F

Stream-Gaging Methods for Short-Term Studies

Discharge, Q, is the volume rate of flow [$L^3\ T^{-1}$] through a stream cross-section at right angles to the flow direction:

$$Q = U \cdot A = U \cdot B \cdot Y. \qquad \textbf{(F-1)}$$

Here, U is average velocity through the cross-section [$L\ T^{-1}$], A is the area of the cross-section [L^2], B is the water-surface width at the cross-section [L], and Y is the average depth at the cross-section [L]. The process of measuring discharge is called **stream gaging.**

Methods for determining the discharge occurring at the time of observation can be classified as shown in Table F-1. Discharge can be measured directly by several methods or determined indirectly by (1) observing the **stage,** Z_s, defined as

$$Z_s \equiv Z_w - Z_0, \qquad \textbf{(F-2)}$$

where Z_w is the elevation of the water surface and Z_0 is the elevation of an arbitrary datum, and (2) using a previously established relation between stage and discharge. The stage–discharge relation is called a **rating curve** (or **rating table**); its form is determined by the configuration of the stream channel in the measurement reach. This configuration may be that of the natural channel (**natural control**) or it may be that of an artificial structure such as a weir or flume (**artificial control**).

This appendix describes stream-gaging methods that are suitable for short-term and special-purpose studies: direct measurement via the velocity-area method (Section F.2) and dilution gaging (Section F.3), indirect measurement via weirs (Section F.4) and flumes (Section F.5), and the measurement of stage (Section F.6). Section F.7 describes methods for estimating the discharge of a peak flow that occurred prior to the time of measurement.

The details of constructing permanent stream-gaging installations for long-term measurement stations were described by Reinhart and Pierce (1964), Buchanan and Somers (1969), Gregory and Walling (1973), Herschy (1999), and Shaw (1988). Herschy (1999) provided a complete discussion of measurement errors using various techniques.

F.1 SELECTION OF MEASUREMENT LOCATION

The overall purposes of a study determine the general location of a stream-gaging location. For example, when water-balance information is sought, the gage might be sited near the outlet of a drainage basin or at the mouths of major streams entering a lake. When information on flows is to be used in the design of structures or land-use plans, the gage should be located near the site of the structure or plan. Gages used as part of a general-purpose hydrologic network may be variously placed to provide information on a range of drainage-basin types and sizes.

The specific location of a stream gage depends on (1) accessibility; (2) minimization of flow bypassing the section as ground water beneath the channel or in flood channels at high flows; (3) absence of

TABLE F-1
Classification of Stream-Gaging Methods.

I. Direct measurement
  A. Volumetric gaging
  B. Velocity–area gaging
  C. Dilution gaging
II. Indirect measurement via stage–discharge relation
  A. Empirical rating curve (natural control)
  B. Theoretical rating curve (artificial control)
    1. Weirs
    2. Flumes

backwater conditions due to high water levels in a stream or lake to which the gaged stream is tributary, or to tidal fluctuations; and (4) the presence of conditions suitable for the particular measurement method to be used.

F.2 VELOCITY-AREA METHOD

The **velocity-area method** involves direct measurement of the components of discharge [Equation (F-1)] at successive locations (called **verticals**) along a stream cross-section and numerical integration of the measured values to give the total discharge.

Figure F-1 defines the quantities involved. The more verticals used in a measurement (N), the more accurate it will be (other things equal); $N \geq 25$ is usually recommended, with the spacing of verticals adjusted such that < 5% of the total discharge occurs in each subsection. Thus, in general, spacings between verticals will vary, being closer together where the flow is deeper and faster and farther apart where the flow is slower and shallower.

Velocity at each vertical (U_i) is measured by a current meter suspended from a rod or cable, depth (Y_i) is measured by rod or weighted cable, and cross-section location (X_i) by measurement tape or range finder.

F.2.1 Selection of Measurement Section

The quality of a velocity-area measurement is strongly influenced by the nature of the measurement cross-section. Accuracy and precision are enhanced in sections with the following characteristics:

1. converging flow (i.e., cross-sectional area decreasing downstream) without areas of near-zero velocity or eddies,
2. absence of backwater conditions (due, for example, to high water levels in a stream or lake to which the gaged stream is tributary),
3. smooth cross-section with minimal flow obstructions upstream or downstream, and
4. velocities and depths not exceeding the range for which the velocity- and depth-measuring devices give accurate results and for which one can safely negotiate the section.

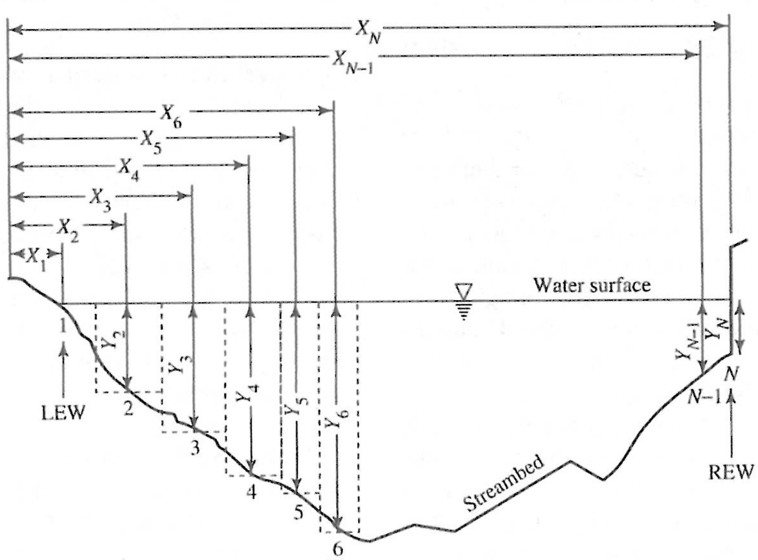


FIGURE F-1
Definitions of terms for measurement of discharge by the velocity–area method [Equations (F-3) and (F-4)]. Dashed lines delineate individual subsections, numbered consecutively $i = 1, 2, \ldots, N$. Left and right edges of water (LEW and REW) are defined for an observer facing *downstream*; sections can be numbered starting on left or right bank.

The accuracy of measurements at natural sections can be improved by removing obstructions in and above and below the section, since this will not affect discharge. If this is done, however, the measured velocities and depths will not be representative of the natural values.

Cross-sections may be negotiated by wading, by boat, or from bridges or specially constructed cableways (Buchanan and Somers 1969; Herschy 1999). In wading measurements, the current meter is fixed to a hand-held vertical rod (called a **wading rod**) that is also used to gage the depth. In measurements from bridges, cableways, and boats, the current meter is suspended on a weighted cable and depth is measured by sounding the bottom with the weight.

In wading measurements (Figure F-2), the location of the cross-section is usually marked by a measuring tape, and the location of a vertical is measured against the tape. For small streams, the accuracy measurements can be improved by making them from a specially constructed temporary wooden bridge on which cross-stream distances are permanently marked (Figure F-3).

Cross-sections should be perpendicular to the dominant velocity vector, and all velocities used in the computation should be perpendicular to the section. Where the vector is not perpendicular to the section at a particular vertical, the perpendicular velocity should be measured or the velocity used in the computations must be corrected by multiplying by the cosine of the angle between the perpendicular and the actual velocity vector.

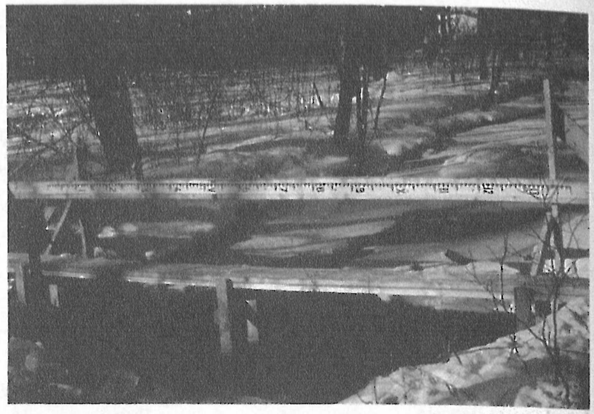


FIGURE F-3
Temporary structure used in velocity–area gaging via wading rod. Note the cross-section distances permanently fixed to the structure. Photo by author.

