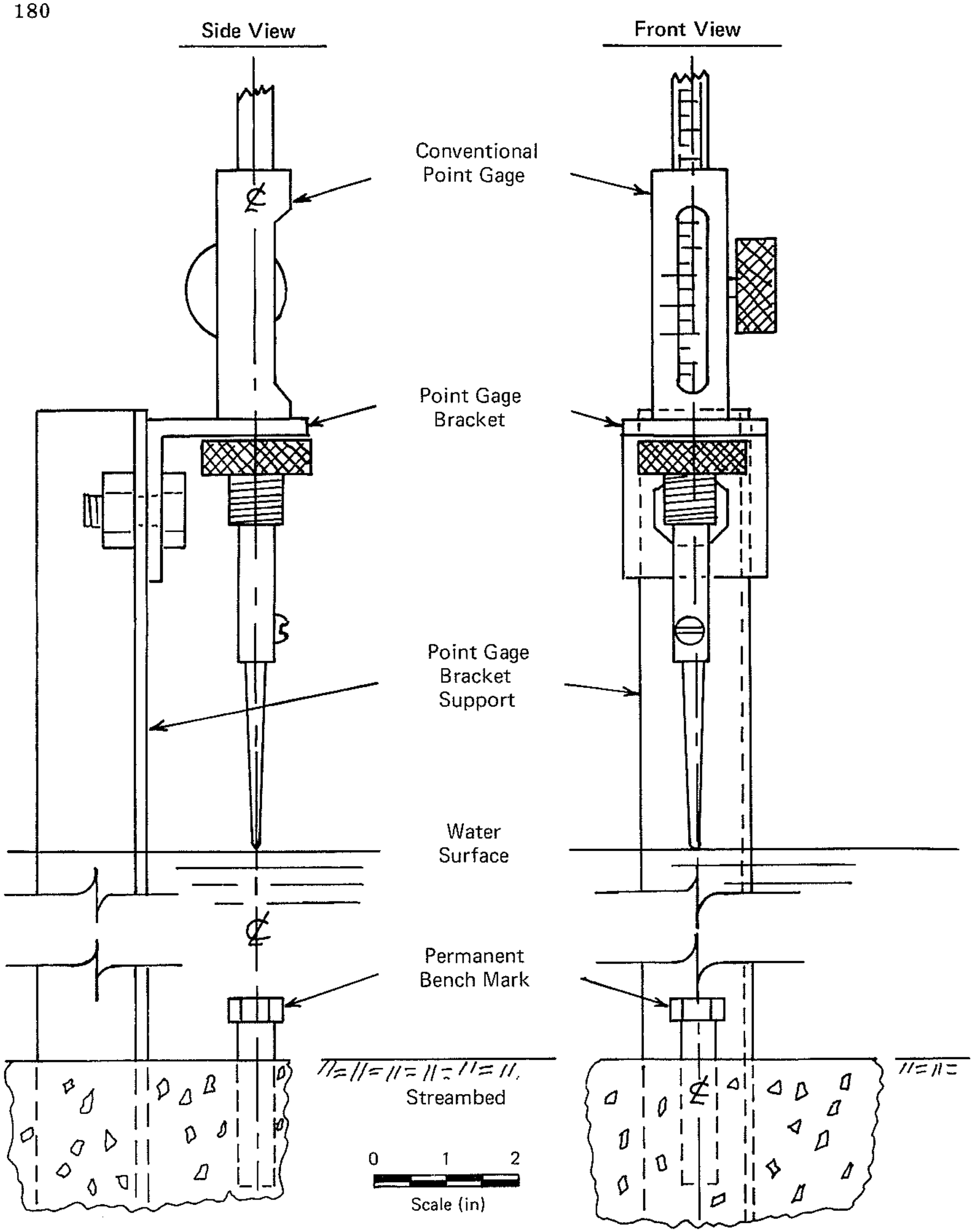


FIGURE 2.76.—Support and permanent bench mark for a portable conventional point gage.

GAGE DATUM INSPECTION

Headwater ( ) Tailwater ( ) Gage at ______________________________

Party ______________________________ Date ______________________

Instrument No. ____________________ Gage Zero ______________________

Level Notes

Station	+B.S.	H.I.	-F.S.	Observed Elevation	Correct Elevation	Elevation Corrected to

Inside Gage

(1) Elevation of R.P. = ____________

(2) Bottom of tape weight or staff gage R.P. before adjustment = ____________

(3) Observed gage zero (1) - (2) = ____________

Gage zero not changed, corrected to ________ft by adjusting tape weight to ________ft or adjusting R.P. to ________ft.

Comments:

Outside Gage

(1) Elevation of R.P. = ____________

(2) Bottom of tape weight or staff gage R.P. before adjustment = ____________

(3) Observed gage zero (1) - (2) = ____________

Gage zero not changed, corrected to ________ft by adjusting tape weight to ________ft or adjusting R.P. to ________ft.

Comments:

FIGURE 2.77.—Gage datum inspection form.

- Depth of water in pond with respect to same reference.
- Signature of person changing the chart.

Since a mechanical device makes the recording, it needs to be serviced periodically. Once a year, the recorder should be cleaned and oiled. At the same time, the recorder should be calibrated to assure that the clock is measuring time accurately and the travel of the float is proportioned properly on the chart. Plans should be made to repair occasional breakdown or damage caused by unexpected catastrophes or vandalism.

Items such as stilling well, instrument shelter, access ramps, and intake pipes are associated with mechanical recorders. These items will require general maintenance and upkeep. Intake to the stilling well must be watched closely to assure that the well is not plugged. Pond outflow structures must be maintained in good condition to assure accurate measurement of outflow from the pond if the outflow should spill.

Not as many explicit instructions regarding service and maintenance can be made for these instruments because configuration of such instrumentation varies from the old-style mechanical recorders fitted with an attached transducer driving an analog recorder (which may need weekly attention) to specially designed systems that can collect data from remote locations for long periods with no special servicing of the instruments. Any type of instrument will require periodic maintenance and calibration, and records must be collected with all the necessary identification.

Data Reduction and Processing

Runoff chart processing and basic runoff computations are accelerated with modern electronic equipment such as analog-to-digital converters and high-speed digital computers. If this equipment is unavailable, desk-sized calculators and slide rules should be used. Instructions for both types of computations are included in this section. Although both methods provide the same answers, electronic methods are faster, should have fewer errors, and can be made at lower costs where the number of computations is voluminous.

Analog Chart Record Processing

Special attention should be given to the recorder charts as soon as they are removed. Detect and note on the analog trace such abnormalities as faulty records due to clock stoppage; water level float failing to respond to water level changes in the stream because of ice, friction, or other causes; and debris lodged on the control or clogging of intake to recorder stilling well. By comparison with rainfall and runoff records from nearby stations, adjustments to the chart should be made to represent, as closely as possible, the true record. High watermarks observed at gaging stations and marked on the chart may help construct the true stage graph. Standardized procedures for chart annotation are needed to insure that all required information is recorded. Local situations usually dictate procedures to be used.

For procedures oriented around digital computer processing, the basic data, including breakpoint information adjusted for time or flow depth shift (or both if needed), should be tabulated directly on special forms for card punching. The format of these forms will depend on the computer program used to reduce and process the data (*38*).

Tabulation of data from water level recorder charts should supply sufficient detail to provide reasonable accuracy in determining instantaneous rates and total runoff. The method to be used in runoff calculation will influence the frequency with which points must be selected. Fewer points will be necessary where no pondage corrections are figured. Experience may indicate that greater or less refinement than the criteria given here will best fit a particular area. The record from water stage recorder chart (fig. 2.63) is tabulated in figure 2.78 as follows:

Fill in all data called for in the heading. Most items require no special explanation. The file number should include station designation, year, and sheet or storm number. The dates shown should include those periods with no runoff to give continuity of record to the tabulations.

On the first line of the form, show the period since the last runoff if it is more than one calendar day. These dates are included in the heading.

Enter the month, day, and time in column 1 to correspond to the gage heights selected in column 3.

Enter gage heights obtained from water-level recorder charts in column 3 in accordance with the following steps:

1. Select groups of uniform time intervals (using whole minutes only) so that the gage-height interval during any time interval is approximately 0.30 foot (9.1 cm), where no pondage correction is to be applied; 0.20 foot (6.1 cm), when necessary to apply pondage correction; and 0.10 foot (3.1 cm) or less, where the pond area is more than 2.5 percent of the drainage area. These limits apply to that part of the chart with data of rapidly rising or falling stages. For slowly rising or falling stages, the increments of stage suggested must

Location Fennimore, Wisconsin
File No. W-4-1943-7
Chart No. 233
W-4 (Horton)
(Station designation)

Gage datum elevation of point of zero flow 1.50 feet
Gage datum elevation of point of zero pondage 1.0 feet
Gage datum elevation of "float at rest" ...... feet
Gage datum elevation of lowest intake 1.093 feet

RECORD OF RUN-OFF

Slide Rule Computation

Drainage area 171 acres ...... square miles
Dates: July 17 to August 12, 1943
Rating table: RT-1

Date and time (1)	Time interval (2)	Gage height (3)	Rate of change in stage (4)	Pondage correction (5)	Observed discharge (6)	Rate of run-off (Discharge corrected for pondage) (7)	(8)	Average rate of run-off for time interval (9)	Total run-off: For time interval (10)	Accumulated (11)	Remarks (12)
Hr. and min.	Minutes	Feet	Feet per min.	Second-feet	Second-feet	Second-feet	Inches per hour	in/hr	Inches	Inches	
			No Runoff July 17 to August 12, 1943								
Aug. 12	a.m.										
4:37		1.10					0		0	0	Beginning of Rise
4:45	8	1.52					0	0	0	0	
:48	3	1.73					.0017	.0008	0	0	
:51	3	1.91					.0075	.0046	.0002	.0002	
:54	3	2.00					.0126	.0100	.0005	.0007	
:56	2	2.13					.0229	.0177	.0006	.0013	
:57	1	2.27					.0393	.0311	.0005	.0018	
:58	1	2.47					.0720	.0556	.0009	.0027	
:59	1	2.62					.104	.0880	.0015	.0042	
5:00	1	3.00					.224	.1640	.0027	.0069	
:01	1	3.65					.573	.3985	.0066	.0135	
:02	1	4.12					.963	.7680	.0128	.0263	
:03	1	4.21					1.06	1.0115	.0169	.0432	
:04	1	4.25					1.10	1.0800	.0180	.0612	Maximum Stage
:06	2	4.23					1.07	1.0850	.0362	.0974	
:10	4	4.15					.992	1.0310	.0687	.1661	
:15	5	3.90					.766	.8790	.0731	.2392	
:20	5	3.62					.553	.6595	.0549	.2941	
:25	5	3.36					.393	.4730	.0394	.3335	
:30	5	3.12					.273	.3330	.0278	.3613	
:40	10	2.74					.136	.2045	.0341	.3954	
:50	10	2.48					.0737	.1049	.0175	.4129	
6:00	10	2.30					.0436	.0586	.0098	.4227	
:15	15	2.12					.0221	.0328	.0082	.4309	
:30	15	2.01					.0132	.0177	.0044	.4353	
7:00	30	1.85					.0052	.0092	.0046	.4399	
:30	30	1.73					.0017	.0035	.0017	.4416	
8:00	30	1.66					.0007	.0012	.0006	.4422	
:30	30	1.60					.0002	.0004	.0002	.4424	
9:00	30	1.56					.0001	.0002	.0001	.4425	
:30	30	1.54					0	0			End of Runoff
10:00	30	1.52					0				
:40	40	1.50					0				Total Aug. 12 = .4425"

Conversion factor for drainage area: 1 cubic foot per second .0058 inches per hour.

Tabulated by G. R. Lloyd 9/7/43 (Date) Checked by ...... (Date)

Computed by G. R. Lloyd 9/8/43 (Date) Checked by N. E. Minshall 10/10/43 (Date)

Sheet ...... of ...... sheets

FIGURE 2.78.—Tabulation of a stage record at Fennimore, Wis.

be reduced. Where pondage corrections are required, use smaller intervals of gage height for some time before the peak stage is reached. Do not use a time interval of less than 1 minute. Computations will be easier if time intervals of 1, 2, 3, 5, or 6; 10 or 12; 20 or 30; and multiples of 60 minutes are used. Make the tabulation in accordance with the preceding steps until a head of 0.2 foot (6.1 cm) over the notch is reached on the falling stages.

2. For the part of the record below a head of 0.2 foot (6.1 cm), use the preceding time intervals with gage-height intervals of 0.02 to 0.05 foot (0.6 to 1.5 cm) (depending on the steepness of the recession) until the GDE of the notch or a reasonably constant stage is reached.

3. Where the stage becomes reasonably constant at a head of 0.2 foot (6.1 cm) or more, supply the tabulation in accordance with the first step to within 0.2 foot (6.1 cm) of the constant stage and continue in accordance with the second step until constant stage is reached.

4. Supply the gage-height intervals at and near peaks and troughs in about the same detail as for stages within 0.2 foot (6.1 cm) of the notch or constant stages.

5. Tabulate gage heights at midnight and at any noticeable break in the pen trace for runoffs involving more than 1 day.

Corrected data from the water stage recorder chart (fig. 2.63) are tabulated in columns 1 and 3 of figure 2.78. Use the criteria for a station where no pondage correction is to be applied. Note that on the rising stage the difference in stage from 5:00 to 5:01 a.m. is 0.65 foot (19.8 cm), which is controlled by the minimum time interval of 1 minute. Increments of stage from 5:10 to 6:00 average slightly less than the 0.3-foot (9.1 cm) limit for stations without pondage correction. From 7:30 to 9:00, the increments are about 0.05 foot (1.5 cm). Failure to pick enough points on the recession will result in values that are too high for total runoff.

Any constriction of the stream channel, such as the construction of a dam or spillway, will cause a certain amount of backwater or pondage upstream from the constriction. The effect of this pondage on the rate of flow should be considered in selecting the procedure for calculating runoff. If the runoff is flashy, a moderate quantity of pondage water may significantly reduce the peak outflow rates from the area and pondage corrections may be desirable. In areas where runoff rises and falls at moderate rates, these considerations may be generalized.

- If a pondage correction is applied on areas where the pond is small and the rate of change in stage is slow, the resultant hydrograph will shift a few minutes, with little difference in the shape or the peak rate. This shift probably will apply to pond areas of less than 0.5 percent of the drainage area.
- Where the pond area is 0.5 to 2.5 percent of the drainage area, a pondage correction may be necessary to obtain rates of inflow to the pond. After several storms have been computed, check to see if the pondage correction is justified. Plot the peak rate of inflow to the pond against the peak discharge over the weir for several storms. If the points fall in a reasonably straight line and do not vary more than 5 percent from a 45° line, assume that application of the pondage correction is unwarranted.
- On areas where the pond surface is more than 2.5 percent of the drainage area, consider the rain falling on the pond in addition to the pondage correction when figuring rates of runoff from the drainage area.

Since the rates of runoff are to be calculated in inches per hour and the quantities of runoff in inches, time can be saved if discharge values of the rating tables are converted to inches per hour. This will eliminate the necessity of computing rates of runoff in cubic feet per second and then applying a factor to each such rate to convert it to inches per hour. The conversion factor is obtained by dividing 0.99174 by the drainage area in acres.

A method of calculating runoff rates and amounts using a desk calculator was described by McGuinness and Royer (*26*). High-speed digital computers are much faster and more accurate and should be used when they are available.

Digital Punch Tape Processing

Although development of the analog-to-digital recorder (ADR) was a significant step in automation of streamflow data processing, final accuracy of streamflow computations depends largely on the person who collects and edits ADR tapes and computer output. This section outlines a general procedure for automatic streamflow data processing and discusses five essential steps in the editing process (*14*). This procedure must be modified to meet specific needs and equipment capabilities and limitations.

The first step in a good edit program is to complete an inspection report, as shown in figure 2.66, each time the recorder is checked. Abnormalities in the recorder operation or in the record are noted on the reverse of this sheet. This form serves as a valuable permanent record when it is necessary to recheck past operation or to interpret some anomaly in flow. ADR's are to be inspected weekly. Five-minute and 15-minute tapes are changed each month and every 3 months, respectively. When a new tape is installed, the date, time, station identification, gage reading, and other identifying data should be written on the tape and initialed by the person performing the operation. The same information is written on the end of the tape being removed. At the beginning of each tape, eight punchouts are made manually before the starting time, without any skips between punches. These punches are needed to advance the translator to the proper beginning point for tape-to-card translation.

ADR tapes have punchout intervals that can vary from 2 minutes to 1 hour. The translator can be set to read each punchout on the tape or every second tape, fourth tape, sixth tape, and so forth. When streamflow rate is relatively steady, the translator can be set to read once each hour. The number of readouts punched on cards is thus reduced and translation is speeded considerably. Frequency of translation may be decided by referring to visual trace from a monitor watershed equipped with a chart recorder or by using a rain gage record. In this manner, 105,000 punchouts per year on the 5-minute tape are reduced to about 40,000. On the 15-minute tape, 35,000 punchouts are reduced to less than 20,000. Translator costs can be reduced considerably if this method is used rather than reading every point on the tape. Since these costs vary with machine time required for translation, tape should be edited to show the translation frequency desired.

The second step in a good edit program is to fill out and denote translation frequency (fig. 2.79) on a translator operator instructions form. Other remarks, such as missing data, also should be noted on this form. The tape editor must estimate the missing data and must attach a listing of time-head values (5-minute or hourly time intervals for each day of missing data) to this form. These data will be keypunched and inserted into the card deck during translation. Skipped sections of the tape and offset punches also should be noted with the time and date of each occurrence (head and time corrections can best be applied in later edit steps).

