Introduction to Hydrometry

Hydrometry means literally water measurement. In the past hydrometric engineers were particularlyinvolved in stream ow measurements. Today many aspects of water measurements are included.Hydrometry is de ned in this course as the measurement of ow inopen watercourses, supported or complemented by the measurements of water levels, bed levels and sediment transport.

5. STREAMFLOW MEASUREMENTS

Continuous streamflow records are necessary in the design of water supply systems, in designing hydraulic structures, in the operation of water management systems, and in estimating sediment or chemical loads of streams. To do these, systematic records of stage and discharge are essential. This chapter elaborates common methods practiced in Ethiopia to measure streamflow.

River stage is the elevation above some arbitrary zero datum of the water surface at a streamflow gauging station. The datum is sometimes taken as mean sea level but more often is slightly below the point of zero flow at the gauging station. The elevation datum is set with reference to at least three permanent reference marks or benchmarks located in stable ground separate from the recorder structure following standard surveying work.

It is difficult to make a direct, continuous measurement of discharge in a stream or river but relatively simple to obtain a continuous record of stage. Thus measurements of river stage provides the best alternative. Before we discuss some methods of measuring river stage, first we discuss criteria fort selecting a stream gauging station.

5.1 Stream gauging site selection

The principles of network design and the proposed use of data should govern the selection of streams to be gauged. Dense network of gauging station is required for research works related to runoff estimation, soil erosion estimation, and water balance calculation at different watershed sizes. Whereas the goal is to construct a dam to impound water, light network of stream gauging stations is sufficient - one station at or near the dam site can be adequate. A general-purpose network must, however, provide the ability to estimate hydrological parameters over a wide area using for example a regional regression model.

Despite the development of a variety of objectives and statistically based methods for streamflow and rainfall network design, judgment and experience are still indispensable. The WMO Guide to Hydrological Practice recommendations for network density as a staring point for network design is given in Table 5.1.

Table 5-1.Recommended minimum density of hydrometric stations

Type of region	Range of norms for minimum network, area, km^2 per station	Range of provisional norms tolerated in difficult conditions area, km^2 per station
Flat regions of tropical, temperate and Mediterranean zones	1000 - 25000	3000 - 10000
Mountainous regions of tropical, temperate and Mediterranean zones	300 -100	1000 - 5000
Small mountainous islands with very irregular rainfall, very dense stream network	140 - 300	
Arid zone	5000 - 20000	

5.1.1 Selection of gauging site

The selection of a particular site for the gauging station on a given stream should be guided by the following criteria for an ideal gauge site (WMO 1981):

i. The general course of the stream is straight for about 100 meters upstream and downstream from the gauging site.

ii. The total flow is confined into the channel at all stages and no flow bypasses the site as sub-surface flow.

iii. The streambed is not subject to scour and fill and is free of aquatic growth.

iv. Banks are permanent, high enough to contain floods, and are free of brush.

v. Unchanging natural controls are present in the form of a bedrock outcrop or other stable riffle for low flow, and a channel constriction for high flow, or a fall or cascade that is un-submerged at all stages to provide a stable relation between stage and discharge. If no satisfactory natural low-water control exists, a suitable site is available for installing an artificial control.

vi. A site is available, just upstream from the control, for housing the stage recorder where the potential for damage by water-borne debris is minimal during flood stages; the elevation of the stage recorder itself should be above any floods likely to occur during the life of the station.

vii. The gauge site is far enough upstream from the confluence with another stream.

viii. A satisfactory reach for measuring discharge at all stage is available within reasonable proximity of the gauge site. It is not necessary that low and high flows be measured at the same cross-section.

ix. The site is readily accessible for ease in installation and operation of the gauging station.

x. Facilities for telemetry can be made available, if required.

xi. Typical streamflow gauging station installed in the Wabi Shebele river at upstream fo Melka Wakana Dam is shown in Figure 5.1. In practice rarely will an ideal site be found for a gauging station and judgement must be exercised in choosing between possible sites. A gauging site should be located at a point along the stream where there is a high correlation between stage and discharge, featuring a one to one correspondence between stage and discharge. Either section or channel control is necessary for the rating to be single-valued.

xii. A rapid or fall located immediately downstream of gauging site forces critical flow through it, providing a section control. In the absence of a natural section control, an artificial control – for instance, a concrete weir – can be built to force the rating being single-valued. This type of control is very stable under low and average flow conditions.

xiii. A long downstream channel of relatively uniform cross-sectional shape, constant slope, and bottom friction provides a channel control. However, a gauging site relying on channel control requires periodic re-calibration to check its stability. To improve channel control, the gauging site should be located far from downstream backwater effects caused by reservoirs and large river confluence.

Figure 5-1: Typical streamflow gauging station installed in the Wabi river near Dodola town upstream of the Melkawakana reservoir (February 2002).

5.2 Stage measurement

Basically there are two modes of stage measurements. The first is discrete stage measurements using manual gauges, and the second is continuous stage measurements using recorders. For the measurement of stage, uncertainties should not be worse than ± 10 mm or 0.1 % of the range.

5.2.1 Manual gauge

The simplest way to measure river stage is by means of a staff gage. A staff gauge is vertically attached to a fixed feature such as a bridge pier or a pile (Figure 5.2). The scale is positioned so that all possible water levels can be read promptly and accurately. Another type of manual gauge is the wire gauge. Wire gauge consists of a reel holding a length of light cable with a weight affixed to the end of the cable. The reel is mounted in affixed position – for instance, on a bridge span – and the water level is measured by unreeling the cable until the weight touches the water surface. Each revolution of the reel unwinds a specific length of cable, permitting the calculation of the distance to the water surface. Manual gages are used where stages do not vary greatly form one measurement to another measurement. They are impractical in small or flashy streams, where substantial changes in stages may occur between readings.

5.2.1 Recording gauge

A recording gage measures stages continuously and records them on a strip chart. The mechanism of a recording gauge is either float actuated or pressure actuated. In a float actuated recorder, a pen recording the water level on a strip chart is actuated by a float on the surface of the water. The recorder and float is housed on suitable enclosure on the top of a stilling well connected to the stream by two intake pipes (two intake pipes are used incase one of them become clogged) (Figure 5.2). The stilling well protects the float from debris and ice and dampens the effect of wave action. This type of gage is commonly used for continuous measurements of water levels in rivers and lakes.

The pressured actuated recorder or the bubble gage senses the water level by bubbling a continuous stream of gas (usually CO_2) into the water. The bubble gauge consists of a specially designed servo-manometer, gas-purge system, and recorder. Nitrogen fed through a tube bubble freely into the stream through an orifice positioned at a fixed location below the water surface. The pressure in the tube, equal to that of the piezo-meteric head above the orifice, is transmitted to the servo-manometer, which converts changes in pressure in the gas-purge system into pen movements on a strip-chart recorder. Bubble-type water level sensors are used in applications where a stilling well is either impractical or too expensive and where the stream carries a heavy sediment load. The Awash river at Awash town is equipped with pressured actuated recorder.

