Measurement and Computation of Streamflow: Volume 1. Measurement of Stage and Discharge

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PREFACE

The science and, in part, art of stream gaging has evolved through the years, largely from the collective experiences and innovations of its practitioners. The earliest truly comprehensive manual on stream-gaging procedures and equipment, based on techniques developed up to that time, was published in 1943 by the Geological Survey as Water-Supply Paper 888, "Stream-Gaging Procedures," by D. M. Corbett and others. That report was an instant classic; it was received enthusiastically by the hydraulic-engineering profession throughout the world and became a major training document for at least two generations of stream gagers, hydrologists, and hydraulic engineers, both in the United States and abroad.

The need for updating Water-Supply Paper 888 has been obvious for some time as a result of later developments in stream-gaging techniques and equipment. Furthermore, it has been felt for some time that the scope of that report should be expanded to cover not only the newer field techniques but also the office procedures required to produce a published annual record of stream discharge. In recognition of those needs, the Geological Survey has sporadically produced supplementary reports that update the description of specific techniques and equipment or that describe specific field or office procedures not covered by Water-Supply Paper 888. These reports—some administrative, some open-file, and others in the published series titled "Techniques of Water-Resources Investigations"—are fragmentary in the sense that they deal only with selected aspects of the broad subject of stream gaging. Similar, but less detailed, reports on selected topics have also been produced by international agencies in the form of Technical Notes by the World Meteorological Organization and in the form of Recommendations and Standards by the International Standards Organization.

The present report evolved from that background. Its two volumes provide a comprehensive compilation of time-tested techniques presented as an up-to-date (1980) standardized manual of stream-gaging procedures.

Acknowledgments.—The material presented in the manual is based on the work of many hydraulic engineers and technicians, each of whom has contributed to the current state-of-the-art in the measurement and computation of streamflow. Many of those who contributed directly to the manual are listed in the references appended to each chapter. However, the authors wish to acknowledge a special group of colleagues who have made major contributions to this manual in the form of written (cited) methods and techniques, significant suggestions for correction and revision of the manuscript and illustrations, and all-out efforts to bring about development and publication of the material. These colleagues include:

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The authors also wish to acknowledge with thanks the suggestions of their many other colleagues, who are too numerous to mention.

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CONVERSION FACTORS

[Factors for converting inch-pound to metric units are shown to four significant figures However, in the text the metric equivalents, where shown, are carried only to the number of significant figures consistent with the values for the English units.]

Inch-pound	*Multiply by—*	*Metric*
acres	4.047×10^{3}	m^2 (square meters)
acre-ft (acre-feet)	1.233×10^{3}	m^3 (cubic meters)
acre-ft/yr (acre feet per year)	1.233×10^{3}	m^3/yr (cubic meters per year)
ft (feet)	3.048×10^{-1}	m (meters)
ft/hr (feet per hour)	3.048×10^{-1}	m/hr (meters per hour)
ft/s (feet per second)	3.048×10^{-1}	m/s (meters per second)
ft^3/s (cubic feet per second)	2.832×10^{-2}	m^3/s (cubic meters per second)
in (inches)	2.540×10	mm (millimeters)
lb (pounds)	4.536×10^{-1}	kg (kilograms)
mi (miles)	1.609	km (kilometers)
mi^2 (square miles)	2.590	km^2 (square kilometers)
oz (ounces)	2.835×10^{-2}	kg (kilograms)

MEASUREMENT AND COMPUTATION OF STREAMFLOW

VOLUME 1. MEASUREMENT OF STAGE AND DISCHARGE

By S. E. RANTZ and others

CHAPTER 1.—INTRODUCTION

PURPOSE OF THE MANUAL

The purpose of this manual is to provide a comprehensive description of state–of–the–art standardized stream-gaging procedures, within the scope described below. The manual is intended for use as a training guide and reference text, primarily for hydraulic engineers and technicians in the U.S. Geological Survey, but the manual is also appropriate for use by other stream-gaging practitioners, both in the United States and elsewhere.

SCOPE OF THE MANUAL

The technical work involved in obtaining systematic records of streamflow is discussed, in two volumes, in accordance with the following six major topics:

Volume 1. Measurement of stage and discharge
- a. Selection of gaging-station sites
- b. Measurement of stage
- c. Measurement of discharge

Volume 2. Computation of discharge
- d. Computation of the stage-discharge relation
- e. Computation of daily-discharge records
- f. Presentation and publication of stream-gaging data

In order to make the text as broadly usable as possible, discussions of instrumentation and measurement are aimed at the technician, and discussions of computational procedure are aimed at the junior engineer who has a background in basic hydraulics.

Many of the procedures for determining discharge that are discussed in volume 2 require specialized instrumentation to obtain field data that supplement the observation of stage. The descriptions of such specialized equipment and associated observational techniques are given in appropriate chapters in volume 2 so that the reader may have unified discussions of the methodologies applicable to each type of problem in determining discharge.

In general the authors have attempted to prepare a manual that will stand independently—references are given to supplementary published material, but the reader should find relatively few occasions when there is pressing need to consult those references. There are three notable exceptions to that statement.

1. The subject of indirect determination of peak discharge (v. 1, chap. 9) is treated here only in brief because of space limitations; the subject is treated fully in five reports in the Geological Survey report series, "Techniques of Water-Resources Investigations." The five reports are named in the reference section of chapter 9.
2. Among the methods discussed in this manual for computing the discharge of tidal streams are four mathematical techniques for evaluating the differential equations of unsteady flow (v. 2, chap. 13). The four techniques are given only cursory treatment because a detailed description of the complex mathematical techniques is considered to be beyond the scope of the manual.
3. The processing of streamflow records by digital computer (v. 2, chap. 15) is a subject that is given only generalized treatment here. It was not practicable to include a detailed description of each step in the sequence of operation of an automated computing system because of space limitations, and also because the particulars of each step are somewhat in a state of flux in response to continual improvement in storage and access procedures.

STREAMFLOW RECORDS

Streamflow serves man in many ways. It supplies water for domestic, commercial, and industrial use; irrigation water for crops; dilution and transport for removal of wastes; energy for hydroelectric power generation; transport channels for commerce; and a medium for recreation. Records of streamflow are the basic data used in developing reliable surface-water supplies because the records provide information on the availability of streamflow and its variability in time and space. The records are therefore used in the planning and design of surface-water related projects, and they are also used in the management or operation of such projects after the projects have been built or activated.

Streamflow, when it occurs in excess, can create a hazard—floods cause extensive damage and hardship. Records of flood events obtained at gaging stations serve as the basis for the design of bridges, culverts, dams, and flood-control reservoirs, and for flood-plain delineation and flood-warning systems.

The streamflow records referred to above are primarily continuous records of discharge at stream-gaging stations, a gaging station being a stream-site installation so instrumented and operated that a continuous record of stage and discharge can be obtained. Networks of stream-gaging stations are designed to meet the various demands for streamflow information, including inventory of the total water resource. The networks of continuous-record stations, however, are often augmented by auxiliary networks of partial-record stations to fill a particular need for streamflow information at relatively low cost. For example, an auxiliary network of sites, instrumented and operated to provide only instantaneous peak-discharge data, is often established to obtain basic information for use in regional flood-frequency studies. An auxiliary network of uninstrumented sites for measuring low flow only is often established to provide basic data for use in regional studies of drought and of fish and wildlife maintenance or enhancement.

GENERAL STREAM-GAGING PROCEDURES

After the general location of a gaging station has been determined from a consideration of the need for streamflow data, its precise location is so selected as to take advantage of the best locally available conditions for stage and discharge measurement and for developing a stable stage-discharge relation.

A continuous record of stage is obtained by installing instruments that sense and record the water-surface elevation in the stream. Discharge measurements are initially made at various stages to define the relation between stage and discharge. Discharge measurements are then made at periodic intervals, usually monthly, to verify the stage-discharge relation or to define any change in the relation caused by changes in channel geometry and (or) channel roughness. At many sites the discharge is not a unique function of stage; variables other than stage must also be continuously measured to obtain a discharge record. For example, stream slope is measured by the installation of a downstream auxiliary stage gage at stations where variable backwater occurs. At other sites a continuous measure of stream velocity at a point in the cross section is obtained and used as an additional variable in the discharge rating. The rate of change of stage can be an important variable where flow is unsteady and channel slopes are flat.

Artificial controls such as low weirs or flumes are constructed at some stations to stabilize the stage-discharge relations in the low-flow range. These control structures are calibrated by stage and discharge measurements in the field.

The data obtained at the gaging station are reviewed and analyzed by engineering personnel at the end of the water year. Discharge ratings are established, and the gage-height record is reduced to mean values for selected time periods. The mean discharge for each day and extremes of discharge for the year are computed. The data are then prepared for publication.

SELECTED REFERENCE

Carter, R. W., and Davidian, Jacob, 1968, General procedure for gaging streams: U.S. Geol. Survey Techniques Water-Resources Inv., book 3, chap. A6, p. 1–2.

CHAPTER 2.—SELECTION OF GAGING-STATION SITES

INTRODUCTION

The general location of a gaging station is dependent on the specific purpose of the streamflow record. If the streamflow record is needed for the design or operation of a water project, such as a dam and reservoir, the general location of the gaging station obviously will be in the vicinity of the water project. The selection of a gaging-station site becomes complicated, however, when the station is to be one of a network of stations whose records are required for study of the general hydrology of a region. Such studies are used to inventory the regional water resource and formulate long-range water-development plans. In that situation, attention to hydrologic principles is required in selecting the general locations of the individual stations in the network to ensure that optimum information is obtained for the money spent in data collection.

A discussion of the design of gaging-station networks is beyond the scope of this manual and for the purpose of this chapter we will assume that the general location of a proposed gaging station has been determined. The discussion that follows will be concerned with the hydraulic considerations that enter into the selection of the precise location of the gage to obtain the best locally available conditions for the measurement of stage and discharge and for the development of a stable discharge rating.

CONSIDERATIONS IN SPECIFIC SITE SELECTION

After the general location of a gaging station has been determined, a specific site for its installation must be selected. For example, if the outflow from a reservoir is to be gaged to provide the streamflow data needed for managing reservoir releases, the general location of the gaging station will be along the stretch of stream channel between the dam and the first stream confluence of significant size downstream from the dam. From the standpoint of convenience alone, the station should be established close to the dam, but it should be far enough downstream from the outlet gates and spillway outlet to allow the flow to become uniformly established across the entire width of the stream. On the other hand the gage should not be located so far downstream that the stage of the gaged stream may be affected by the stage of the confluent stream. Between those upstream and downstream limits for locating the gage, the hydraulic features should be investigated to obtain a site that presents the best possible conditions for stage and discharge measurement and for developing a stable stage-discharge relation. If the proposed gaging station is to be established for purely hydrologic purposes, unconnected with the design or operation of a project, the general location for the gage will be the stretch of channel between two large tributary or confluent streams. The same consideration will apply in the sense that the gage should be far enough downstream from the upper tributary so that flow is fairly uniformly established across the entire width of stream, and far enough upstream from the lower stream confluence to avoid variable backwater effect. Those limits often provide a reach of channel of several miles whose hydraulic features must be considered in selecting a specific site for the gage installation.

The ideal gage site satisfies the following criteria:

1. The general course of the stream is straight for about 300 ft (approx. 100 m) upstream and downstream from the gage site.
2. The total flow is confined to one channel at all stages, and no flow bypasses the site as subsurface flow.
3. The streambed is not subject to scour and fill and is free of aquatic growth.
4. Banks are permanent, high enough to contain floods, and are free of brush.
5. 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 falls or cascade that is unsubmerged at all stages (chap. 3).
6. A pool is present upstream from the control at extremely low stages to ensure a recording of stage at extremely low flow, and to avoid high velocities at the streamward end of gaging-station intakes during periods of high flow.
7. The gage site is far enough upstream from the confluence with another stream or from tidal effect to avoid any variable influence the other stream or the tide may have on the stage at the gage site.
8. A satisfactory reach for measuring discharge at all stages is available within reasonable proximity of the gage site. (It is not nec-

essary that low and high flows be measured at the same stream cross section.)

9. The site is readily accessible for ease in installation and operation of the gaging station.

Rarely will an ideal site be found for a gaging station and judgment must be exercised in choosing between adequate sites, each of which has some shortcomings. Often, too, adverse conditions exist at all possible sites for installing a needed gaging station, and a poor site must be accepted. For example, all streams in a given region may have unstable beds and banks, which result in continually changing stage-discharge relations.

The reconnaissance for a gaging site properly starts in the office where the general area for the gage site is examined on topographic, geologic, and other maps. Reaches having the following pertinent characteristics should be noted: straight alinement, exposed consolidated rock as opposed to alluvium, banks subject to overflow, steep banks for confined flow, divided channels, possible variable backwater effect from a tributary or confluent stream or from a reservoir, and potential sites for discharge measurement by current meter. The more favorable sites will be given critical field examination; they should be marked on the map, access roads should be noted, and an overall route for field reconnaissance should be selected.

In the field reconnaissance the features discussed earlier are investigated. With regard to low flow, a stable well-defined low-water control section is sought. In the absence of such a control, the feasibility of building an artificial low-water control is investigated. If a site on a stream with a movable bed must be accepted—for example, a sand-channel stream—it is best to locate the gage in as uniform a reach as possible, away from obstructions in the channel, such as bridges, which tend to intensify scour and fill. (See page 377.) Possible backwater resulting from aquatic growth in the channel should also be investigated. If the gage is to be located at a canyon mouth where the stream leaves the mountains or foothills to flow onto an alluvial plain or fan, reconnaissance current-meter measurements of discharge should be made during a low-flow period to determine where the seepage of water into the alluvium becomes significant. The station should be located upstream from the area of water seepage in order to gage as much of the surface flow as possible; the subsurface flow or underflow that results from channel seepage is not "lost" water, but is part of the total water resource.

With regard to high stages, high-water marks from major floods of the past are sought and local residents are questioned concerning historic flood heights. Such information is used by the engineer in making a judgment decision on the elevation at which the stage-recorder must be placed to be above any floods that are likely to occur in the future. The recorder shelter should be so located as to be sheltered from waterborne debris during major floods. Evidence is also sought concerning major channel changes, including scour and deposition at streambanks, that occurred during notable floods of the past. That evidence, if found, gives some indication of changes that might be expected from major floods of the future.

The availability of adequate cross sections for current-meter measurement of discharge should also be investigated. Ideally, the measurement cross section should be of fairly uniform depth, and flow lines should be parallel and fairly uniform in velocity throughout the cross section. The measurement section should be in reasonable proximity to the gage to avoid the need for adjusting measured discharge for change in storage, if the stage should change rapidly during a discharge measurement. However a distance of as much as 0.5 mi (approx. 1 km) between gage and measuring section is acceptable if such a distance is necessary to provide both a good stage-measurement site and a good discharge-measurement site. Low-flow discharge measurements of all but the very large streams are made by wading. For flows that cannot be safely waded, the current meter is operated from a bridge, cableway, or boat. It is most economical to use an existing bridge for that purpose, but in the absence of a bridge, or if the measuring section at a bridge site is poor, a suitable site should be selected for constructing a cableway. If construction of a cableway is not feasible because of excessive width of the river, high-water measurements will be made by boat when safe to do so. The cross section used for measuring high flows is rarely suitable for measuring low flows, and wading measurements are therefore made wherever measuring conditions are most favorable.

Consideration should be given to the possibility of variable backwater effect from a stream confluence or reservoir downstream from the general location of the required gaging station. Without knowledge of stage and discharge at a potential gage site and of concurrent stage at the stream confluence or reservoir, the engineer can only conjecture concerning the location on the stream where backwater effect disappears for various combinations of discharge and stage. A safe rule is the following: Given a choice of several acceptable gaging sites on a stream, the gaging site selected should be the one farthest upstream from the possible source of variable backwater. If it is necessary to accept a site where variable backwater occurs, a uniform reach for measurement of slope should be sought, along with a site for the installation of an auxiliary gage. If a gaging station must be placed in a tide-affected reach, the unsteady flow that must be gaged will also require an auxiliary gage, but in addition line power must be available to insure the synchronized recording of stage at the two gages. The availability of line power or telephone lines is also a consideration, where needed for special instrumentation or for the telemetering units that are often used in flood-forecasting and flood-warning systems.

In cold regions the formation of ice always presents a problem in obtaining reliable winter records of streamflow. However in regions that are only moderately cold, and therefore subject to only moderate ice buildup, forethought in the selection of gage sites may result in streamflow records that are free of ice effect. Gage sites that are desirable from that standpoint are as follows:

1. Below an industrial plant, such as a paper mill, steel mill, thermal powerplant, or coal mine. "Waste" heat may warm the water sufficiently or impurities in the water may lower the freezing point to the extent that open-water conditions always prevail.
2. Immediately downstream from a dam with outlet gages. Because the density of water is maximum at a temperature of 4°C, the water at the bottom of a reservoir is commonly at or near that temperature in winter. Most outlet gates are placed near the bottom of the dam, and the water released is therefore approximately 4°C above freezing. It would take some time for that water to lose enough heat to freeze.
3. On a long fairly deep pool just upstream from a riffle. A deep pool will be a tranquil one. Sheet ice will form readily over a still pool, but the weather must be extremely cold to give complete cover on the riffle. At the first cold snap, ice will form over the pool and act as an insulating blanket between water and air. Under ice cover the temperature of the streambed is generally slightly above the freezing point and may, by conduction and convection, raise the water temperature slightly above freezing, even though water enters the pool at 0°C. That rise in temperature will often be sufficient to prevent ice formation on the riffle.

After the many considerations discussed on the preceding pages have been weighed, the precise sites for the recording stage gage and for the cableway for discharge measurements (if needed) are selected. Their locations in the field are clearly marked and referenced. The maximum stage at which the low-water control will be effective should be estimated; the intakes to the stage recorder should be located upstream from the low-water control, a distance equal to at least three times the depth of water on the control at that estimated maximum stage. If the intakes are located any closer than that to the control, they may lie in a region where the streamlines have vertical curvature; intake location in that region is hydraulically undesirable.

The gaging station on the Kaskaskia River at Bondville, Ill., shown in figure 1, satisfies most of the requirements discussed in this chapter. Low-flow measurements are made by wading upstream from the control. The bridge site provides accessibility, convenience to power lines, and a good location for an outside reference gage, which is shown on the downstream parapet wall of the bridge.

Up to this point there has been no discussion of specific site location for crest-stage gages. Those gages provide peak-discharge data only. Where possible they should be installed upstream from road culverts, which act as high-water controls. The specific site for a gage is at a distance of one culvert width upstream from the culvert inlet. In the absence of such control structures, the crest-stage gage should be installed in a straight reach of channel that can be utilized in computing peak discharge by the slope-area method (chap. 9).

SELECTED REFERENCES

Carter, R. W., and Davidian, Jacob, 1968, General procedure for gaging streams: U.S. Geol. Survey Techniques Water-Resources Inv., book 3, chap. A6, p. 2–3.

World Meteorological Organization, 1974, Guide to hydrometeorological practices [3d ed.]: WMO-no. 168, p. 3.1–3.4, 3.13–3.16.

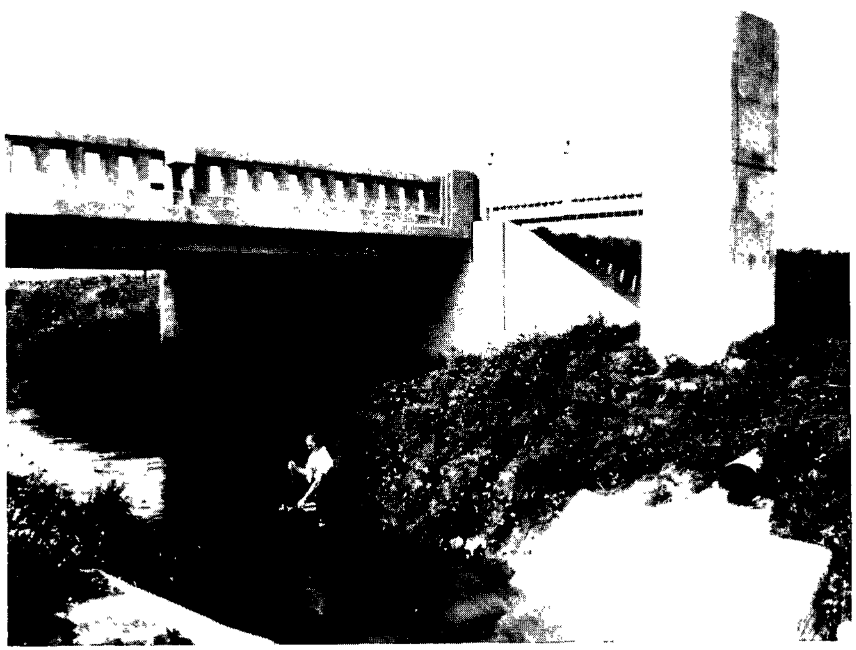

FIGURE 1.—Gage, concrete control, outside gage on bridge, and an engineer making a wading measurement, Kaskaskia River at Bondville, Ill.

CHAPTER 3.—GAGING-STATION CONTROLS

TYPES OF CONTROL

The conversion of a record of stage to a record of discharge is made by the use of a stage-discharge relation. The physical element or combination of elements that controls the relation is known as a control. The major classification of controls differentiates between section controls and channel controls. Another classification differentiates between natural and artificial controls. (Artificial controls are structures built for the specific purpose of controlling the stage-discharge relation; a highway bridge or paved floodway channel that serves incidentally as a control is not classed as an artificial control.) A third classification differentiates between complete, partial, and compound controls.

Section control exists when the geometry of a single cross section a short distance downstream from the gage is such as to constrict the channel, or when a downward break in bed slope occurs at the cross section. The constriction may result from a local rise in the streambed, as at a natural riffle or rock ledge outcrop, or at a constructed weir or dam; or it may result from a local constriction in width, which may occur naturally or be caused by some manmade channel encroachment, such as a bridge whose waterway opening is considerably narrower than the width of the natural channel. Examples of a downward break in bed slope are the head of a cascade or the brink of a falls.

Channel control exists when the geometry and roughness of a long reach of channel downstream from the gaging station are the elements that control the relation between stage and discharge. The length of channel that is effective as a control increases with discharge. Generally speaking, the flatter the stream gradient, the longer the reach of channel control.

A complete control is one that governs the stage-discharge relation throughout the entire range of stage experienced at the gaging station. More commonly, however, no single control is effective for the entire range of stage, and so the result is a compound control for the gaging station. A common example of a compound control is the situation where a section control is the control for low stages and channel control is effective at high stages. The compound control sometimes includes two section controls, as well as channel control. In that situation the upstream section control is effective for the very low stages, a section control farther downstream is effective for intermediate stages, and channel control is effective at the high stages.

With regard to complete controls, a section control may be a complete control if the section control is a weir, dam, cascade, or falls of such height that it does not become submerged at high discharges. A channel control may be a complete control if a section control is absent, as in a sand channel that is free of riffles or bars, or in an artificial channel such as a concrete-lined floodway.

A partial control is a control that acts in concert with another control in governing the stage-discharge relation. That situation exists over a limited range in stage whenever a compound control is present. As an example consider the common situation where a section control is the sole control for low stages and channel control is solely operative at high stages. At intermediate stages there is a transition from one control to the other, during which time submergence is "drowning out" the section control. During that transition period the two controls act in concert, each as a partial control. In effect we have a dam (low-flow control) being submerged by tailwater; before complete submergence (channel control), the upstream stage for the particular discharge is dependent both on the elevation of the dam and on the tailwater elevation. Where the compound control includes two section controls, the degree of submergence of the upstream section control will be governed by the downstream section control, during a limited range in stage. When that occurs, each of the section controls is acting as a partial control. A constriction in channel width, unless unusually severe, usually acts as a partial control, the upstream stage being affected also by the stage downstream from the constriction.

ATTRIBUTES OF A SATISFACTORY CONTROL

The two attributes of a satisfactory control are permanence (stability) and sensitivity. If the control is stable, the stage-discharge relation will be stable. If the control is subject to change, the stage-discharge relation is likewise subject to change, and frequent discharge measurements are required for the continual recalibration of the stage-discharge relation. That not only increases the operating cost of a gaging station, but results in impairment of the accuracy of the streamflow record.

The primary cause of changes in natural controls is the high velocity associated with high discharge. Of the natural section controls, a rock ledge outcrop will be unaffected by high velocities, but boulder, gravel, and sand-bar riffles are likely to shift, boulder riffles being the most resistant to movement and sand bars the least resistant. Of the natural channel controls those with unstable bed and banks, as found in sand-channel streams, are the most likely to change as a result of velocity-induced scour and deposition.

Another cause of changes in natural controls is vegetal growth. The growth of aquatic vegetation on section controls increases the stage for a given discharge, particularly in the low-flow range. Vegetal growth on the bed and banks of channel controls also affects the stage-discharge relation by reducing velocity and the effective waterway area. In the temperate climates, accumulations of waterlogged fallen leaves on section controls each autumn clog the interstices of alluvial riffles and raise the effective elevation of all natural section controls. The first ensuing stream rise of any significance usually clears the control of fallen leaves.

Controls, particularly those for low flow, should be sensitive; that is, a small change in discharge should be reflected by a signficant change in stage. To meet that requirement it is necessary that the width of flow at the control be greatly constricted at low stages. In a natural low-water control such constriction occurs if the control is in effect notched, or if the controlling cross section roughly has a flat V-shape or a flat parabolic shape. Those shapes will ensure that the width of flow over the control decreases as discharge decreases. Generally speaking, a low-water control is considered to be sensitive if a change of no more than 2 percent of the total discharge is represented by a change of one unit of recorded stage. For example, in the U.S.A., stage is recorded in units of hundredths of a foot; therefore, for the low-water control to be regarded as sensitive, a change in stage of 0.01 ft (0.003 m) should represent a change of no more than about 2 percent of the total discharge.

In the interest of economy a gaging station should be located upstream from a suitable natural control (fig. 2). However, where natural conditions do not provide the stability or the sensitivity required, artificial controls should be considered. The artificial controls are all section controls; it is not feasible to pave or otherwise improve a long reach of channel solely for the purpose of stabilizing the stage-discharge relation.

ARTIFICIAL CONTROLS

An artificial control is a structure built in a stream channel to stabilize and constrict the channel at a section, and thereby simplify the procedure of obtaining accurate records of discharge. The artificial controls built in natural streams are usually broad-crested weirs that conform to the general shape and height of the streambed. (The term "broad-crested weir," as used in this manual, refers to any type of weir other than a thin-plate weir.) In canals and drains, where the range of discharge is limited, thin-plate weirs and flumes are the controls commonly built. Thin-plate weirs are built in those channels whose flow is sediment free and whose banks are high enough to accommodate the increase in stage (backwater) caused by the installation of a weir (fig. 3). Flumes are largely self cleaning and can therefore be used in channels whose flow is sediment laden, but their principal advantage in canals and drains is that they cause relatively little backwater (head loss) and can therefore be used in channels whose banks are relatively low. Flumes are generally more costly to build than weirs.

Flumes may be categorized with respect to the flow regime that principally controls the measured stage; that is, a flume is classed as either a critical-flow flume or a supercritical-flow flume. The Parshall flume (fig. 4) is the type of critical-flow flume most widely used. Supercritical-flow flumes are used in streams that transport heavy loads of sediment that include rocks that are too large to pass through a critical-flow flume without being deposited in the structure. Of those flumes the trapezoidal supercritical-flow flume (fig. 5) is the most widely used by the U.S. Geological Survey.

Artificial controls eliminate or alleviate many of the undesirable characteristics of natural section controls. Not only are they physically stable, but they are not subject to the cyclic or progressive growth of aquatic vegetation other than algae. Algal slimes that sometimes form on artificial controls can be removed with a wire brush, and the controls are self cleaning with regard to fallen leaves. In moderately cold climates artificial controls are less likely to be affected by the formation of winter ice than are natural controls. The artificial control can, of course, be designed to attain the degree of sensitivity required for the gaging station. In addition, an artificial

FIGURE 2.—Gage and natural control, Little Spokane River at Elk, Wash.

control may often provide an improved discharge-measurement section upstream from the control by straightening the original angularity of flow lines in that cross section.

In canals or drains, where the range of discharge is limited, artificial controls are usually built to function as complete controls throughout the entire range in stage. In natural channels it is generally impractical to build the control high enough to avoid submergence at high discharges, and the broad-crested weirs that are usually built are effective only for low, or for low and medium, discharges. Figures 6 and 7 illustrate broad-crested weirs of two different shapes; the crests of both have a flat upward slope from the center of the stream to the banks, but in addition, the weir in figure 7 has a shallow V-notch in the center for greater sensitivity.

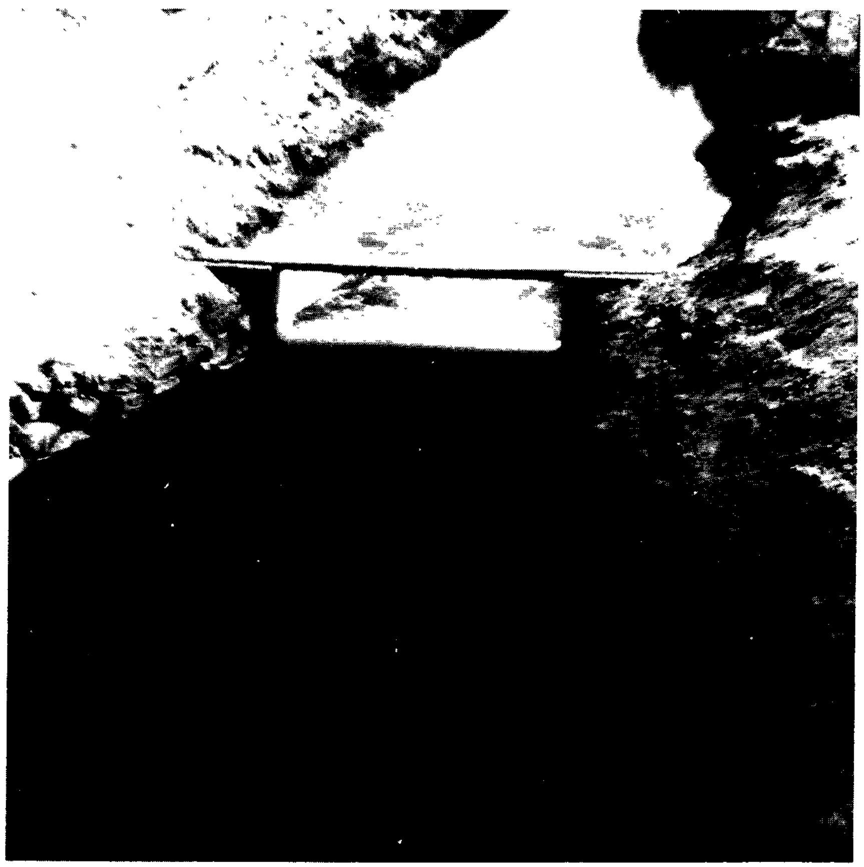

FIGURE 3.—Rectangular weir on Trifolium Drain 23 of Imperial Irrigation District, near Westmorland, Calif. (View looking downstream.)

The attributes desired in an artificial control include the following:

1. The control should have structural stability and should be permanent. The possibility of excessive seepage under and around the control should be considered, and the necessary precautions should be taken for the prevention of seepage by means of sheet piling or concrete cut-off walls and adequate abutments.
2. The crest of the control should be as high as practicable to possibly eliminate the effects of variable downstream conditions or to limit those effects to high stages only.
3. The profile of the crest of the control should be designed so that a small change in discharge at low stages will cause a measurable change in stage. If the control is intended to be effective at all stages, the profile of the crest should be designed to give a stage-discharge relation (rating curve) of such shape that it can be extrapolated to peak stages without serious error.
4. The shape of the control structure should be such that the passage of water creates no undesirable disturbances in the channel upstream or downstream from the control.

FIGURE 4.—Four-foot Parshall flume discharging 62 ft^3/s (1.76 m^3/s) under free-flow conditions. Scour protection is required with this height of fall. (Courtesy U.S. Bureau of Reclamation.)

5. If the stream carries a heavy sediment load, the artificial control should be designed to be self-cleaning. Flumes have that attribute; broad-crested weirs can often be made self-cleaning by a design modification in which the vertical upstream face of the weir is replaced by an upstream apron that slopes gently from the streambed to the weir crest.

Artificial controls are often built in conformance with the dimensions of laboratory-rated or field-rated weirs or flumes. The question arises whether to use the precalibrated rating or to calibrate in place each new installation. There are two schools of thought on the subject. In many countries, the precalibrated rating is accepted, and discharge measurements by current meter or by other means are made only periodically to determine if any statistically significant changes in the rating have occurred. If a change is detected, the new rating is defined by as many discharge measurements as are deemed neces-

FIGURE 5.—Flow through a 3-ft trapezoidal supercritical-flow flume showing transition from subcritical to supercritical flow.

sary. In other countries, including the U.S.A., the position is taken that it is seldom desirable to accept the rating curve prepared for the model structure without checking the entire rating of the prototype structure in the field by current-meter measurements, or by other methods of measuring discharge. The experience in the U.S.A. and elsewhere has been that differences between model and prototype invariably exist, if only in approach-channel conditions, and these differences are sufficient to require complete in place calibration of the prototype structure. In place calibration is sometimes dispensed with where the artificial control is a standard thin-plate weir having negligible velocity of approach.

CHOICE OF AN ARTIFICIAL CONTROL

Cost is usually the major factor in deciding whether or not an artificial control is to be built to replace an inferior natural control. The cost of the structure is affected most by the width of the stream and by the type or condition of bed and bank material. Stream width governs the size of the structure, and bed and bank material govern the type of construction that must be used to minimize leakage under and around the structure.

If an artificial control is to be used, the type and shape of the structure to be built is dependent upon channel characteristics, flow conditions, range of discharge to be gaged, sensitivity desired, and the maximum allowable head loss (backwater).

FIGURE 6.—Concrete artificial control on Mill Creek near Coshocton, Ohio.

CHOICE BETWEEN WEIR AND FLUME

As a general rule a weir is more advantageous for use as a control structure than a flume for the following reasons.

1. Weirs are usually cheaper to build than flumes.
2. A weir can be designed to have greater sensitivity at low flows with less sacrifice of range of discharge that can be accommodated, than is possible with a flume. Sensitivity is increased for a weir by notching the crest or by shaping the longitudinal profile of the crest in the form of a flat V or catenary. Sensitivity at low flow for a flume is usually attained by narrowing the effective width (converging sidewalls), or by using a trapezoidal cross section (narrower width at low stages), or by doing both; those measures significantly reduce the capacity of the flume.
3. The high water end of the stage-discharge relation for a weir can be extended beyond the stages at which the weir is effective as a control, with more confidence than can be done for a flume. In other words the stage-discharge relation for a flume becomes completely uncertain when any part of the structure upstream from the stage-measurement site is overtopped; the stage-discharge relation for a weir becomes completely uncertain only when the bank or abutment is overtopped and flow occurs around the end(s) of the weir.

If a flume is installed with the expectation that it will be overtop-

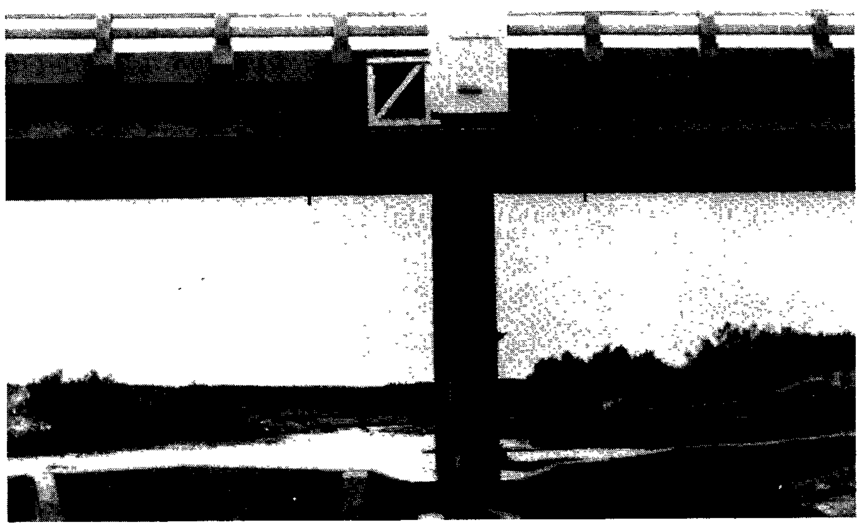

FIGURE 7.—Artificial control on Delaware River near Red Bluff, New Mex., with shallow V notch in the broad-crested weir.

ped by floodflows, it is advisable to install a nonrecording stage gage in a straight reach of channel, upstream or downstream from the flume but beyond the influence of the flume structure. Simultaneous observations of stage at the flume and in the unobstructed channel can be used to obtain a relation between the two stage gages. A stage-discharge relation for the site of the nonrecording gage can thus be derived from the flume rating, up to the discharge at which overflow starts at the flume. If for some reason, such as one of those discussed on page 273, high-flow discharge measurements are not available, the stage-discharge relation for the nonrecording gage can be extrapolated to flood stages; that extrapolation can then be used with the stage relation to define the overflow portion of the discharge rating for the flume.

However, in the following situations the use of a flume is more advantageous than the use of a weir.

1. If a heavy sediment load is carried by the stream, a weir will trap the sediment; accumulation of the sediment in the approach reach will alter the stage-discharge relation by changing the velocity of approach. Flumes however are usually self cleaning by virtue of their converging sidewalls and (or) steep floor. They therefore are more likely to maintain an unchanging stage-discharge relation. However, if the size of the sediment is not large, it is often possible to make a broad-crested weir self cleaning by including in its design an upstream apron that slopes downward from crest. An apron slope of 1 vertical to 5 horizontal will usually result in the transport of small sediment over the weir.
2. Weirs are usually not suitable for use in steep channels where the Froude number ($V/\sqrt{gd}$) is greater than about 0.75. Best results are obtained with weirs where the velocity head is a very minor part of the total head, and that is not the case in steep channels where the velocity head becomes excessively large. Furthermore, as explained above, weirs act as sediment traps on steep streams.
3. Flumes create less backwater for a given discharge than do weirs. Consequently it is more advantageous to use a flume if the channel has low banks. However for most natural streams it is impractical to design any type of structure to act solely as a metering device for the entire range of stage that may be experienced. In other words, for most natural streams sporadic overbank flooding is to be expected, with or without a control structure in the stream. Consequently the advantage of reduced backwater for a flume is usually realized only where the flow can be controlled to give some preassigned value of maximum dis-

charge. That situation occurs only in diversion canals or in natural streams having a bypass floodway.

CHOICE BETWEEN CRITICAL-FLOW FLUME AND SUPERCRITICAL-FLOW FLUME

After it has been decided for a particular site that the use of a flume is more desirable than the use of a weir, a decision must be made as to whether to use a critical-flow flume or a supercritical-flow flume.

Both types of flume will transport debris of considerable size without deposition in the structure, but if the transported rocks are excessively large, they may be deposited at or immediately upstream from the critical-depth section of a critical-flow flume. That will cause a change in the discharge rating of the flume. Therefore where that situation is likely to occur, a supercritical-flow flume should be selected for use.

If a critical-flow flume will pass the transported sediment load, that type of flume should be selected for use because the discharge rating for a critical-flow flume is more sensitive than that for a supercritical-flow flume.

SUMMARY

The artificial controls recommended for natural streams are as follows:

Sediment-transport characteristics	*Recommended structure*
Light load, small-size sediment	Weir.
Medium load, medium-size sediment	Parshall flume.
Heavy load, large-size sediment	Trapezoidal supercritical-flow flume.

The adjectives used above—light, medium, heavy, small, and large—are relative. Observation of both the study site and of the operation of control structures installed in an environment similar to that of the study site will provide the principle basis for the selection of the optimum type of control for use. In short, do not use a flume where a weir will do; if the use of a flume is indicated, do not use a supercritical-flow flume where a critical-flow flume (Parshall flume) will do.

The above recommendations also apply to canals or natural channels whose flow is controlled to limit the maximum discharge to some preassigned value. However, there is one exception; if the banks have little freeboard at the maximum discharge before the installation of an artificial control, a flume installation is generally preferable to a weir, in order to minimize the backwater caused by the structure.

DESIGN OF AN ARTIFICIAL CONTROL

Having decided on the type of artifical control to be used—weir, Parshall flume, or trapezoidal supercritical-flow flume—the next step is to design the structure. A standard design will usually be used, although channel conditions may make it necessary to make minor modification of the standard dimensions of the structure selected. The four factors—channel characteristics, range of discharge to be gaged, sensitivity desired, and maximum allowable head loss (backwater)—must be considered simultaneously in the precise determination of the shape, dimensions, and crest elevation of the control structure. However, two preliminary steps are necessary.

First, the head-discharge relations for various artificial controls of standard shape and of the type selected are assembled. Several such relations are to be found in hydraulics handbooks (King and Brater, 1963; World Meterological Organization, 1971). In addition, head-discharge relations for artificial controls that were field calibrated will usually be available in the files of water-resources agencies in the area—some are given in chapter 10. The head-discharge relations that are assembled need only be approximately correct, because channel conditions at the site of the proposed control will seldom match those for the model control.

The second preliminary step is to determine an approximate stage-discharge relation for the anticipated range in stage in the unobstructed channel at the site of the proposed control. That may be done by the use of an open-channel discharge equation, such as the Manning equation, in which uniform flow is assumed for the site and a value of the roughness coefficient is estimated. The reliability of the computed stage-discharge relation will be improved if one or more discharge measurements are made to verify the value of the roughness coefficient used in the computations. The purpose of the computations is to determine the tailwater elevation that is applicable to any given discharge after an artificial control is installed.

The next step is to consider the lower discharges that will be gaged. The tailwater elevations corresponding to those discharges are used to determine the minimum crest elevation permissible for the proposed artificial control at the lower discharges, under conditions of free flow or allowable percentage of submergence. If a flume is to be installed, the throat section should be narrow enough to ensure sensitivity at low discharges. If a weir is to be installed and the stream is of such width that a horizontal crest will be insensitive at low discharges, the use of a flat V crest is recommended—for example, one whose sides have a slope of 1 vertical to 10 horizontal. The sensitivity desired is usually such that the discharge changes no more than 2 percent for each change in stage of 0.01 ft (0.003 m). For a weir or critical-flow flume (Parshall flume) it is also desirable that the minimum discharge to be gaged have a head of at least 0.2 ft (0.06 m) to eliminate the effects of surface tension and viscosity.

Stage-discharge relations for the several controls under consideration are next prepared for the anticipated range in discharge. A structure is then selected that best meets the demands of the site in acting as a control for as much of the range as possible, without exceeding the maximum allowable backwater (head loss) at the higher stages and with minor submergence effect and acceptable sensitivity at lower stages. In other words, a high crest elevation minimizes submergence but maximizes backwater effect which may cause or aggravate flooding; a low crest elevation maximizes submergence but minimizes backwater effect; and where flumes are concerned, the attainment of the desired range of discharge may require some sacrifice of sensitivity at extremely low discharges. The engineer must use judgment in selecting a control design that is optimum for the local conditions. For example, to design a flume whose throat is sufficiently wide to accommodate the desired range of discharge, it may be necessary to relax the 2-percent sensitivity criterion at extremely low flows; in that situation the engineer may accept a sensitivity such that a change in stage of 0.01 ft (0.003 m) results in as much as a 5-percent change in discharge.

From a practical viewpoint the use of artificial controls is limited to streams with stable channels. Artificial controls seldom operate satisfactorily in sand channels having highly mobile beds. The transport of sediment as bedload continually changes the characteristics of the approach channel, and the control itself may be buried or partially buried by the movement of sand dunes.

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CHAPTER 4.—MEASUREMENT OF STAGE

GENERAL

The stage of a stream or lake is the height of the water surface above an established datum plane. The water-surface elevation referred to some arbitrary or predetermined gage datum is called the "gage height." Gage height is often used interchangeably with the more general term "stage," although gage height is more appropriate when used to indicate a reading on a gage. Stage or gage height is usually expressed in feet and hundredths of a foot, or in meters and hundredths or thousandths of a meter.

Records of gage height are used with a stage-discharge relation in computing records of stream discharge. The reliability of the discharge record is therefore dependent on the reliability of the gage-height record as well as on the accuracy of the stage-discharge relation. Records of stream stage are also useful in themselves for such purposes as the design of structures affected by stream elevation and the planning of flood-plain use. The gage-height record of a lake or reservoir provides, in addition to elevations, indexes of surface area and volume of the water body.

A record of stage may be obtained by systematic observations of a nonrecording gage or by means of a water-stage recorder. Special-purpose gages that do not give a complete record of stage are discussed on pages 74–78. The advantages of the nonrecording gage are the low initial cost and the ease of installation. The disadvantages are the need for an observer and the lack of accuracy of the estimated continuous-stage graph drawn through the plotted points of observed stage. For long-term operation the advantages of the recording gage far outweigh those of the nonrecording gage, and therefore the use of the nonrecording gage as a base gage is not recommended. However, at a recording-gage station, one or more nonrecording gages should be maintained as auxiliary gages for the operation of the station (p. 53–54). Telemetering systems are often used to transmit gage-height information to points distant from the gaging station (p. 54–59).

DATUM OF GAGE

The datum of the gage may be a recognized datum, such as mean sea level, or an arbitrary datum plane chosen for convenience. An arbitrary datum plane is selected for the convenience of using relatively low numbers for gage heights. To eliminate the possibility of minus values of gage height, the datum selected for operating purposes is below the elevation of zero flow on a natural control. Where an artificial control is used, the gage datum is usually set at the elevation of zero flow.

As a general rule a permanent datum should be maintained so that only one datum for the gage-height record is used for the life of the station. An exception occurs at gage sites where excessive streambed scour, after installation of the station, results in low-flow stages having a negative gage height. In that situation a change in gage datum to eliminate the negative numbers is recommended, to avoid possible confusion involving the algebraic sign of the gage heights. Another exception occurs when channel changes at a station make it impractical to maintain the station at the existing site. It may then be necessary to move the station a distance, such that there is significant fall in the water-surface elevation between the old and new sites, even though discharges at the two sites are equivalent. In that situation there is generally little to be gained by establishing the datum at the new site at the same sea-level elevation as the datum at the original site. That is especially true if the station is moved downstream a distance such that negative gage heights would result from use of the original datum. When datum changes are made for whatever reason, a record of the change should be a part of the published station description.

In any event, when a station is established, it should be assumed that a permanent datum will be maintained at the site. To maintain a permanent datum, each gaging station requires at least two or three reference marks; that is, permanent points of known gage-height elevation that are independent of the gage structure. The datum at each gaging station is periodically checked by running levels from the reference marks to the gages at the station.

If an arbitrary datum plane is used, it is desirable that it be referred by levels to a bench mark of known elevation above mean sea level, so that the arbitrary datum may be recovered if the gage and reference marks are destroyed.

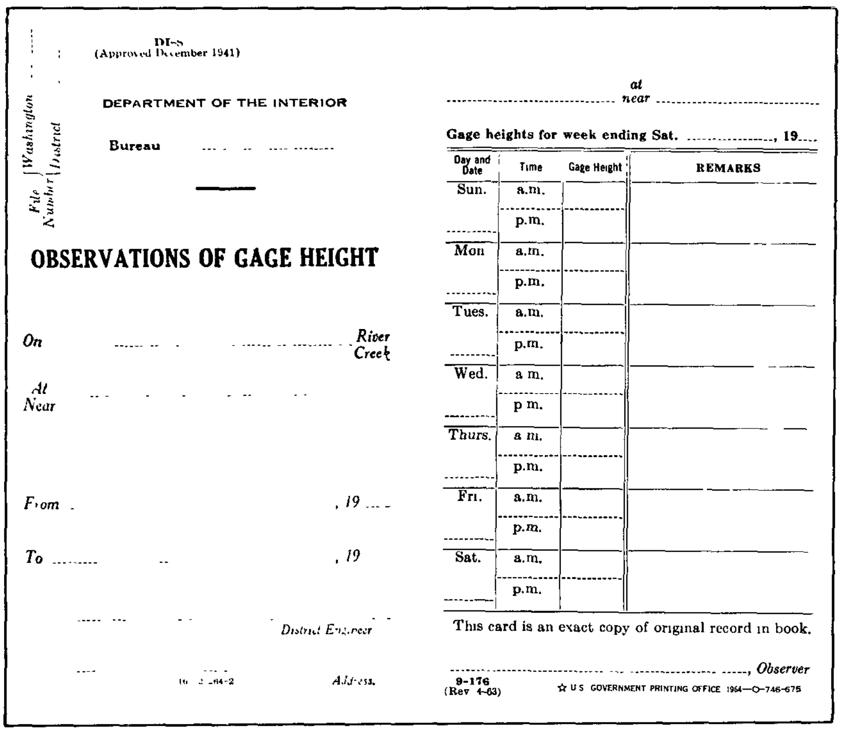
NONRECORDING STREAM-GAGING STATIONS

On page 23 it was mentioned that a record of stage could be obtained by systematic observation of a nonrecording gage. The advantages (low initial cost) and disadvantages (need for an observer, lack of accuracy) of a nonrecording gaging station were briefly discussed. On pages 53–54 the use of nonrecording gages as auxiliary and as reference (base) gages at recording-gaging stations will be discussed. Chapter 15 (p. 559–560) will describe the manner in which an observer's gage readings are used to compute a record of stage at a nonrecording gaging station. This section of the manual describes the various types of nonrecording gages.

Of the five types to be described, the two most generally used at nonrecording stations are either the staff or wire-weight gage. At such stations the nonrecording gage is usually read twice daily by an observer, and additional readings are made during periods of rapidly changing stage. The observer systematically records and reports his readings to headquarters. The record book and report cards shown in figure 8 are used by the U.S. Geological Survey, the book being the permanent record of nonrecording-gage readings. The weekly report card serves as an interim report of the observations made. The weekly report cards are read promptly on their arrival at the headquarters office so that any problems or difficulties that arise can be handled without delay.

On each routine visit to a nonrecording stream-gaging station, the hydrographer also visits the observer to enter in the stage-record book the gage reading(s) that the hydrographer has made. At that time he also inspects the record book to check for discrepancies in the observer's readings. Such visits by the hydrographer are important; if they are not made the observer tends to feel that no one pays any attention to his work, and he may become less conscientious about his gage readings.

Descriptions of five types of nonrecording stage gages follow. The special-purpose crest-stage gage and its use as an adjunct to the non-recording gaging station are described on pages 77–78.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(Approved December 1941)

File Number { Washington / District

DEPARTMENT OF THE INTERIOR

Bureau

OBSERVATIONS OF GAGE HEIGHT

On .... River / Creek

At / Near

From .... , 19....

To .... , 19

District Engineer

Address.

at / near

Gage heights for week ending Sat. ...., 19....

Day and Date	Time	Gage Height	REMARKS
Sun.	a.m.		
	p.m.		
Mon	a.m.		
	p.m.		
Tues.	a.m.		
	p.m.		
Wed.	a.m.		
	p.m.		
Thurs.	a.m.		
	p.m.		
Fri.	a.m.		
	p.m.		
Sat.	a.m.		
	p.m.		

This card is an exact copy of original record in book.

........, Observer

9-176
(Rev 4-63)

☆ U S GOVERNMENT PRINTING OFFICE 1964—O-746-675

FIGURE 8.—Cover of book and weekly report card for recording manual gage observations.

STAFF GAGE

Staff gages are either vertical or inclined. The standard Geological Survey vertical staff gage consists of porcelain-enameled iron sections, each 4 in (0.1 m) wide, 3.4 ft (1.04 m) long, and graduated every 0.02 ft (0.0067 m). (See fig. 9.) The vertical staff gage is used in stilling wells as an inside reference gage, or in the stream as an outside gage.

An inclined staff gage is usually a graduated heavy timber securely attached to permanent foundation piers. Inclined gages built flush with the streambank are less likely to be damaged by floods, floating ice, or drift than are projecting vertical staff gages. Copper barrelhoop staples and bronze numerals are generally used for the graduations. Inclined gages are used only as outside gages.

WIRE-WEIGHT GAGE

The wire-weight gage used in the U.S.A. is known as the type A wire-weight gage. It consists of a drum wound with a single layer of cable, a bronze weight attached to the end of the cable, a graduated disc, and a Veeder counter, all within a cast-aluminum box. (See fig. 10.) The disc is graduated in tenths and hundredths of a foot and is permanently connected to the counter and to the shaft of the drum. The cable is made of 0.045-in-diameter stainless-steel wire and is guided to its position on the drum by a threading sheave. The reel is equipped with a pawl and ratchet for holding the weight at any desired elevation. The diameter of the drum of the reel is such that each complete turn represents a 1-ft (0.305 m) movement of the weight. A horizontal checking bar is mounted at the lower edge of the instrument so that when it is moved to the forward position the bottom of the weight will rest on it. The gage is set so that when the bottom of the weight is at the water surface, the gage height is indicated by the combined readings of the counter and the graduated disc. The type A wire-weight gage is commonly mounted on a bridge handrail, parapet wall, or pier for use as an outside gage.