In addition to the translator operator instructions form, the tapes must be marked to show point of frequency change and the beginning and ending points of translation. The best method is to mark the tape with red and green felt-tipped pens (red for stop and green for start). The beginning point should be marked by a green line completely across the tape on the correct time line. When frequency changes are necessary, a red line is required at midnight of the day before the change, and a green line is required on the next point to read out (5-min, 15-min, or hourly line). The final point to be translated on the tape should be marked with a red line. Note: A complete day's record must be read at the same time interval; frequency of translation cannot be switched during a day's record. When tape editing is finished, the tape should be rewound so the beginning point is on the outside of the roll.

Translators such as Fischer and Porter will read three channels leaving off the units position, or they will read all four channels. Translator output may result in 1 card per day for 60-minute readouts, 4 cards for 15-minute readouts, or 12 cards for 5-minute readouts. The cards also contain watershed number, date, day of week, and card number within the day. The card format can be altered to fit the needs of the user. The card deck from the translator is edited in the third step.

The third edit step is done by the collector of the record either on the printed cards from the translator or on a printout of these cards. The card deck or printout is scanned for indications of anomalies caused by trash in the notch, wild

Tape Type 5 min.

Date May 1969 Gage No. Computation Code No.

Time	Frequency	Remarks
	5 min.	
	Hourly	
	"	
	"	
	"	Last Punch 1410 Missing Data Punched
	"	Break in Tape
	"	
	"	
	"	
	"	
	"	

FIGURE 2.79.—Translator operator instructions form.

values, incorrect translation points, incorrect number of days in month, and so forth. Head corrections are penciled onto the printout, and the printout or a listing of errors is returned to the translator. The card deck will be corrected by replacing erroneous cards with corrected cards (magnetic or paper tape output from the translator is also available).

After the translation step and edits, cards are processed through the condense and edit program, which performs four separate functions in preparing data for final discharge integration and computations. These functions are further reduction of points, insertions of unit's position digit in head readings, generation of time increments between remaining head readings, and computer edit of the data.

Head values are dropped systematically by the computer when the difference between readings is less than prescribed limits. These limits are set so that during storm periods every punchout needed to define the storm is retained. To be sure that the hydrograph will be defined adequately during periods of gradual change, however, a head reading is retained at the end of each 3-hour interval regardless of the number retained during the interval (if desired, all punchouts can be retained, but this adds to computing costs). This technique reduces the number of points retained to between 3,000 and 5,000 per year.

A digital computer accumulates time intervals and inserts the time of day in hours and minutes for each retained head reading. The computer also checks the cards for sequence, day of month, day of week, and year.

Output from this program is a set of time-head cards and a listing. These cards contain watershed number, date, and eight sets of time-head readings. The listing contains the same information on each printed line.

The fourth edit step uses the printout from the condensed program. This printout is returned to the user for editing. The listing is scanned for errors not detected by computer and previous edit steps. Corrections are penciled on the listing. Time and head corrections also can be applied at this time. Rain gages or adjacent units can be compared to check for errors. The listing is returned to the translator for corrections in the card deck. The cards are now ready for the discharge integration program.

The final step of routine streamflow data processing is programed for computer analysis. Formulas (preferably) or rating tables are used to convert head readings to discharge. Watershed areas also may be used to convert head readings directly to discharge rates in cubic feet per second per square mile (CSM). Computer programs may be used at the same time to determine discharge volume of the hydrograph, streamflow summaries, flow frequencies, and time—CSM coordinates. Programs will be written to provide specific information needed.

All computer output listings are returned to the user for a final check for errors. This is accomplished by comparing daily and monthly flows with those obtained from rainfall or nearby catchments. If discrepancies appear, the original records and all intermediate steps must be examined to determine if errors exist. If errors do exist, corrections should be penciled in for recomputation. If no errors occur after correction, all cards and printouts are ready for further use or for storage as permanent records of stream discharge.

Discharge Measurement Calculation

Obtain velocity values from a rating table (fig. 2.80) for the current meter and suspension used (rod, or 15, C - 0.5, and so forth). If shorter time intervals (20 to 35 seconds) are used, it is convenient to double the time and number of revolutions to obtain the velocity directly from the rating table. These values are listed in the "At point" column for each 0.2 and 0.8 depth observation of revolutions and time. For station 18, the velocity at 0.2 depth, taken from the rating table at 20 revolutions in 40 seconds, was 1.17 ft/s (35.7 cm/s); that for the 0.8 depth was 0.942. The average of these two values is 1.056—the mean velocity in the vertical. Since the current angle-correction factor was 0.98, the true mean velocity in the vertical is 0.98 times 1.056 = 1.035 ft/s (32.2 cm/s). As the 0.6-depth observation represents the mean velocity in the vertical, its value is recorded directly as the "Mean in Vertical." (See value for station 18, fig. 2.74.)

The area to which the "mean in vertical" velocity is applied is taken as the depth at the section, times a width extending halfway to the preceding and following observation points. Thus, the area at station 18 is equal to the depth since the stations on either side are a distance of 1 foot (30.5 cm). Discharge for each station is the product of the area times the velocity.

Total discharge (fig. 2.74) is the sum of that for each section; total area is the sum of all section areas. These totals are recorded in figure 2.81. This form can be filled in at the office after computing the measurement data. Area and discharge values are rounded off to the nearest three significant figures. Total discharge of 31.23 becomes 31.2 ft^3/s (0.94 m^3/s).

Gage height of the discharge measurement (fig. 2.74) usually is determined by averaging the gage height at the beginning and end of the measurement. A weighted mean gage height must be determined for discharge measurements where the change in stage during the measurement is several tenths of a foot or more, and for stream cross sections where a large part of the flow is measured in a small part of the total stream width (fig. 2.81). Total stream width would occur where there there is a wide overflow with low velocities and a narrow deep channel of high-velocity flow. For these streams, manual readings or gage-height values from the chart must be supplied several times during the measurement. These times are noted on the form illustrated in figure 2.74 at the point of velocity and depth observations. They are transferred later to the front sheet of this form (fig. 2.73). Using the times recorded in column one of figure 2.73, one can obtain gage-height values from the recorder chart (fig. 2.82). A method using a two-man team at alternate stations across a stream is reported by Blanchard and DeCoursey (*5*) for use on rapidly changing streams.

Stage-discharge relation is developed from current-meter measurements that are plotted (stage vs. discharge) on log-log paper (fig. 2.83).

It is usually desirable to plot the low-water section in this relationship separately for better definition. A best fit line is drawn through the plotted measurements.

High-stage measurements on small watersheds are difficult to obtain. Slope-area measurements or contracted opening measurements using flood peak high watermarks help to establish the upper end of the relation. These observations should be made as soon as possible after the flood.

With a permanent control at the runoff station, the stage-discharge relation should not shift. Once the relationship is established, occasional check measurements are made. Shifting of flow control due to backwater needs specific instructions.

RATING TABLE FOR TYPE Price CURRENT METER NO. SCS 13

Suspension Rod Rated 3-22, 1936

Index 437-416

Equations V=2.260R+0.040

Limits of Actual Rating 2.58 to 9.07 feet per sec. at Bureau of Standards, Wash., D.C.

Condition of meter New

No. SCS-13

Time in seconds	Velocity in feet per second for following revolutions: 3	5	7	10	15	20	25	30	40	Time in seconds
40	0.201	0.315	0.429	0.600	0.885	1.17	1.45	1.74	2.31	40
41	.197	.308	.419	.586	.864	1.14	1.42	1.70	2.25	41
42	.193	.301	.410	.573	.844	1.12	1.39	1.66	2.20	42
43	.189	.295	.401	.560	.825	1.09	1.36	1.62	2.15	43
44	.185	.289	.393	.548	.807	1.07	1.33	1.58	2.10	44
45	.182	.283	.385	.537	.790	1.04	1.30	1.55	2.06	45
46	.179	.278	.377	.526	.773	1.02	1.27	1.52	2.01	46
47	.176	.273	.370	.515	.758	1.00	1.24	1.49	1.97	47
48	.172	.268	.362	.505	.742	.980	1.22	1.45	1.93	48
49	.170	.263	.356	.495	.728	.961	1.19	1.43	1.89	49
50	.167	.258	.349	.486	.714	.942	1.17	1.40	1.85	50
51	.164	.254	.343	.477	.701	.924	1.15	1.37	1.82	51
52	.162	.249	.337	.468	.688	.907	1.13	1.34	1.78	52
53	.159	.245	.331	.460	.675	.890	1.11	1.32	1.75	53
54	.157	.241	.326	.452	.663	.874	1.09	1.30	1.72	54
55	.154	.237	.320	.445	.652	.859	1.07	1.27	1.69	55
56	.152	.234	.315	.437	.641	.844	1.05	1.25	1.66	56
57	.150	.230	.310	.430	.630	.830	1.03	1.23	1.63	57
58	.148	.227	.305	.423	.620	.816	1.01	1.21	1.60	58
59	.146	.223	.301	.416	.610	.803	.996	1.19	1.56	59
60	.144	.220	.296	.410	.600	.790	.980	1.17	1.55	60
61	.142	.217	.292	.404	.591	.778	.964	1.15	1.53	61
62	.140	.214	.387	.398	.582	.765	.949	1.13	1.50	62
63	.139	.211	.283	.392	.573	.754	.935	1.12	1.48	63
64	.137	.208	.279	.386	.564	.742	.921	1.10	1.45	64
65	.135	.205	.276	.381	.556	.732	.907	1.08	1.43	65
66	.134	.203	.272	.375	.548	.721	.894	1.07	1.41	66
67	.132	.200	.268	.370	.540	.711	.881	1.05	1.39	67
68	.131	.198	.265	.365	.533	.701	.868	1.04	1.37	68
69	.129	.195	.261	.360	.526	.691	.856	1.02	1.35	69
70	.128	.193	.258	.356	.519	.681	.844	1.01	1.33	70

Time in seconds	Velocity in feet per second for following revolutions: 50	60	80	100	150	200	250	300	350	Time in seconds
40	2.86	3.43	4.56	5.69	8.52	11.34	14.16	16.99	19.82	40
41	2.80	3.35	4.45	5.55	8.31	11.06	13.88	16.58	19.33	41
42	2.73	3.27	4.34	5.42	8.11	10.80	13.49	16.18	18.57	42
43	2.67	3.19	4.24	5.30	7.92	10.55	13.18	15.81	18.44	43
44	2.61	3.12	4.15	5.18	7.74	10.31	12.88	15.45	18.02	44
45	2.55	3.05	4.06	5.06	7.57	10.08	12.60	15.11	17.62	45
46	2.50	3.99	3.97	4.95	7.41	9.87	12.32	14.78	17.24	46
47	2.44	3.93	3.89	4.85	7.85	9.66	12.06	14.47	16.87	47
48	2.39	2.86	3.81	4.75	7.10	9.46	11.81	14.16	16.52	48
49	2.35	2.81	3.73	4.65	6.96	9.26	11.57	13.88	16.18	49
50	2.31	2.75	3.66	4.56	6.82	9.08	11.34	13.60	15.86	50
51	2.27	2.70	3.59	4.47	6.69	8.90	11.12	13.33	15.55	51
52	2.22	2.65	3.52	4.39	6.56	8.73	10.91	13.08	15.25	52
53	2.18	2.60	3.45	4.30	6.44	8.57	10.70	12.83	14.96	53
54	2.14	2.55	3.39	4.23	6.32	8.41	10.50	12.60	14.69	54
55	2.10	2.51	3.33	4.15	6.20	8.26	10.31	12.37	14.42	55
56	2.07	2.46	3.27	4.08	6.09	8.11	10.13	12.15	14.16	56
57	2.03	2.42	3.21	4.00	5.99	7.97	9.95	11.93	13.92	57
58	2.00	2.38	3.16	3.94	5.88	7.83	9.78	11.73	13.68	58
59	1.96	2.34	3.10	3.87	5.79	7.70	9.62	11.53	13.45	59
60	1.93	2.31	3.05	3.81	5.69	7.57	9.46	11.34	13.22	60
61	1.90	2.27	3.00	3.74	5.60	7.45	9.30	11.15	13.01	61
62	1.87	2.24	2.96	3.69	5.51	7.33	9.15	10.98	12.80	62
63	1.84	2.20	2.91	3.63	5.42	7.21	9.01	10.80	12.60	63
64	1.81	2.17	2.86	3.57	5.34	7.10	8.87	10.63	12.40	64
65	1.78	2.13	2.82	3.52	5.26	6.99	8.73	10.47	12.21	65
66	1.76	2.10	2.78	3.46	5.18	6.89	8.60	10.31	12.02	66
67	1.73	2.07	2.74	3.41	5.10	6.79	8.47	10.16	11.85	67
68	1.71	2.04	2.70	3.36	5.03	6.69	8.35	10.01	11.67	68
69	1.68	2.01	2.66	3.32	4.95	6.59	8.23	9.87	11.50	69
70	1.66	1.98	2.62	3.27	4.88	6.50	8.11	9.73	11.34	70

Computed by RH 3-3-37

Checked by AL 3-3-37

FIGURE 2.80.—Rating table for a current meter.

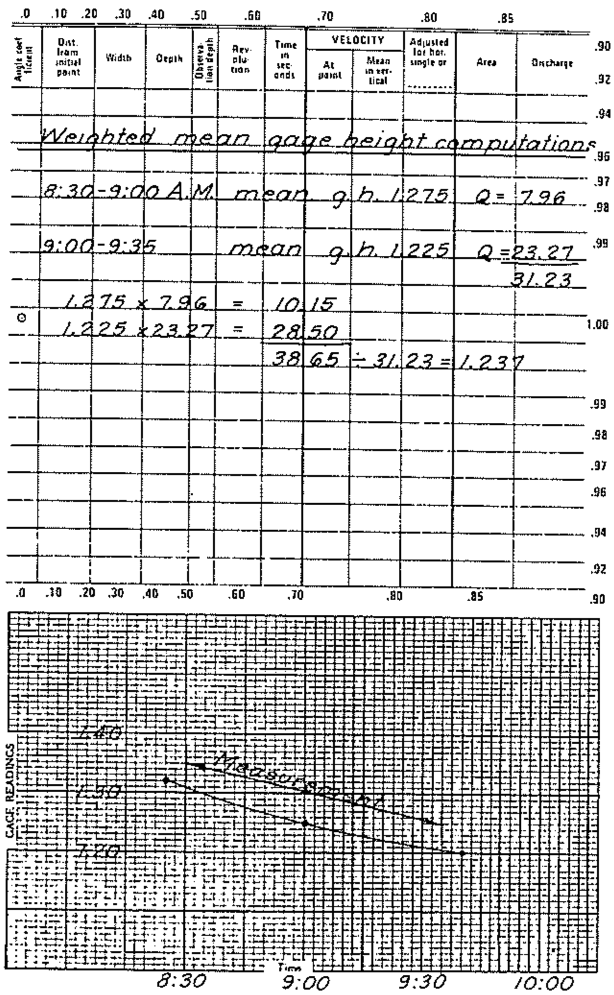


FIGURE 2.81.—Worksheet for computing mean gage height and curve.

The rating table for stage-discharge relation (fig. 2.84) is prepared from the graph sheet (fig. 2.83). Pick off points for each 0.1 foot or less from the graph line, such as 3.06 ft^3/s(0.09 m^3/s) at stage of 1.00 foot (30.48 cm); 4.48 ft^3/s (0.13 m^3/s) at 1.10 feet (33.5 cm), and so on. Tabulate these values on the rating table. As this table is set up in 0.01-foot (0.3 cm) intervals, interpolated values of cubic feet per second must be computed for those points within the selected graph values. This can be done by distributing the difference between consecutive graph values into the number of 0.01-foot (0.3 cm) intervals. For this table, the graph interval from 1.00 to 1.10 feet (30.5 cm to 33.5 cm) is 1.42 ft^3/s (0.04 m^3/s). If this interval is divided into 10 parts, each 0.01-foot (0.3 cm) stage charge will be 0.142 ft^3/s (0.004 m^3/s). The difference in cubic feet per second per 0.01-foot (0.3 cm) stage is not uniform throughout the range from the 1.00 to 1.10-foot (30.5 cm to 33.5 cm) stage. The increment of 0.142 ft^3/s (0.004 m^3/s) per 0.01-foot (0.3 cm) stage applies to the center of the overall reach; that is, at about 1.15-foot (35.1 cm) stage (figure 2.83). Increment cubic feet per second values for the reach 1.00- to 1.03-foot (30.5 cm to 31.4 cm) stages are smaller, and those from 1.06 to 1.10 (32.3 to 33.5 cm) are larger than 0.142 ft^3/s (0.004 m^3/s). In making the rating table differences in ft^3/s for the 0.01-foot (0.3 cm) stage, changes are to vary uniformly and smoothly without abrupt changes. Streambed flow can be converted from cubic feet per second to units of watershed inches per hour.

Missing Records

The greatest cause of missing records is usually failure of the observer to wind the clock, ink the pen, place the pen on the paper, tighten the float pulley after adjustment, flush the intake pipes, change the clock on schedule, or adjust the paper properly in the recorder. Training the observer and providing adequate

COMPUTATION OF WEIGHTED MEAN GAGE HEIGHT

Time	Gage height		Discharge in intervals	Gage height times discharge
	Reading	Average		
	Feet	*Feet*	*Ft^3/s*	
8:32 a.m.	4.26			
8:42	4.12	4.19	3.46	14.50
8:57	3.92	4.02	26.72	107.41
9:17	3.62	3.77	15.60	58.81
9:32	3.42	3.52	5.22	18.37
Totals			51.00	199.09

Weighted mean gage height: 199.09 ÷ 51.00 = 3.90 ft.