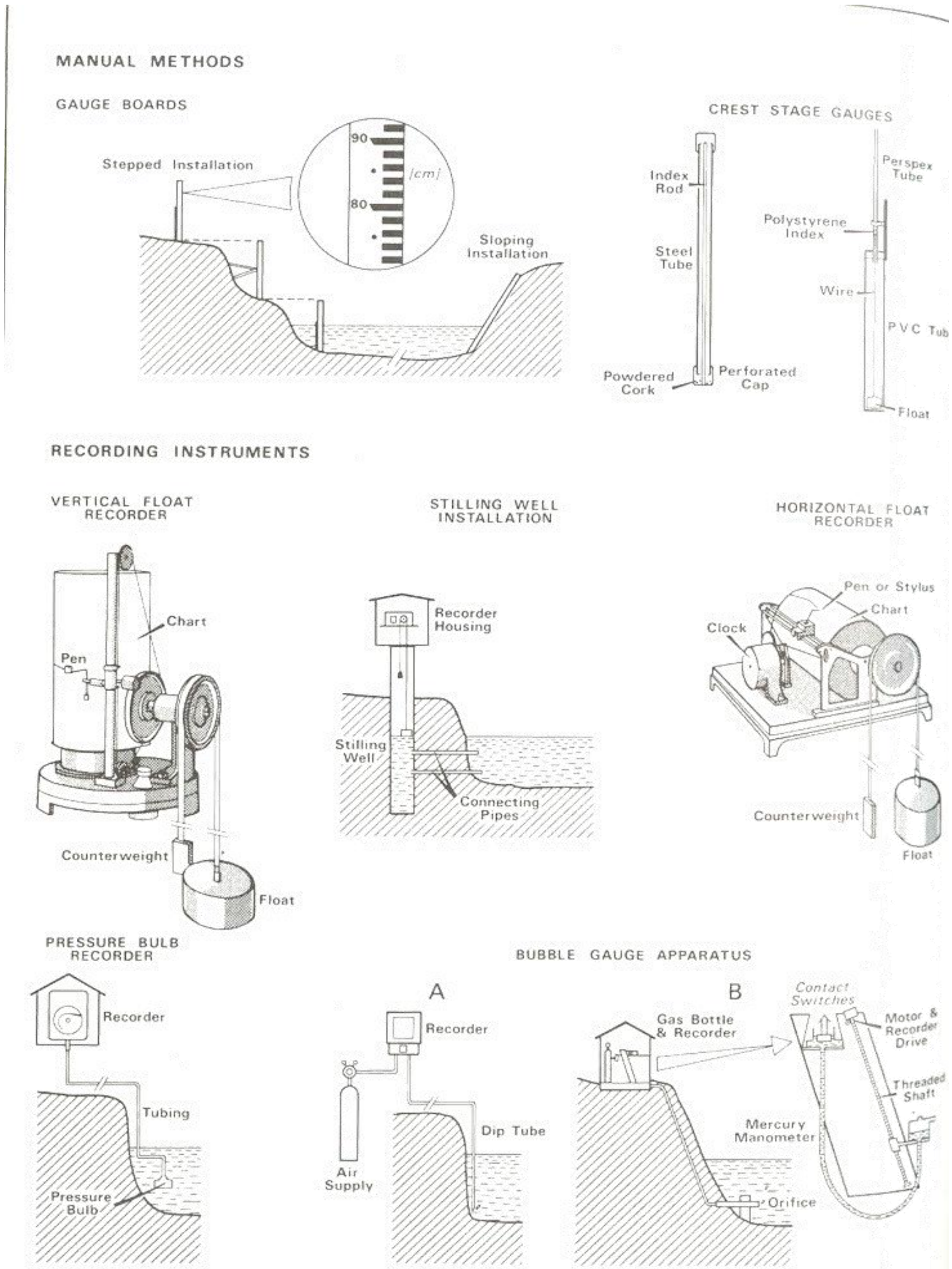


Figure 5-2. **The measurement of stage through manual methods and recording instruments (after Gregory and Walling, 1973)**

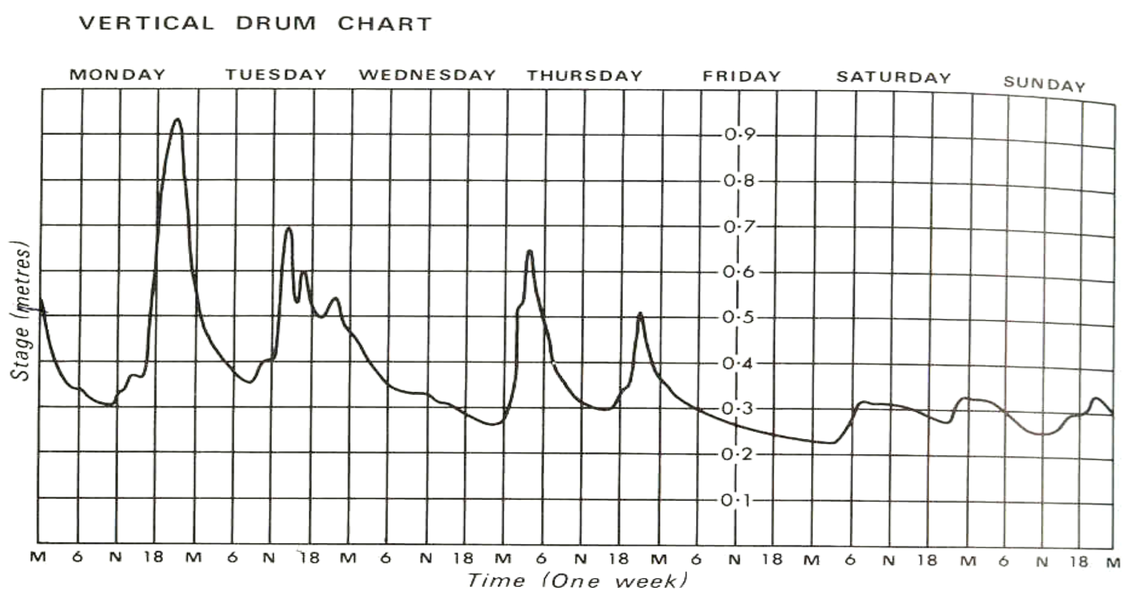


Figure 5-3: A typical chart from vertical float recorder.

Crest stage gage. This is used to obtain a record of flood crest at sites where recording gages are not installed. A crest stage gage consists of a wooden staff gage scale, situated inside a pipe that has small holes for the entry of water. A small amount of cork is placed in the pipe, floats as the water rises, and adheres to the staff or scale at the highest water level.

Telemetric gages. Gages with automatic data transmittal capabilities are called self-reporting gages, or stage sensors. Self-reporting gauges are of the float-actuated or pressure actuated type. These instruments use telemeters to broadcast stage measurement in real time, from a stream gauging location to a central site. This type of gauge is ideally suited for applications where speed of processing is of utmost important, e.g., for operational hydrology or real-time flood forecasting.

5.3 Flow velocity measurement and discharge computation

Flow velocity. The velocity of flow in a stream can be measured with a current meter. Current meters are propeller devices (Figure 5.4) placed in the flow, the speed with which the propeller rotates being proportional to the flow velocity.