FLOAT-TAPE GAGE

A float-type gage consists of float, graduated steel tape, counterweight, and pulley. (See fig. 11.) The float pulley is usually 6 in (0.152 m) in diameter, grooved on the circumference to accommodate the tape, and mounted in a standard. An arm extends from the standard to a point slightly beyond the tape to carry an adjustable index. The tape is connected to the float by a clamp that also may be used for making adjustments to the tape reading if the adjustments necessary are too

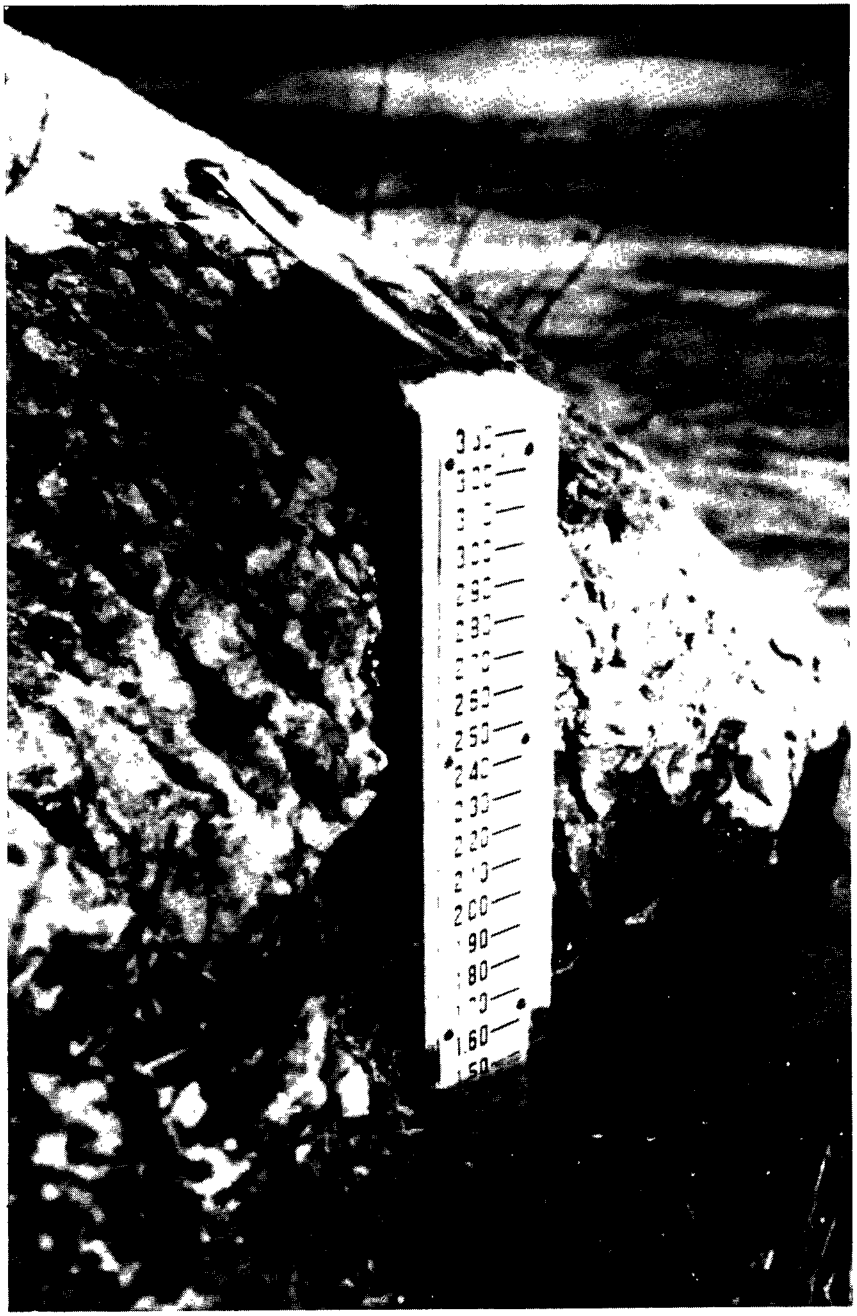

FIGURE 9.—Vertical staff gage.

large to be accommodated by the adjustable index. A 10-in (0.25 m)-diameter copper float and a 2-lb (0.9 kg) lead counterweight are normally used. The float-type gage is used chiefly in stilling wells as an inside reference or auxiliary gage.

ELECTRIC-TAPE GAGE

The electric-tape gage consists of a steel tape graduated in feet and hundredths of a foot, to which is fastened a cylindrical weight, a reel in a frame for the tape, a 4½-volt battery., and a voltmeter. (See fig. 12.) One terminal of the battery is attached to a ground connection, and the other to one terminal of the voltmeter. The other terminal of the voltmeter is connected through the frame, reel, and tape, to the weight. The weight is lowered until it contacts the water surface; this contact completes the electric circuit and produces a signal on the voltmeter. With the weight held in the position of first contact, the tape reading is observed at the index provided on the reel mounting. The electric-tape gage is used as an inside reference gage and occasionally as an outside gage. If oil is floating on the water surface, the gage will give the gage height of the interface, because oil is a dielectric. In some electric-tape gages a light or audible signal is substituted for the voltmeter.

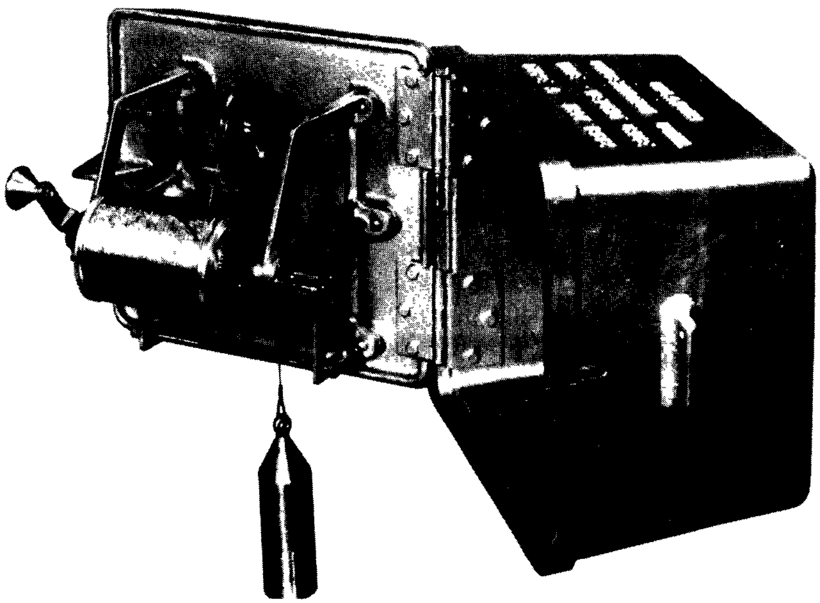

FIGURE 10.—Type A wire-weight gage.

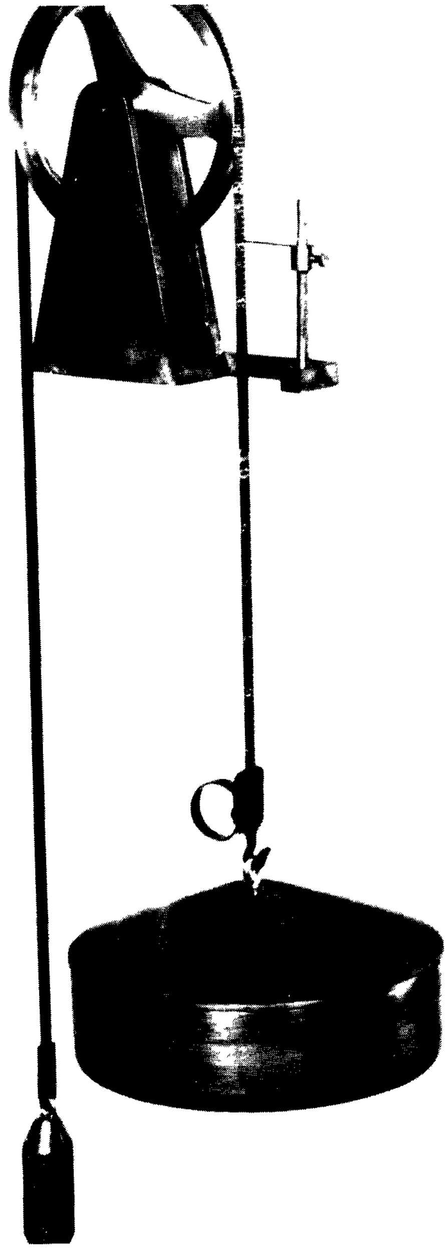

FIGURE 11.—Float-tape gage.

FIGURE 12.—Electric-tape gage.

CHAIN GAGE

A chain gage (fig. 13) is used where outside staff gages are difficult to maintain and where a bridge, dock, or other structure over the water is not available for the installation of a wire-weight gage. The chain gage can be mounted on a cantilevered arm which extends out over the stream, or which is made in such a way that it can be tilted to extend over the stream.

The chain gage consists of the cantilevered arm, one or more enamelled gage sections mounted horizontally on the cantilever, and a heavy sash chain that runs over a pulley on the streamward end of the cantilever. (See figure 13.) The chain is mounted so that it moves along the gage sections. A weight is attached to the streamward end of the chain, and an index marker (*M* in fig. 13) is attached near the other end at a distance from the weight that is appropriate for reading the gage height of low flows. Additional index markers can be attached to the chain at appropriate intervals to read gage heights greater than those directly obtainable from the mounted gage sections.

Stage is determined by lowering the weight until the bottom of the

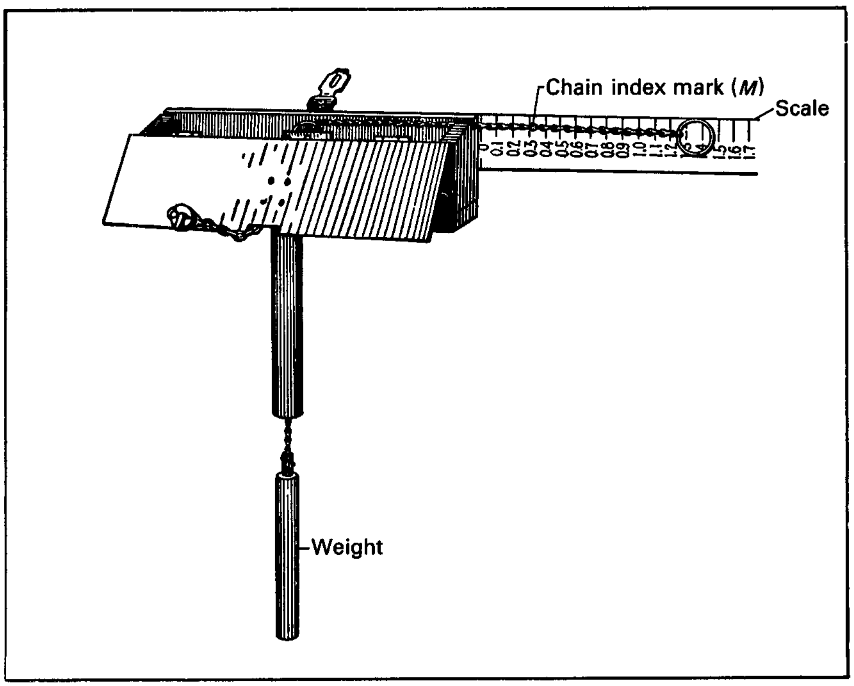

FIGURE 13.—Chain gage (cantilevered mount not shown).

weight just touches the water surface. The gage height then is read from the mounted gage plate at the location of the appropriate chain index marker.

RECORDING STREAM-GAGING STATIONS

METHODS OF SENSING STAGE FOR AUTOMATIC RECORDING

Stage is sensed for automatic recording by a float in a stilling well, or by a gas-purge system that transmits the pressure head of water in a stream to a manometer. The latter system, which does not require a stilling well, is known as a bubble gage.

FLOAT SENSOR

The float sensor consists of a tape or cable passing over a pulley, with a float in a stilling well attached to one end of the tape or cable and a counterweight attached to the other end. (See fig. 11.) The float follows the rise and fall of the water level, and the water level can be read by using an index and graduated tape, or the pulley can be attached to a water-stage recorder to transmit the water level to the recorder.

BUBBLE-GAGE SENSOR

The bubble-gage sensor (Barron, 1963) consists of a gas-purge system, a servomanometer assembly, and a servocontrol unit. (See fig. 14.) The gas-purge system transmits the pressure head of water in the stream to the manometer location. A gas, usually nitrogen, is fed through a tube and bubbled freely into the stream through an orifice at a fixed elevation in the stream. The gas pressure in the tube is equal to the piezometric head on the bubble orifice at any gage height.

The servomanometer converts the pressure in the gas-purge system to a shaft rotation for driving a water-stage recorder. Mercury is used as the manometer liquid to keep the overall manometer length to a minimum. The manometer has a sensitivity of 0.005 ft (0.0015 m) of water and can be built to record ranges in gage height in excess of 120 ft (36 m). The use of mercury in the manometer permits positioning of the pressure reservoir to maintain the float-switch contacts in null position. In this position, the vertical distance between mercury surfaces will be 1/13.6 times the head of water. A change in pressure at the reservoir displaces the mercury which in turn activates the float switch. This causes movement of the pressure reservoir until the distance of head of water divided by 13.6 is again maintained. This motion, in turn, is translated to the recorder.

The servocontrol unit provides the relay action necessary to permit the sensitive float switch to control the operation of the servomotor;

the unit also provides an appropriate time delay between the closing of the float switch and the starting of the motor.

Several bubble-gage sensors, differing in minor detail from that described above, are commercially available. Also commercially available is another type of bubble-gage sensor in which the nitrogen bubble tubing transmits the river-stage pressure to a bellows; the bellows, through a mechanical linkage, actuates a recording pen.

The proper placement of the orifice is essential for an accurate stage record. The orifice should be located where the weight of water above it represents the stage in the river. If the orifice is partly buried in sand or mud, the recorded stage will be greater than that in the river. An orifice preferably should not be installed in swift currents. If this is unavoidable, the orifice must be kept at right angles to the direction of flow. A recommended mounting for high-velocity flow is one in which the orifice is installed flush with the wall of the mounting structure. Care should also be taken to keep the orifice out of highly turbulent flow.

In unstable streambeds it is sometimes advantageous to place the bubble orifice in a vented well point driven into the unstable bottom.

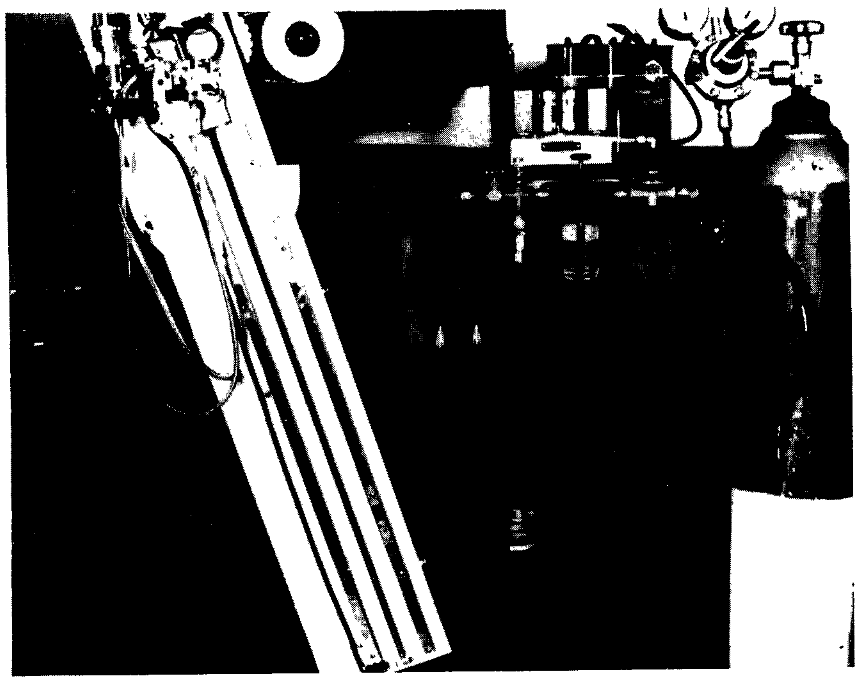

FIGURE 14.—Major units of the bubble gage.

If oil (generally kerosene) is to be added to prevent freezing in the vent pipe, the top of the well screen should be a sufficient distance below the minimum expected stream stage to retain the required depth of oil (fig. 15). To prevent variations in the depth of oil from affecting the manometer reading, the bubble orifice should be below the top of the screen so that the bubbles emerge into the water.

It is emphasized that for satisfactory operation of the well point the streambed material should not be so finely grained as to unduly impede the passage of river water to the well point and thereby cause lag in the recorded stage. To prevent clogging of the well-point screen, the screen should be made of material that will inhibit chemical reaction with substances in the water and (or) in the bed material. A stainless-steel screen set 1 to 3 ft (0.3 to 1.0 m) below the streambed is recommended.

The bubble gage is used primarily at sites where it would be expensive to install a stilling well. It is also used on sand-channel streams because the gas tends to keep the orifice from being covered with sand and the tube may be easily extended to follow a stream channel that shifts its location. However, the float stilling-well installation is cheaper to install at many sites, and its performance is usually more reliable than that of the bubble gage. The two systems have about the same accuracy—±0.01 ft (0.003 m). The choice of systems thus depends on the characteristics of the gage site.

WATER-STAGE RECORDERS

A water-stage recorder is an instrument for producing a graphic or punched-tape record of the rise and fall of a water surface with respect to time. It consists of a time element and a gage-height element which, when operating together, produce on a chart or on a tape a record of the fluctuations of the water surface. The time element is controlled by a clock that is driven by a spring, by a weight, or by electricity. The gage-height element is actuated by a float or a bubble gage.

If a float sensor is used, the float pulley is attached to the recorder. The float and counterweight are suspended on a perforated steel tape or on a plain or beaded cable. Cone-shaped protrusions on the circumference of the float-tape pulley match performations in the tape. As the float rises or falls the float pulley rotates in proportion to the change in stage; the rotation of the pulley is transmitted to the recorder and the appropriate gage height is thereby recorded. A copper float 10 in (0.25 m) in diameter is normally used, but other sizes are also used depending on the type of recorder, gage-height scale, and accuracy requirements.

If a bubble-gage sensor is used, the stage is translated to the recorder by a chain and sprocket arrangement. (See fig. 14.)

Stage recorders are either digital or graphic. Both types may be

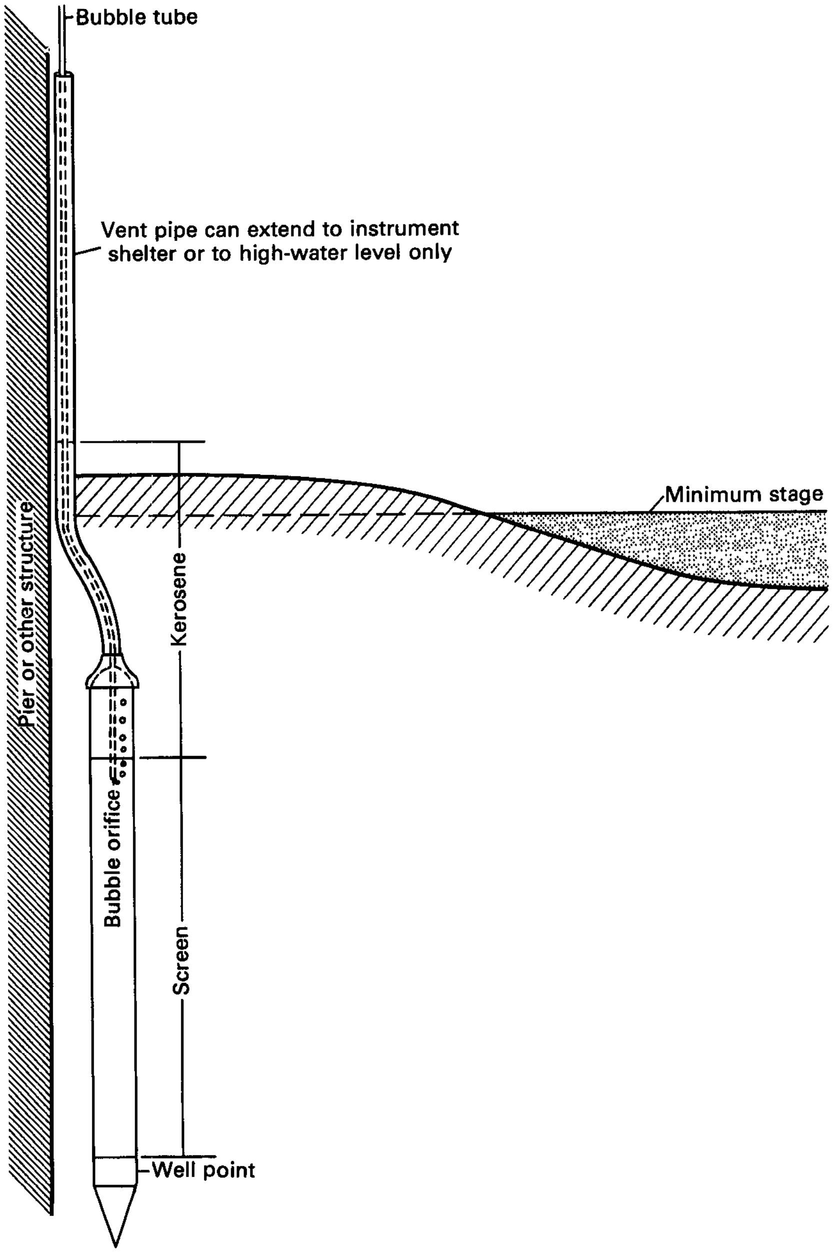

FIGURE 15.—Installation of bubble orifice in unstable streambed.

used with the float or bubble gage. Digital recorders are gradually replacing strip-chart (graphic) recorders at gaging stations in the United States. The two recorders are about equal in accuracy, reliability, and cost, but the digital recorder is compatible with the use of electronic computers in computing discharge records. This automated system offers greater economy and flexibility in the computation-publication process than do manual methods associated with graphic recording. However, the use of graphic recorders should be continued at those sites where a graphic record is necessary to detect ice effects, backwater, or frequent malfunctions of the recording system.

DIGITAL RECORDER

The digital recorder used by the U.S. Geological Survey (Isherwood, 1963) is a battery-operated slow-speed paper-tape punch which re-

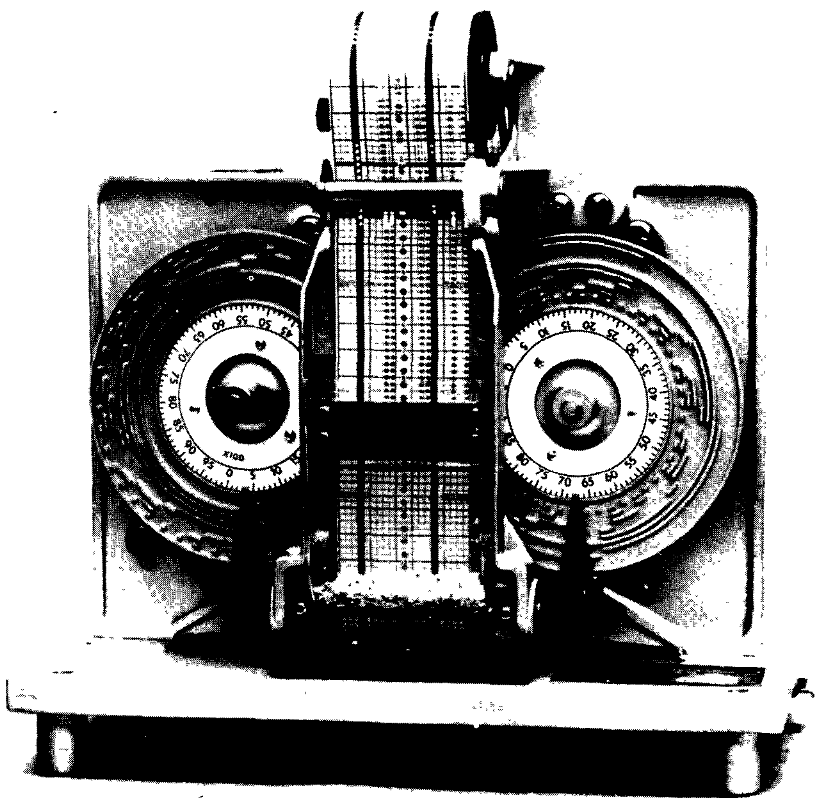

FIGURE 16.—Digital recorder.

cords a 4-digit number on a 16-channel paper tape at preselected time intervals. (See fig. 16.)

Stage is recorded by the instrument in increments of a hundredth of a foot from zero to 99.99 ft (30.5 m) and is transmitted to the instrument by rotation of the input shaft. Shaft rotation is converted by the instrument into a coded punch-tape record that is simple enough to be read directly from the tape. The code consists of four groups of four punches each. In each group, the first punch represents "1," the second "2," the third "4," and the fourth "8." Thus a combination of up to three punches in a group represents digits from 1 to 9, with a blank space for 0, and the four groups of punches represent all numbers from 1 to 9,999. (See fig. 17.)

Coding is done by means of two discs containing raised ridges in accordance with the punch code outlined above. One disc is mounted directly on the input shaft. The second code disc is connected to the first by a 100:1 worm gear so that one hundred revolutions of the input shaft rotate the second, or high-order disc, one complete revolution. A paper tape is moved upward through a punch block which is mounted on a movable arm hinged at the base of the recorder. The punch block contains a single row of 18 pins, 16 pins for the information punches and 2 for punching feed holes.

The tape is punched when the punch block with its protruding pins is forced against the code discs by spring action. Those pins, which strike the raised ridges of the discs, punch through the paper tape and record the position of the discs at that instant. The readout cycle begins with an impulse from the timer that causes a 6-volt motor to turn a sequencing camshaft. The sequence of operations for one reading includes punching the paper, advancing the paper, and compressing the punch spring for the next readout cycle.

The timers (fig. 18) used on the digital recorders are electromechanical timing devices that are powered by the same 7½-volt battery that operates the 6-volt motor. The timers provide contact closure for actuating the digital recorder at preselected time intervals of 5, 15, 30, or 60 minutes by using a different cam for each different time interval.

The cam on the timer corresponds to the minute hand on a clock; that is, it makes one revolution per hour in a clockwise direction. If the cam has one dropoff point, the recorder will punch hourly; if it has two dropoff points, it will punch every 30 minutes; and if it has four dropoff points, it will punch every 15 minutes. The timer in figure 18 has four dropoff points. The arm positioned by the cam operates a single-pole double-throw switch. When the cam dropoff point passes the arm, the switch initiates the major part of the readout cycle which includes punching of the tape. A preset action returns the switch to the initial position prior to the next readout cycle. Alternating-current timers can be used with the digital recorders at places where

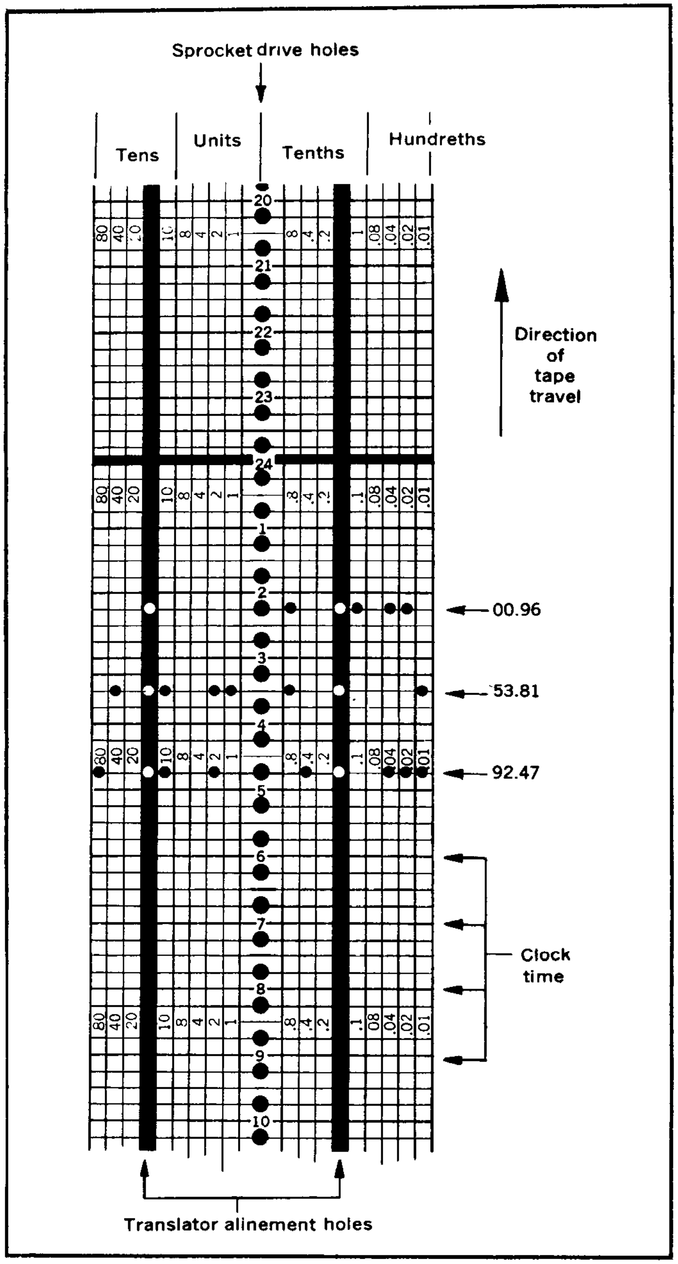

FIGURE 17.—Sample digital-recorder tape.

reliable alternating-current power is available.

Digital recorders may miss the absolute peak stage especially on flashy streams. However, a measure of the maximum peak that occurs between inspections of the recorder can be obtained by attaching a wire clip (similar to a paper clip) or small magnet to the float tape just below the instrument shelf in such a manner that it will slide along the tape as the stage rises but remain in a fixed position as the stage declines. (See p. 60–61).

Mechanically punched tape is the most practical for field use under widely varying conditions of temperature and moisture. Electronic translators are used to convert the 16-channel punch-tape records to a tape suitable for input into a digital computer for computation of a daily mean gage height and daily mean discharge.

In the metric version of the digital recorder, stage is recorded in increments of 1 millimeter from 0 to 9.999 m.

GRAPHIC RECORDER

The graphic, or analog, recorder furnishes a continuous trace of water stage, with respect to time, on a chart. Usually the gage-height element moves the pen or pencil stylus and the time element moves the chart, but in some recorders those actions are reversed. In the U.S.A. the common range of available gage-height scales is from 10 in=1 ft (10:12) to 10 in=20 ft (1:24). The width of strip charts is usually 10 in (0.25 m). The range of available time scales is from 0.3 to 9.6 in (0.0076 m to 0.244 m) per day. Normally the gage-height

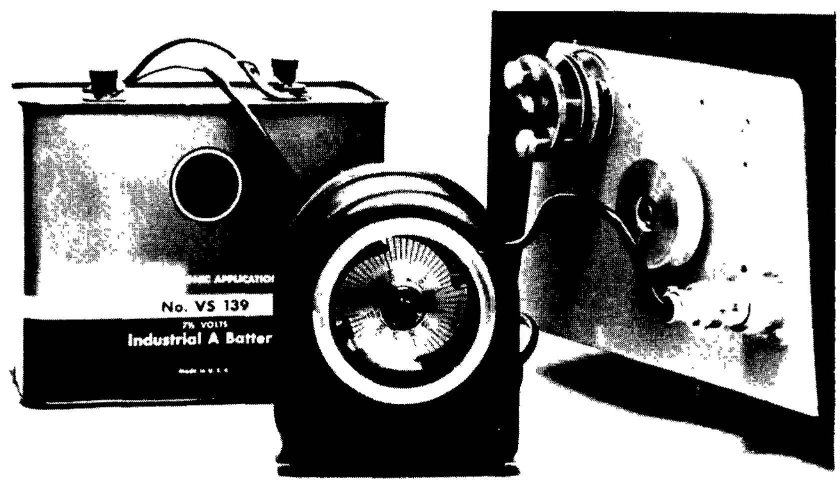

FIGURE 18.—Digital-recorder timer.

scale of 10 in=5 ft (1:6) or 10 in=10 ft (1:12) is used with a time scale of 1.2, 2.4, or 4.8 in per day.

Most graphic recorders can record an unlimited range in stage by a stylus-reversing device or by unlimited rotation of the drum.

Most strip-chart recorders will operate for several months without servicing. Drum recorders require attention at weekly intervals. Figure 19 shows a commonly used continuous strip-chart graphic recorder, and figure 20 a horizontal-drum recorder that must be serviced at weekly intervals. Attachments are available for the recorder

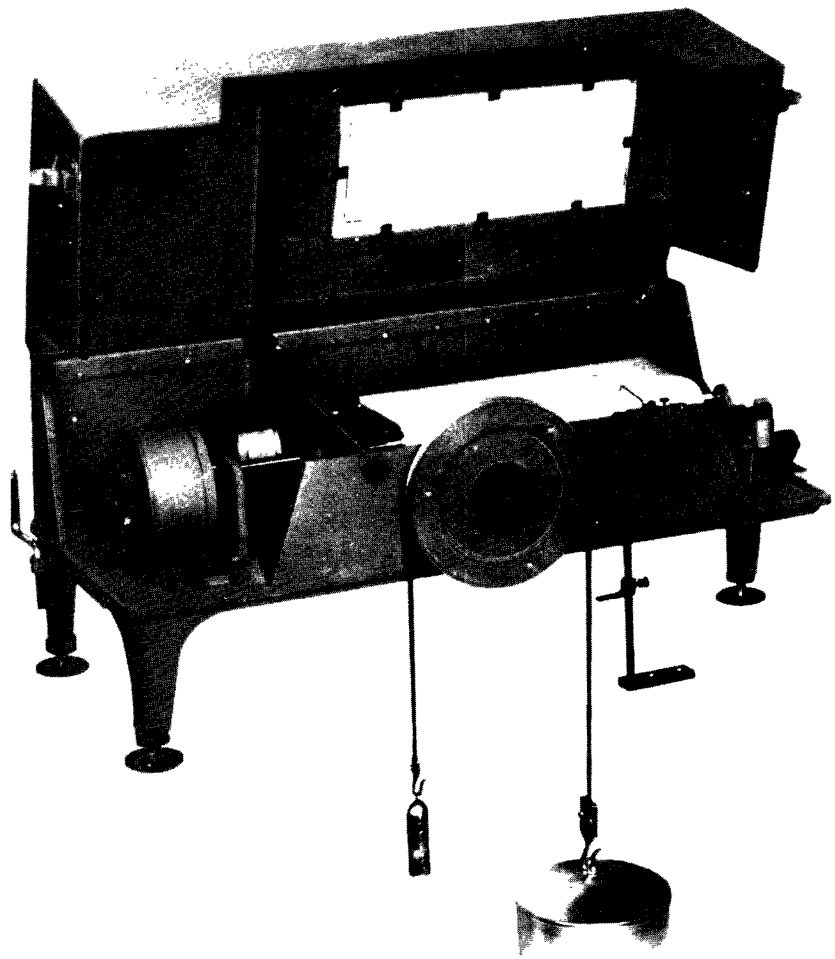

FIGURE 19.—Continuous strip-chart recorder.

shown in figure 19 to record water temperature or rainfall on the same chart with stage. Figure 21 is a section of a typical strip chart whose gage-height scale is 5:12 and whose time scale is 4.8 in per day.

STILLING WELLS

The stilling well protects the float and dampens the water-surface fluctuations in the stream caused by wind and turbulence. Stilling wells are made of concrete, reinforced concrete block, concrete pipe, steel pipe, and occasionally wood. They are usually placed in the bank of the stream (figs. 22–26), but often are placed directly in the stream and attached to bridge piers or abutments. (See figs. 27 and 28.) The

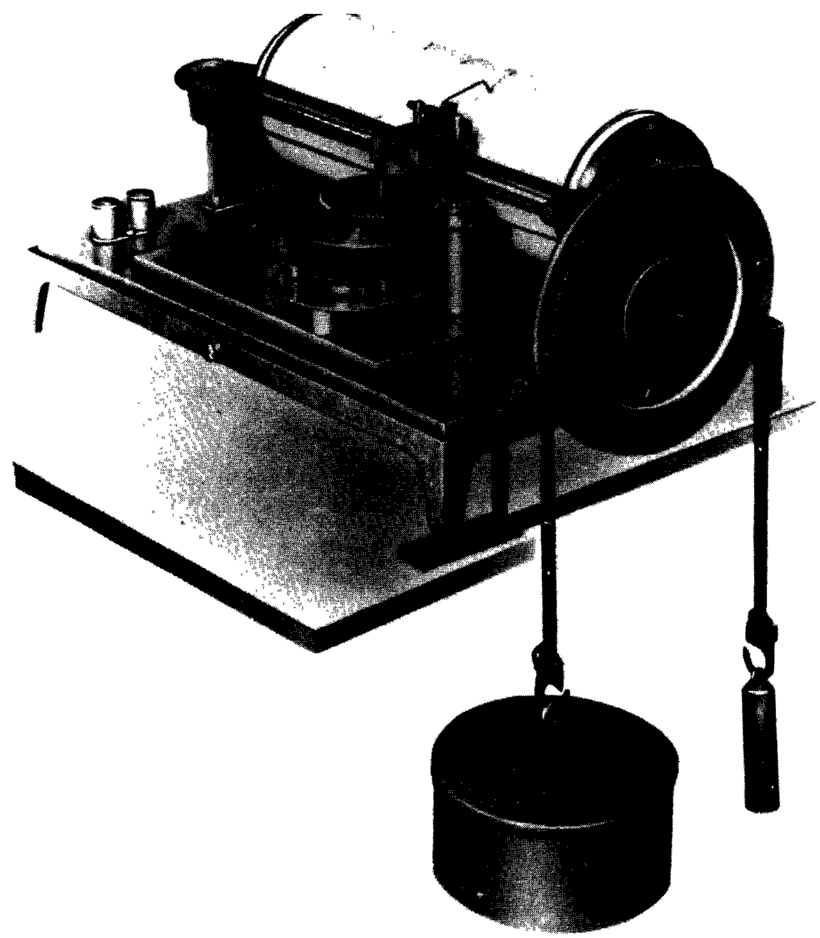

FIGURE 20.—Horizontal-drum recorder.

stilling well should be long enough for its bottom to be at least 1 ft (0.3 m) below the minimum stage anticipated, and its top preferably should be high enough so that the recording instrument will be above the level of the 100-year flood.

The inside of the well should be large enough to permit free operation of all the equipment to be installed. Usually a pipe 4 ft (1.2 m) in diameter or a well with inside dimensions 4 by 4 ft (1.2 by 1.2 m) is of satisfactory size, but pipes 1.5 ft (0.5 m) in diameter have been used for temporary installations where a conventional water-stage recorder was the only equipment to be installed. The 4 by 4 ft well provides ample space for the hydrographer to enter the well to clean it or to repair equipment. The smaller metal wells and the deep wells

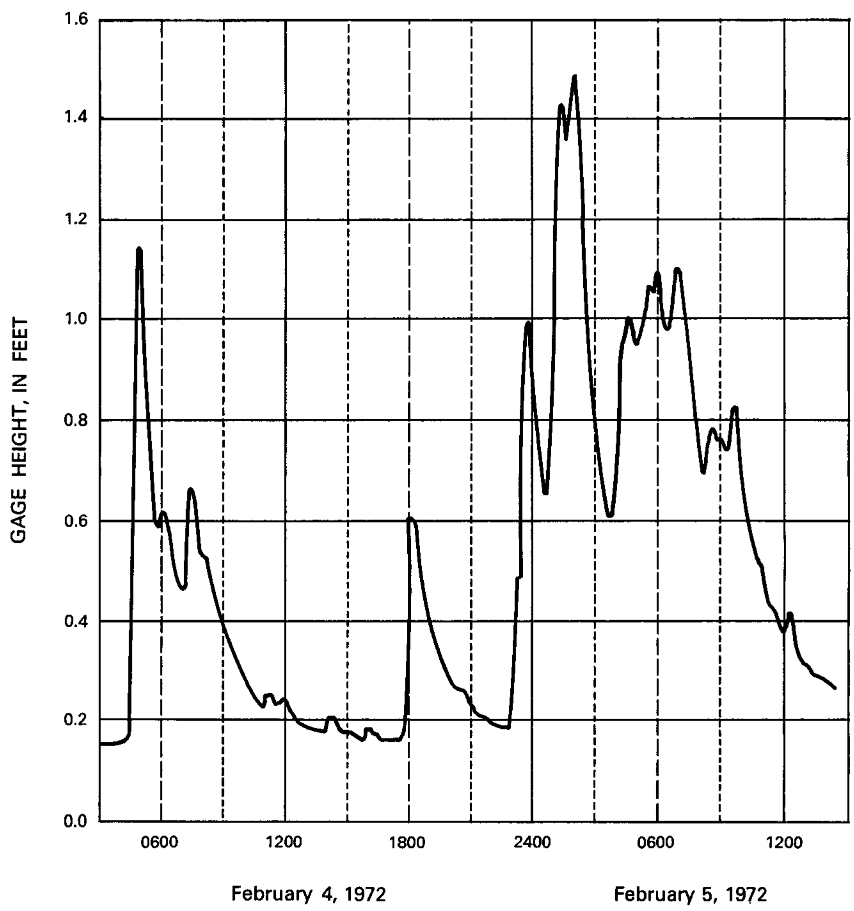

FIGURE 21.—Section of a typical strip chart.

should have doors at various elevations to facilitate cleaning and repairing. (See figs. 22 and 25.)

When placed in the bank of the stream the stilling well should have a sealed bottom so that ground water cannot seep into it nor stream water leak out.

Water from the stream enters and leaves the stilling well through the intake so that the water in the well is at the same elevation as the water in the stream. If the stilling well is in the bank of the stream, the intake consists of a length of pipe connecting the stilling well and the stream. The intake should be at an elevation at least 0.5 ft (0.15 m) lower than the lowest expected stage in the stream, and at least 0.5 ft above the bottom of the stilling well to prevent silt buildup from plugging the intake. In cold climates the intake should be below the

FIGURE 22.—Reinforced concrete well and shelter. (Note clean-out door.)

frostline. If the well is placed in the stream, holes drilled in the stilling well may act as an intake, taking the place of a length of pipe. Some wells placed in the stream have a cone-shaped hopper bottom that serves as an intake and is self cleaning.

Two or more pipe intakes are commonly installed at vertical intervals of about 1 ft (0.3 m). During high water, silt may cover the streamward end of the lower intakes while the higher ones will continue to operate. The intakes should be properly located and sized to minimize surge.

Most stations that have intakes subject to clogging are provided with flushing systems (see fig. 29) whereby water under several feet of head can be applied to the gage-well end of an intake. Ordinarily a pump raises water from the well to an elevated tank. The water is then released through the intake by the operation of a valve. Intakes without flushing systems may be cleaned with a plumber's snake or rod, or by building up a head of water in the well with a portable pump to force an obstruction out of the intakes.

The intakes for stations placed in the bank of the stream are usually galvanized-steel pipe. The most common size used is 2-in (0.05 m)-diameter pipe, but for some wells pipe diameters as large as 4 in (0.1 m) are used. After the size and location of the well have been decided upon, the size and number of intakes should be determined.

FIGURE 23.—Concrete well and wooden shelter with asphalt-shingle siding.

The intake pipe should be large enough for the water in the well to follow the rise and fall of stage without significant delay. The follow-

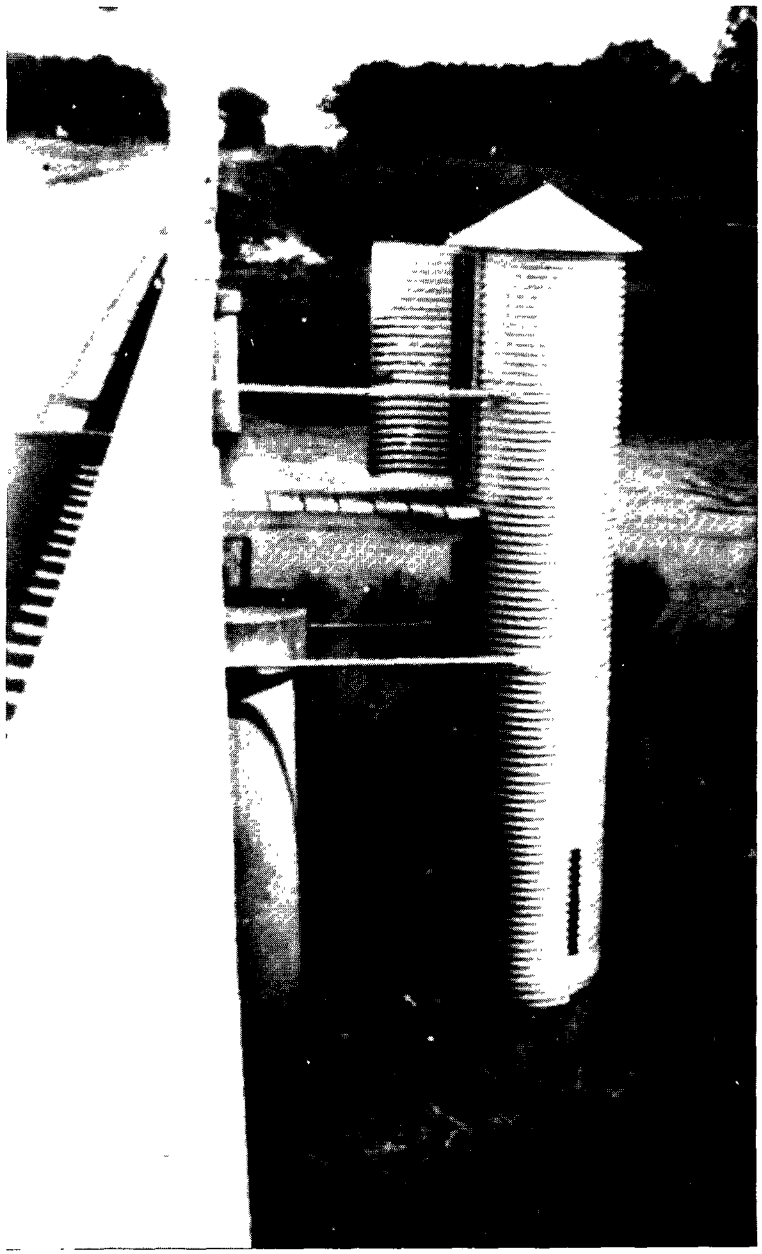

FIGURE 24.—Corrugated-galvanized-steel-pipe well and shelter.

ing relation may be used to predict the lag for an intake pipe for a given rate of change of stage:

$$\Delta h = \frac{0.01}{g} \frac{L}{D} \left(\frac{A_w}{A_p}\right)^2 \left(\frac{dh}{dt}\right)^2$$

in which

Δh = lag, in feet (or meters),

g = acceleration of gravity, in feet (or meters) per second per second,

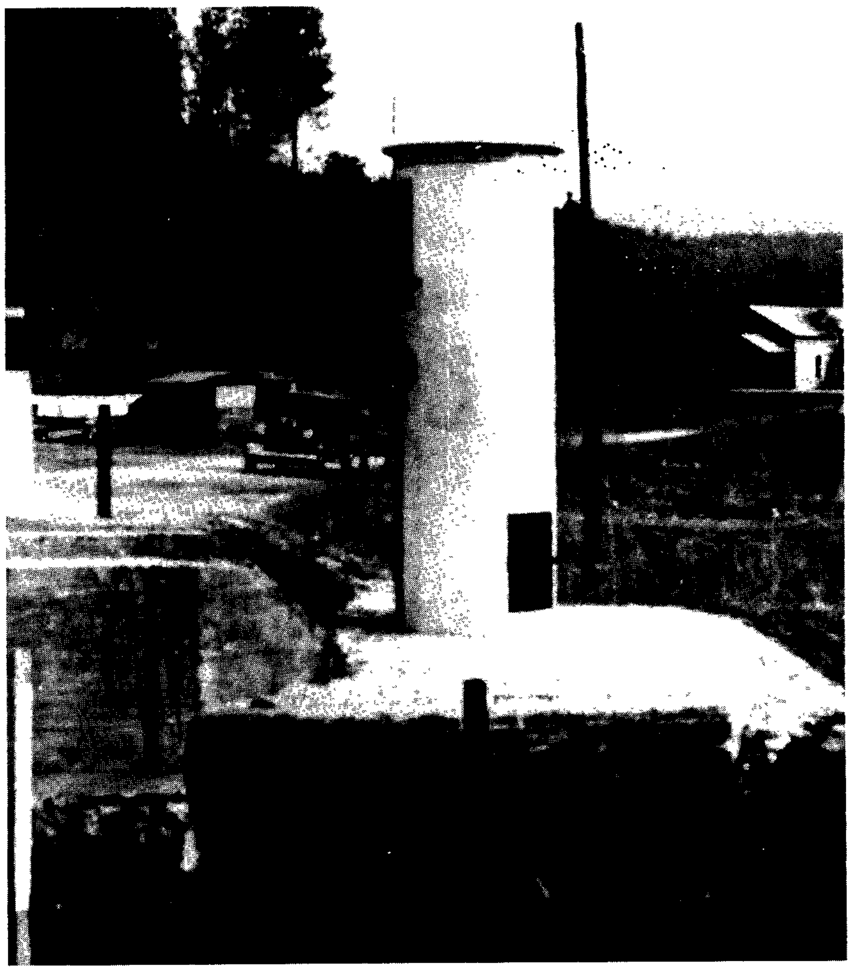

FIGURE 25.—Concrete-pipe well and shelter. (Note clean-out door, staff gage, and upper intake pipe.)

L = intake length in feet (or meters),
D = intake diameter, in feet (or meters),
A_w = area of stilling well, in square feet (or square meters),
A_p = area of intake pipe, in square feet (or square meters), and
$\frac{dh}{dt}$ = rate of change of stage, in feet (or meters) per second.

Smith, Hanson, and Cruff (1965) have studied intake lag in stilling-well systems, relating it to the rate of change of stage of the stream and to the various types and sizes of components used in the stilling-well intake system.

The intake pipe should be placed at right angles to the direction of flow, and it should be level. If the stream velocity at the end of the intake is high, either drawdown or pileup may affect the water level in the stilling well, depending on the angle of the flow at the intake opening. Drawdown causes the water level in the stilling well to be lower than that in the stream; pileup has the opposite effect. Drawdown commonly occurs at high velocities even when the intake pipe is at an angle of 90° with the flow. To reduce or possibly eliminate drawdown or pileup, a static tube should be attached to the streamward end of the intake pipe. A static tube is a short length of pipe attached to an elbow or tee on the end of the intake pipe and extending horizontally downstream. (See fig. 30.) The end of the static tube is capped, and water enters or leaves through holes drilled in the tube.

The usual means of preventing the formation of ice in the well

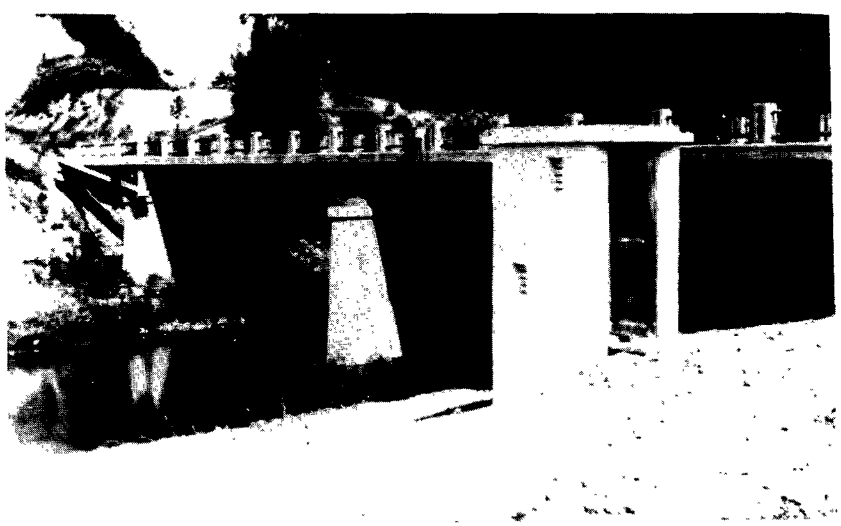

FIGURE 26.—Concrete-block shelter.

during cold weather are: (1) subfloors, (2) heaters, and (3) oil. Subfloors are effective if the station is placed in the bank and has plenty of fill around it. If the subfloor is built in the well below the frostline in the ground, ice will seldom form in the well as long as the stage remains below the subfloor. Holes are cut in the subfloor for the recorder float and weights to pass through, and removable covers are placed over the holes. Subfloors prevent air circulation in the well and the attendant heat loss.

An electric heater or heat lamps with reflectors may be used to keep the well free of ice. The cost of operation and the availability of electric service at the gaging station are governing factors. Heating cables are often placed in intake pipes to prevent ice from forming.

Oil is used in two ways: (1) Where the well is small and leakproof, the oil may be poured into the well; and (2) where the well is large or not leakproof, the oil—usually kerosene, fuel oil, or diesel oil—is poured into an oil tube. The oil tube, which is open ended and of sufficiently large diameter to accommodate the recorder float, is supported in the well in a vertical position with its lower end just above

FIGURE 27.—Steel-pipe well and look-in shelter attached to bridge abutment.

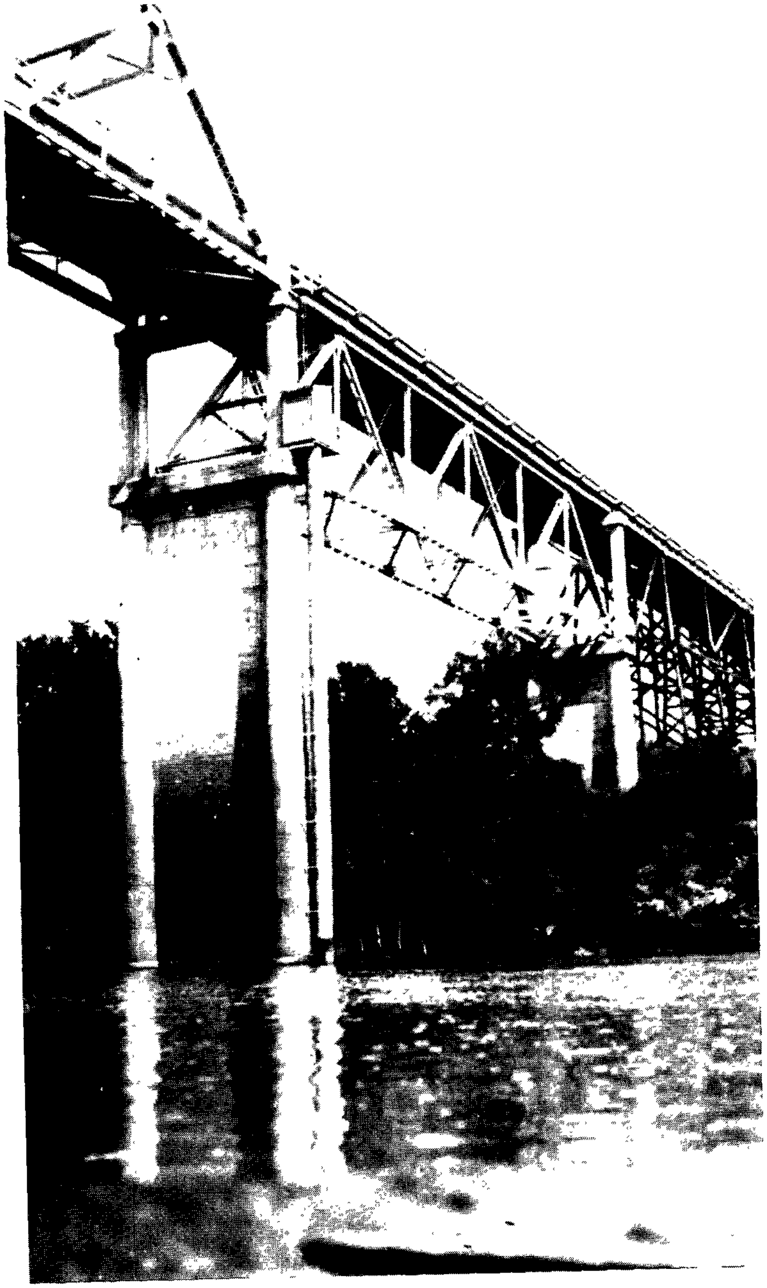

FIGURE 28.—Corrugated-steel-pipe well and wooden shelter attached to bridge pier.