FIGURE 2.82.—Method of computing weighted mean gage height.

checking forms are excellent means of preventing missing records.

When records are lost for any reason, take the following steps:

- Determine high watermarks at the gage site for the period.
- Examine runoff records from the nearest station to establish probable trends in the missing data.
- Examine precipitation and temperature records from the nearest stations.
- Correlate past runoff records with precipitation and runoff from nearby stations.
- Adjust data by an analysis of hydrograph recessions for similar seasonal events of record.
- Further adjust the probable hydrograph by an equation that accounts for difference in watershed retention.

Estimating Recessions

A visit to the runoff station should be scheduled immediately following or during any significant runoff event to prevent long periods of missing records caused by silting of intakes and instrument malfunctions. This action may provide a few valuable data points for fitting the recession curve.

The recession for a watershed may be plotted by comparing it with recessions from other storms on the same watershed. A template matched to a series of recession curves will provide a reasonable estimate of the missing data. This overlay may be prepared as follows:

- Search the runoff records and select flows that have complete recessions and are not influenced by precipitation (elongated). Use flows of similar gage height.

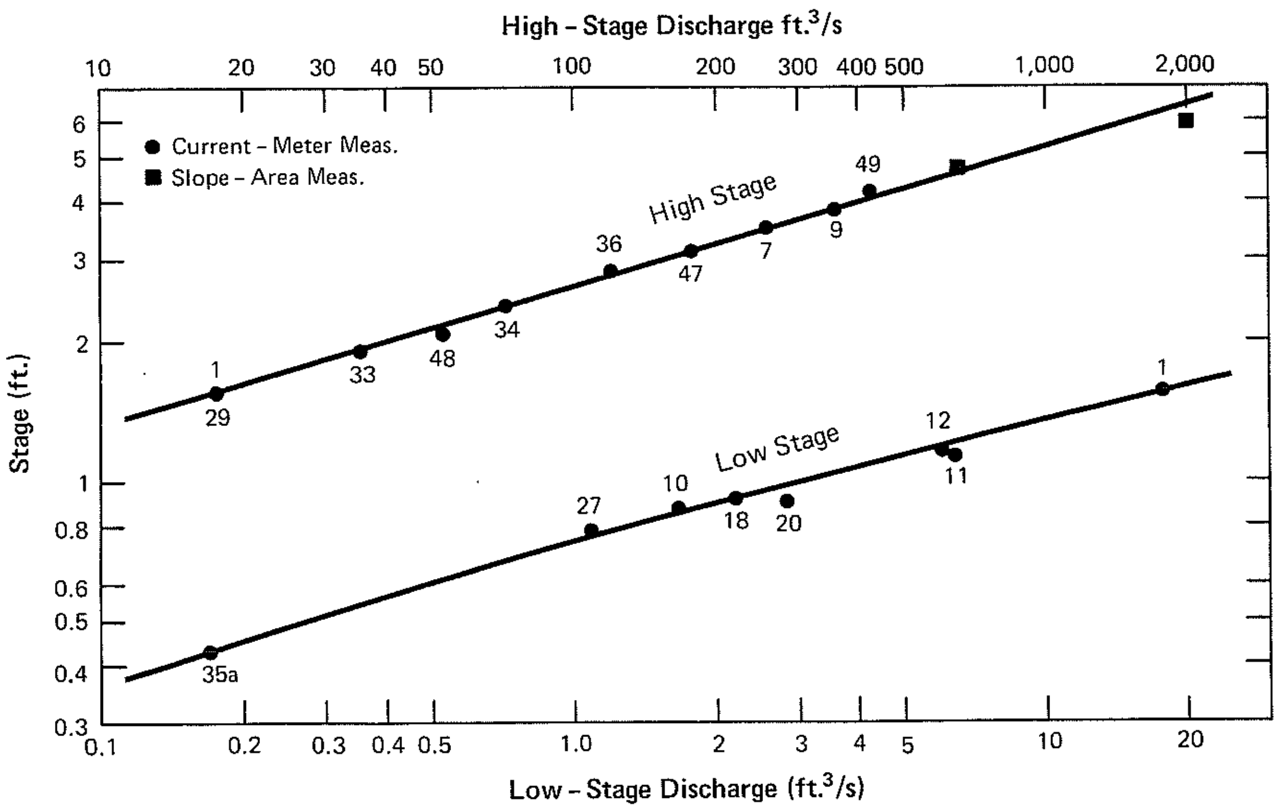


FIGURE 2.83.—Low-water stage and high-water stage discharge relation developed from current meter measurements.

Rating table for

Crooked River nr Central, **Dated** October 10, 19 48

from June 1, 1946 *to* October 10, 1948; *from* ______ *to* ______

from ______ *to* ______; *from* ______ *to* ______

Gage height	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09	Difference
Feet	*Cfs*	*Cfs*	*Cfs*	*Cfs*	*Cfs*	*Cfs*	*Cfs*	*Cfs*	*Cfs*	*Cfs*	*Cfs*
0.0											
.1											
.2					0.00	0.01	0.01	0.02	0.03	0.03	0.04
.3	0.04	0.05	0.05	0.06	.07	.08	.09	.10	.11	.12	.09
.4	.13	.14	.15	.17	.18	.19	.21	.23	.24	.26	.14
.5	.27	.29	.31	.33	.35	.37	.39	.41	.44	.46	.22
.6	.49	.52	.55	.58	.61	.64	.67	.70	.73	:76	.31
.7	.80	.84	.89	.93	.98	1.03	1.09	1.14	1.20	1.26	.52
.8	1.32	1.38	1.45	1.52	1.60	1.68	1.76	1.84	1.92	2.00	.76
.9	2.08	2.17	2.27	2.37	2.47	2.58	2.68	2.79	2.90	3.01	1.05
1.0	3.13	3.26	3.40	3.53	3.66	3.80	3.94	4.09	4.24	4.39	1.41
.1	4.54	4.71	4.89	5.09	5.29	5.40	5.59	5.78	5.97	6.16	1.81
.2	6.35	6.58	6.81	7.04	7.27	7.50	7.74	7.98	8.22	8.46	2.35
.3	8.70	8.95	9.20	9.46	9.73	10.0	10.3	10.6	10.9	11.2	2.80
.4	11.5	11.8	12.2	12.5	12.9	13.2	13.6	13.9	14.3	14.6	3.50
.5	15.0	15.4	15.8	16.2	16.6	17.0	17.4	17.8	18.2	18.6	4.00
.6	19.0	19.4	19.8	20.2	20.6	21.0	21.4	21.9	22.3	22.7	4.20
.7	23.2	23.7	24.1	24.6	25.0	25.5	26.0	26.5	27.0	27.5	4.80
.8	28.0	28.5	29.1	29.6	30.1	30.7	31.3	31.9	32.4	33.0	5.60
.9	33.6	34.2	34.8	35.4	36.0	36.7	37.4	38.0	38.6	39.3	6.40
2.0	40.0	40.7	41.3	41.9	42.6	43.3	44.0	44.8	45.5	46.2	7.00
.1	47.0	47.7	48.4	49.1	49.8	50.6	51.5	52.4	53.2	54.1	8.00
.2	55.0	55.9	56.7	57.6	58.4	59.3	60.2	61.1	62.1	63.0	9.00
.3	64.0	64.9	65.8	66.7	67.6	68.5	69.5	70.5	71.5	72.5	9.50
.4	73.5	74.5	75.5	76.5	77.5	78.5	79.6	80.7	81.8	82.9	10.5
.5	84.0	85.1	86.3	87.5	88.7	90.0	91.2	92.4	93.6	94.8	12
.6	96.0	97.3	98.6	99.9	101	102	104	105	106	108	13
.7	109	110	112	113	115	116	117	119	120	122	14
.8	123	125	126	128	129	131	132	133	135	136	15
.9	138	140	141	143	144	146	148	149	151	153	17

Computed by ABC 10/6/19 48; *Checked by* DEF 10/10/19 48 *Remarks* ______

FIGURE 2.84.—Rating table for stage discharge relation.

- Tabulate these flows and plot the recessions ($\log_{10}$ discharge vs time) on semilog paper. The curves are then superimposed so that an average curve could be chosen to represent the standard recessions curve. After superimposing several recession records, draw a straight line through the lower portion of the curves to extend the recession.
- Transfer the curve to be used for the standard recession from discharge into stage height readings. Plot this curve on a typical recorder chart and draw a line through the points to form the standard recession curve.
- Make a mylar overlay to place on the charts for easy appraisal where the standard recession curve should be applied if needed. Since each runoff is a unique event, this curve is used for estimation only. The recessions for a watershed vary according to moisture, seasons of the year, and changes in vegetation cover.

Analytical methods also can be used to estimate missing stage records. Polynomial equations can be used when the beginning and ending time and stage record are known, as was demonstrated by Mills and Snyder (*27*).

High Watermarks

Flood debris lines on walls, stilling wells, streambanks, and vegetation are valuable as a check on maximum recorded gage height and as a singular estimate of the peak discharge rate. Correlation of peak runoff rate with flood volume for similar events may provide much useful information in estimating runoff volumes for periods of missing or erroneous records. At peak stage in a pond the instantaneous runoff rates of inflow and outflow are used without pondage correction.

Comparison With Runoff or Retention From Nearby Watersheds

In comparing runoff from two watersheds, select periods that will not be affected by the conditions of cover and tillage. First, select a series of storms common to both areas and make sure each area has about the same cover conditions for all storms. This does not imply that both areas must be in the same cover. For example, an area could be 100 percent in hay crops and still obtain a comparable result. This method is best adapted to small adjacent areas and involves plotting runoff from one watershed against runoff from the other, for all storms selected, to develop an average relation.

Figure 2.85 illustrates the method applied to two small watersheds: W-I, 27.2 acres (11 ha) and W-II, 50 acres (20.2 ha) near Edwardsville, Ill. Watershed W-II was in 100 percent pasture during the entire period of record. Watershed W-I was in 100 percent alfalfa except for 4 years of record, when 96 percent was cultivated. The relationship when W-I was 100 percent alfalfa differed from the relationship pertaining to W-II when W-I was cultivated. Land use

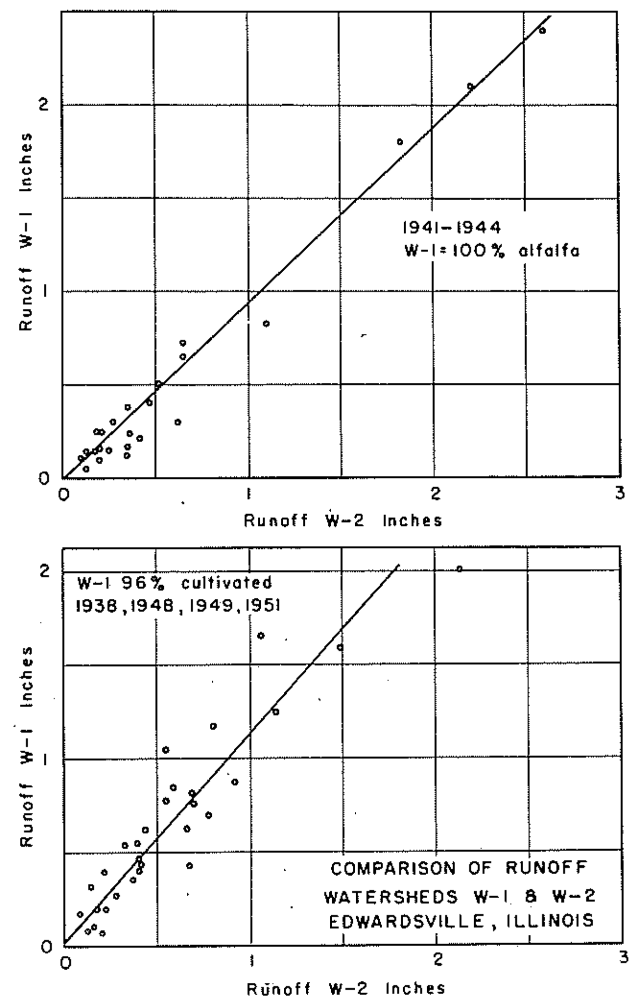


FIGURE 2.85.—Comparison of runoff from two adjoining watersheds as an aid in estimating data of missing records, Edwardsville, Ill.

during the period of missing record must be considered in selecting the proper curve for estimating total runoff from one watershed, based upon its relationship to the other watershed. In these comparisons, different relations may exist for different seasons.

This method may not produce satisfactory results for some areas because rainfall varies between the areas being compared. These results are illustrated in figure 2.86, which is a comparison of two areas near Fennimore, Wis. Both areas have approximately the same shape and topography, but they vary considerably in size—W-2 is 22.8 acres (9.2 ha) and W-4 is 171 acres (69.2 ha). During 1942, 1943, 1950, and 1951, 0 to 7 percent of W-2 was cultivated and 45 to 50 percent of W-4 was cultivated. During the years 1944 and 1945, 60 percent of both watersheds was cultivated.

As shown in the lower graph of figure 2.86, land use affects the runoff relationship between these two watersheds, but scatter values during the 1944-45 period were not great enough to establish the curve. For these watersheds, a better relationship is obtained by comparing amounts of retention (that is, rainfall minus runoff) as in the upper graph of figure 2.86. Retention on the area for the missing record can be estimated by finding the retention on the area used for comparison, entering it on the chart, and reading the retention on the area in question. Estimated retention is subtracted from rainfall to estimate runoff.

Size of the drainage areas used in the preceding comparison need not be the same. In fact, drainage areas can be considerably different in size if all runoff is surface runoff resulting from precipitation in excess of infiltration rates, and if no base flow or interflow occurs.

In areas where the geology is such that streams are classed as gaining or losing or where the base flow or interflow represents a considerable part of the total runoff, it may become necessary to compare only watersheds that are approximately the same size. This method also may be useful in estimating peak rates of runoff. Maximum rates of runoff from the watershed are plotted, storm by storm, against maximum rates of runoff from a comparable watershed, to derive a curve for estimating missing records of peak flow for a given

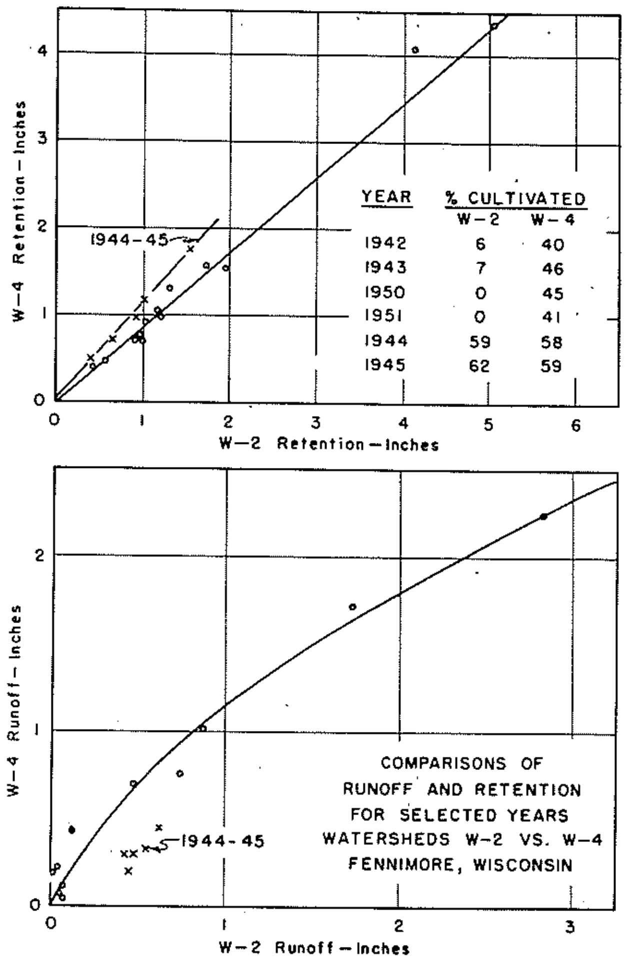


FIGURE 2.86.—Comparison of runoff and moisture retention on similar watersheds as a method of estimating data of missing records, Fennimore, Wis.

storm. As shown for total runoff, a comparison of retention rates of one area against another may give more consistent results.

Estimate of Flow During Ice Periods

Estimation of flow during ice periods is a complex problem at natural channel sections. Detailed site surveys are required to determine the area, wetter perimeter, roughness coefficients of the bed, and covering ice layer and water surface slope (*10*). At best, estimates usually apply only for short periods since an increase or decrease in rate of flow floods the ice layer or causes the ice to form a bridge over the flowing water. Frequent observations are necessary for reliable estimates of flow.

Flow at weirs and flumes often can be estimated by periodically removing the ice layer upstream and downstream from the control section. Higher velocity through the control often prevents serious ice formations. At locations where ice is a continuing problem, the weir or flume should be covered by a heated enclosure capable of resisting snow and wind loads expected in the area (fig. 2.87). Electric heating cables attached to the bottom and stilling well of metal flumes or buried in concrete, help to reduce the effect of ice on the runoff record.