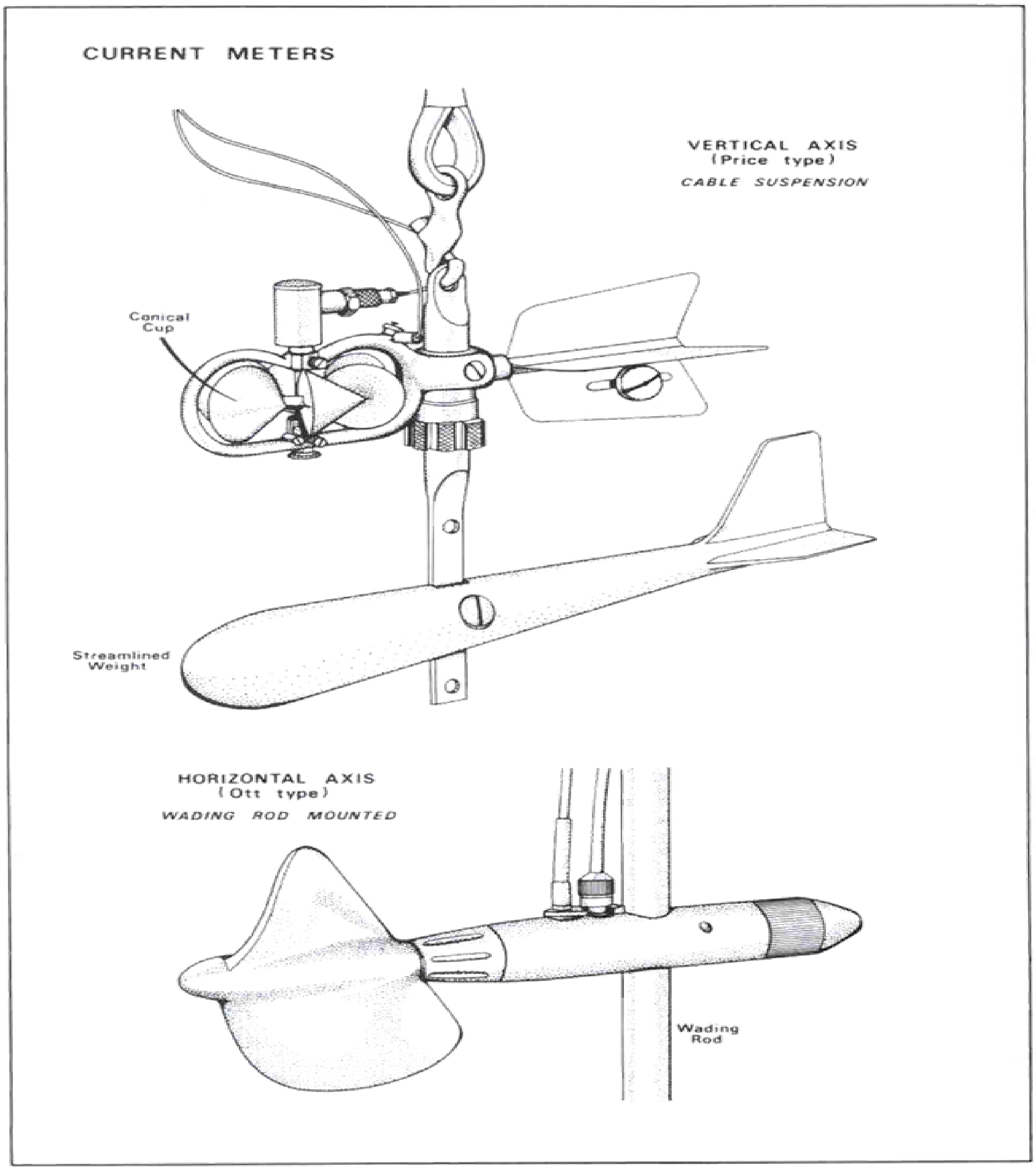


Figure 5-4: Vertical and horizontal axis current meters and wading rod and cable suspension mounting of the meter body.

The relation between measured revolution per second of the meter cups N and water

$$V = a + bN \tag{5.1}$$

velocity V is given by
where:

a = the starting velocity or velocity required to overcome mechanical friction.
b = the constant of proportionality, and

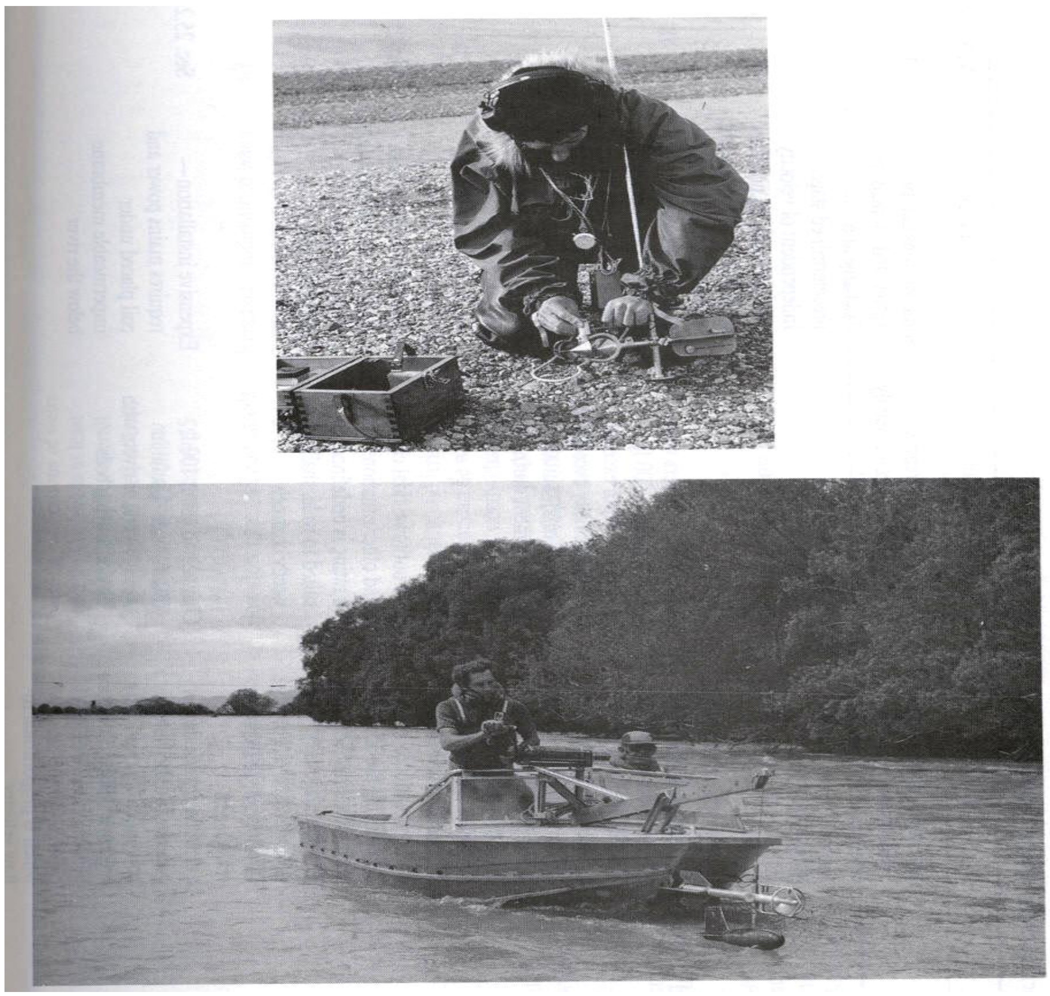


Figure 5-5: Top: current meter mounted on a measuring rod, (bottom) suspended on a cable from the bow of a jet-boat. Wide rivers flow (usually greater than 100 m) are often measured using a boat- the Baro river near Sudan border is the case in Ethiopia.