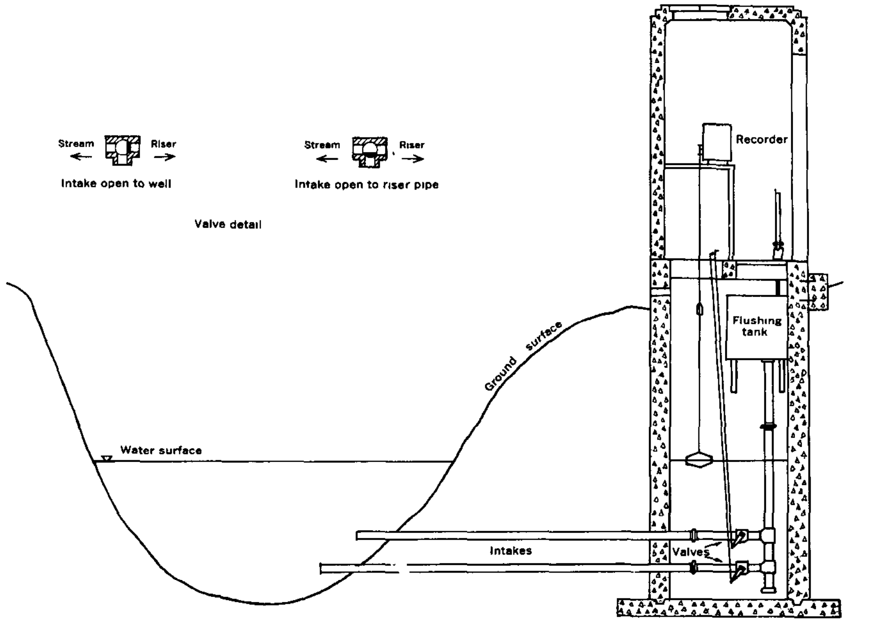

FIGURE 29.—Flushing system for intakes.

FIGURE 30.—Static tube for intakes. (Note outside reference gage.)

the floor of the well. The oil tube should be long enough to contain the oil throughout the range in stage expected during the winter. When oil is put in a well, the oil surface stands higher than the water surface in the stream. A correction must therefore be made to obtain the true river stage. The depth of oil required usually ranges from 0.5 to 2.0 ft (0.15 to 0.6 m) depending on the severity of the climate and the exposure of the well.

Stilling wells often fill with sediment, expecially those located in arid or semiarid regions. If a well is located on a stream carrying heavy sediment loads, it must be cleaned often to maintain a continuous record of stage. In those locations sediment traps are helpful in reducing the frequency and labor of sediment removal. A sediment trap is a large boxlike structure that occupies a gap in the lower intake line, streamward from the stilling well. The bottom of the sediment trap is usually about 3 ft (1.0 m) below the elevation of the intake. Inside the trap are one or more baffles to cause suspended sediment to settle in the trap, rather than pass into the well. A removable top to the trap provides access to the interior of the trap for the removal of trapped sediment.

The operation of type of recording stream-gaging station requires the installation of one or more nonrecording gages for use as auxiliary or reference gages. Reference gages are discussed on pages 53–54.

INSTRUMENT SHELTERS

Shelters are made of almost every building material available and in various sizes depending on local custom and conditions. (See figs. 22–28.) The most convenient type of shelter is one that the hydrographer can enter standing. A shelter with inside dimensions 4 by 4 ft (1.2 by 1.2 m) and with ceiling height 7 ft (2.2 m) above the floor is usually of adequate size, unless the shelter is also to be used to store sediment-sampling equipment and (or) house telemetry equipment (p. 54–59). Look-in shelters (fig. 27) are also used at sites where a limited amount of equipment is to be installed and a portable and inexpensive shelter is desired.

In humid climates, shelters are well ventilated and have a tight floor to prevent entry of water vapor from the well. Screening and other barriers are used over ventilators and other open places in the well or shelter to prevent the entry of insects, rodents, and reptiles.

The bubble gage does not require a stilling well. The instrument shelter for a bubble gage may be installed at any convenient location above the reach of floodwaters. This gage may be used to take advantage of existing natural or artificial features in a stream without costly excavation for well or intake and without need for any external structural support. The bubble gage is especially well suited for short-term installations because the entire station is readily dismantled and relocated with practically no loss of investment.

A shelter with inside dimensions 4 by 4 by 7 ft (1.2 by 1.2 by 2.2m) is needed to accommodate the equipment for a bubble gage. Shelters similar to those in figures 22, 23, and 26 would be adequate. The shelter can be placed on a concrete slab or other suitable foundation. The bubble orifice is placed at least 0.5 ft (0.15 m) below the lowest expected stage in the stream. The plastic tube connecting the orifice and the instrument is encased in metal pipe or conduit, or buried to protect it from the elements, animals, and vandalism. A typical bubble-gage installation is shown in figure 31. The streamward end of the metal pipe should be flush with the streamward face of the pier shown in figure 31 to prevent disturbance of the streamlines of river flow in the vicinity of the orifice. The streamward end of the orifice line should also have a downward slope, as shown in the figure. It has been found that if the end of the orifice line is installed in a horizontal position, water tends to run up the bubble tube after each bubble is formed, and the induced surge is recorded by the stage recorder.

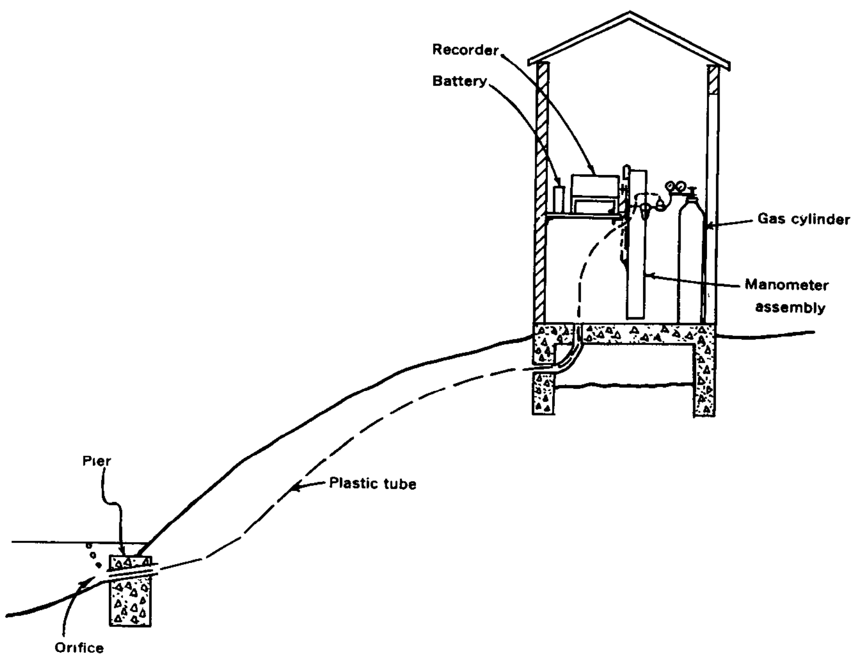

FIGURE 31.—Typical bubble-gage installation.

REFERENCE AND AUXILIARY GAGES AT RECORDING GAGING STATIONS

Nonrecording gages, which were discussed on pages 24–32, are used both as reference and auxiliary gages at recording stream-gaging stations. As used in this manual a reference gage is the gage to which the recording instrument is set; it is the base gage for the recording station. All other nonrecording gages at the recording station are considered to be auxiliary gages. A detailed discussion follows.

Outside gage—At bubble-gage stations a nonrecording gage, established in close proximity to the bubble orifice, acts as the base or reference gage for checking and resetting the gage height indicated by the water-stage recorder.

At stations equipped with a stilling well for the operation of a float-operated stage recorder, there is always the possibility that the stage in the stilling well may not be representative of the stage of the stream. For example, intakes can become plugged, floats can spring a leak, or oil can leak out of wells or oil tubes. Consequently, a nonrecording auxiliary gage is installed outside the stilling well so that the water level of the stream can be determined directly for comparison with the stage in the stilling well. It is not necessary that the two observed stages agree precisely; hydraulic conditions at the station may be such that precise agreement is not possible. If a reading of the outside auxiliary gage indicates that an unsatisfactory record is being obtained by the recording gage, the trouble is rectified immediately. If immediate repairs are not feasible, or if there has been an instrument failure, the outside auxiliary gage is used as a temporary substitute for the recording gage. The outside auxiliary gage can be read as needed by a local observer to continue the record of stage until the malfunction of the recording station is rectified.

Staff or wire-weight gages are usually used as outside auxiliary gages. Outside staff gages located in the pools near the gage structures are visible in figures 5, 6, 22, 25, and 30; a wire-weight auxiliary gage on the parapet wall of the bridge is visible in figure 1, and another is seen in figure 27.

Inside gage—At gaging stations equipped with a stilling well a nonrecording gage inside the structure is used to indicate the water-surface elevation in the stilling well. Readings on this inside gage are compared with readings on the outside auxiliary gage to assure that the stilling-well intakes are functioning properly. If the intakes are functioning properly, the inside gage is used for checking and resetting the gage height indicated by the water-stage recorder. In short, the inside gage is the base or reference gage for the station. Float- or electric-tape gages, or vertical staff gages, are the inside reference gages most commonly used in stilling wells.

On occasion a reference mark or reference point of known elevation, with respect to gage datum, is used in place of either a reference or auxiliary gage. The stage is then determined by measuring from the reference mark or point down to the water surface either in the stream or in the stilling well, as the case may be. While the practice of using a reference mark or point is acceptable, it is not nearly as convenient as the practice of using a standard nonrecording gage. (The distinction between a reference mark and reference point should be noted here: Both have elevations that are known, but a reference point is a point on the gage structure itself; a reference mark is a point that is not on the gage structure, and whose elevation is therefore unaffected by any movement of the gage structure.)

The practice described above of using the inside gage, rather than the outside gage, as the reference or base gage for a recorder equipped with a stilling well is followed in the U.S.A. and in many other countries. The reasoning behind that practice is that recorded (inside) gage heights will be used to determine discharge, and if differences exist between inside and outside stages, those differences will be known only for those times when both gages are read. If the outside gage is used as the base gage, corrections, known or assumed, must be applied to all recorded gage heights to convert them to outside stages. Furthermore, outside gages are often difficult to read with precision because of the action of wind and waves. In other countries the outside gage is used as the reference gage for the reasons that (1) river stage is often as important as discharge to the user of the record and (2) a stage-discharge relation is dependent on river stage, rather than on the stage inside a stilling well. The validity of both those reasons is recognized in the U.S.A. Therefore outside high-water marks are obtained for each flood event, and where the elevation of the marks differs significantly from the peak inside stage, both inside and outside stages are published. Also, where there is significant difference between inside and outside gage readings, the stage-discharge relation is first developed on the basis of outside-gage readings observed at the times when discharge is measured. The relation is then adjusted to correspond with inside-gage readings observed at the times of discharge measurement.

TELEMETERING SYSTEMS

Telemetering systems are used when current information on stream stage is needed at frequent intervals and it is impractical to visit the gaging station each time the current stage is needed. Current stage information is usually necessary for reservoir operation, flood forecasting, prediction of flows, and for current-data reporting. The types of telemetering systems are:

1. Those that continuously indicate or record stage at a distance from the gage site. Examples of this type are the position-motor and impulse telemetering systems.
2. Those that report instantaneous gage readings on call or at predetermined intervals. Examples of this type are the Telemark and resistance telemetering systems, and the experimental satellite data-collection systems.

POSITION-MOTOR SYSTEM

The position-motor system provides remote registering of water levels on graphic recorders or on counter or dial indicators over distances up to 15 mi (25 km). This system employs a pair of self-synchronizing motors—one on the transmitter, whose rotor is actuated by a float-tape gage or a bubble gage, and the other on the receiving unit, whose rotor follows the rotary motion of the transmitting motor to which it is electrically connected. Alternating current is used to operate the system, and a five-wire transmission line is required—two excitation wires and three line wires.

IMPULSE SYSTEM

The impulse system provides remote registering of water levels on graphic recorders or on counter and dial indicators over longer distances than does the position-motor system. This system will operate over leased telephone lines or other metallic circuits. The impulse sender at the gaging station is actuated by a float-tape gage or a bubble gage and sends electrical impulses over the line connecting it to the receiver. This system usually has a battery for the power source at the sender and alternating current at the receiver, though direct current or alternating current may be used at both ends. The advantage of this system over the position-motor system is that it will operate over long distances.

TELEMARK SYSTEM

The Telemark system codes instantaneous stage and signals this information either audibly over telephone circuits or by coded pulses for transmission by radio. The distance of transmission is unlimited because signals can be sent over long-distance telephone circuits or by radio. Telemark response to a telephone ring is automatic. When used in radio transmission, the signals are started by a timing device set for a predetermined broadcast schedule, or the Telemark may be interrogated by radio channel to start the signal.

The Telemark consists of (1) the positioning element which is actuated by a float-tape gage or a bubble gage (fig. 32) and (2) the signaling element which, when signaled, drives a contact across the signaling drums that are positioned in correspondence with the stage. The Telemark may be operated by either alternating current or by batteries.

A Telemark that operates directly off a digital recorder is available and will probably be increasingly used. This Telemark does not need its own stage sensor; it uses that of the digital recorder. A memory system is used so that when the Telemark is signaled, the last gage height recorded on the digital recorder is transmitted.

Telemarks for radio reporting are equipped with an auxiliary switch and coding bar for transmitting identifying radio station call letters and numbers in international Morse code, in addition to transmitting the stage.

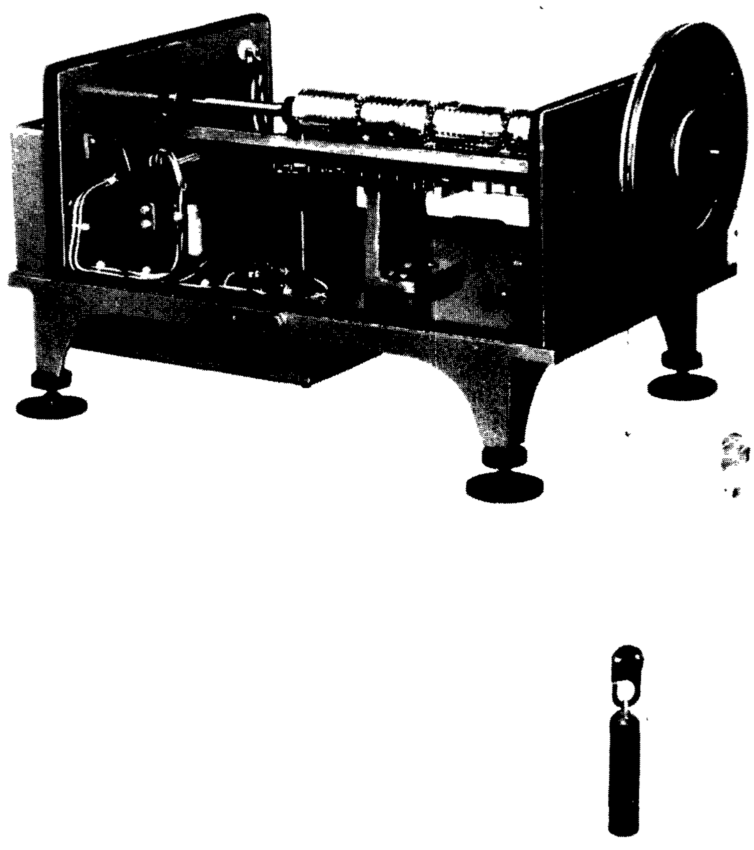

FIGURE 32.—Telemark gage.

RESISTANCE SYSTEM

The resistance system, as used in the U.S.A., was developed by the U.S. Weather Bureau. It provides remote indications of water level for distances up to about 40 mi (about 65 km). Two models are available, one for distances of about a mile (1.6 km) and the other for longer distances. The system consists of two potentiometers in a wheatstone-bridge circuit with a microammeter null indicator. One of the potentiometers is located in the gage house and is actuated by a float and pulley assembly. (See fig. 33.) The other potentiometer and the null indicator are housed at the observation site. (See fig. 34.) By adjusting this potentiometer for a null balance on the meter, the gage height can be read directly to tenths of a foot from a dial coupled to the potentiometer shaft. This system operates on batteries, and three wires connect the unit.

SATELLITE DATA-COLLECTION SYSTEM

The U.S. Geological Survey has been experimenting for several years with the collection of stream-stage data by use of an orbiting satellite which receives radio transmissions of stage and transmits

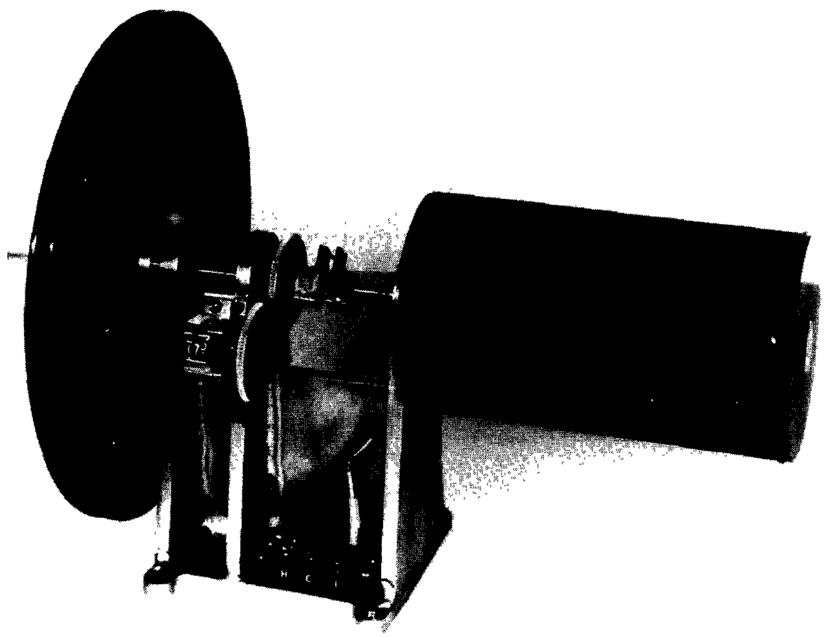

FIGURE 33.—Resistance system transmitter unit.

the data to central receiving stations. An operational system of that kind would make stage data continuously available for use by the managers of water projects who require current ("real time") information for project operation. Because the system is still in the experimental stage it will be described only briefly.

Experimental work on the system began soon after the launch of the Earth Resources Technology Satellite (ERTS) in July 1972. At numerous stream-gaging stations that are dispersed over a wide geographic area, inexpensive battery-operated radios, called data-collection platforms (DCP), are used to transmit stage data through ERTS to the National Aeronautics and Space Administration (NASA) receiving stations at Goldstone, California, and at Greenbelt, Maryland. The data are processed and distributed to experimenters from the ERTS Operations and Control Center in Greenbelt.

Data can be provided to a DCP either directly from a digital water-stage recorder or through an intermediate memory device. In the

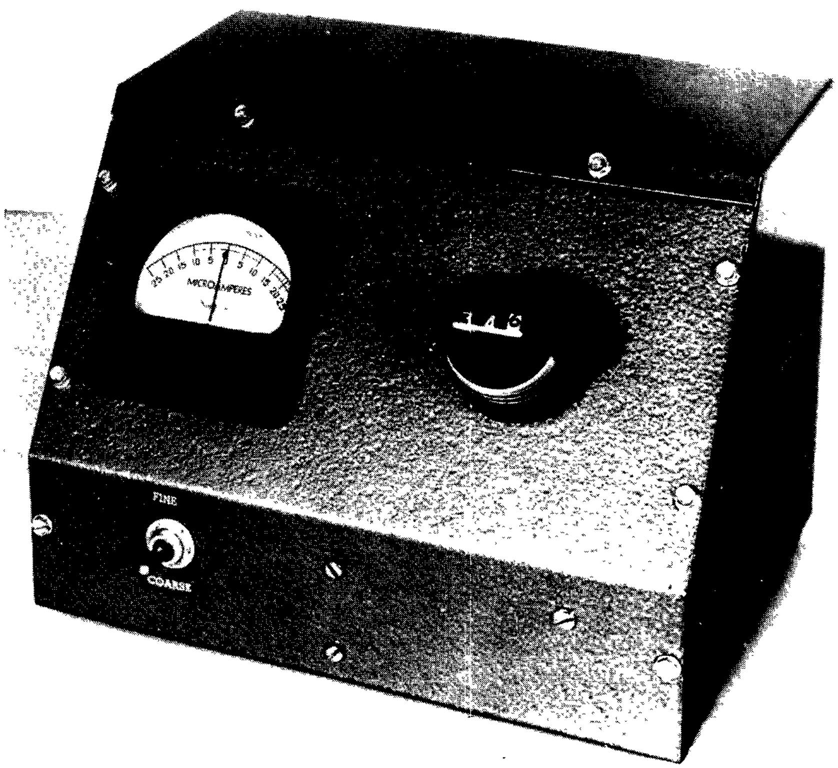

FIGURE 34.—Resistance system indicator unit.

direct communication the most recently recorded digital stream stage is continuously available to the DCP for inclusion in a 0.038-second radio message that is transmitted every 3 minutes. Several times daily the ERTS passes within 1,800 mi of the DCP, and the data are at that time relayed to the receiving stations. Where the intermediate memory device is used, the data are formatted efficiently in the device for inclusion in the periodic DCP message. By use of the memory device virtually all 24 of the hourly stream stages collected daily at North American stream-gaging stations can be accumulated and relayed, as opposed to the 5 or 6 hourly stages that are relayed daily when the DCP is directly connected to the digital-stage recorder. Tests of the satellite data-collection system have been successful to date (1980) and indicate that the satellite relay of environmental data from widely dispersed and remotely located gaging stations can be effectively and reliably performed.

OPERATION OF A RECORDING STREAM-GAGING STATION

Strip-chart or digital recorders are designed to give a continuous record of stage, but careful attention to details is necessary at each visit to the station to ensure a reliable and uninterrupted record during the period (commonly 4–6 weeks) that usually elapses between such visits. This section of the manual presents only general instructions for servicing recording stream-gaging stations. It is not practical to attempt to provide detailed instructions here for servicing stage-recorders because of the large numbers of such instruments of varying design that are available. For detailed instructions concerning any particular stage-recorder, it is necessary that the hydrographer consult the service manual prepared by the manufacturer of the instrument or prepared by the stream-gaging agencies that use the particular instrument.

The first thing done by the hydrographer on his visit to the recording station is to determine if the clock or timer is running. If it is a strip-chart recorder that is being serviced, the point at which the pen or pencil is resting is then circled; if it is a digital recorder that is being serviced, the instrument is caused to punch the digital tape and the set of punched holes is circled. The time by the hydrographer's watch is noted and the base nonrecording reference gage is read. Those observations are written on the chart or digital tape in proximity to the circled gage height. The hydrographer then reads all gages and notes the chart position or digital-tape position, with respect to time and gage height, of the circled pen mark or punched holes. All that information is also written on the chart or digital tape.

The various gage readings and the recorded gage heights are compared to determine if intakes are functioning properly or if there is a malfunction in the gas-purge system of the bubble gage. The time indicated by the chart or digital tape is compared with the time indicated by the hydrographer's watch to determine if the instrument clock or timer is operating satisfactorily. The section of chart or digital tape bearing the gage-height record is next removed and is examined to determine if there has been any recorder malfunction since the previous servicing of the station.

The clock is then wound, or if the clock is battery driven, the battery voltage is checked and the chart or digital tape is rethreaded into the take-up roll. The pen or punch mechanism is next set to agree with the gage height indicated by the base reference gage, and the chart or digital tape is advanced to give agreement with the hydrographer's watch time. By resetting the recorder at this point, before completing his inspection of the station facilities, the hydrographer gives himself another opportunity to check the operation of the recorder after his work is completed and the instrument has been operating for a period of time.

If the station has a float-operated recorder, the hydrographer inspects the float to determine if it leaks and checks the float-clamp screw to make sure there can be no slippage of the float tape where it joins the float. He also checks the stilling well to be sure there is no unduly large accumulation of sediment in the well. If the well is equipped with an oil tube for winter operation he uses a point on the top of the tube to measure down to the oil surface within the tube and to the water surface outside the tube. The differential between the two measurements, when divided by a value equal to 1.0 minus the specific gravity of the oil, equals the depth of oil in the oil tube. The hydrographer can decide from that computation whether oil has leaked from the tube and the additional amount of oil, if any, to be added. If the stilling well is equipped with a flushing device, the intakes should be flushed as a matter of course. If no flushing device has been provided, and there are indications that the intakes are lagging, the intakes should be cleaned by forcing a plumber's snake through them.

If a high discharge has occurred since the previous visit to a stilling-well-equipped station, high-water marks should be sought both in the stilling well and outside the well, as a check on the peak stage shown by the stage recorder. After making that check, the high-water mark should be cleaned from the inside of the well to prevent confusion with high-water marks that will be left by subsequent peak discharges.

Another means of checking recorded peak stages is by having a wire clip (similar to a paper clip) or magnet attached to the float tape immediately below the shelf through which the tape passes. As the stage rises, the wire clip or magnet, being too large to pass through the hole in the shelf, retains its position at the bottom of the shelf, and the moving float tape slides past it. When the stage recedes, the clip or magnet remains attached to the tape and moves downward as the float moves downward. Any one of several alternative methods may be used to determine the peak stage that had been attained. In one method the peak stage is obtained by subtracting a correction constant from the tape reading at the top of the wire clip. That correction constant is equal to the difference in elevation between the index pointer for the float gage and the bottom of the shelf. In a second method the visiting hydrographer computes the difference between readings on the float tape at the bottom of the shelf and at the top of the wire clip or magnet. He adds that difference to the current gage height to obtain the peak stage attained. A third method of determining the peak stage is to raise the float until the wire clip or magnet is at the bottom of the shelf; the corresponding tape-gage reading is the peak stage that had been attained. Regardless of the method used, after determining the peak stage the wire clip or magnet should be moved back to the bottom of the shelf for subsequent indication of peak stage.

If an indicator of minimum stage is desired, a similar device—wire clip or magnet—can be attached to the float tape immediately below the instrument shelf, but on the counterweight side of the float sheave. The operation of the minimum-stage indicator is similar to that described above for the peak-stage indicator.

If the station has a bubble-gage sensor, the bubble orifice should be inspected to make sure that it has not been buried by a deposit of sediment. A log of gas-feed rate, gas consumption, and gas-cylinder replacement (fig. 35) should be kept to insure a continuous supply of gas and to help in checking for leakage in the system. There can be no serious leak in the gas-purge system if (1) the manometer operates to indicate stage correctly and (2) the gas consumption based on the average bubble rate over a period of time corresponds with the gas consumption computed from the decrease in cylinder pressure. If a gas leak is evident, its location can be determined by isolating various parts of the gas-purge system by the sequential closing of valves in the system. If a high discharge has occurred since a previous visit to the station, a high-water mark should be sought in the vicinity of the base reference gage, as a check on the peak stage shown by the stage recorder.

After the hydrographer has completed his inspection of the station facilities, he returns to the stage recorder and repeats the first steps he had taken in servicing the recorder. He determines that the clock or timer is running; circles the point at which the pen is resting on the

BUBBLE GAGE LOG SHEET Station: Double Trouble Creek near Dry Bone, Ky.

Date	Time	Gage height			Air temp. °F	Pressure, psi		Bubbles per min.	Ft^3 of gas	Delay	Remarks
		Outside	Counter	Pen		Cylinder	Feed				
10-2-58	9:30a	1.31	1.32	1.32	75	1500	34	46	90	On	
11-1-58	1:00p	6.30	6.31	6.31	40	1275	36	44	88	On	Installed new batteries

FIGURE 35.—Sample log sheet for bubble gage.

strip chart; observes his watch time; reads all gages, and writes his observations on the strip chart or digital tape. He does not leave the station without assuring himself that: the recorded gage height and time agree with the gage height of the base reference gage and his watch time; the clock is running; all necessary valves are open; the float wheel (if any) is engaged; the pen (if any) is marking.

A few generalities may be stated concerning the maintenance of recording stream-gaging stations to increase the accuracy and improve the continuity of the stage record. Malfunctions of the recorder can be reduced by the periodic cleaning and oiling of the recorder and clock or timer. Each year intakes, stilling wells, and sediment traps should be thoroughly cleaned. Excessive humidity and temperatures in the gage house should be reduced to a minimum by proper ventilation, and if feasible, extremely cold temperatures in the gage house should be modified by the use of heating units or insulation. Humidity and temperature control reduce the errors associated with paper expansion and contraction. Experience in the U.S.A. has shown that a program of careful inspection and maintenance will result in a complete gage-height record about 98 percent of the time.

FACTORS AFFECTING THE ACCURACY OF THE STAGE RECORD

Continuous records of discharge at a gaging station are computed from the record of stage and the stage-discharge relation. For that purpose stage records having an accuracy of ±0.01 ft (±0.003 m) are generally required. That accuracy can usually be attained by use of the continuous stage-recording systems previously described. The record obtained by sketching a continuous-stage graph through the plotted points representing periodic observations of a nonrecording gage cannot attain such accuracy, of course, but with care the individual gage observations can usually have an accuracy within ±0.01 ft.

In the discussions that follow, nonrecording gages will be treated first and then recording gages will be discussed. A remark appropriate to a discussion of the accuracy of any gage concerns the maintenance of the gage datum to the accuracy criterion of 0.01 ft. That is achieved by running levels to reference marks for the gage (p. 24) and, if necessary, adjusting the gage to restore the original datum. Levels should be run at least once every 2–3 years, and oftener if conditions are known to be unstable. If a nonrecording gage is used only as an outside auxiliary gage for a stilling well, a larger error in gage datum may be tolerated, and adjustment to the gage need not be made for datum discrepancies that do not exceed 0.02 ft (0.006 m). That is so because seldom will inside and outside gages agree exactly even when both are set precisely to the same datum. The inside reference gage, as explained earlier (p. 54) is the base gage for the station; the primary purpose of the outside gage at a station equipped with a stilling well is to indicate whether or not the intakes are operating properly. (As mentioned on page 54, in some countries the outside gage is used as the reference or base gage.)

NONRECORDING GAGES

STAFF GAGE

Settlement or uplift of the structure(s) supporting the staff gage may disturb the gage datum. Where levels from a reference mark show that the datum of an inclined staff gage (p. 26) has been disturbed, the gage is recalibrated by removing the staples used for the graduations and replacing them at the proper elevations. Vertical staff gages are usually made up of several porcelain-enameled iron sections, each about 3.4 ft (1.04 m) long and bearing permanent graduations. Where levels from a reference mark show disturbance of the datum of a vertical staff gage, it is necessary to reset the individual gage sections. The graduations of the manufactured gage plates often have minor discrepancies, and therefore if the gage plates must be reset, they should be reset so that a graduation near the center of each gage plate is at the proper datum.

It is often difficult to accurately detect the water line when making staff-gage observations under the conditions of poor light and (or) clear water. Under those conditions it is helpful to float a matchstick or some similar floatable material against the gage and thereby define the water line. When the water surface is surging rapidly as a result of wave action, the stage to be recorded is the mean of the elevations of the peak and trough of the waves.

WIRE-WEIGHT GAGE

Wire-weight gages (p. 26) are usually mounted on bridges, and changes in gage datum often result from the settlement of bridge abutments or piers, or from changes in the deflection of the bridges resulting either from differences in traffic loading at the times of observation or from seasonal changes in air temperature. In addition to errors attributable to datum changes, erroneous readings of the type A wire-weight gage may also be caused by slippage of the graduated disc of the gage; that slippage results from insecure tightening of the disc screws. The latter condition can be detected if a reading of the gage height of the horizontal checking bar is always made prior to lowering the weight to the water surface.

In checking the datum of the type A wire-weight gage by levels from a reference mark, the elevation of the horizontal checking bar and that of the bottom of the weight at various heights above the water surface are compared with gage readings. When necessary, adjustment of the gage to give accurate readings is made by loosening the graduated disc, rotating the disc to the true gage height of the bottom of the weight as determined from levels, and then retightening the disc screws.

Reliable observations are difficult to obtain by wire-weight gage when the water surface is disturbed by waves; the stage sought under those circumstances is the mean of the elevations of the peak and trough of the waves. On the other hand, it is also difficult to sense the water surface from a high bridge when the water is quiescent. The ideal condition for sensing the water surface occurs when a slow-moving water current is present under the weight. When observations are made on a windy day, the wire that supports the weight will bow rather than hang vertically, thereby causing the gage to underregister. The error introduced will depend on the intensity of the wind and on the height of the gage above the water surface. The combination of a high wind and high bridge may cause appreciable underregistration of the gage.

Another source of error, and one that also increases with height above the water surface, is in the tendency of the weight to rotate about its vertical axis as the weight is being lowered. The rotation twists the wire, and sequences of untwisting and twisting of the wire follow. During each period of twisting the wire shortens and the weight ascends; during each period of untwisting the wire lengthens to its full length and the weight descends. Observation of the gage height of the water surface should not be made until the weight has ceased to rotate.

FLOAT-TAPE GAGE

Change in the datum of a float-tape gage (p. 26) may result from movement of the structure supporting the gage or from slippage of the adjustable index. A float-tape gage will also read incorrectly if the tape has slipped in the clamp connecting the tape and float (gage will overregister) or if the float has sprung a leak and has taken in water (gage will underregister). The first steps in checking a float-tape gage therefore are to make sure that the clamp screw bears tightly on the tape and to shake the float for indications of water sloshing within the float.

Where levels from a reference mark show the elevation of the adjustable index to be in error, the screw holding the index is loosened, the index is raised or lowered to its proper position, and the index screw is retightened. The elevation of the water surface in the stilling well is next obtained by measuring with a steel tape to the water surface from the index or other suitable point of known elevation. Any further adjustment that is needed to make the gage height agree with the water-surface elevation is made by adjusting the length of tape at the float clamp. After all adjustments are complete, a record should be made for future reference of the tape graduation at which the tape enters the tape clamp. If the stage of the stream is changing, the valves on the stilling-well intakes should be closed while adjustments to the float-tape gage are being made in order to maintain a constant water level in the stilling well.

ELECTRIC-TAPE GAGE

Change in the datum of an electric-tape gage (p. 28) can only result from movement of the structure supporting the gage, because the position of the index of the gage is permanent with respect to the instrument. An electric-tape gage may also read incorrectly (overregister) if the tape and weight are insecurely clamped together and slippage of the tape has occurred. Where levels from a reference mark show the gage index to be in error, nothing can be done about moving the index, but its new (and true) gage height is made a matter of record. The original datum of the gage can be maintained, however, by adjusting the effective length of the tape, that is, by changing the length of tape that is inserted and clamped in the gage weight (fig.12). In other words, when the tape is unreeled so that the bottom of the weight is, for example, 1.00 ft (0.3 m) below the index, the tape gage should read 1.00 ft (0.3 m) less than the gage height of the index. After the adjustment is completed, a record should be made, for future reference, of the tape graduation at which the tape enters the weight. If the stage of the stream is changing rapidly at the time a reading is to be taken in the stilling well, it is helpful to close the valves on the stilling-well intakes in order to obtain a constant water level in the stilling well.

Use of the electric-tape gage as the inside reference gage is practically a necessity where a small diameter stilling well is used in a cold climate. In that situation oil must be added to prevent freezing, and because the stilling well is too small to accommodate an oil tube (p. 48), the oil must be added in the well itself. Because the oil is lighter than water, the surface of the floating oil in the stilling well will be higher than the water level at the streamward end of the stilling-well intakes. It is the gage height of the water level (GH) that is sought, and it may be computed by either of two equations:

$$GH = \left[\text{gage height of oil surface}\right] - \left[(1.0 - \text{specific gravity of oil}) \times (\text{depth of oil})\right] \quad (1)$$

or

$$GH = \left[\text{gage height of interface of oil and water}\right] + \left[(\text{specific gravity of oil}) \times (\text{depth of oil})\right]. \quad (2)$$

Use of an electric-tape gage makes it a simple matter to measure the distances needed in equation 2. That part of the tape above the weight is rubbed with carpenter's chalk for a distance of about 1.5 ft (0.5 m) above the weight, and the weight is then lowered until a signal is received on the voltmeter. At that point the electric-tape gage reading obtained will be that of the oil-water interface (p. 28). The tape is then reeled up and the lower end inspected. The oil will have wetted the chalk on the tape leaving a sharp demarcation between the wet and dry lengths of tape. The depth of oil is equal to the distance between the demarcation line and the bottom of the weight. If the depth of the oil is less than the length of the weight, the oil will not reach the tape. In that event, the weight is lowered a known distance below the oil-water interface—say, 1.0 ft (0.3 m)—and the apparent oil depth that is determined is reduced by 1.0 ft (0.3 m), the additional distance that the weight was lowered. The computed oil depth and the observed gage height of the oil-water interface are used in equation 2 to compute the true gage height.

CHAIN GAGE

Change in the datum of a chain gage (p. 31) can result from movement of the structure on which the gage is mounted, but there are many other sources of error. They include stretching of the chain, foreign material between chain links, wear of the chain links, and wear of the pulley wheel. For that reason the chain length between the bottom of the weight and the chain index markers should be measured at each visit by the hydrographer, using a 4-lb (2-kg) pull on the chain. A small spring balance is usually used to put the proper tension on the chain when measuring its length. If the chain length differs by more than 0.02 ft (0.006 m) from the "true" value established the last time the datum was checked by levels, the length of the chain is adjusted at the point where the chain is attached to the weight. The adjustment is easily made because the chain and weight are attached by means of a cotter pin through one of a series of holes in the neck of the weight, the spacing between holes being about 0.02 ft (0.006 m).

In checking the gage datum by levels from a reference mark, the gage height of a point of known elevation on the supporting structure (reference point) is first checked. Next, the elevation of the bottom of the weight at one or more heights above the water surface, as determined by levels from a reference mark, is compared with gage readings. When necessary, adjustment of the chain length to give accurate gage readings is made in the manner indicated above. The length of the chain is measured and recorded before and after the adjustment.

The reliability of gage readings of the water surface is affected by wind, waves, and water current, in a manner similar to that discussed for the wire-weight gage (p. 65). There is no tendency, however, for the weight to rotate when lowered, as described in the discussion of the wire-weight gage.

ACCURACY OF FLOAT-OPERATED RECORDERS

This section of the report discusses the inaccuracies inherent in any float-operated instrument; the discussion is not concerned with such sources of error as datum changes, faulty intake operation, float leakage, float-tape slippage, and paper expansion, which were discussed in the preceding sections. The principal sources of error inherent in a float-operated instrument are float lag, line shift, and submergence of the counterweight (Stevens, 1921). With regard to the algebraic sign of the errors discussed below, a positive (+) sign indicates that the instrument shows a stage higher than the true stage, and a negative (−) sign indicates that the instrument underregisters.

Float lag.—If the float-operated recorder is set to the true water level while the water level is rising, it will thereafter show the correct water level, as far as float lag is concerned, for all rising stages. For falling stages, however, the recorded stage will be above the true water level (positive error) by the amount of float lag or change in flotation depth of the float. A reverse effect occurs if the original gage setting is made when the water level is falling. Float lag varies directly with the force (F) required to move the mechanism of the recorder and inversely as the square of the float diameter (D). F commonly ranges from 1 to 5 oz (0.03 to 0.15 kg) depending on the type and condition of the instrument.

The equation for maximum float-lag error ($MFLE$) is

$$MFLE = 0.37\ F/D^2 \text{ (English units)}, \qquad (3)$$

where $MFLE$ is expressed in feet, D is expressed in inches, and F is expressed in ounces. The equation is

$$MFLE = (0.00256)\, F/D^2 \text{ (metric units)}, \tag{4}$$

where $MFLE$ and D are expressed in meters, and F is expressed in kilograms.

If we assume a value of 3 oz (0.08 kg) for F and a value of 8 in (0.2 m) for D, $MFLE$ equals 0.017 ft (0.005 m). If the recorder was set to the true water level while the float was rising, the record on a falling stage will be in error by +0.017 ft. If however, the index were set at the true level at a stationary stage—that is, at the peak or trough of a changing stage, or when the valves in a stilling well were closed—then the error will be halved on a changing stage. The error will be +0.0085 ft for falling stages and −0.0085 for rising stages.

Line shift.—With every change of stage a part of the float tape passes from one side of the float pulley to the other, and the change in weight changes the depth of flotation of the float. The magnitude of change depends on the change in stage (ΔH) since the last correct setting of the recorder, the unit weight (U) of the tape, and the float diameter (D). The error will be positive (+) for a rising stage and a negative (−) for a falling stage.

The equation for line-shift error (LSE) is

$$LSE = 0.37 \left(\frac{U}{D^2}\right) \Delta H \text{ (English units)}, \tag{5}$$

where U is expressed in ounces per foot, D is in inches, and LSE and ΔH are in feet.

The equation is

$$LSE = (0.00256) \left(\frac{U}{D^2}\right) \Delta H \text{ (metric units)}, \tag{6}$$

where U is expressed in kilograms per meter, and LSE, D, and ΔH are expressed in meters.

If we assume a value of 0.14 oz/ft (0.013 kg/m) for U, a value of 50 ft (15 m) for ΔH, and a value of 8 in (0.2 m) for D, LSE equals 0.04 ft (0.012 m).

Submergence of the counterweight.—When the counterweight and any part of the float line become submerged as the stage rises, the pull on the float is reduced and its depth of flotation is increased. The converse is true when the submerged counterweight emerges from the water on a falling stage. Thus, the error caused by submergence or emergence is opposite to that of the line-shift error and tends to compensate for the line-shift error. The submergence error is dependent on the weight of the counterweight (c) and the float diameter (D).

The equation for submergence error (SE) is

$$SE = 0.017 \frac{c}{D^2} \text{(English units)}, \tag{7}$$

where SE is expressed in feet, c is in ounces, and D is in inches.

The equation is

$$SE = (0.000118) \frac{c}{D^2} \text{(metric units)}, \tag{8}$$

where SE and D are expressed in meters and c is expressed in kilograms.

If we assume a value of 1.25 lb (0.57 kg) for c and a value of 8 in (0.2 m) for D, $SE = 0.0053$ ft (0.0016 m).

Although not related to errors inherent in a float-operated stage recorder, it might be mentioned here that error in recorded stage may be caused by expansion or contraction of the stilling well of a tall gage structure that is exposed to large temperature changes. For example, a steel well 80 ft (24 m) high, exposed to an increase in temperature of 40°C, will have its instrument shelf raised 0.04 ft (0.012 m), assuming, of course, that the instrument shelter is attached to the well.

Summary.—The errors inherent in a float-operated stage recorder will affect computed discharges. The effect of float lag can be reduced to any desired level by the use of an appropriately large float, and that accuracy level may then be maintained by keeping the instrument cleaned and oiled to reduce friction. The recorder should be set to the gage height in the stilling well when the stage in the well is constant, because the float-lag error in subsequent recorded gage-height will then be only half as large as it would be if the recorder setting were made during a period of changing stage in the stilling well. Even if the stage of the stream is changing, the stage in the well may be kept constant by keeping the intake valves closed while the recorder is being set.

The error resulting from counterweight submergence is a constant function of stage and becomes an integral part of the stage-discharge relation. The same would be true of line-shift error if the recorder settings were made only when the stage is low, but that practice is usually impractical. Line shift can be a significant source of error only at gaging stations that experience a wide range of stage.

ACCURACY OF BUBBLE-GAGE RECORDERS

This section of the report discusses the inaccuracies inherent in any bubble-gage system for sensing stage; the discussion is not concerned with such sources of error as datum corrections, sediment deposition on the bubble orifice, and leaks in the system, which were discussed in the preceding sections. The principal sources of error inherent in a bubble-gage recorder are variation in gas friction, variation in required bubble-feed rate with rate of increase in stage, and variation in weight of gas column with stage.

Variation in gas friction.—Friction created by the flow of gas through the bubble tubing results in the pressure at the manometer being slightly higher than that at the orifice. If the bubble-feed rate could be kept constant and temperature did not vary, the friction would remain constant and the accuracy of the gage would be unaffected, because the manometer is always set to agree with the water-surface elevation. However, changes in gas-feed rates cause variation in the friction of the gas flowing through the tube, and where long bubble tubes are used, the variation in friction can produce significant error in recorded gage height.

Inaccuracies due to variation in gas friction can be eliminated by using two gas tubes—one to feed gas to the bubble orifice, the other to act as a static pressure tube to transmit pressure from a point at or near the orifice back to the manometer.

As a conservative criterion for determining when the use of dual tubing is desirable, it is suggested that variations in gas friction be limited to 0.01 ft (0.003 m). If, for a given length of orifice line, an error no greater than 0.01 ft results from a 100 percent increase in bubble rate, a single bubble tube will be satisfactory. Figure 36, based on laboratory tests using the standard U.S. Geological Survey bubble gage shows the relation between the length of bubble tubing and a 100-percent variation in bubble rate for a gas-friction error of 0.01 ft. To illustrate the use of figure 36, assume that a 100-percent variation in bubble rate represents a change from 40 bubbles per minute to 80 bubbles per minute. The corresponding length of bubble tubing indicated by the diagram is 370 ft (113 m). Thus any length of single tubing up to 370 ft could be used in this particular case without introducing a friction error in excess of 0.01 ft. If more than 370 ft of tubing is required at this particular site, and a gage-height error greater than 0.01 ft cannot be tolerated, two gas tubes should be used. When two tubes are used, they should be joined at a T-connector, from whose vertical leg a single tube extends to the orifice. The T-connector should be located above the normal high-water elevation to prevent the entry of water into both legs of the system if pressure loss occurs. Additional valves must also be provided so that the static pressure tube can be separately purged, if necessary.

Variation in required bubble-feed rate with rate of increase in stage.—During rapid rises in stage the instrument will lag if the

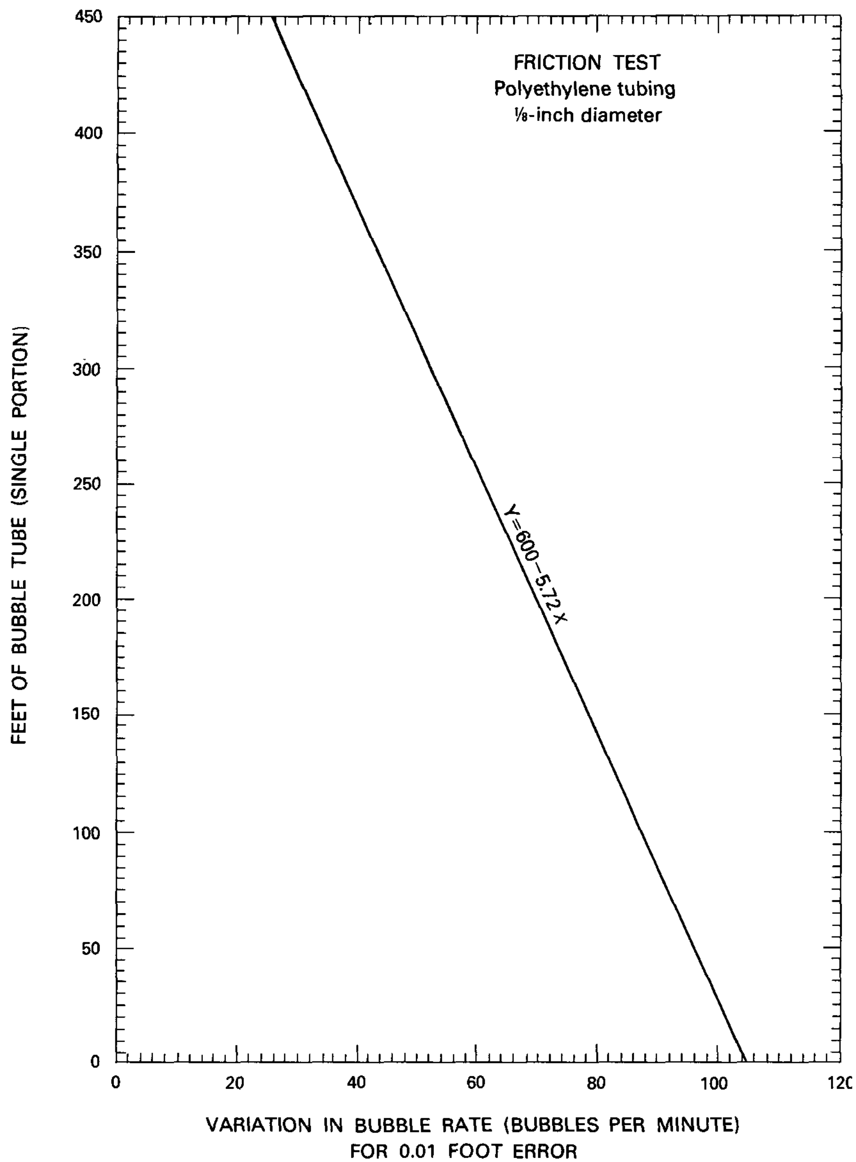

FIGURE 36.—Diagram for determining allowable length of bubble tubing for maximum friction error of 0.01 ft.

bubble-feed rate is too low. Laboratory tests on the standard U.S. Geological Survey bubble gage indicate that figure 37 may be used to

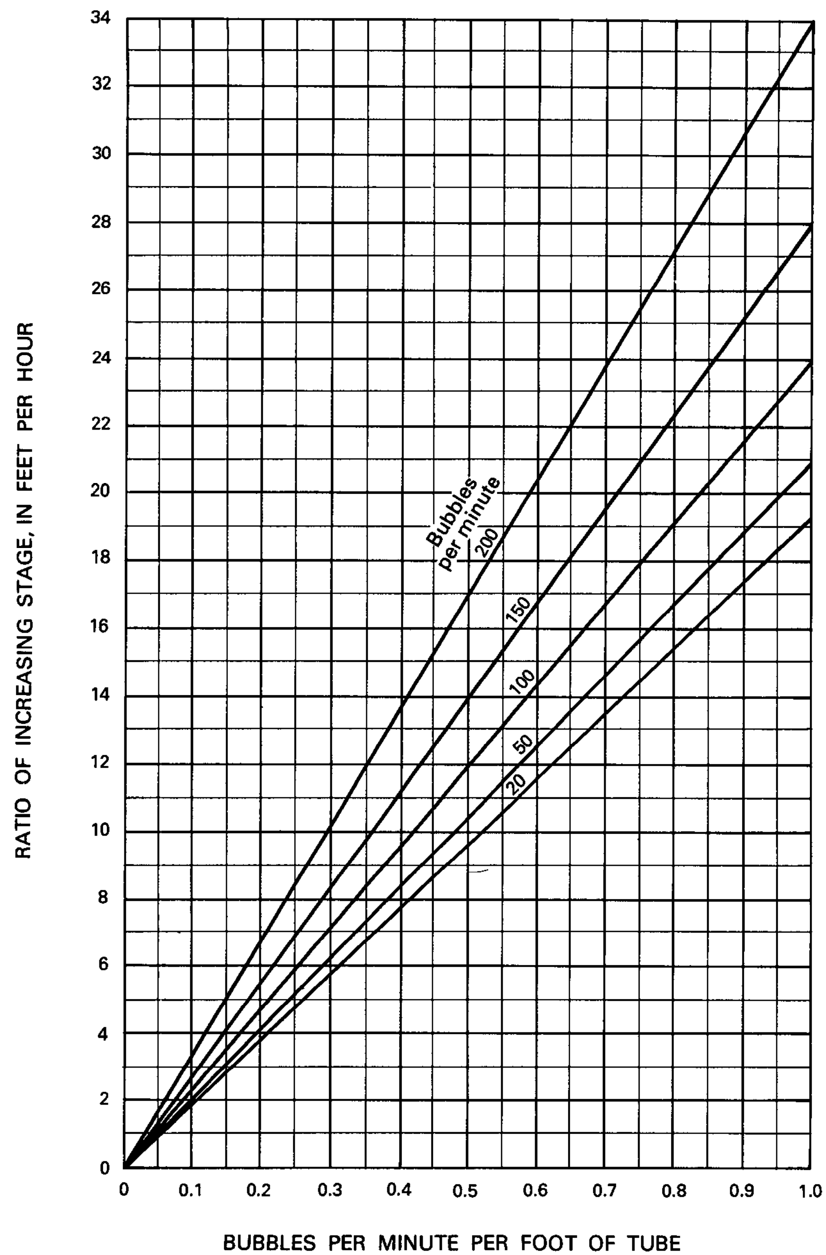

FIGURE 37.—Diagram for determining required bubble rate.

determine the bubble rate required for the maximum expected rate of increase of stage, where the minimum expected stage above the orifice will be less than 10 ft (3.0 m) for long periods of time. For periods when a minimum stage of more than 10 ft is expected to prevail, a lower bubble rate should be used to conserve gas, because the rate of increase of stage that a given bubble rate will support increases directly with stage. It was found that for installations where the minimum stage will continuously be higher than 10 ft, the maximum expected rate of increase of stage should be divided by $\frac{H_{(min)} + 33}{33}$ before entering figure 37. ($H_{(min)}$ is the minimum expected stage above the orifice.) For example, for a given expected rate of increasing stage, the adequate bubble rate for a minimum stage of 100 ft (30.0m) will be one-fourth of that for a minimum stage of 10 ft—the factor $\frac{100 + 33}{33}$ equals 4.

Variation in weight of gas column with stage.—At installations where the manometer is high above the bubble orifice, and large fluctuations in stage occur, the variation in weight of the gas column with stage will cause the manometer to read low (underregister) at high stages. The error varies almost linearly with stage and for that reason the manometer readout in that situation can be corrected without too much difficulty. Depending on the type of instrument used, either the inclination of the manometer is adjusted or a change is made in the gearing between the servosystem and the recording equipment.

As a matter of fact any source of error that varies linearly with stage—for example, the density of the water may increase with stage as a result of increased sediment load—may be compensated for by the adjustments mentioned above. Changes in temperature are usually a negligible source of error, except in high-head installations having large fluctuations in stage. In that situation it is recommended that the bubble-gage system be equipped with a temperature-compensated servomanometer to minimize the error attributable to temperature changes.

SPECIAL PURPOSE GAGES

MODEL T RECORDER

The model T recorder (fig. 38) is a graphic recorder that has a 1:6 gage-height scale and a time scale of 2.4 in. per day. Use of the recorder is limited to a 3-ft (0.9 m) range in stage and it should be serviced weekly, although it will record more than one trace on the 7-day chart. The model T recorder is operated by a motor-wound spring-driven timer, and power is supplied to the timer by a 4½-volt battery. A timer is also available that operates on a 1½-volt battery. The housing of the recorder is designed to fit on the top of a vertical 3-in (0.076 m)-diameter pipe that can be used as the stilling well. This recorder is much cheaper and more compact than the conventional graphic recorders and is well suited for temporary installations for low-flow studies, particularly those concerned with diurnal fluctuation in discharge during low-flow periods. The model T recorder is not used at continuous-record gaging stations.