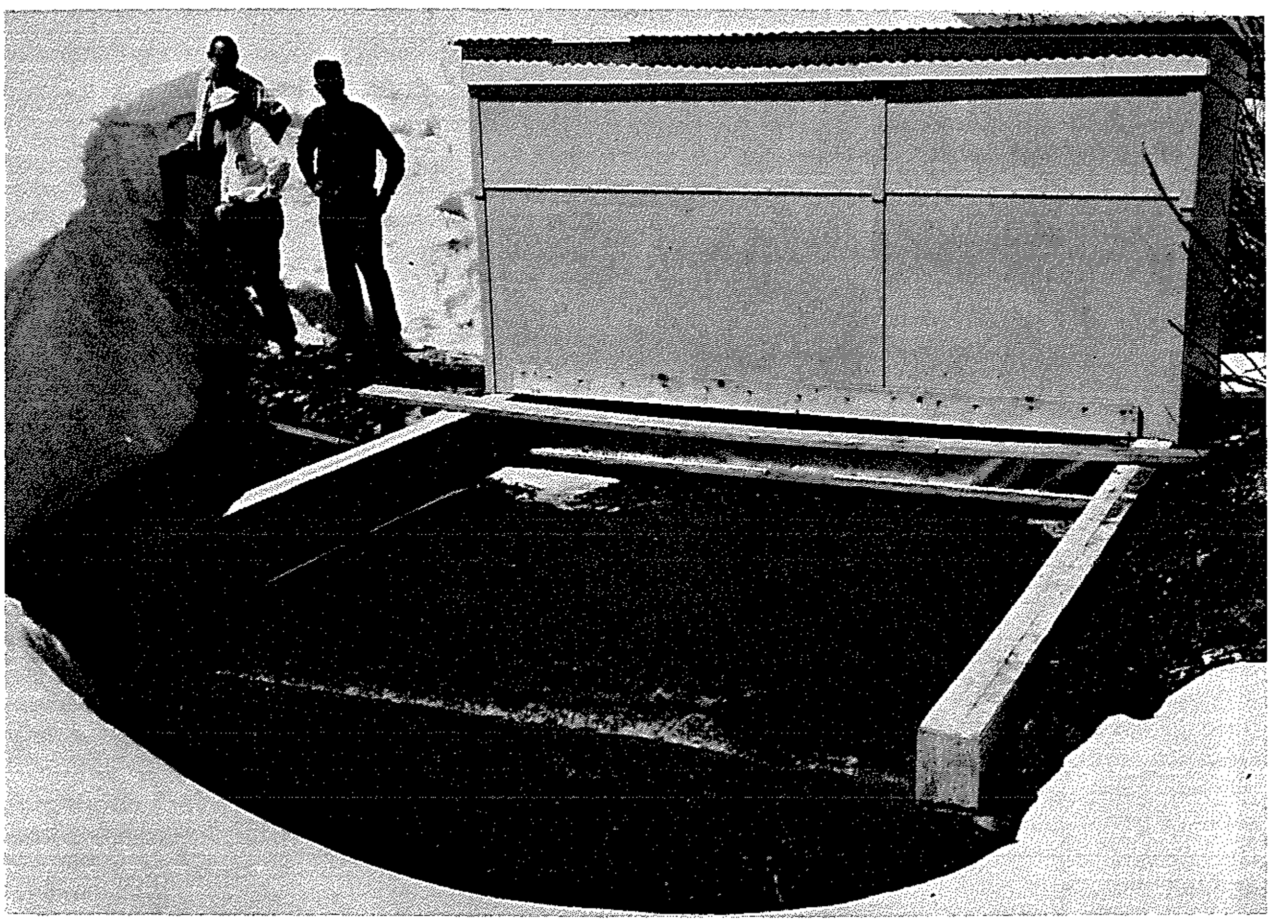
Manual Calculations

Manual calculations without pondage corrections are fairly simple using a desk calculator. For pondage or a large pond requiring subtraction for precipitation on the pond, calculations must be made carefully of runoff.

Calculations for Pondage Corrections

The area of temporary pondage above the weir on the 50-acre watershed at Edwardsville, Ill., ranges from 544 ft^2 (50 m^2) at notch elevation, G.D.E. 1.50, to 33,000 ft^2 (0.14 to 2970 m^2), or 1.5 percent of the drainage area at maximum design stage, G.D.E. 5.20. The rating table for this station, prepared in accordance with instructions (*42*) is shown in figure 2.88. Follow these instructions:

- Enter data in columns 1 and 3 in figure 2.89. Note that the difference in stage between



PN-5890

FIGURE 2.87.—Typical weir enclosure for ice conditions.

Index: Edwardsville, Ill. W-II RT-1

RATING TABLE FOR STATION W-II (LOVE) EDWARDSVILLE, ILL.

G.D.E. = Gage Datum Elevation

G.D.E. of zero flow over weir = 1.50

Reference: Drawing No. 5-S-5069-D

G.D.E. of original point of zero pondage = 1.0

G.D.E. of Lowest Intake = 1.10

Cross sections taken June 1938

Observed Discharge over Weir (q_w) (3:1 Side Slopes) — Inches per Hour for each 0.01 ft. of Stage

Stage G.D.E. (feet)	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
1.10	--	--	--	--	--	--	--	--	--	--
1.20	--	--	--	--	--	--	--	--	--	--
1.30	--	--	--	--	--	--	--	--	--	--
1.40	--	--	--	--	--	--	--	--	--	--
1.50	--	--	--	--	--	.0001	.0001	.0002	.0003	.0003
1.60	.0005	.0006	.0008	.0009	.0011	.0014	.0016	.0018	.0021	.0024
1.70	.0027	.0031	.0034	.0038	.0043	.0047	.0052	.0057	.0063	.0068
1.80	.0074	.0080	.0087	.0094	.0101	.0109	.0117	.0125	.0134	.0143
1.90	.0152	.0162	.0172	.0183	.0193	.0207	.0216	.0228	.0240	.0254
2.00	.0268	.0282	.0296	.0310	.0326	.0341	.0357	.0373	.0391	.0409
2.10	.0427	.0445	.0465	.0484	.0504	.0524	.0546	.0568	.0588	.0611
2.20	.0633	.0657	.0681	.0706	.0732	.0758	.0784	.0812	.0840	.0868
2.30	.0897	.0927	.0956	.0986	.102	.105	.108	.111	.115	.118
2.40	.122	.125	.129	.133	.136	.140	.144	.148	.151	.155
2.50	.159	.163	.168	.172	.176	.181	.185	.190	.194	.201
2.60	.204	.210	.214	.220	.222	.228	.232	.238	.242	.248
2.70	.252	.258	.264	.268	.274	.280	.286	.292	.296	.302
2.80	.308	.314	.320	.325	.331	.337	.343	.349	.355	.361
2.90	.367	.375	.383	.391	.399	.407	.415	.423	.431	.439
3.00	.446	.455	.463	.471	.479	.486	.494	.502	.510	.518
3.10	.526	.534	.542	.550	.558	.566	.576	.586	.596	.606
3.20	.616	.625	.635	.645	.655	.665	.675	.685	.695	.705
3.30	.715	.724	.734	.744	.754	.764	.776	.788	.800	.812
3.40	.824	.836	.848	.860	.872	.883	.895	.907	.919	.931
3.50	.943	.954	.967	.978	.990	1.01	1.02	1.03	1.04	1.05
3.60	1.07	1.08	1.10	1.11	1.12	1.14	1.15	1.17	1.18	1.19
3.70	1.21	1.22	1.24	1.25	1.26	1.27	1.29	1.30	1.32	1.33
3.80	1.35	1.36	1.38	1.40	1.41	1.43	1.44	1.46	1.48	1.49
3.90	1.51	1.52	1.54	1.56	1.58	1.59	1.61	1.63	1.65	1.66
4.00	1.68	1.70	1.72	1.74	1.75	1.77	1.79	1.81	1.82	1.84
4.10	1.86	1.88	1.90	1.91	1.93	1.95	1.97	1.98	2.01	2.03
4.20	2.04	2.07	2.09	2.10	2.12	2.14	2.16	2.18	2.20	2.22
4.30	2.24	2.26	2.28	2.30	2.32	2.36	2.38	2.40	2.42	2.44
4.40	2.46	2.48	2.50	2.54	2.56	2.58	2.60	2.62	2.66	2.68
4.50	2.70	2.72	2.74	2.78	2.80	2.82	2.84	2.86	2.90	2.92
4.60	2.94	2.96	3.00	3.02	3.04	3.08	3.10	3.12	3.14	3.18
4.70	3.20	3.22	3.25	3.27	3.29	3.33	3.35	3.37	3.39	3.43
4.80	3.45	3.47	3.51	3.53	3.57	3.59	3.61	3.65	3.67	3.71
4.90	3.73	3.77	3.79	3.83	3.85	3.89	3.91	3.95	3.97	4.01
5.00	4.03	4.07	4.09	4.11	4.13	4.17	4.19	4.23	4.25	4.29
5.10	4.31	4.35	4.37	4.41	4.43	4.47	4.51	4.55	4.57	4.59
5.20	4.63									

Pondage Correction (q_p) — Inches per Hour for each 0.01 ft. of Stage For a Rate of Change in Stage of 1 ft. per Minute

Stage G.D.E. (feet)	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
1.10	.0228	.0242	.0255	.0270	.0284	.0298	.0324	.0349	.0375	.0401
1.20	.0427	.0453	.0478	.0504	.0530	.0556	.0580	.0600	.0627	.0651
1.30	.0675	.0698	.0722	.0746	.0770	.0794	.0824	.0854	.0883	.0913
1.40	.0942	.0972	.100	.103	.106	.109	.128	.147	.166	.185
1.50	.202	.222	.242	.260	.278	.298	.321	.347	.373	.397
1.60	.421	.447	.472	.496	.520	.546	.572	.596	.619	.645
1.70	.671	.695	.718	.744	.770	.794	.820	.845	.871	.897
1.80	.923	.948	.974	1.00	1.03	1.05	1.08	1.10	1.12	1.15
1.90	1.17	1.20	1.22	1.24	1.27	1.29	1.32	1.34	1.37	1.39
2.00	1.42	1.45	1.47	1.50	1.52	1.55	1.57	1.60	1.62	1.65
2.10	1.67	1.70	1.72	1.75	1.77	1.80	1.82	1.85	1.87	1.90
2.20	1.92	1.95	1.97	1.99	2.03	2.05	2.06	2.08	2.12	2.14
2.30	2.16	2.18	2.20	2.24	2.26	2.28	2.30	2.34	2.36	2.38
2.40	2.42	2.44	2.46	2.48	2.52	2.54	2.58	2.60	2.64	2.68
2.50	2.70	2.74	2.78	2.82	2.84	2.88	2.92	2.96	3.00	3.04
2.60	3.08	3.12	3.15	3.19	3.23	3.27	3.31	3.35	3.37	3.41
2.70	3.45	3.49	3.53	3.55	3.59	3.63	3.67	3.71	3.73	3.77
2.80	3.81	3.85	3.89	3.91	3.95	3.99	4.03	4.05	4.09	4.13
2.90	4.17	4.19	4.23	4.27	4.29	4.33	4.37	4.39	4.43	4.45
3.00	4.49	4.53	4.55	4.59	4.60	4.64	4.68	4.70	4.74	4.76
3.10	4.80	4.84	4.86	4.90	4.92	4.96	5.00	5.02	5.06	5.08
3.20	5.12	5.16	5.18	5.22	5.24	5.28	5.32	5.34	5.38	5.40
3.30	5.44	5.48	5.50	5.54	5.56	5.60	5.64	5.66	5.68	5.72
3.40	5.74	5.78	5.80	5.84	5.88	5.90	5.94	5.96	6.00	6.02
3.50	6.05	6.09	6.11	6.15	6.17	6.21	6.23	6.27	6.31	6.33
3.60	6.35	6.39	6.43	6.45	6.47	6.51	6.55	6.57	6.59	6.63
3.70	6.67	6.69	6.71	6.75	6.79	6.81	6.83	6.87	6.89	6.93
3.80	6.95	6.97	7.01	7.03	7.05	7.09	7.11	7.13	7.17	7.21
3.90	7.22	7.24	7.28	7.30	7.34	7.36	7.38	7.42	7.44	7.48
4.00	7.50	7.52	7.56	7.58	7.62	7.64	7.66	7.70	7.72	7.74
4.10	7.78	7.80	7.82	7.84	7.88	7.90	7.92	7.96	7.98	8.00
4.20	8.02	8.06	8.08	8.10	8.14	8.16	8.18	8.20	8.24	8.26
4.30	8.28	8.30	8.32	8.36	8.38	8.40	8.42	8.44	8.48	8.50
4.40	8.52	8.54	8.56	8.60	8.62	8.64	8.66	8.68	8.72	8.74
4.50	8.76	8.78	8.80	8.84	8.86	8.88	8.90	8.94	8.98	9.00
4.60	9.01	9.05	9.09	9.11	9.13	9.17	9.21	9.23	9.27	9.29
4.70	9.33	9.37	9.39	9.43	9.45	9.49	9.53	9.57	9.59	9.63
4.80	9.67	9.71	9.75	9.77	9.81	9.85	9.89	9.93	9.97	10.01
4.90	10.05	10.07	10.11	10.15	10.2	10.2	10.3	10.3	10.3	10.4
5.00	10.4	10.4	10.5	10.5	10.6	10.6	10.6	10.6	10.7	10.7
5.10	10.7	10.7	10.7	10.8	10.8	10.8	10.8	10.9	10.9	10.9
5.20	10.9									

FIGURE 2.88.—Discharge values for 3-to-1 weir and pondage correction data for station W-II, Edwardsville, Ill.

Location Edwardsville, Illinois
File No. W-2-1952-17
Chart No. 1105
W-2 (Love)
(Station designation)
Drainage area 50 acres .... square miles
Dates: June 11 to July 3, 1952
Rating table: RT-1-1938

Gage datum elevation of point of zero flow 1.50 feet
Gage datum elevation of point of zero pondage 1.0 feet
Gage datum elevation of "float at rest" .... feet
Gage datum elevation of lowest intake 1.20 feet

RECORD OF RUN-OFF

Monroe-Matic Calculator

Date and time (1)	Time interval (2)	Gage height (3)	Rate of change in stage (4)	Pondage correction (5)	Observed discharge (6)	Rate of run-off (Discharge corrected for pondage) (7)	(8)	Average rate of run-off for time interval (9)	Total run-off: For time interval (10)	Accumulated (11)	Remarks (12)
Hr and min	Minutes	Feet	Feet per min	In/hr	In/hr	Second-feet	In/hr	Time Factor	Inches	Inches	
			No Runoff June 11 to July 2, 1952								
July 2 p.m.											
11:15		1.32	0	0	0		0			0	Beginning of Rise
:18	3	1.40	.0333	.0041	0		.0041	.025		.0001	
:21	3	1.52	.0550	.0133	0		.0133	.025		.0005	
:24	3	1.73	.0952	.0708	.0038		.0746	.025		.0027	
:27	3	2.09	.1085	.1790	.0409		.2199	.025		.0101	
:30	3	2.38	.1158	.273	.115		.3880	.025		.0253	
:32	2	2.65	.1575	.515	.228		.7430	.017		.0445	
:33	1	2.83	.1750	.685	.325		1.010	.008		.0585	
:34	1	3.00	.1850	.830	.446		1.276	.008		.0768	
:35	1	3.20	.2000	1.024	.616		1.640	.008		.1002	
:36	1	3.40	.2000	1.148	.824		1.972	.008		.1291	
:37	1	3.60	.2100	1.333	1.070		2.403	.008		.1641	
:38	1	3.82	.1900	1.333	1.38		2.713	.008		.2050	Peak Discharge
:39	1	3.98	.1250	.930	1.65		2.580	.008		.2473	
:40	1	4.07	.0850	.655	1.81		2.465	.008		.2877	
:41	1	4.15	.0650	.514	1.95		2.465	.008		.3271	
:42	1	4.20	.0350	.281	2.04		2.321	.008		.3654	
:43	1	4.22	0	0	2.09		2.090	.008		.4007	Maximum Stage
:45	2	4.21	-.0175	-.141	2.07		1.929	.017		.4690	
:48	3	4.12	-.0350	-.274	1.90		1.626	.025		.5579	
:51	3	4.00	-.0533	-.400	1.68		1.280	.025		.6306	
:54	3	3.80	-.0600	-.417	1.35		.933	.025		.6859	
:57	3	3.64	-.0550	-.356	1.12		.764	.025		.7283	
12 md't	3	3.47	-.0518	-.307	.907		.600	.025		.7624	Total July 2 = .7624"
July 3 a.m.											
12:06	6	3.19	-.0409	-.208	.606		.398	.050		.8123	
:12	6	2.98	-.0308	-.136	.431		.295	.050		.8470	
:18	6	2.82	-.0234	-.091	.320		.229	.050		.8732	
:24	6	2.70	-.0192	-.066	.252		.186	.050		.8939	
:30	6	2.59	-.0186	-.057	.201		.144	.050		.9104	
:40	10	2.42	-.0145	-.0357	.1290		.0933	.083		.9301	
:50	10	2.30	-.0110	-.0238	.0897		.0659	.083		.9433	
1:00	10	2.20	-.0100	-.0192	.0633		.0441	.083		.9524	
:10	10	2.10	-.0090	-.0150	.0427		.0277	.083		.9584	
:20	10	2.02	-.0070	-.0103	.0296		.0193	.083		.9623	
:40	20	1.90	-.0052	-.0061	.0152		.0091	.167		.9670	
2:00	20	1.81	-.0038	-.0036	.0080		.0044	.167		.9693	
:20	20	1.75	-.0027	-.0021	.0047		.0026	.167		.9705	
:40	20	1.70	-.0025	-.0017	.0027		.0010	.167		.9711	
3:00	20	1.65	-.0019	-.0010	.0014		.0004	.167		.9713	
:30	30	1.61	-.0012	-.0005	.0006		.0001	.250		.9714	
4:00	30	1.58	-.0007	-.0003	.0003		0	.250		.9715	End of Runoff
5:00	60	1.55					0				Total July 3 = .2091"
							33.0298	2.372			

Conversion factor for drainage area: 1 cubic foot per second .... inches per hour.