Initial values of a and b can be found from the calibration tables provided by the manufacturer. With time the values of a and b are changing and regular recalibration is essential. This may be done by towing the current meter through still water in a tank at a series of known velocities.

The current meter can be hand-held in the flow in a small stream (measurement by wading), suspended from a bridge or cable way across a large stream, or lowered from the bow of a boat (Figure 5.5).

Velocity distribution: The flow velocity varies with depth in a stream . Over the cross-section of an open channel, the velocity distribution depends on the character of the river banks and of the bed and on the shape of the channel. The maximum velocities tend to be found just below the water surface and away from the retarding friction of the banks.

The average velocity occurs say about 0.6 of the depth. It is standard practice to measure velocity at 0.2 and 0.8 of the depth when the depth is more than 60 cm and to average the two velocities to determine the average velocity for the vertical section. For shallow rivers and near the banks on deeper rivers where the depths are less than 0.6 m, velocity measurements are made at 0.6 of depth of flow.

Discharge computation.

The discharge computation of a stream is calculated from measurements of velocity and depth. A marked line is stretched across the stream. At regular intervals along the line, the depth of the water is measured with a graduated rod or by lowering a weighted line from the surface to the stream bed, and the velocity is measured using a current meter. The discharge Q at a cross-section of area A is found by

(5.2)

$$Q = \int V.dA$$

Where V = streamflow velocity
A = cross sectional area of the flow

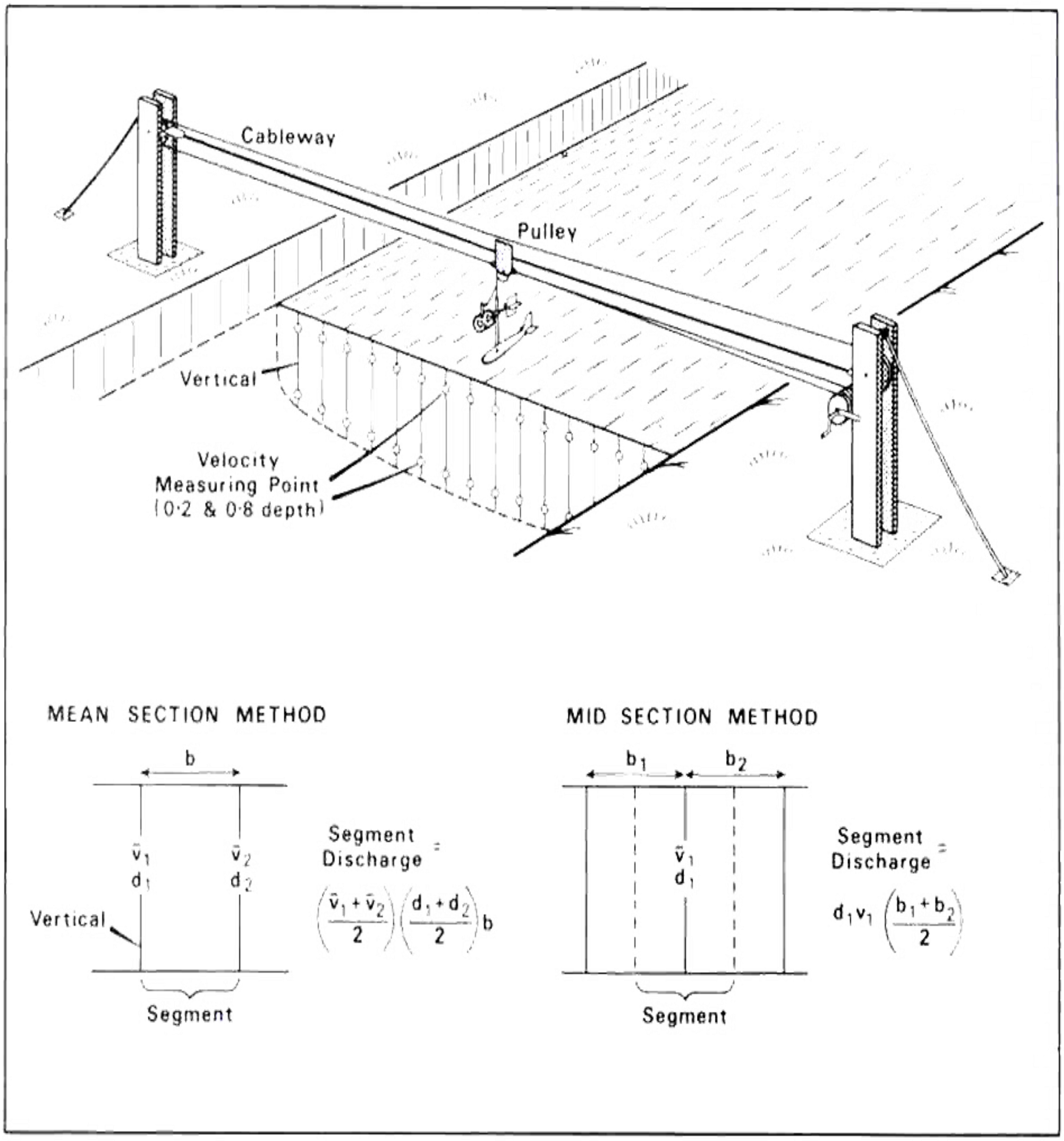


Figure 5-6. The velocity area technique of discharge measurements: a cable way is used on large streams for positioning the current meter in the verticals and a special cable drum can be used to obtain accurate readings of depth and spacing of verticals. The mean section and mid-section methods are commonly used to compute the discharge of the individual segments.

in which the integral is approximated by summing the incremental discharges

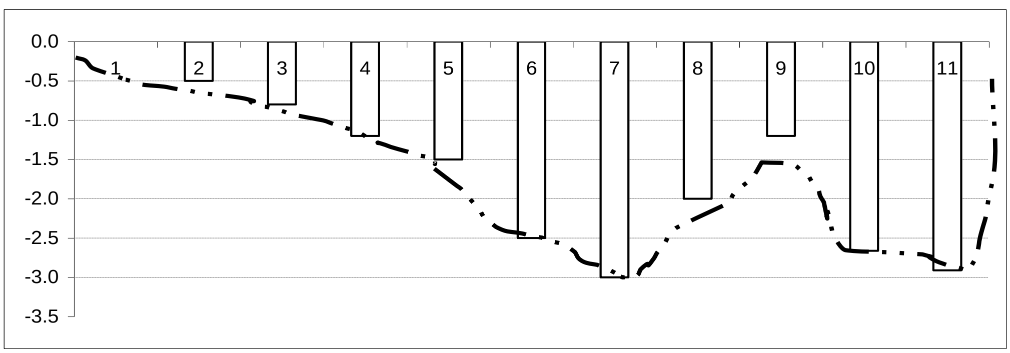
$$Q = \sum_{i=1}^{n} V_i \, d_i \, \Delta W_i \tag{5.3}$$

calculated from
each measurement i, i = 1, 2, ..., n of velocity V_i and depth d_i.
The measurements represent average values over width Δw_i of the stream.

