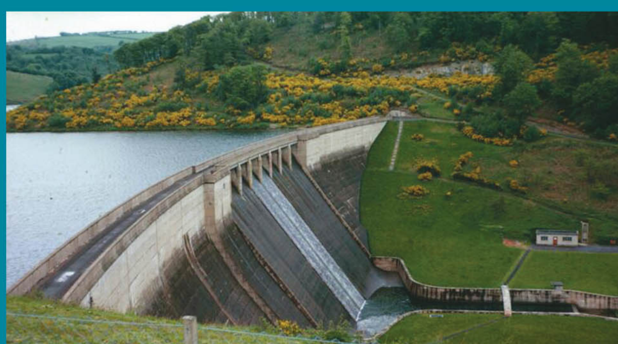
Third Edition

Streamflow Measurement

Reginald W. Herschy



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Third edition

Reginald W. Herschy

Chairman British Standards Institution Technical Committee on Hydrometry

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Water, water, every where,
Nor any drop to drink.
Samuel Taylor Coleridge (1772–1834)

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Preface

Since the publication of the second edition of the book in 1995, significant advances have taken place worldwide in streamflow measurement in both instrumentation and methodology.

The scope of this third edition of the book has therefore been extended to include new methods and instrumentation and a new Chapter 6 on the acoustic Doppler current profiler (ADCP) has been included to meet the growing need worldwide for this method.

The moving boat and electromagnetic methods are now included in a reduced form in Chapter 2 as is a new method now under research – the seismic flowmeter.

There are hardly any large floods which have been measured directly by standard methods and a section on indirect methods of flood peak estimation has therefore been included under Chapter 5 – Special problems.

The opportunity has been taken to rewrite certain sections of the book in view of technical developments and new or revised international standards. Chapters 9 and 15 come into this category. However, care has been taken to retain existing instrumentation, such as chart recorders, which are still used efficiently in many countries.

The book has been written in a global context as streamflow measurements are carried out to similar standards worldwide. It is indeed true to say that a gauging is being made by someone somewhere in the world at any moment in time.

The need for better and extended streamflow measurements is more necessary today than ever before especially with the need to address climate change and its effects on streamflow.

The international monitoring of streamflow measurement will become crucial in a changing hydrological world where our precious water resources are required to be audited and carefully managed.

Reginald W. Herschy
Reading

Chapter 1

Introduction

1.1 Global water

The management of our global water resources will require renewed effort by all concerned in the water industry and measurement of the world's rivers will play a large part in the distribution of these resources. In the world today over one quarter of the population still do not have safe drinking water.

The amount of fresh water available is small. Only about 0.6% of the global water is available for use and of that only 1% is in rivers; the rest is groundwater.

However, this 1% is only 0.006% of the total global water and it is this water that requires to be measured as streamflow; in fact about 70,000 km^3.

With tens of millions of people worldwide relying on fresh water, the importance of global streamflow measurement is crucial.

Figure 1.1 shows diagrammatically the components of the global water cycle.

1.2 Climate change

Considerable research has been undertaken into climate change by the IPCC (Intergovernment Panel on Climate Change). Two important elements for hydrometry to address are rainfall and streamflow. Climate change can be expected to lead to changes in precipitation and streamflow. Water supply in arid and semi-arid regions is very sensitive to small changes in rainfall and evaporation although the latter may be reduced because of increased CO_2 concentrations. In the runoff scenario, it is believed that a doubling of CO_2 might increase river flows by between 40 and 80% in certain parts of the world. Such estimates of runoff extremes, if confirmed, would require careful streamflow monitoring as well as considerable modification in the design of water related structures including flood control works and conveyance capacity.

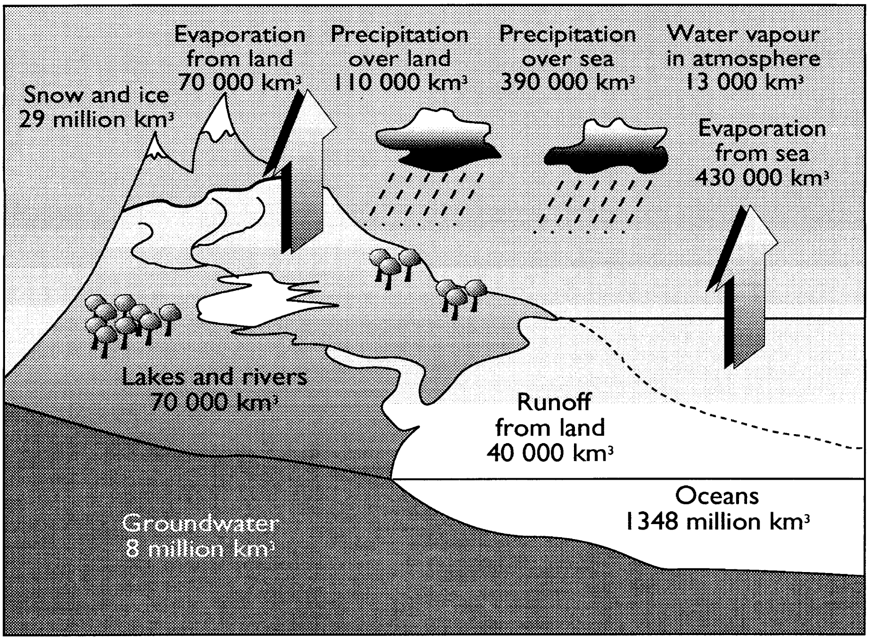

Figure 1.1 The hydrological (water) cycle with major reservoirs.
Note: The fluxes of evaporation, precipitation and annual run off are in km^3 $year^{-1}$

1.3 Field measurements

Streamflow is the combined result of all climatological and geographical factors that operate in a drainage basin. It is the only phase of the hydrological cycle in which the water is confined in well-defined channels which permit accurate measurements to be made of the quantities involved. Other measurements of the hydrological cycle are point measurements for which the uncertainties, on an areal basis, are difficult, if not impossible, to estimate.

Good water management is founded on reliable streamflow information and the final reliability of the information depends on the initial field measurements. The hydrologist making these measurements has therefore the responsibility of ensuring raw data of acceptable quality are collected. The successful processing and publication of the data depend largely on the quality of the field measurements.

This book is therefore for field hydrologists and for students of hydrology in universities and colleges.