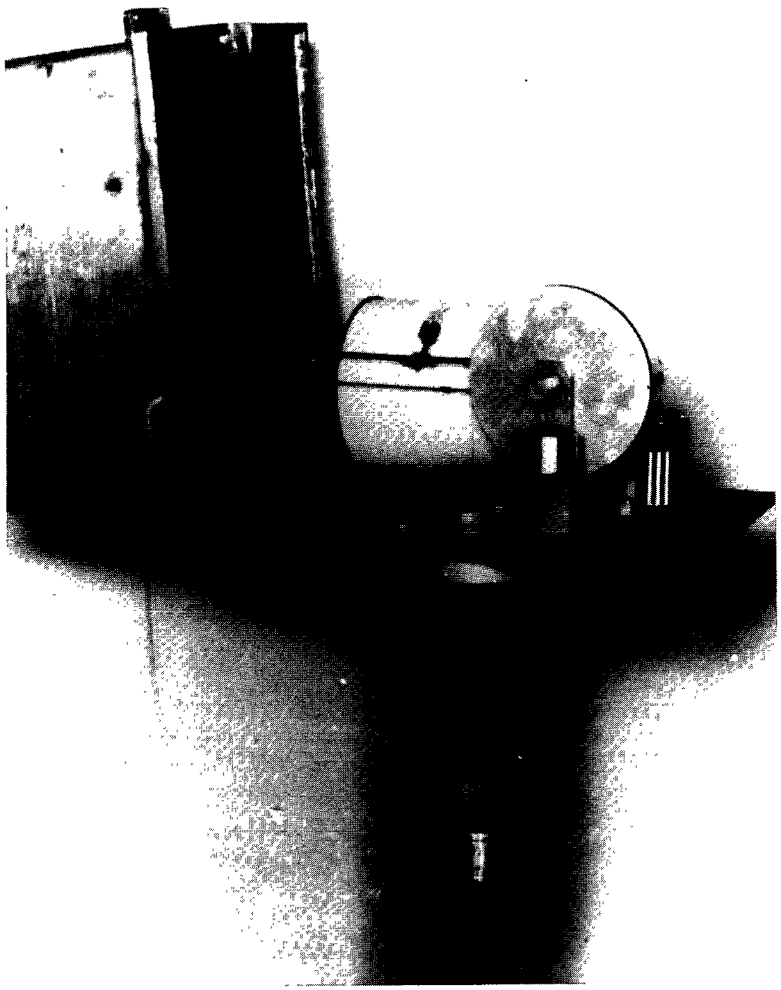

FIGURE 38.—Model T recorder.

SR RECORDER

The model SR recorder (fig. 39) is a graphic recorder that records flood stages and rainfall. A 5-in (0.127m)-diameter flat circular disc is rotated by a battery-wound clock. The power source is a 1½-volt battery. The chart is circular and turns one revolution in 24 hr. Three ranges in stage are available for the effective chart width of 2 in (0.05 m): 5, 10, or 20 ft. The recorder sits on a 2-in-diameter pipe that serves as the stilling well (similar to that for the model T). Five inches of rainfall can also be recorded on the effective width of the chart, but the rainfall reservoir can be equipped with a siphon that will allow an unlimited amount of rainfall to be recorded. The rainfall reservoir is a separate 2-in-diameter pipe that fills at the rate of 1 ft (0.3 m) for each 1 in (0.025 m) of rainfall. After the pipe has filled to a height of 5 ft (1.52 m), the siphon is tripped to empty the pipe.

The SR recorder is much cheaper and more compact than the conventional graphic recorders and is used for studying rainfall-runoff

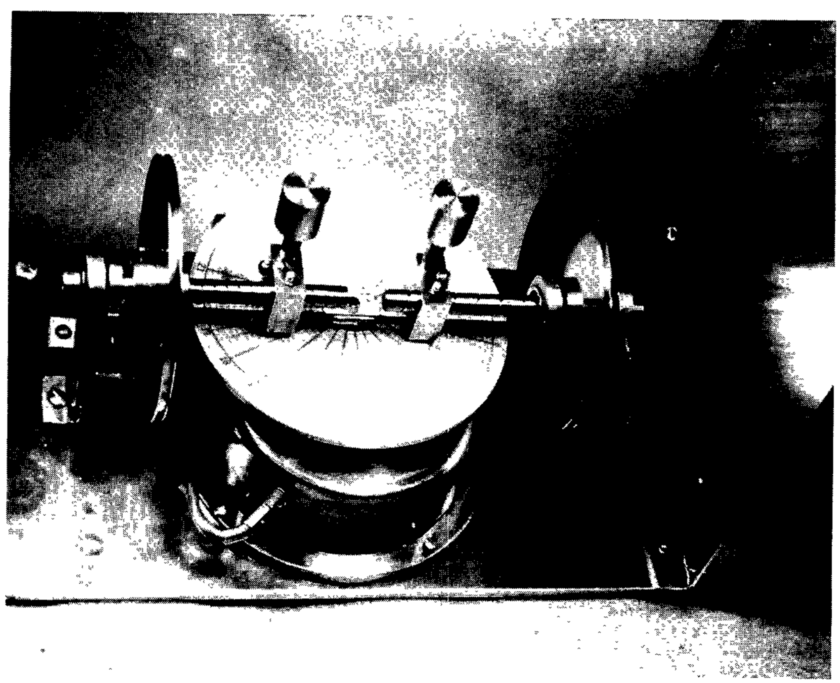

FIGURE 39.—Model SR recorder.

relations for isolated storms on small watersheds. The SR recorder is not used at continuous-record gaging stations.

A word of caution is in order at this point. The SR recorder has not proven to be an unqualified success, although it has performed satisfactorily at many installations. Its use is therefore recommended reservedly.

CREST-STAGE GAGE

The crest-stage gage is a device for obtaining the elevation of the flood crest of streams. The gage is widely used in the U.S.A. because it is simple, economical, reliable, and easily installed. Because of those attributes the crest-stage gage has become a basic instrument in regional studies of flood frequency. For such studies the network of standard gaging stations is augmented by a network of crest-stage gages, thereby providing flood-peak information at a great many sites in the region at reasonable cost.

A crest-stage gage is also a valuable adjunct to the nonrecording gage at nonrecording gaging stations. It provides a record of the peak stages of stream rises, and those stages can be used with the observer's routine readings when sketching the estimated continuous-stage graph through the plotted points of observed stage.

Many different types of crest-stage gages have been tested by the U.S. Geological Survey. (See, for example, Friday, 1965, and Carter and Gamble, 1963.) The one found most satisfactory is a vertical piece of 2-in (0.05 m) galvanized pipe containing a wood or aluminum staff held in a fixed position with relation to a datum reference. (See fig. 40.) The bottom cap has six intake holes located as shown in figure 40 to minimize nonhydrostatic drawdown or superelevation inside the pipe. Tests have shown this arrangement of intake holes to be effective with velocities up to 10 ft/s (3 m/s) and at angles up to 30 degrees with the direction of flow. The top cap contains one small vent hole.

The bottom cap, or a perforated tin cup or copper screening in cup shape attached to the lower end of the staff, contains regranulated cork. As the water rises inside the pipe the cork floats on the water surface. When the water reaches its peak and starts to recede the cork adheres to the staff inside the pipe, thereby retaining the crest stage of the flood. The gage height of a peak is obtained by measuring the interval on the staff between the reference point and the floodmark. Scaling can be simplified by graduating the staff. The cork should be cleaned from the staff before replacing the staff in the pipe to prevent confusion with high-water marks that will be left by subsequent peak discharges.

The datum of the gage should be checked by levels run from a reference mark to the top of the staff, the graduated staff being of

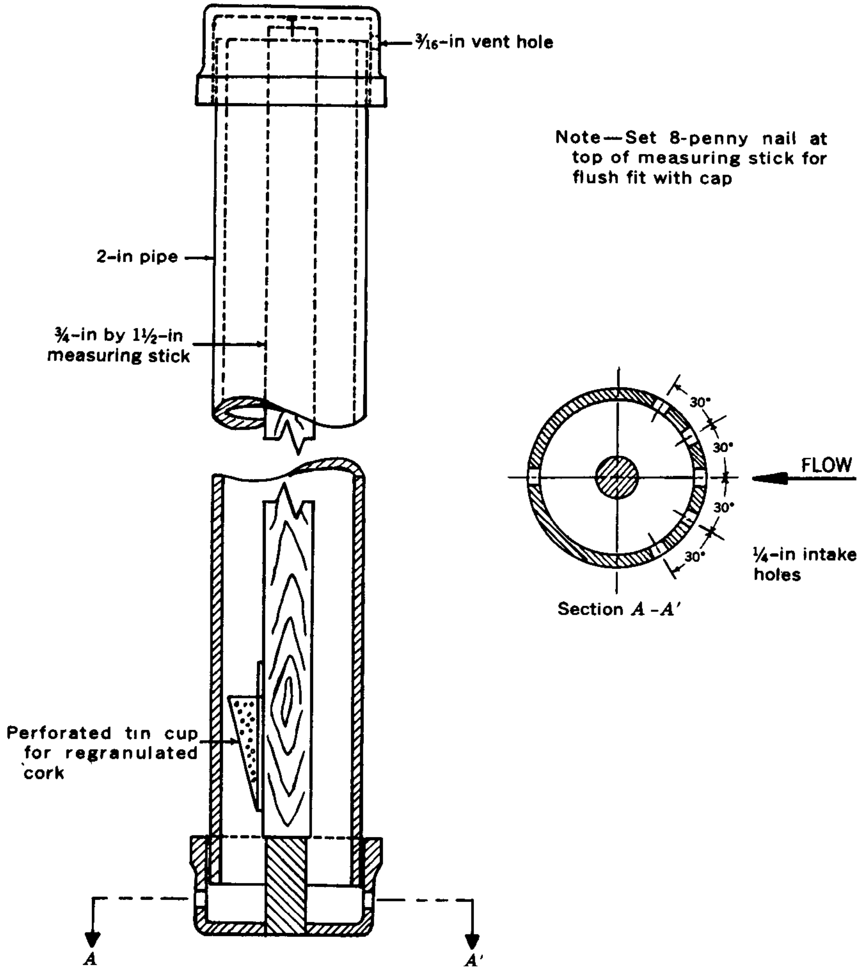

FIGURE 40.—Crest-stage gage.

known length. The gage itself should be serviced on a regular basis. However, the staff should not be removed from the pipe for any reason when the stage is high. If, after such removal, the staff is reinserted when water stands high in the pipe, the resulting surge of the water displaced by the staff will leave an artificial "high-water mark" on the staff.

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CHAPTER 5.—MEASUREMENT OF DISCHARGE BY CONVENTIONAL CURRENT-METER METHOD

INTRODUCTION

Streamflow, or discharge, is defined as the volume rate of flow of water, including any substances suspended or dissolved in the water. Discharge is usually expressed in cubic feet per second or cubic meters per second.

Discharge measurements are made at each gaging station to determine the discharge rating for the site. The discharge rating may be a simple relation between stage and discharge or a more complex relation in which discharge is a function of stage, slope, rate of change of stage, or other factors. Initially the discharge measurements are made with the frequency necessary to define the station rating, as early as possible, over a wide range of stage. Measurements are then made at periodic intervals, usually monthly, to verify the rating or to define any changes in the rating caused by changes in stream-channel conditions.

Discharge measurements may be made by any one of the methods discussed in chapters 5–8. However, the conventional current-meter method is most commonly used in gaging streams. When using this method, observations of width, depth, and velocity are taken at intervals in a cross section of the stream, while the hydrographer is wading or supported by a cableway, bridge, ice cover, or boat. A current meter is used to measure velocity. This chapter describes the conventional current-meter method.

GENERAL DESCRIPTION OF A CONVENTIONAL CURRENT-METER MEASUREMENT OF DISCHARGE

A current-meter measurement is the summation of the products of the subsection areas of the stream cross section and their respective average velocities. The formula

$$Q = \Sigma(a\ v) \tag{9}$$

represents the computation, where Q is total discharge, a is an individual subsection area, and v is the corresponding mean velocity of the flow normal to the subsection.

In the midsection method of computing a current-meter measurement, it is assumed that the velocity sample at each vertical represents the mean velocity in a rectangular subsection. The subsection area extends laterally from half the distance from the preceding observation vertical to half the distance to the next, and vertically from the water surface to the sounded depth. (See fig. 41.)

The cross section is defined by depths at verticals 1, 2, 3, 4, . . . n. At each vertical the velocities are sampled by current meter to obtain the mean velocity for each subsection. The subsection discharge is then computed for any subsection at vertical x by use of the equation,

$$q_x = v_x \left[\frac{(b_x - b_{(x-1)})}{2} + \frac{(b_{(x+1)} - b_x)}{2}\right] d_x$$

$$= v_x \left[\frac{b_{(x+1)} - b_{(x-1)}}{2}\right] d_x \tag{10}$$

where

q_x = discharge through subsection x,
v_x = mean velocity at vertical x,
b_x = distance from initial point to vertical x,
$b_{(x-1)}$ = distance from initial point to preceding vertical,
$b_{(x+1)}$ = distance from initial point to next vertical, and
d_x = depth of water at vertical x.

Thus, for example, the discharge through subsection 4 (heavily outlined in fig. 41) is

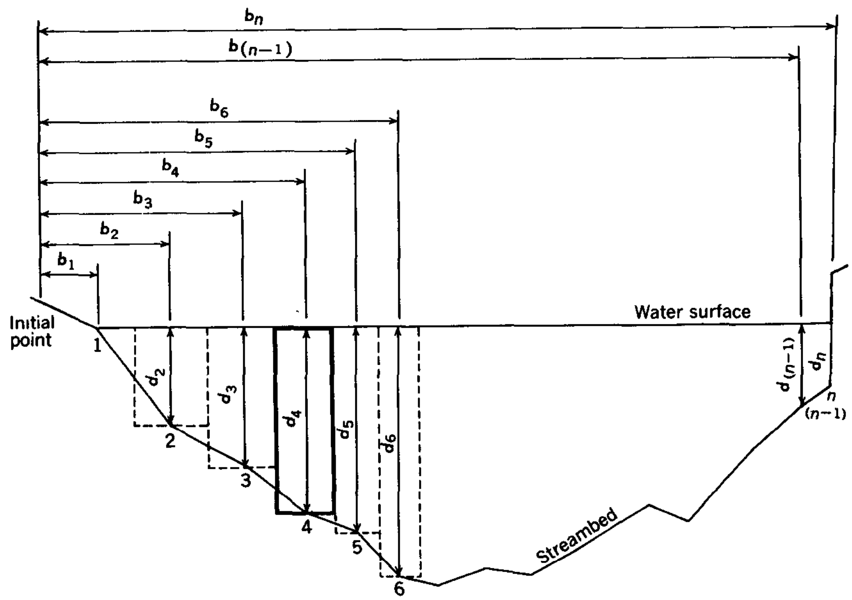
$$q_4 = v_4 \left[\frac{b_5 - b_3}{2}\right] d_4.$$

The procedure is similar when x is at an end section. The "preceding vertical" at the beginning of the cross section is considered coincident with vertical 1; the "next vertical" at the end of the cross section is considered coincident with vertical n. Thus,

$$q_1 = v_1 \left[\frac{b_2 - b_1}{2}\right] d_1$$

and

$$q_n = v_n \left[\frac{b_n - b_{(n-1)}}{2}\right] d_n.$$



EXPLANATION

1, 2, 3 .... n	Observation verticals
$b_1, b_2, b_3, \ldots . b_n$	Distance, in feet or meters, from the initial point to the observation vertical
$d_1, d_2, d_3, \ldots . d_n$	Depth of water, in feet or meters, at the observation vertical
Dashed lines	Boundaries of subsections; one heavily outlined is discussed in text

FIGURE 41.—Definition sketch of midsection method of computing cross-section area for discharge measurements.

For the example shown in figure 41, q_1 is zero because the depth at observation point 1 is zero. However, when the cross-section boundary is a vertical line at the edge of the water as at vertical n, the depth is not zero and velocity at the end vertical may or may not be zero. The formula for q_1 or q_n is used whenever there is water only on one side of an observation vertical such as at piers, abutments, and islands. It is necessary to estimate the velocity at an end vertical, usually as some percentage of the adjacent vertical, because it is impossible to measure the velocity accurately with the current meter close to a boundary. There is also the possibility of damage to the equipment if the flow is turbulent. Laboratory data suggest that the mean vertical velocity in the vicinity of a smooth sidewall of a rectangular channel can be related to the mean vertical velocity at a distance from the wall equal to the depth. The tabulation below gives values that define the relation.

Distance from wall, as a ratio of the depth	*Mean vertical velocity, as related to V_D*
0.00	$0.65V_D$
.25	$.90V_D$
.50	$.95V_D$
1.00	$1.00V_D$

NOTE—V_D is the mean vertical velocity at a distance from the vertical wall equal to the depth.

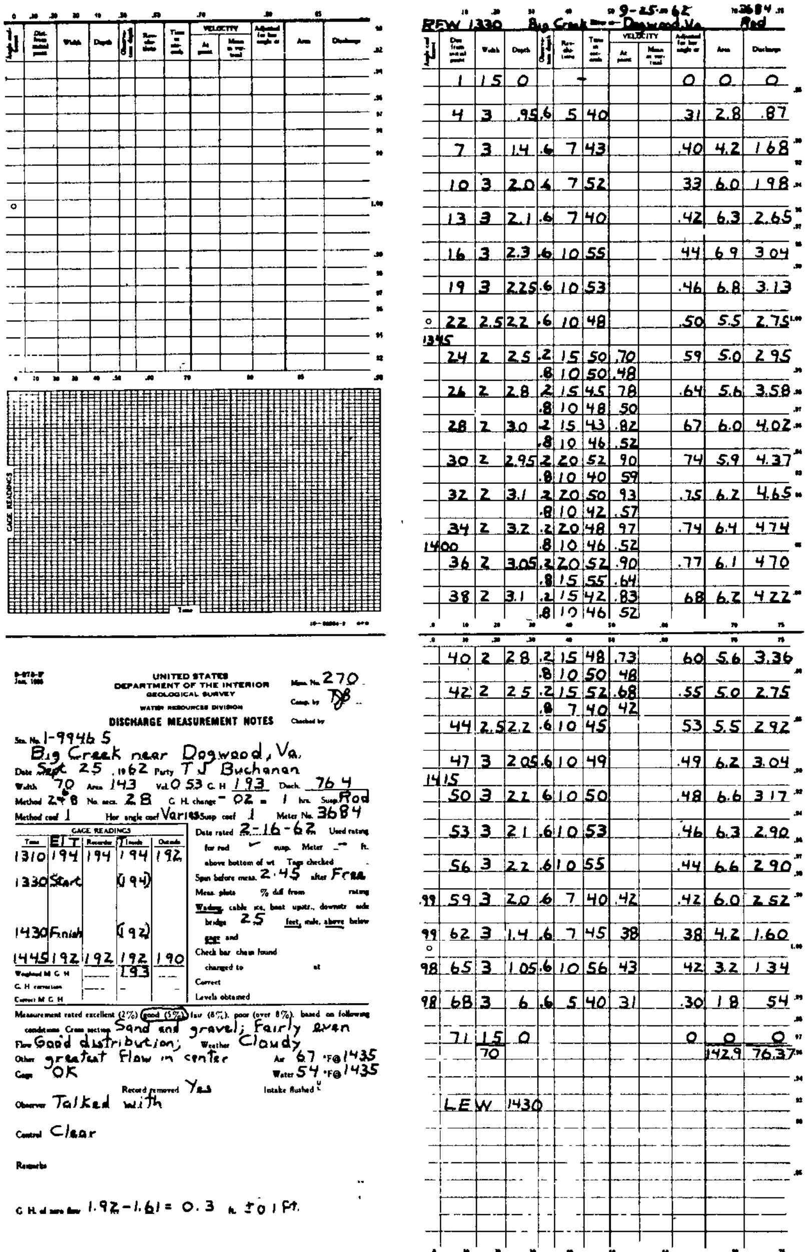
The summation of the discharges for all the subsections—usually 25 to 30 in number—is the total discharge of the stream. An example of the measurement notes used by the U.S. Geological Survey is shown in figure 42.

The mean-section method, used by the U.S. Geological Survey prior to 1950, differs from the midsection method, described above, in computation procedure. In the older method discharges are computed for subsections between successive observation verticals. The velocities and depths at successive verticals are each averaged, and each subsection extends laterally from one observation vertical to the next. Subsection discharge is the product of the average of two mean velocities, the average of two depths, and the distance between observation verticals. In both methods total discharge is the sum of the subsection discharges. A study by Young (1950) concluded that the midsection method is simpler to compute and is a slightly more accurate procedure than the mean-section method.

Current-meter meaurements are usually classified in terms of the means used to cross the stream during the measurement, such as wading, cableway, bridge, boat, or ice cover.

INSTRUMENTS AND EQUIPMENT

Current meters, timers, and a means of counting meter revolutions are needed for the measurement of discharge, along with additional


9-279-F
Jan. 1955

UNITED STATES
DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY
WATER RESOURCES DIVISION
DISCHARGE MEASUREMENT NOTES

Meas. No. 270
Comp. by TJB
Checked by

Sta. No. 1-9946 5
Big Creek near Dogwood, Va.
Date Sept 25, 1962 Party T J Buchanan
Width 70 Area 143 Vel. 0 53 G. H. 1 93 Disch. 76 4
Method 2-8 No. secs. 28 G. H. change - 02 in 1 hrs. Susp. Rod
Method coef 1 Hor. angle coef Varies Susp. coef 1 Meter No. 3684

GAGE READINGS				
Time	EIT	Recorder	T Inside	Outside
1310	194	194	194	192
1330	Start		(194)	
1430	Finish		(192)	
1445	192	192	192	190
Weighted M. G. H.		—	193	—
G. H. correction		—	—	—
Correct M. G. H.				

Date rated 2-16-62 Used rating for rod ✓ susp. Meter — ft. above bottom of wt. Tags checked
Spin before meas. 2.45 after Free
Meas. plots % diff. from rating
Wading, cable, ice, boat, upstr., downstr., side bridge 25 feet, mile, above, below gage, and
Check bar, chain found
changed to at
Correct
Levels obtained

Measurement rated excellent (2%), good (5%), fair (8%), poor (over 8%), based on following conditions: Cross section Sand and gravel; Fairly even
Flow Good distribution; Weather Cloudy
Other greatest flow in center Air 67 °F @ 1435
Gage OK Water 54 °F @ 1435
Record removed Yes Intake flushed
Observer Talked with
Control Clear
Remarks

G. H. of zero flow 1.92 - 1.61 = 0.3 ft. ± 0.1 ft.

REW 1330 Big Creek near Dogwood, Va. 9-25-62 3684 Rod

Angle coef.	Dist. from initial point	Width	Depth	Observation depth	Revolutions	Time in seconds	Velocity at point	Mean in vertical	Adjusted for hor. angle	Area	Discharge
	1	1.5	0						0	0	0
	4	3	.95	.6	5	40			.31	2.8	.87
	7	3	1.4	.6	7	43			.40	4.2	1 68
	10	3	2.0	.6	7	52			33	6.0	1 98
	13	3	2.1	.6	7	40			.42	6.3	2.65
	16	3	2.3	.6	10	55			44	6 9	3 04
	19	3	2.25	.6	10	53			.46	6.8	3.13
○	22	2.5	2.2	.6	10	48			.50	5.5	2.75
1345											
	24	2	2.5	.2	15	50	.70		59	5.0	2 95
				.8	10	50	.48				
	26	2	2.8	.2	15	45	78		.64	5.6	3.58
				.8	10	48	50				
	28	2	3.0	.2	15	43	.82		67	6.0	4.02
				.8	10	46	.52				
	30	2	2.95	.2	20	52	90		74	5.9	4.37
				.8	10	40	59				
	32	2	3.1	.2	20	50	93		.75	6.2	4.65
				.8	10	42	.57				
	34	2	3.2	.2	20	48	97		.74	6.4	4 74
1400				.8	10	46	.52				
	36	2	3.05	.2	20	52	.90		.77	6.1	4 70
				.8	15	55	.64				
	38	2	3.1	.2	15	42	.83		68	6.2	4 22
				.8	10	46	52				
	40	2	2 8	.2	15	48	.73		60	5.6	3.36
				.8	10	50	48				
	42	2	2 5	.2	15	52	.68		.55	5.0	2.75
				.8	7	40	42				
	44	2.5	2.2	.6	10	45			53	5.5	2 92
	47	3	2 05	.6	10	49			.49	6.2	3.04
1415											
	50	3	2.2	.6	10	50			.48	6.6	3 17
	53	3	2 1	.6	10	53			.46	6.3	2.90
	56	3	2.2	.6	10	55			.44	6.6	2 90
.99	59	3	2.0	.6	7	40	.42		.42	6.0	2.52
.99	62	3	1.4	.6	7	45	38		38	4.2	1.60
○											
.98	65	3	1 05	.6	10	56	43		42	3.2	1 34
.98	68	3	.6	.6	5	40	31		.30	1 8	.54
	71	1.5	0						0	0	0
		70								142.9	76.37
	LEW		1430								

FIGURE 42.—Computation notes of a current-meter measurement by the midsection method.

equipment that depends on the manner in which the measurement is to be made—that is, whether by wading, cableway, bridge, boat, or from ice cover. Instruments and equipment used in making the current-meter measurements are described in this section of the manual under the following categories: current meters, sounding equipment, width-measuring equipment, equipment assemblies, and miscellaneous equipment.

CURRENT METERS

A current meter is an instrument used to measure the velocity of flowing water. The principle of operation is based on the proportionality between the velocity of the water and the resulting angular velocity of the meter rotor. By placing a current meter at a point in a stream and counting the number of revolutions of the rotor during a measured interval of time, the velocity of water at that point is determined.

The number of revolutions of the rotor is obtained by an electrical circuit through the contact chamber. Contact points in the chamber are designed to complete an electrical circuit at selected frequencies of revolution. Contact chambers can be selected having contact points that will complete the circuit twice per revolution, once per revolution, or once per five revolutions. The electrical impulse produces an audible click in a headphone or registers a unit on a counting device. The intervals during which meter revolutions are counted are timed with a stopwatch. A discussion of the required time interval follows.

Turbulent flow, which is ordinarily found in natural streams and in artificial channels, is always accompanied by local eddying, which results in pulsations in the velocity at any point. Figure 43, taken from a study by Pierce (1941), shows the pulsations observed in a laboratory flume for two different mean velocities. The greater magnitude of the pulsations, relative to the mean, at the lower velocity explains why current-meter observations at a point should cover a longer period when low velocities are being measured than when higher velocities are being measured. At high velocities, pulsations have only minor effect on the current-meter observations. In the U.S.A. it is customary to observe velocity at a point by current meter for a period that ranges from 40 to 70 s. It is recognized that the use of a period of from 40 to 70 s is not long enough to insure the accuracy of a single point-observation of velocity. However, because the pulsations are random and because velocity observations during a discharge measurement are made at 25 to 30 verticals, usually with two observations being made in each vertical, there is little likelihood that the pulsations will bias the total measured discharge of a stream. (See p. 181–182.) Longer periods of current-meter observation at a point are not used, because it is desirable to complete a discharge measurement before the stage changes significantly and because the use of longer observation periods may add significantly to the operating cost of a large number of gaging stations.

Current meters generally can be classified with respect to two main types: those meters having vertical-axis rotors and those having horizontal-axis rotors. The comparative characteristics of these two types are summarized below:

1. Vertical-axis rotor with cups or vanes.
   a. Operates in lower velocities than do horizontal-axis meters.
   b. Bearings are well protected from silty water.
   c. Rotor is repairable in the field without adversely affecting the rating.
   d. Single rotor serves for the entire range of velocities.
2. Horizontal-axis rotor with vanes.
   a. Rotor disturbs flow less than do vertical-axis rotors because of axial symmetry with flow direction.
   b. Rotor is less likely to be entangled by debris than are vertical-axis rotors.
   c. Bearing friction is less than for vertical-axis rotors because bending moments on the rotor are eliminated.

VERTICAL-AXIS CURRENT METERS

A common type of vertical-axis current meter is the Price meter, type AA. (See fig. 44.) This meter is used extensively by the U.S. Geological Survey. The standard Price meter has a rotor 5 in (0.127

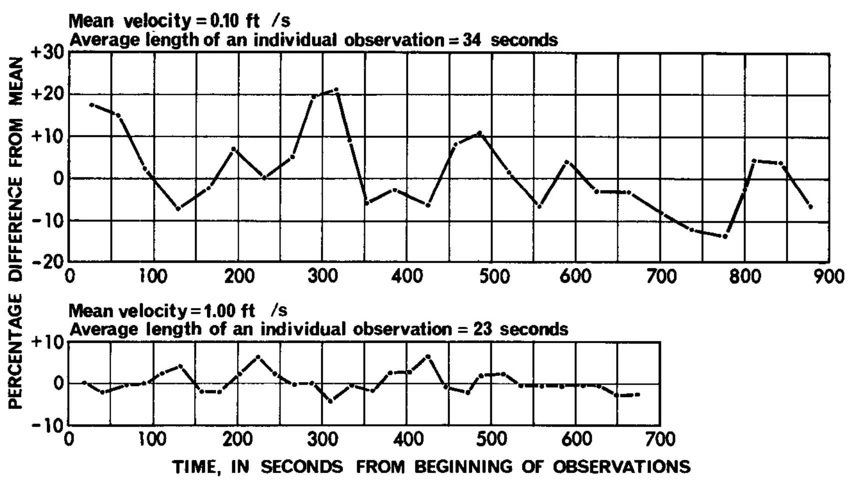


FIGURE 43.—Comparison of pulsations for two different mean velocities measured in a laboratory flume, 12 ft wide.

m) in diameter and 2 in (0.05 m) high with six cone-shaped cups mounted on a stainless-steel shaft. A pivot bearing supports the rotor shaft. The contact chamber houses both the upper part of the shaft and a slender bronze wire (cat's whisker) attached to a binding post. With each revolution an eccentric contact on the shaft makes contact with a bead of solder at the end of the cat's whisker. A separate reduction gear (pentagear), wire, and binding post provide a contact each time the rotor makes five revolutions. A tailpiece keeps the meter pointing into the current.

In addition to the standard type AA meter for general use there is a type AA meter for low velocities. No pentagear is provided. This modification reduces friction. The shaft usually has two eccentrics making two contacts per revolution. The low-velocity meter normally is rated from 0.2 to 2.5 ft/s (0.06 to 0.76 m/s) and is recommended for use when the mean velocity at a cross section is less than 1 ft/s (0.3 m/s).

In addition to the type AA meters, the U.S. Geological Survey uses a Price pygmy meter in shallow depths. (See fig. 44.) The pygmy meter is scaled two-fifths as large as the standard meter and has neither a tailpiece nor a pentagear. The contact chamber is an integral part of the yoke of the meter. The pygmy meter makes one contact per revolution and is used only with rod suspension.

The U.S. Geological Survey has recently developed a four-vane vertical-axis meter. (See fig. 45.) This meter is useful for measurements under ice cover because the vanes are less likely to fill with slush ice and because it requires a much smaller hole for passage through the ice. One yoke of the vane meter is made to be suspended at the end of a rod and will fit holes made by an ice drill. Another yoke

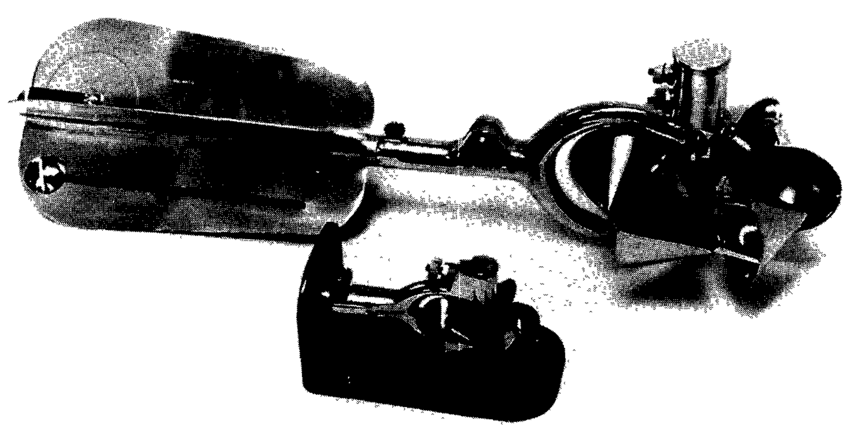


FIGURE 44.—Price type AA meter, top; Price pygmy meter, bottom.

is made for regular cable or rod suspension. (See fig. 45.) The vane meter has the disadvantage of not responding as well as the Price type AA meter at velocities less than 0.5 ft/s (0.15 m/s).

A new contact chamber has been designed by the U.S. Geological Survey to replace the wiper contact of the type AA and vane meters. The new contact chamber contains a magnetic switch, glass enclosed in a hydrogen atmosphere and hermetically sealed. The switch assembly is rigidly fixed in the top of the meter head just above the tip of the shaft. The switch is operated by a small permanent magnet rigidly fastened to the shaft. The switch quickly closes when the magnet is alined with it and then promptly opens when the magnet moves away. The magnet is properly balanced on the shaft. Any type AA meter can have a magnetic switch added by replacing the shaft and the contact chamber. The magnetic switch is placed in the special contact chamber through the tapped hole for the binding post. The rating of the meter is not altered by the change. An automatic counter (p. 130) is used with the magnetic-switch contact chamber. If a headphone is used, arcing may weld the contacts.

A Price meter accessory that indicates the direction of flow is described on page 129.

Vertical-axis current meters do not register velocities accurately when placed close to a vertical wall. A Price meter held close to a right-bank vertical wall will underregister because the slower water velocities near the wall strike the effective (concave) face of the cups. The converse is true at a left-bank vertical wall. (The terms "left bank" and "right bank" designate direction from the center of a stream for an observer facing downstream.) The Price meter also

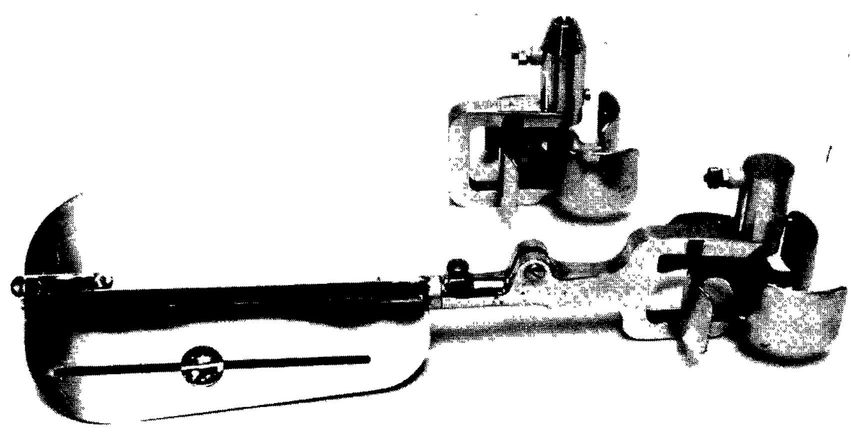


FIGURE 45.—Vane ice meter, top; vane meter with cable suspension yoke, bottom.

underregisters when positioned close to the water surface or close to the streambed.

HORIZONTAL-AXIS CURRENT METERS

The types of horizontal-axis meters most commonly used are the Ott, Neyrpic, Haskell, Hoff, and Braystoke. The Ott meter is made in Germany, the Neyrpic meter in France, and both are used extensively in Europe. The Haskell and Hoff meters were developed in the United States, where they are used to a limited extent. The Braystoke meter is used extensively in the United Kingdom. The Ott meter (fig. 46) is a precision instrument but is not widely used in the U.S.A. because it is not as durable as the Price meter under extreme conditions. The makers of the Ott meter have developed a component propeller that in oblique currents automatically registers the velocity component at right angles to the measuring section for angles as great as 45° and

FIGURE 46.—Ott current meter.

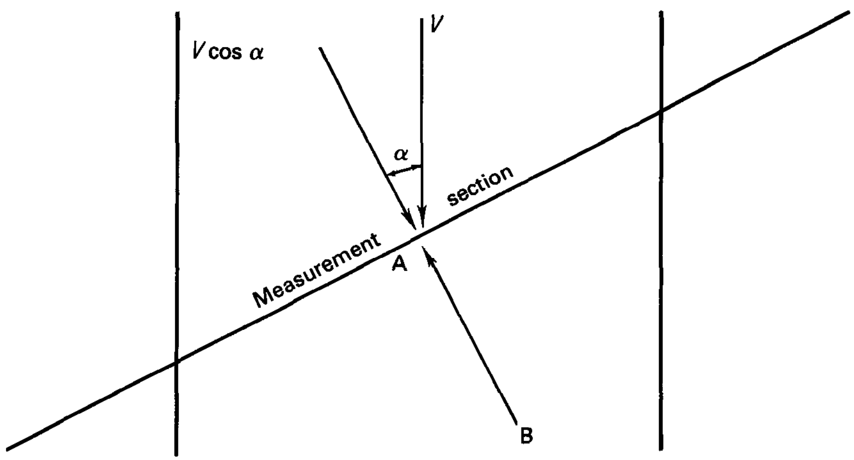


FIGURE 47.—Velocity components measured by Ott and Price current meters.

for velocities as great as 8 ft/s (2.4 m/s). For example, if this component propeller were held in the position *AB* in figure 47 it would register $V \cos \alpha$ rather than V, which the Price meter would register.

The Neyrpic meter is used rarely in the U.S.A. for the same reason that the Ott meter is rarely used there.

The Haskell meter has been used by the U.S. Lake Survey, Corps of Engineers, in streams that are deep, swift, and clear. By using propellers with a variety of screw pitches, a considerable range of velocity can be measured. The Haskell meter is more durable than most other horizontal-axis current meters.

The Hoff meter (fig. 48) is another current meter used in the U.S.A. The lightweight propeller has three or four vanes of hard rubber. The meter is suited for the measurement of low velocities but is not suited for rugged use.

COMPARISON OF PERFORMANCE OF VERTICAL-AXIS AND HORIZONTAL-AXIS CURRENT METERS

Comparative tests of the performance of vertical-axis and horizontal-axis current meters, under favorable measuring conditions, indicate virtually identical results from use of the two types of meter. That was the conclusion reached in 1958 by the U.S. Lake Survey, Corps of Engineers, after tests made with the Price, Ott, and Neyrpic current meters (Townsend and Blust, 1960). The results of one of their tests is shown in figure 49.

Between the years 1958 and 1960, the U.S. Geological Survey made 19 simultaneous discharge measurements on the Mississippi River using Price and Ott meters. The average difference in discharge between results from the two meters was −0.15 percent, using the measurements made with the Price meter as the standard for comparison. The maximum differences in discharge measured by the two meters was −2.76 and +1.53 percent.

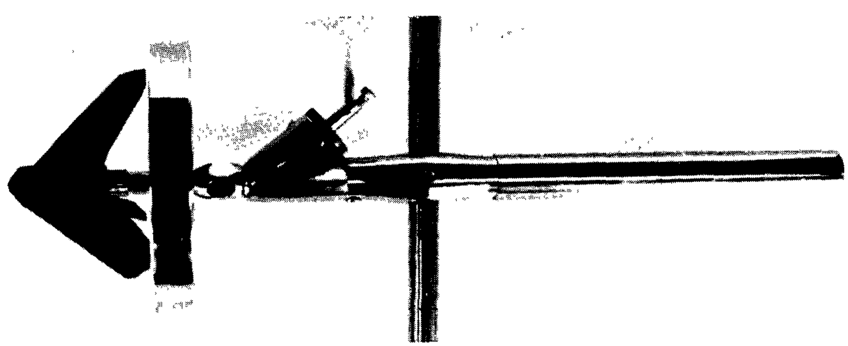


FIGURE 48.—Hoff current meter.

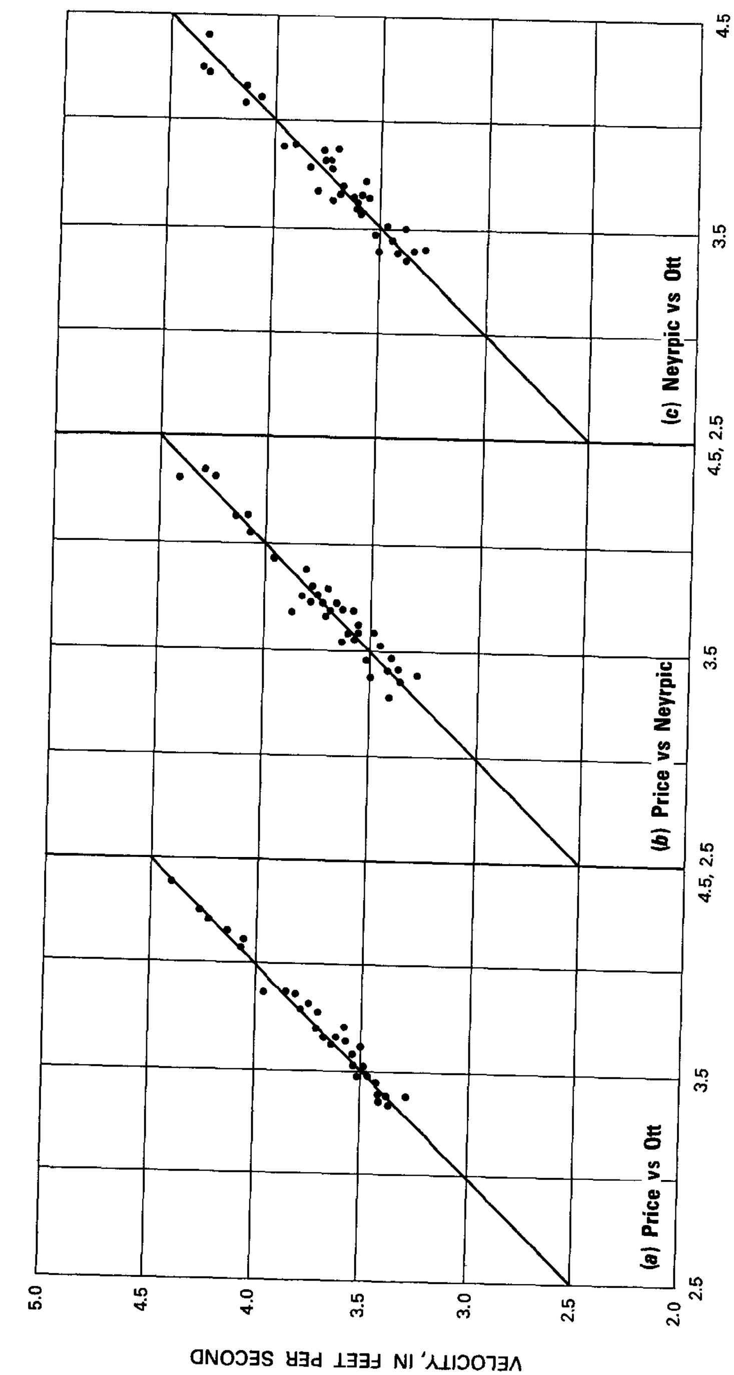


FIGURE 49.—Comparison of mean *velocities* measured simultaneously by various current meters during 2-min periods, Stella Niagara section, Panel point 5. (After Townsend and Blust, 1960.)

OPTICAL CURRENT METER

In recent years the U.S. Geological Survey has developed an optical current meter (fig. 50). The meter and its use have been described by Chandler and Smith (1971). The optical current meter is designed to measure surface velocities in open channels without immersing equipment in the stream. However, because it measures only surface velocity, the optical meter is not considered a substitute for conventional equipment in those situations where good measurements can be made by standard techniques. It is a device that has extended the capability of making discharge measurements to a range of situations under which standard current-meter techniques cannot be used. Those situations include flood velocities that are too high to be measured by conventional meter—for example, supercritical velocities in floodways—or the presence of a debris load during flood periods that makes it hazardous to immerse a current meter.

Basically, the meter is a stroboscopic device consisting of a low-power telescope, a single oscillating mirror driven by a cam, a variable-speed battery-operated motor, and a tachometer. The water surface is viewed from above through the meter, while gradually changing the speed of the motor to bring about synchronization of the

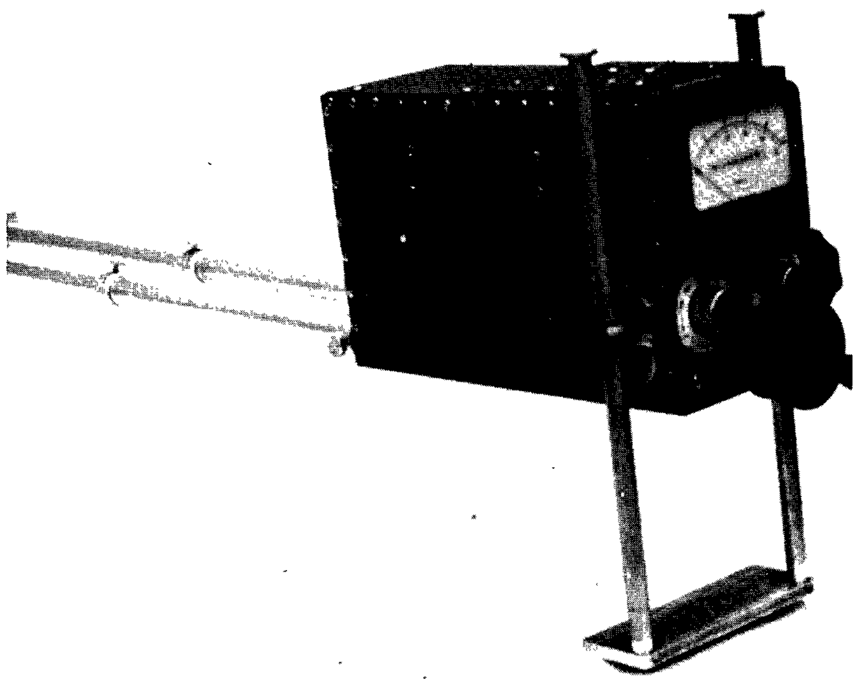


FIGURE 50.—Optical current meter.

angular velocity of the mirror and the surface velocity of the water. Synchronization is achieved when the motion of drift or disturbances on the water surface, as viewed through the meter, is stopped. A reading of the tachometer and height of the meter above the water surface are the only elements needed to compute the surface velocity.

The velocity measurement may be made from any bridge, walkway, or other structure that will support the optical meter. The vertical axis of the meter must be perpendicular to the water surface. Surface velocity (V_s) is computed from the equation

$$V_s = KRD,$$

where K is the constant for the meter, R is the readout from the tachometer, and D is the distance to the water surface in feet. The tachometer is scaled to produce a value of K equal to 1.00. The surface velocity computed from the above equation must be corrected by an appropriate coefficient to obtain the mean velocity in the vertical. The precise coefficient applicable is, of course, unique to the particular stream and to the location of the vertical in the stream cross section. However, data abstracted from conventional current-meter measurements show that application of a coefficient of 0.90 will not introduce errors of more than ±5 percent in concrete-lined channels. For natural channels a coefficient of 0.85 has been used.

A unique feature of the optical current meter is the automatic correction that is made for variations in the direction of the streamlines of flow. If the flow approaches the cross section at an angle other than the perpendicular, and if the axis of the oscillating mirror in the meter is parallel to the cross section, then at the null point of observation, the water will appear to move laterally across the field of view. The meter measures only the velocity vector normal to the cross section, and there is no need to apply horizontal angle corrections.

The range of velocities that can be measured with the optical current meter is limited at the lower end by the accuracy of the tachometer and at the upper end by the physical limitations of the human eye. Table 1 shows the range of velocities that can be meas-

TABLE 1.—*Range of velocities that can be measured with optical current meter*

Observation height (D)		Maximum velocity		Minimum velocity for ±5 percent resolution	
(ft)	(m)	(ft/s)	(m/s)	(ft/s)	(m/s)
5	1.52	25	7.62	1.6	0.49
10	3.05	50	15.2	3.2	.98
15	4.57	75	22.9	4.8	1.46
20	6.10	100	30.5	6.4	1.95

ured from various heights above the water surface with the U.S. Geological Survey model of the meter. The minimum velocities shown in column 3 of the table can be measured with an error of ±5 percent; the higher velocities at the various observation heights will be measured with lesser error.

CARE OF THE CURRENT METER

To insure reliable observations of velocity it is necessary that the current meter be kept in good condition. The care of conventional meters will be discussed first.

Before and after each discharge measurement the meter cups or vanes, pivot and bearing, and shaft should be examined for damage, wear, or faulty alinement. Before using the meter its balance on the cable-suspension hanger should be checked, the alinement of the rotor when the meter is on the hanger or wading rod should also be checked, and the conductor wire should be adjusted to prevent interference with meter balance and rotor spin. During measurements, the meter should periodically be observed when it is out of the water to be sure that the rotor spins freely.

Meters should be cleaned and oiled daily when in use. If measurements are made in sediment-laden water, the meter should be cleaned immediately after each measurement. For vertical-axis meters the surfaces to be cleaned and oiled are the pivot bearing, pentagear teeth and shaft, cylindrical shaft bearing, and thrust bearing at the cap.

After oiling, the rotor should be spun to make sure that it operates freely. If the rotor stops abruptly, the cause of the trouble should be sought and corrected before using the meter. The duration of spin should be recorded on the field notes for the discharge measurement. A significant decrease in the duration of spin indicates that the bearings require attention. In vertical-axis current meters the pivot requires replacement more often than other meter parts, and it therefore should be examined after each measurement. The pivot and pivot bearing should be kept separated, except during measurements, by use of the raising nut provided in the Price meter or by replacing the pivot with a brass plug in the pygmy meter. Fractured, worn, or rough pivots should be replaced.

Meter repairs by the hydrographer should be limited to minor damage only. That is particularly true of the rotor, where small changes in shape can significantly affect the meter rating. In vertical-axis meters, minor dents in the cups can often be straightened to restore the original shape of the cups, but in case of doubt, the entire rotor should be replaced with a new one. Badly sprung yokes, bent yoke stems, misalined bearings and tailpieces should be reconditioned in shops equipped with the specialized facilities needed.

There are only few details connected with care of the optical current meter. The meter should be transported in a shock-proof carrying case and the battery should be checked periodically. Field performance of the tachometer should also be checked periodically. Three steps are involved in the checking process. First, a cam speed is measured by counting and timing mirror oscillations with a stopwatch, and the corresponding dial reading of the tachometer is observed. Next, a tachometer dial readout is computed from the measured cam speed and the known scale factor of the tachometer dial. In the final step the observed dial reading and the computed dial readout are compared.

RATING OF CURRENT METERS

To determine the velocity of the water from the revolutions of the rotor of a conventional current meter, a relation must be established between the angular velocity of the rotor and the velocity of the water that spins the rotor. That relation is known as the rating of the current meter. The rating is established by first towing the meter at a constant velocity through a long water-filled trough, and then relating the linear and rotational velocities of the current meter. The following paragraphs describe the rating of meters by the U.S. Geological Survey at the Hydrologic Instrumentation Facility in Mississippi.

The rating trough used is a sheltered concrete tank 450 ft (137 m) long, 12 ft (3.7 m) wide, and 12 ft (3.7 m) deep. An electrically driven car rides on rails extending the length of the tank. The car carries the current meter at a constant rate through the still water in the basin. Although the rate of travel can be accurately adjusted by means of an electronic regulating gear, the average velocity of the moving car is determined for each run by making an independent measurement of the distance it travels during the time that the revolutions of the rotor are electrically counted. Eight pairs of runs are usually made for each current meter. A pair of runs consists of two traverses of the basin, one in each direction, at the same speed. Practical considerations usually limit the ratings to velocities ranging from 0.1 to about 15 ft/s (0.03 to about 4.6 m/s), although the car can be operated at lower speeds. Unless a special request is made for a more extensive rating, the lowest velocity used in the rating is about 0.2 ft/s (0.06 m/s), and the highest is about 8.0 ft/s (2.5 m/s).

For convenience in field use, the data from the current-meter ratings are reproduced in tables, a sample of which is shown in figure 51.

UNITED STATES
DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY
Water Resources Division

RATING TABLE FOR TYPE AA CURRENT METER

EQUATIONS V=2 180R + 020 (2 200) 2 170R + 030 Std Rating No 1

Time in Seconds	VELOCITY IN FEET PER SECOND — Revolutions 3	5	7	10	15	20	25	30	40	Time in Seconds
40	183	292	401	565	.837	1.11	1 38	1 65	2 20	40
41	180	286	392	552	818	1 08	1.35	1 62	2 15	41
42	176	280	383	539	799	1 06	1 32	1 58	2 10	42
43	172	273	375	527	780	1 03	1 29	1 54	2 05	43
44	.169	268	367	515	.763	1 01	1 26	1 51	2 00	44
45	165	262	359	504	747	989	1.23	1 47	1 96	45
46	162	257	352	494	.731	968	1 20	1 44	1 92	46
47	159	252	345	484	716	948	1.18	1 41	1 88	47
48	156	247	338	474	701	928	1 16	1 38	1 84	48
49	153	242	331	.465	687	910	1 13	1 35	1 80	49
50	.151	238	325	.456	.674	892	1.11	1 33	1 76	50
51	148	234	319	447	661	875	1 09	1 30	1 73	51
52	146	230	313	.439	649	858	1 07	1 28	1 70	52
53	143	226	308	431	637	843	1 05	1 25	1 67	53
54	141	222	303	424	626	827	1 03	1 23	1 63	54
55	139	218	297	416	615	813	1 01	1 21	1 61	55
56	.137	215	292	409	604	.799	.993	1 19	1 58	56
57	135	211	288	402	594	785	976	1 17	1 55	57
58	133	208	283	396	584	.772	960	1 15	1 52	58
59	131	205	279	389	574	759	944	1 13	1 50	59
60	129	202	274	383	565	747	928	1 11	1 47	60
61	127	199	270	377	556	735	913	1 09	1 45	61
62	125	196	266	.372	547	723	899	1.07	1 43	62
63	.124	193	.262	366	539	712	885	1 06	1 40	63
64	122	190	258	.361	531	.701	.872	1 04	1 38	64
65	121	188	255	355	523	691	858	1 03	1 36	65
66	119	185	251	350	515	681	846	1 01	1 34	66
67	118	.183	248	345	508	.671	.833	996	1 32	67
68	116	180	.244	341	501	661	821	982	1 30	68
69	115	178	241	336	494	652	810	968	1 28	69
70	113	176	238	331	487	643	799	954	1 27	70
	3	5	7	10	15	20	25	30	40	

UNITED STATES
DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY
Water Resources Division

RATING TABLE FOR TYPE AA CURRENT METER

Actual Rating Limits 0 25 to 8 0 feet per second Date 03-05-70

Time in Seconds	VELOCITY IN FEET PER SECOND — Revolutions 50	60	80	100	150	200	250	300	350	Time in Seconds
40	2 74	3 28	4 37	5 45	8.17	10 88	13 59	16 30	19 02	40
41	2 68	3 21	4 26	5 32	7 97	10 62	13 26	15 91	18 55	41
42	2 61	3 13	4 16	5 20	7 78	10 36	12 95	15 53	18 11	42
43	2 55	3 06	4 07	5 08	7 60	10 12	12 65	15.17	17 69	43
44	2 50	2 99	3 98	4 96	7 43	9 89	12 36	14 83	17 29	44
45	2 44	2 92	3 89	4 85	7 26	9.67	12 09	14 50	16 91	45
46	2 39	2 86	3 80	4 75	7 11	9 46	11 82	14 18	16 54	46
47	2 34	2 80	3 72	4 65	6 96	9 26	11 57	13 88	16 19	47
48	2 29	2 74	3 65	4 55	6 81	9 07	11 33	13 59	15 85	48
49	2 24	2 69	3 57	4 46	6 67	8 89	11 10	13 32	15 53	49
50	2 20	2 63	3 50	4 37	6 54	8 71	10 88	13 05	15 22	50
51	2 16	2 58	3 43	4 28	6 41	8 54	10 67	12 79.	14 92	51
52	2 12	2 53	3 37	4 20	6 29	8 38	10 46	12 55	14 64	52
53	2 08	2 49	3 31	4 12	6 17	8 22	10 27	12 31	14 36	53
54	2 04	2 44	3 24	4 05	6 06	8 07	10 08	12 09	14 09	54
55	2 00	2 40	3 19	3 98	5 95	7 92	9 89	11 87	13 84	55
56	1 97	2 35	3 13	3 90	5 84	7 78	9 72	11 65	13 59	56
57	1 93	2 31	3 08	3 84	5 74	7 64	9 55	11 45	13 35	57
58	1 90	2 27	3 02	3 77	5 64	7 51	9 38	11 25	13 12	58
59	1 87	2 24	2 97	3 71	5 55	7 39	9 22	11 06	12 90	59
60	1 84	2 20	2 92	3 65	5 45	7 26	9 07	10 88	12 69	60
61	1 81	2 16	2 88	3 59	5 37	7 14	8 92	10 70	12 48	61
62	1 78	2 13	2 83	3 53	5 28	7 03	8 78	10 53	12 28	62
63	1 75	2 10	2 79	3 47	5 20	6 92	8 64	10 36	12 09	63
64	1 72	2 06	2 74	3 42	5 12	6 81	8 51	10 20	11 90	64
65	1 70	2 03	2 70	3 37	5 04	6 71	8 38	10 05	11 71	65
66	1 67	2 00	2 66	3 32	4 96	6 61	8 25	9 89	11 54	66
67	1 65	1 97	2 62	3 27	4 89	6 51	8 13	9 75	11 37	67
68	1 62	1 94	2 58	3 22	4 82	6 41	8 01	9 60	11 20	68
69	1 60	1 92	2 55	3 17	4 75	6 32	7 89	9 46	11 04	69
70	1 58	1 89	2 51	3 13	4 68	6 23	7 78	9 33	10 88	70
	50	60	80	100	150	200	250	300	350	

FIGURE 51.—Current-meter rating table.