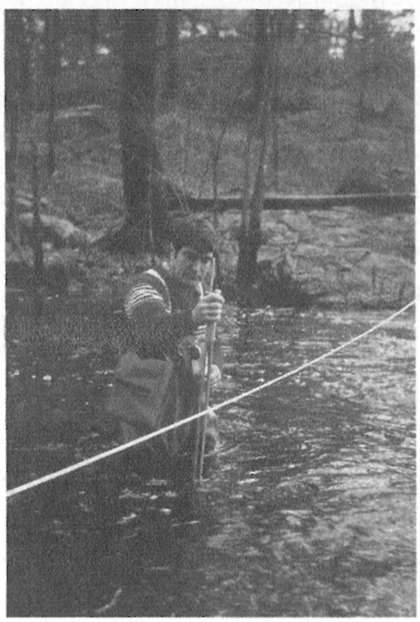


FIGURE F-2
Velocity–area stream gaging using a wading rod. The tape marking the cross section is visible just in front of the observer. Photo by J.V.Z. Dingman.

F.2.2 Method of Integration

Discharge through the cross-section is given by

$$Q = \sum_{i=1}^{N} U_i \cdot A_i, \qquad \text{(F-3)}$$

where the area of each subsection, A_i, is computed as

$$A_i = Y_i \cdot \frac{|X_{i+1} - X_{i-1}|}{2}, \qquad \text{(F-4)}$$

the X_i are cross-stream distances to successive verticals measured from an arbitrary horizontal datum, the U_i are the vertically averaged velocities at each vertical, and the Y_i are the depths at each vertical. X_1 and X_N are located at the ends of the section (left and right edges of water[1]), and X_{N-1} and X_{N+1} are taken as zero. The velocities at the ends of the section (U_1 and U_N) will always equal zero, and Y_1

[1] "Left" and "right" edges of water are defined relative to an observer facing *downstream*.

and Y_N will also equal zero (unless the corresponding bank is vertical).

Equation (F-4) is called the **mid-section method,** and it is the method used by the U.S. Geological Survey, which is the federal agency responsible for collecting streamflow data in the United States. It has been shown (Hipolito and Loureiro 1988) that integration using Equation (F-4) gives the most precise measurements.

F.2.3 Measurement of Velocity

Vertical Velocity Profile

The vertically averaged velocity is usually estimated by assuming that the velocity is logarithmically related to distance above the bottom, as for wind flow over the ground surface [Equation (D-22)]:

$$u(y_i) = \frac{1}{k} \cdot u_{*i} \cdot \ln\left(\frac{y_i}{y_{0i}}\right). \tag{F-5}$$

Here $u(y_i)$ is velocity at a distance y_i above the bottom at vertical i, u_{*i} is the friction velocity at vertical i, and y_{0i} is the roughness height at vertical i. (The zero-plane displacement height is taken as zero in open-channel flows.) The derivation of Equation (F-5) was discussed by Dingman (1984).

The local friction velocity for water flow can be directly calculated as

$$u_{*i} = (g \cdot Y_i \cdot S_c)^{1/2}, \tag{F-6}$$

where g is gravitational acceleration, Y_i is local flow depth, and S_c is channel slope.

The value of y_{0i} is determined by the height of the roughness elements on the channel bed, which can be represented by the median diameter of the bed particles, d_{50}. If

$$d_{50} < \frac{4 \cdot \mu}{\rho \cdot u_{*1}}, \tag{F-7}$$

where μ is viscosity and ρ is mass density, then the flow is **hydraulically smooth,** and

$$y_{0i} = \frac{0.11 \cdot \mu}{\rho \cdot u_{*i}} \tag{F-8}$$

Where the criterion of Equation (F-7) is not fulfilled, the flow is **hydraulically rough,** and

$$y_{0i} \approx 0.033 \cdot d_{50}. \tag{F-9}$$

Estimating Average Velocity in a Vertical

Six-Tenths-Depth Method If Equation (F-5) applies and y_{0i} is very small relative to the flow depth, it can be shown that the average velocity occurs at a distance of $0.368 \cdot Y_i$ above the bottom (Dingman 1984). Based on this, the **Six-Tenths-Depth Method** assumes that the velocity measured at a distance of $0.6 \cdot Y_i$ below the surface ($0.4 \cdot Y_i$ above the bottom) is the average velocity at that point in the cross-section.

Standard U.S. Geological Survey practice is to use the Six-Tenths-Depth Method where $Y_i < 2.5$ ft (0.75 m).

Two-Tenths-and-Eight-Tenths-Depth Method If the velocity is given by Equation (F-5), it can be shown that

$$u(0.4 \cdot Y_i) = \frac{u(0.2 \cdot Y_i) + u(0.8 \cdot Y_i)}{2}. \tag{F-10}$$

Thus average vertical velocity can be estimated as the average of the velocities at $0.2 \cdot Y_i$ and $0.8 \cdot Y_i$.

It has been found that the Two-Tenths-and-Eight-Tenths-Depth Method gives more accurate estimates of average velocity than does the Six-Tenths-Depth Method (Carter and Anderson 1963), and standard U.S. Geological Survey practice is to use the Two-Tenths-and-Eight-Tenths-Depth Method where $Y_i > 2.5$ ft (0.75 m).

General Two-Point Method If velocity is measured at two points, each an arbitrary fixed distance above the bottom, the relative depths of those sensors will change as the discharge changes. Again assuming a logarithmic vertical velocity distribution, Walker (1988) derived the following expression for calculating the average vertical velocity from two sensors fixed at arbitrary distances above the bottom, y_{i1} and y_{i2}, where $y_{i2} > y_{i1}$:

$$U_i = \frac{[1 + \ln(y_{i\,2})] \cdot u(y_{i\,1}) + [1 + \ln(y_{i\,1})] \cdot u(y_{i\,2})}{\ln(y_{i\,2}/y_{i\,1})}. \tag{F-11}$$

Walker (1988) also calculated the error in estimating U_i for sensors located at various combinations of relative depths.

Multi-Point Method As noted, the standard formulas for calculating average vertical velocity assume a logarithmic vertical velocity distribution with $y_{0i} << Y_i$. These assumptions may not be appropriate for channels with roughness elements (boulders, aquatic vegetation) whose heights are a significant fraction of depth and which have significant obstructions upstream and downstream of the measurement section. In these cases, Buchanan and Somers (1969) recommended estimating U_i as

$$U_i = 0.5 \cdot u_i(0.4 \cdot Y_i) + 0.25 \cdot [u_i(0.2 \cdot Y_i) + u_i(0.8 \cdot Y_i)]. \quad \textbf{(F-12)}$$

However, the highest accuracy in these situations is assured by measuring velocity several heights at each vertical, with averages found by numerical integration over each vertical or over the entire cross section. Alternatively, a statistical sampling approach may be appropriate (Dingman 1989).

Surface-Velocity (Float) Method If it is not possible to accurately measure velocities at various depths, the average velocity can be estimated by observing the time it takes floats inserted at representative locations across the stream to travel a given distance. The measurement distance should be about 10 times the stream width.

The average velocity for each path can then be estimated as

$$U_i = f\left(\frac{d_{50}}{Y_i}\right) \cdot u(Y_i), \quad \textbf{(F-13)}$$

where $u(Y_i)$ is the velocity of the ith float and $f(d_{50}/Y_i)$ is a proportion that depends on the ratio of the average height of channel roughness elements, d_{50}, to the flow depth. Figure F-4 shows this relation, again assuming a logarithmic velocity profile. A reasonable general value is $f(d_{50}/Y_i) = 0.85$.

Current Meters

Several types of current meters are available, including horizontal-axis (propeller or screw type), vertical-axis (Price-type; this is standard for the U.S. Geological Survey), and electromagnetic instruments. Accurate measurements require carefully calibrated and maintained current meters (Smoot and Novak 1968) and, at each measurement point, averaging the velocity over time (usually 30 to 60 s) to eliminate fluctuations due to turbulence.