Objectives of a streamflow programme

There are many different uses of streamflow data within the broad context of water management, such as water supply, pollution control, irrigation, flood control, energy generation and industrial water use. The importance placed on any one of these purposes may vary from country to country. In India and China, for example, emphasis may be placed upon irrigation and flood control whereas in the United Kingdom water supply may be given priority. The emphasis for any one need may also change over short or longer periods of time. What appears to be axiomatic, however, is that none of these needs can be met without reliable streamflow data being available at the right time, the right place and the right quality.

Categories of streamflow data

The type of streamflow information required may be classified into two distinct categories. The first is that required for planning and design while the second is that required for current use, i.e. operational management.

Data for planning and design may not necessarily have an immediate use but are valuable in the long term for civil engineering works of various types and for flood forecasting and control. Planning and design data are also used to examine long-term trends as are data on the stream environment.

Current use data have an immediate high return value since they are invariably required initially for operation and control. Current use streamflow stations are operated for as long as the need remains.

Designers of water control and water-related facilities increasingly use the statistical characteristics of streamflow rather than flow over specific historic periods. The probability that the historical sequence of flow history at a given site will occur again is remote. Indeed, when a hydrologist makes just one measurement of discharge it is probable that the exact conditions under which the discharge occurred may rarely happen again.

It is often desirable to consider the future, not in terms of specific events, but in terms of probability of occurrence over a span of years. For example, many highway bridges are designed on the basis of the flood that will be exceeded on the average only once in 50 years. Storage reservoirs are designed on the basis of the probability of failure of a particular capacity to sustain a given draft rate. The water available for irrigation, dilution of waste or other purposes may be stated in terms of the mean flow, or probability of flow magnitudes, for periods of a year, season, month, week or day. In addition there is a trend towards flow simulation based on statistical characteristics, such as the mean, standard deviation and skew. To define statistical characteristics, a record of at least 30 years is desirable for reliable results and a study of Section 13.9 (Chapter 13) suggests caution in estimating trends or probabilities from short-term periods.

1.4 Cost effectiveness

In most countries the cost effectiveness of streamflow data collection is an important consideration; this is particularly the case where streamflow is included in the budget for water management. Cost effectiveness may be measured by the benefit:cost ratio, but to estimate this ratio for streamflow is difficult, mainly due to the problems associated with assessing the benefits accruing. This problem sometimes leaves the hydrological service at a disadvantage in bidding for funds.

The wide variety of uses of streamflow data also makes the estimation of national benefits difficult. The question of marginal gains through network changes is therefore not straightforward. It is, however, a fact that costs have risen sharply in providing gauging stations and in data capture and publication. The gains on the other hand are not easily quantified and each use of streamflow data may demand different and perhaps sophisticated analysis before benefits of streamflow data collection can be realised. This, however, is not usually the case in developing countries where the gain from a flood control scheme or an irrigation scheme may be enough to cover the cost of the entire network many times over. Benefit:cost ratios in these circumstances may be as high as 50:1 or more.

In other countries, however, a period of years may elapse before a useful record is generated to quantify the benefits of current data and even then any satisfactory assessment is complicated.

The first objective therefore is to develop a suitable method to identify potential quantifiable benefits to various types of data user. Such benefits are usually to be found in data required for reservoir design, water abstraction, flood warning, flood control including flood proofing, irrigation, highway bridge design, hydroelectric power generation, river pollution control, sewage purification and so on.

The costs of providing these services are quantified over a defined period and the benefits accruing from streamflow data are calculated for each. More often the benefits may have to be determined from an agreed percentage of the cost of the services.

Flood proofing, for example, reduces the cost of flood damage and this figure can be conveniently quantified from flood damage records. If no flood damage records exist, a percentage benefit based on the cost of the scheme can usually be calculated. The benefits are calculated for each use to which the streamflow data are put and totalled. This total is divided by the cost of obtaining the streamflow data. This is usually the cost of the operation of the gauging stations and processing the data or, more conveniently, the sum of capital and staff costs.

In a benefit–cost study of the UK streamflow network carried out for the Department of the Environment (DOE) in 1989, the annual benefits were found to be in the range US$16.5–90 million depending on how these were quantified, the best estimate being US$31.5 million. The annual cost of operating the network, including overheads, management, data processing, etc. was US$13.5 million. The benefit:cost ratio was therefore in the range 1.2–7 with a best estimate of 2.3. It was concluded, therefore, that even at the lowest level of benefit:cost ratio, the UK streamflow network represents a sound economic investment. The ratio would have been higher if some of the intangibles (e.g. consents for discharge effluents) could have been quantified. Not surprisingly only small annual economic benefits could be quantified from flood forecasting, flood warning or flood alleviation.

1.5 International standards in stream gauging

Water in a stream in a specific locality knows no jurisdictional boundaries, local or national. That same water may eventually move to any other part of the earth through the hydrological cycle. Streamflow data are therefore needed from all parts of the earth to enable hydrologists to discover the quantity of the earth's water resources on a comprehensive and continuous basis. Streamflow records that have been gathered by non-standard methods may be suspect. For this and other reasons, the International Organization for Standardization (ISO) set up in 1956, a technical committee on streamflow measurement. This committee, known as TC113, has produced a number of international standards on streamflow which are now used worldwide. Of the 104 ISO member countries, some 37 are members of TC113.

The methods described in this book generally follow the principles and recommendations of the ISO Standards.

In addition, the World Meteorological Organization (WMO), publishes guides and technical reports on stream gauging and selected ISO Standards, in the form of technical regulations which are circulated to some 187 WMO member countries.

Standardisation activity at the European level is the responsibility of CEN (European Committee for Standardization – Comité Européen de Normalisation) and CENELEC (European Committee for Electrotechnical Standardization). Together these bodies make up the Joint European Standards Institution (ESI). The aim of European standardisation is the harmonisation of standards on a Euro-wide basis in order to facilitate the exchange of goods and services by eliminating barriers to trade which might result from requirements of a technical nature. The national standardisation institutes of 27 countries support CEN. In addition, other European countries have affiliate status. Streamflow is under TC318 'Hydrometry', formed in 1994.

1.6 Summary of methods

A summary of the methods of streamflow measurement follows together with a reference to the chapters in which each method is discussed.

Velocity–area method (Chapter 2)

The discharge is derived from the sum of the products of stream velocity, depth and distance between verticals (Fig. 1.2), the stream velocity usually being obtained by a current meter. For a continuous record of discharge in a stable prismatic open channel with no variable backwater effects, a unique relation exists between water level (stage) and discharge. Once established, this stage–discharge relation is used to derive discharge values from recordings of stage. With the exception of the dilution method, which is a direct method, it could be inferred that all methods of streamflow measurement are based as the velocity–area principle.