The velocities corresponding to a range of 3 to 350 revolutions of the rotor within a period of 40 to 70 s are listed in the tables. This range in revolution and time has been found to cover general field requirements. To provide the necessary information for extending a table for the few instances where extensions are required, the equations of the rating table are shown in the spaces provided in the heading. The term, R, in the equations refers to revolutions of the rotor per second. The equation to the left of the figure in parentheses (2.20 in fig. 51) is the equation for velocities less than 2.20 ft/s (0.67 m/s), and the equation to the right is for velocities greater than 2.20 ft/s. The velocity 2.20 ft/s is common to both equations.

It should be noted that the equations given are those of the rating table and not necessarily those of the actual rating. If a rating table already on file matches a rating within close tolerances, that table is selected in preference to preparing a new one. The tolerances are listed below:

Revolutions of rotor per second (R)	*Tolerance, in percent*
Less than 1.0	1.0
1.0 and greater	.5

Because of the rigid control in the manufacture of the Price meter, virtually identical meters are now being produced, and their rating equations tend to be identical. Therefore, the U.S. Geological Survey now feels no need to calibrate the meters individually. Instead, an average standard rating is established by calibrating a large number of meters that have been constructed to U.S. Geological Survey specifications, and that rating is then supplied with each meter. To insure that all meters are virtually identical, the dies and fixtures for the construction of Price meters are supplied to the manufacturer.

Price meters that have been first rated using a wading-rod suspension, and then rated using a cable suspension with U.S. Geological Survey Columbus-type weights and hangers, have not shown significant differences in rating. Therefore, no suspension coefficients are needed, and none should be used, if Columbus-type weights and hangers are properly used. Tests that compared meters were discussed on pages 89–90. In those tests Columbus-type weights were used with all meters. The close agreement of results for all meters indicate that no suspension coefficients are required when horizontal-axis current meters are used with Columbus-type weights.

The rating of the optical current meter is relatively simple. Its operation is based on precise mathematical principles, and, given an accurate tachometer, meter accuracy is dependent only on the configuration of the cam that oscillates the mirror. A master cam is used in the manufacture of the individual meter cams. The meter is rated by observation of a long endless belt driven at constant speed. That known belt speed is checked against the speed computed by multiplying the height of the meter above the belt by the tachometer reading. If the comparison of known and computed speeds shows a lack of agreement, the tachometer scaling is changed to bring about agreement.

SOUNDING EQUIPMENT

Sounding (determination of depth) is usually done mechanically, the equipment used being dependent on the type of measurement being made. Depth and position in the vertical are measured by a rigid rod or by use of a sounding weight suspended from a cable. The cable length is controlled either by a reel or by a handline. A sonic sounder is also available, but it is usually used in conjunction with a reel and a sounding weight.

Sounding equipment used by the U.S. Geological Survey is described in the following categories: wading rods, sounding weights, sounding reels, handlines, and sonic sounder.

WADING RODS

The two types of wading rods commonly used are the top-setting rod and the round rod. The top-setting rod is preferred because of the convenience in setting the meter at the proper depth and because the hydrographer can keep his hands dry in the process. The round rod can be used in making ice measurements as well as wading measurements and has the advantage that it can be disassembled to 1-ft (0.3-m) lengths for storing and transporting.

The top-setting wading rod has a 1/2-in (12.7-mm) hexagonal main rod for measuring depth and a 3/8-in (9.5-mm) diameter round rod for setting the position of the current meter (fig. 52).

The rod is placed in the stream so the base plate rests on the streambed, and the depth of water is read on the graduated main rod. When the setting rod is adjusted to read the depth of water, the meter is positioned automatically for the 0.6-depth method. (See p. 134.)The 0.6-depth setting might also be described as the 0.4-depth position measured up from the streambed. When the depth of water is divided by 2 and this new value is set, the meter will be at the 0.2 depth position measured up from the streambed. When the depth of water is multiplied by 2 and this value is set, the meter will be at the 0.8-depth position measured up from the streambed. These two positions represent the conventional 0.2- and 0.8-depth positions. (See p. 134.)

The round wading rod consists of a base plate, lower section, three or four intermediate sections, sliding support, and a rod end (not essential). The parts are assembled as shown in figure 53. The meter

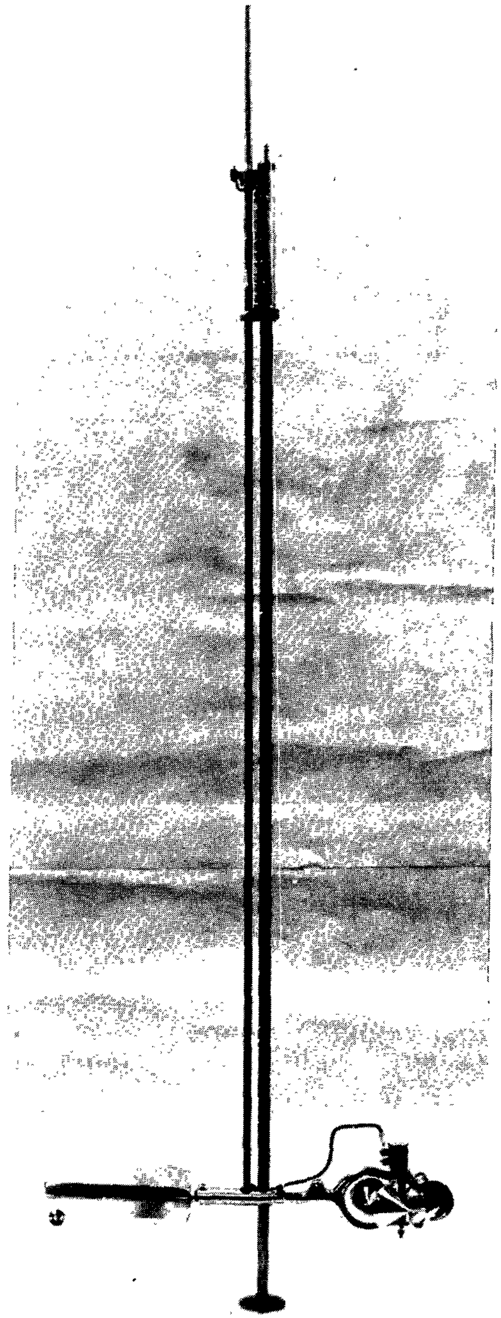


FIGURE 52.—Top-setting wading rod with meter attached.

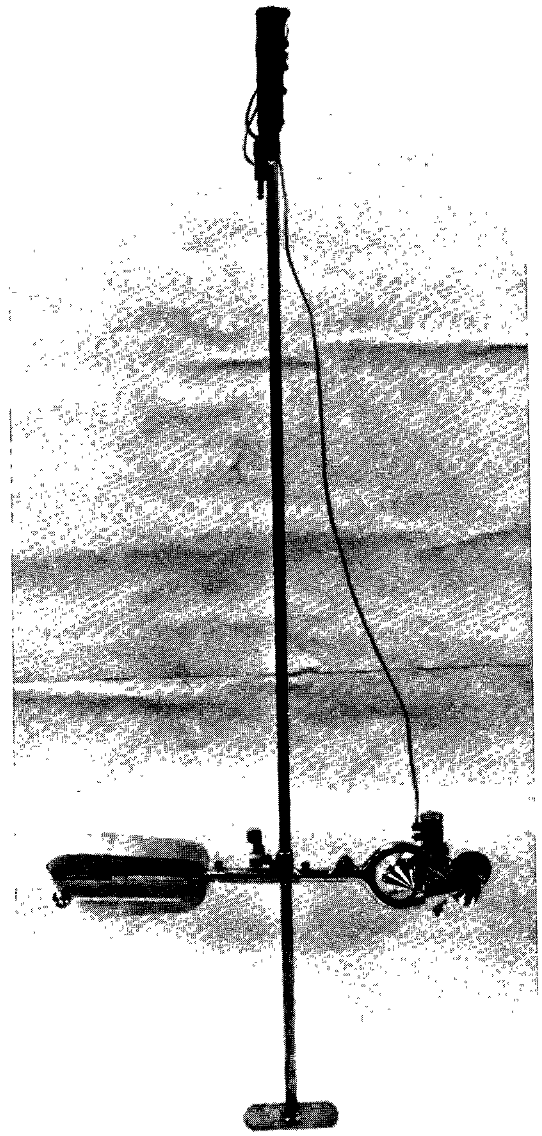


FIGURE 53.—Round wading rod with meter attached.

is mounted on the sliding support and is set at the desired position on the rod by sliding the support.

The round rod is also used in making ice measurements. Intermediate sections of the round rod are screwed together to make an ice

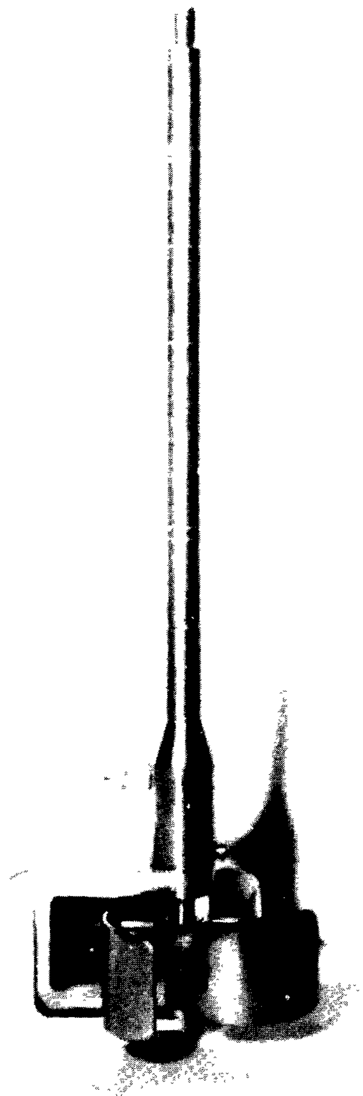


FIGURE 54.—Lower section of ice rod for use with vane ice meter.

rod of desired length (fig. 54). The most convenient length for an ice rod is about 3 ft (1 m) longer than the maximum depth of water to be found in a cross section. About 12 ft (4 m) is the maximum practical length for an ice rod; depths greater than 10 ft (3 m) are usually measured with a sounding weight and reel. The base plate, sliding support, and lower section are not used on an ice rod. Instead, a special lower section is screwed directly into the top of the contact chamber of the vane ice meter. (See fig. 54.) If a Price meter is used under ice cover, another special lower section is used to hold the meter by means of the hanger screw. (See fig. 55.) All lower sections for ice rods are now made so that the center of the vanes or cups is at the 0-ft point on the rod.

SOUNDING WEIGHTS AND ACCESSORIES

If a stream is too deep or too swift to wade, the current meter is suspended in the water by cable from a boat, bridge, or cableway. A sounding weight is suspended below the current meter to keep it

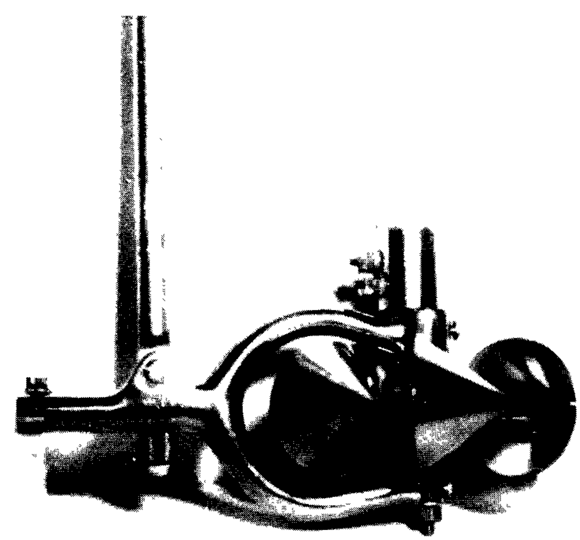


FIGURE 55.—Lower section of ice rod for use with Price meter.

stationary in the water. The weight also prevents damage to the meter when the assembly is lowered to the streambed.

The sounding weights now used in the U.S.A. are the Columbus weights, commonly called the C type. (See fig. 56.) The weights are streamlined to offer minimum resistance to flowing water. Each weight has a vertical slot and a drilled horizontal hole to accommodate a weight hanger and securing pin.

The weight hanger (fig. 57) is attached to the end of the sounding line by a connector. The current meter is attached beneath the connector, and the sounding weight is attached to the lower end of the hanger by means of the hanger pin.

In addition to the weights shown in figure 56, weights of 150, 200 and 300 lb (68, 91, and 136 kg) are used for measuring the discharge of deep, swift rivers. The sounding-weight hangers shown in figure 57 are designed to accommodate the weights of the various sizes. The height of the meter rotor above the bottom of the sounding weight must be considered in calculations to position the meter for velocity observations at various percentages of the stream depth.

SOUNDING REELS

A sounding reel has a drum for winding the sounding cable, a crank and ratchet assembly for raising and lowering the weight or holding it in any desired position, and a depth indicator. The U.S. Geological Survey has five types or sizes of sounding reel in common use, the choice of reel being dependent on the depth of water to be measured and on the weight required for sounding. The lightest of the reels, known as the Canfield reel (fig. 58), can be used with either single- or two-conductor cable; the other four reels use two-conductor cable, whose diameter ranges from 0.084 in to 0.125 in (2.13 to 3.18 mm),

FIGURE 56.—Columbus 15-, 30-, 50-, 75-, and 100-lb sounding weights.

depending on the weight to be handled. The three smaller reels have a hand crank for raising and lowering the meter and weight (fig. 58);

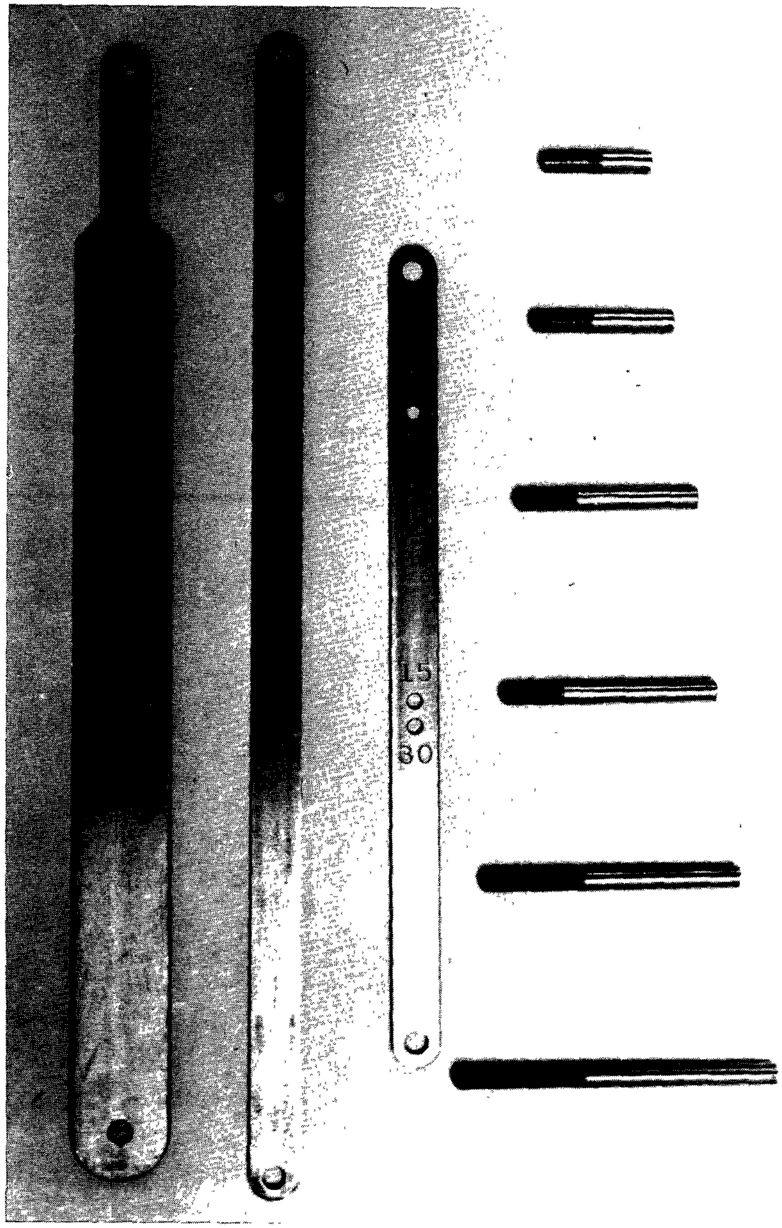


FIGURE 57.—Sounding-weight hangers and hanger pins.

the largest of the five reels is operated by a battery-powered unit but has a handcrank for emergency use; the second largest reel (fig. 59) may be operated either by a hand crank or a power unit. Specially designed connectors are used to join the end of the reel cable to the sounding-weight hanger.

The two smaller reels (Canfield and A-pack reels) are equipped with counters for indicating depth (fig. 58); the three larger reels are equipped with computing depth indicators (figs. 59 and 60). On the computing depth indicator, depth is indicated by a pointer. Tens of feet are read on a numbered dial through an aperture near the top of the main dial. The main dial also has a graduated spiral to indicate directly the 0.8-depth position (p. 134) for depths up to 30 ft (9.15 m).

HANDLINES

When discharge measurements that are to be made from a bridge require light sounding weights—15 or 30 lb (6.8 or 13.6 kg)—the weight and meter are often suspended on a handline (figs. 61 and 62). Handlines can also be used from cable cars, but they seldom are because a sounding reel mounted on the cable car is much more convenient to use.

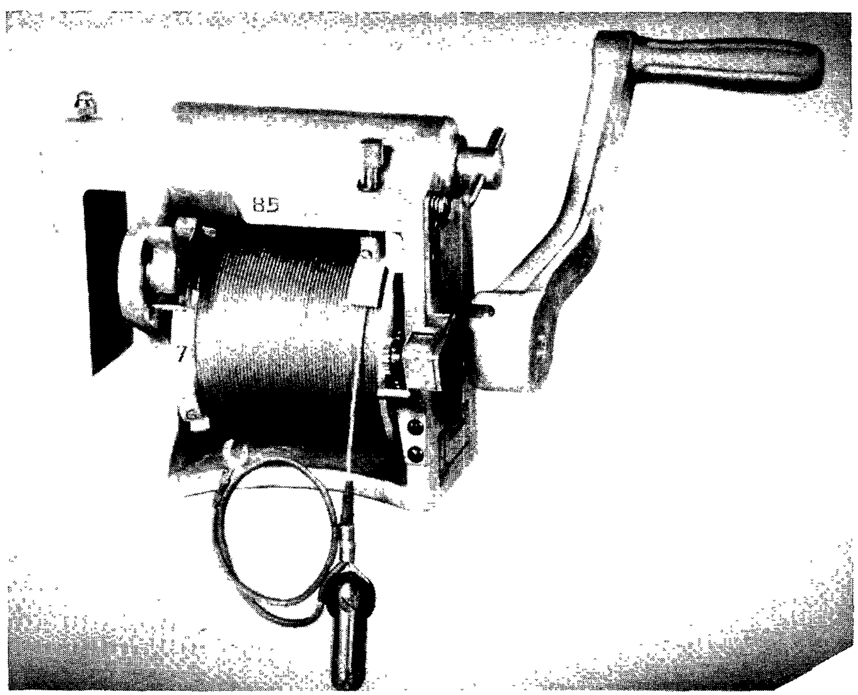


FIGURE 58.—Canfield reel.

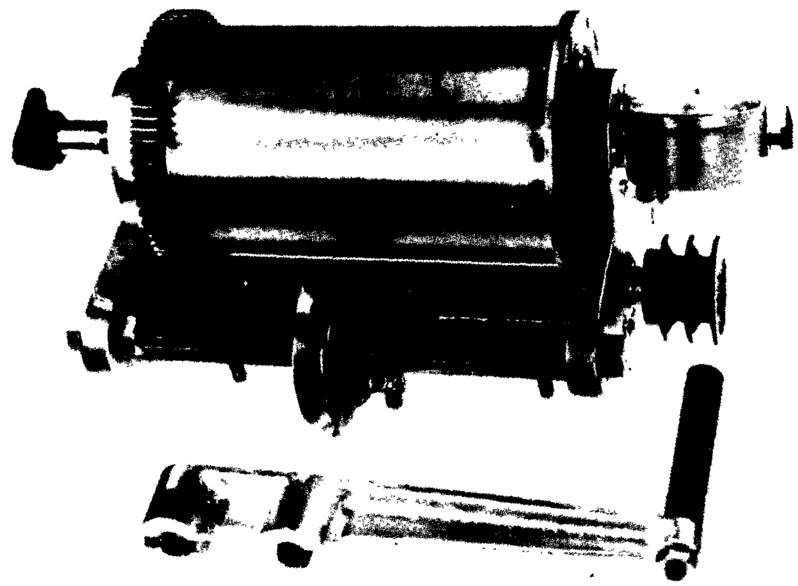


FIGURE 59.—B–56 reel.

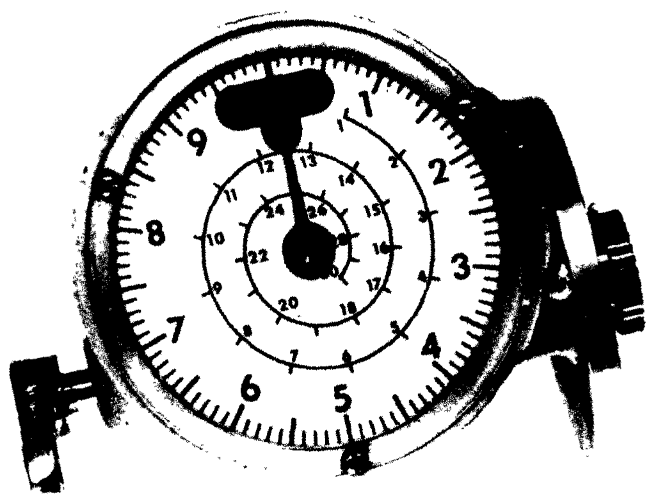


FIGURE 60.—Computing depth indicator.

The handline is made up of two separate cables that are electrically connected at a reel (fig. 63). The upper or hand cable is that part of the handline that is used above the water surface. It is a heavy-duty two-conductor electric cable, whose thick rubber protective covering makes the cable comfortable to handle. At its upper end is a connection for the headphone. The lower or sounding cable is a light reverse-lay steel cable with an insulated core. A connector joins the lower end of the sounding cable to the hanger that is used as a mount for the current meter and sounding weight. Sounding cable in excess of the length needed to sound the stream being measured is wound on

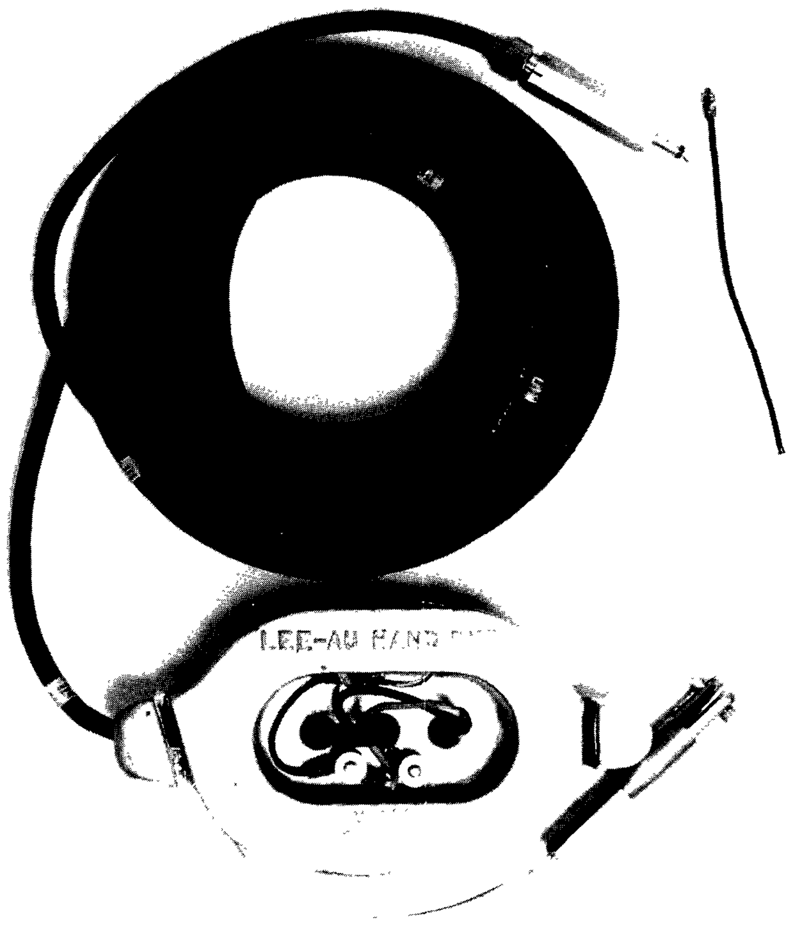


FIGURE 61.—Handline.

the reel. The sounding cable is tagged at convenient intervals with streamers of different colored binding tapes, each colored streamer being a known distance above the current-meter rotor. The use of these tags in determining depth is described on page 150.

The advantages of the handline are ease in assembling the equipment for a discharge measurement and relative ease in making discharge measurements from certain types of bridges, particularly truss bridges that do not have cantilevered sidewalks. The disadvantages of the handline are a lesser degree of accuracy in depth determination than that obtained with a sounding reel, more physical exertion is required in making the discharge measurement, and the handline can seldom be used for high-water measurements of large

FIGURE 62.—Handline in use from a bridge.

streams because of the heavy sounding weights needed for such measurements.

SONIC SOUNDER

A commercial, compact, portable sonic sounder has been adopted by the U.S. Geological Survey to measure stream depth. (See figs. 64 and 65.)

The sounder is powered by either a 6- or 12-volt storage battery and will operate continuously for 10 hr on a single battery charge. Three recording speeds are available, 36, 90, or 180 in (0.91, 2.29,4.57 m) per hr. Four operating ranges, 0–60, 60–120, 120–180, and 180–240 ft (0–18.3, 18.3–36.6, 36.6–54.9, and 54.9–73.2 m) allow intervals of 60 ft (18.3 m) of depth to be recorded. The sounder is portable, weighing only 46 lb (20.9 kg). The transducer has a narrow beam angle of 6° which minimizes errors on inclined streambeds and allows the hydrographer to work close to piers or other obstructions.

In swift debris-laden streams measurements can be made with this equipment without lowering the meter and weight to the streambed. As soon as the weight is in the water, the depth will be recorded. The meter can then be set at the 0.2 depth or just below the water surface for a velocity observation. The observed velocity can be converted to mean velocity in the vertical by applying an appropriate coefficient. (See p. 135–137.)

FIGURE 63.—Handline reels; Lee-Au (top) and Morgan (bottom).

Temperature change affects the sound-propagation velocity, but error from that source is limited to about ±2 percent in fresh water. That error can be eliminated completely by adjusting the sounder to

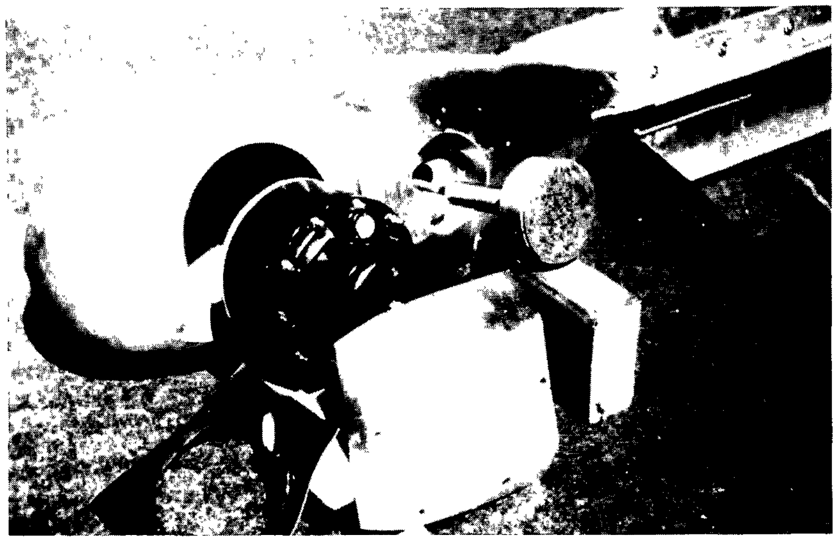


FIGURE 64.—Sounding weight with compass and sonic transducer ready for assembly.

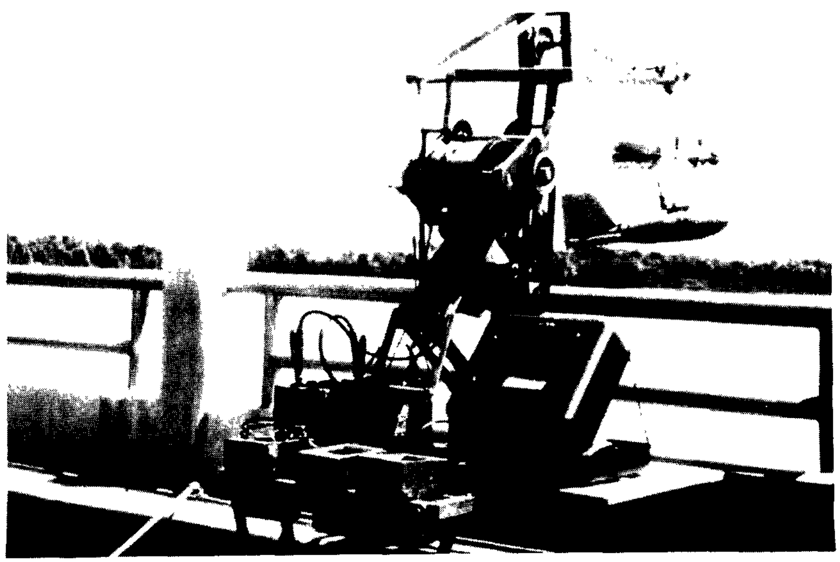


FIGURE 65.—Sonic measuring assembly.

read correctly at an appropriate average depth determined by other means.

WIDTH-MEASURING EQUIPMENT

The distance to any point in a cross section is measured from an initial point on the bank. Cableways and bridges used regularly for making discharge measurements are commonly marked at 2-, 5-, 10-, or 20-ft (0.61-, 1.52-, 3.05-, or 6.10-m) intervals by paint marks. Distance between markings is estimated, or measured with a rule or pocket tape.

For measurements made by wading, from boats, or from unmarked bridges, steel or metallic tapes or tag lines are used. Tag lines are made of 1/32-, 1/16-, 3/32-, or 1/8-in (0.79-, 1.59-, 2.38-, or 3.18-mm)-diameter galvanized steel aircraft cord having solder beads at measured intervals to indicate distances. The standard arrangement of solder beads or tags used by the U.S. Geological Survey is as follows:

- one tag every 2 ft for the first 50 ft of tag line;
- one tag every 5 ft for stations between 50 and 150 ft on the tag line;
- one tag every 10 ft for stations between 150 ft and the end of the tag line.

For identifying the stationing of the tags, an additional tag (total of two tags) is used at stations 0, 10, 20, 30, 40, 50, 150, 250, 350, and 450 ft. Two additional tags (total of three tags) are used at stations 100, 200, 300, 400, and 500 ft.

The standard lengths of tag line are 300, 400 and 500 ft (91.4, 122, and 152m), but other sizes are available.

Three types of tag-line reels in use are Lee-Au, Pakron, and Columbus type A (fig. 66). Larger reels designed particularly for use with boats are described on page 120. It is practically impossible to string a tag line for discharge measurements from a boat when the width of the stream is greater than 2,500 ft (750 m). The methods used to determine width at such sites are described on pages 156–157.

EQUIPMENT ASSEMBLIES

Special equipment is necessary for each type of current-meter measurement. The meters, weights, and reels used have already been described. The additional equipment needed is described in this section.

The special equipment assemblies have been divided into five basic groups: cableway, bridge, boat, ice, and velocity-azimuth-depth assembly (VADA) equipment.

CABLEWAY EQUIPMENT

The cableway provides a track for the operation of a cable car from which the hydrographer makes a current-meter measurement. Cable cars also support the sounding reel and other necessary equipment. Both sitdown and standup types of cable cars are used in stream gaging. (See figs. 67 and 68). Pierce (1947) describes plans for both types. Normally, sitdown cars are used for cableway spans less than 400 ft (122 m) and for those spans where the lighter sounding weights are used. The standup car is used on the longer spans and where heavy sounding weights are needed.

The cars are moved from one point to another on the cableway by means of cable-car pullers. (See fig. 69.) The standard car puller is a cast aluminum piece with a snub attached to act as a brake. The snub, usually four-ply belting, is placed between one of the car sheaves and the cable to prevent movement of the car along the cable. A second-type puller is used when a car is equipped with a follower brake (fig. 69). A third type, the Colorado River cable-car puller, is the same in principle as the puller used on cars equipped with a follower brake.

Power-operated cable cars are available for extremely long spans or other special situations. (See fig. 70.)

Sitdown cable cars have a variety of means of supporting the sounding reel. A-pack and Canfield reels are designed to clamp on the side of the car (fig. 71). Permanent or portable reel seats are attached to the cable cars for larger reels. (See figs. 67 and 72.)

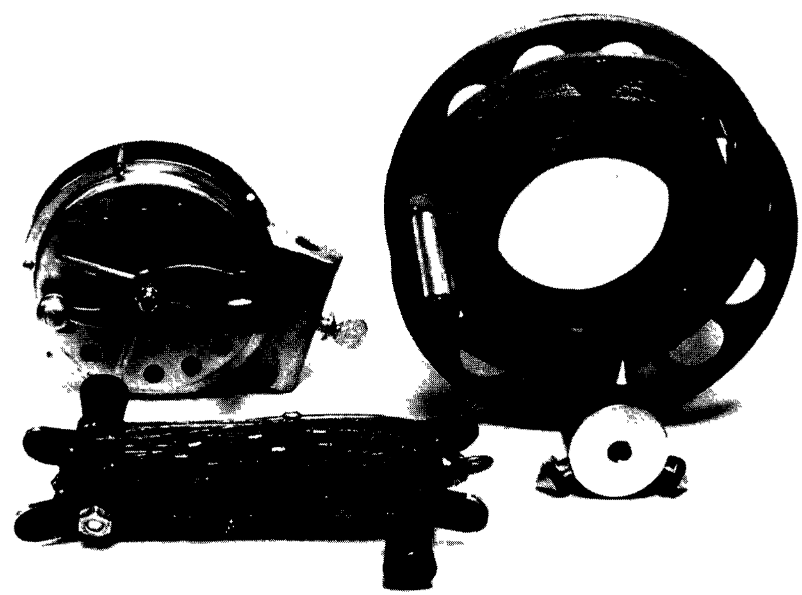


FIGURE 66 —Tag-line reels: top left, Pakron; top right, Lee-Au with removable hub in front; bottom, Columbus type A.

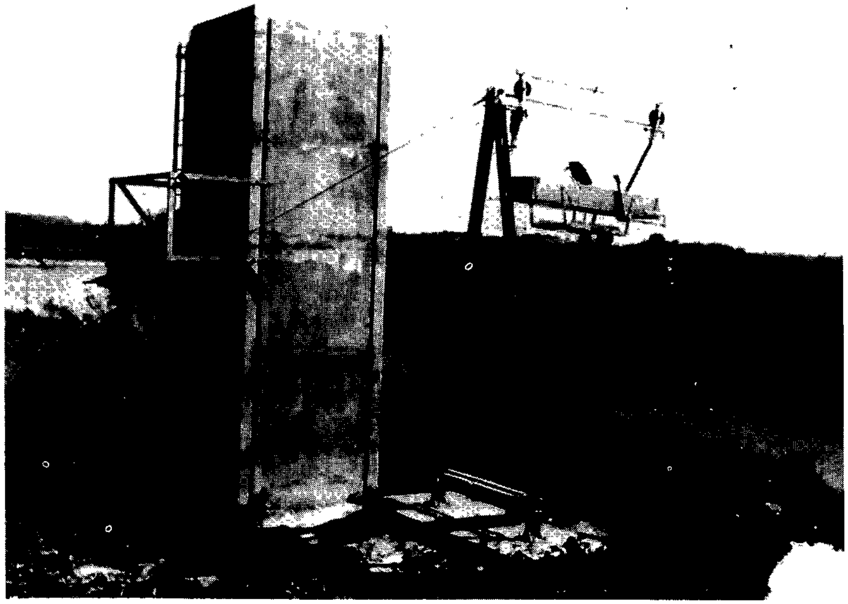


FIGURE 67.—Sitdown cable car.

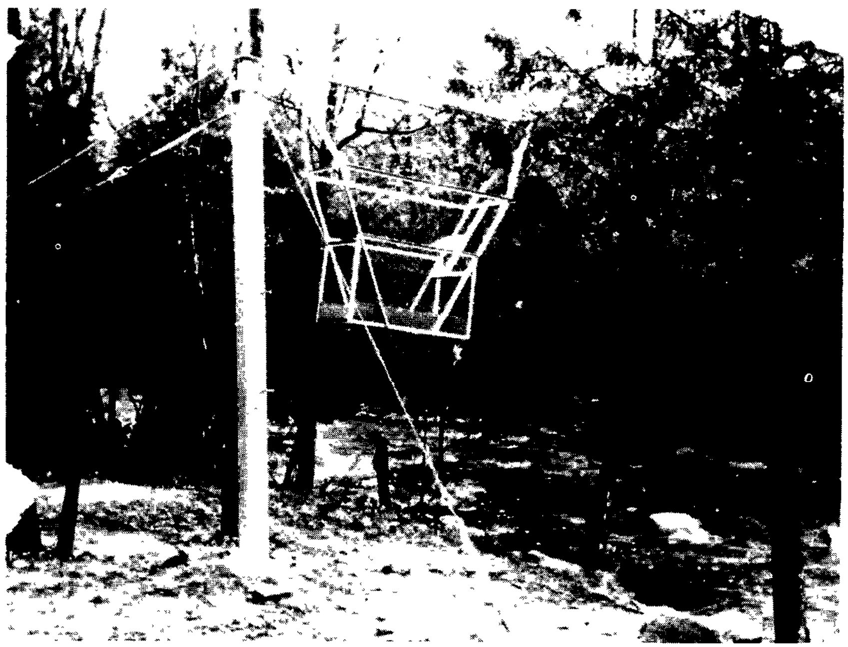


FIGURE 68.—Standup cable car.

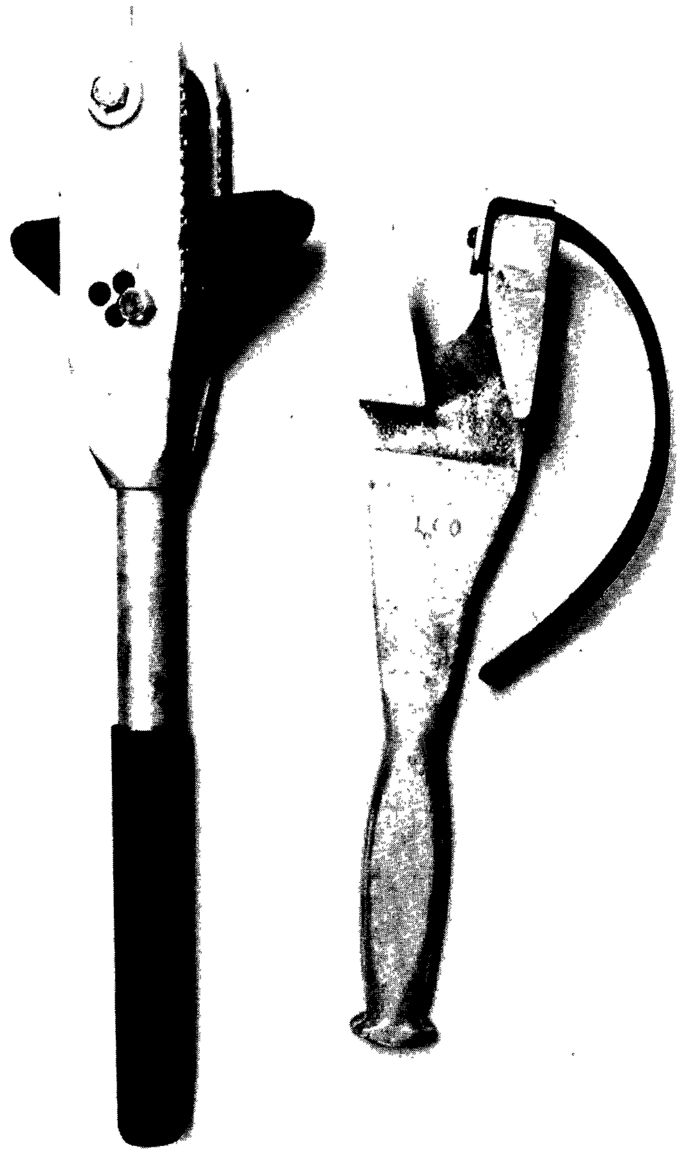


FIGURE 69.—Cable-car puller for follower-brake cable cars, left; for standard cars, right.

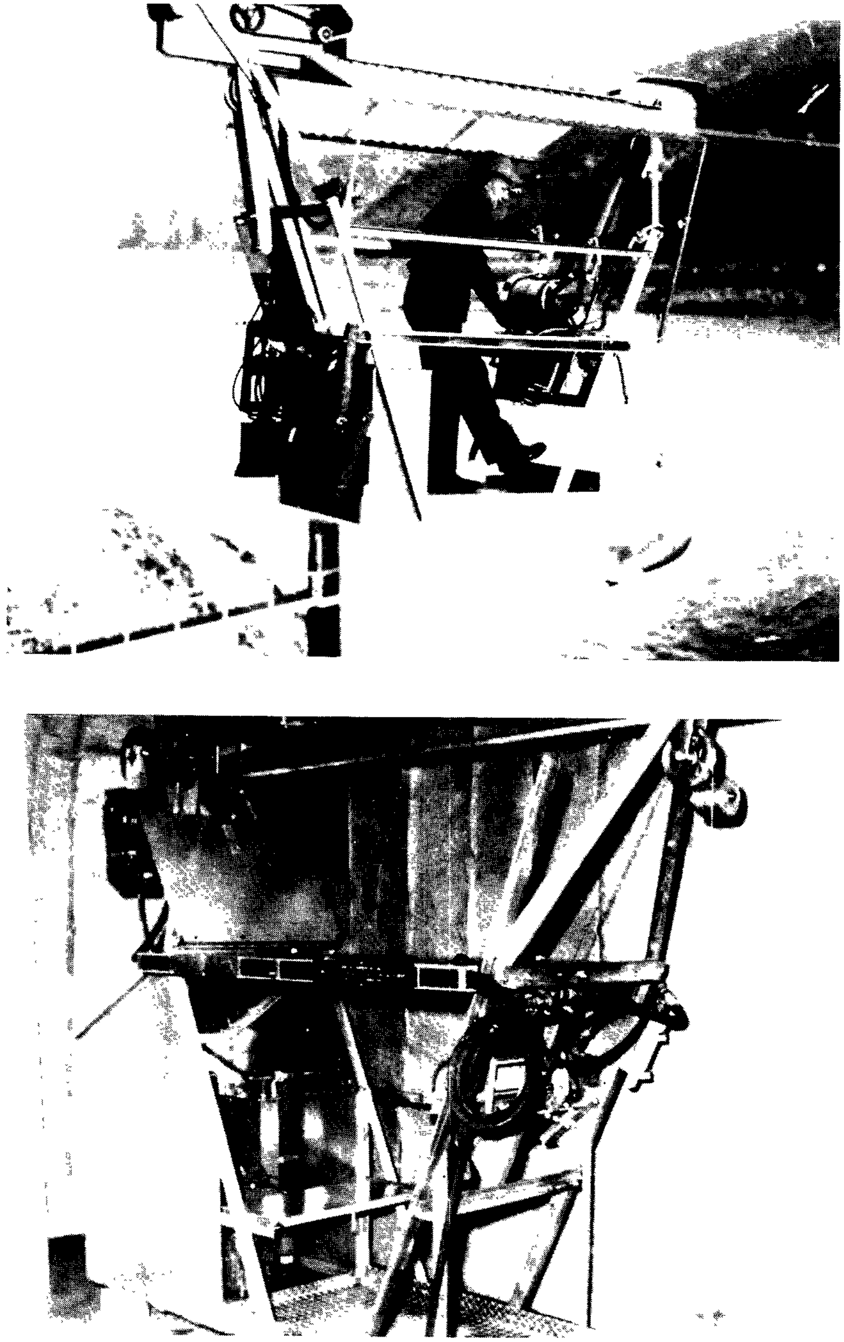


FIGURE 70.—Power-operated cable cars. (*A*) battery-powered car; (*B*) gasoline-powered car.

Standup cable cars have reel seats attached to the structural members of the car (fig. 68). A sheave attached to the structural members carries the sounding line so that the sounding weight and current meter will clear the bottom of the car. Power reels can also be used on standup cable cars.

Carrier cableways are sometimes used on the smaller streams for measuring discharge as well as for sediment sampling. They are used in areas where it is impossible to wade, where no bridges are avail-

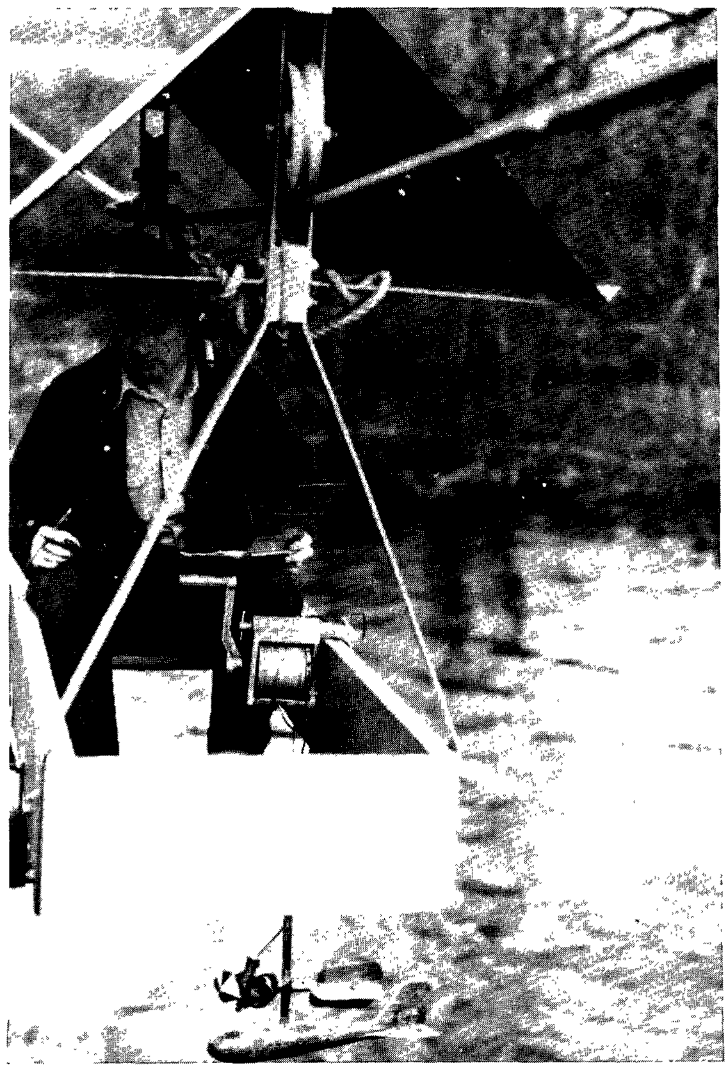


FIGURE 71.—Sitdown cable car with Canfield reel clamped to side of car.

able, and where it has been impractical to build a complete cableway. The assembly is operated from the shore (fig. 73). Carrier cables are

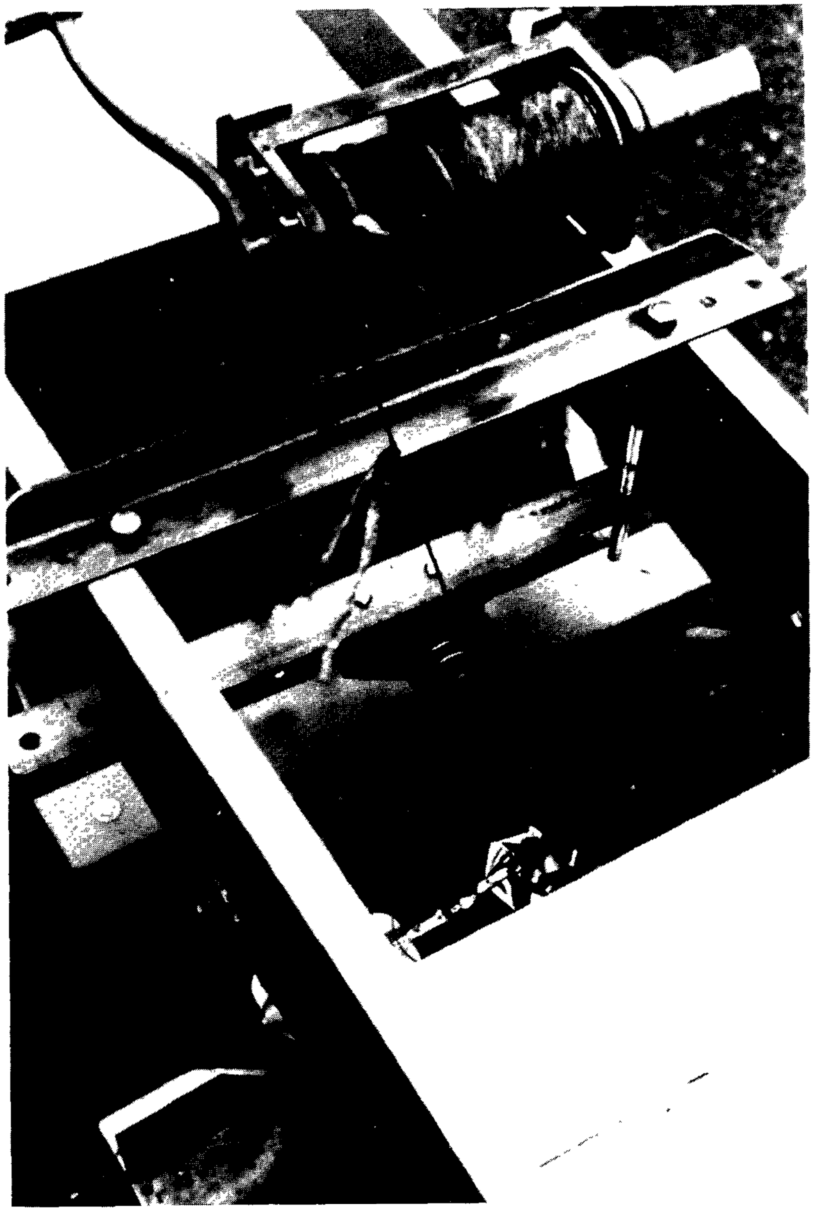


FIGURE 72.—Portable reel seat on sitdown-type cable car. (Note tags on sounding cable.)

more widely used in Europe, particularly in the United Kingdom, than they are in the U.S.A.

BRIDGE EQUIPMENT

When one measures from a bridge, the meter and sounding weight can be supported by a handline, or by a sounding reel mounted on a crane, or by a bridge board. The handline has been described on pages 104–108.

Two types of hand-operated portable cranes are the type A (figs. 74 and 75) for use with weights up to 100 lb (45.4 kg) and the type E for heavier weights.

All cranes are designed so that the superstructure can be tilted

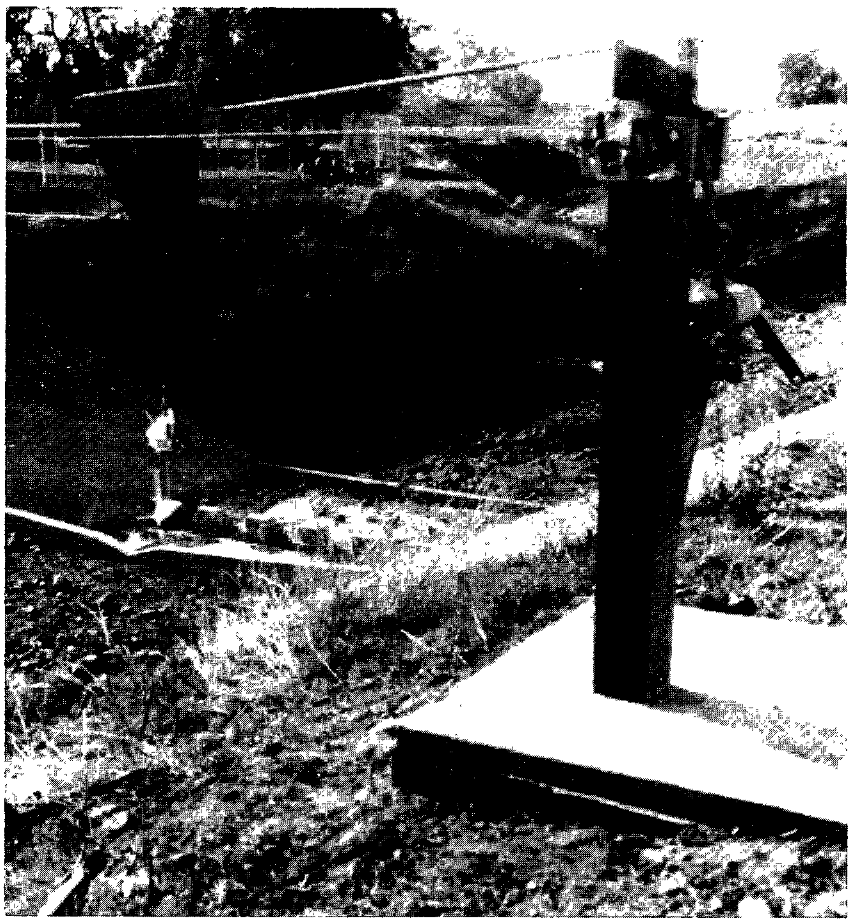


FIGURE 73.—Carrier (bank-operated) cableway. Two reels are used; one moves the meter assembly laterally, and the other moves the current meter vertically.

forward over the bridge rail far enough for the meter and weight to clear most rails. Where bridge members obstruct the movement of the crane from one vertical to the next, the weight and meter can be brought up, and the superstructure can be tilted back to pass by the obstruction. (See figs. 74 and 75.)