Tabulated by John Jones 9/12/52 (Date) Checked by George Scott 9/12/52 (Date)

Computed by John Jones 9/13/52 (Date) Checked by Kenneth Draner 10/16/52 (Date)

Sheet ...... of ...... sheets

FIGURE 2.89.—Computations performed by using a desk calculator for pondage corrections of runoff at Edwardsville, Ill.

successive time intervals is approximately 0.2 foot (6.1 cm).

- Enter in column 2 the difference in minutes between successive entries in column 1.
- Determine the rate of change in stage in column 4. The theoretical rate of change in stage at a point would be the tangent to the stage graph at that point. As the chart is curvilinear, the stage graph must be plotted on rectangular coordinate paper and the slope of the tangent must be determined at each tabulated point with a straightedge or template. Although it may require more points for accuracy, a satisfactory method is to determine the difference between gage heights corresponding to equal time intervals preceding and succeeding the point under consideration. Divide this difference by the total time interval between the gage heights. This gives the slope of the chord between two points immediately before and after the time in question. If the times between stage intervals are sufficiently small, the chord is approximately parallel to the tangent.
- Computations of the slope of the chords that are taken as the rates of change in stage may be simplified by use of a table of half rates of ponding (fig. 2.90). The rate of change in stage is obtained by adding the half rates for the interval before and after the time in question. Half rates for stage increments in excess of 0.10 foot can be interpolated by adding the half rate of the pertinent hundredth foot stage increment to that of the pertinent tenth foot increment. Thus, the half rate for the stage increment of 0.27 foot (0.20+0.07) in 2 minutes preceding 11:32 p.m. in figure 2.90 is computed as the sum of 0.050+0.0175=0.0675 ft/min. The half rate for the stage increment of 0.18 foot (0.10+0.08) in 1 minute following 11:32 p.m. is 0.0500+0.0400=0.0900. The tangent or rate of pondage at 11:32 p.m. is equal to the sum of the half rates: 0.0675+0.0900=0.1575 ft/min (4.72 cm/m). If the time between stages is more than 1 hour, the rate of change in stage normally will be so small that it may be disregarded. The rate of change in stage is positive for rising stages and negative for falling stages. At peaks and troughs of the pen trace (maximum-minimum gage heights) the rate of change in stage is always zero.
- The pondage correction in column 5 of figure 2.89 is obtained from the right side of the rating table (fig. 2.88). In using this part of the table, determine the value q_p for any rate of change in stage at a given gage height by multiplying the tabular value for the gage height by the rate of change in stage. Thus, at 11:32 the G.D.E. is 2.65, and the rate of change in stage is 0.1575 ft/min (4.72 cm/m); from the rating table, the G.D.E. 2.65 q_p is equal to 3.27 in/hr (8.3 cm/h) and the rate of ponding is 0.1575×3.27=0.515 in/hr (1.3 cm/h). The sign of the pondage correction is positive for rising stages and negative for falling stages.
- Obtain values of observed discharge q_w in column 6 directly from the rating table for each tabulated gage height.
- These entries are the discharges corrected for pondage or the algebraic sums of entries in columns 5 and 6. Carry the entries in column 8 to three significant figures, but never more than four decimal places.
- Obtain column 9 by averaging column 8 for the prior and current discharge. Obtain column 10 by multiplying column 9 by the time interval of column 2 and dividing by 60 minutes. Obtain column 11 by accumulating column 10. At the time of the peak rate of runoff in column 8, the pondage correction for this storm was almost equal to the discharge over the weir.

On areas with large ponds, apply corrections for ponding and for rain falling on the surface of the pond. The area of the pond at elevation of the spillway station at McCredie, Mo., is about 16 acres, or almost 10 percent of the drainage area. The drainage area in this instance is variable, depending on the stage of the pond; thus, a rating table cannot be made directly in inches per hour. The chart from this station for the period from June 6 to June 7, 1945, is shown in figure 2.91.

- Plot the stage graph G.D.E. versus time on cross-section paper, using an expanded time scale, and draw a smooth curve through the plotted points (fig. 2.92). Use large scales for these graphs, as they will give greater accuracy to the final results. Plot the graph only for that part of the runoff during which rainfall occurred, that is, 2:50 a.m. to 7:00 a.m. on June 7.
- Plot the accumulated rainfall diagram di-

Time-interval (minutes)	Half rates of ponding during time-interval for change of stage of- (Ten-thousandths of a foot per minute)													
	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09	0.10	0.20	0.30	0.40	0.50
1	50	100	150	200	250	300	350	400	450	500	1000	1500	2000	2500
2	25	50	75	100	125	150	175	200	225	250	500	750	1000	1250
3	17	33	50	67	83	100	117	133	150	167	333	500	667	833
4	12	25	38	50	62	75	88	100	112	125	250	375	500	625
5	10	20	30	40	50	60	70	80	90	100	200	300	400	500
6	8	17	25	33	42	50	58	67	75	83	167	250	333	417
7	7	14	21	29	36	43	50	57	64	71	143	214	286	357
8	6	12	19	25	31	38	44	50	56	62	125	187	250	312
9	6	11	17	22	28	33	39	44	50	56	111	167	222	278
10	5	10	15	20	25	30	35	40	45	50	100	150	200	250
11	5	9	14	18	23	27	32	36	41	45	91	136	182	228
12	4	8	12	17	21	25	29	33	38	42	83	125	167	208
13	4	8	12	15	19	23	27	31	35	38	77	115	154	192
14	4	7	11	14	18	21	25	29	32	36	71	107	143	179
15	3	7	10	13	17	20	23	27	30	33	67	100	133	167
16	3	6	9	12	16	19	22	25	28	31	62	94	125	156
17	3	6	9	12	15	18	21	24	26	29	59	88	118	147
18	3	6	8	11	14	17	19	22	25	28	56	83	111	139
19	3	5	8	11	13	16	18	21	24	26	53	79	105	132
20	2	5	8	10	12	15	18	20	22	25	50	75	100	125
21	2	5	7	10	12	14	17	19	21	24	48	71	95	119
22	2	5	7	9	11	14	16	18	20	22	45	68	91	114
23	2	4	7	9	11	13	15	17	20	22	43	65	87	109
24	2	4	6	8	10	12	15	17	19	21	42	62	83	104
25	2	4	6	8	10	12	14	16	18	20	40	60	80	100
26	2	4	6	8	10	12	13	15	17	19	38	58	77	96
27	2	4	6	7	9	11	13	15	17	19	37	56	74	93
28	2	4	5	7	9	11	12	14	16	18	36	54	71	89
29	2	3	5	7	9	10	12	14	16	17	34	52	69	86
30	2	3	5	7	8	10	12	13	15	17	33	50	67	83
35	1	3	4	6	7	9	10	11	13	14	29	43	57	72
40	1	2	4	5	6	8	9	10	12	12	25	37	50	62
45	1	2	3	4	6	7	8	9	10	11	22	33	44	55
50	1	2	3	4	5	6	7	8	9	10	20	30	40	50
55	1	2	3	4	5	5	6	7	8	9	18	27	36	45
60	1	2	2	3	4	5	6	7	7	8	17	25	33	42

FIGURE 2.90.—Tabular data of half rates of ponding during time interval, Edwardsville, Ill.

rectly below the gage height curve, using the scales for the stage graph.

- Subtract the accumulated rainfall curve from the G.D.E. curve at a sufficient number of points to permit construction of another smooth curve. No fixed rule can be given for spacing these points except that they must be closer together where curvature in the stage graph is considerable at significant changes in rainfall intensities.
- Draw a smooth curve through points mentioned in the preceding step. This curve, shown as a solid line, represents the rise in water surface due to surface runoff in excess of outflow. When more than one sheet is necessary for plotting, duplicate a small portion of this sheet on the next sheet.
- Select and indicate groups of uniform time intervals on the graph, step 1, on slowly rising or falling stages. Use a uniform stage increment (0.01 foot (0.3 cm)) rather than a uniform time interval increment. Enter these times and stages in columns 1 and 3 of figure 2.93.
- Enter in column 2 the difference in time between successive entries in column 1.
- In column 4, tabulate the rate of change in stage for each time listed in column 1. For that part of the stage record plotted in figure 2.93, obtain the rate of change in stage at a given time from the stage curve corrected for rainfall by (1) placing a straightedge or triangle tangent to the curve, (2) reading the difference in intercepts on the vertical scale over a 100-minute interval, and (3) pointing off two decimals.
- The straight line shown in figure 2.94 is tangent to the curve at 4:50 a.m. The intercepts at 4:00 a.m. and 5:40 a.m. (100 min later) are 20.25 and 20.51, respectively. The rate of ponding (due to surface runoff) at this time is, therefore, $(20.51-20.25) \div 100 = +0.0026$ ft/min. The rate of change in stage is positive for rising stages and negative for falling stages.
- Use a template prepared on celluloid or heavy transparent plastic material, similar to the template shown in figure 2.94, to determine the rate of change in stage. To use this template, place the stage graph of figure 2.92 on a table or drawing board, using a T-square to line up the base line. Place the bottom of the template against the T-square and slide it along until one of the lines coincides with the curve, corrected for rainfall in step 4, at each time tabulated in column 1. Read the value on this line or interpolate between lines. The rate of change in stage for the part after rainfall ceases is determined according to the procedure used for figure 2.84.
- Obtain the rates of change as outlined. The pondage correction, (q_p, in column 5, fig. 2.93), will be obtained from the rating table (fig. 2.95). Values of q_p are given for each 0.1 foot (3 cm) of stage and can be interpolated for the nearest 0.01 foot (0.3 cm). Error generally will be less than 1 percent, however, if the nearest 0.1 foot(3 cm) is used. Obtain the value of q_p for any rate of change in stage at a given gage height by multiplying the tabular value of q_p for this gage height by the rate of change in stage. Thus, computation can be made with a

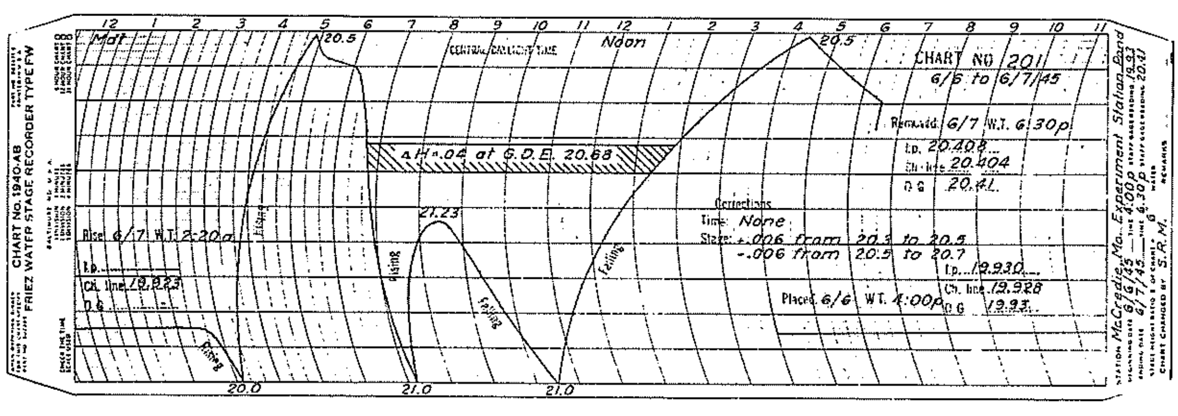


FIGURE 2.91.—Chart record of stage height on pond during runoff period at McCredie, Mo.

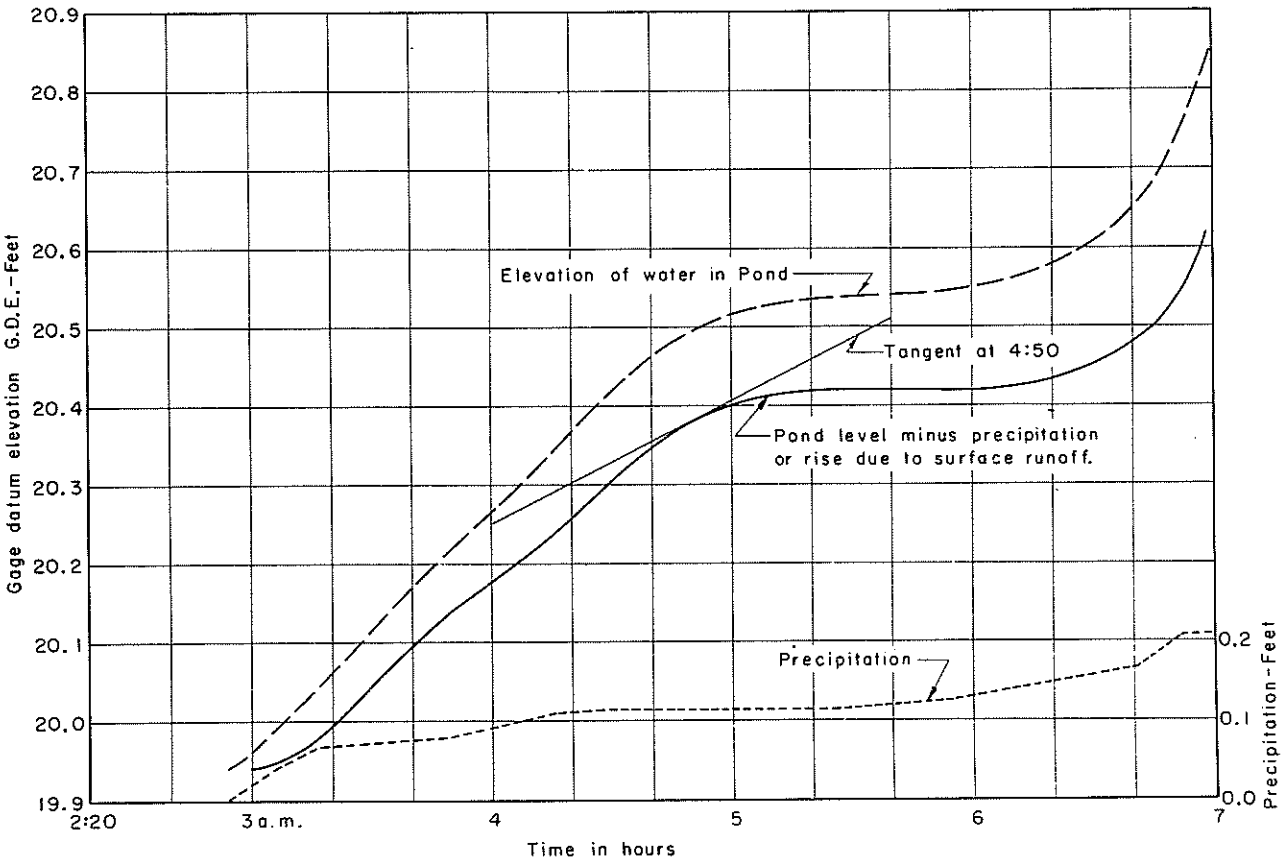


FIGURE 2.92.—Stage record corrected for rainfall on pond surface, McCredie, Mo.

slide rule. The sign of the pondage correction, q_p, is determined by the sign of the rate of change in stage.

- Obtain observed discharges (q_s) in column 6 directly from the rating table for each gage height.
- Obtain entries in column 7, "rate of runoff," by adding the entries in columns 5 and 6; carry entries to three significant figures but never more than four decimals.
- Obtain the values of q in inches per hour, column 8, by multiplying the values in column 7 by the conversion factor given at the bottom of the sheet. The conversion factor is taken as a constant for the entire storm period except when variation in size of the pond surface during a storm is more than 3 percent of the watershed area. If the range in stage is sufficient to produce significant variation in the pond surface area, vary the conversion factor accordingly. Conversion factors for elevations of the pond at the McCredie station are plotted in figure 2.91. Read the conversion factor from this curve from each gage height.
- Perform the rest of the computations in accordance with the instructions for figure 2.89.

Check Method for Total Runoff

An independent, quick method for checking total runoff often is desirable (*37*). Methods outlined in the first edition of this handbook (*42*) reduce the labor of computations without sacrificing accuracy. Gage heights are converted to discharge, and total runoff is determined from the area under the pen trace of the water level recorder chart. Two methods of summation are the use of horizontal strips and the use of vertical strips (fig. 2.96).