The stage–discharge relation is covered in Chapter 4 and since the measurement of stage is one of the most important factors in all methods, a separate chapter is devoted to it (Chapter 3). Special problems, associated with velocity–area stations in particular, such as corrections for soundings from cableways, stilling well lag and draw-down, rapidly changing discharge and measurements under ice cover, are presented in Chapter 5.

Acoustic Doppler Current Profiler (ADCP) (Chapter 6)

The ADCP instrument is mounted on a motorised boat that moves across the river perpendicular to the current. Velocities are measured when the ADCP transmits acoustic pulses along three or four beams at a constant frequency. These beams are positioned at precise horizontal angles from each other and

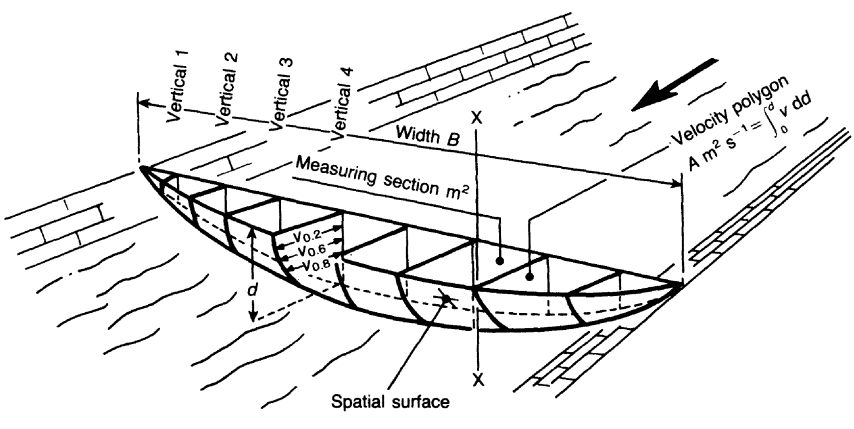

Figure 1.2 The measuring section. The volume of water is bounded by the measuring section, the water surface, the bed and the spatial surface as shown schematically. At any section XX, the area of the velocity polygon is the integral *vdd* (with limits from 0 to *d*) and equal to A m^2s^{-1}. The volume of water passing per second is then found from the integral of *Adb* (with limits from 0 to *b*) and equal to the integral of *vdd db* (with limits from 0 to *d* and 0 to *b*) which is equal to the total flow *Q* in m^3s^{-1}.

directed at a known angle from the vertical, typically 20° or 30° The instrument processes echoes throughout the water column along each beam. The difference in frequency (Doppler shift) between transmitted pulses and received echoes (Doppler effect) can be used to measure the relative velocity between the instrument and the suspended material in the water that reflects the pulses back to the instrument (backscattering). The ADCP uses the Doppler effect to compute a velocity component along each beam and the system software calculates velocity in three dimensions using trigonometric relations. The ADCP may therefore be regarded as a velocity–area method giving a single value of discharge, usually to provide a point on the stage–discharge curve.

Float gauging (Chapter 7)

The water velocity is measured by recording the time taken for a float to travel a known distance along the channel. Observations are made using floats at different positions across the channel and discharge is derived from the sum of the products of velocity, width and depth.

Generally, this method is used only when the flow is either too fast or too slow to use a current meter or where ice floes would cause damage to the meter.

Slope–area method (Chapter 8)

The discharge is derived from measurements of the slope of the water surface and the cross-section of the channel over a fairly straight reach, assuming a roughness coefficient for the channel boundaries.

Stage–fall–discharge method (Chapter 9)

In a stable open channel affected by backwater, a relation is established between fall (slope) and discharge.

Weirs and flumes (Chapter 10)

The relation between stage (or head) and discharge over a weir or through a flume is established from laboratory (or field) calibration. The discharge is subsequently derived from this rating equation.

Dilution method (Chapter 11)

A tracer liquid is injected into the channel and the water is sampled at a point further downstream where turbulence has mixed the tracer uniformly throughout the cross-section. The change in concentration between the solution injected and the water at the sampling station is converted into a measure of the discharge.

Moving boat method (Chapter 12)

A current meter is suspended from a boat which traverses the channel normal to the streamflow. The component of the velocity in the direction of the stream is computed from the resultant velocity and the angle of this resultant. The discharge is the sum of the products of the stream velocity, depth and distance between observation points.

Ultrasonic method (Chapter 12)

The velocity of flow is measured by transmitting an ultrasonic pulse diagonally across the channel in both directions simultaneously. The difference in time transits is a measure of the velocity which has to be multiplied by the cross-sectional area to derive discharge. The ultrasonic method therefore also follows the principles of velocity–area measurements.

Electromagnetic method (Chapter 12)

The discharge is found by measuring the electromotive force (emf) produced by a moving conductor (the flowing water) through a magnetic field produced by a coil placed either below or above the open channel. The emf is proportional to the discharge.

Accuracy (Chapter 13)

Considerable research into uncertainties in streamflow measurement over recent years has led to the publication of several international standards and this chapter has been updated from the first edition to address the latest methods of the assessment of uncertainties.

Hydrometric data processing (Chapter 14)

In view of the advances made in solid-state recording and data processing of streamflow measurements, this chapter has been completely rewritten but autographic chart recording methods have been retained.

Flow in pipes (Chapter 15)

Because of today's need for hydrologists to address all types of flow measurement, this chapter has been added to the present edition and includes flow in closed conduits under pressure and flow in partially filled pipes using both existing theory and practice and modern concepts.

1.7 Selection of method

Velocity–area method

Generally, consideration is given first to the possibility of installing a velocity–area station especially if it is known that a relation can be established between stage and discharge. Discharge measurements may then be carried out using a current meter by wading (when the depth and velocity permit), by cableway (when the span permits the installation of a cableway, and the river is too deep to wade), by boat (if the river is too wide for a cableway installation), by moving boat (if the river is wide enough), by floats (if the velocity is too low or too high to use a current meter or there are ice floes in the river), by slope–area (if no other method is suitable during floods) or from bridges (if these are considered suitable).

ADCP

The ADCP is now used in many countries giving very good results in the measurement of a single measurement of discharge. Systems now available are able to measure both large and small rivers and deep or shallow rivers when mounted on motor launches or small remote-controlled or tethered rafts or catamarans. A great advantage of the method is its speed whereby an ADCP measurement may be as much as ten-times quicker than a conventional measurement. The equipment now is considerably reduced in size and weight and units available now may be only a few kilograms in weight and less than 30 centimetres in height.