F.2.4 Accuracy

Carter and Anderson (1963) evaluated the accuracy of velocity-area measurements using standard U.S. Geological Survey techniques. They determined discharge from measurements of velocity and depth at over 100 verticals in 127 cross-sections in different streams, then recomputed the discharge using smaller numbers of observations at each site. Care-

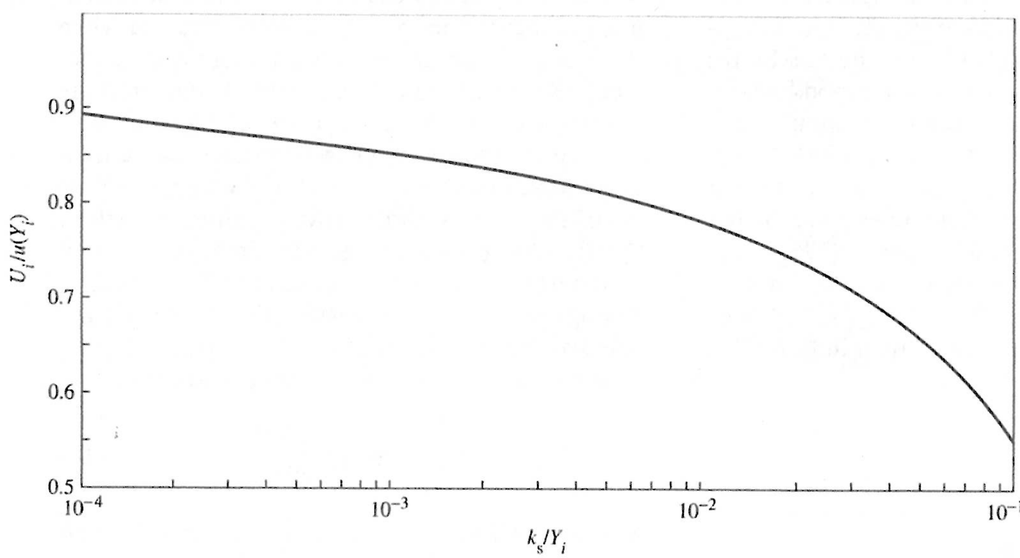


FIGURE F-4
Theoretical ratio of average velocity, U_i, to surface velocity, $u(Y_i)$, as a function of the ratio of roughness height, d_{50}, to depth, Y_i, for a logarithmic velocity profile.

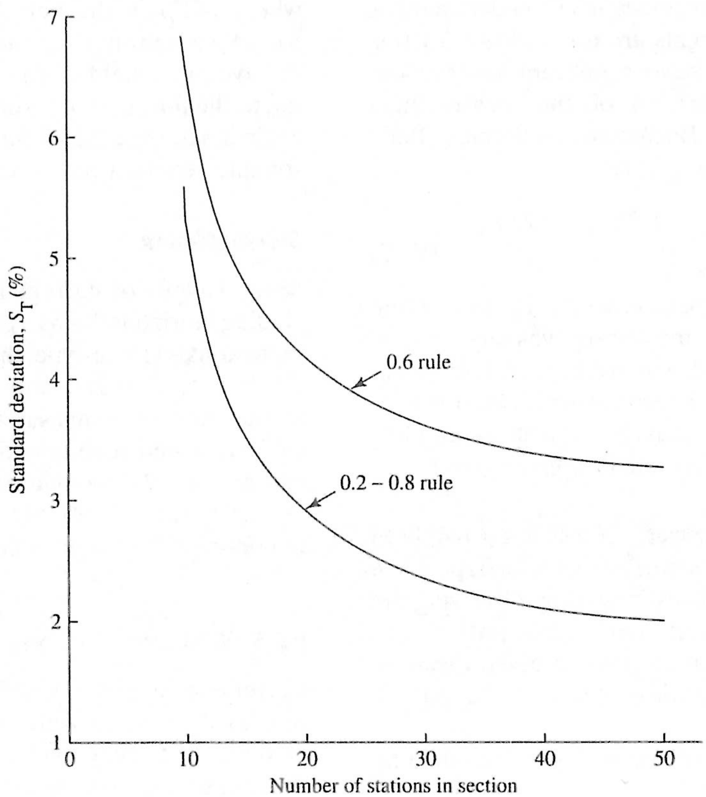


FIGURE F-5
Standard deviation of percentage errors as a function of number of verticals used in velocity–area measurements using the Six-Tenths-Depth Method and the Two-Tenths-and-Eight-Tenths-Depth Method. From Carter and Anderson (1963).

fully calibrated current meters were used in these observations.

The results are summarized in Figure F-5: They show the standard deviation of the error as a function of number of verticals (N) for the Six-Tenths and Two-Tenths-and-Eight-Tenths methods. About two-thirds of discharge measurements should have percentage errors less than the values given by the curves, and about 95% of measurements should have errors less than twice the values given by the curves. Although Carter and Anderson (1963) gave no information about the nature of the streams they measured, it is likely that errors for highly irregular channels would be larger than indicated by Figure F-5.

F.3 DILUTION GAGING

Dilution gaging involves introducing a tracer into the flow at an upstream location and measuring the rate of arrival of the tracer at a downstream location. It is usually a more accurate method than velocity-area gaging in small, highly turbulent streams with rough, irregular channels. As with velocity-area measurements, the method is most accurate when there is no change in discharge during the measurement. Kilpatrick and Cobb (1985) and Herschy (1999) gave a complete discussion of the method.

The distance between the two measurement locations must be long enough to allow complete mixing of the tracer with the flow, but short enough so that the change in discharge is insignificant. It is best to determine the distance required for complete mixing empirically, by observing the mixing of a visible dye such as fluorescein in the reach of interest. Alternatively, this length, L_{mix}, may be estimated as

$$L_{mix} = K_{mix} \cdot \frac{C \cdot B^2}{g^{1/2} \cdot Y}, \qquad \textbf{(F-14)}$$

where C is Chézy's C [estimated from Table 9-6 and Equation (9-20)], B is average reach width, Y is average reach depth, g is gravitational acceleration, and K_{mix} is found from Table F-2 (Kilpatrick and Cobb 1985).

TABLE F-2
Mixing Coefficients for Dilution Gaging.

Number and Location of Injection Points	K_{mix}
1 point at center of flow	0.500
2 points, 1 at center of each half of flow	0.125
3 points, 1 at center of each third of flow	0.055
1 point at edge of flow	2.00

From Kilpatrick and Wilson (1989)

Gregory and Walling (1973) summarized the requirements for a tracer. The substance should (1) be readily soluble, (2) have zero or very low natural concentration in the stream, (3) not be chemically reactive with or physically absorbed by substances in the stream, (4) be easily detectable at low concentrations, (5) be harmless to the observer and to stream life, and (6) be of reasonable cost. Sodium chloride (NaCl) is probably the best choice in most situations; its concentration can usually be readily detected by developing a calibration curve between electrical conductivity and concentration in the stream water and subsequently measuring the conductivity.

There are two techniques for dilution gaging: (1) constant-rate injection and (2) slug (or gulp) injection (Figure F-6). Constant-rate injection requires a somewhat more elaborate installation than slug injection. The tracer solution is injected at a constant rate, Q_T, for a period of time sufficient for the downstream concentration to reach a steady equilibrium value, C_{eq}. Then the discharge, Q, is calculated as

$$Q = Q_T \cdot \frac{C_T - C_{eq}}{C_{eq} - C_b}, \qquad \textbf{(F-15)}$$

where C_T is the concentration of the tracer solution and C_b is the natural background concentration of the tracer in the stream.

Slug injection involves dumping a volume, V_T, of tracer solution with concentration C_T into the stream at the upstream site. Concentration at the downstream site, $C_d(t)$, is then measured as a function of time until it recedes to its background value C_b. Stream discharge is then given by

$$Q = \frac{(C_T - C_b) \cdot V_T}{\int_0^{\infty} [C_d(t) - C_b] \cdot dt}, \qquad \textbf{(F-16)}$$

where the integral is evaluated by graphically measuring the area under the $C_d(t)$ vs. t curve.