Location McCredie, Missouri
File No.
Chart No. 201
Station Reservoir
(Station designation)

Gage datum elevation of point of zero flow 20.0 feet
Gage datum elevation of point of zero pondage 5.6 feet
Gage datum elevation of "float at rest" feet
Gage datum elevation of lowest intake feet

RECORD OF RUN-OFF

Drainage area acres square miles
Dates: May 29 to June 7, 1945
Rating table:

Date and time (1) Hr. and min.	Time interval (2) Minutes	Gage height (3) Feet	Rate of change in stage (4) Feet per min.	Pondage correction (5) Second-feet	Observed discharge (6) Second-feet	Rate of run-off (Discharge corrected for pondage) (7) Second-feet	(8) Inches per hour	Average rate of run-off for time interval (9) Time Factor	Total run-off: For time interval (10) Inches	Accumulated (11) Inches	Remarks (12)
			No Runoff May 29 to			June 7,	1945				
June 7	a.m.										
2:50		19.93	0			0	0			0	
3:00	10	19.96	.0005	5.7	0	5.7	.0373	.083		.0031	Beginning of Runoff
:10	10	20.01	.0026	29.7	.018	29.7	.1945	.083		.0223	
:20	10	20.06	.0047	53.9	.33	54.2	.3550	.083		.0679	
:30	10	20.11	.0052	60.0	.88	60.9	.3990	.083		.1305	
:40	10	20.17	.0048	55.8	1.79	57.6	.3774	.083		.1950	
:50	10	20.22	.0039	45.5	2.72	48.2	.3158	.083		.2525	
4:00	10	20.27	.0036	42.3	3.80	46.1	.3020	.083		.3038	
:10	10	20.32	.0040	47.2	5.00	52.2	.3420	.083		.3572	
:20	10	20.37	.0046	54.6	6.33	60.9	.3990	.083		.4187	
:30	10	20.42	.0047	56.0	7.77	63.8	.4180	.083		.4865	
:40	10	20.47	.0037	44.8	9.32	54.1	.3544	.083		.5507	
:50	10	20.50	.0026	31.6	10.3	41.9	.2744	.083		.6028	
5:00	10	20.51	.0017	20.5	10.7	31.2	.2143	.083		.6434	
:10	10	20.53	.0009	10.9	11.3	22.2	.1455	.083		.6733	
:20	10	20.54	.0003	3.6	11.7	15.3	.1002	.083		.6937	
:30	10	20.54	0	0	11.7	11.7	.0766	.083		.7083	
:40	10	20.54	-.0002	-2.4	11.7	9.3	.0609	.083		.7198	
:50	10	20.55	-.0001	-1.2	12.1	11.9	.0779	.083		.7313	
6:00	10	20.55	.0002	2.4	12.1	14.5	.0950	.083		.7456	
:10	10	20.56	.0006	7.3	12.4	19.7	.1290	.083		.7642	
:20	10	20.58	.0014	17.0	13.1	30.1	.1971	.083		.7913	
:30	10	20.61	.0023	28.1	14.3	42.4	.2778	.083		.8307	
:40	10	20.65	.0035	42.9	15.8	58.7	.3845	.083		.8857	
:45	5	20.68	.0047	57.8	17.0	74.8	.4900	.042		.9224	
:50	5	20.74	.0072	89.2	19.6	108.8	.7125	.042		.9729	
:55	5	20.80	.0120	150	22.2	172.2	1.1280	.042		1.0502	
:57	2	20.82	.0124	155	23.1	178	1.1660	.017		1.0892	Peak Discharge
7:00	3	20.86	.0121	152	25.0	177	1.1590	.025		1.1473	
:02	2	20.88	.0114	143	26.0	169	1.1070	.017		1.1858	
:05	3	20.91	.0104	131	27.4	158	1.0350	.025		1.2394	
:10	5	20.97	.0087	111	30.4	141.4	.9260	.042		1.3218	
:15	5	21.01	.0073	93.2	32.4	125.6	.8230	.042		1.3952	
:20	5	21.05	.0064	82.1	34.5	116.6	.7640	.042		1.4619	
:25	5	21.08	.0055	70.8	36.1	106.9	.7000	.042		1.5234	
:30	5	21.11	.0044	56.8	37.7	94.5	.6190	.042		1.5788	
:40	10	21.15	.0032	41.5	40.0	81.5	.5340	.083		1.6745	
:50	10	21.18	.0025	32.6	41.7	74.3	.4870	.083		1.7592	
8:00	10	21.21	.0017	22.2	43.5	65.7	.4303	.083		1.8353	
:10	10	21.22	.0009	11.8	44.1	55.9	.3660	.083		1.9014	
:20	10	21.23	.0002	2.6	44.7	47.3	.3100	.083		1.9575	Maximum Stage
:25	5	21.23	-.0001	-1.3	44.7	43.4	.2840	.042		1.9825	
:30	5	21.23	-.0003	-3.9	44.7	40.8	.2670	.042		2.0056	
:45	15	21.22	-.0013	-17.0	44.1	27.1	.1775	.125		2.0612	
:58	13	21.20	-.0017	-22.20	42.9	20.7	.1355	.108		2.0950	
9:09	11	21.18	-.0019	-24.70	41.7	17.0	.1114	.092		2.1177	
:19	10	21.16	-.0021	-27.2	40.6	13.4	.0878	.083		2.1342	
:28	9	21.14	-.0022	-28.5	39.4	10.9	.0714	.075		2.1462	
:42	14	21.11	-.0023	-29.7	37.7	8.0	.0524	.117		2.1607	
:55	13	21.08	-.0024	-30.9	36.1	5.2	.0341	.108		2.1700	
10:08	13	21.05	-.0023	-29.5	34.5	5.0	.0328	.108		2.1772	
:22	14	21.02	-.0023	-29.4	32.9	3.5	.0029	.117		2.1814	
:35	13	20.99	-.0022	-28.0	31.4	3.4	.0023	.108		2.1820	
:50	15	20.96	-.0021	-26.6	29.9	3.3	.0216	.125		2.1850	
11:04	14	20.93	-.0021	-26.5	28.4	1.9	.0124	.117		2.1889	
:19	15	20.90	-.0020	-25.2	26.9	1.7	.0111	.125		2.1919	
:34	15	20.87	-.0019	-23.9	25.5	1.6	.0105	.125		2.1946	
:50	16	20.84	-.0018	-22.5	24.1	1.6	.0105	.133		2.1974	
12:07	17	20.81	-.0017	-21.2	22.7	1.5	.0098	.142		2.2003	
:25	18	20.78	-.0016	-19.9	21.3	1.4	.0092	.150		2.2031	
:45	20	20.75	-.0015	-18.6	20.0	1.4	.0092	.167		2.2062	
1:06	21	20.72	-.0014	-17.3	18.7	1.4	.0092	.175		2.2094	
:27	21	20.69	-.0014	-17.2	17.5	0.3	.0020	.175		2.2114	
:50	23	20.66	-.0013	-16.0	16.2	0.2	.0013	.192		2.2120	
2:15	25	20.63	-.0013	-15.9	15.0	-0.9	0	.208		2.2123	End of Runoff
							19.6503	5.703			Total June 7 = 2.2123"

Conversion factor for drainage area: 1 cubic foot per second .00655 inches per hour.

Tabulated by John Doe 8/8/45 (Date) Checked by Henry Roe 8/10/45 (Date)
Computed by John Doe 8/11/45 (Date) Checked by Allen Poe 8/16/45 (Date)

Sheet of sheets

FIGURE 2.93.—Computations for pond stage record of runoff at McCredie, Mo.

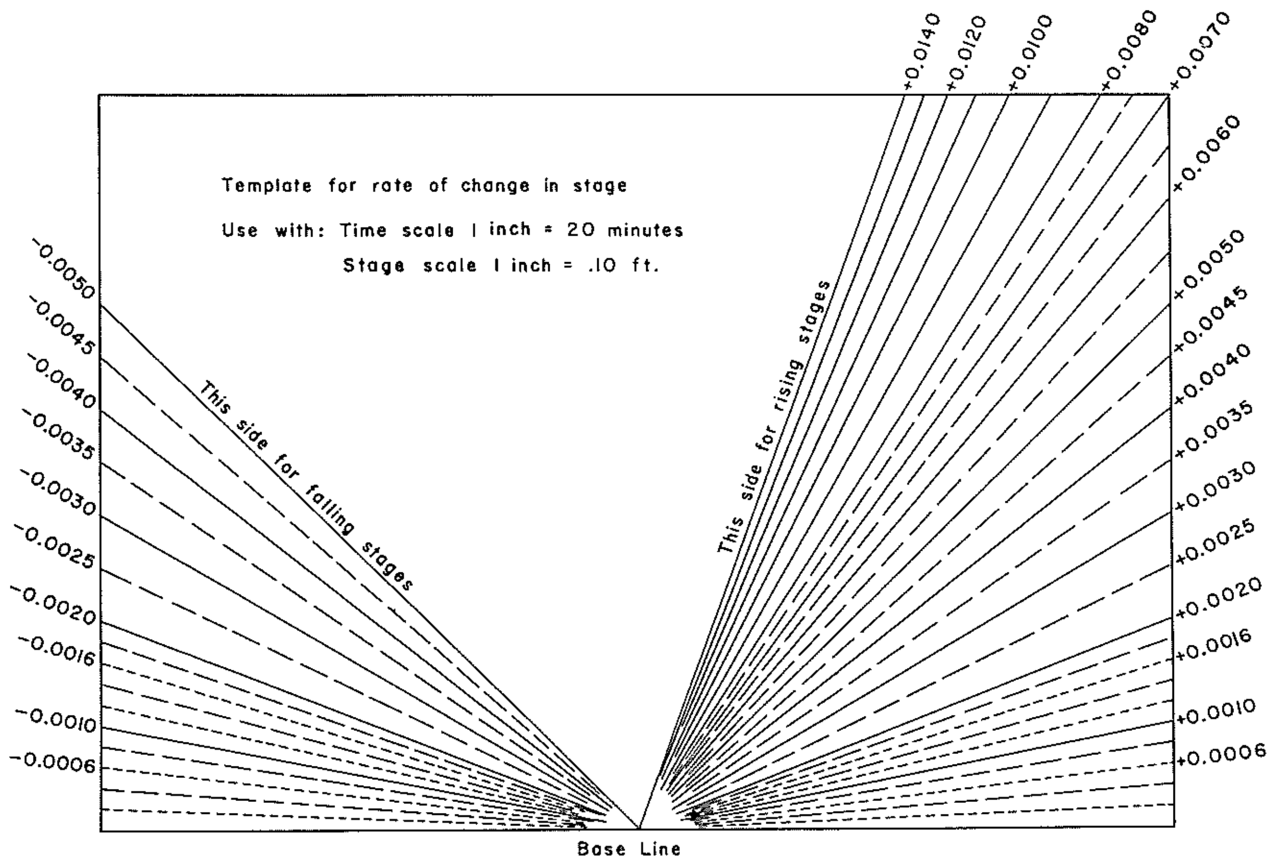


FIGURE 2.94.—Use of template for obtaining tangents to curves on rectangular coordinates.

Computer Calculations

Digital computer processing of basic runoff data is more rapid, accurate, and efficient. Selection of the process depends on basic data, computer hardware available, programing talent available, and desired end results. A few principles will be discussed, and two examples will be given. Figure 2.97 illustrates the flow of runoff data processing at the Southern Great Plains Watershed Research Center (*36*). Figure 2.98 illustrates the process at the Southwest Watershed Research Center. Although both procedures have the same objective, each is tailored to specific needs and available computer hardware.

First, all analog records must be digitized, or digital information must be transcribed to punched cards, magnetic tape, or whatever is used as computer input. Digitized or transcribed data generally contain more than time and stage height values. Identification information, codes pertaining to processing programs, estimates, and machine operator signatures also are included. The runoff data format used by the Southwest Watershed Research Center is shown in figure 2.99. This format includes:

- Beginning time with elapsed time or the military time of each datum point;
- Estimation of time and stage height;
- A correction factor for adjustment of th. stage heights;
- Codes to call for the addition of standard recessions to the record;
- Codes for standard or daylight savings time (all records in daylight savings time are converted to standard time); and

RATING TABLE FOR MCCREDIE EXPERIMENT STATION RESERVOIR

G.D.E. = Gage datum elevation
G.D.E. of zero flow over weir = 20.0
Reference: Drawing No. P-8242

q_s values from laboratory calibration of model at Saint Anthony Falls Hydraulics Laboratory, or $q = 31.9h^{1.625}$

G.D.E. of zero pondage = 5.5
Pond surveyed: 1940-41

Stage G.D.E. (feet)	Observed discharge over spillway (q_s) (Discharge in cubic feet per second)									
	0	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
20.0	0	0.018	0.056	0.107	0.171	0.245	0.330	0.423	0.525	0.635
20.1	.755	.881	1.01	1.15	1.30	1.46	1.62	1.79	1.96	2.14
20.2	2.33	2.52	2.72	2.92	3.13	3.35	3.57	3.80	4.03	4.27
20.3	4.51	4.75	5.00	5.26	5.52	5.79	6.06	6.33	6.61	6.89
20.4	7.18	7.47	7.77	8.07	8.38	8.69	9.00	9.32	9.64	9.97
20.5	10.3	10.7	11.0	11.3	11.7	12.1	12.4	12.8	13.1	13.5
20.6	13.9	14.3	14.7	15.0	15.4	15.8	16.2	16.6	17.0	17.5
20.7	17.9	18.3	18.7	19.1	19.6	20.0	20.4	20.9	21.3	21.8
20.8	22.2	22.7	23.1	23.6	24.1	24.5	25.0	25.5	26.0	26.4
20.9	26.9	27.4	27.9	28.4	28.9	29.4	29.9	30.4	30.9	31.4
21.0	31.9	32.4	32.9	33.4	34.0	34.5	35.0	35.6	36.1	36.6
21.1	37.2	37.7	38.3	39.0	39.4	40.0	40.6	41.1	41.7	42.3
21.2	42.9	43.5	44.1	44.7	45.3	45.9	46.5	47.1	47.7	48.3
21.3	48.8	49.4	50.1	50.7	51.3	51.9	52.6	53.2	53.9	54.5
21.4	55.1	55.8	56.4	57.1	57.7	58.4	59.1	59.7	60.4	61.1
21.5	61.7	62.4	63.1	63.8	64.5	65.2	65.8	66.5	67.2	67.9
21.6	68.6	69.3	70.0	70.7	71.4	72.1	72.8	73.5	74.2	75.0
21.7	75.7	76.4	77.1	77.9	78.6	79.3	80.0	80.8	81.5	82.3
21.8	83.0	83.7	84.5	85.2	86.0	86.7	87.5	88.2	89.0	89.7
21.9	90.5	91.3	92.1	92.8	93.6	94.4	95.2	96.0	96.8	97.6
22.0	98.4	99.2	100	100	102	102	103	104	105	106
22.1	107	107	108	109	110	111	112	112	113	114
22.2	115	116	117	118	118	119	120	121	122	123
22.3	124	--	--	--	--	--	--	--	--	--

Stage G.D.E. (feet)	Pondage correction (q_p) (Cubic feet per second for each 0.10 ft. of stage for a rate of change in stage of 1 ft. per minute)									
	0	0.10	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90
5.0	--	--	--	--	--	0	15	35	55	77
6.0	100	123	152	181	210	243	277	310	349	388
7.0	426	465	512	558	605	665	725	785	856	926
8.0	997	1067	1160	1253	1345	1447	1548	1650	1744	1838
9.0	1932	2025	2103	2181	2260	2342	2424	2505	2587	2669
10.0	2750	2832	2921	3010	3100	3197	3294	3390	3488	3586
11.0	3684	3782	3863	3944	4025	4117	4209	4300	4383	4466
12.0	4549	4632	4718	4804	4890	4970	5050	5130	5218	5306
13.0	5394	5482	5548	5614	5680	5752	5824	5895	5968	6040
14.0	6112	6185	6257	6329	6400	6467	6533	6600	6674	6748
15.0	6823	6897	6965	7032	7100	7175	7250	7325	7401	7477
16.0	7553	7630	7708	7786	7865	7943	8022	8100	8186	8272
17.0	8358	8443	8529	8615	8700	8783	8867	8950	9039	9128
18.0	9217	9307	9405	9502	9600	9700	9800	9900	10004	10108
19.0	10212	10317	10431	10546	10660	10773	10887	11000	11131	11261
20.0	11392	11522	11648	11774	11900	12043	12187	12330	12471	12612
21.0	12753	12893	13055	13217	13380	13540	13700	13860	14032	14204
22.0	14376	14548	14719	14890	15060	15240	15420	15600	15782	15964
23.0	16146	16328	--	--	--	--	--	--	--	--

FIGURE 2.95.—Discharge values for spillway and pondage correction data for McCredie reservoir.

- Initials of the technician reading the record. Estimates should be added to the record.

Computer Check of Digital Record

Any logical or possible error situations in recording or preparing the digital data may be detected with a computer program. A complete checking program depends on the processing system used. This program usually is developed by trial and error. Chronological order, proper sequence of codes, and other items, should be checked, depending on the system used. The only efficient way to make checks is by use of a computer program. The following situations are checked by the Southwest Watershed Research Center checking program:

1. Header card preceding first event of new year for a given location.
2. First card of event preceded by end card for last event.
3. End card on event preceding header card.
4. Header card present where location and year do not change.
5. Location same as location on previous header card.
6. Header card out of order—station does not increase or year does not increase.
7. Endrun card in place of header card (events following endrun card).

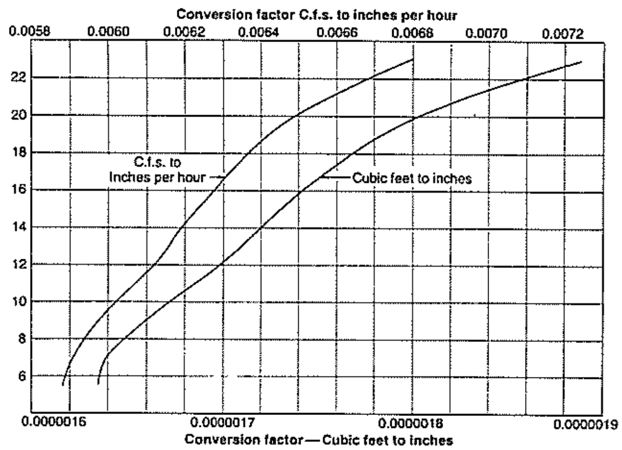


FIGURE 2.96.—Chart record of variation in conversion factors due to changes in net area of the McCredie watershed.