Example 5.1: Given the following stream gauging data, calculate the discharge.

Vertical No.	1	2	3	4	5	6	7	8	9	10	11
Distance to refernce point (m)	15.0	20.0	25.0	30.0	35.0	40.0	45.0	50.0	55.0	60.0	65.0
Sounding depth di (m)	0.0	0.5	0.8	1.2	1.5	2.5	3.0	2.0	1.2	2.7	2.9
Velocity at 0.2 di (m/s)	0.0	0.5	0.7	0.9	1.2	1.4	1.7	1.3	0.9	1.7	1.8
Velocity at 0.8 di (m/s)	0.0	0.4	0.6	0.7	0.8	1.1	1.3	1.0	0.7	1.3	1.4

Solution: To use Eq. (5.3) first the average velocity at each sounding depth is calculated, then the partial width which is constant in this example is calculated (20-15) = 5 m



Vertical No.	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0
Distance to reference point (m)	15.0	20.0	25.0	30.0	35.0	40.0	45.0	50.0	55.0	60.0	65.0
Sounding depth (m)	0.0	0.5	0.8	1.2	1.5	2.5	3.0	2.0	1.2	2.7	2.9
Velocity at 0.2 m (m/s)	0.0	0.5	0.7	0.9	1.2	1.4	1.7	1.3	0.9	1.7	1.8
Velocity at 0.8 m (m/s)	0.0	0.4	0.6	0.7	0.8	1.1	1.3	1.0	0.7	1.3	1.4
Average Velocity (m/s)	0.0	0.5	0.7	0.8	1.0	1.3	1.5	1.2	0.8	1.5	1.6
Partial area Ai (msq)	0.0	2.5	4.0	6.0	7.5	12.5	15.0	10.0	6.0	13.3	4.8
Partial discharge (m3/s)	0.0	1.1	2.6	4.8	7.5	15.6	22.5	11.5	4.8	19.5	7.6

Total Q = 97.58 (m3/s) **Total A = 81.61 msq**

Average velocity (m/s) = Q/ A = 1.196 m/s

5.4 Dilution gauging

Dilution gauging method of measuring the discharge in a stream is made by adding a chemical solution or tracer of known concentration to the flow and then measuring the dilution of the solution downstream where the chemical is completely mixed with the stream water. A tracer is a substance that is not normally present in the stream and that is not likely to be lost by chemical reaction with other substances. Salt, fluorescein dye, and radioactive materials are commonly used as tracers. In general there are two methods of dilution gauging, sudden-injection methods and constant rate injection method. The two methods are described as follows.

5.4.1 Sudden Injection method

In this method a quantity of tracer volume V_1 and concentration C_1 is added to the river by suddenly emptying a flask of tracer solution that is gulp injection. At the sampling station downstream the entire tracer cloud is monitored to find the relation between concentration and time. The quantity of tracer or mass of tracer M is then equal to C_1V_1. If t_1 is the time before the leading edge of the tracer cloud arrives at the sampling station and t_2 is the time after all the tracer has

$$C_1V_1 = Q\int_{t_1}^{t_2}(C_2 - C_0)dt \qquad (5.5)$$

passed this station the quantity of tracer is given by

Where: C_2 = sustained final (equilibrium) concentration of the chemical in the well mixed flow (mg/l)

C_0 = the base value concentration, already present in the river (mg/l)

Using principle of conservation of mass we may estimate the streamflow Q:

$$Q = \frac{C_1V_1}{\int_{t_1}^{t_2}(C_2 - C_0)dt} \qquad (5.6a)$$

$$Q = \frac{V_1 C_1}{T C_2} \quad (5.6b)$$

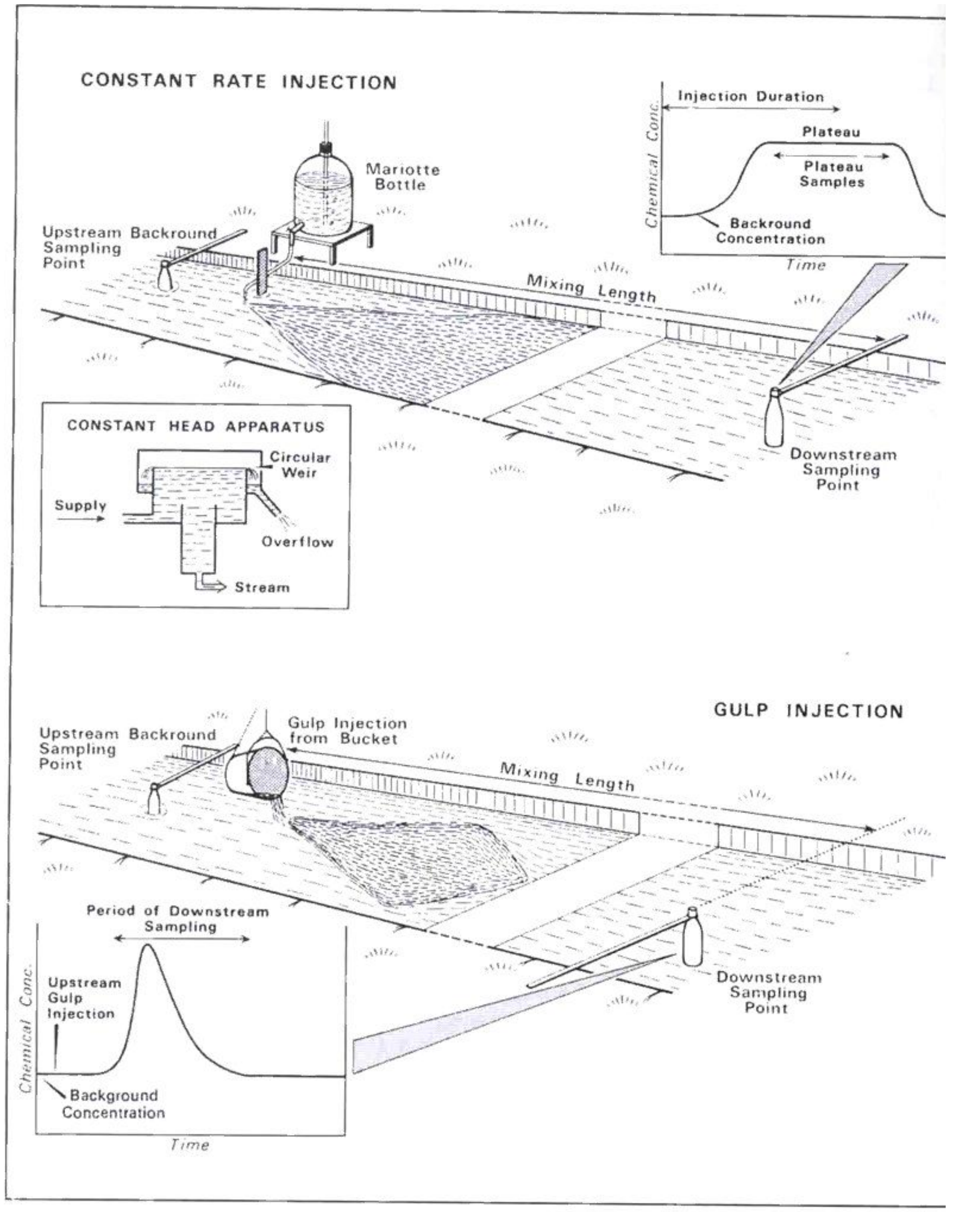


Figure 5-7. Dilution gauging: constant rate injection and gulp injection.