F.4 SHARP-CRESTED V-NOTCH WEIRS

Accurate discharge measurements in small streams can be made using portable V-notch weirs in temporary installations (Figure F-7). The weir plate can be constructed of plywood or metal. The notch should be sharp-edged so that the water "springs free" even at low discharges. Thus, if plywood is used, the notch itself should be formed of metal strips.

The weir should be installed in the stream such that all the flow is diverted through it, and such that a virtually horizontal pool extends some distance upstream. The weir plate must be carefully leveled so that the V-notch is symmetric about a vertical line.

Discharge through a weir is determined by measuring the elevation of the water surface, Z_w, above the point of the V-notch, Z_v, and relating that elevation to discharge. Thus precise measurement of Z_w is essential.

Z_w should be measured where the water surface is virtually horizontal in the pool formed by the weir; this should ideally be at an upstream distance at least twice the vertical dimension of the weir opening. However, acceptably approximate Z_w measurements may be obtained by observing the water level on a scale fixed to the upstream face of the weir plate far enough from the notch to avoid the effects of drawdown. This latter arrangement eliminates the need for precise leveling to establish the relation between elevations observed upstream and the elevation of the point of the V-notch.

The relation between Z_w and discharge, Q, is given by

$$Q = C_w \cdot g^{1/2} \cdot \tan\left(\frac{\theta_v}{2}\right) \cdot (Z_w - Z_v), \qquad \textbf{(F-17)}$$

where g is gravitational acceleration, θ_v is the angle of the V-notch, Z_v is the elevation of the base of the V-notch, and C_w is a weir coefficient. When $(Z_w - Z_v) > 0.3 \cdot (Z_v - Z_b)$, where Z_b is the elevation of the streambed at the upstream face of the weir, $C_w = 0.43$. Values of C_w for smaller values of

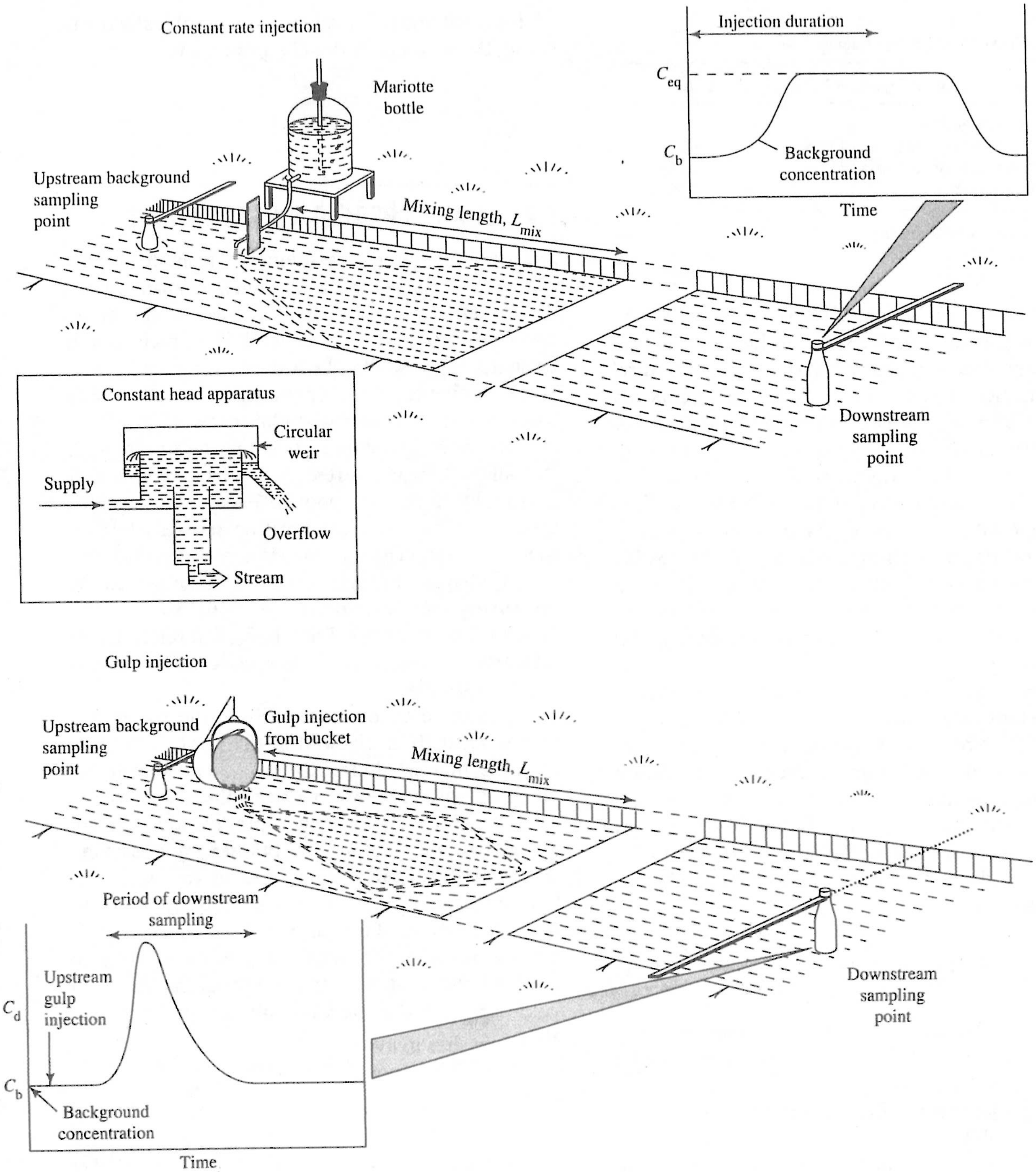
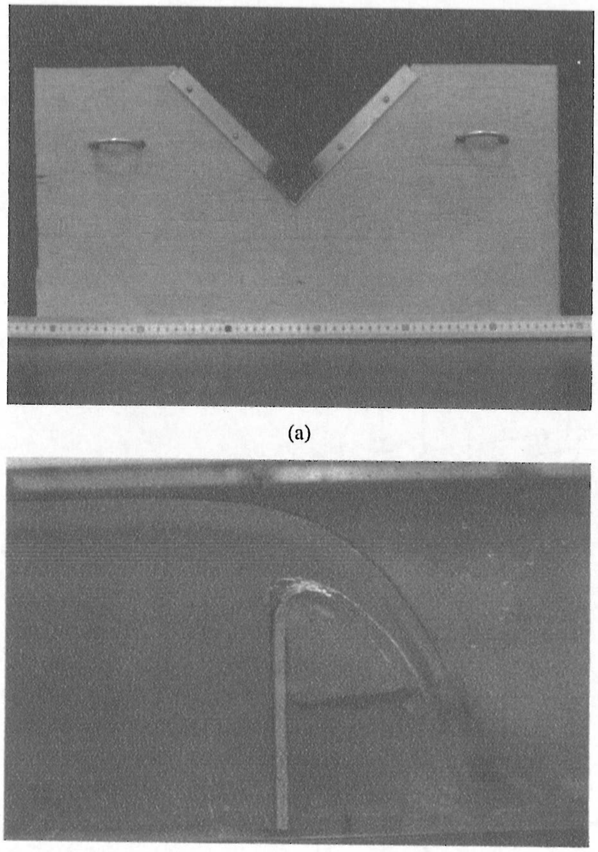


FIGURE F-6
Dilution-gaging techniques. From Gregory and Walling (1973).



(a)

(b)

FIGURE F-7
(a) Construction of a V-notch weir in a plywood weir plate. (b) The notch should have a sharp edge so that the water springs free even at low discharges. Photos by author.