Cast-iron counterweights weighing 60 lb (27.2 kg) each are used with four-wheel-base cranes. (See fig. 75.) The number of such weights needed depends upon the size of the sounding weight being supported, the depth and velocity of the stream, and the amount of debris being carried by the stream.

A protractor is used on cranes to measure the angle the sounding line makes with the vertical when the weight and meter are dragged downstream by the water. The protractor is a graduated circle clamped to an aluminum plate. A plastic tube, partly filled with colored antifreeze, fitted in a groove between the graduated circle and

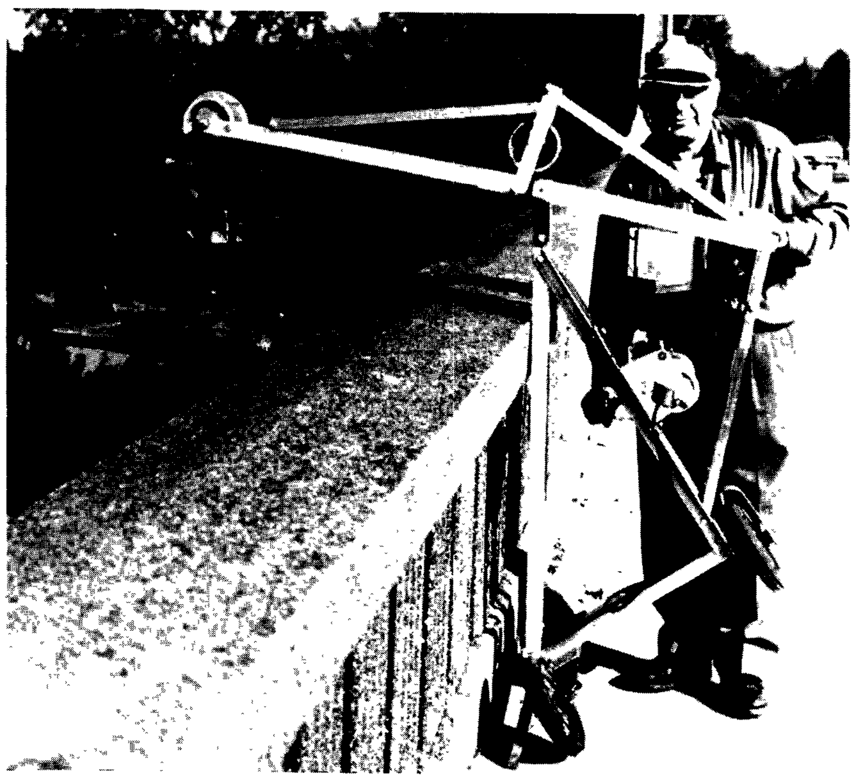


FIGURE 74.—Type-A crane with three-wheel base. (During soundings and velocity observations the crane is tilted against the bridge rail. An A–55 reel is mounted on the crane.)

the plate, is the protractor index. A stainless-steel rod is attached to the lower end of the plate to ride against the downstream side of the sounding cable. The protractor will measure vertical angles from −25° to +90°. The cranes in figures 74 and 75 are equipped with protractors at the outer end of the boom.

Bridge boards may be used with an A-pack or A-55 sounding reel and with weights up to 50 lb (22.7 kg). A bridge board is usually a plank about 6 to 8 ft (1.8 to 2.4 m) long with a sheave at one end over which the meter cable passes and a reel seat near the other end. The board is placed on the bridge rail so that the force exerted by the sounding weight suspended from the reel cable is counterbalanced by the weight of the sounding reel (fig. 76). The bridge board may be hinged near the middle to allow one end to be placed on the sidewalk or roadway.

Many special arrangements for measuring from bridges have been

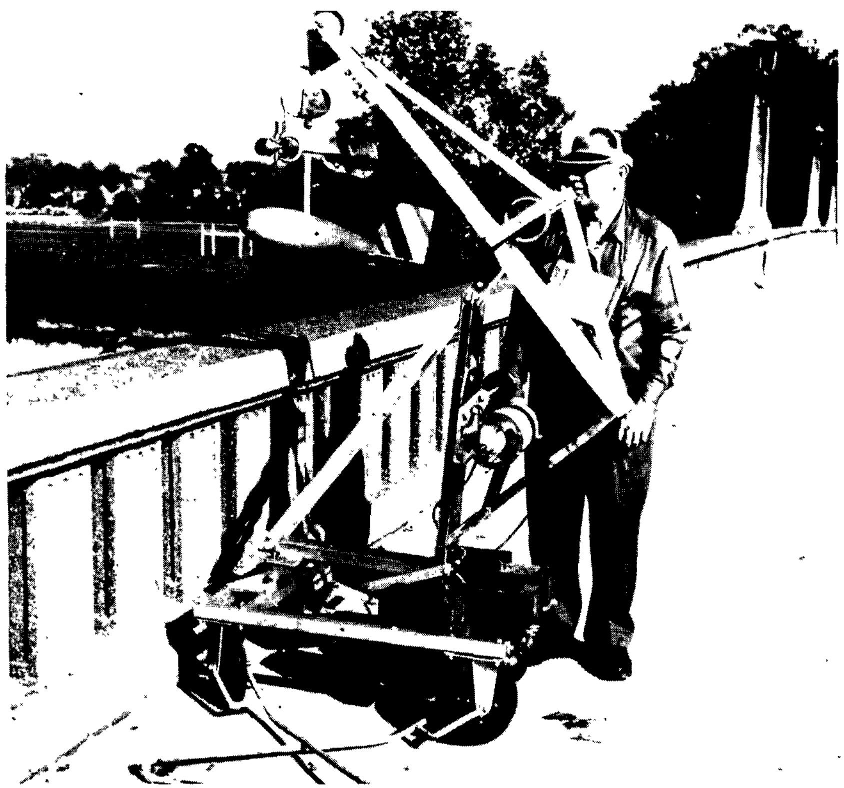


FIGURE 75.—Type-A crane with four-wheel base with boom in retracted position. (A B–56 reel is mounted on crane. Note fluid protractor on outer end of boom.)

devised to suit particular purposes. Truck-mounted cranes are often used for measuring from bridges over the larger rivers. (See fig. 77.) Monorail stream-gaging cars have been developed for large rivers. The car is suspended from the substructure of the bridge by means of an I-beam. The car is attached to the I-beam track by trolleys and is propelled by a forklift motor having a wheel in contact with the bottom of the beam. The drive mechanism and sounding equipment are powered by a 430-ampere-hour, 450-lb (204-kg), 12-volt battery.

BOAT EQUIPMENT

Measurements made from boats require special equipment not used for other types of measurement. Extra-large tag-line reels are used on wide streams. Three different tag-line reels are used by the U.S. Geological Survey for boat measurements:

1. A heavy duty horizontal-axis reel without a brake and with a capacity of 2,000 ft (610 m) of ⅛-in (3.18-mm)-diameter cable (fig. 78).

FIGURE 76.—Bridge board in use.

2. A heavy-duty horizontal-axis reel with a brake and with a capacity of 3,000 ft (914 m) of ⅛-in-diameter cable (fig. 79).
3. A vertical-axis reel without a brake and with a capacity of 800 ft (244 m) of ⅛-in-diameter cable (fig. 80).

A utility line of 30 ft (9 m) of 3/32-in (2.38-mm)-diameter cable with a harness snap at one end and a pelican hook at the other is connected to the free end of the boat tag line and fastened around a tree or post, thereby preventing damage to the tag line. After the tag line is strung across the stream, the reel is usually bolted to a plank and chained to a tree. The tag line has station markers at appropriate intervals.

Special equipment is necessary to suspend the meter from the boat when the depths are such that a rod suspension cannot be used. A crosspiece reaching across the boat is clamped to the sides of the boat and a boom attached to the center of the cross piece extends out over the bow. (See fig. 81.) The crosspiece is equipped with a guide sheave and clamp arrangement at each end to attach the boat to the tag line and make it possible to slide the boat along the tag line from one station to the next. A small rope can be attached to these clamps so

FIGURE 77.—Truck-mounted crane used on the Mississippi River.

that in an emergency a tug on the rope will release the boat from the tag line. The crosspiece also has a clamp that prevents lateral movement of the boat along the tag line when readings are being made. The boom consists of two structural aluminum channels, one telescoped within the other to permit adjustments in length. The boom is equipped with a reel plate on one end; on the other end is a sheave over which the meter cable passes. The sheave end of the boom is designed so that by adding a cable clip to the sounding cable, a short distance above the connector, the sheave end of the boom can be retracted when the meter is to be raised out of the water. The raised

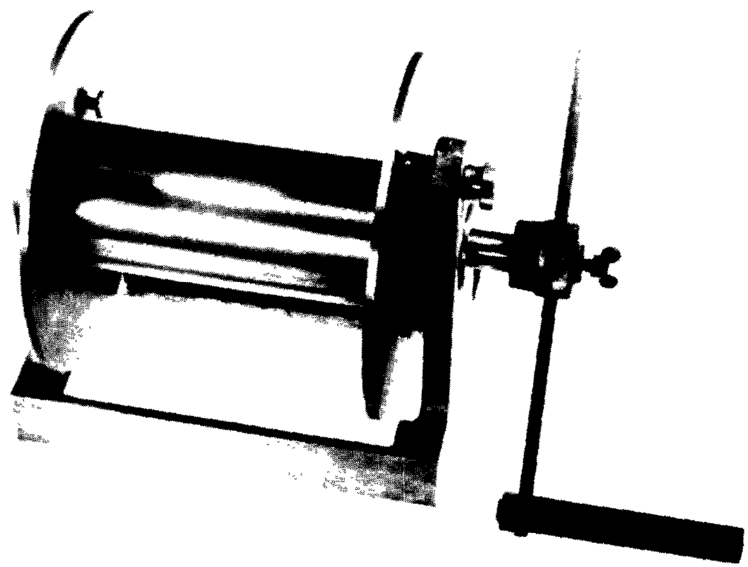


FIGURE 78.—Horizontal-axis boat tag-line reel without a brake.

meter is easy to clean and is in a convenient position when not being operated.

All sounding reels fit the boat boom except the A-pack and the Canfield reels, which can be made to fit by drilling additional holes in the reel plate on the boom.

In addition to the equipment already mentioned, the following items are needed when making boat measurements:

1. A stable boat big enough to support the hydrographers and equipment.
2. A motor that can move the boat with ease against the maximum current in the stream.
3. A pair of oars for standby use.
4. A life preserver for each hydrographer.
5. A bailing device.

Figure 82 shows the equipment assembled in a boat.

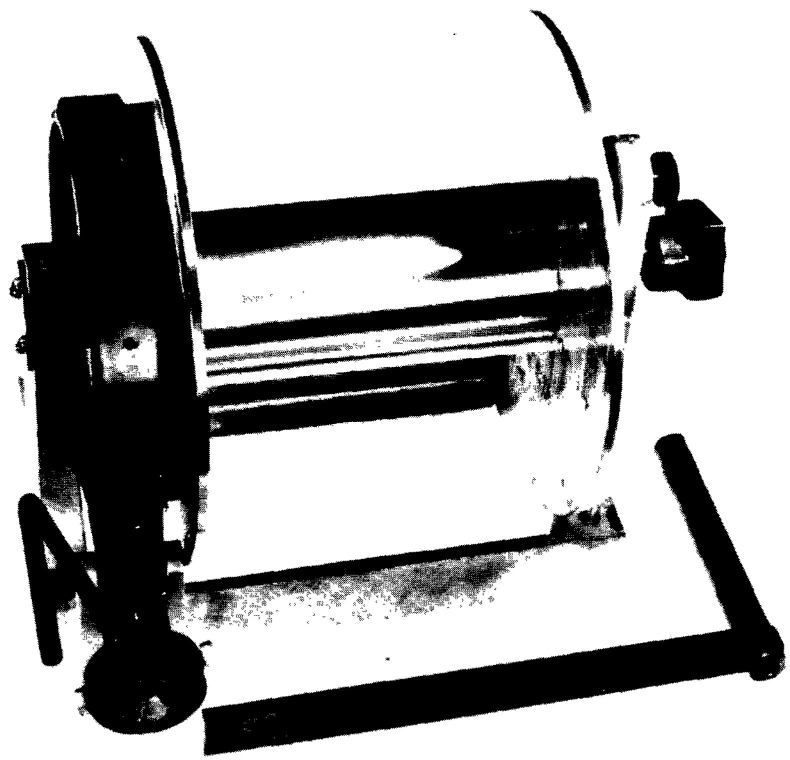


FIGURE 79.—Horizontal-axis boat tag-line reel with a brake.

ICE EQUIPMENT

Current-meter measurements made under ice cover require special equipment for cutting holes in the ice through which to suspend the meter.

Cutting holes through the ice on streams to make discharge measurements has long been a laborious and time-consuming job. The development of power ice drills, however, has eliminated many of the difficulties and has reduced considerably the time required to cut the holes. Holes are often cut with a commercial ice drill that cuts a 6-in (0.15-m) diameter hole (fig. 83). The drill weighs about 30 lb (13.6

FIGURE 80.—Vertical-axis boat tag-line reel (when in use the axis of the reel is vertical).

kg) and under good conditions will cut through 2 ft (0.6 m) of ice in about a minute.

Where it is impractical to use the ice drill, ice chisels are used to cut the holes. Ice chisels used are usually 4 or 4½ ft (about 1.3 m) long and weigh about 12 lb (5.5 kg). The ice chisel is used when first crossing an ice-covered stream to determine whether the ice is strong enough to support the hydrographer. If a solid blow of the chisel blade does not penetrate the ice, it is safe to walk on the ice, providing the ice is in contact with the water.

Some hydrographers supplement the ice chisel with a Swedish ice auger. The cutting blade of this auger is a spadelike tool of hardened steel that cuts a hole 6–8 in (0.15–0.20 m) in diameter. The auger is operated by turning a bracelike arrangement on top of the shaft.

When holes are cut in the ice the water, which is usually under pressure because of the weight of the ice, rises in the hole. To determine the effective depth of the stream (p. 153–154), ice-measuring sticks are used to measure the distance from the water surface to the

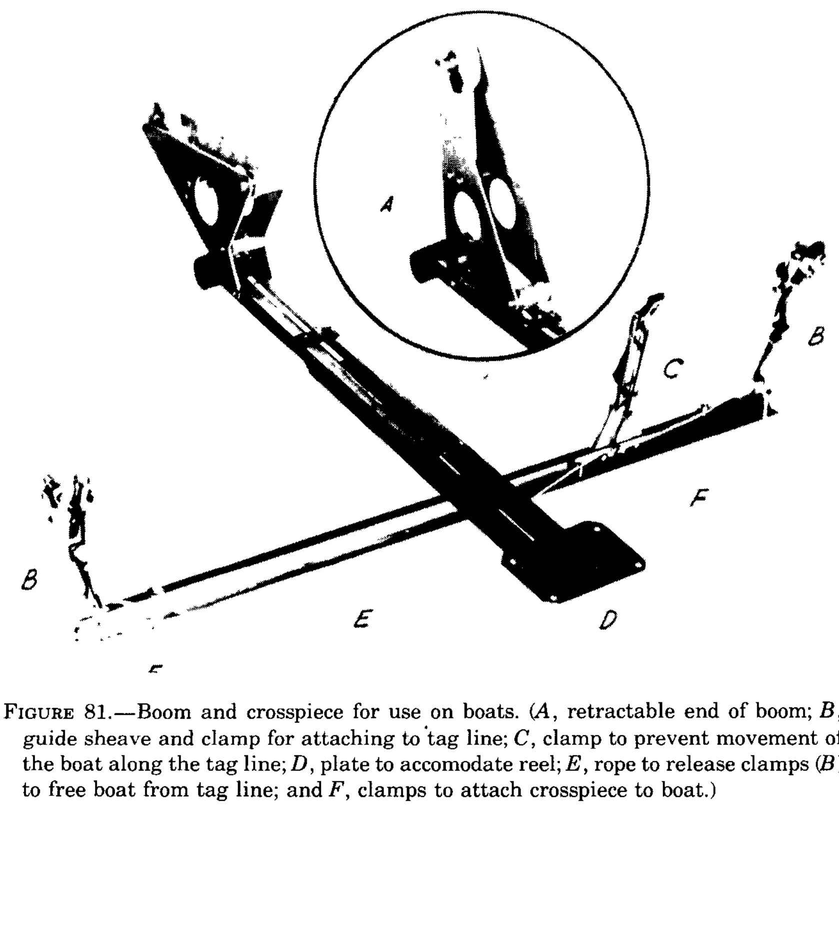


FIGURE 81.—Boom and crosspiece for use on boats. (*A*, retractable end of boom; *B*, guide sheave and clamp for attaching to tag line; *C*, clamp to prevent movement of the boat along the tag line; *D*, plate to accomodate reel; *E*, rope to release clamps (*B*) to free boat from tag line; and *F*, clamps to attach crosspiece to boat.)

bottom of the ice. This is done with a bar about 4 ft long (1.2 m), made of strap steel or wood, which is graduated in feet and tenths of a foot and has an L-shaped projection at the lower end. The horizontal part of the L is held on the underside of the ice, and the depth to that point

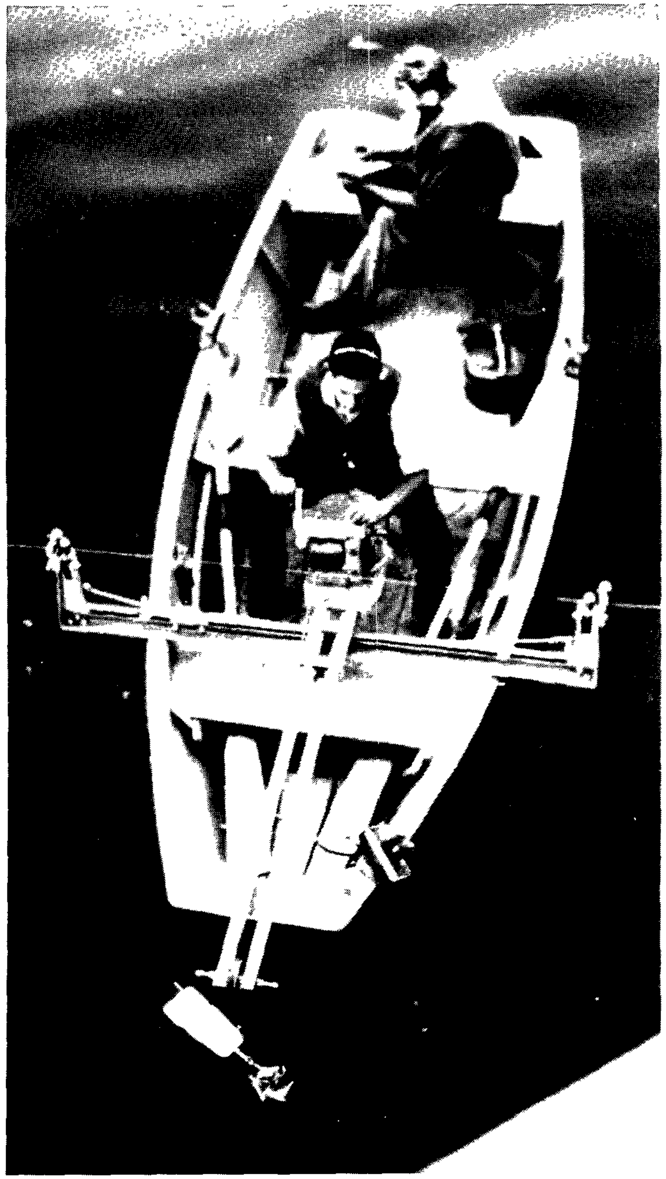


FIGURE 82.—Measuring equipment set up in a boat.

is read at the water surface on the graduated part of the stick. The horizontal part of the L is at least 4 in (0.1 m) long so that it may

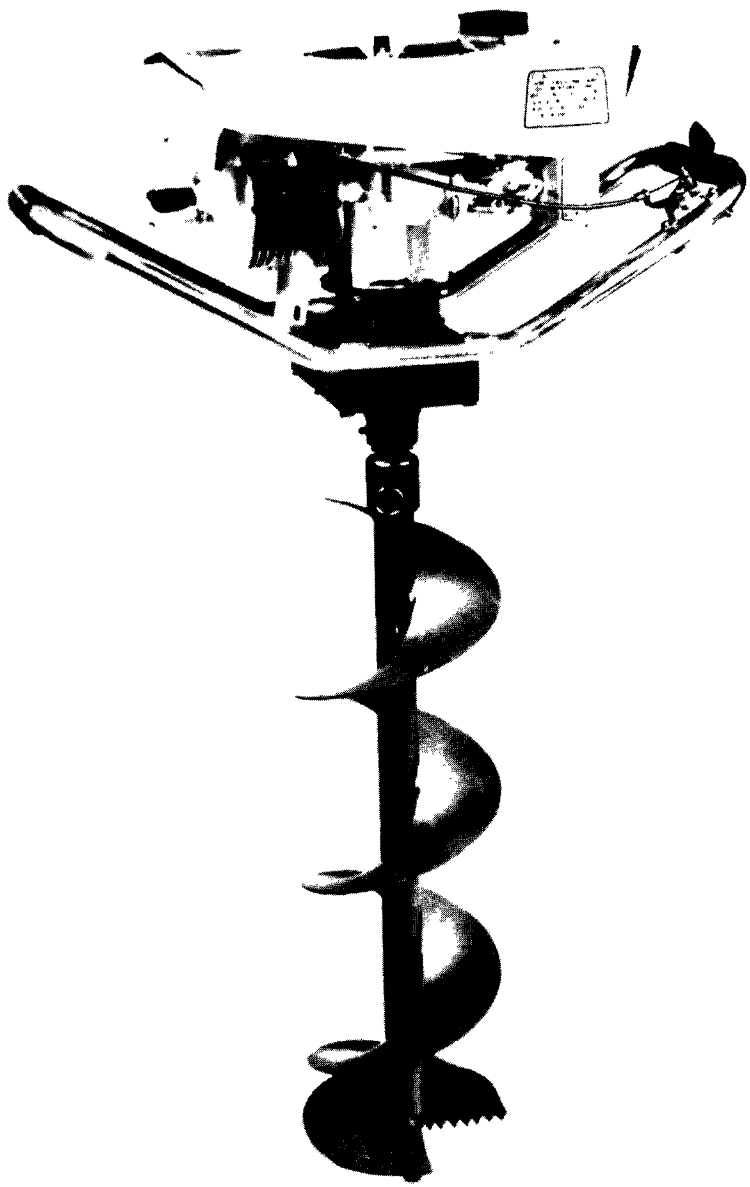


FIGURE 83.—Gasoline-powered ice drill. (Photograph by permission of General Equipment Co.)

extend beyond any irregularities on the underside of the ice.

When the total depth of water under ice cover is greater than 10 or 12 ft (3.0 or 3.6 m), a sounding reel or handline is usually used. The sounding reel is mounted on a collapsible support set on runners. (See fig. 84.)

A special ice-weight assembly is used for sounding under ice be-

FIGURE 84.—Collapsible reel support and ice-weight assembly.

cause a regular sounding weight will not fit through the hole cut by the ice drill (fig. 84). The weights and meter are placed in a framework that will pass through the drilled hole.

VELOCITY-AZIMUTH-DEPTH ASSEMBLY

The velocity-azimuth-depth assembly, commonly called VADA, combines a sonic sounder with a remote-indicating compass and Price current meter to record depth, indicate the direction of flow, and permit observations of velocity at any point.

In figure 85, the azimuth-indicating unit is shown mounted on a four-wheel crane. Incorporated within the remote-indicator box is the battery for the current-meter circuit, the headphone jacks, and the two-conductor jack for the sonic sounder. A switch allows the remote-indicating unit to be used separately or in conjunction with the sonic sounder. The sonic sounder is described on page 108. This assembly is useful in tidal investigations and in other special studies, as well as at regular gaging stations, where it is desirable to determine the

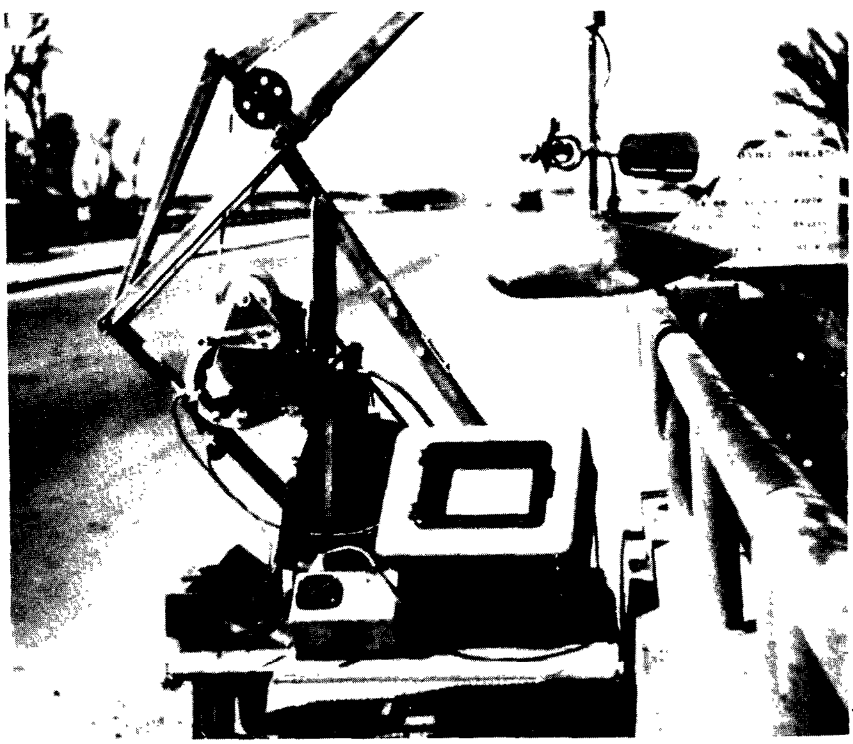


FIGURE 85.—Velocity-azimuth-depth assembly.

direction of flow beneath the water surface, because of the possibility that it may differ from that at the surface.

MISCELLANEOUS EQUIPMENT

Several miscellaneous items that have not been described are necessary when current-meter measurements are made. Three classifications of this equipment are timers, counting equipment, and waders and boots.

In order to determine the velocity at a point with a current meter, it is necessary to count the revolutions of the rotor in a measured interval of time, usually 40–70 s. The velocity is then obtained from the meter-rating table (fig. 51). The time interval is measured to the nearest second with a stopwatch.

The revolutions of the meter rotor during the observation of velocity are counted by an electric circuit that is closed each time the contact wire touches the single or penta eccentric of the current meter. A battery and headphone are part of the electrical circuit, and a click is heard in the headphone each time the contact wire touches. (See fig. 86.) Compact, comfortable hearing-aid phones have been adapted by some to replace the headphones.

A magnetic-switch contact chamber has been developed to replace the contact-wire chamber (p. 87). An automatic electric counter has also been developed for use with the magnetic contact chamber (fig. 86). The counter can register up to 999 and has a reset button. A metal clip is attached to the counter so that it may be easily carried on the hydrographer's belt. The electric counter should not be used with the contact-wire chamber, because at low velocities the contact wire wipes irregularly thereby sending several signals to the counter for each revolution.

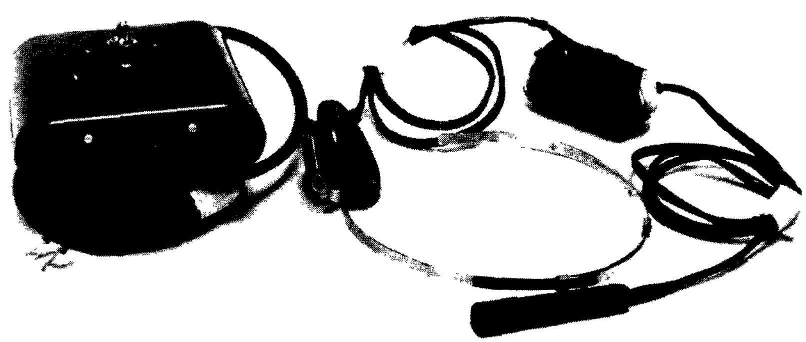


FIGURE 86.—Automatic counter (left) and headphone (right).

Waders or boots are needed when wading measurements are made. Waders should be loose fitting even after allowance has been made for heavy winter clothing. Ice creepers strapped on the shoe of boots or waders should be used on steep or icy streambanks and on rocky or smooth and slippery streambeds. (See fig. 87.)

MEASUREMENT OF VELOCITY

The current meter measures velocity at a point. The method of making discharge measurements at a cross section requires determination of the mean velocity in each of the selected verticals. The mean velocity in a vertical is obtained from velocity observations at many points in that vertical, but it can be approximated by making a few velocity observations and using a known relation between those velocities and the mean in the vertical. The more commonly used methods of determining mean vertical velocity are:

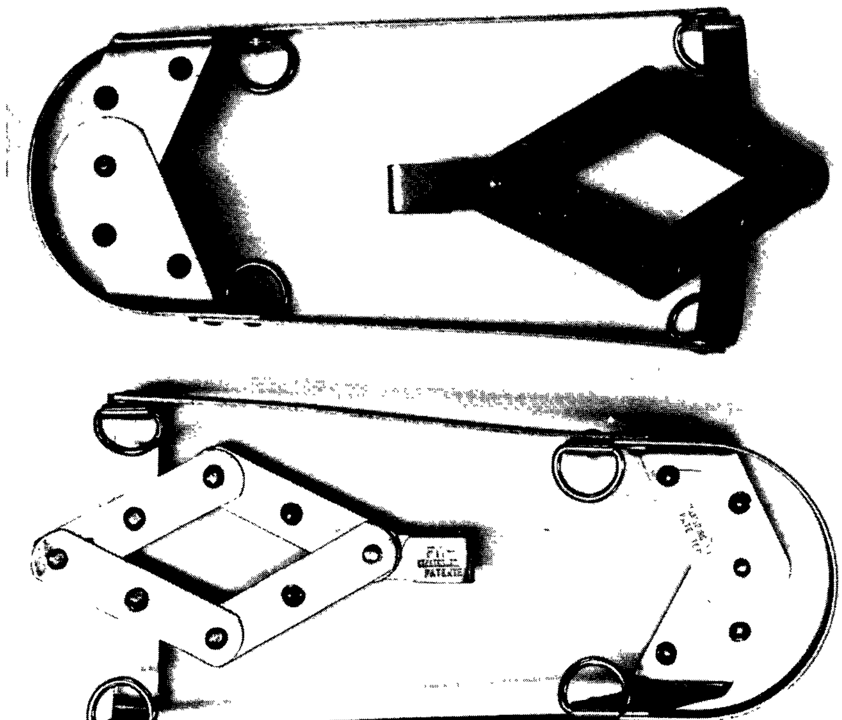


FIGURE 87.—Ice creepers for boots and waders.

1. Vertical-velocity curve.
2. Two-point.
3. Six-tenths-depth.
4. Three-point.
5. Two-tenths depth.
6. Subsurface-velocity.
7. Surface-velocity.
8. Integration.

Less commonly used are the following multipoint methods of determining mean vertical velocity:

9. Five-point.
10. Six-point.

VERTICAL-VELOCITY CURVE METHOD

In the vertical-velocity curve method a series of velocity observations at points well distributed between the water surface and the streambed are made at each of the verticals. If there is considerable curvature in the lower part of the vertical-velocity curve, it is advisable to space the observations more closely in that part of the depth. Normally, the observations are taken at 0.1-depth increments between 0.1 and 0.9 of the depth. Observations are always taken at 0.2, 0.6, and 0.8 of the depth so that the results obtained by the vertical-velocity curve method may be compared with those obtained by the more commonly used methods of velocity observation. Observations are made at least 0.5 ft (0.15m) below the water surface and above the streambed when the Price AA meter or the vane meter is used and are made at least 0.3 ft (0.09 m) from those boundaries when the Price pygmy meter is used; those meters underregister velocity when placed closer to the water surface or streambed.

The vertical-velocity curve for each vertical is based on observed velocities plotted against depth (fig. 88). In order that vertical-velocity curves at different verticals may be readily compared, it is customary to plot depths as proportional parts of the total depth. The mean velocity in the vertical is obtained by measuring the area between the curve and the ordinate axis with a planimeter, or by other means, and dividing the area by the length of the ordinate axis.

The vertical-velocity curve method is valuable in determining coefficients for application to the results obtained by other methods but is not generally adapted to routine discharge measurements because of the extra time required to collect field data and to compute the mean velocity.

Intensive investigation of vertical-velocity curves by Hulsing, Smith, and Cobb (1966) resulted in table 2 which shows average ordinates of the vertical-velocity curve.

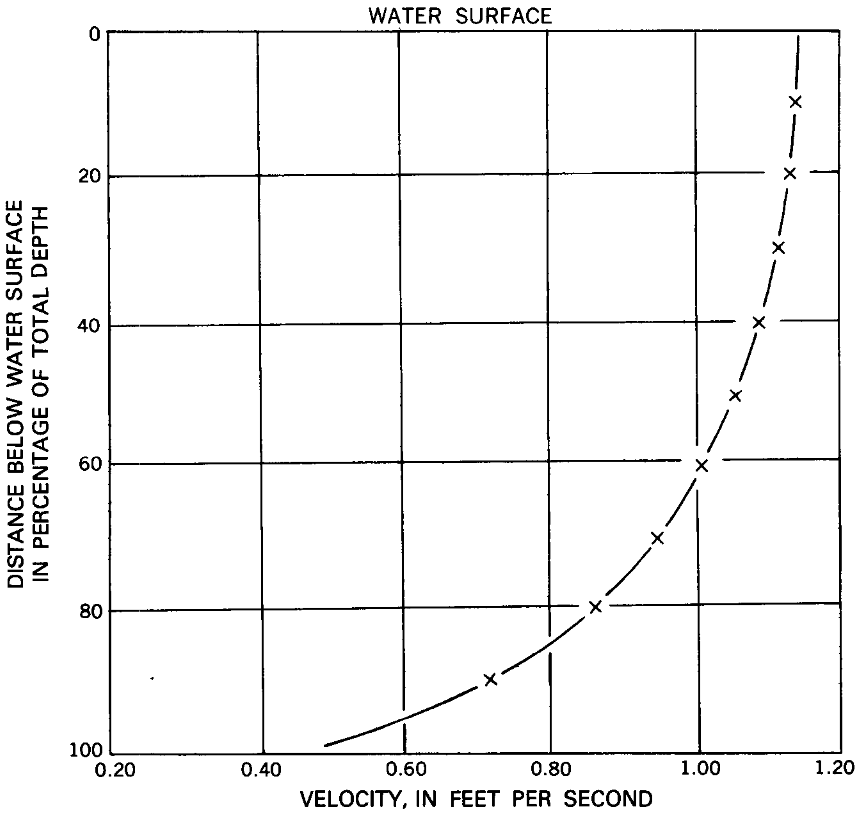


FIGURE 88.—Typical vertical-velocity curve.

TABLE 2.—*Coefficients for standard vertical-velocity curve*

Ratio of observation depth to depth of water	Ratio of point velocity to mean velocity in the vertical
0.05	1.160
.1	1.160
.2	1.149
.3	1.130
.4	1.108
.5	1.067
.6	1.020
.7	.953
.8	.871
.9	.746
.95	.648

TWO-POINT METHOD

In the two-point method of measuring velocities, observations are made in each vertical at 0.2 and 0.8 of the depth below the surface. The average of those two observations is taken as the mean velocity in the vertical. This method is based on many studies involving actual observation and mathematical theory. Experience has shown that this method gives more consistent and accurate results than any of the other methods listed, other than the five-point, six-point, and vertical-velocity curve methods. The two-point method is the one generally used by the U.S. Geological Survey. Table 2 indicates that the two-point method, on the average, gives results that are within 1 percent of the true mean velocity in the vertical.

The two-point method is not used at depths less than 2.5 ft (0.76 m) when measuring with a Price current meter, because the meter would then be too close to the water surface and to the streambed to give dependable results.

The vertical-velocity curve will be distorted by overhanging vegetation that is in contact with the water or by submerged objects, such as large rocks and aquatic growth, if those elements are in close proximity, either in the upstream or downstream direction, to the vertical in which velocity is being measured. Where that occurs the two-point method will not give a reliable value of the mean velocity in the vertical, and an additional velocity observation at 0.6 of the depth should be made. The three observed velocities should then be used in the three-point method (p. 135). A rough test of whether or not the velocities at the 0.2 and 0.8 depths are sufficient for determining mean vertical velocity is given in the following criterion: the 0.2-depth velocity should be greater than the 0.8-depth velocity but less than twice as great.

SIX-TENTHS DEPTH METHOD

In the 0.6-depth method, an observation of velocity made in the vertical at 0.6 of the depth below the surface is used as the mean velocity in the vertical. Actual observation and mathematical theory have shown that the 0.6-depth method gives reliable results. (See table 2.) The U.S. Geological Survey uses the 0.6-depth method under the following conditions:

1. Whenever the depth is between 0.3 ft (0.09 m) and 1.5 ft (0.46 m) and a Price pygmy meter is being used, or between 1.5 ft (0.46 m) and 2.5 ft (0.76 m) and a Price type AA (or type A) meter is being used. (See table 3 for depth and velocity limitations of each meter.)
2. When large amounts of slush ice or debris make it impossible to

observe the velocity accurately at the 0.2 depth. That condition prevents the use of the two-point method.

3. When the distance between the meter and the sounding weight is too great to permit placing the meter at the 0.8 depth. That circumstance prevents the use of the two-point method.
4. When the stage in a stream is changing rapidly and a measurement must be made quickly.

Although, the preceding paragraph states that the 0.6-depth method may be used with a pygmy meter when water depths are as shallow as 0.3 ft (0.09 m), strictly speaking, the 0.6-depth method should not be used when depths are less than 0.75 ft (0.23 m). This follows from the fact that the pygmy meter underregisters when set closer to the streambed than 0.3 ft (p. 132). From a practical standpoint, however, when it is necessary to measure velocities where water depths are as shallow as 0.3 ft, the 0.6-depth method is used. It is recognized, however, that the results obtained in that situation are only approximate values that underestimate the true velocity. Efforts made to date to define shallow-depth coefficients for natural streams have been unsuccessful.

THREE-POINT METHOD

In the three-point method velocities are observed at 0.2, 0.6, and 0.8 of the depth, thereby combining the two-point and 0.6-depth methods. The mean velocity is computed by averaging the 0.2- and 0.8-depth observations and then averaging that result with the 0.6-depth observation. When more weight to the 0.2- and 0.8-depth observations is desired, the arithmetical mean of the three observations may be used. The first procedure is usually followed, however.

The three-point method is used when the velocities in the vertical are abnormally distributed, as explained on page 134. When a Price type AA (or type A) current meter is used, the three-point method cannot be applied unless the depths are greater than 2.5 ft (0.76 m). (See table 3.)

TWO-TENTHS-DEPTH METHOD

In the 0.2-depth method the velocity is observed at 0.2 of the depth below the surface and a coefficient is applied to the observed velocity to obtain the mean in the vertical. The method is principally used for measuring flows of such high velocity that it is not possible to obtain depth soundings or to position the meter at the 0.8 or 0.6 depth.

A standard cross section or a general knowledge of the cross section at a site is used to compute the 0.2 depth when it is impossible to obtain soundings. A sizable error in an assumed 0.2 depth is not critical in the determination of velocity because the slope of the vertical-velocity curve at this point is usually nearly vertical. (See fig. 88.) The 0.2 depth is also used in conjunction with the sonic sounder for flood measurements. (See p. 108.)

The measurement made by the 0.2-depth method is normally computed by using the 0.2-depth velocity observations without coefficients, as though each observation were a mean in the vertical. The approximate discharge thus obtained divided by the area of the measuring section gives the weighted mean value of the 0.2-depth velocity. Studies of many measurements made by the two-point method show that for a given measuring section the relation between the mean 0.2-depth velocity and the true mean velocity either remains constant or varies uniformly with stage. In either circumstance, this relation may be determined for a particular 0.2-depth measurement by recomputing measurements made at the site by the two-point method using only the 0.2-depth velocity observation as the mean in the vertical. The plotting of the true mean velocity versus the mean 0.2-depth velocity for each measurement will give a velocity-relation curve for use in adjusting the mean velocity for measurements made by the 0.2-depth method.

If at a site too few measurements have been made by the two-point method to establish a velocity-relation curve, vertical-velocity curves are needed to establish a relation between the mean velocity and the 0.2-depth velocity. The usual coefficient to adjust the 0.2-depth velocity to the mean velocity is about 0.87. (See table 2.)

The 0.2-depth method is not as reliable as either the two-point method or the 0.6-depth method when conditions are equally favorable for a current-meter measurement by any of the three methods.

SUBSURFACE-VELOCITY METHOD

In the subsurface-velocity method the velocity is observed at some arbitrary distance below the water surface. That distance should be at least 2 ft (0.6 m), and preferably more for deep swift streams to avoid the effect of surface disturbances. The subsurface-velocity method is used, in the absence of an optical current meter, when it is impossible to obtain soundings and the depths cannot be estimated with sufficient reliability to even approximate a 0.2-depth setting for a conventional current-meter measurement. Coefficients are necessary to convert the velocities observed by the subsurface-velocity method to the mean velocity in the vertical. A prerequisite in obtaining those coefficients is to determine the depths during the measurement from soundings made after the stage has receded enough for that purpose. Those depths are used with the known setting of the current meter below the water surface to compute ratios of depth of observation to total depth during the measurement. The coefficients to be used with the subsurface-velocity observations can then be computed either by use of the data in table 2 or by obtaining vertical-velocity curves at the reduced stage of the stream.

SURFACE-VELOCITY METHOD

If an optical current meter (p. 91–93) is available, the surface-velocity method is used in preference to the subsurface-velocity method described above. In a natural channel a surface-velocity coefficient of 0.85 or 0.86 is used to compute mean velocity on the basis of the data in table 2. In a smooth artificial channel a surface-velocity coefficient of 0.90 is used. If the artificial channel has smooth vertical walls, the coefficients shown in figure 89 are used in the vicinity of the walls. Figure 89 is based on data obtained in a laboratory study. The fact that the coefficients close to the wall are greater than unity is explained by the fact that the secondary currents that form near

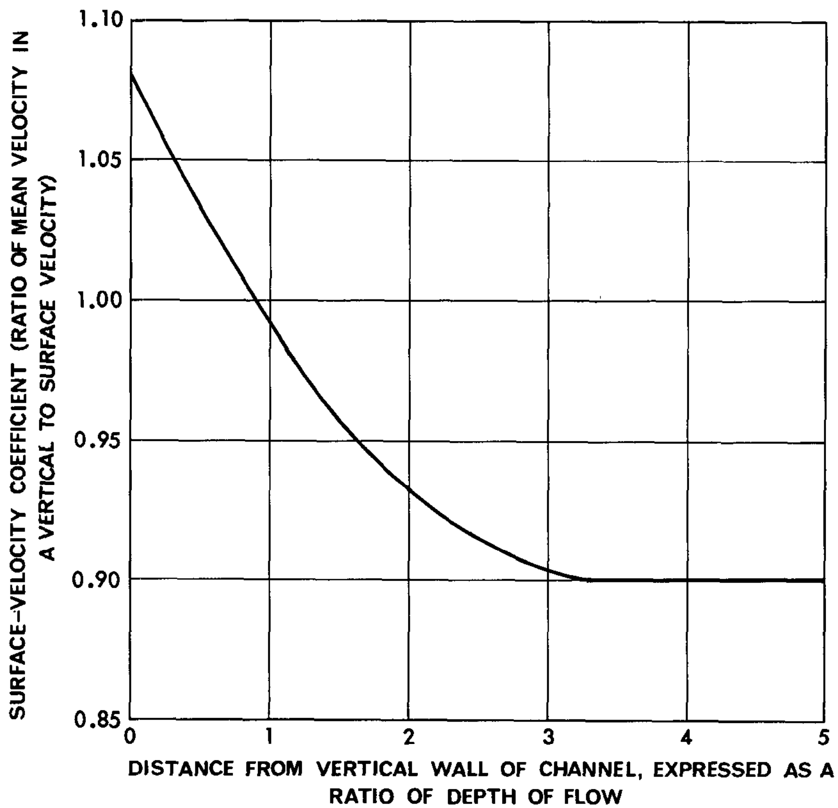


FIGURE 89.—Relation of surface-velocity coefficient to distance from vertical wall of a smooth rectangular channel.

the walls depress the position of the filaments of maximum velocity; that is, the maximum velocity in a vertical close to the wall does not occur at the water surface.

INTEGRATION METHOD

In the integration method the meter is lowered in the vertical to the bed of the stream and then raised to the surface at a uniform rate. During this passage of the meter the total number of revolutions and the total elapsed time are used with the current-meter rating table to obtain the mean velocity in the vertical. The integration method cannot be used with a vertical-axis current meter because the vertical movement of the meter affects the motion of the rotor; consequently, the method is not used in the U.S.A., where the Price meter is the standard current meter. However, the integration method is used to a degree in European countries where horizontal-axis meters are the standard current meters. The accuracy of the measurement is dependent on the skill of the hydrographer in maintaining a uniform rate of movement of the meter. A disadvantage of the method is the inability of the meter to measure streambed velocities because the meter cannot be placed that low. Coefficients smaller than unity are therefore required to correct the observed integrated velocity.

FIVE-POINT METHOD

The five-point method is rarely used in the U.S.A. Velocity observations are made in each vertical at 0.2, 0.6, and 0.8 of the depth below the surface, and as close to the surface and to the streambed as practical. The European criteria for surface and bottom observations state that the horizontal axis of the current meter should not be situated at a distance less than 1½ times the rotor height from the water surface, nor should it be situated at a distance less than 3 times the rotor height from the streambed. In addition, no part of the meter should break the surface of the water.

The velocity observations at the five meter positions are plotted in graphical form, and the mean velocity in the vertical is determined by the use of a planimeter, as explained on page 132. As an alternative, the mean velocity may be computed from the equation

$$V_{\text{mean}} = 0.1(V_{\text{surface}} + 3V_{0.2} + 3V_{0.6} + 2V_{0.8} + V_{\text{bed}}).$$

SIX-POINT METHOD

The six-point method is rarely used in the U.S.A., but is sometimes used in European countries in situations where the existence of a distorted vertical-velocity distribution is known or suspected; for example, in the presence of aquatic growth or under ice cover. Velocity observations are made in each vertical at 0.2, 0.4, 0.6, and 0.8 of the depth below the surface, and also close to the surface and to the streambed. The criteria for surface and streambed observations are those given in the discussion above.

The velocity observations at the six meter positions are plotted in graphical form and the mean velocity in the vertical is determined by planimetering the area bounded by the vertical-velocity curve and the ordinate axis, as explained on page 132. The mean velocity may also be computed mathematically from the equation,

$$V_{mean} = 0.1(V_{surface} + 2V_{0.2} + 2V_{0.4} + 2V_{0.6} + 2V_{0.8} + V_{bed}).$$

PROCEDURE FOR CONVENTIONAL CURRENT-METER MEASUREMENT OF DISCHARGE

The first step in making a conventional current-meter measurement of discharge is to select a measurement cross section of desirable qualities. If the stream cannot be waded, and high-water measurements are made from a bridge or cableway, the hydrographer has no choice with regard to selection of a measurement cross section. If the stream can be waded, the hydrographer looks for a cross section of channel with the following qualities:

1. Cross section lies within a straight reach, and streamlines are parallel to each other.
2. Velocities are greater than 0.5 ft/s (0.15 m/s) and depths are greater than 0.5 ft (0.15 m).
3. Streambed is relatively uniform and free of numerous boulders and heavy aquatic growth.
4. Flow is relatively uniform and free of eddies, slack water, and excessive turbulence.
5. Measurement section is relatively close to the gaging-station control to avoid the effect of tributary inflow between the measurement section and control and to avoid the effect of storage between the measurement section and control during periods of rapidly changing stage.

It will often be impossible to meet all of the above criteria, and when that is the case, the hydrographer must exercise judgment in selecting the best of the sites available for making the discharge measurement.

If the stream cannot be waded and the measurement must be made from a boat, the measurement section selected should have the attributes listed above, except for those listed in item 2 concerning depth and velocity. Depth is no consideration in a boat measurement; if the stream is too shallow to float a boat, the stream can usually be waded. However, velocity in the measurement section is an important consideration. If velocities are too slow, meter registration may be affected by an oscillatory movement of the boat, in which the boat, even though fastened to the tag line, moves upstream and downstream as a result of wind action; or, where a vertical-axis meter is used, meter registration may be affected by vertical movement of the boat as a result of wave action (p. 180–181). If velocities are too fast, it becomes difficult to string the tag line across the stream.

Regardless of the type of measurement that is to be made, if the gaging station is downstream from a hydroelectric powerplant, the stage will be changing too rapidly, most of the time, to assure a satisfactory discharge measurement. The hydrographer should obtain a schedule of operations from the powerplant operator, or determine the operating schedule from the gage-height chart, and plan to make his discharge measurements near the crest or trough of the stage hydrograph, or during periods of near-constant discharge from the powerplant.

After the cross section has been selected, the width of the stream is determined. A tag line or measuring tape is strung across the measurement section for measurements made by wading, from a boat, from ice cover, or from an unmarked bridge. Except where a bridge is used, the line is strung at right angles to the direction of flow to avoid horizontal angles in the cross section. For cableway or bridge measurements, use is made of the graduations painted on the cable or bridge rail as described on page 110. Next the spacing of the verticals is determined to provide about 25 to 30 subsections. If previous discharge measurements at the site have shown uniformity of both the cross section and the velocity distribution, fewer verticals may be used. The verticals should be so spaced that no subsection has more than 10 percent of the total discharge. The ideal measurement is one in which no subsection has more than 5 percent of the total discharge, but that is seldom achieved when 25 subsections are used. (The discharge measurement notes in figure 42 show that the subsection with the greatest discharge had 6.2 percent of the total discharge.) It is not recommended that all observation verticals be spaced equally unless the discharge is evenly distributed across the stream. The spacing between verticals should be closer in those parts of the cross section that have the greater depths and velocities.

After the stationing of the observation verticals has been determined, the appropriate equipment for the current-meter measurement is assembled and the measurement note sheets for recording observations are prepared. (See fig. 42.) The following information will be recorded for each discharge measurement:

1. Name of stream and location to correctly identify the established gaging station; or name of stream and exact location of site for a miscellaneous measurement.

2. Date, party, type of meter suspension, and meter number.
3. Time measurement was started using military time (24-hr clock system).
4. Bank of stream that was the starting point.
5. Control conditions.
6. Gage heights and corresponding times.
7. Water temperature.
8. Other pertinent information regarding the accuracy of the discharge measurement and conditions which might affect the stage-discharge relation.

The streambank is identified by the letters LEW or REW (left edge of water or right edge of water, respectively, when facing downstream). The time is recorded periodically in the notes during the course of the measurement. If the gaging station is equipped with a digital recorder it is advantageous to synchronize the time observations with the punch cycle of the recorder. (See fig. 42.) The time observations are important for computing the mean gage height of the discharge measurement, if the measurement is made during a period of appreciable change in stage. (See p. 170–173.) When the discharge measurement is completed, the time is recorded along with the streambank (LEW or REW) where the measurement ended.

We have digressed somewhat in discussing the measurement notes and now return to the details of the discharge measurement. After the note sheet is readied, the meter assembly is checked. The meter should balance on the hanger used and should spin freely; the electric circuit through the meter should operate satisfactorily; and the stopwatch should check satisfactorily in a comparison with the hydrographer's watch. After recording on the note sheet the station (distance from initial point) of the edge of water, the actual measurement is ready to be started.

Depth (if any) at the edge of water is measured and recorded. The depth determines the method of velocity measurement to be used, normally the two-point method (p. 134) or the 0.6-depth method (p. 134). The setting of the meter for the particular method to be used is then computed, and the meter position is recorded, using a designation such as 0.8 or 0.6 or 0.2, as the case may be. After the meter is placed at the proper depth and pointed into the current, the rotation of the rotor is permitted to become adjusted to the speed of the current before the velocity observation is started. The time required for such adjustment is usually only a few seconds if velocities are greater than 1 ft/s (0.3 m/s), but for slower velocities, particularly if the current meter is suspended on a cable, a longer period of adjustment is needed. After the meter has become adjusted to the current, the number of revolutions made by the rotor is counted for a period of 40 to 70 s. The stopwatch is started simultaneously with the first signal or click, which is counted as "zero," and not "one." The count is ended on a convenient number coinciding with one of those given in the column headings of the meter rating table. The stopwatch is stopped on that count and is read to the nearest second or to the nearest even second if the hand of the stopwatch is on a half-second mark. That number of seconds and the number of revolutions are then recorded.

If the velocity is to be observed at more than one point in the vertical, the meter setting for the additional observation is determined, the revolutions are timed, and the data are recorded. The hydrographer moves to each of the observation verticals and repeats the above procedure until the entire cross section has been traversed. For each vertical he records distance from initial point, water depth, meter-position depth, revolutions of the meter, and the time interval associated with those revolutions (fig. 42).

Consideration must be given to the direction of flow, because it is the component of velocity normal to the measurement section that must be determined. The discussion that follows concerns currents that approach the measurement section obliquely, at angle α (fig. 47). If, in a wading measurement, the meter used is a horizontal-axis meter with a component propellor, such as the Ott meter, the propellor should be pointed upstream at right angles to the cross section, but only if α is less than 45°. Such a meter will register the desired component of velocity normal to the cross section, when α is less than 45°. If α is greater than 45°, the component meter should be pointed into the current. All other meters on a wading-rod suspension should likewise be pointed into the current. Any meter on a cable suspension, as is used for the higher stages, will automatically point into the current because of the effect of the meter vanes. When the meter is pointed into an oblique current the measured velocity must be multiplied by the cosine of the angle (α) between the current and a perpendicular to the measurement section in order to obtain the desired normal component of the velocity.