5.4.2 Continuous and constant rate injection method

In this method a tracer of known concentration C_1 is injected continuously at a rate q at a sampling station situated downstream of injection point. The mass rate M at which the tracer enters the test reach is

$$M = qC_1 + QC_0 \tag{5.7}$$

Assuming that satisfactory mixing of tracer has taken place with the entire flow across the cross-section with the measured concentration C_2 (reaching equilibrium concentration), we have

$$qC_1 + QC_0 = (Q+q)C_2 \tag{5.8}$$

Solving for Q we get

$$Q = \frac{C_1 - C_2}{C_2 - C_0} q \tag{5.9a}$$

The dilution method is particularly useful for very turbulent flows, which can provide complete mixing within a relatively short distance. It is also applicable when the cross section is so rough that alternative methods are unfeasible.

It is to be noted that a highly turbulent and narrow reach is desirable. In this regard minimum required mixing length can be estimated by

$$L = \frac{0.13B^2C \ \ (0.7C + 2\sqrt{g})}{gy} \tag{5.9b}$$

Where: L = mixing length

B = average width of the stream

y = average depth of the stream

C = Chezy's coefficient of roughness, varying from 15 to 50 for smooth to rough bed conditions

g = 9.81 m/s^2

Example 5.2 25 g/l solution of a chemical tracer was discharged into a stream at 0.01 l/s. At sufficiently far downstream observation point, the chemical was found to reach an equilibrium concentration of 5 parts per billion. Estimate the stream discharge. The background concentration of the tracer chemical in stream water may be taken as nil.

Solution. q = 0.01 l/s = 10^{-5} m^3/s, C_1 = 25 gm/l = 20 000 mg/l = 20 000 ppm = 20 part per billion

$$Q=\frac{C_1-C_2}{C_2-C_0}q$$

$$Q = (20\,000/0.005)*10^{-5}$$

$$Q = 50 \text{ m}^3/\text{s}$$

Example 5.3 A fluorescent tracer with a concentration of 45 gm/l was injected into a stream at a constant rate of 8 cm^3/s. At a downstream section sufficiently far away from the point of injection, the concentration was found to be 0.008 parts per million. Estimate the discharge in the stream. The background concentration of the tracer in the stream is zero.

Solution. using Eq (5.9a) we get

$$Q=\frac{C_1-C_2}{C_2-C_0}q$$

$$Q=\frac{8*10^{-6}(45000-.008)}{.008-0}$$

$$Q = 45 \text{ m}^3/\text{s}$$

5.5 The slope-area method

Occasionally, the high stages and swift currents that prevails during floods increase the risk of accident and bodily harm. Therefore, it is generally not possible to measure discharge during the passage of a flood. An estimate of peak discharge can be obtained indirectly by the use of open channel flow formula.

The following guidelines are used in selecting a suitable reach:

i. High-water marks should be readily recognizable.
ii. The reach should be sufficiently long so that fall can be measured accurately.
iii. The cross-sectional shape and channel dimensions should be relatively constant.
iv. The reach should be relatively straight, although a contracting reach is preferred to an expanding reach, and
v. Bridges, channel bends, waterfalls, and other features causing flow non-uniformity should be avoided. Note that the accuracy of the slope-area method improves as the reach length increases.

A suitable reach should satisfy one or more of the following criteria:

i. The ratio of reach length to hydraulic depth should be greater than 75.
ii. The fall should be greater than or equal to 0.15 m, and
iii. the fall should be greater than either of the velocity heads computed at the upstream and downstream cross sections.

The procedure consists of the following steps:

I. Calculate the conveyance K at the upstream and downstream sections:

$$K_u = \frac{1}{n} A_u R_u^{2/3} \tag{5.10}$$

$$K_d = \frac{1}{n} A_d R_d^{2/3} \tag{5.11}$$

Where: K = conveyance
A = flow x-sectional area (m^2)
R = hydraulic radius (m)
n = reach Manning roughness coefficient
u and d denotes upstream and downstream, respectively

II. Calculate the reach conveyance K , equal to the geometric mean of the upstream and downstream conveyances:

$$K = (K_u K_d)^{1/2} \tag{5.12}$$

III. Calculate the first approximation of the energy slope S:

$$S = \frac{F}{L} \tag{5.13}$$

Where: F = fall, elevation difference in L;
and L = reach length

IV. Calculate the first approximation to the peak discharge Q_i

$$Q_i = KS^{1/2} \tag{5.14}$$

V. Calculate the velocity heads:

$$h_{vu} = \frac{\alpha_u (Q_i / A_u)^2}{2g} \tag{5.15}$$

$$h_{vd} = \frac{\alpha_d (Q_i / A_u d)^2}{2g} \tag{5.16}$$

Where: h_u and h_d are the velocity heads at upstream and downstream sections respectively,

α_{vu} and α_{vd} are the velocity heads at upstream and downstream sections respectively, and

g = gravitational acceleration.

VI. Calculate an updated value of energy slope S_i:

$$S_i = \frac{F + k(h_{vu} - h_{vd})}{L} \tag{5.17}$$

Where: k = loss coefficient, for expanding flow, i.e., $A_d > A_u$, k = 0.5, for contracting flow that is $A_d < A_u$, k = 1.0

VII. Calculate an updated value of peak discharge

$$Q_i = KS_i^{1/2} \tag{5.18}$$

VIII Compare the updated value of peak discharge with previous estimate, and continue the iteration until you close the difference between the newly estimated peak discharge and the previously estimated peak discharge.

Example 5.3 Use the slope area method to calculate the peak discharge for the following data: Reach length = 600 m, fall = 0.6 m. Manning n = 0.037.

Upstream flow area = 1550 m^2, upstream wetted perimeter = 450 m, upstream velocity head coefficient = 1.10. Downstream flow area = 1450 m^2, downstream wetted perimeter = 400 m, downstream velocity head coefficient = 1.12.