TABLE F-3
Ranges of Flows Measurable by V-Notch Weirs.[a]

	Maximum Q		
	($m^3\ s^{-1}$)	($ft^3\ s^{-1}$)	($L\ s^{-1}$)
	90° Notch		
Maximum H_w (m)			
0.15	0.0117		11.7
0.25	0.0421		42.1
0.30	0.0664		66.4
0.50	0.238		238
Maximum H_w (ft)			
0.50		0.431	
0.80		1.40	
1.00		2.44	
1.50		6.72	
	60° Notch		
Maximum H_w (m)			
0.15	0.00677		6.77
0.25	0.0243		24.3
0.30	0.0383		38.3
0.50	0.137		137
Maximum H_w (ft)			
0.50		0.249	
0.80		0.806	
1.00		1.41	
1.50		3.88	

[a] $H_w = Z_w - Z_v$ in Equation (F-17).

$(Z_w - Z_v)$ should ideally be obtained by calibration. [See Dingman (1984); Herschy (1999).]

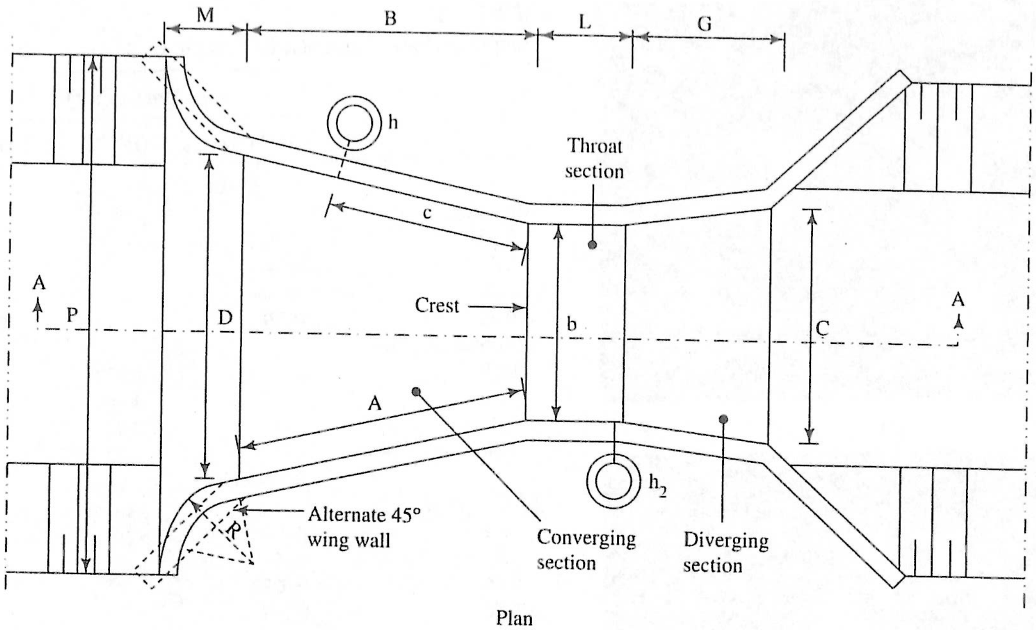
The maximum vertical opening and angle of a V-notch weir are dictated by the anticipated range of flows and the required measurement precision. Narrower angles give greater precision at low flows, but reduce the overall range of measurement for a given size of opening. Table F-3 gives the capacity of 90° and 60° V-notch weirs with various maximum dimensions. Weirs with compound angles can be constructed to give more optimal combinations of low-flow sensitivity and range; these must be calibrated to obtain the stage–discharge relation.

F.5 FLUMES

Flumes are devices that conduct the streamflow through a short reach with a constricted cross-section that accelerates the flow and provides a fixed stage–discharge relation. Large flumes are used in permanent gaging stations, and portable flumes can be used for short-term measurements in small streams. Both flumes and weirs give stable rating curves, but flumes are advantageous where one wishes to avoid inducing sediment deposition or inundating upstream areas.

There are many flume designs, each with its own rating curve (Herschy 1999). Some types may be commercially purchased. Figure F-8 and Tables F-4 and F-5 show the design and ranges of one of the most commonly used portable designs, the **Parshall flume;** and Figure F-9 gives the design of the **modified Parshall flume** used by the U.S. Geological Survey for temporary gaging of small streams.

As with weirs, flumes must be properly leveled, and are usually installed in temporary dams



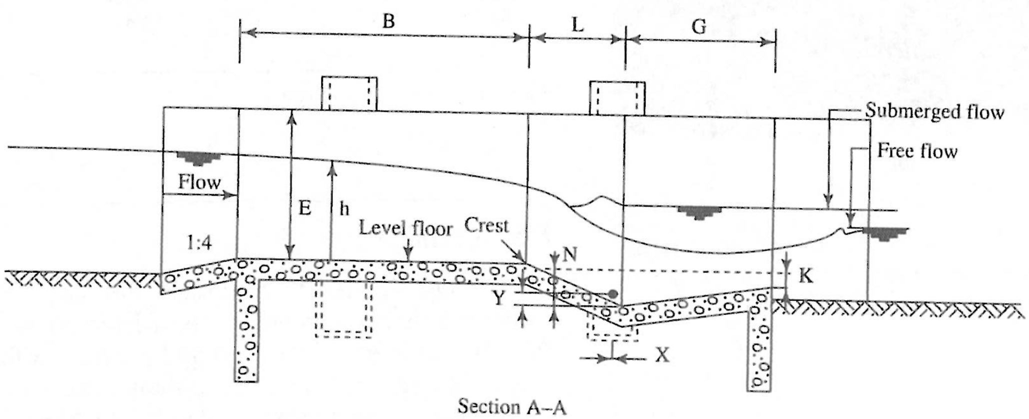


FIGURE F-8
Configuration of the Parshall flume. Actual dimensions corresponding to letters are given in Table F-4 and discharge ranges in Table F-5. From Herschy (1985).

that assure that all the flow passes through the measuring device.

F.6 STAGE MEASUREMENT

Direct measurement of discharge is difficult and time-consuming, and recording it is impossible; however, observing and recording water-surface elevation, or stage [Equation (F-2)], is relatively simple. Thus the stage–discharge relation (rating curve) is an essential component of discharge measurement for which repeated or continuous records are required.

F.6.1. Methods of Measurement

Stage is most simply determined by observing the position of the water surface on a ruler-like **staff gage** (Figure F-10). The zero level on the staff gage

TABLE F-4
Dimensions of Standard Parshall Flumes[a]

											h_2	
b	D	C	B	L	G	E	N	K	A	c	X	Y
0.025	0.167	0.093	0.357	0.076	0.204	0.153–0.229	0.029	0.019	0.363	0.241	0.008	0.013
0.051	0.213	0.135	0.405	0.114	0.253	0.153–0.253	0.043	0.022	0.415	0.277	0.016	0.025
0.076	0.259	0.178	0.457	0.152	0.30	0.305–0.610	0.057	0.025	0.466	0.311	0.025	0.038
0.152	0.396	0.393	0.610	0.30	0.61	0.61	0.114	0.076	0.719	0.415	0.051	0.076
0.229	0.573	0.381	0.862	0.30	0.46	0.76	0.114	0.076	0.878	0.588	0.051	0.076
0.305	0.844	0.610	1.34	0.61	0.91	0.91	0.228	0.076	1.37	0.914	0.051	0.076
0.457	1.02	0.762	1.42	0.61	0.91	0.91	0.228	0.076	1.45	0.966	0.051	0.076
0.610	1.21	0.914	1.50	0.61	0.91	0.91	0.228	0.076	1.52	1.01	0.051	0.076
0.914	1.57	1.22	1.64	0.61	0.91	0.91	0.228	0.076	1.68	1.12	0.051	0.076
1.22	1.93	1.52	1.79	0.61	0.91	0.91	0.228	0.076	1.83	1.22	0.051	0.076
1.52	2.30	1.83	1.94	0.61	0.91	0.91	0.228	0.076	1.98	1.32	0.051	0.076
1.83	2.67	2.13	2.09	0.61	0.91	0.91	0.228	0.076	2.13	1.42	0.051	0.076
2.13	3.03	2.44	2.24	0.61	0.91	0.91	0.228	0.076	2.29	1.52	0.051	0.076
2.44	3.40	2.74	2.39	0.61	0.91	0.91	0.228	0.076	2.44	1.62	0.051	0.076
3.05	4.75	3.66	4.27	0.91	1.83	1.22	0.34	0.152	2.74	1.83		
3.66	5.61	4.47	4.88	0.91	2.44	1.52	0.34	0.152	3.05	2.03		
4.57	7.62	5.59	7.62	1.22	3.05	1.83	0.46	0.229	3.50	2.34		
6.10	9.14	7.31	7.62	1.83	3.66	2.13	0.68	0.31	4.27	2.84		
7.62	10.67	8.94	7.62	1.83	3.96	2.13	0.68	0.31	5.03	3.35		
9.14	12.31	10.57	7.92	1.83	4.27	2.13	0.68	0.31	5.79	3.86		
12.19	15.48	13.82	8.23	1.83	4.88	2.13	0.68	0.31	7.31	4.88		
15.24	18.53	17.27	8.23	1.83	6.10	2.13	0.68	0.31	8.84	5.89		

[a]Letters refer to dimensions in Figure F-8. All values in meters.