Weirs and flumes

In small rivers (under 100 m in width) a measuring structure may be considered, particularly if backwater conditions prevail. The main factors to be assessed for a measuring structure are cost, head loss (afflux) available, Froude number and bed conditions.

Flumes are normally only considered in smaller channels and especially in wastewater treatment works of which there are literally thousands in the UK alone.

Ultrasonic and electromagnetic methods

The ultrasonic and electromagnetic methods provide a continuous measurement of discharge for all designed stages of flow and continue to do so under backwater conditions even if the flow actually reverses due, for example, to tidal influence.

The main restrictions for the ultrasonic method are that a source of electrical power should be available, the river should not be more than about 300 m wide with suitable minimum depth and should have no weed growth or significant sediment transport.

The electromagnetic method also requires a source of power and is restricted to rivers about 40 m wide but continues to measure under weed conditions or heavy sediment load.

Dilution techniques

Dilution gauging is not in such general use as other methods because the technique requires specially trained staff. Nevertheless it is the most suitable method available for discharge measurements in turbulent mountain streams. It is used mainly for spot measurements especially in the calibration of other methods, for example measuring structures, but in certain situations it may be the only suitable method. It is also the only fully direct method for the measurement of discharge since the velocity, depth or area does not enter into the computation.

Stage–fall–discharge and slope–area methods

These methods are indirect methods of measurement, but have their place under conditions where the above methods are not suitable or are unavailable. The stage–fall–discharge method is particularly useful under backwater conditions especially in large rivers, when it may be the only suitable method. The slope–area method is useful in the measurement of floods, either current or historical, the latter from flood marks.

The stage–fall–discharge method may take the form of a permanent station; the slope–area method is used for measurements and may be employed at a permanent velocity–area station for measuring the highest flows. The latter method depends, however, on Manning's '*n*' or Chezy's '*C*' roughness coefficients and, unless these are established on site from measurements, the methods may have a large current or historical uncertainty.

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Chapter 2

The velocity–area method of streamflow measurement

2.1 General

The velocity–area method for the determination of discharge in open channels consists of measurements of stream velocity, depth of flow and distance across the channel between observation verticals. The velocity is measured at one or more points in each vertical by a current meter and an average velocity determined in each vertical. The discharge is derived from the sum of the product of mean velocity, depth and width between verticals. The discharge so obtained is normally used to establish a relation between water level (stage) and streamflow. Once established this stage–discharge relation is used to derive discharge values from records of stages at the gauging station.

Not all current meter measurements, however, are made to establish a stage–discharge relation and for many purposes individual determinations or 'spot measurements' are very often required for management functions. Such measurements may not require the measurement of stages but otherwise the method of measurement is the same. At some stations, however, a record of stages only may be required for purposes such as flood warning. At most gauging stations, however, both stages and discharges are measured to establish a relation between these two variables.

2.2 Spacing of verticals

In order to describe the bed shape and the horizontal and vertical velocity distributions completely, an infinite number of verticals would be necessary; for practical reasons, however, only a finite number is possible. In practice, therefore, the cross-section is divided into segments by spacing verticals at a sufficient number of locations across the channel to ensure an adequate sample of both velocity distribution and bed profile. The spacing and number of verticals are crucial for the accurate measurement of discharge and for this reason between 20 and 30 verticals are normally used. This practice applies to rivers of all widths except where the channel is so narrow that 20 or 30 verticals would be impracticable. We shall see in Chapter 13 that uncertainties in streamflow measurement are expressed as percentages. The percentage uncertainty therefore for using, say, 20 verticals is of the same order for all widths of river notwithstanding the width of the segments (in absolute terms the uncertainty will increase as the width of segment increases).

Verticals may be spaced on the basis of the following criteria:

(a) equidistant;
(b) segments of equal flow;
(c) bed profile.

The choice will depend largely on the flow conditions, the geometry of the cross-section and the width of river. For very wide rivers (over 300m), for example, it is sometimes convenient to make the verticals equidistant; for rivers having an asymmetrical horizontal velocity distribution, or a significant variation in the horizontal velocity distribution, it is normally advisable to space the verticals in such a manner so as to achieve segments of equal flow over the required range; for rivers having abnormalities in the bed profile, the verticals are spaced so as to make allowance for depressions or obtrusions and general irregularities of the bed. A general rule, however, for current meter measurements is to make the width of segments less as the depth and velocities become greater.

Irrespective of which criteria are followed, the spacing of the verticals is arranged so that no segment contains more than, say, 10% of the total flow. The best measurement is normally one having no segment with more than 5% of the total flow.

2.3 Computation of current meter measurements

Mid-section method

In the mid-section method of computation it is assumed that the velocity sampled at each vertical represents the mean velocity in a segment. The segment area extends laterally from half the distances from the preceding vertical to half the distance to the next, and from the water surface to the sounded depth as shown by the hatched area in Fig. 2.1. The segment discharge is then computed for each segment and these are summed up to obtain the total discharge. Referring to Fig. 2.1, which shows diagrammatically the cross-section of a stream channel, the discharge passing through segment 5 is computed as

$$q_5 = \bar{v}_5 \left(\frac{(b_5 - b_4) + (b_6 - b_5)}{2} \right) d_5 \tag{2.1}$$

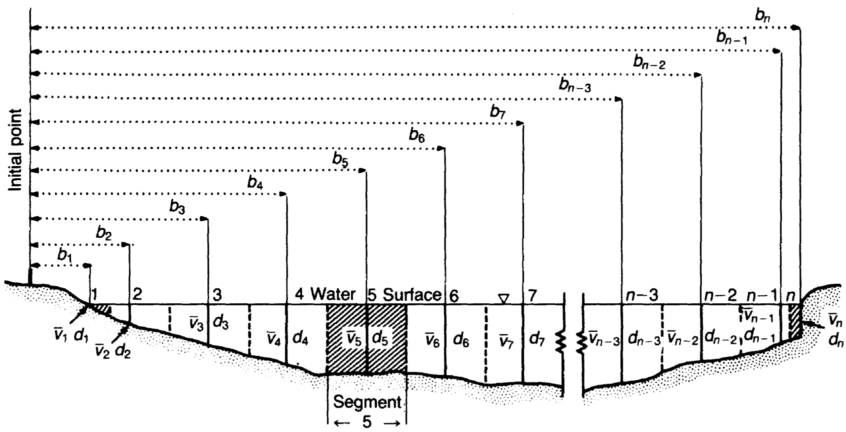

Figure 2.1 The mid-section method of computing current meter measurements. 1, 2, 3, . . ., n, number of vertical; $b_1, b_2, b_3, \ldots, b_n$, distance from initial point; $d_1, d_2, d_3 \ldots, d_n$, depth of flow at verticals; $\bar{v}$, average velocity in verticals.

$$= \bar{v}_5\left(\frac{b_6 - b_4}{2}\right)d_5 \tag{2.2}$$

where q_5 = discharge through segment 5;
$\bar{v}_5$ = mean velocity in vertical 5;
b_4, b_5, b_6 = distance from an initial point on the bank to verticals 4, 5 and 6;
d_5 = depth of flow at vertical 5.