In the U.S.A., either of two methods is used to obtain cosine α (fig. 47). In the first method, use is made of the field notes which have a point of origin (o) printed on the left margin and cosine values on the right margin (fig. 42). The cosine of the angle of the current is measured by holding the note sheet in a horizontal position with the point of origin on the tag line, bridge rail, cable rail, or any other feature parallel to the cross section (fig. 90). With the long side of the note sheet parallel to the direction of flow, the tag line or bridge rail will intersect the value of cosine α on the top, bottom, or right edge of the note sheet. The direction of the current will be apparent from the direction of movement of floating particles. If the water is clear of floating material, small bits of floating material are thrown into the stream and the edge of the note sheet is alined parallel to the direction of movement. If no such material is available, the inelegant, but time-honored method of spitting into the stream is used to obtain an indicator of the direction of flow. The measured velocity is multiplied by the cosine of the angle to determine the velocity component normal to the measurement section.

The second, and more reliable, method of obtaining cosine α involves the use of a folding foot-rule. These rules, which are either 3 or 6 ft long, are graduated in hundreths of a foot and are jointed every half foot. The first 2 ft of the rule is extended, the 2.00-ft marker is placed on the tag line or bridge rail (fig. 91), and the rule is alined with the direction of the current. The rule is folded at the 1-ft mark so that the first foot of the rule is normal to the tag line or bridge rail. That reading, subtracted from 1.00, is cosine α. For example, if the reading on the rule is 0.09 ft, cosine α equals 0.91.

Details peculiar to each of the various types of current-meter measurement are described in the sections of the manual that follow.

CURRENT-METER MEASUREMENT BY WADING

Current–meter measurements are best made by wading, if conditions permit. (See fig. 92.) Wading measurements have a distinct advantage over measurements made from cableways or bridges in that it is usually possible to select the best of several available cross sections for the measurement. The type AA or pygmy meter is used for wading measurements in the U.S.A. Table 3 lists the type of meter and velocity method to be used for wading measurements at various depths.

Some departure from table 3 is permissible. For example, if a type

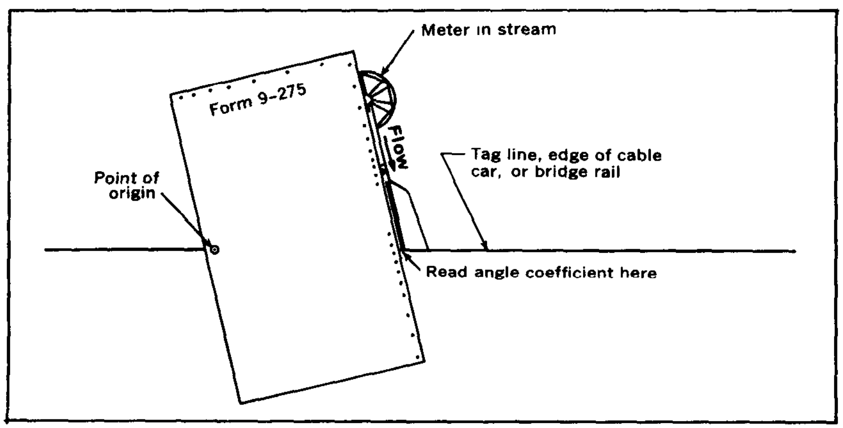


FIGURE 90.—Measurement of horizontal angle with measurement-note sheet.

AA meter is being used in a measurement section that has almost all its depths greater than 1.5 ft (0.46 m), the pygmy meter should not be substituted for the few depths that are less than 1.5 ft, or vice versa. With regard to the use of the type AA meter in depths less than 1.5 ft, strictly speaking, that meter should not be used in depths less than 1.25 ft (0.38 m). A depth of 1.25 ft will accommodate the 0.6-depth method without causing the meter to be set closer than 0.5 ft from the streambed; if the meter is set any closer to the streambed it will underregister the velocity (p. 132). However, if the hydrographer is using the type AA meter in a measurement section that has only few verticals shallower than 1.25 ft, he may use that meter for depths that are even as shallow as 0.5 ft (0.15 m) without changing to a pygmy meter. The hydrographer must make a judgment decision. He knows that his meter is underregistering the velocity by some unknown percentage in the depths shallower than 1.25 ft, but if that shallow-depth flow represents less than 10 percent of the total discharge, his total measured discharge should not be too greatly in error.

Neither the type AA meter nor the pygmy meter should be used for measuring velocities slower than 0.2 ft/s, unless absolutely necessary.

If depths or velocities under natural conditions are too low for a dependable current-meter measurement, the cross section should be modified, if practical, to provide acceptable conditions. It is often pos-

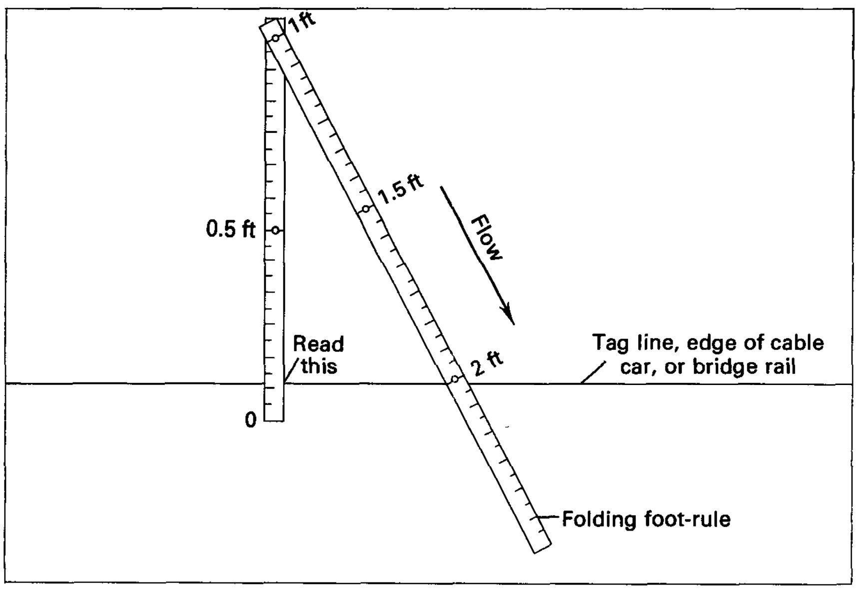


FIGURE 91.—Measurement of horizontal angle with folding foot-rule.

sible to build temporary dikes to eliminate slack water or shallow depths in a cross section, or to improve the cross section by removing rocks and debris within the section and in the reach of channel im-

FIGURE 92.—Wading measurement using top-setting rod.

TABLE 3.—*Current-meter and velocity-measurement method for various depths (wading measurement)*

Depth (ft)	Depth (m)	Meter	Velocity method
2.5 or more	0.76 or more	Type AA (or type A)	0.2 and 0.8
1.5–2.5	0.46–0.76	---- do ----	.6
.3–1.5	.09– .46	Pygmy[1]	.6

[1]Used when velocities are less than 2 5 ft/s (0 76 m/s)

mediately upstream and downstream from the section. After the cross section has been modified, the flow should be allowed to stabilize before starting the discharge measurement.

The hydrographer should stand in a position that least affects the velocity of the water passing the current meter. That position is usually obtained by facing the bank so that the water flows against the side of the leg. The wading rod is held at the tag line by the hydrographer who stands about 3 in (0.07 m) downstream from the tag line and at least 1.5 ft (0.46 m) from the wading rod. He should avoid standing in the water if his feet and legs occupy a significantly large percentage of a narrow cross section. In small streams where the width permits, the hydrographer stands on an elevated plank or other support, rather than in the water.

The wading rod should be held in a vertical position with the meter parallel to the direction of flow while the velocity is being observed. If the flow is not at right angles to the tag line, the angle coefficient should be carefully measured (figs. 90 and 91).

When measuring streams having shifting beds, the soundings or velocities can be affected by the scoured depressions left by the hydrographer's feet. For such streams, the meter should be placed ahead of and upstream from the feet. For such streams, too, the hydrographer's notes should accurately describe the configuration of the streambed and water surface. (See p. 376–379.)

A measurement should be made of the depth of water over the lowest point of the control, either before or after the discharge measurement. The gage height corresponding to this lowest point (gage height of zero flow) is very useful in analyzing the stage-discharge relation for the gaging station. (See p. 333–334).

When the flow is too low for a reliable measurement of discharge by current meter, the discharge is determined by use of (1) a volumetric method of measurement, (2) a portable Parshall flume, or (3) a portable weir plate. Those three methods of discharge determination are discussed in chapter 8.

CURRENT-METER MEASUREMENTS FROM CABLEWAYS

The equipment assemblies for use on cableways are described on pages 110–117.

The size of the sounding weight used in current-meter measurements depends on the depth and velocity in the measurement cross section. A rule of thumb generally used is that the size of the weight (lb) should be greater than the maximum product of velocity (ft/s) and depth (ft) in the cross section. If insufficient weight is used, the meter assembly will be dragged downstream. If debris or ice is flowing or if the stream is shallow and swift, the weight used should be appreciably heavier than that indicated by the above rule. The rule is not rigid, but it does provide a starting point for deciding on the size of weight required at various stages.

The Price type AA current meter is generally used in the U.S.A. when making discharge measurements from a cableway. The depth is measured by use of a sounding reel, and the velocity is measured by setting the meter at the proper position in the vertical. (See table 4.) Table 4 is designed so that no velocity observations will be made with the meter closer than 0.5 ft (0.15 m) to the water surface. In the zone from the water surface to a depth of 0.5 ft, the current meter is known to give erroneous results.

Some sounding reels are equipped with a computing depth indicator. To use the computing spiral, the indicator is set at zero when the center of the current-meter rotor is at the water surface. The sounding weight and meter are then lowered until the weight touches the streambed. If, for example, a 30 C .5 (see table 4) suspension is used and if the indicator reads 18.5 ft when the sounding weight touches the bottom, the depth would be 19.0 ft (18.5 ft + 0.5 ft). To move the meter to the 0.8-depth position (0.8×19.0 ft), the weight and meter are raised until the hand on the indicator is over the 19-ft mark on the graduated spiral (fig. 60); the hand will then be pointing to 15.2 on the main dial. To set the meter at the 0.2-depth position (0.2×19.0 ft), the weight and meter are raised until the hand on the indicator is pointing to 3.8 ft on the main dial.

One problem found in observing velocities from a cableway is that movement of the cable car from one station to the next causes the car to oscillate for a short time after coming to a stop. The hydrographer should wait until this oscillation has been dampened to the extent that it is negligible before counting meter revolutions.

Tags can be placed on the sounding line at known distances above the center of the meter cups as an aid in determining depths. Furthermore, the use of tags allows the meter to be kept submerged throughout the discharge measurement to prevent freezing in cold air during the winter measurements. The tags, which are usually streamers of different colored binding tape, are fastened to the sounding line by solder beads or by small cable clips. Tags are used for determining depth in either of two ways.

1. In the procedure that is usually preferred, a tag is set at the water surface, after which the depth indicator is set at the distance between that particular tag and the center of the meter cups. This is equivalent to setting the indicator at zero when the center of the meter rotor is at the water surface, and the hydrographer then proceeds with his depth settings as described in a preceding paragraph. If debris or ice is flowing, this method prevents damage to the meter.

TABLE 4.—*Velocity-measurement method for various meter suspensions and depths*

Suspension[1]	Minimum depth			
	0.6 method		0.2 and 0.8 method	
	(ft)	(m)	(ft)	(m)
15 C .5, 30 C .5	1.2	0.37	2.5	0.76
50 C .55	1.4	.43	2.8	.85
50 C .9	2.2	.67	4.5	1.37
75 C 1.0, 100 C 1.0, 150 C 1.0	2.5	.76	5.0	1.52
200 C 1.5, 300 C 1.5	[2]3.8	1.16	7.5	2.29

[1]Suspensions shown indicate the size of the sounding weight and the distance from the bottom of the weight to the current-meter axis. Thus "50 C .9" refers to a 50-pound Columbus-type weight, the suspension for which puts the meter 0.9 ft above the bottom of the weight.

[2]Use 0.2-depth method for depths between 2.5 and 3.7 ft, and apply appropriate coefficient (usually 0.87 from table 2).

2. In the second method the sounding weight is first lowered to the streambed, and the depth is then determined by raising the weight until the first tag below the water surface appears at the surface. The total depth is then sum of (a) the distance the weight was raised to bring the tag to the water surface, (b) the distance the tag is above the center of the meter cups, and (c) the distance from the bottom of the weight to the center of the cups. This method is usually used with handlines, and it is also used to simplify the measurement of deep, swift streams (p. 159–163).

If large quantities of debris are carried by the stream, the meter should be periodically raised to the cable car for inspection during the measurement to be certain that the pivot and rotor of the meter are free of debris. However, the meter should be kept in the water during the measurement if the air temperature is well below freezing. The hydrographer should carry a pair of lineman's side-cutter pliers when making measurements from a cableway. These can be used to cut the sounding line to insure safety if the weight and meter become caught on a submerged object or on heavy floating debris and it is impossible to release them. Sometimes the cable car can be pulled to the edge of the water where the entangling debris can be removed.

When a measurement of a deep, swift stream is made from a cableway, the meter and weight do not hang vertically but are dragged downstream. The vertical angle, that is, the angle between the meter-suspension cable and the vertical, should be measured by protractor in order to correct the soundings to give the actual vertical depth. The procedure used to correct soundings is described on pages 159–168.

If a handline is used to suspend the current meter and sounding weight, the measurement procedure to be followed is that described in the section on bridge measurements (p. 150–151).

CURRENT-METER MEASUREMENTS FROM BRIDGES

Bridges are often used for making discharge measurements of streams that cannot be waded. Measurement cross sections under bridges are often satisfactory for current-meter measurements, but cableway sections are usually superior.

No set rule can be given for choosing between the upstream or downstream side of the bridge for making a discharge measurement. The advantages of using the upstream side of the bridge are:

1. Hydraulic characteristics at the upstream side of bridge openings usually are more favorable.
2. Approaching drift can be seen and thus can be more easily avoided.
3. The streambed at the upstream side of the bridge is not likely to be scoured as badly as the downstream side.

The advantages of using the downstream side of the bridge are:

1. Vertical angles are more easily measured because the sounding line will move away from the bridge.
2. The flow lines of the stream may be straightened by passing through a bridge opening with piers.

Whether to use the upstream side or the downstream side of a bridge for a current-meter measurement should be decided individually for each bridge after considering the above factors. Other pertinent factors relate to physical conditions at the bridge, such as location of the walkway, traffic hazards, and accumulation of trash on pilings and piers.

In making the discharge measurement either a handline, or a sounding reel supported by a bridge board or by a portable crane, is used to suspend the current meter and sounding weight from the bridge. The velocity is measured by setting the meter at positions in the vertical as indicated in table 4. If velocities are high, the equipment is used no closer than several feet from piers and abutments. In that situation depths and velocities at the pier or abutments are estimated on the basis of observations in the vertical nearest the pier. (See p. 82.)

Where piers are in the measuring section, it is usually necessary to use more than 25–30 subsections to obtain results as reliable as those obtained with a similar measuring section that has no piers. Piers not only affect the horizontal distribution of velocities, but they frequently affect the direction of the current, causing horizontal angles that must be carefully measured.

Whether or not to exclude the area of a bridge pier from the area of the measurement cross section depends primarily on the relative locations of the measurement section and the end of the pier. If measurements are made from the upstream side of the bridge, it is the relative location of the upstream end (nose) of the pier that is relevant; for measurements made from the downstream side it is the location of the downstream end (tail) of the pier that is relevant. If any part of the pier extends into the measurement cross section, the area of the pier is excluded. However, bridges quite commonly have cantilevered walkways from which discharge measurements are made. In that case the measurement cross section lies beyond the end of the pier—upstream from the nose or downstream from the tail, depending on which side of the bridge is used. In that situation it is the position and direction of the streamlines that determines whether or not the pier area is to be excluded. The hydrographer, if he had not previously noted the stationing of the sides of the pier when projected to the measurement cross section, does so now. If there is negligible or no downstream flow in that width interval (pier subsection)—that is, if only stagnation and (or) eddying exists upstream from the nose or downstream from the tail, whichever is relevant—the area of the pier is excluded. If there is significant downstream flow in the pier subsection, the area of the pier is included in the area of the measurement cross section. The horizontal angles of the streamlines in and near the pier subsection will usually be quite large in that circumstance.

Footbridges are sometimes used for measuring the discharge of canals, tailraces, and small streams. Often a rod suspension can be used for the meter when measuring from a footbridge. In low velocities the procedure for determining depth when using a rod suspension is the same as that used for wading measurements. For higher velocities depth is obtained from the difference in readings at an index point on the bridge when the base plate of the rod is at the water surface and when it is on the streambed. The measurement of depth by that method eliminates errors in reading the depth caused by the fast-moving water piling up on the upstream face of the rod. Handlines, bridge cranes, and bridge boards are also used from footbridges.

When using a sounding reel, depths and velocities are measured by the methods described in the preceding section of the manual on cableway measurements (p. 146–148). When using a handline, depth is determined by first lowering the sounding weight to the streambed and then raising the weight until one of the tags is at the water surface. The distance that the weight is raised is measured along the rubber-covered cable (p. 106) with either a steel or metallic tape, a folding foot-rule, or a graduated rod. The total depth of water is then the summation of (1) the distance the particular tag is above the meter cups, (2) the measured distance the meter and weight were raised, and (3) the distance from the bottom of the weight to the meter cups.

Another, less widely used, method of determining depth requires that the meter cups be set at the water surface, after which the sounding weight is lowered to the streambed while measuring, with tape or rule, the length of line that is let out. This measured distance, plus the distance from the bottom of the sounding weight to the meter cups, is the depth of water.

When using a handline, enough cable is unwound from the handline reel to keep the reel out of water when the sounding weight is on the streambed at the deepest part of the measuring section. That prevents submergence of the reel and thick rubber-covered cable and attendant drag on the equipment in high-velocity flow, and unless the bridge is relatively close to the water surface, it still permits the hydrographer to raise and lower the meter by means of the rubber-covered cable rather than the bare-steel cable. When the meter is set for a velocity observation, the hydrographer stands on the rubber-covered cable or ties it to the handrail to hold the meter in place. By doing so his hands are free to operate the stopwatch and record the data.

If the bridge has vertical truss members in the plane of the measurement cross section, the handline can be disconnected from the headphone wire and passed around the truss member with the sounding weight on the bottom. This eliminates the need for raising the weight and meter to the bridge each time a move is made from one vertical to another and is the principal advantage of a handline.

CURRENT-METER MEASUREMENTS FROM ICE-COVER

Discharge measurements under ice cover (fig. 93) are usually made under conditions that range from uncomfortable to severe, but it is extremely important that they be made, because the reliability of a large part of the computed discharge record for a winter period may depend on one such measurement.

Cross sections for possible use for measuring under ice cover should be selected during the open-water season when channel conditions can be observed and evaluated. Commonly the most desirable measurement section will be just upstream from a riffle because slush ice that collects under the ice cover is usually thickest at the upstream end of the pools created by riffles. The equipment used for cutting or drilling holes in the ice was described on pages 124–129.

The danger of working on ice-covered streams should never be underestimated. When crossing the stream, the hydrographer should test the strength of the ice with solid blows using a sharp ice chisel. Ice thickness may be irregular, especially late in the season when a thick snow cover may act as an insulator. Water just above freezing can slowly melt the underside of the ice, creating thin spots. Ice

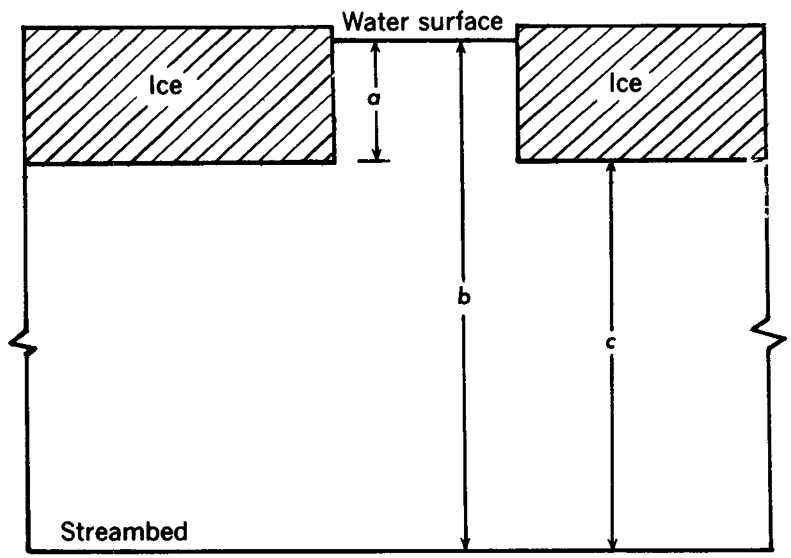
FIGURE 93.—Ice rod being used to support current meter for a discharge measurement, top; ice drill being used to cut holes, bottom.

bridged above the water may be weak, even though relatively thick.

At the cross section selected for measurement three holes, one at each quarter point of the width, are cut to check on the possible presence of slush ice or possible maldistribution of flow. If poor conditions are found, other cross sections are similarly investigated to find one that is free of slush ice and has a favorable horizontal distribution of flow. After finding a favorable cross section, at least 20 holes are cut for the current-meter measurement of discharge. The holes should be spaced so that no subsection carries more than 10 percent of the total discharge. On narrow streams it may be simpler to remove all the ice in the cross section.

The effective depth of the water (fig. 94) is the total depth of water minus the distance from the water surface to the bottom of the ice. The vertical pulsation of water in the holes in the ice sometimes causes difficulty in determining the depths. The total depth of water is usually measured with an ice rod or with a sounding weight and reel, depending on the depth.

The distance from the water surface to the bottom of the ice is measured with an ice-measuring stick (p. 125), unless slush is present at the hole. In that situation the effective depth is the total depth



a = Water surface to bottom of ice
b = Total depth of water
c = Effective depth ($c = b - a$)

0.2-depth setting $= a + 0.2c$
0.8-depth setting $= b - 0.2c$
0.6-depth setting $= b - 0.4c$

FIGURE 94.—Method of computing meter settings for measurements under ice cover.

minus the distance from the water surface to the interface between water and slush. To locate that interface, the current meter is suspended at a depth below the slush ice where the meter rotor turns freely. The meter is then slowly raised until the rotor stops; that point is considered the interface, and its depth below the water surface is measured for determining the effective depth. The effective depth is then used to compute the proper position of the meter in the vertical.

The vane ice meter is recommended for use under ice cover because (1) the vanes do not become filled with slush ice as the cups of the Price meter often do, (2) the yoke of the vane meter will fit in the hole made by the ice drill, and (3) the yoke and ice rod can serve as an ice-measuring stick. The contact chamber of the vane meter can be rotated to any position; its binding post is therefore placed perpendicular to the axis of the yoke to avoid interference when using the top of the yoke as the horizontal leg of an ice-measuring stick.

Because of the roughness of the underside of the ice cover, the location of the filament of maximum velocity is some distance below the underside of the ice. Figure 95 shows a typical vertical-velocity

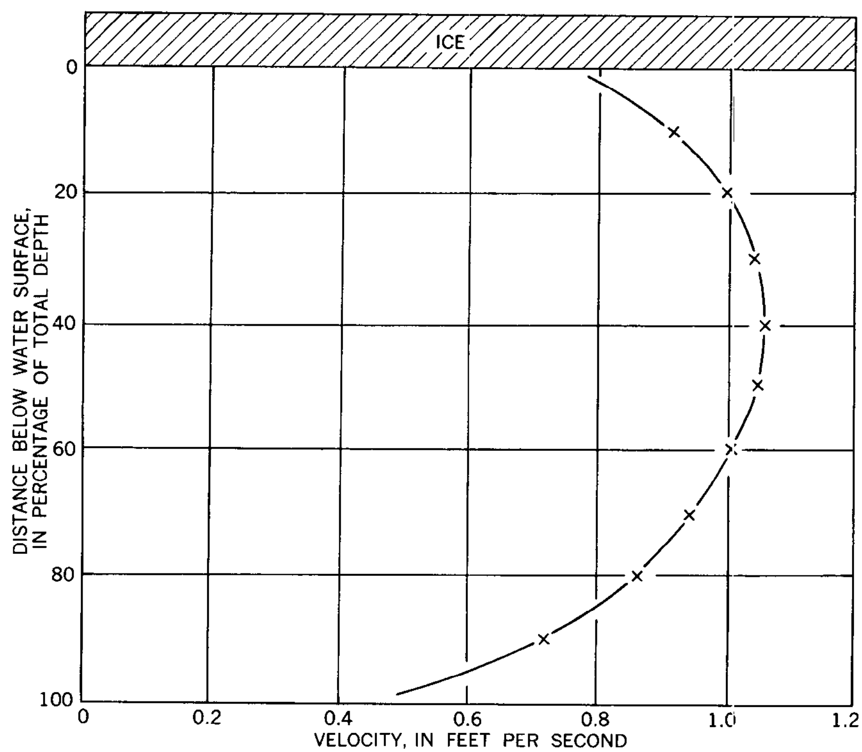


FIGURE 95.—Typical vertical-velocity curve under ice cover.

curve under ice cover. In making a discharge measurement the 0.2- and 0.8-depth method is recommended in the U.S.A. for effective depths equal to or greater than 2.5 ft, and the 0.6-depth method for effective depths less than 2.5 ft. It is also recommended that two vertical-velocity curves be defined when ice measurements are made to determine whether any coefficients are necessary to convert the velocity obtained by the 0.2- and 0.8-depth method, or by the 0.6-depth method, to the mean velocity. Normally the average of the velocities obtained by the 0.2- and 0.8-depth method gives the mean velocity, but a coefficient of about 0.92 usually is applicable to the velocity obtained by the 0.6-depth method. In Europe a three-point method is commonly used in which velocity observations are made at 0.15, 0.5, and 0.85 of the effective depth.

When measuring the velocity, the meter is kept as far upstream as possible to minimize any effect that the vertical pulsation of water in the hole might have on the meter registration. The meter is exposed as little as possible to the cold air so that its operation will not be impaired by the formation of ice on exposed parts.

If there is only partial ice cover on the measuring section, the procedure described above is used for the ice-covered observation verticals, and open-water methods are used for the open-water verticals.

A sample sheet of discharge-measurement notes for a measurement made under ice cover is shown in figure 96. Vertical-velocity curves that had been defined for that measurement had indicated that mean velocity in a vertical was given by the 0.2- and 0.8-depth method, and that the 0.6-depth method required a coefficient of 0.92.

CURRENT-METER MEASUREMENTS FROM BOATS

Discharge measurements are made from boats where no cableways or suitable bridges are available and where streams are too deep to wade. Personal safety is the limiting factor in the use of boats on streams having high velocities.

In making a boat measurement the tag line is first strung across the measurement section by unreeling the line as the boat moves across the stream. Some tag-line reels are equipped with brakes (fig. 79) to control the line tension during the unreeling. If a tag line whose reel is unequipped with a brake has been strung across a stream, the slack is taken up by means of a block and tackle attached to the reel and to an anchored support on the bank. If there is traffic on the river, one man must be stationed on the bank to lower and raise the tag line to allow river traffic to pass. Streamers should be attached to the tag line so that it may be seen by boat pilots. The method of positioning the boat for measuring depths and velocities, by sliding the boat along the tag line from one observation vertical to another, was described on page 121.

If the flow of traffic on the river is continual or if the width of the river is too great for a tag line to be used, other means are needed to position the boat. One method that dispenses with the tag line involves keeping the boat lined up with flags positioned on each end of

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WATER RESOURCES DIVISION

Meter
V-378

Date March 18, 1962 Number of Meas. 365

DISCHARGE MEASUREMENT NOTES—ICE COVER

Frozen ~~River at~~ Creek, near Hot Springs, N.Y.

	REW Dist. from initial point	Width	Total depth of water	W.S. to bot. ice	Effective depth	Depth of meter below water surface	Revolutions	Time in seconds	Velocity: At point	Velocity: Mean in vertical	Area	Discharge
	1315											
Veloc. coef.	24	3	0	-	-	-	-	-		0	0	0
.92	30	6	2.6	1.6	1.0	2.2	10	47	.50	.46	6.0	2.8
.92	36	6	3.6	1.8	1.8	2.9	15	49	.71	.65	10.8	7.0
.92	42	5	4.3	2.0	2.3	3.4	15	44	.79	.73	11.5	8.4
1.0	46	4	4.5	1.8	2.7	2.3	20	43	1.07	.87	10.8	9.4
						4.0	15	52	.67			
	50	4	4.7	1.7	3.0	2.3	20	40	1.15	.92	12.0	11.0
						4.1	15	50	.70			
	54	4	4.6	1.7	2.9	2.3	25	49	1.17	.96	11.6	11.1
	1325					4.0	15	46	.76			
	58	4	4.9	1.6	3.3	2.3	20	40	1.15	.92	13.2	12.1
						4.2	15	50	.70			
	62	4	4.8	1.6	3.2	2.2	25	48	1.20	.98	12.8	12.5
						4.2	15	46	.76			
	66	3.5	5.0	1.5	3.5	2.2	25	44	1.30	1.08	12.2	13.2
						4.3	15	41	.85			
	69	3	5.3	1.6	3.7	2.3	25	40	1.43	1.15	11.1	12.8
						4.6	15	40	.87			
	72	3	5.1	1.5	3.6	2.2	25	41	1.40	1.16	10.8	12.5
	1335					4.4	20	51	.91			
		49.5									122.8	112.8

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FIGURE 96.—Part of notes for discharge measurement under ice cover.

the measurement cross section. Flags on one bank would suffice, but it is better to have them on both banks. The position of the boat in the cross section can be determined by means of a transit on the shore and a stadia rod held in the boat (fig. 97). Another method of determining the position of the boat is by setting a transit on one bank at a convenient measured distance from, and at right angles to, the cross section. The position of the boat is computed by measuring the angle to the boat, as shown in figure 98. A third method of determining the position of the boat requires a sextant in the boat. The sextant is used to read the angle between a flag at the end of the cross section and another at a known distance perpendicular to the cross section (fig. 98). The boat position can be computed from the measured angle and the known distance between the flags on the shore. Unless anchoring is more convenient, the boat must be held stationary by its motor when readings are being taken.

Boat measurements are not recommended where velocities are

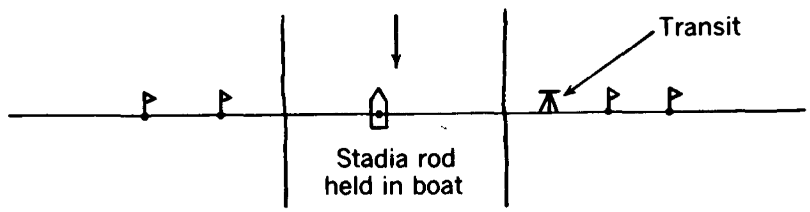
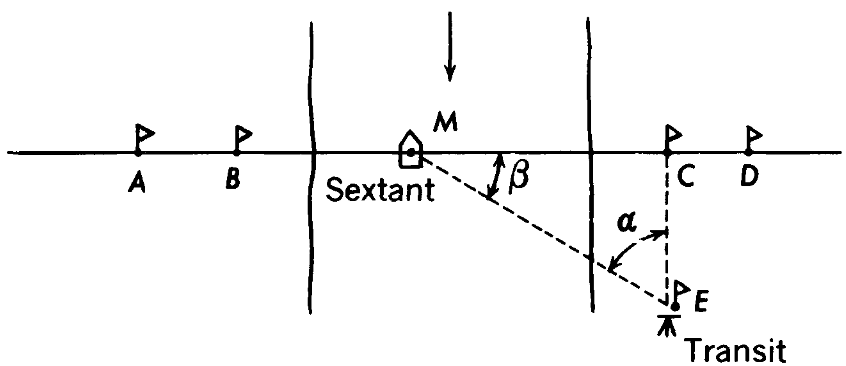


FIGURE 97.—Determining position in the cross section, stadia method.



$$MC = CE \tan \alpha \text{ (transit)}$$

$$MC = \frac{CE}{\tan \beta} \text{ (sextant)}$$

FIGURE 98.—Determining position in the cross section, angular method.

slower than 1 ft/s (0.3 m/s) when the boat is subject to the action of wind and waves. (See p. 180–181.) If the maximum depth in the cross section is less than 10 ft (3 m) and the velocity is low, a rod is usually used for measuring the depth and supporting the current meter. For greater depths and velocities, a cable suspension with reel, boat boom, and sounding weight is used. The procedure for measuring discharge from a boat using the boat boom and crosspiece (p. 122–123) is the same as that for measuring from a bridge or cableway, once the special equipment has been set up and the method of positioning the boat has been established.

A special method of measuring discharge from a boat without stopping the boat at observation stations—the moving-boat method—is described in detail in chapter 6.

NETWORKS OF CURRENT METERS

Occasional special measurements require the simultaneous determination of velocities at several points in a cross section, distributed either laterally or vertically. For example, it may be necessary to measure a vertical-velocity profile quickly in unsteady flows and to check it frequently in order to determine the changes in shape of the vertical profile as well as the rates of those changes. Another example is the measurement of tide-affected streams where it is desirable to measure the total discharge continously during at least a full tidal cycle (approximately 13 hr). The need for so many simultaneous velocity determinations (one at each vertical in the cross section) for so long a period can be an expensive and laborious process using conventional techniques of discharge measurement.

A grouping of 21 current meters and special supplemental instrumentation has been used by the U.S. Geological Survey to facilitate measurements of the types just described. Only a few persons are required to operate the system. The 21 meters are connected so that the spacing between any two adjacent meters can be varied to distances as great as 200 ft (61 m). Furthermore, each meter has sufficient handline cable to be suspended vertically from a bridge for as much as 200 ft. The meters have a standard calibration. Revolutions of the rotors are recorded by electronic counters that are grouped compactly in one box at the center of the bank of meters. The operator, by flipping one switch, starts all 21 counters simultaneously and, after an interval of several minutes, stops all counters. The indicated number of revolutions for the elapsed time interval is converted to a velocity for each meter. The distance between meters is known; a record of stage is maintained to evaluate depth; prior information at the site is obtained to convert point velocities in the verticals to mean velocities in those verticals. All of the information necessary to compute discharge in the cross section is therefore available and is tabulated for easy conversion to discharge.

Other countries have developed similar equipment; for example, a grouping of 40 meters is used in the United Kingdom.

SPECIAL PROBLEMS IN CONVENTIONAL CURRENT-METER MEASUREMENTS

MEASUREMENT OF DEEP, SWIFT STREAMS

The measurement of deep, swift streams by current meter usually presents no serious problems when adequate sounding weights are used and when floating ice or drift is not excessive. Normal procedures must sometimes be altered, however, when measuring streams under particularly adverse conditions, the four most common situations of that kind being represented by the following cases:

Case A. Possible to sound, but weight and meter drift downstream.

Case B. Not possible to sound, but a standard cross section is available.

Case C. Not possible to sound, and no standard cross section is available.

Case D. Not possible to submerge the meter in the water.

Procedures are described below for discharge measurements made under those adverse conditions. The procedures for cases B, C, and D are actually applicable only where channel conditions in the measurement cross section or reach are stable, meaning that no significant scour or deposition occurs.

CASE A. DEPTH CAN BE SOUNDED

Where it is possible to sound the depth but the weight and meter drift drownstream, the depths as measured by the usual methods will be in error, being too large (fig. 99). The correction for the error has two parts, the air correction and the wet-line correction. The air correction is shown in figure 99 as the distance cd. The wet-line correction in figure 99 is shown as the difference betwen the wet-line depth de and the vertical depth dg.

As shown in figure 99, the air correction depends on the vertical angle P and the distance ab. The correction is computed as follows:

$$ab = ac$$

$$\cos P = \frac{ab}{ad} = \frac{ab}{ac+cd} = \frac{ab}{ab+cd}$$

$$ab+cd = \frac{ab}{\cos P}$$

$$cd = \frac{ab}{\cos P} - ab = ab\left[\frac{1}{\cos P} - 1\right]. \qquad (11)$$

The air correction for even-numbered angles between 4° and 36° and for vertical lengths between 10 and 100 ft is shown in table 5. The correction is applied to the nearest tenth of a foot; hundredths are given to aid in interpolation. Table 6 is a similar table in metric units.

The air correction may be nearly eliminated by using tags at

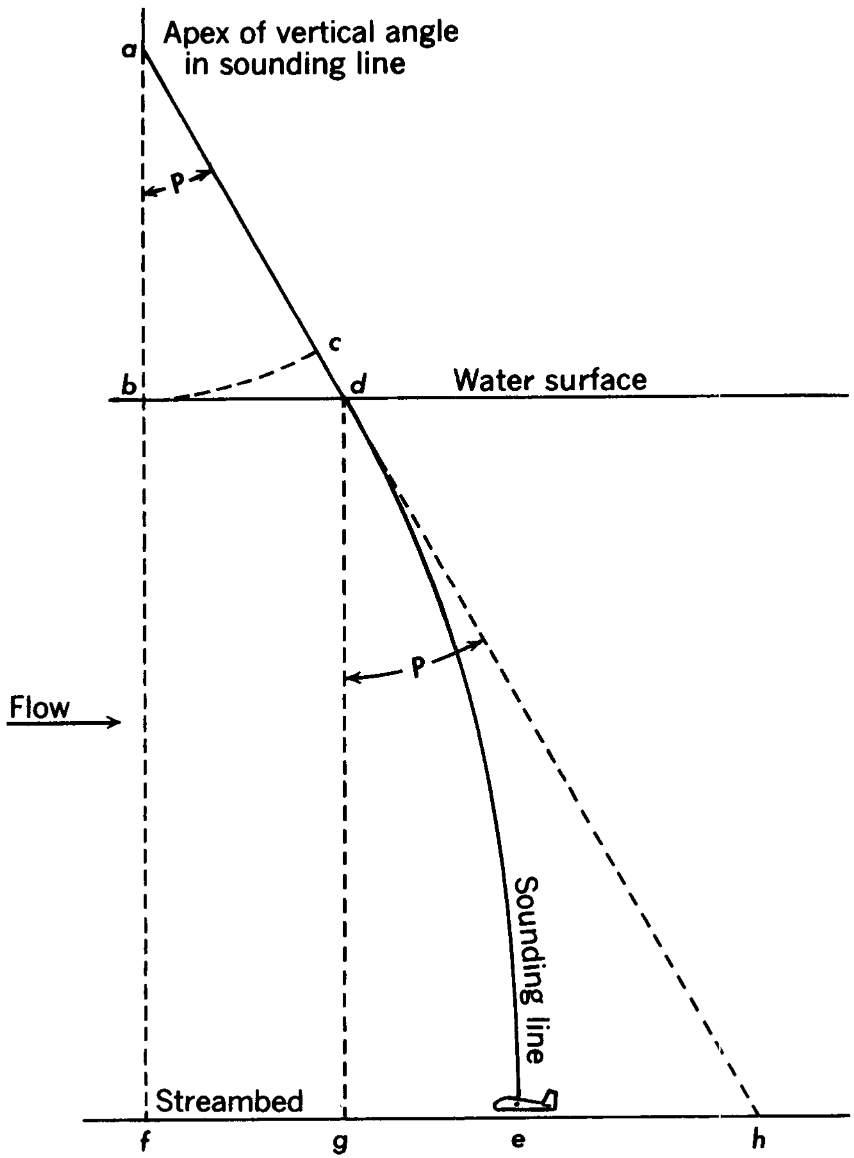


FIGURE 99.—Position of sounding weight and line in deep, swift water.

TABLE 5.—*Air-correction table, giving difference, in feet, between vertical length and slant length of sounding line above water surface for selected vertical angles*

Vertical length (ft)	Vertical angle of sounding line at protractor																	Vertical length (ft)
	4°	6°	8°	10°	12°	14°	16°	18°	20°	22°	24°	26°	28°	30°	32°	34°	36°	
10	0.02	0.06	0.10	0.15	0.22	0.31	0.40	0.51	0.64	0.79	0.95	1.13	1.33	1.55	1.79	2.06	2.36	10
12	.03	.07	.12	.19	.27	.37	.48	.62	.77	.94	1.14	1.35	1.59	1.86	2.15	2.47	2.83	12
14	.03	.08	.14	.22	.31	.43	.56	.72	.90	1.10	1.32	1.58	1.86	2.17	2.51	2.89	3.30	14
16	.04	.09	.16	.25	.36	.49	.64	.82	1.03	1.26	1.51	1.80	2.12	2.48	2.87	3.30	3.78	16
18	.04	.10	.18	.28	.40	.55	.73	.93	1.16	1.41	1.70	2.03	2.39	2.78	3.23	3.71	4.25	18
20	.05	.11	.20	.31	.45	.61	.81	1.03	1.28	1.57	1.89	2.25	2.65	3.09	3.58	4.12	4.72	20
22	.05	.12	.22	.34	.49	.67	.89	1.13	1.41	1.73	2.08	2.48	2.92	3.40	3.94	4.54	5.19	22
24	.06	.13	.24	.37	.54	.73	.97	1.24	1.54	1.88	2.27	2.70	3.18	3.71	4.30	4.95	5.67	24
26	.06	.14	.26	.40	.58	.80	1.05	1.34	1.67	2.04	2.46	2.93	3.45	4.02	4.66	5.36	6.14	26
28	.07	.15	.28	.43	.63	.86	1.13	1.44	1.80	2.20	2.65	3.15	3.71	4.33	5.02	5.77	6.61	28
30	.07	.17	.29	.46	.67	.92	1.21	1.54	1.93	2.36	2.84	3.38	3.98	4.64	5.38	6.19	7.08	30
32	.08	.18	.31	.49	.71	.98	1.29	1.65	2.05	2.51	3.03	3.60	4.24	4.95	5.73	6.60	7.55	32
34	.08	.19	.33	.52	.76	1.04	1.37	1.75	2.18	2.67	3.22	3.83	4.51	5.26	6.09	7.01	8.03	34
36	.09	.20	.35	.56	.80	1.10	1.45	1.85	2.31	2.83	3.41	4.05	4.77	5.57	6.45	7.42	8.50	36
38	.09	.21	.37	.59	.85	1.16	1.53	1.96	2.44	2.98	3.60	4.28	5.04	5.88	6.81	7.84	8.97	38
40	.10	.22	.39	.62	.89	1.22	1.61	2.06	2.57	3.14	3.79	4.50	5.30	6.19	7.17	8.25	9.44	40
42	.10	.23	.41	.65	.94	1.29	1.69	2.16	2.70	3.30	3.97	4.73	5.57	6.50	7.53	8.66	9.91	42
44	.11	.24	.43	.68	.98	1.35	1.77	2.26	2.82	3.46	4.16	4.95	5.83	6.81	7.88	9.07	10.39	44
46	.11	.25	.45	.71	1.03	1.41	1.85	2.37	2.95	3.61	4.35	5.18	6.10	7.12	8.24	9.49	10.86	46
48	.12	.26	.47	.74	1.07	1.47	1.93	2.47	3.08	3.77	4.54	5.40	6.36	7.43	8.60	9.90	11.33	48
50	.12	.28	.49	.77	1.12	1.53	2.02	2.57	3.21	3.93	4.73	5.63	6.63	7.74	8.96	10.31	11.80	50
52	.13	.29	.51	.80	1.16	1.59	2.10	2.68	3.34	4.08	4.92	5.86	6.89	8.04	9.32	10.72	12.28	52
54	.13	.30	.53	.83	1.21	1.65	2.18	2.78	3.47	4.24	5.11	6.08	7.16	8.35	9.68	11.14	12.75	54
56	.14	.31	.55	.86	1.25	1.71	2.26	2.88	3.59	4.40	5.30	6.31	7.42	8.66	10.03	11.55	13.22	56
58	.14	.32	.57	.89	1.30	1.78	2.34	2.98	3.72	4.55	5.49	6.53	7.69	8.97	10.39	11.96	13.69	58
60	.15	.33	.59	.93	1.34	1.84	2.42	3.09	3.85	4.71	5.68	6.76	7.95	9.28	10.75	12.37	14.16	60
62	.15	.34	.61	.96	1.39	1.90	2.50	3.19	3.98	4.87	5.87	6.98	8.22	9.59	11.11	12.79	14.64	62
64	.16	.35	.63	.99	1.43	1.96	2.58	3.29	4.11	5.03	6.06	7.21	8.48	9.90	11.47	13.20	15.11	64
66	.16	.36	.65	1.02	1.47	2.02	2.66	3.40	4.24	5.18	6.25	7.43	8.75	10.21	11.83	13.61	15.58	66
68	.17	.37	.67	1.05	1.52	2.08	2.74	3.50	4.36	5.34	6.44	7.66	9.01	10.52	12.18	14.02	16.05	68
70	.17	.39	.69	1.08	1.56	2.14	2.82	3.60	4.49	5.50	6.62	7.88	9.28	10.83	12.54	14.44	16.52	70
72	.18	.40	.71	1.11	1.61	2.20	2.90	3.71	4.62	5.65	6.81	8.11	9.55	11.14	12.90	14.85	17.00	72
74	.18	.41	.73	1.14	1.65	2.27	2.98	3.81	4.75	5.81	7.00	8.33	9.81	11.45	13.26	15.26	17.47	74
76	.19	.42	.75	1.17	1.70	2.33	3.06	3.91	4.88	5.97	7.19	8.56	10.08	11.76	13.62	15.67	17.94	76
78	.19	.43	.77	1.20	1.74	2.39	3.14	4.01	5.01	6.13	7.38	8.78	10.34	12.07	13.98	16.09	18.41	78
80	.20	.44	.79	1.23	1.79	2.45	3.22	4.12	5.13	6.28	7.57	9.01	10.61	12.38	14.33	16.50	18.89	80
82	.20	.45	.81	1.27	1.83	2.51	3.30	4.22	5.26	6.44	7.76	9.23	10.87	12.69	14.69	16.91	19.36	82
84	.20	.46	.83	1.30	1.88	2.57	3.39	4.32	5.39	6.60	7.95	9.46	11.14	12.99	15.05	17.32	19.83	84
86	.21	.47	.85	1.33	1.92	2.63	3.47	4.43	5.52	6.75	8.14	9.68	11.40	13.30	15.41	17.73	20.30	86
88	.21	.48	.87	1.36	1.97	2.69	3.55	4.53	5.65	6.91	8.33	9.91	11.67	13.61	15.77	18.15	20.77	88
90	.22	.50	.88	1.39	2.01	2.75	3.63	4.63	5.78	7.07	8.52	10.13	11.93	13.92	16.13	18.56	21.25	90
92	.22	.51	.90	1.42	2.06	2.82	3.71	4.73	5.90	7.22	8.71	10.36	12.20	14.23	16.48	18.97	21.72	92
94	.23	.52	.92	1.45	2.10	2.88	3.79	4.84	6.03	7.38	8.90	10.58	12.46	14.54	16.84	19.38	22.19	94
96	.23	.53	.94	1.48	2.14	2.94	3.87	4.94	6.16	7.54	9.09	10.81	12.73	14.85	17.20	19.80	22.66	96
98	.24	.54	.96	1.51	2.19	3.00	3.95	5.04	6.29	7.70	9.27	11.03	12.99	15.16	17.56	20.21	23.13	98
100	.24	.55	.98	1.54	2.23	3.06	4.03	5.15	6.42	7.85	9.46	11.26	13.26	15.47	17.92	20.62	23.61	100

TABLE 6.—*Air-correction table, giving difference, in meters, between vertical length and slant length of sounding line above water surface for selected vertical angles*

Vertical length (m)	Vertical angle of sounding line at protractor (degrees)																											
	5	8	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35
1	0.00	0.01	0.02	0.02	0.02	0.03	0.03	0.04	0.04	0.05	0.05	0.06	0.06	0.07	0.08	0.09	0.09	0.10	0.11	0.12	0.13	0.14	0.16	0.17	0.18	0.19	0.21	0.22
2	.01	.02	.03	.04	.04	.05	.06	.07	.08	.09	.10	.11	.13	.14	.16	.17	.19	.21	.22	.24	.26	.29	.31	.33	.36	.38	.41	.44
3	.01	.03	.04	.05	.07	.08	.09	.10	.12	.14	.16	.17	.19	.21	.24	.26	.28	.31	.34	.37	.40	.43	.46	.50	.54	.58	.62	.66
4	.02	.04	.06	.07	.09	.11	.12	.14	.16	.18	.21	.23	.26	.28	.32	.34	.38	.42	.45	.49	.53	.57	.62	.67	.72	.77	.82	.88
5	.02	.05	.08	.09	.11	.14	.16	.18	.20	.23	.26	.29	.32	.35	.40	.43	.47	.52	.56	.61	.66	.72	.78	.84	.90	.96	1.03	1.10
6	.05	.05	.10	.10	.15	.15	.20	.20	.25	.30	.30	.35	.40	.45	.45	.50	.55	.60	.65	.75	.80	.85	.95	1.00	1.05	1.15	1.25	1.35
7	.05	.05	.10	.15	.15	.20	.20	.25	.30	.30	.35	.40	.45	.50	.55	.60	.65	.75	.80	.85	.90	1.00	1.10	1.15	1.25	1.35	1.45	1.55
8	.05	.10	.10	.15	.20	.20	.25	.30	.35	.35	.40	.45	.50	.55	.65	.70	.75	.85	.90	1.00	1.05	1.15	1.25	1.35	1.45	1.55	1.65	1.75
9	.05	.10	.15	.15	.20	.25	.30	.30	.35	.40	.45	.50	.60	.65	.70	.75	.85	.95	1.00	1.10	1.20	1.30	1.40	1.50	1.60	1.75	1.85	2.00
10	.05	.10	.15	.20	.20	.25	.30	.35	.40	.45	.50	.55	.65	.70	.80	.85	.95	1.05	1.10	1.20	1.30	1.45	1.55	1.65	1.80	1.90	2.05	2.20
11	.1	.1	.2	.2	.2	.3	.3	.4	.4	.5	.6	.6	.7	.8	.9	1.0	1.0	1.1	1.2	1.3	1.4	1.6	1.7	1.8	2.0	2.1	2.3	2.4
12	.1	.1	.2	.2	.3	.3	.4	.4	.5	.6	.6	.7	.8	.8	1.0	1.0	1.1	1.2	1.3	1.5	1.6	1.7	1.9	2.0	2.2	2.3	2.5	2.6
13	.1	.1	.2	.2	.3	.4	.4	.5	.5	.6	.7	.7	.9	.9	1.0	1.1	1.2	1.4	1.5	1.6	1.7	1.9	2.0	2.2	2.3	2.5	2.7	2.9
14	.1	.1	.2	.2	.3	.4	.4	.5	.6	.6	.7	.8	.9	1.0	1.1	1.2	1.3	1.5	1.6	1.7	1.8	2.0	2.2	2.3	2.5	2.7	2.9	3.1
15	.1	.2	.2	.3	.3	.4	.5	.5	.6	.7	.8	.9	1.0	1.1	1.2	1.3	1.4	1.6	1.7	1.8	2.0	2.1	2.3	2.5	2.7	2.9	3.1	3.3
16	.1	.2	.2	.3	.4	.4	.5	.6	.7	.7	.8	.9	1.0	1.1	1.3	1.4	1.5	1.7	1.8	2.0	2.1	2.3	2.5	2.7	2.8	3.1	3.3	3.5
17	.1	.2	.3	.3	.4	.5	.5	.6	.7	.8	.9	1.0	1.1	1.2	1.3	1.5	1.6	1.8	1.9	2.1	2.2	2.4	2.6	2.8	3.0	3.3	3.5	3.8
18	.1	.2	.3	.3	.4	.5	.6	.6	.7	.8	.9	1.0	1.2	1.3	1.4	1.5	1.7	1.9	2.0	2.2	2.4	2.6	2.8	3.0	3.2	3.5	3.7	4.0
19	.1	.2	.3	.3	.4	.5	.6	.7	.8	.9	1.0	1.1	1.2	1.4	1.5	1.6	1.8	2.0	2.1	2.3	2.5	2.7	2.9	3.2	3.4	3.6	3.9	4.2
20	.1	.2	.3	.4	.4	.5	.6	.7	.8	.9	1.0	1.1	1.3	1.4	1.6	1.7	1.9	2.1	2.2	2.4	2.6	2.9	3.1	3.3	3.6	3.8	4.1	4.4

selected intervals on the sounding line, and then using the tags to reference the water surface. This practice is almost equivalent to moving the reel to a position just above the water surface.

The correction for excess length of line below the water surface is obtained by using an elementary principle of mechanics. If a known horizontal force is applied to a weight suspended on a cord, the cord takes a position of rest at some angle with the vertical, and the tangent of the vertical angle of the cord is equal to the horizontal force divided by the vertical force of the weight. If several additional horizontal and vertical forces are applied to the cord, the tangent of the angle in the cord above any point is equal to a summation of the horizontal forces below that point, divided by the summation of the vertical forces below the point.

The distribution of total horizontal drag on the sounding line is in accordance with the variation of velocity with depth. The excess in length of the curved line over the vertical depth is the sum of the products of each tenth of depth and the function $(1/\cos P)-1$ of the corresponding angles; the function is derived for each tenth of depth by means of the tangent relation of the forces acting below any point.

The wet-line correction for even-numbered angles between 4° and 36° and for wet-line depths between 10 and 100 ft is shown in table 7. (Table 8 is a similar table in metric units.) The correction is applied to the nearest tenth of a foot. The wet-line correction cannot be determined until the air correction has been deducted from the observed depth.

The following assumptions were used in deriving the wet-line correction tables:

1. The weight will go to the bottom despite the force of the current.
2. The sounding is made when the weight is at the bottom but entirely supported by the line.
3. Drag on the streamlined weight in the sounding position is neglected.
4. The table is general and can be used for any size sounding weight or line that is designed to offer little resistance to the current.

If the direction of flow is not perpendicular to the measuring section, the angle of the measuring line as indicated by the protractor will be less than the true angle of the line. The air correction and wet-line correction will then be too small. To correct for this it is necessary to either measure by protractor the horizontal angle between the direction of flow and a perpendicular to the measurement section, or determine the horizontal-angle coefficient by the methods described on pages 142–143.