From Herschy (1985).

may be taken as the datum if it is below the level of zero discharge.

Stage can also be determined via a float–counterweight system (Figure F-11) and with instruments that directly measure water pressure (pressure transducers).

Whatever system is used, one should always establish by survey the elevation of the gage relative to some point whose elevation will not change, and periodically check that elevation to avoid errors due to disturbance of the staff gage.

Continuous records of discharge are obtained by recording stage by means of a float or pressure sensor attached to an analog or digital recorder and using the rating curve to convert to discharge. Buchanan and Somers (1968) and Herschy (1999) described many approaches to measuring and recording stage.

F.6.2 Measurement Location

To establish a useful stage–discharge relation, stage must be measured within a few stream widths of the discharge-measurement cross-section. Stage should be observed where it is sensitive to discharge variations (i.e., where dZ_s/dQ is relatively large) and where it can be accurately measured (usually to within 0.01 ft or 0.003 m). These conditions are usually met in an area of quiet water not far upstream of a reach of accelerating flow and near the bank, where wave action is minimal.

If necessary, wave action can be virtually eliminated by measuring stage in a **stilling well.** Such a device can be constructed by surrounding the staff gage with a section of metal or plastic barrel into which holes have been punched, or simply with piled rocks. Construction of more elaborate stilling wells dug into stream banks and communicating to the stream via pipes is discussed by Buchanan and Somers (1968) and Herschy (1999).

F.6.3 Stage–Discharge Relations at Natural Controls

When artificial controls are used the rating curve has a theoretical form [e.g., Equation (F-17)], although calibration of the relation by direct measurements may be required over at least some flow ranges. For natural controls, the rating curve is es-

TABLE F-5
Discharge Characteristics of Parshall Flumes

Throat width b (m)	Discharge Range ($m^3 s^{-1} \times 10^{-3}$) Minimum	Maximum	Equation $Q = Kh^u$ ($m^3 s^{-1}$)
0.025	0.09	5.4	$0.0604h^{1.55}$
0.051	0.18	13.2	$0.1207h^{1.55}$
0.076	0.77	32.1	$0.1771h^{1.55}$
0.152	1.50	111	$0.3812h^{1.58}$
0.229	2.50	251	$0.5354h^{1.53}$
0.305	3.32	457	$0.6909h^{1.522}$
0.457	4.80	695	$1.056h^{1.538}$
0.610	12.1	937	$1.428h^{1.550}$
0.914	17.6	1427	$2.184h^{1.566}$
1.219	35.8	1923	$2.953h^{1.578}$
1.524	44.1	2424	$3.732h^{1.587}$
1.829	74.1	2929	$4.519h^{1.595}$
2.134	85.8	3438	$5.312h^{1.601}$
2.438	97.2	3949	$6.112h^{1.607}$
3.048	0.16[a]	8.28[a]	$7.463h^{1.60}$
3.658	0.19[a]	14.68[a]	$8.859h^{1.60}$
4.572	0.23[a]	25.04[a]	$10.96h^{1.60}$
6.096	0.31[a]	37.97[a]	$14.45h^{1.60}$
7.620	0.38[a]	47.14[a]	$17.94h^{1.60}$
9.144	0.46[a]	56.33[a]	$21.44h^{1.60}$
12.192	0.60[a]	74.70[a]	$28.43h^{1.60}$
15.240	0.75[a]	93.04[a]	$35.41h^{1.60}$

[a]Values in $m^3 s^{-1}$.

From Herschy (1985)

tablished empirically by concurrent direct measurement of discharge and observation of stage over a range of discharges.

At any cross-section, there is usually a relation between discharge, Q, and stage, Z_s, of the form

$$Q = a \cdot Z_s^b, \tag{F-18}$$

where a and b are determined by the configuration of the stream reach, the exact location at which stage is measured, and the datum for Z_s. The datum for stage measurement, Z_0, [Equation (F-2)] is simply a convenient elevation below the elevation corresponding to zero discharge. The values of a and b in Equation (F-18) typically vary for different ranges of Z_s in natural streams, and the rating curve must be empirically determined by measurement of Z_s and Q over the range of interest.

Ideally, stage should be measured where (1) hysteresis in the stage–discharge relation is minimal and (2) the relation will not change with time due to erosion or deposition in the measurement reach. However, these conditions may be difficult to find in many streams. Figure F-12 shows a rating curve where shifts have occurred; this demonstrates that one must continually check the rating curve by frequent discharge measurements.

F.7 SLOPE-AREA MEASUREMENTS

It is often possible to estimate the discharge of a recent flood peak from observations of high-water marks in a reach. The procedure involves surveying the slope of the water surface as revealed by the high-water marks and the cross-sectional area beneath the marks, and hence is known as the **slope-area method.**

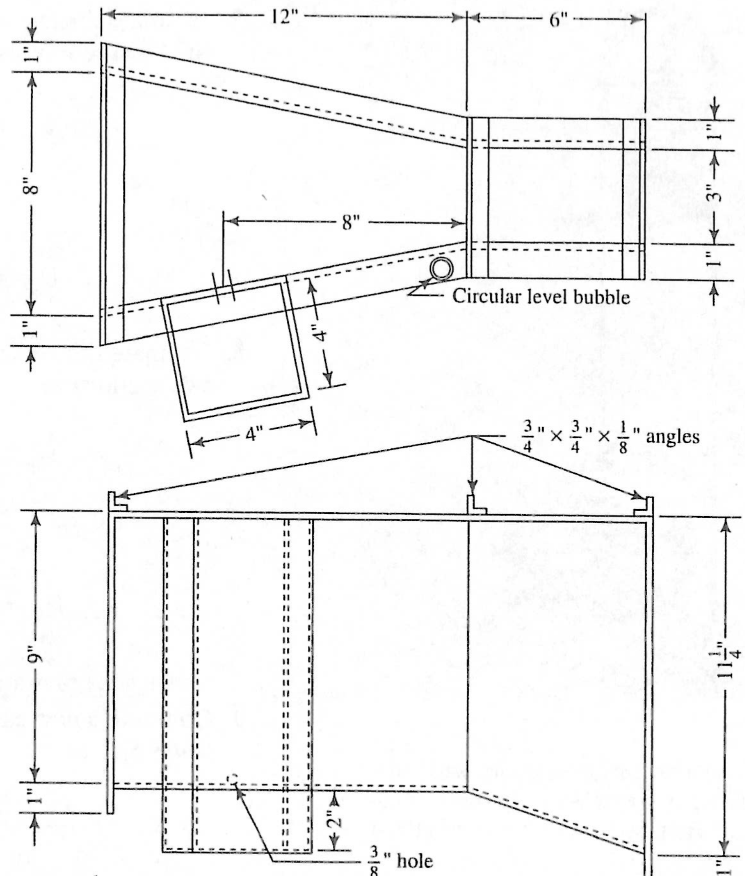
F.7.1 Standard Method

In the standard slope-area method, an estimated channel-resistance value is used along with the surveyed cross-sectional dimensions in the equation for uniform flow [Equation (9-18) or (9-19)] to compute the discharge. Dalrymple and Benson (1967) described the field measurements needed to apply the method and noted that the selection of a suitable reach (distance between upstream and downstream cross-sections) is the most important element of a slope-area measurement. The presence of unequivocal high-water marks is essential, and the stream reach should be relatively straight and uniform without pronounced constrictions, protrusions, or free overfalls. The accuracy of the method generally increases with reach length, and the reach should meet one or more of the following criteria: (1) reach length at least 75 times flow depth, (2) vertical water-surface fall through the reach greater than 0.15 m, and (3) water-surface fall greater than computed velocity heads.