For the end segment, 1, shown hatched, the discharge may be computed as

$$q_1 = \bar{v}_1\left(\frac{b_2 - b_1}{2}\right)d_1 \tag{2.3}$$

and the end segment, n, as

$$q_n = \bar{v}_n\left(\frac{b_n - b_{n-1}}{2}\right)d_n. \tag{2.4}$$

The preceding segment at the beginning of the cross-section is therefore considered coincident with vertical 1 and the next vertical at the end of the cross-section is considered coincident with vertical n.

In the example in Fig. 2.1, q_1 is zero because the depth at vertical 1 is zero. However, when the cross-section boundary is vertical at the edge of the water, the depth is not zero and the velocity at the end vertical may or may not be zero. The equations for q_1 and q_n are used whenever there is water on only one side of a vertical, such as piers, abutments and islands. It is usually necessary to estimate the velocity at the end segments as a percentage of the velocity on the adjacent vertical because it is not possible to locate the current meter close to a boundary. Alternatively, a current meter observation may be made as near the edge as possible and this velocity used in computing the discharge in the end segments. However, if the verticals 2 and $n - 1$ are placed as close as possible to the banks and the cross-section is wide, the discharge in in the end segments can normally be neglected. A typical computation of a current meter measurement employing the mid-section method is shown in Table 2.1. It will be noted in this example that 22 verticals have been used and that the discharge in any one segment does not exceed 10% of the total discharge.

Mean-section method

Segment discharges are computed between successive verticals. An example of one such segment is shown hatched in Fig. 2.2. The velocities and depths for successive verticals are each averaged, the segment discharge being the product of the two averages.

Referring to Fig. 2.2, the discharge passing through segment 5–6 is computed as

$$q_{5\text{–}6} = \left(\frac{\bar{v}_5 + \bar{v}_6}{2}\right)\left(\frac{d_5 + d_6}{2}\right)(b_6 - b_5) \tag{2.5}$$

where $q_{5\text{–}6}$ = discharge through segment 5–6;
$\bar{v}_5, \bar{v}_6$ = mean velocities in verticals 5 and 6;
d_5, d_6 = depth of flow at verticals 5 and 6;
b_5, b_6 = distance from an initial point on the bank to verticals 5 and 6.

It will be noted that the depth of flow at vertical 1 is zero and the problem of computing the flow in the end segments does not arise in this method nor does it arise when the bank is vertical and the velocity can be taken as approximately zero at the end vertical. The computation is therefore carried out for the end segments in exactly the same way as for the other segments. Nevertheless this facility does not give the mean-section method an overall advantage over the mid-section method, the latter being simpler to compute and therefore quicker if the calculations are being performed manually. There is little difference in time, however, if a pocket calculator is employed for the calculation.

Table 2.1 Typical computation for a current meter measurement by the mid-section method

(1) Verticals	*(2) Distance from initial point (m)*	*(3) Depth (m)*	*(4) Meter position*	*(5) Revs*	*(6) Time (s)*	*(7) Velocity*	*(8)*	*(9) Width (m)*	*(10) Area (m^2)*	*(11) Discharge ($m^3 s^{-1}$)*
						At point ($m s^{-1}$)	*Mean in vertical ($m s^{-1}$)*			
RB	4	0		0	0	0	0	0	0	0
1	5	0.31	0.6	40	60	0.193	0.193	1	0.31	0.060
2	6	0.40	0.6	45	59	0.219	0.219	1	0.40	0.089
3	7	0.51	0.6	51	61	0.238	0.238	1	0.51	0.121
4	8	0.85	0.6	52	61	0.243	0.243	1	0.85	0.206
5	9	1.23	0.2	55	60	0.260	0.235	1	1.23	0.289
			0.8	44	60	0.211				
6	10	1.58	0.2	58	62	0.265	0.240	1	1.58	0.379
			0.8	46	61	0.216				
7	11	1.69	0.2	60	61	0.278	0.251	1	1.69	0.424
			0.8	48	61	0.225				
8	12	1.71	0.2	65	62	0.295	0.274	1	1.71	0.468
			0.8	51	63	0.253				
9	13	1.87	0.2	70	62	0.317	0.287	1	1.87	0.537
			0.8	58	64	0.257				
10	14	1.84	0.2	69	62	0.313	0.287	1	1.84	0.528
			0.8	58	63	0.262				

11	15	1.71	0.2	66	61	0.305	0.278	1	1.71	0.475
			0.8	55	62	0.252				
12	16	1.65	0.2	62	61	0.287	0.262	1	1.65	0.432
			0.8	52	62	0.238				
13	17	1.50	0.2	60	61	0.278	0.258	1	1.50	0.387
			0.8	50	60	0.238				
14	18	1.36	0.2	58	62	0.265	0.241	1	1.36	0.328
			0.8	47	62	0.217				
15	19	1.19	0.2	55	61	0.257	0.228	1	1.19	0.271
			0.8	42	63	0.193				
16	20	1.17	0.2	51	62	0.235	0.211	1	1.17	0.247
			0.8	39	60	0.188				
17	21	0.92	0.6	46	61	0.216	0.216	1	0.92	0.199
18	22	0.81	0.6	41	63	0.188	0.188	1	0.81	0.152
19	23	0.70	0.6	39	61	0.184	0.184	1	0.70	0.129
20	24	0.63	0.6	36	63	0.167	0.167	1	0.63	0.105
21	25	0.55	0.6	31	61	0.150	0.150	1	0.55	0.082
22	26	0.48	0.6	26	64	0.125	0.125	1	0.36	0.045
LB	26.5	0		0	0	0	0	0	0	0
								Σ	24.54	5.953