Solution: Basic parameters calculations:

	Flow area (m^2)	Wetted perimeter (m)	Hydraulic radius (m)	Conveyance (m3/s)	Reach conveyance =	93999.14
upstream section	1550	450	3.44	95,545.32	First App. of S =	0.001
downstream section	1450	400	3.63	92,477.97		

Peak discharge computation:

Iter. No.	h_{vu} (m)	h_{vd} (m)	Energy slope (m/m)	Peak discharge (m^3/s)
1			0.001	2972.515
2	0.20619	0.2399	0.0009438	2887.815
3	0.19461	0.22642	0.000946	2892.639
4	0.19526	0.22718	0.0009468	2892.368

Hence, the final value converges to Q = 2892.3 m3/s

5.6 Orifice formula for bridge opening

The discharge may be calculated from measurements taken on an existing bridge over the river by using the orifice formula (TRRL, 1992):

$$Q = C_o (2g)^{1/2} L D_d [(D_u - D_d) + (1+e)\frac{V^2}{2g}]^{(1/2)} \tag{5.19}$$

Where:

Q = Discharge at a section just downstream of the bridge (m^3/s)

g = Acceleration due to gravity (9.81 m/s^2)

L = Linear waterway, i.e. distance between abutments minus width of piers, measured perpendicular to the flow (m)

D_u = Depth of water immediately upstream of the bridge measured from marks left by the river in flood (m)

D_d = Depth of water immediately downstream of the bridge measured from marks on the piers, abutments or wing walls (m)

V = Mean velocity of approach (m/s)

C_o and e are coefficients to account for the effect of the structure on flow, as listed in Table 5.2. Definition sketch of the Orifice formula is shown in Figure 5.8.

Table 5-2. Values of C_o and e in the orifice formula, L = Width of waterway, and W =unobstructed width of the stream as defined in Figure 5.9:

L/W	C_o	e
0.50	0.892	1.050
0.55	0.880	1.030
0.60	0.870	1.000
0.65	0.867	0.975
0.70	0.865	0.925
0.75	0.868	0.860
0.80	0.875	0.720
0.85	0.897	0.510
0.90	0.923	0.285
0.95	0.960	0.125

It is to be noted that whenever possible, flow volumes should be calculated by both the area-velocity and orifice formula methods, and compared.

Example 5.4 Calculate the discharge passing through a bridge with a waterway width of 18 m across a stream 30 m wide. In flood the average depth of flow just downstream of the bridge is 2.0 m and the depth of flow upstream is 2.2 m.

Solution. Discharge at a section just upstream of the bridge, assuming a rectangular section is A. V = 2.2 x 30 V = 66 V m^3/s. Discharge at a section just downstream of the bridge will be the same and will be given by the orifice formula:

$$Q = C_o(2g)^{1/2} LD_d[(D_u - D_d) + (1+e)\frac{V^2}{2g}]^{(1/2)}$$

L =18, W = 30, L/W = 0.6, thus C_0 and e are 0.87 and 1.00 respectively.

$$Q = 0.87(2*9.81)^{1/2} 18*2[(2.2-2) + (1+1)\frac{V^2}{2*9.81}]^{(1/2)}$$

Q = 138.6[0.2 + 0.102V^2/(2*9.81)]$^{0.5}$

Substituting for Q, 66V, we get 66V = 138.6[0.2 + 0.102V^2/(2*9.81)]$^{0.5}$, V= 1.265

Now Q = 66 V = 66 *1.265 = 83.5 m/s

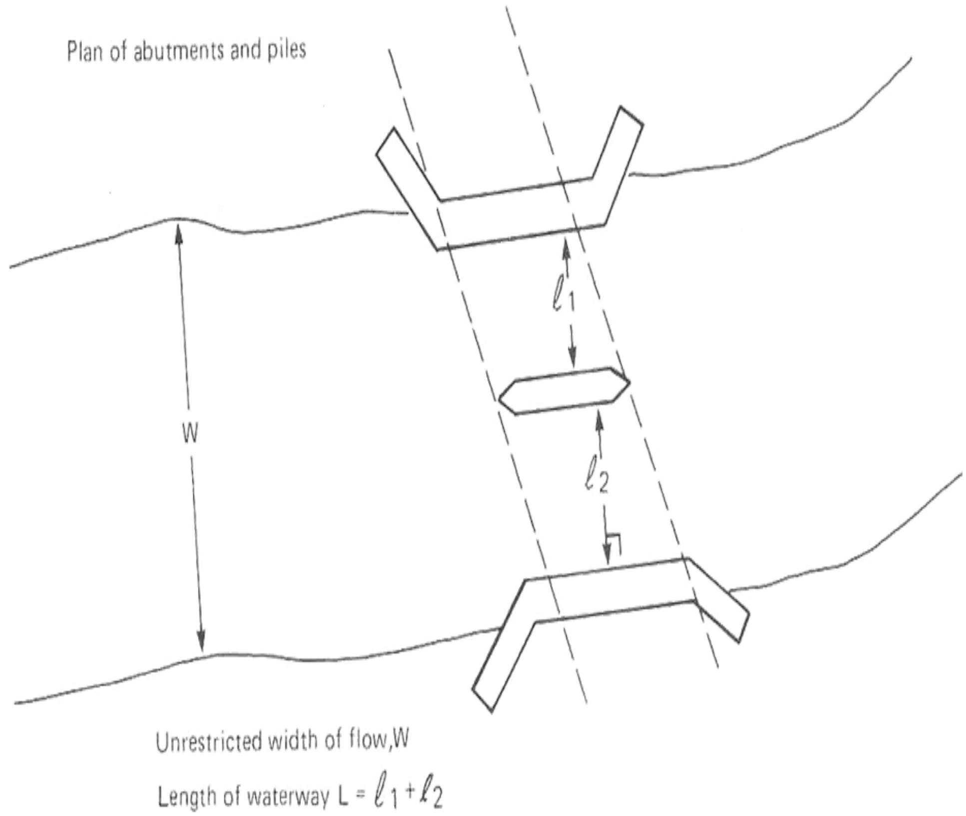


Fig. 4.5 ay at a bridge

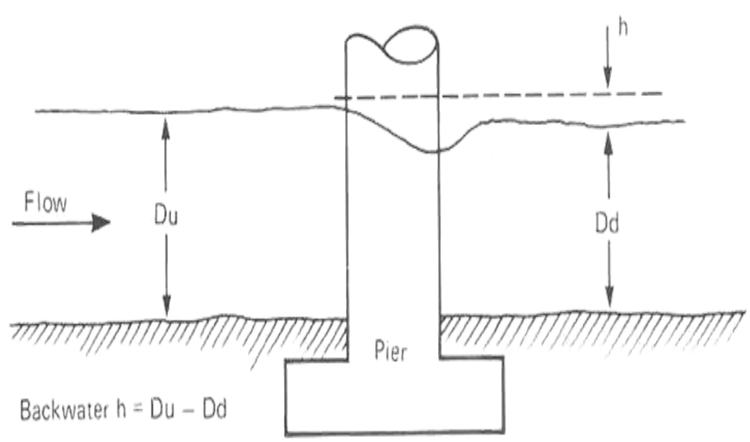
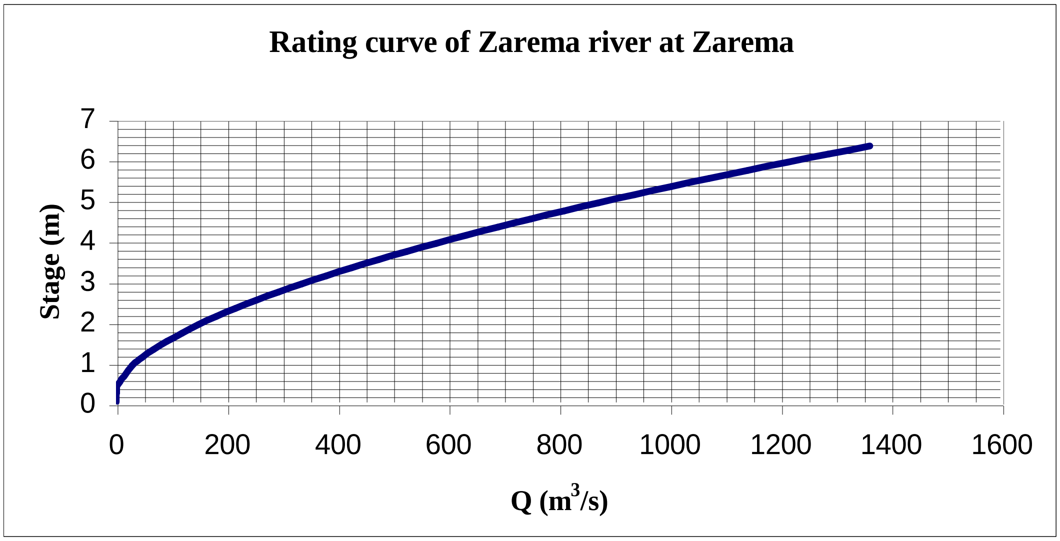