8. Location of begin card of event compared to location on header card.
9. Location on records of an event compared to location of the begin card of event.
10. Station and year compared in same manner as location for (8) and (9).
11. Correct time note for single-record events.
12. Improper time note on regular events.
13. Correct time note for single-card event but incorrect position code.
14. Illegal depth note (not 1, 0, blank, or E).
15. Illegal position code (not 1, 2, 3, 4, or blank).
16. Illegal standard recession code (not 0, blank, 1, 2, or 3).
17. Standard recession code present on record with depth less than or equal to 1.50 (applies to certain locations).
18. Illegal type of time code (not 0, 1, or blank).
19. Standard recession code present on card other than end card of event.
20. Single-card event preceded by single-card event or end-card of event.
21. Date decreases within year; date decreases, station does not change (no header card before event).
22. Time interval within event greater than 24 hours.
23. Begin card after header card.
24. Middle card of event preceded by begin card or another middle card.
25. End card preceded by middle card or begin card of event.
26. Date decreases within event.
27. Time does not increase within station.
28. Date not changed at 2401.
29. Gage height constant does not equal zero—warning.
30. Depth value blank.
31. Depth value greater than 1.50, and no standard recession code present (applies to certain locations).

Updating the Basic Data

Once errors in the basic data have been detected by a checking program, they need to be corrected. The method selected to make the corrections depends on how the basic record is stored. If the basic data are processed from punched cards, corrected cards must be punched and filed. If these data are processed from magnetic tape or disc, corrections will be made by a program with the corrections as input. Once the corrections have been made, the basic data can be processed by a program to make desired tabulations.

Instantaneous Discharge Values

The measured depth of flow past the measuring station is associated with a discharge rate. This association is expressed by a rating table or a functional relation. It may be used with a computer program to determine the discharge rate each time a stage measurement is made. Once the flow rate is associated with the depth, the sequence of values can be integrated numerically to obtain the volume of the flow.

Numerical Integration With No Adjustment for Ponding

The following calculations must be programed to make the numerical integration and adjust for the influences of ponding.

- Determine the $n = 1$ to N rates of discharge [in units of volume per unit time (ft³/s) or depth per unit time (in/hr)] from the N stage values.

• Calculate the average rate between two successive discharge values by adding the two values and dividing by 2.

$$RAVE\ (n+1) = \frac{R(n)+R(n+1)}{2}$$

where $RAVE$ is average discharge $R(n)$ in rate interval (n).

• Calculate the increment of volume by multiplying the average rate by the duration of time increment.

$$\text{Vol.}\ (n+1) = [RAVE\ (n+1)]\ [T\ (n+1) - T\ (n)]$$

where $T(n)$ is the time in interval (n).

• Obtain the total volume by summing the several increment volumes.

$$\text{Total vol.} = \sum_{n=1}^{N} \text{Vol.}\ (n).$$

Numerical Integration With Adjustment for Ponding

The following calculations must be programed to make the numerical integration with adjustment for ponding.

• Determine the $n = 1$ to N rates of discharge [in units of volume per unit time (ft^3/s) or depth per unit time (in/hr)] from the N stage values.

• Make provision in the program to also input a pondage correction table (amounts of volume for each increment in stage [that is, 0.01 foot (0.02 cm) of stage] for a constant rate of change of stage [that is, 1 ft/min(30.5 cm/m))].

• From the basic data, calculate the change in stage. Theoretically, it would be the first derivative of the stage function at the particular time (or tangent to the recorded stage vs. time curve at the time of interest). Since the record of stage has been digitized (sampled at a finite number of points), a good approximation of the tangent at a point is to evaluate the slope of the chord between two points *equal* time increments preceding and succeeding the point under consideration.

• Multiply the rate of change of stage times the value from the pondage correction table of flow rate per unit rate of change in stage to obtain the rate of flow going into or coming out of storage.

• Add the amount of flow going into pond storage to the measured outflow for the total flow rate. Subtract the amount coming out of storage from the measured outflow for the total flow rate.

• Once adjustments for ponding have been made to the flow rate, continue the program with the previously described procedure for processing with no adjustments (p. 204) and continue with the steps outlined there.

Numerical Integration With Adjustment for Pondage and Rain on the Pond

Increase in the pond stage (depth) due to rain falling directly into the pond is deducted by subtracting the accumulated depth (in feet) of rainfall from the record of stage (in feet) during the rainfall. The unadjusted stage record is used to determine the $n = 1$ to N rates of discharge past the measuring structure. With the adjusted record of stage from step A, proceed with the computations in step B of the previous section, "Numerical Integration With Adjustment for Ponding."

Alternative Method for Discharge Determination

Rates of inflow to a pond or small reservoir are calculated in the same way pondage corrections are made to flow through a flow-measuring structure. Use the outlined procedures described in section "Numerical Integration With Adjustment for Ponding" to calculate the flow rate for the particular situation.

An alternate method of calculating the rate of flow into a pond is (*9*):

1. Prepare gage height versus volume relation.
    - Planimeter successive contours on topographic map of pond.
    - Calculate volumes between contours, using the formula:

$$V_i = 1/2\,(A_{i-1} + A_i)\,h,$$

where A is the area enclosed by the

contour and h is the difference in elevation of the contours. [Make volume calculations in ft³ (m)].

- Plot gage height versus volume relation.

2. Prepare gage height versus volume table. [The plot is used only for the foot contours since the scale is not accurate enough to read the volumes at 0.10- or 0.01-foot (3.05 or 0.3 cm) intervals.]

- From the plot of gage height versus volume, read the volumes at successive 1-foot (30.5 cm) contours.
- Using the method of increasing first differences, find the volumes at each 0.10 and 0.01 foot (3.05 and 0.3 cm)

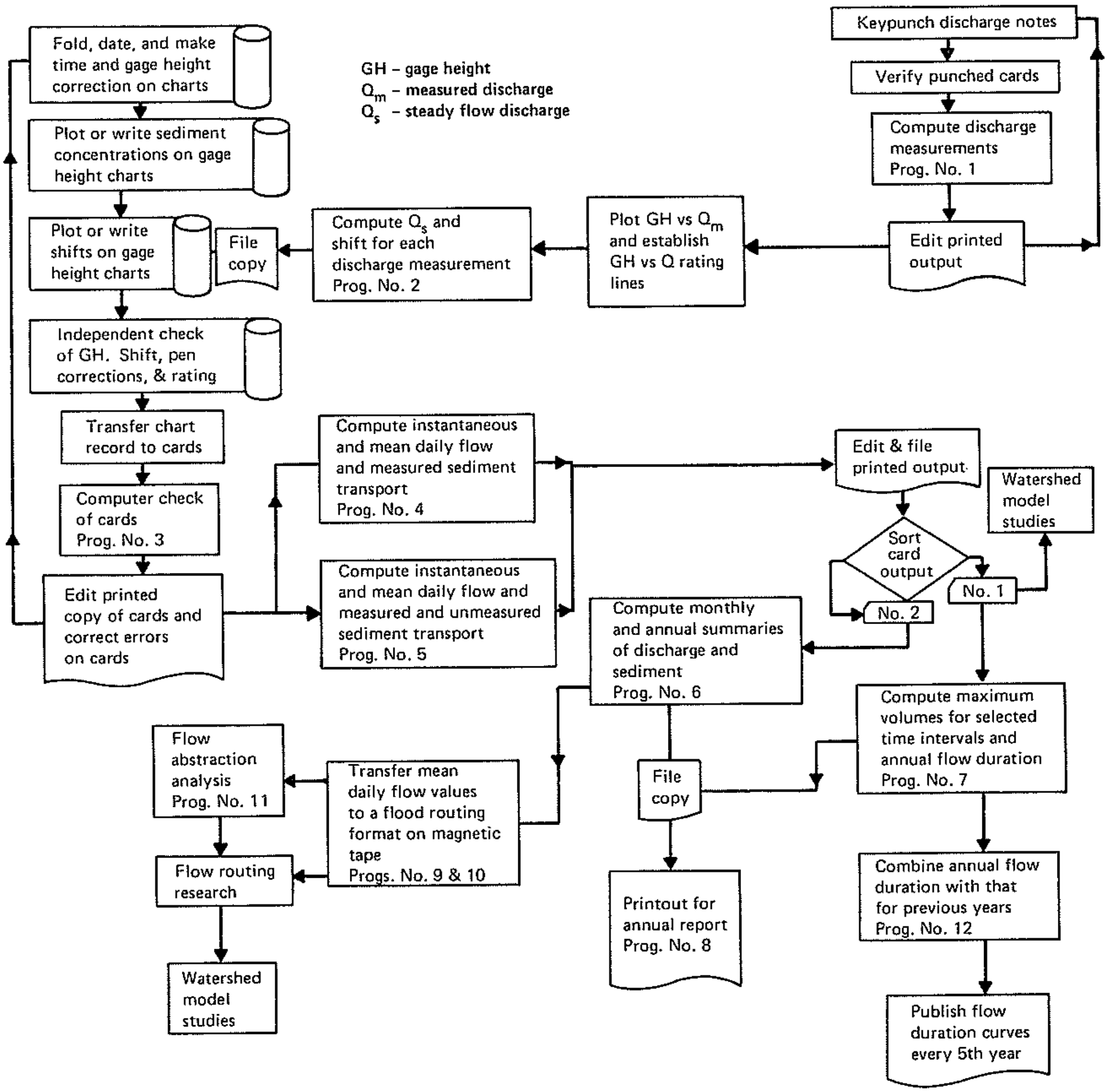


FIGURE 2.97.—Data processing system for streamflow and sediment transport at the Southern Great Plains Watershed Research Center.

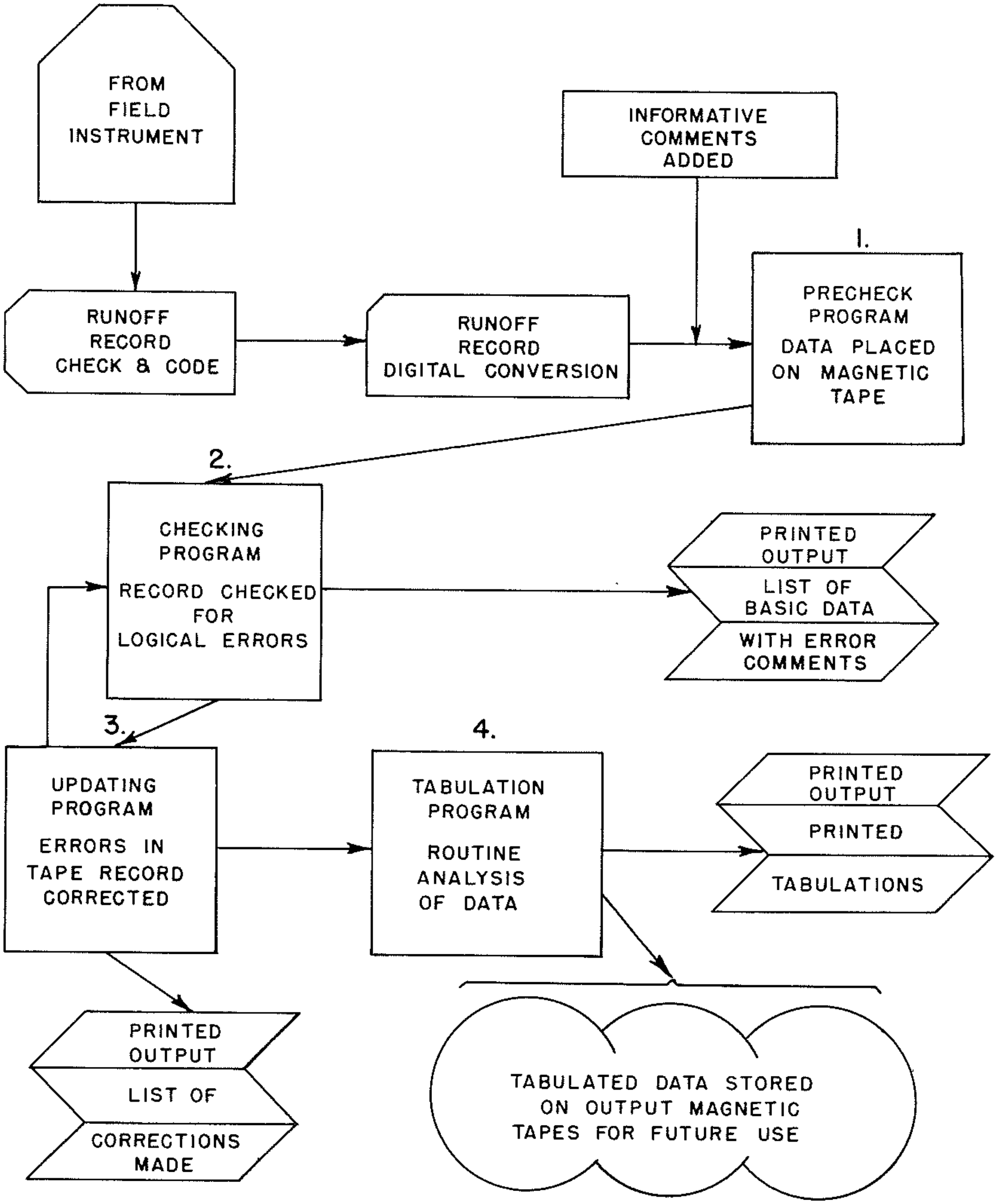
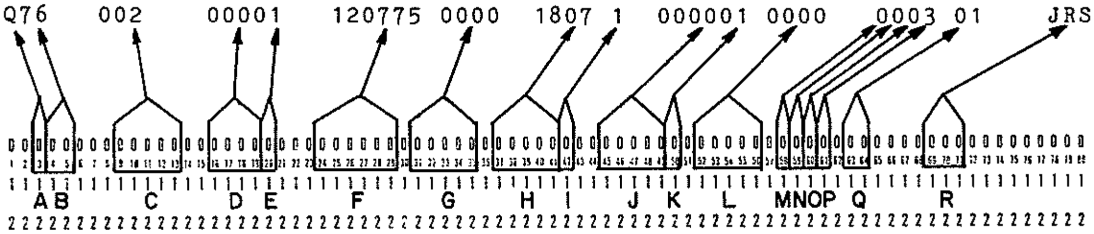


FIGURE 2.98.—Processing scheme for runoff data at the Southwest Watershed Research Center.

Q76	002	00001	120775	0000	1721	0	000000	0000	0001	01	JRS
Q76	002	00001	120775	0000	1724	0	000300	0000	0002	01	JRS
Q76	002	00001	120775	0000	1726	0	000500	0000	0002	01	JRS
Q76	002	00001	120775	0000	1728	0	000600	0000	0002	01	JRS
Q76	002	00001	120775	0000	1730	0	000600	0000	0002	01	JRS
Q76	002	00001	120775	0000	1731	0	009000	0000	0002	01	JRS
Q76	002	00001	120775	0000	1732	0	010800	0000	0002	01	JRS
Q76	002	00001	120775	0000	1733	0	012800	0000	0002	01	JRS
Q76	002	00001	120775	0000	1734	0	013700	0000	0002	01	JRS
Q76	002	00001	120775	0000	1735	0	014000	0000	0002	01	JRS
Q76	002	00001	120775	0000	1736	0	013700	0000	0002	01	JRS
Q76	002	00001	120775	0000	1737	0	012900	0000	0002	01	JRS
Q76	002	00001	120775	0000	1738	0	011700	0000	0002	01	JRS
Q76	002	00001	120775	0000	1739	0	011000	0000	0002	01	JRS
Q76	002	00001	120775	0000	1740	0	010500	0000	0002	01	JRS
Q76	002	00001	120775	0000	1741	0	010000	0000	0002	01	JRS
Q76	002	00001	120775	0000	1742	0	008300	0000	0002	01	JRS
Q76	002	00001	120775	0000	1743	0	007000	0000	0002	01	JRS
Q76	002	00001	120775	0000	1744	0	006000	0000	0002	01	JRS
Q76	002	00001	120775	0000	1745	0	005000	0000	0002	01	JRS
Q76	002	00001	120775	0000	1746	0	004300	0000	0002	01	JRS
Q76	002	00001	120775	0000	1747	0	003500	0000	0002	01	JRS
Q76	002	00001	120775	0000	1748	0	002900	0000	0002	01	JRS
Q76	002	00001	120775	0000	1750	0	002100	0000	0002	01	JRS
Q76	002	00001	120775	0000	1752	0	001400	0000	0002	01	JRS
Q76	002	00001	120775	0000	1755	0	000900	0000	0002	01	JRS
Q76	002	00001	120775	0000	1807	1	000001	0000	0003	01	JRS



A. Data type code (Q = runoff).
B. Watershed I.D.
C. Station.
D. Response note. Cols. 16 & 17, give count of total number of runoff responses for a single rain. Cols. 18 & 19, give the position of the event among the several multiple responses.
E. Peak note. (1 = Simple; 2 = Complex).
F. Date (day, month, year).
G. Begin time of event. For the begin time-elapsed time mode of noting time (24 hour clock time).
H. The 24 hour clock time of each reading or the elapsed time.
I. Time note for indication of estimations.
J. Depth.
K. Depth note for indication of estimations.
L. Correction factor.
M. Time mode code (0 = 24 hour clock time for each data point; 1 = begin time and elapsed time for each reading).
N. Standard recession code.
O. Type of time code (0 = Standard time; 1 = Daylight savings or War time).
P. Card Position Code.
Q. Rating Table I.D.
R. Coder's initials.

FIGURE 2.99.—Runoff data format from the Southwest Watershed Research Center.

between contours. Complete gage height versus volume table (fig. 2.83).