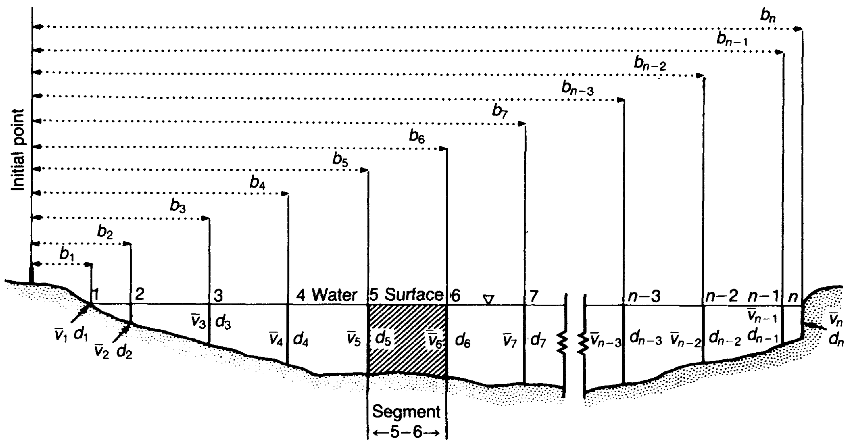

Figure 2.2 The mean-section method of computing current meter measurements. 1, 2, 3, . . . , n, number of vertical; $b_1, b_2, b_3, \ldots, b_n$, distance from initial point; $d_1, d_2, d_3, \ldots, d_n$, depth of flow at verticals; $\bar{v}$, average velocity in verticals.

Velocity–depth integration method

Whereas the previous two methods may be termed arithmetical methods of computing discharge, the velocity–depth integration method is a graphical method. If sufficient current meter observations have been made in the verticals, a curve of mean velocity × depth of flow (area of vertical velocity curve) may be drawn over the cross-section. The area of this curve represents the total discharge.

Referring to Fig. 2.3, the procedure is as follows:

(a) Draw the vertical velocity curve for each vertical by plotting the velocity observations against their corresponding depths of flow.
(b) Measure the area contained by each curve by planimeter.
(c) Plot these areas over the water surface line of the cross-section and draw a smooth curve through the points. The area enclosed between this curve and the water surface line represents the total discharge.

The areas contained by the curves are best measured by planimeter but if graph paper with millimetre divisions is used, the 10 mm squares can be counted to calculate the area with acceptable accuracy, making allowance for scale factors.

Velocity–contour method

This is also a graphical method and like the velocity–depth method described above requires a number of current meter observations in the verticals.

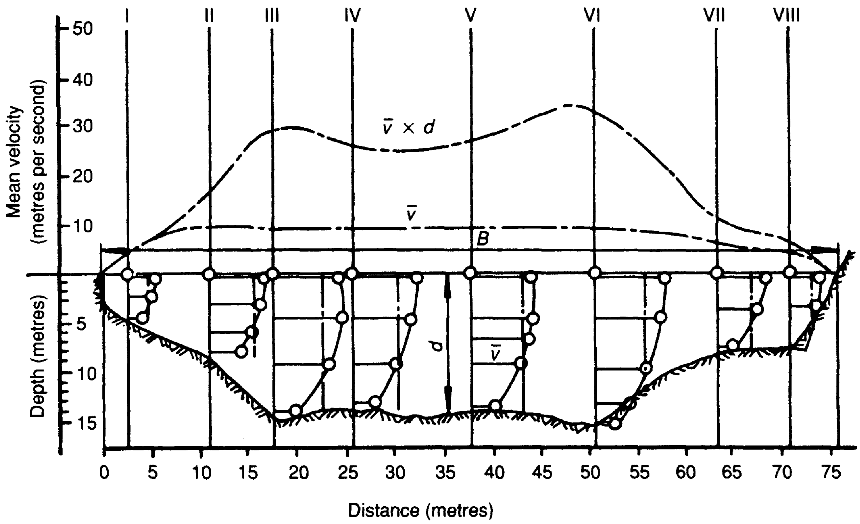

Figure 2.3 The velocity–depth integration method of computing current meter measurements. $Q = \Sigma_0^B \bar{v}d\,\Delta B$.

Referring to Fig. 2.4, the procedure is as follows:

(a) Vertical velocity distribution curves are drawn for each vertical.
(b) These curves are interpolated for convenient intervals of velocity (e.g. 0.25, 0.5, 0.75 m s^{-1}).
(c) Curves or contours of equal velocity (isovels) are drawn as shown in Fig. 2.4(a).
(d) Starting from the maximum, the areas enclosed by successive velocity contours are measured by planimeter and plotted on a diagram, as shown in Fig. 2.4(b), with the ordinate indicating velocity and the abscissa indicating the corresponding area enclosed by the respective velocity contour. The summation of the area enclosed by this curve represents the total discharge.

It can be seen from Fig. 2.4(b) that the maximum velocity plotted on the ordinate is 3.05 m s^{-1}, which in this example is found from the surface velocity distribution curve (Fig. 2.4(a)) and the maximum area plotted on the abscissa is about 1138 m^2, being the sum of the areas enclosed by each velocity contour and the water surface line.

Of the four methods of computation of discharge described above, the mid-section and mean-section methods are used almost universally. The two graphical methods are normally employed in special studies and in the investigation of velocity distribution. The use of graphical methods, however, does not relax the rule for the number or spacing of verticals.

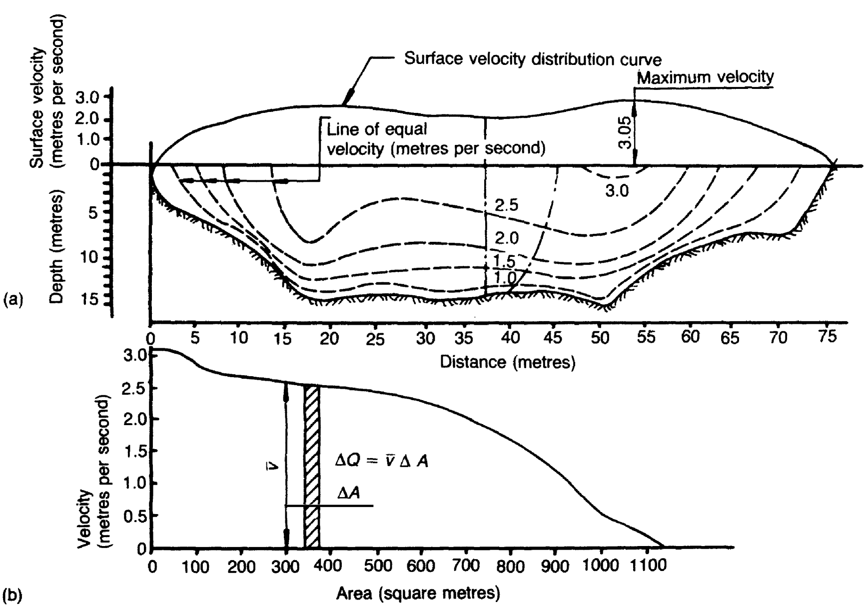

Figure 2.4 The velocity contour method of computing current meter measurements. $Q = \Sigma_0^A \bar{v}\,\Delta A$. (a) Velocity contours in a section, and (b) Total flow.