Figure 5-8. Definition sketch of the orifice formula

5.7 Stage discharge relationship - rating curve

The rating curve is constructed by plotting successive measurements of the discharge and gage height on a graph. Figure 5.9 shows an example of rating curves in linear and logarithmic scale for Zarema river a tributary of Tekeze river near the foot of Semen Mountains.



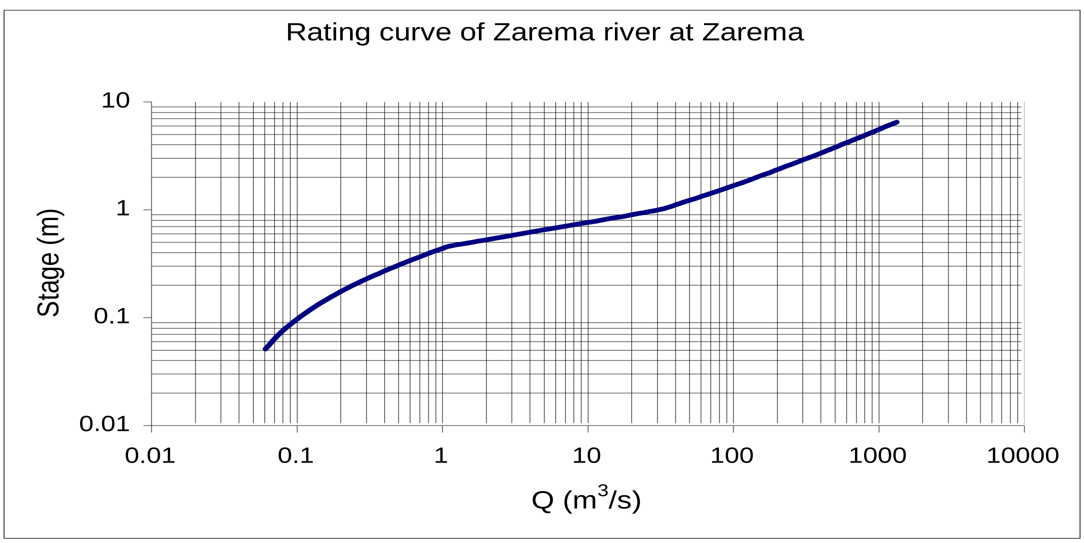


Figure 5-9. Rating curves in linear (Top) and logarithmic scale of Zarema river near Zarema, a tributary of Tekeze river (MWR)..

A rating curve must be checked periodically to ensure that the relationship between Q and H has remained constant. Scouring of the stream bed or deposition of sediment in the stream can cause the rating curve to change so that the same recorded gage height produces different Q.

The rating curve is described by a rating equation of the form

$$Q = a(H + H_0)^b \tag{5.20}$$

where a, b, and H_0 are coefficients.

Having paired measured data of (H, Q) the coefficients a and b can be estimated by taking a trial value of H_0 which gives a straight line of the equation:

$$\log Q = \log a + b \log(H + H_0) \tag{5.21}$$

Or value of H_0 adjustment for the low flow, can be estimated with the following method. Three values of discharge (Q_1, Q_2, Q_3) are selected from known portion of the curve. One of these should be near the middle of the curve, and the other value should be near the upper end of the curve. Then the third intermediate value is estimated by

$$Q_2 = \sqrt{Q_1 Q_3} \tag{5.22}$$

If H_1, H_2, and H_3 represent the gage heights corresponding to Q_1, Q_2, and Q_3 then H_o is estimated by

$$H_0 = \frac{H_1 H_3 - H_2^2}{H_1 + H_3 - 2H_2} \tag{5.23}$$

A full rating curve can consist of different rating equation, e.g., one for low flows and one for high flows, and often for low flows b > 2, for high flows b < 2.

Example5.4 Developed rating curve for the river Zarema near Zarema a tributary of the Tekeze river having a watershed area of 3259 km^2 has the following rating curves:

$$Q = 4.241(H + 0.201)^{3.057} \qquad \textit{for low flow upto } H \leq 0.43m$$

$$Q = 69.997(H + (-0.345))^{1.680} \qquad \textit{for high flow upto } H > 0.43m$$

where H is in m and Q in m^3/s. Figure 5.10 shows the graphical form of the above equations.

Extrapolation of ratings. Extrapolation of ratings is necessary when a water level is recorded above the highest level and flow gauged level. Without considering the cross-section geometry and controls, large error may result. Where the cross section is stable, a simple method is to extend the *stage-area* and *stage velocity* curves and, for given stage values, take the product of velocity and cross-section area to give discharge values beyond the stage values that have been gauged.

Stage-area and stage velocity curves can easily produced from the data that are used for establishing the rating curve of the same river. The stage-area curve then can be extended above the active channel by using standard land-surveying methods. Extrapolation of the stage-velocity curve requires understanding of the high stage control. Where there is channel control and where Manning roughness is not varying with stage, the Manning equation may be used to estimate the extrapolated velocity. It is to be noted that an upper bound on velocity is normally imposed by the Froude number $V/(gh)^{0.5}$ knowing that the Froude number rarely exceeds unity in alluvial channels.

Shifting in rating curves. The stage-discharge relationship can vary with time, in response to degradation, aggradation, or a change in channel shape at the control section. Shifts in rating curves are best detected from regular gaugings and become evident when several gaugings deviate from the established curve. Sediment accumulation or vegetation growth at the control will cause deviation which increases with time, but a flood can flush away sediment and aquatic weed and cause a sudden reversal of the rating curve shift.

In gravel-bed rivers a flood may break up the armoring of the surface gravel material, leading to general degradation until a new armoring layer becomes established, and rating tend to shift between states of quasi-equilibrium. It may then be possible to shift the rating curve up or down by the change in the mean bed level, as indicated by plots of stage and bed level versus time. Most of the rivers in Wollo, such as Mille and Logia exhibit such phenomenon.

In rivers with gentle slopes. discharge for a given stage when the river is rising may exceed discharge for the same stage when the river is falling (flood subsiding). In such cases adjustment factors must be applied in calculating discharge for rising and falling stages.