In the following steps, we assume that we have information on two cross-sections; the subscript u designates the upstream cross-section, and d the downstream cross-section. These steps follow the approach described by Ponce (1989). Dalrymple and Benson (1967) also gave equations for applying the method using more than two cross-sections.

1. Survey the upstream and downstream cross-sections below the high-water marks and determine the cross-sectional areas (A_u, A_d) and hydraulic radii (R_u, R_d) for each,

FIGURE F-9
Configuration of a 3-inch modified Parshall flume. Its maximum discharge capacity is about 0.5 ft^3 s^{-1} (0.014 m^3 s^{-1} = 14 L s^{-1}). From Buchanan and Somers (1968).



Material: $\frac{1}{8}$-in. aluminum
Welded construction
Note: This stilling well can accommodate a 3-in. float and be used with a recorder if continuous measurement is desired for a period.

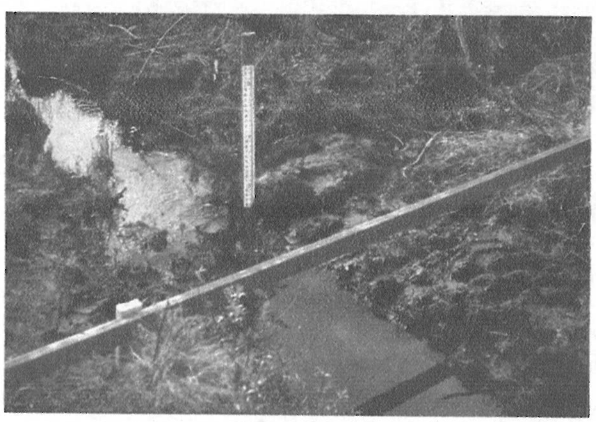


FIGURE F-10
Staff gage installed on a small stream. Photo by author.

the distance between the sections (L), and the elevation difference between the high-water marks at the two sections (Δz).

2. Estimate the Manning resistance factor (n) for the reach from Table 9-6 or by comparison with photographs (Barnes 1967; Hicks and Mason 1991).
3. Compute the section **conveyances,** K_u and K_d, as

$$K_u \equiv \frac{u_m \cdot R_u^{2/3} \cdot A_u}{n}, \quad \textbf{(F-19a)}$$

and

$$K_d \equiv \frac{u_m \cdot R_d^{2/3} \cdot A_d}{n}, \quad \textbf{(F-19b)}$$

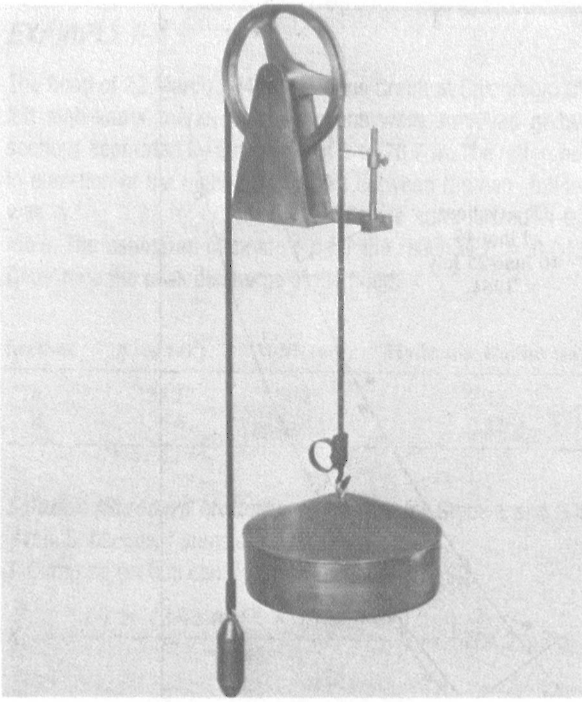


FIGURE F-11
Simple float–counterweight system for measuring water-surface elevation. The pulley can be attached to a recorder to obtain a continuous record. From Buchanan and Somers (1968).

where u_m is the unit-conversion factor for the Manning equation (Table 9-6).

4. Compute the average reach conveyance, K, as the geometric mean of the section conveyances:

$$K = (K_u \cdot K_d)^{1/2}. \quad \text{(F-20)}$$

An iterative procedure is now used to compute Q. Successive iterations are denoted in the following steps by subscripts $i = 1, 2, \ldots$ Usually no more than five iterations are required for convergence to an effectively constant value.

5. Compute the first estimate of the **friction slope,** S_1, as

$$S_1 = \frac{\Delta z}{L}. \quad \text{(F-21)}$$

6. Compute the first estimate of the discharge, Q_1, as

$$Q_1 = K \cdot S_1^{1/2}. \quad \text{(F-22)}$$

7. Estimate the average velocities (U_{ui}, U_{di}) at the two sections as

$$U_{ui} = \frac{Q_1}{A_u} \quad \text{(F-23a)}$$

and

$$U_{di} = \frac{Q_1}{A_d}. \quad \text{(F-23b)}$$

8. Estimate the **velocity heads** (h_{ui}, h_{di}) at the two sections as

$$h_{ui} = \frac{U_{ui}^2}{2 \cdot g} \quad \text{(F-24a)}$$

and

$$h_{di} = \frac{U_{di}^2}{2 \cdot g}. \quad \text{(F-24b)}$$

where g is gravitational acceleration.

9. Compute a new estimate of the friction slope, S_{i+1}, as

$$S_{i+1} = \frac{\Delta z + k \cdot (h_{ui} - h_{di})}{L}, \quad \text{(F-25)}$$

where k is an **eddy coefficient,** empirically found to be given by

$$k = 0.5 \text{ if } A_d > A_u \quad \text{(F-26a)}$$

and

$$k = 1 \text{ if } A_d \leq A_u. \quad \text{(F-26b)}$$

10. Compute a new estimate of the discharge, Q_{i+1}, as

$$Q_{i+1} = K \cdot S_{i+1}^{1/2}. \quad \text{(F-27)}$$

11. Compare Q_{i+1} with Q_i and repeat steps 7–11 until successive discharge estimates do not change.

Herschy (1985) indicated that the usual range of error with the slope-area method is 10 to 20%, and Kirby (1987) gave a complete analysis of the errors involved in slope-area estimates. Accuracy is improved by careful selection of the measurement reach and by using more cross-sections, but is inher-