2.4 Measurement of velocity

The mean velocity in each vertical is determined by current meter observations by any of the following methods.

The velocity distribution method

In this method velocity observations are made in each vertical at a sufficient number of points distributed between the water surface and bed to define effectively the vertical velocity curve, the mean velocity being obtained by dividing the area between the curve and the plotting axes by the depth. The number of points required depends on the degree of curvature, particularly in the lower part of the curve, and usually varies between six and ten. Observations are normally made at 0.2, 0.6, and 0.8 of the depth from the surface, so that the results from the vertical velocity curve can be compared with various combinations of reduced points methods, and the highest and lowest points should be located as near to the water surface and bed as possible.

This method is the most accurate if done under ideal, steady-stage conditions but is not considered suitable for routine gauging due to the length of time required for the field observations and for the ensuing computation. It is used mainly for checking velocity distribution when the station is first established and for checking the accuracy of the reduced points methods.

The velocity curve may be extrapolated to the bed by the use of the following equation

$$v_x = v_a \left(\frac{x}{a} \right)^{1/c} \tag{2.6}$$

where v_x is the point velocity required in the extrapolated zone at distance x from the bed;
v_a is the velocity at the last measuring point on the velocity curve at distance a from the bed;
c is a constant varying from 5 for coarse beds to 7 for smooth beds and generally taken as 6.

Note: if $x = 0$ (bed level), v_x (at bed level) $= 0$.

An example of the use of equation (2.6) is as follows. In a velocity distribution measurement the lowest observation in the vertical was at a point 0.25 m from the bed. The value of the velocity at this point was 0.15 m s^{-1}. Find the approximate velocity at a point 0.1 m from the bed in order to complete the vertical velocity curve.

From equation (2.6)

$$\begin{aligned} v_x &= 0.15 \left(\frac{0.10}{0.25} \right)^{1/6} \\ &= 0.13 \text{ m s}^{-1}. \end{aligned}$$

An alternative method of obtaining the velocity in the region beyond the last measuring point, and so to complete the vertical velocity curve, is based on the assumption that the velocity for some distance up from the bed may often be taken as being proportional to the logarithm of the distance x from the bed. If the observed values of velocities, therefore, are plotted against corresponding values of $\log x$, the best-fitting straight line through these points can be extended to the bed. The required velocities close to the bed may then be read directly from the graph.

The 0.6 depth method

Velocity observations are made at a single point at 0.6 of the depth from the surface and the value obtained is accepted as the mean for the vertical. This assumption is based both on theory and on results of analysis of many vertical velocity curves, which showed that in the majority of cases the 0.6 method produced results of acceptable accuracy. The value of the method is its essential reliability, the ease and speed of setting the meter at a single point, and the reduced time necessary for completion of a gauging.

The 0.2 and 0.8 depth method

Velocity is observed at two points at 0.2 and 0.8 of the depth from the surface and the average of the two readings is taken as the mean for the vertical. Here again this assumption is based on theory and on the study of vertical velocity curves; experience has confirmed its essential accuracy. Generally the minimum depth of flow should be about 0.75 m when the 0.2 and 0.8 depth method is used.

Six-point method

Velocity observations are made by taking current meter readings on each vertical at 0.2, 0.4, 0.6 and 0.8 of the depth below the surface and as near as possible to the surface and bed. The mean velocity may be found by plotting in graphical form and using a planimeter, or from the equation

$$\bar{v} = 0.1\ (v_{\text{surface}} + 2v_{0.2} + 2v_{0.4} + 2v_{0.6} + 2v_{0.8} + v_{\text{bed}}) \tag{2.7}$$

where v is the velocity.

Five-point method

Velocity observations are made by taking current meter readings on each vertical at 0.2, 0.6, and 0.8 of the depth below the surface and as near as possible to the surface and bed. The mean velocity may be found by plotting in graphical form and using a planimeter, or from the equation

$$\bar{v} = 0.1\ (v_{\text{surface}} + 3v_{0.2} + 3v_{0.6} + 2v_{0.8} + v_{\text{bed}}). \tag{2.8}$$

Equations (2.7) and (2.8) are established from the area of a plane surface by a simple arithmetical procedure. In the six-point method, for example, the surface area of the curve ($\bar{v}D$) is approximately

$$(v_1 \times 0.1D + v_2 \times 0.2D + v_3 \times 0.2D + v_4 \times 0.2D + v_5 \times 0.2D + v_6 \times 0.1D)\ \mathrm{m^2s^{-1}}$$

and the average velocity is found by dividing the total depth by D, giving

$$0.1\ (v_1 + 2v_2 + 2v_3 + 2v_4 + 2v_5 + v_6)\ \mathrm{m\ s^{-1}}.$$

Similarly, equation (2.8) is established from the surface area of the curve giving

$$(v_1 \times 0.1D + v_2 \times 0.3D + v_3 \times 0.3D + v_4 \times 0.2D + v_5 \times 0.1D)\ \mathrm{m^2s^{-1}}$$

and dividing by D gives the average velocity as

$$0.1\ (v_1 + 3v_2 + 3v_3 + 2v_4 + v_5)\ \mathrm{m\ s^{-1}}.$$

Three-point method

Velocity observations are made by taking current meter readings on each vertical at 0.2, 0.6 and 0.8 of the depth below the surface. The average of the three values may be taken as the mean velocity in the vertical. Alternatively the 0.6 measurement may be weighted and the mean velocity obtained from the equation

$$\bar{v} = 0.25\ (v_{0.2} + 2v_{0.6} + v_{0.8}). \tag{2.9}$$

The origin of the average velocity occurring at 0.6 of the depth and also at the average of 0.2 and 0.8 of the depth from the surface is based essentially on the theoretical velocity distribution of velocity in an open channel. For the condition of turbulent flow over a rough boundary the vertical velocity curves have approximately the form of a parabola whose axis, coinciding with the filament of maximum velocity, is parallel with the surface and is in general situated between the surface and one-third of the depth of the water from the bed. As the depth and velocity increases, however, the curve approaches a vertical line in its limiting position (this fact is used to advantage in the moving boat method (p. 82) where the current meter is located at approximately 1 m from the surface).