3. From a tabulation of gage height versus time (read from the recorder chart), calculate $\Delta S/\Delta t$, the change in storage with respect to time.

- Reading values from the derived table, tabulate the total storage versus time.
- Calculate the actual storage for the time increment with the formula:

$$\Delta S_{i+1} = S_{i+1} - S_i.$$

- Calculate $\Delta S/\Delta t$ by dividing ΔS by the time increment in seconds. The result will be $\Delta S/\Delta t$ in cubic feet per second.

From $\Delta S/\Delta t$, determine the inflow hydrograph. The outflow, if any, is determined from the gage height and spillway cross section.

The inflow hydrograph is computed from:

$$I = \Delta S/\Delta t + Q \tag{2-44}$$

where

I is the inflow and Q is the outflow with I located at the midpoint of the interval Δt.

Use of the volume increment method to calculate a pond inflow hydrograph eliminates the intermediate step of calculating a rate of change of pond storage for a given uniform rate of change of stage relation. Comparisons of the results from the two methods (fig. 2.100 and 2.101) also show that little difference exists between the two methods, especially when the increment is small.

Flow Duration Computations

The duration of flow within any period (whether it is the entire period of record, a calendar year, or any other specified length of time) can be tallied by a computer program. Where total flow duration is desired over a long period, a storage register can be set and the duration of each event can be added as it is tabulated. If the entire period of record or an entire year is not processed in one pass on the

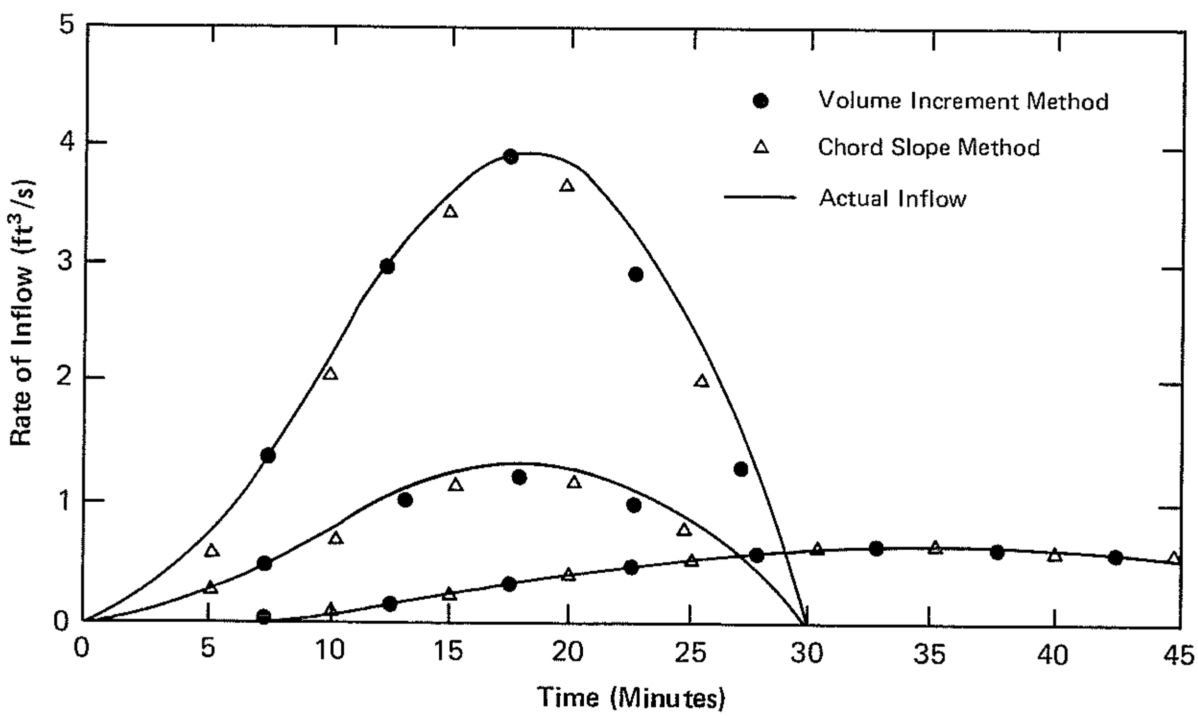


FIGURE 2.100.—Actual inflow rates with superimposed rates calculated by the volume increment method and chord slope method, comparing parabolic input to a parabolic pond cross section of unit length.

computer, tally information must be carried over to the succeeding processes. Programs of the Southwest Watershed Research Center maintain a tally of flow duration that was estimated so that a comparison of estimated flow duration to total flow duration (including estimated duration) may be used to evaluate quality of the record.

If the desired information is duration of flow rates above set threshold values, slightly more elaborate accounting procedures must be programed. An adequate procedure has been described by Woolhiser and Saxton (*43*). A description of the procedure and a definition sketch will be given for reference.

Routine for Flow Durations and Volume

After each incremental volume is computed, the portion of the volume Δt_1 above a flow rate, *CLASS* (*J*), is accumulated as *VOLD* (*J*) and *DUR* (*J*). By using several flow rates, *CLASS* (*J*), as specified in the basic data cards, a flow-duration series is computed.

The volume of flow at rates equal to or greater than the rate *CLASS* (*J*) is shaded in the hydrograph of figure 2.102. A straight-line variation of discharge between tabulated points is assumed, and incremental volumes and durations are computed on this basis. For example, the equation for the incremental duration in triangle 1 is:

$$\text{Length}(ac) = \frac{[Q3 - CLASS(J)] \times (t3 - t2)}{Q3 - Q2}. \quad (2\text{–}45)$$

The incremental volume is:

$$\text{Area}(1) - \frac{[Q3 - CLASS(J)]^2 \times (t3 - t2)}{2(Q_3 - Q_2)}. \quad (2\text{–}46)$$

Maximum (or Minimum) Volume for Selection Time Interval Computations

Programs can be prepared to calculate the maximum (or minimum) flow volume for any

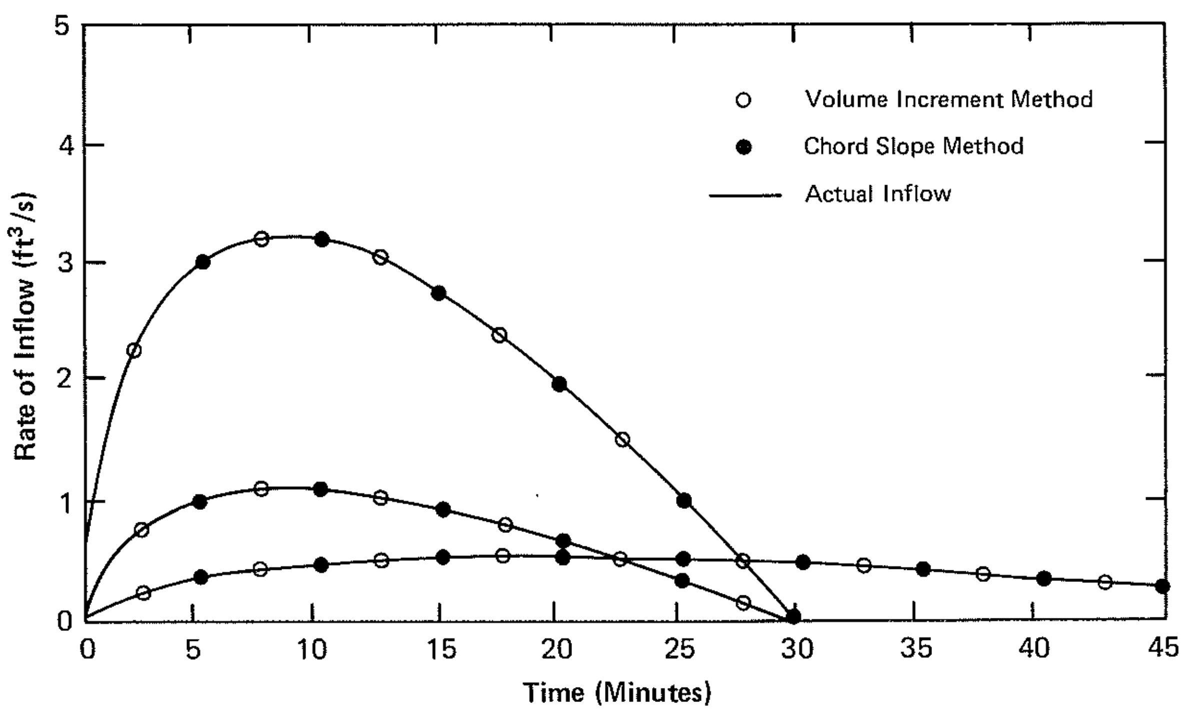


FIGURE 2.101.—Actual inflow rate with superimposed rates calculated by the volume increment method and cosine method, comparing parabolic input to a parabolic pond cross section of unit length.

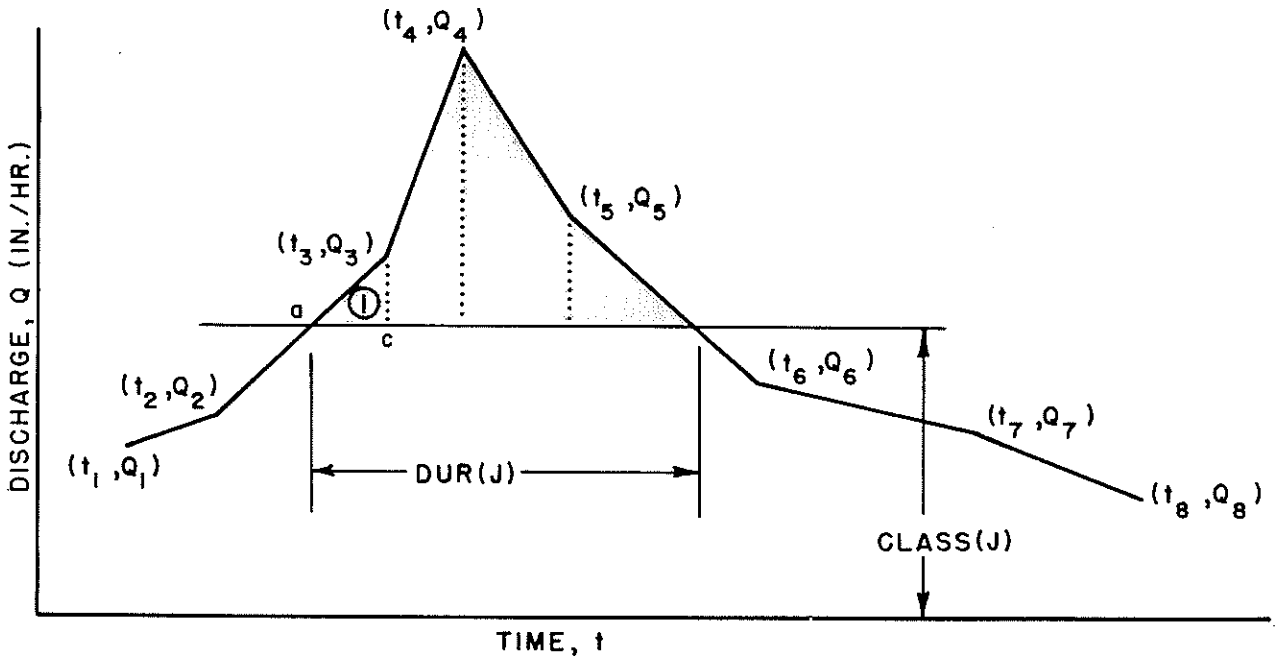


FIGURE 2.102.—Hydrograph for determining flow-duration value.

desired period of record, from a single event to the entire period. A good computer program procedure for these calculations was reported by Woolhiser and Saxton (*43*). This procedure is given for reference.

Annual maximum volumes of runoff occurring within the selected time intervals P (J)are determined by a search of the previously calculated data. Two indexed variables, V (L) and T (L), are used. V (L) is the accumulated runoff in inches, and T (L) is its corresponding time in hours from the beginning of the calendar year.

The limit of storage in the computer for these values is set in the DIMENSION statement. Since one unit of storage for each variable is required for each input reading, the maximum number needed can be estimated. Since this represents a sizable amount of storage on the smaller computers, it may be necessary to store fewer values, search more often, and combine the results.

A mesh of time length P (J) is set up to make this search. The mesh is indexed at $J = 1$, which sets its length equal to the first value specified in the basic constants. This mesh is shifted through the stored data to obtain the maximum volume accumulated (during this length of time and the beginning time) of this maximum amount. The mesh is reindexed, and the procedure is repeated for each successive length of time.

To make this search, *LL* indexes the leading edge of this mesh and *LT* indexes the trailing edge. These indices are assigned values equal to the index of a time value. Use the time value at which the mesh edge occurs if the mesh edge is coincidental with a time value or use the value immediately following if the mesh edge occurs between two time values. This substitution was made in the following three equations when incorporating them into the program.

Three possible positions of the search mesh with respect to the stored values of time as it is shifted through the data are:

- The leading and trailing edges exactly coincide with the stored values of T_L as shown in figure 2.103. Thus, no interpolation is required, and the volume V_J in the time period P (J) is given by:

$$V_J = F_{L+N} - V_L. \tag{2-47}$$

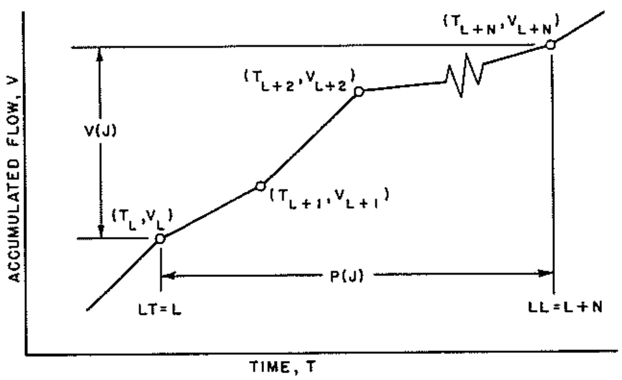


FIGURE 2.103.—Maximum volume search with leading and trailing edges coinciding with the time interval.

• The leading edge falls between two values of time as shown in figure 2.104. An interpolation is required at the leading edge of the mesh, and the volume V_J is given by:

$$V_J = V_{L+N-1} + \frac{P\,(J) - T_{L+N-1} + T_L}{T_{L+N} - T_{L+N-1}} \times V_{L+N} - V_{L+N-1} - V_L. \qquad (2\text{–}48)$$

• The trailing edge falls between two time values, as shown in figure 2.105. An interpolation is required at the trailing edge of the mesh; and, the volume V_J is given by:

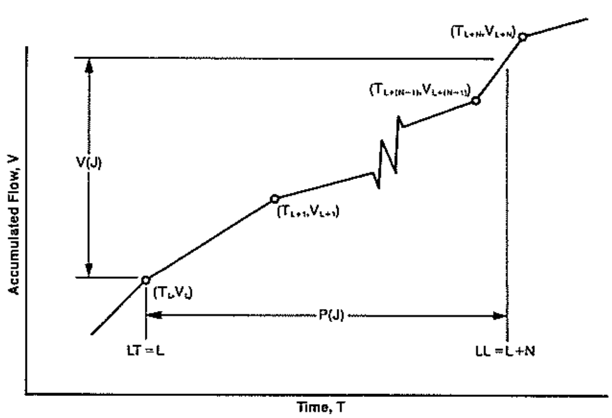


FIGURE 2.104.—Maximum volume search with leading edge interpolation.

$$V_J = V_{L+N} - V_L + \frac{T_{L+N} - T_L - P\,(J)}{T_{L+N} - T_L} \times (V_{L+1} - V_L). \qquad (2\text{–}49)$$

Summary Information

Summary information of the runoff data tabulation may be calculated and formated for use by a computer program. The following information may be prepared by computer programs as a summary:

1. Number of runoff events.
2. Number of estimated events.
3. Number of known events but no record obtained.
4. Total (including estimated amounts) event flow duration per year.
5. Estimated event flow duration per year.
6. Table of durations of flow above selected rates.
7. Table of monthly volumes for each measured station.
8. Yearly total volumes for each measuring station.
9. Monthly average volume for period of record for each measuring station.
10. Yearly average volume for period of record for each measuring station.
11. Maximum peak discharge for the year.
12. Average peak discharge.

Incremental plotters are commonly available. Programs may be expanded or specifically written to prepare machine plottings of hydrographs, mass curves, and other information for which a plotting is desired. These programs

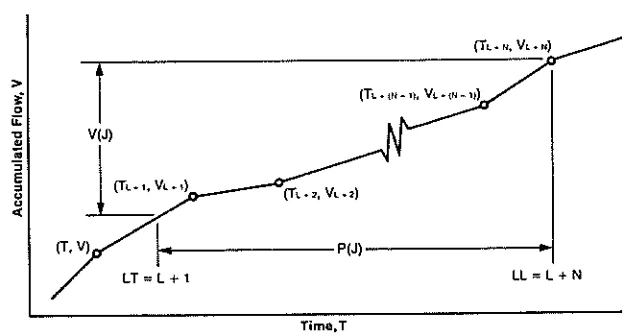


FIGURE 2.105.—Maximum volume search with trailing edge interpolation.

depend on the input data, hardware available, and programing talent available. A plotting program will call special system subroutines (supplied by the manufacturer of the plotting equipment) and will prepare a magnetic tape with plotting instructions. This tape is used as input to the plotting device on which the instructed plotting is done. Plotting programs can be incorporated within large overall processing programs as options to use, or they can remain as separate smaller programs using output tapes (see fig. 2.98) for their source data.

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