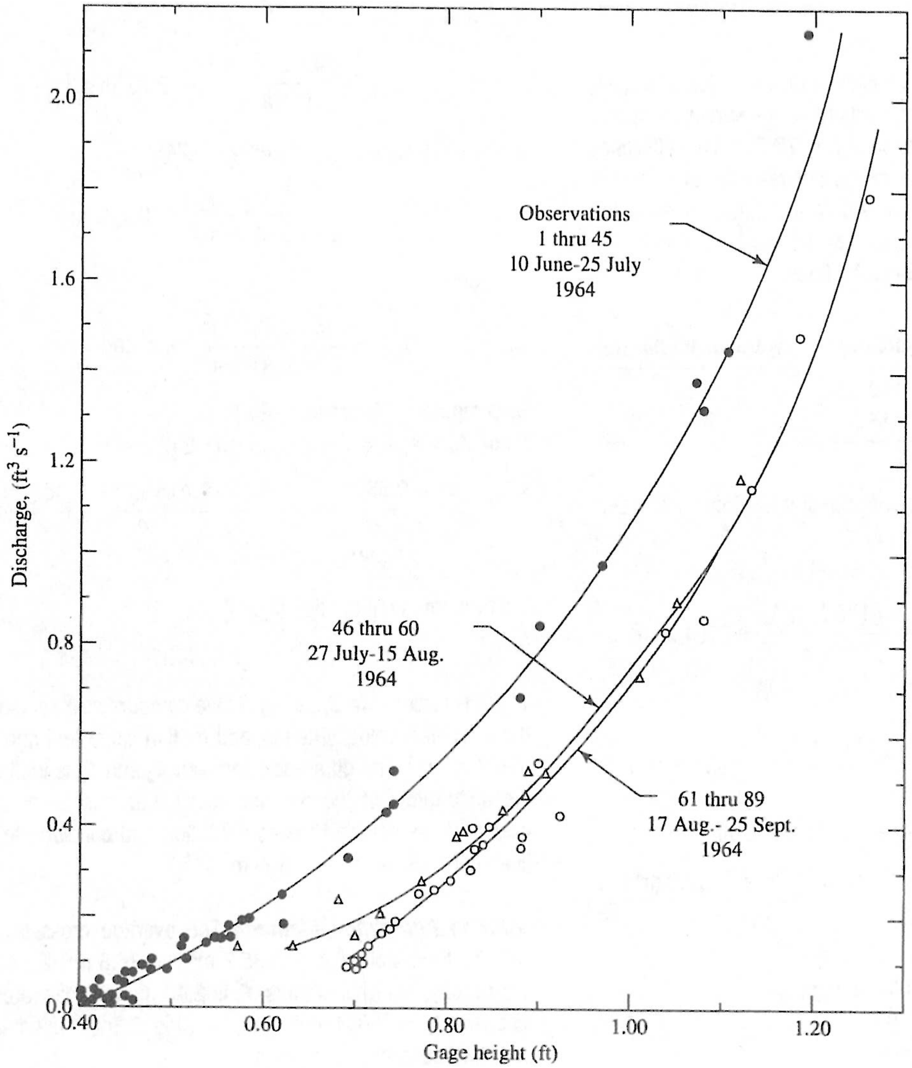


FIGURE F-12
Rating curve for a natural-cross section in a small stream in Alaska. Note that the relation has shifted with time due to siltation of the channel. From Dingman (1970).

ently limited by the need to estimate the channel resistance (n) and the fact that the flow is seldom truly uniform.

F.7.2 Simplified Method

Based on statistical analysis of over 600 measured flows in natural channels, Dingman and Sharma (1997) developed the following simplified equation that eliminates the need to estimate channel resistance in slope-area calculations:

$$Q = 1.564 \cdot A^{1.173} \cdot Y^{0.400} \cdot S^{-0.056 \cdot \log(S)}. \quad \text{(F-28)}$$

In this equation, Q is in m^3 s^{-1}, A is the average cross-sectional area for the reach in m^2, Y is average depth in m, and S is the water-surface slope tangent.

Equation (F-28) gave good estimates of discharge for in-bank flows over a wide range of channel sizes ($0.41 \text{ m}^2 \leq A \leq 8520 \text{ m}^2$) and slopes ($0.00001 \leq S \leq 0.0418$), but tended to over-estimate for flows in which both $Q < 3$ m^3 s^{-1} and $Fr < 0.2$, where Fr is the **Froude number,** defined as

$$Fr \equiv \frac{U}{(g \cdot Y)^{1/2}}. \quad \text{(F-29)}$$

EXAMPLE F-1

The flood of 22 March 1948 on Esopus Creek at Coldbrook, NY, left high-water marks. Cross-sections were surveyed at two sections separated by a distance of $L = 78.7$ m. The difference in elevation of the high-water marks between the two stations was $\Delta Z = 0.35$ m; other survey data are summarized in the table. The estimated Manning's n for the reach is $n = 0.043$. Determine the peak discharge of the flood.

Section	Area (m²)	Width (m)	Hydraulic Radius (m)
u	135.7	55.2	2.43
d	136.6	53.4	2.52

Solution (Standard Method) Information for Steps 1 and 2 is given. Subsequent steps are as follows:

3. Compute section conveyances [Equation (F-19)]:

$$K_u = \frac{1.00 \times (2.43\ \text{m})^{2/3} \times (135.7\ \text{m}^2)}{0.043} = 5704.2\ \text{m}^3\,\text{s}^{-1}$$

and

$$K_d = \frac{1.00 \times (2.52\ \text{m})^{2/3} \times (136.6\ \text{m}^2)}{0.043} = 5891.2\ \text{m}^3\,\text{s}^{-1}.$$

4. Compute average conveyance [Equation (F-20)]:

$$K = [(5704.2\ \text{m}^3\text{s}^{-1}) \times (5891.2\ \text{m}^3\text{s}^{-1})]^{1/2} = 5797.0\ \text{m}^3\text{s}^{-1}.$$

5. Compute S_1 [Equation (F-21)]:

$$S_1 = \frac{0.35\ \text{m}}{78.7\ \text{m}} = 0.00446.$$

6. Compute Q_1 [Equation (F-22)]:

$$Q_1 = (5797.0\ \text{m}^3\,\text{s}^{-1}) \times (0.00446)^{1/2} = 387.0\ \text{m}^3\,\text{s}^{-1}.$$

7. Compute U_{u1} and U_{u2} [Equation (F-23)]:

$$U_{u1} = \frac{387.0\ \text{m}^3\,\text{s}^{-1}}{135.7\ \text{m}^2} = 2.85\ \text{m}\,\text{s}^{-1}$$

and

$$U_{d1} = \frac{387.0\ \text{m}^3\,\text{s}^{-1}}{136.6\ \text{m}^2} = 2.83\ \text{m}\,\text{s}^{-1}$$

8. Compute h_{u1} and h_{d1} [Equation (F-24)]:

$$h_{u1} = \frac{(2.85\ \text{m}\,\text{s}^{-1})^2}{2 \times 9.81\ \text{m}\,\text{s}^{-2}} = 0.415\ \text{m}.$$

and

$$h_{u1} = \frac{(2.83\ \text{ms}^{-1})^2}{2 \times 9.81\ \text{ms}^{-2}} = 0.409\ \text{m}.$$

9. Compute S_2 [Equation (F-25)]:
Since $A_d > A_u$, $k = 0.5$ [Equation (F-24)] and

$$S_2 = \frac{(0.35\ \text{m}) + 0.5 \times (0.415\ \text{m} - 0.409\ \text{m})}{78.7\ \text{m}} = 0.00449.$$

10. Compute Q_2 [Equation (F-27)]:

$$Q_2 = (5797.0\ \text{m}^3\,\text{s}^{-1}) \times (0.00449)^{1/2} = 394.8\ \text{m}^3\,\text{s}^{-1}.$$

11. Q_2 is larger than Q_1. Using Q_2, we compute new estimates of the velocities, velocity heads, and friction slope and find $Q_3 = 394.9\ \text{m}^3\,\text{s}^{-1}$. The difference between Q_2 and Q_3 is well within the uncertainty of the method (recall that n is an estimated value), so we can end the computation and conclude that the peak flood discharge was $395\ \text{m}^3\,\text{s}^{-1}$.

Solution (Simplified Method): The average cross-sectional area for the reach is $A = (135.7\ \text{m}^2 + 136.6\ \text{m}^2)/2 = 136.2\ \text{m}^2$; average hydraulic radius, R, is 2.48 m; and the water-surface slope $S = 0.00446$ as found in Step 5. Entering these values into Equation (F-28) yields

$$Q = 1.564 \times 136.2^{1.173} \times 2.48^{0.400} \times 0.00446^{-0.056 \cdot \log(0.00446)} = 360\ \text{m}^3\,\text{s}^{-1}.$$

For this example the standard and simplified estimates differ by less than 10%, within the usual uncertainty range for slope-area estimates.