The vertical distribution of velocity may be expressed approximately by the equation

$$v = \left(\frac{D-d}{a}\right)^{1/c} \tag{2.10}$$

where v is the velocity at depth d below the water surface, c is a coefficient usually having a value of 6 (equation (2.6)), D is the total depth of flow and a is a constant numerically equal to the distance above the bottom of the channel of a point at which the velocity has unit value.

Now integrating equation (2.10) for $\bar{v}$ (average velocity in the vertical)

$$\bar{v} = \frac{1}{D}\int_0^D v\ \mathrm{d}d$$

$$= \frac{1}{D}\int_0^D \left(\frac{D-d}{a}\right)^{1/c} \mathrm{d}d$$

$$= \frac{1}{D}\left[-\frac{ac}{c+1}\left(\frac{D-d}{a}\right)^{1/c+1}\right]_0^D \tag{2.11}$$

then

$$\bar{v} = \frac{c}{c+1}\left(\frac{D}{a}\right)^{1/c}. \tag{2.12}$$

Now making $v = \bar{v}$ in equation (2.10)

$$\frac{c}{c+1}\left(\frac{D}{a}\right)^{1/c} = \left(\frac{D-d}{a}\right)^{1/c} \tag{2.13}$$

and

$$\left(\frac{c}{c+1}\right)^{c} = \frac{D-d}{D}.$$

Hence

$$\frac{d}{D} = 1 - \left(\frac{c}{c+1}\right)^{c} \tag{2.14}$$

and substituting values of c between 5 and 8 in equation (2.14), d/D is approximately equal to 0.6.

Now if $v_{0.2}$ is the velocity at depth $0.2D$, $v_{0.8}$ is the velocity at depth $0.8D$ and $\bar{v} = \frac{1}{2}(v_{0.2} + v_d)$, then from equations (2.10) and (2.12)

$$\frac{1}{2}\left[\left(\frac{D-0.2D}{a}\right)^{1/c} + \left(\frac{D-d}{a}\right)^{1/c}\right] = \frac{c}{c+1}\left(\frac{D}{a}\right)^{1/c}$$

so

$$\frac{D-d}{a} = \frac{D}{a}\left[\frac{2c}{c+1} - (0.8)^{1/c}\right]^{c}$$

and

$$d = D - D\left[\frac{2c}{c+1} - (0.8)^{1/c}\right]^c. \tag{2.15}$$

Substituting $c = 6$ in equation (2.15) gives d/D approximately equal to 0.82. Therefore

$$\bar{v} \doteqdot \tfrac{1}{2}(v_{0.2} + v_{0.8}).$$

Similarly if $\bar{v} = \frac{1}{2}(v_d + v_{0.8})$ then

$$d = D - D\left[\frac{2c}{c+1} - (0.2)^{1/c}\right]^c \tag{2.16}$$

and substituting $c = 6$ in equation (2.16) gives d/D approximately equal to 0.27. Therefore $\bar{v} \doteqdot \frac{1}{2}(v_{0.2} + v_{0.8})$, as before.

The foregoing theory is normally applicable to large rivers when the time of exposure of the current meter is sufficient to equalise pulsations but in general the 0.6 and 0.2 + 0.8 depth methods are almost universally used and give acceptable results. Also it will be noted that in Chapter 13 the uncertainty in the measurement of velocity due to the limited number of points taken in the vertical, X_p, is divided by m, the number of verticals. However, many rivers do not necessarily follow the theoretical parabolic velocity distribution even when the time of exposure of the meter is several minutes. In such situations and where sufficient depth is available, special gaugings are sometimes taken using the five-point or six-point method using a single rod with five or six meters attached to it and employing a special counter box to record the current meter observations.

2.5 Current meters

The current meter is still the most universally used instrument for velocity determination. The principle is based upon the relation between the speed of the water and the resulting angular velocity of the rotor. By placing a current meter at a point in a stream and counting the number of revolutions of the rotor during a measured time interval, the velocity of the water at that point can be determined. The number of revolutions of the rotor is obtained by various means depending on the design of the meter but normally this is achieved by an electric circuit through the contact chamber. Contact points or a reed switch in the chamber are designed to complete an electric circuit at selected frequencies of revolution, normally once per revolution, but also at frequencies of twice per revolution or once for five revolutions depending on design. In the case of the Braystoke propeller meter, a diametrically opposed pair of small cylindrical permanent magnets is inserted at the rear of the impeller boss. This operates an electrical reed switch enclosed in a glass envelope located in the current meter body. From the switch a twin-conductor cable carries the electric pulses generated by each revolution of the impeller to the counter device. In all types of design the electrical impulses produce either a signal which registers a unit on a counting device or an audible signal in a headphone. Intervals of time are measured by a stopwatch or by an automatic timing device. Latest developments in current meter design include the introduction by the United States Geological Survey of an optical head pick-up which improves low velocity response. This new pick-up system utilises a pivot bearing in the head and is actuated by a rotating fibre-optic bundle. The system generates four counts per revolution.

The SonTek Flow Tracker hand-held acoustic Doppler current meter for wading measurement can now be classed as a unique member of the current meter genre (Fig. 2.7c). Like the propeller and cup-type meters it is a point velocity meter but with the important innovation that it has an optional two or three dimensional velocity probe. It has a powerful software-friendly package with a keypad custom-designed for velocity and discharge measurements.

The electromagnetic current meter (Figure 2.6d and also Figure 2.12b) is a point velocity meter as distinct from the electromagnetic total flow meter described in Chapter 2. It has no moving parts and is of solid-state construction; it has the ability to measure very low velocities of the order of millimetres per second.

Another innovation is the solid-state current meter digitiser used in both the United States and the United Kingdom. This device counts the number of revolutions and at the end of the count period illuminates revolutions and seconds on a light emitting diode (LED) display. This value is held for a few seconds and then the velocity is displayed based on the time and counts recorded and the rating equations for the specific current meter in use.

Cup-type and propeller-type current meters

Current meters can be classified generally as those having vertical axis rotors (Fig. 2.5) and those having horizontal axis rotors (Fig. 2.6(a)(b)(c)), the former being known as cup-type current meters and the latter as propeller-type meters.

The cup-type current meter consists of a rotor revolving about a vertical shaft and hub assembly, bearings, main frame, a contact chamber containing the electrical contact, tail fin and means of attaching the instrument to rod or cable suspension equipment. The rotor is generally constructed of six conical cups fixed at equal angles on a ring mounted on the vertical shaft. This assembly is retained in the main frame by means of an upper shaft bearing and a lower pivot bearing.