If the horizontal angle of the direction of flow is called H, the measured vertical angle P, and the true vertical angle X, the relation

TABLE 7.—*Wet-line table, giving difference, in feet, between wet-line length and vertical depth for selected vertical angles*

Wet-line length (ft)	Vertical angle of sounding line at protractor																	Wet-line length (ft)
	4°	6°	8°	10°	12°	14°	16°	18°	20°	22°	24°	26°	28°	30°	32°	34°	36°	
10	0.01	0.02	0.03	0.05	0.07	0.10	0.13	0.16	0.20	0.25	0.30	0.35	0.41	0.47	0.54	0.62	0.70	10
12	.01	.02	.04	.06	.09	.12	.15	.20	.24	.30	.36	.42	.49	.57	.65	.74	.84	12
14	.01	.02	.04	.07	.10	.14	.18	.23	.29	.35	.41	.49	.57	.66	.76	.87	.98	14
16	.01	.03	.05	.08	.12	.16	.20	.26	.33	.40	.47	.56	.65	.76	.87	.99	1.12	16
18	.01	.03	.06	.09	.13	.18	.23	.30	.37	.45	.53	.63	.73	.85	.98	1.12	1.26	18
20	.01	.03	.06	.10	.14	.20	.26	.33	.41	.50	.59	.70	.82	.94	1.09	1.24	1.40	20
22	.01	.04	.07	.11	.16	.22	.28	.36	.45	.55	.65	.77	.90	1.04	1.20	1.36	1.54	22
24	.01	.04	.08	.12	.17	.24	.31	.39	.49	.60	.71	.84	.98	1.13	1.31	1.49	1.68	24
26	.02	.04	.08	.13	.19	.25	.33	.43	.53	.64	.77	.91	1.06	1.23	1.41	1.61	1.81	26
28	.02	.04	.09	.14	.20	.27	.36	.46	.57	.69	.83	.98	1.14	1.32	1.52	1.74	1.95	28
30	.02	.05	.10	.15	.22	.29	.38	.49	.61	.74	.89	1.05	1.22	1.42	1.63	1.86	2.09	30
32	.02	.05	.10	.16	.23	.31	.41	.52	.65	.79	.95	1.12	1.31	1.51	1.74	1.98	2.23	32
34	.02	.05	.11	.17	.24	.33	.44	.56	.69	.84	1.01	1.19	1.39	1.60	1.85	2.11	2.37	34
36	.02	.06	.12	.18	.26	.35	.46	.59	.73	.89	1.07	1.26	1.47	1.70	1.96	2.23	2.51	36
38	.02	.06	.12	.19	.27	.37	.49	.62	.78	.94	1.12	1.33	1.55	1.79	2.07	2.36	2.65	38
40	.02	.06	.13	.20	.29	.39	.51	.66	.82	.99	1.18	1.40	1.63	1.89	2.18	2.48	2.79	40
42	.03	.07	.13	.21	.30	.41	.54	.69	.86	1.04	1.24	1.47	1.71	1.98	2.28	2.60	2.93	42
44	.03	.07	.14	.22	.32	.43	.56	.72	.90	1.09	1.30	1.54	1.80	2.08	2.39	2.73	3.07	44
46	.03	.07	.15	.23	.33	.45	.59	.75	.94	1.14	1.36	1.61	1.88	2.17	2.50	2.85	3.21	46
48	.03	.08	.15	.24	.35	.47	.61	.79	.98	1.19	1.42	1.68	1.96	2.27	2.61	2.98	3.35	48
50	.03	.08	.16	.25	.36	.49	.64	.82	1.02	1.24	1.48	1.75	2.04	2.36	2.72	3.10	3.49	50
52	.03	.08	.17	.26	.37	.51	.67	.85	1.06	1.29	1.54	1.82	2.12	2.45	2.83	3.22	3.63	52
54	.03	.09	.17	.27	.39	.53	.69	.89	1.10	1.34	1.60	1.89	2.20	2.55	2.94	3.35	3.77	54
56	.03	.09	.18	.28	.40	.55	.72	.92	1.14	1.39	1.66	1.96	2.28	2.64	3.05	3.47	3.91	56
58	.03	.09	.19	.29	.42	.57	.74	.95	1.18	1.44	1.72	2.03	2.37	2.74	3.16	3.60	4.05	58
60	.04	.10	.19	.30	.43	.59	.77	.98	1.22	1.49	1.78	2.10	2.45	2.83	3.26	3.72	4.19	60
62	.04	.10	.20	.31	.45	.61	.79	1.02	1.26	1.54	1.84	2.17	2.53	2.93	3.37	3.84	4.33	62
64	.04	.10	.20	.32	.46	.63	.82	1.05	1.31	1.59	1.89	2.24	2.61	3.02	3.48	3.97	4.47	64
66	.04	.11	.21	.33	.48	.65	.84	1.08	1.35	1.64	1.95	2.31	2.69	3.12	3.59	4.09	4.61	66
68	.04	.11	.22	.34	.49	.67	.87	1.12	1.39	1.69	2.01	2.38	2.77	3.21	3.70	4.22	4.75	68
70	.04	.11	.22	.35	.50	.69	.90	1.15	1.43	1.74	2.07	2.45	2.86	3.30	3.81	4.34	4.89	70
72	.04	.12	.23	.36	.52	.71	.92	1.18	1.47	1.79	2.13	2.52	2.94	3.40	3.92	4.46	5.03	72
74	.04	.12	.24	.37	.53	.73	.95	1.21	1.51	1.84	2.19	2.59	3.02	3.49	4.03	4.59	5.17	74
76	.05	.12	.24	.38	.55	.74	.97	1.25	1.55	1.88	2.25	2.66	3.10	3.59	4.13	4.71	5.30	76
78	.05	.12	.25	.39	.56	.76	1.00	1.28	1.59	1.93	2.31	2.73	3.18	3.68	4.24	4.84	5.44	78
80	.05	.13	.25	.40	.58	.78	1.02	1.31	1.63	1.98	2.37	2.80	3.26	3.78	4.35	4.96	5.58	80
82	.05	.13	.26	.41	.59	.80	1.05	1.34	1.67	2.03	2.43	2.87	3.35	3.87	4.46	5.08	5.72	82
84	.05	.13	.27	.42	.60	.82	1.08	1.38	1.71	2.08	2.49	2.94	3.43	3.96	4.57	5.21	5.86	84
86	.05	.14	.28	.43	.62	.84	1.10	1.41	1.75	2.13	2.55	3.01	3.51	4.06	4.68	5.33	6.00	86
88	.05	.14	.28	.44	.63	.86	1.13	1.44	1.80	2.18	2.60	3.08	3.59	4.15	4.79	5.46	6.14	88
90	.05	.14	.29	.45	.65	.88	1.15	1.48	1.84	2.23	2.66	3.15	3.67	4.25	4.90	5.58	6.28	90
92	.06	.15	.29	.46	.66	.90	1.18	1.51	1.88	2.28	2.72	3.22	3.75	4.34	5.00	5.70	6.42	92
94	.06	.15	.30	.47	.68	.92	1.20	1.54	1.92	2.33	2.78	3.29	3.84	4.44	5.11	5.83	6.56	94
96	.06	.15	.31	.48	.69	.94	1.23	1.57	1.96	2.38	2.84	3.36	3.92	4.53	5.22	5.95	6.70	96
98	.06	.16	.31	.49	.71	.96	1.25	1.61	2.00	2.43	2.90	3.43	4.00	4.63	5.33	6.08	6.84	98
100	.06	.16	.32	.50	.72	.98	1.28	1.64	2.04	2.48	2.96	3.50	4.08	4.72	5.44	6.20	6.98	100

TABLE 8.—*Wet-line table, giving difference, in meters, between wet-line length and vertical depth for selected vertical angles*

Wet-line length (m)	Vertical angle of sounding line at protractor (degrees)												
	10	11	12	13	14	15	16	17	18	19	20	21	22
1	0.01	0.01	0.02	0.02	0.02	0.03	0.03	0.03	0.03	0.04	0.04	0.04	0.05
2	.02	.02	.03	.03	.04	.04	.05	.05	.06	.06	.07	.08	.08
3	.03	.04	.04	.05	.05	.06	.06	.07	.08	.08	.09	.10	.11
4	.03	.04	.04	.05	.06	.07	.08	.09	.10	.11	.12	.13	.14
5	.04	.05	.05	.05	.07	.08	.09	.10	.11	.13	.14	.15	.17
6	.05	.05	.05	.05	.10	.10	.10	.10	.15	.15	.15	.15	.20
7	.05	.05	.05	.10	.10	.10	.10	.15	.15	.15	.20	.20	.25
8	.05	.05	.10	.10	.10	.10	.15	.15	.15	.20	.20	.25	.25
9	.05	.10	.10	.10	.10	.15	.15	.20	.20	.20	.25	.25	.30
10	.05	.10	.10	.10	.15	.15	.15	.20	.20	.25	.25	.30	.30
11	.1	.1	.1	.1	.2	.2	.2	.2	.2	.3	.3	.3	.3
12	.1	.1	.1	.1	.2	.2	.2	.2	.3	.3	.3	.3	.4
13	.1	.1	.1	.1	.2	.2	.2	.2	.3	.3	.3	.4	.4
14	.1	.1	.1	.1	.2	.2	.2	.3	.3	.3	.4	.4	.4
15	.1	.1	.1	.2	.2	.2	.2	.3	.3	.3	.4	.4	.5
16	.1	.1	.1	.2	.2	.2	.3	.3	.3	.4	.4	.4	.5
17	.1	.1	.2	.2	.2	.2	.3	.3	.3	.4	.4	.5	.5
18	.1	.1	.2	.2	.2	.3	.3	.3	.4	.4	.5	.5	.6
19	.1	.1	.2	.2	.2	.3	.3	.3	.4	.4	.5	.5	.6
20	.1	.2	.2	.2	.2	.3	.3	.4	.4	.5	.5	.6	.6

Wet-line length (m)	Vertical angle of sounding line at protractor (degrees)												
	23	24	25	26	27	28	29	30	31	32	33	34	35
1	0.05	0.06	0.06	0.07	0.07	0.08	0.08	0.09	0.10	0.10	0.11	0.12	0.12
2	.09	.09	.10	.11	.11	.12	.13	.14	.15	.16	.17	.18	.19
3	.12	.13	.14	.15	.16	.18	.19	.20	.21	.23	.24	.26	.27
4	.16	.17	.18	.19	.21	.22	.24	.25	.27	.28	.30	.32	.34
5	.18	.20	.22	.24	.26	.27	.29	.31	.33	.35	.38	.40	.42
6	.20	.25	.25	.25	.30	.30	.35	.35	.40	.40	.45	.45	.50
7	.25	.25	.30	.30	.35	.35	.40	.40	.45	.50	.50	.55	.55
8	.30	.30	.35	.35	.40	.40	.45	.45	.50	.55	.55	.60	.65
9	.30	.35	.35	.40	.45	.45	.50	.55	.55	.60	.65	.70	.70
10	.35	.35	.40	.45	.45	.50	.55	.60	.60	.65	.70	.75	.80
11	.4	.4	.4	.5	.5	.6	.6	.6	.7	.7	.8	.8	.9
12	.4	.4	.5	.5	.6	.6	.6	.7	.7	.8	.8	.9	.9
13	.4	.5	.5	.6	.6	.7	.7	.8	.8	.9	.9	1.0	1.0
14	.5	.5	.6	.6	.7	.7	.8	.8	.9	.9	1.0	1.0	1.1
15	.5	.5	.6	.6	.7	.7	.8	.8	.9	1.0	1.0	1.1	1.2
16	.5	.6	.6	.7	.7	.8	.9	.9	1.0	1.1	1.1	1.2	1.2
17	.6	.6	.7	.7	.8	.8	.9	1.0	1.0	1.1	1.2	1.2	1.3
18	.6	.7	.7	.8	.8	.9	1.0	1.0	1.1	1.2	1.2	1.3	1.4
19	.6	.7	.7	.8	.9	.9	1.0	1.1	1.2	1.2	1.3	1.4	1.5
20	.7	.7	.8	.8	.9	1.0	1.1	1.1	1.2	1.3	1.4	1.5	1.6

between the angles is expressed by the equation

$$\tan X = \frac{\tan P}{\cos H} \text{ (fig. 100).} \tag{12}$$

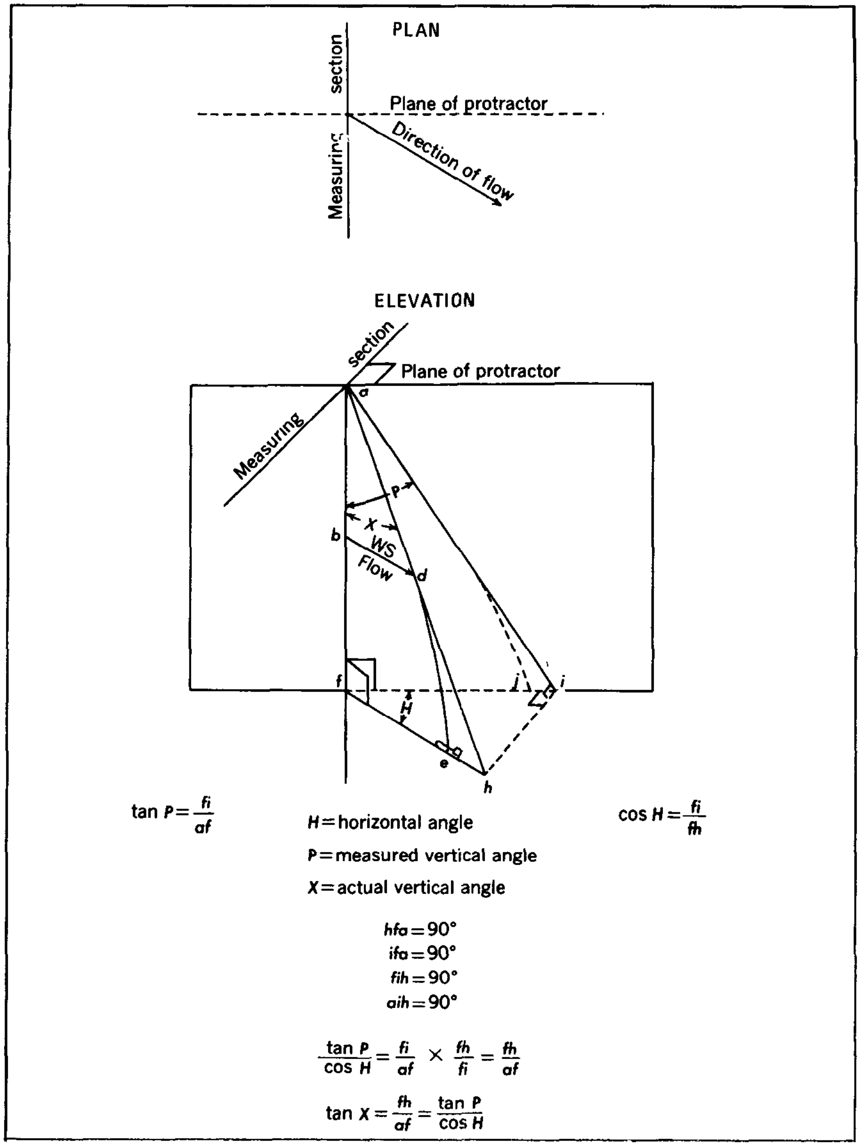


FIGURE 100.—Sketch of geometry of relation of actual to measured vertical angle when flow direction is not normal to measurement section.

Table 9 gives the quantities in tenths of degrees, to be added to observed vertical angles to obtain the true vertical angles for a range of horizontal angles between 8° and 28°.

The conditions that cause error in sounding the depth also cause error in setting the meter at selected depths. The correction tables are not strictly applicable to the problem of setting the meter for velocity observations because of the increased horizontal force on the sounding weight caused by higher velocities when the weight is raised from the streambed. A meter placed in deep, swift water by the ordinary methods for observations at selected percentages of the depth will be too high in the water. The use of tables 5–9 will tend to eliminate this error in the placement of the meter, and although not strictly applicable, their use for this purpose has become general.

For the 0.2-depth position, the curvature of the wet line is assumed to be negligible, and the length of sounding line from the apex of the vertical angle to the weight is considered a straight line. The method used to place the meter at the 0.2-depth position is as follows:

1. Compute the 0.2 value of the vertical depth.
2. Lower the meter this depth into the water and read the vertical angle.
3. Obtain the air correction from table 5 or 6. The vertical length used to obtain the air correction is the sum of (a) 0.2 of the vertical depth, (b) the distance from the water surface to the apex of the angle, and (c) the distance from the bottom of the weight to the meter.
4. Let out an additional amount of line equal to the air correction.
5. If the angle increases appreciably when the additional line is let out, let out more line until the total additional line, the angle, and the vertical distance are in agreement with figures in the air-correction table.

To set the meter at the 0.8-depth position, a correction to the amount of line reeled in must be made for the difference, if any, between the

TABLE 9.—*Degrees to be added to observed vertical angles to obtain actual vertical angles when flow direction is not normal to measurement section*

Observed vertical angle	Horizontal angle					
	8° cos=0.99	12° cos=0 98	16° cos=0 96	20° cos=0 94	24° cos=0 91	28° cos=0.88
8°	0.1	0.2	0.3	0.5	0.8	1.0
12°	.1	.3	.5	.8	1.1	1.5
16°	.1	.4	.6	1.0	1.4	2.0
20°	.2	.4	.7	1.2	1.7	2.4
24°	.2	.5	.8	1.4	2.0	2.8
28°	.2	.5	1.0	1.5	2.2	3.0
32°	.2	.6	1.0	1.6	2.4	3.3
36°	.2	.6	1.1	1.7	2.5	3.4

air correction for the sounding position and that for the 0.8-depth position. This difference is designated as m in table 10. If the angle increases for the 0.8-depth position, the meter must be lowered; if the angle decreases the meter must be raised.

In setting the 0.8-depth position of the meter, the wet-line correction may require consideration if the depths are more than 40 ft and if the change in vertical angle is more than 5 percent. If the vertical angle remains the same or decreases, the wet-line correction (table 7 or 8) for the 0.8-depth position is less than the wet-line correction for the sounding position by some difference designated as n in table 10. If the vertical angle increases, the difference in correction n deminishes until the increase in angle is about 10 percent; for greater increases in angle, the difference between corrections also increases. Table 10 summarizes the effect on air- and wet-line corrections caused by raising the meter from the sounding position to the 0.8-depth position.

For slight changes in the vertical angle, because of the differences m and n in the air- and wet-line corrections, the adjustments to the wet-line length of the 0.8-depth position are generally small and usually can be ignored. Table 10 indicates, however, that the meter may be placed a little too low if the adjustments are not made. Because of this possibility, the wet-line depth instead of the vertical depth is sometimes used as the basis for computing the 0.8-depth position, and no adjustments are made for the differences m and n.

CASE B. DEPTH CANNOT BE SOUNDED BUT STANDARD CROSS SECTION IS AVAILABLE

On occasion it is not possible to sound the bottom, but a standard measurement cross section at the bridge or cableway may be available from previous measurements that were made. Such a cross section

TABLE 10.—*Summary table for setting the meter at 0.8-depth position in deep, swift streams*

Change in vertical angle	Air correction		Wet-line correction	
	Direction of change	Correction to meter position	Direction of change	Correction to meter position
None	None	None	Decrease	Raise meter the distance n.
Decrease	Decrease	Raise meter the distance m.	do	Do.
Increase	Increase	Lower meter the distance m.	Decrease, then increase.	([1])

[1]Raise meter the distance n unless the increase in angle is greater than about 10 percent, then it is necessary to lower the meter the distance n.

will be useful only if all discharge measurements use the same permanent initial point for the stationing of verticals across the width of the stream, and if there is an outside reference gage or reference point on the bank or bridge to which the water-surface elevation at the measurement cross section may be referred. In the situation described above, the following procedure is used:

1. Determine depths at the observation verticals from the standard cross section and the known water-surface elevation at the measurement cross section.
2. Measure the velocity at 0.2 of the depth.
3. Compute the measurement in the normal manner using the measured velocities as though they were the mean velocities in the vertical, and using the depths from step 1.
4. Determine the coefficient to adjust the 0.2-depth velocity to mean velocity in the cross section, as explained on pages 135–136.
5. Apply the coefficient from step 4 to the computed discharge from step 3.

CASE C. DEPTH CANNOT BE SOUNDED AND NO STANDARD CROSS SECTION IS AVAILABLE

When it is not possible to sound the depth and a standard cross section is not available, the following procedure is used:

1. Refer the water-surface elevation before and after the measurement to an elevation reference point on a bridge, on a driven stake, or on a tree at the water's edge. (It is assumed here that no outside reference gage is available at the measurement cross section.)
2. Estimate the depth and observe the velocity at 0.2 of the estimated depth. The meter should be at least 2.0 ft (0.6 m) below the water surface. Record in the notes the actual depth the meter was placed below the water surface. If an estimate of the depth is impossible, place the meter 2.0 ft below the water surface and observe the velocity there.
3. Make a complete measurement at a lower stage and include some vertical-velocity curves.
4. Use the complete measurement and difference in stage between the two measurements to determine the cross section of the first measurement. To determine whether the streambed has shifted, the cross section should be compared with one obtained in a previous measurement at that site.
5. Use vertical-velocity curves, or the relationship between mean velocity and 0.2-depth velocity, to adjust the velocities observed in step 2 to mean velocity.
6. Compute the measurement in the normal manner using the depths from step 4 and the velocities from step 5.

CASE D. METER CANNOT BE SUBMERGED

If it is impossible to keep the meter and weight in the water because of high velocities and (or) floating drift, use the following procedure:

1. Obtain depths at the measurement verticals by the method explained for case B if a standard cross section is available, or by the method explained above for case C if no standard cross section is available.
2. Measure surface velocities with an optical current meter, as explained on pages 91–93, 137–138.
3. Compute the measurement in the normal manner using the surface velocities as though they were the mean velocities in the vertical, and using the depths from step 1.
4. Apply the appropriate velocity coefficient to the discharge computed in step 3; use a coefficient of 0.86 for a natural channel and 0.90 for an artificial channel.

If an optical current meter is not available, time floating drift over a measured course. (See p. 261–262.)

It should be noted here that the amount of floating drift or ice is usually greatly reduced just after the crest of a rise in stage. It may be possible at that time to obtain velocity observations with a standard current meter.

COMPUTATION OF MEAN GAGE HEIGHT OF A DISCHARGE MEASUREMENT

The mean gage height of a discharge measurement represents the mean stage of the stream during the measurement period. Because the mean gage height for a discharge measurement is one of the coordinates used in plotting the measurements to establish the stage-discharge relation, an accurate determination of the mean gage height is as important as an accurate measurement of the discharge. The computation of the mean gage height presents no problem when the change in stage is uniform and no greater than about 0.15 ft (0.05 m), for then the mean may be obtained by averaging the stage at the beginning and end of the measurement. However, measurements must often be made during periods when the change of stage is neither uniform nor slight.

As a prerequisite for obtaining an accurate mean gage height, the clock time at the beginning and end of the measurement should be recorded on the measurement notes, and additional readings of the clock time should be recorded on the notes at intervals of 15 to 20 min during the measurement. After the discharge measurement has been completed, the recorder chart should be read, and breaks in the slope of the gage-height graph that occurred during the measurement should be noted The breaks in slope are useful in themselves and are also used to determine the gage height corresponding to the clock times noted during the measurement. If the station is equipped with a digital recorder, the gage-height readings punched during the measurement are to be read. At nonrecording stations the only way to obtain intermediate readings is for the stream gager to stop a few times during the measurement to read the gage, or to have someone else do this for him.

If the change in stage is greater than 0.15 ft (0.05 m) or if the change in stage has not been uniform, the mean gage height is obtained by weighting the gage heights corresponding to the clock-time observations. The weighting is done by using either partial discharge or time as the weighting factor. In the past the weighting in the U.S.A. was always done on the basis of partial discharges, but recent study indicates that discharge-weighting usually tends to overestimate the mean gage height, whereas time-weighting usually tends to underestimate the mean gage height. On the basis of the present state of our knowledge, it is suggested that the mean gage height for a discharge measurement be computed by both methods, after which the two results are averaged. A description of the two methods follows.

In the discharge-weighting process, the partial discharges measured between clock observations of gage height are used with the mean gage heights for the periods when the partial discharges were measured. The formula used to compute mean gage height is

$$H = \frac{q_1h_1 + q_2h_2 + q_3h_3 \ldots\ldots\ldots + q_nh_n}{Q}, \tag{13}$$

in which

H = mean gage height (ft or m),

Q = total discharge measured (ft^3/s or m^3/s) = $q_1 + q_2 + q_3 \ldots. + q_n$,

where

$q_1, q_2, q_3, \ldots. q_n$ = discharge (ft^3/s or m^3/s) measured during time interval 1, 2, 3, ... n and

$h_1, h_2, h_3, \ldots. h_n$ = average gage height (ft or m) during time interval 1, 2, 3, ... n.

Figure 101 shows the computation of a discharge-weighted mean gage height. The graph at the bottom of figure 101 is a reproduction of the gage-height graph during the discharge measurement. The discharges are taken from the current-meter measurement notes shown in figure 42. The upper computation of the mean gage height in figure 101 shows the computation using equation 13. The lower computation has been made by a shortcut method to eliminate the multiplication of large numbers. In that method, after the average gage height for each time interval has been computed, a base gage height, which is usually equal to the lowest average gage height, is chosen. Then, the

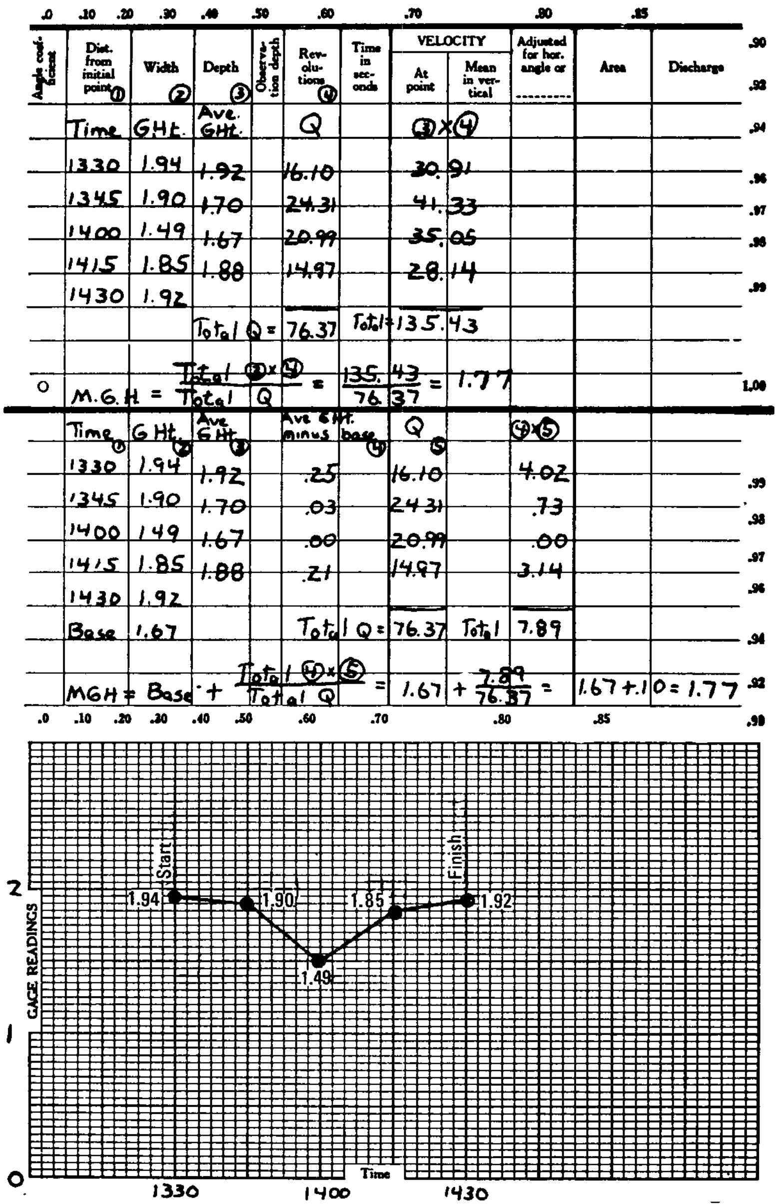


FIGURE 101.—Computation of discharge-weighted mean gage height.

differences between the base gage height and the average gage heights are used to weight the discharges. When the mean difference has been computed, the base gage height is added to it.

In the time-weighting process, the arithmetic mean gage height for time intervals between breaks in the slope of the gage-height graph are used with the duration of those time periods. The formula used to compute mean gage height is

$$H = \frac{t_1h_1 + t_2h_2 + t_3h_3 \ldots\ldots + t_nh_n}{T} \qquad (14)$$

in which

H = mean gage height, in feet or meters,

T = total time for the measurement, in minutes = $t_1 + t_2 + t_3 \ldots\ldots t_n$,

$t_1, t_2, t_3 \ldots\ldots t_n$ = duration of time intervals between breaks in slope of the gage-height graph, in minutes, and

$h_1, h_2, h_3 \ldots\ldots h_n$ = average gage height, in feet or meters, during time interval 1, 2, 3, ... n.

Using the data from figure 101, the computation of the time-weighted mean gage height is as follows:

Average gage height (h)	*Time interval* (t)	$h \times t$
1.92	15	28.80
1.70	15	25.50
1.67	15	25.05
1.88	15	28.20
Total	60	107.55

Mean gage height = 107.55/60 = 1.79 ft

In the example used above there is little difference between the discharge-weighted mean gage height (1.77 ft) and the time-weighted mean gage height (1.79 ft); the average of the two values, 1.78 ft, is the preferred mean gage height for the discharge measurement.

When extremely rapid changes in stage occur during a measurement, the weighted mean gage height is not truly applicable to the discharge measured. To reduce the range in stage during the measurement, measurements under those conditions should be made more rapidly than those made under constant or slowly changing stage. It should be realized, however, that shortcuts in the measurement procedure usually reduce the accuracy of the measured discharge. Therefore measurement procedures during rapidly changing stage must be optimized to produce a minimal combined error in measured discharge and computed mean gage height.

MEASUREMENT PROCEDURES DURING RAPIDLY CHANGING STAGE

The preceding discussion on computing the mean gage height of discharge measurements demonstrated that under conditions of rapidly changing stage, measurement procedures must be streamlined, even at the expense of some accuracy. The reduction in measurement time makes it possible to obtain a gage-height value that is representative of the measured discharge. Where streams are uncontrolled, flood rises are more rapid on small streams than on large streams, because small streams are subject to flash floods that may rise and fall with sufficient rapidity to produce peak flows of almost momentary duration. Consequently the discussion that follows distinguishes between the procedures to be followed for measuring large streams and those for small streams, during periods of rapidly changing stage. The procedure to be followed for measuring streams whose flow is controlled by hydroelectric powerplants was discussed on page 140.

CASE A. LARGE STREAMS

During periods of rapidly changing stage on large streams, the time consumed in making a discharge measurement may be reduced by modifying the standard measurement procedure in the following manner:

1. Use the 0.6-depth method (p. 134). The 0.2-depth method (p. 135) or the subsurface method (p. 136) may be used if placing the meter at the 0.6-depth creates vertical angles requiring time-consuming corrections, or if the vertical angle increases because of drift collecting on the sounding line.
2. Reduce the velocity-observation time to about 20–30 s.
3. Reduce the number of sections taken to about 15–18.

By incorporating all three of the above practices a measurement can be made in 15–20 min. If the subsurface method of observing velocities is used, some vertical-velocity curves will be needed later to establish coefficients to convert observed velocity to mean velocity.

Carter and Anderson (1963) have shown that discharge measurements having 30 verticals, for which the two-point method of observation was used with a 45-s period of observation, will have a standard error of 2.2 percent (see p. 181–183). That means that two-thirds of the measurements made using standard procedures would be in error by 2.2 percent or less. They have also shown that the standard error for a 25-s period of observation, using the 0.6-depth method with depth and velocity observed at 16 verticals, is 4.2 percent. The error caused by using the shortcut method is generally less than the error to be expected as a result of the shifting flow patterns that commonly occur during periods of rapidly changing stage, and in addition, uncertainty concerning the appropriate mean gage height for the measurement is eliminated.

CASE B. SMALL STREAMS

The discussion that follows deals with the measurement of flash floods on small streams. Flash floods begin and end with such abruptness that if the flow is to be measured, the hydrographer must have advance warning of the occurrence of such an event. The warning will enable him to reach the measuring site and make all necessary preparations for current-meter measurements before the stream starts to rise at the site. Once the rise begins it is essential that the many point observations required be made as rapidly as possible because of the rapidly changing discharge.

After arriving at the measuring site where the flash flood is expected, the hydrographer first marks the location of the observation verticals he intends to use. These marks are placed on the bridge rail or cableway that is used for discharge measurements. He then determines the elevation of the streambed, referred to gage datum, at those verticals. That is done both to save time during the actual discharge measurement and because he may be unable to sound the streambed when the flood is in progress. An auxiliary staff gage that can be read from the measuring bridge or cableway should be part of the gaging equipment.

In measuring the discharge during a flash flood, the procedure differs in the following ways from that used in making a conventional current-meter discharge measurement.

1. *Use 6 to 10 observation verticals in the measurement cross section.*—The actual number of verticals used will depend on the width and uniformity of the cross section. Current-meter observations are started when the stage starts to rise and are continued until the flow recedes to normal, or near-normal stage. After completing one traverse of the cross section, the next traverse is started immediately in the opposite direction, and observations continue to be made back and forth across the stream.

2. *Time is saved by making a single velocity observation at each observation vertical.*—If depths, velocities, and the absence of floating drift permit, the 0.6-depth method (p. 134) or 0.2-depth method (p. 135) is used. Otherwise, an optical current meter is used in the surface-velocity method (p. 137).

3. *Readings of the auxiliary staff gage are made at every third velocity observation and clock time is also recorded.*—That is done because the rapid change in stage will commonly make it impossible to later obtain accurate stages, corresponding to the time of each velocity

observation, from the automatic gaging-station record. Furthermore, during periods of rapidly changing stage, a staff-gage record is usually more reliable than an automatic-gage record because of "drawdown" at the intake or because of well or intake lag. Moreover if the gaging station is equipped with a digital recorder, the frequency of punches will seldom be adequate for a flash flood.

After the stream has receded, determinations of streambed elevation at the observation verticals are again made to learn if scour or fill has occurred. If there has been a change in streambed elevation, the change is prorated with time, or in accordance with the best judgment of the hydrographer, to provide the values of depth needed to compute discharge. The most reliable discharge results are obtained, of course, where the streambed is stable or relatively so, leaving no serious uncertainty about stream depths during the measurement.

4. *Computation procedure*—Normally, the discharge of a stream is computed for each current-meter traverse of the measurement cross section, using observed velocities, depths, and incremental channel widths. Because of the rapid change of stage that occurs during the course of a velocity-observation traverse, that conventional computation procedure should not be used when measuring the discharge of flash floods. If the conventional procedure is used there is great uncertainty as to the stage that applies to the computed discharge value. The recommended computation procedure for a flash flood is as follows.

The first step is to construct an individual relation of mean velocity to stage for each observation vertical. The mean velocity, it will be recalled, is obtained by applying an appropriate coefficient to each observed value of surface or subsurface velocity. For each vertical, mean velocity is plotted against stage, and each point is identified by clock time. A single smooth curve is usually fitted to the points, but the scatter of the points may indicate the need for two curves—one for the rising limb of the hydrograph and the other for the falling limb.

In either event, all the data needed are now available for constructing the stage-discharge relation for the entire cross section. The distance between observation verticals (incremental width) is known, and for any selected stage the corresponding depth and mean velocity at each observation vertical are likewise known. Those data are then used, in the conventional manner, to compute the total discharge corresponding to the selected stage. By repeating that operation for several stages, one obtains a stage-discharge relation for the entire range of stage, or, if necessary, two such relations—one for the rising limb of the hydrograph and one for the falling limb. As a final step, the stage-discharge relation(s) is applied to the stage hydrograph to compute the discharge hydrograph. In the absence of a reliable automatic stage record, the numerous visually observed values of stage provide the stage hydrograph.

CORRECTION OF DISCHARGE FOR STORAGE DURING MEASUREMENT

If a discharge measurement is made at a significant distance from the gage during a change in stage, the discharge passing the gage during the measurement will not be the same as the discharge at the measurement section because of the effect of channel storage between the measurement section and the gage.

Adjustment is made for channel storage by applying to the measured discharge a quantity obtained by multiplying the channel surface area by the average rate of change in stage in the reach. The equation used is

$$Q_G = Q_m \pm WL\,\frac{\Delta h}{\Delta t}, \tag{15}$$

where

Q_G = discharge passing the gage control (ft^3/s or m^3/s),
Q_m = measured discharge (ft^3/s or m^3/s),
W = average width of stream between measurement section and control (ft or m),
L = length of reach between measurement section and control (ft or m),
Δh = average change in stage in the reach L during the measurement (ft or m), and
Δt = elapsed time during measurement (s).

A reference point or a temporary gage is set at the measurement cross section if channel storage is likely to be significant. The water-surface elevations at the section and at the gage are determined before and after the measurement to compute Δh. If the measurement is made upstream from the control, the adjustment will be plus for falling stages and minus for rising stages; if made downstream from the control, the adjustment will be minus for falling stages and plus for rising stages.

Figure 102 shows the front sheet of a measurement that has been made 0.6 mi upstream from the control during a period of changing stage. The computation of the adjustment for storage for the measurement shown in figure 102 follows:

Measurement made 0.6 mi upstream, L = 3,170 ft.
Average width (W) between measurement section and control = 150 ft.
Change in stage at control, 5.84 to 6.74 ft = +0.90 ft.
Change in stage at measurement section, 12.72 to 13.74 = +1.02 ft. (Readings taken at measurement section from a reference point before and after measurement.)

Average change in stage (Δh) = (0.90 + 1.02) ÷ 2 = 0.96 ft.
Elapsed time during measurement = 1¼ hr = 4,500 s.
Measured discharge Q_m = 8,494 ft³/s.

$$Q_G = 8{,}494 - 150\,(3{,}170)\,\frac{0.96}{4{,}500} = 8{,}494 - 101 = 8{,}393 \text{ ft}^3\text{/s. Use}$$

8,390 ft³/s.

9-275-F
Jan. 1960

UNITED STATES
DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY
WATER RESOURCES DIVISION

DISCHARGE MEASUREMENT NOTES

Meas. No. 264
Comp. by TJB
Checked by

Sta. No. 1-9946.5
Big Creek near Dogwood, Va.
Date Mar. 26 1962 Party T. J. Buchanan
Width 140 Area 1,040 Vel 8.07 G. H. 6.29 Disch.* 8,390
Method .2-.8 No. secs. 30 G. H. change +.90 in 1¼ hrs Susp 75C
Method coef. 1 Hor angle coef Varies Susp coef. 1.00 Meter No. 3684

EST GAGE READINGS				
Time	El. T	Recorder	T Inside	Outside
1415	5.54	5.54	5.54	5.52
1440	Start	5.84		
1500		6.08		
1530		6.44		
1555	Finish	6.74		
1630	7.16	7.16	7.16	7.14
Weighted M. G. H.				
G. H. correction				
Correct M. G. H.				

Date rated 2-16-62 Used rating
for rod ✓ susp Meter 1.0 ft
above bottom of wt Tags checked Yes
Spin before meas 2:55 after 2:50
Meas plots % diff from rating
Wading, cable, ice, boat, upstr., downstr., side bridge 0.6 feet, mile, above, below gage, and
Check-bar, chain found
changed to at
Correct
Levels obtained

Measurement rated excellent (2%), good (5%), fair (8%), poor (over 8%), based on following conditions Cross section Fairly even; stone and gravel bottom.
Flow Good distribution; some Weather Raining
Other debris flowing. Air 44 °F@ 1635
Gage OK Water 38 °F@ 1635
Record removed Yes Intake flushed
Observer Talked with
Control Clear
Remarks * Discharge adjusted for storage effect
Qm = 8,494
G. H. of zero flow ft. Elev. RP = 30.00
RP to WS @ 1440 = 17.28; 30.00 − 17.28 = 12.72
RP to WS @ 1555 = 16.26; 30.00 − 16.26 = 13.74

16—76791-1

FIGURE 102.—Discharge-measurement notes with discharge adjusted for channel storage.

The adjusted discharge figure is the one used for defining the stage-discharge relation.

The adjustment of measured discharge for storage during a period of changing discharge is a separate and distinct problem from that of making adjustments for variable slope caused by changing discharge. (See p. 418–421.) Regardless of whether or not the discharge is to be adjusted later for variable slope, the storage adjustment to discharge is made immediately after completion of the discharge measurement.

SUMMARY OF FACTORS AFFECTING THE ACCURACY OF A DISCHARGE MEASUREMENT

The factors that affect the accuracy of a discharge measurement have been discussed in appropriate sections of the preceding text. This section provides a brief recapitulation of those factors.

1. *Equipment.*—Accurate measurement requires that measurement equipment be properly assembled and maintained in good condition. To avoid damage in transport, the equipment should be packed in appropriate containers or compartments of the vehicle used by the hydrographer. Current meters are especially susceptible to damage when in use, because measurements must often be made when drift or floating ice is present in a stream.

2. *Characteristics of the measurement section.*—The basic characteristics of the measurement section affect measurement accuracy. The attributes desired in a measurement section are those listed on page 139. If possible, the section should be deep enough to permit use of the two-point method of measuring velocity. The presence of bridge piers in or near the measurement section adversely affects the distribution of velocities. Piers also tend to induce local bed scour which affects the uniformity of depth. Those adverse effects are increased if the bridge piers tend to collect drift on their upstream faces.

3. *Spacing of observation verticals.*—The spacing of observation verticals in the measurement section can affect the accuracy of the measurement. Twenty-five to 30 verticals should normally be used, and the verticals should be spaced so that each subsection will have approximately equal discharge. However, a measurement vertical should be located fairly close to each bank and at "breaks" in depth. The spacing of the verticals should also be reduced in the vicinity of bridge piers. If many bridge piers are present in the section or if the streambed is nonuniform, more verticals than the recommended 25–30 should be used.

4. *Rapidly changing stage.*—When the stage changes rapidly during a discharge measurement, the computed discharge figure loses some of its significance, and there is uncertainty as to the appropriate gage height to apply to that discharge figure. Consequently, the standard procedure for making discharge measurements should be

shortened when the stage is changing rapidly, as explained on pages 174–177, even at the expense of some accuracy. The reduction in measurement time makes it possible to obtain a mean gage height that is representative of the measured discharge.

5. *Measurement of depth and velocity.*—Inaccuracies in sounding and in the placement of the current meter are most likely to occur in those sections having great depths and velocities. Heavy sounding weights should be used to reduce the vertical angle made by the sounding line, and where vertical angles exist, tags and (or) correction tables should be used in determining vertical distances. Where velocities are not perpendicular to the measurement section, the cosine of the angle between the perpendicular and the direction of the current must be determined. If a velocity-azimuth-depth assembly (p. 129–130) is not used, it is necessary to assume that the angle of the surface current prevails throughout the vertical; that assumption may be erroneous.

6. *Ice in the measuring section.*—Reliable measurements may usually be made when measuring from ice cover if the measurement verticals are free of slush ice. Slush ice interferes with the operation of the current-meter rotor and also causes difficulty in determining the effective depth of water. If the effective depth is considered to be that portion of the depth in which the current meter indicates velocity, the assumed effective depth may be too small if slush ice is interfering with free operation of the rotor. Collections of slush ice are generally thickest near the upstream end of ice-covered pools, and those areas should therefore be given little consideration as measurement sections. If the ice cover is layered so that there is water flowing between ice layers, it is almost impossible to obtain a reliable discharge measurement, particularly if the water layers are too thin to permit insertion of the meter between ice layers. The exposure of a wet current meter to subfreezing air temperatures may cause serious underregistration of the current meter as a result of ice forming in the meter bearings and contact chamber. Therefore, once the measurement is started, the current meter should be kept in the water as much as possible to avoid exposure to the cold air.

7. *Wind.*—Wind may affect the accuracy of a discharge measurement by obscuring the angle of the current, by creating waves that make it difficult to sense the water surface prior to sounding the depth, and by affecting the velocity at 0.2-depth in shallow streams, thereby distorting the vertical-velocity distribution. When making boat measurements, the wind-caused waves may induce vertical motion in a cable-suspended meter, or the wind may cause an oscillatory horizontal movement of the boat against the tag line; either movement may affect the operation of the current meter. Table 11 sum-

marizes the results of an investigation (Kallio, 1966) on the effect of vertical motion on the operation of Price, vane-type, and Ott (cosine rotor 8646–A) current meters. The plus signs in table 11 indicate overregistration by the meter; the minus signs indicate underregistration.

ACCURACY OF A DISCHARGE MEASUREMENT MADE UNDER AVERAGE CONDITIONS

Carter and Anderson (1963) made a statistical analysis of the error in discharge measurements made in natural streams under average measuring conditions. They tested the following four assumptions on which the computation of discharge measurements is based:

1. The rating of the current meter is applicable to the conditions of the measurement.
2. The velocity observed at a point is a true time-averaged velocity.
3. The ratio is known between the velocity of selected points in the vertical and the mean velocity in the vertical.

TABLE 11.—*Registration errors, in percentage of stream velocity, caused by vertical motion of current meter*

Stream velocity (ft/s)	Vertical motion (ft/s)						
	0.2	0.4	0.6	0.8	1.0	1.2	1.5
Price current meter (suspended by a cable)							
0.5	−2.0	+10	+36	+72	+120	+150	+210
1.0	−3.0	−1.0	+10	+24	+40	+50	+56
1.5	−6.7	−6.7	−4.0	+1.3	+8.0	+25	+27
2.0	−2.5	−2.5	−2.5	−2.0	0	+4.0	+14.0
2.5	0	0	0	0	0	+.8	+4.0
3.0	0	0	0	0	−2.3	−2.0	0
4.0	0	0	0	0	−1.3	−1.3	0
5.0	+.4	+1.0	+.6	0	−.2	0	+.8
7.0	−.7	−.4	0	+.1	−.4	−.7	−.4
10.0	−.5	−.3	0	0	−.3	−.7	−1.3
Vane-type current meter (suspended by a rod)							
0.5	+4.0	+6.0	+20	+44	+72	+100	+160
1.0	+5.0	+10	+12	+10	+11	+15	+26
1.5	+3.3	+8.7	+10	+6.7	+3.3	+3.3	+10
2.0	+2.0	+6.5	+9.0	+9.5	+8.5	+6.0	+7.5
2.5	+2.0	+4.4	+6.4	+7.6	+8.0	+7.2	+6.4
3.0	−1.7	+3.7	+5.3	+6.7	+7.3	+7.7	+6.7
4.0	+1.2	+.8	+.3	+1.0	+2.5	+3.8	+3.3
5.0	−1.0	−2.6	−2.8	−2.0	−.4	−.2	−2.0
7.0	−.7	−.7	−.3	0	0	+.3	−.4
Ott current meter (cosine rotor 8646-A, standard tailpiece without vertical stabilizer, and two-pin attachment to cable hanger)							
0.5	0	+6.0	+10	+20	+30	+44	+70
1.0	0	0	0	+4.0	+9.0	+15	+30
1.5	0	0	0	+1.3	+4.0	+7.3	+17
2.0	0	0	0	+.5	+2.0	+4.5	+9.5
2.5	0	0	0	0	+1.6	+2.8	+6.4
3.0	0	0	0	+.3	+1.0	+2.3	+6.0
4.0	0	0	+.5	+1.0	+1.8	+2.5	+3.8
5.0	+.4	+.6	+.4	+.6	+1.0	+1.4	−2.0
7.0	0	0	0	0	+.3	+.7	+1.4

4. The depth measurements are correct, and the velocity and depth vary linearly with distance between verticals.

Assumption 1 was tested by comparing the ratings obtained for Price current meters when rated in flumes of different sizes. It was found that the ratings can be repeated within a fraction of 1 percent. When a Price meter was tested in a wind tunnel under differing degrees of turbulence, its performance was not affected by increased turbulence. The similarity of results using the Price, Ott, and Neyrpic current meters has already been discussed on page 89. It was therefore assumed that the standard deviation (S_{R_i}) of the error ratio between measurement results obtained with different current meters equals 1 percent.

Assumption 2 was tested in 23 different rivers where velocities for consecutive time periods of 15, 30, 45, 90, 120, and 240 s were observed for a 1-hr period at points corresponding to 0.2, 0.4, 0.6, and 0.8 depth. The measurement verticals ranged in depth from 2.4 to 26.7 ft (0.7 to 8.1 m), and velocities ranged from 0.43 to 7.9 ft/s (0.13 to 2.4 m/s). Statistical analysis showed that velocity fluctuations were randomly distributed in time and space and that if 45-s observations were taken at the 0.2- and 0.8-depth positions in 30 verticals, the standard deviation (S_{R_t}) of the error ratio between observed and true point velocity was 0.8 percent.

Assumption 3 was tested using more than 100 stream sites. The standard deviation (S_{R_s}) of the error ratio between the mean velocity obtained from 0.2- and 0.8-depth observations and the true vertical velocity, using 30 verticals for the discharge measurement, was 1.15 percent.

Assumption 4 was tested using discharge measurements made at 127 stream sites, in which more than 100 verticals were measured in each cross section. The discharge for each site was again computed using the data for 1/2, 1/4, 1/5, 1/7, and 1/10 of the total number of verticals in each cross section. Error ratios between those computed discharges and the discharges computed using all observation verticals were determined. When 30 observation verticals were used, the standard deviation (S_N) of the error ratios was 1.6 percent.

The standard error of a discharge measurement (S_T) was computed from the equation

$$S_T = \sqrt{(S_{R_i})^2 + (S_{R_t})^2 + (S_{R_s})^2 + (S_N)^2}. \tag{16}$$

For a measurement using velocity observations of 45 s at the 0.2- and 0.8-depth positions in each of 30 verticals, S_T equaled 2.2 percent. That means that if single discharge measurements were made at a number of gaging sites using the standard method recommended in this manual, the errors of two-thirds of the measured discharges would be less than 2.2 percent.

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CHAPTER 6.—MEASUREMENT OF DISCHARGE BY THE MOVING-BOAT METHOD

INTRODUCTION

On large streams and estuaries the conventional methods of measuring discharge by current meter are frequently impractical and involve costly and tedious procedures. There may be no suitable facilities at remote sites. Where suitable facilities do exist, they may be inundated or inaccessible during floods. At some sites, unsteady flow conditions require that measurements be made as rapidly as possible. Measurements on tide-affected rivers must not only be made rapidly, but often continually, throughout a tidal cycle. The moving-boat technique is a method of rapidly measuring the discharge of large streams. It requires no fixed facilities, and it lends itself to the use of alternate sites if conditions make this desirable.

The moving-boat technique is similar to the conventional current-meter measurement in that both use the velocity-area approach in determining discharge. (See chapter 5.) In each method, a measurement is the summation of the products of the subsections of the stream cross section and their respective average velocities. Both techniques require that the following information be obtained:

1. Location of sampling verticals 1, 2, 3,. . .n across the stream in reference to the distance from an initial point.
2. Stream depth, d, at each observation vertical.
3. Stream velocity, V, perpendicular to the cross section at each observation vertical.

During a traverse of the boat across the stream, a sonic sounder records the profile of the cross section, and a continuously operating current meter senses the combined stream and boat velocities. A vertical vane alines itself in a direction parallel to the movement of water past it, and an angle indicator attached to the vane assembly indicates the angle between the direction of the vane and the true course of the boat. The data from these instruments provide the information necessary for computing the discharge for the cross section. Normally, data are collected at 30 to 40 observation points in the cross section for each run. Experience has shown that discharges determined by the moving-boat technique match, within 5 percent, discharges determined by conventional means.

The principal difference between the conventional measurement and the moving-boat measurement lies in the method of data collection. The standard current-meter method of measurement uses what might be called a static approach in its manner of sampling; that is, the data are collected at each observation point in the cross section while the observer is in a stationary position. This is in contrast to the dynamic approach to data collection utilized in the moving-boat method. Here, data are collected at each observation point while the observer is aboard a boat that is rapidly traversing the cross section.

THEORY OF THE MOVING-BOAT METHOD

The moving-boat measurement is made by traversing the stream along a preselected path that is normal to the streamflow. The traverse is made without stopping, and data are collected at intervals along the path. During a traverse of the cross section, the boat operator maintains course by "crabbing" into the direction of the flow sufficiently to remain on line (fig. 103). The velocity, V_b, of the boat with respect to the stream-bed along the selected cross-section path is the velocity at which the current meter is being pushed through the water by the boat. The force exerted on the current meter, then, is a combination of two forces acting simultaneously: one force resulting from the movement of the boat through the water along the cross-section path and the other a consequence of the natural streamflow normal to that path.

The velocity measurement taken at each of the sampling points in the cross section is a vector quantity that represents the relative velocity of water past the vane and meter. This velocity, V_V, is the vector sum of V, the component of stream velocity normal to the cross section at the sampling point, and V_b, the velocity of the boat with respect to the streambed along the selected path. The vector diagram in figure 104 depicts this relation.

The sampling data recorded at each observation point provide the necessary information to define V_V. The pulses-per-second reading from the rate-indicator unit is used in conjunction with a rating table to obtain the vector magnitude, V_V, while the angle reading, α, representing the angle the vane makes with the cross-section path, defines the direction of the vector.

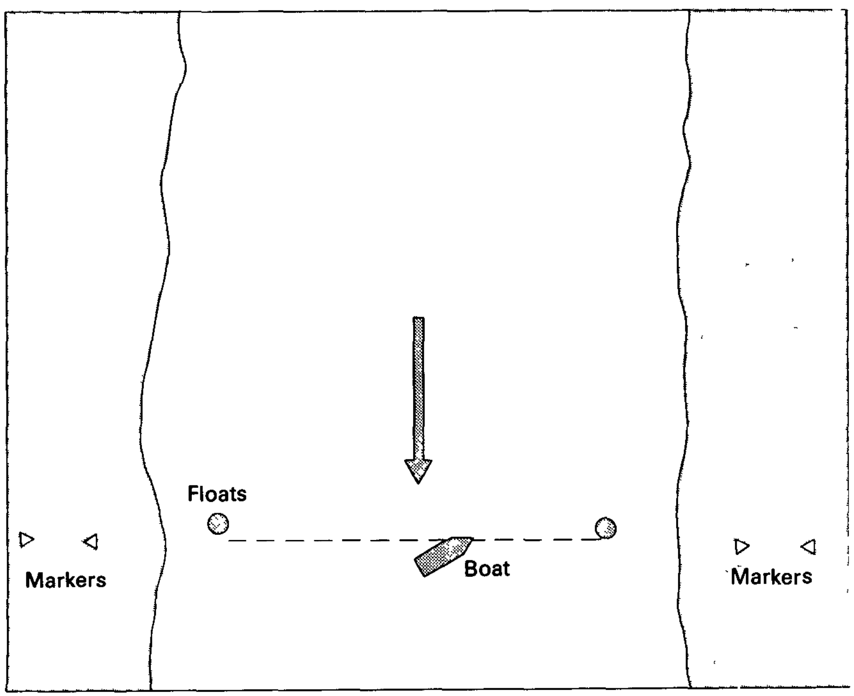


FIGURE 103.—Sketch of stream with markers.

Stream velocity, V, perpendicular to the boat path (true course) at each sampling point, 2, 3, 4, . . . $(n-1)$, can be determined from the relation

$$V = V_V \sin \alpha. \tag{17}$$

The solution of the above equation yields an answer which represents that component of the stream velocity that is perpendicular to the true course even though the direction of flow may not be perpendicular. This is the desired component.

From the same vector diagram, it can be seen that

$$L_b = \int V_V \cos \alpha \, dt, \tag{18}$$

where L_b is the distance that the boat has traveled along the true course between two consecutive observation points, provided the stream velocity is perpendicular to the path. Where the velocity is not perpendicular, an adjustment is required as explained on pages 207–208, where the adjustment of total width and area is discussed.

If one assumes that α is approximately uniform over the relatively short distance that makes up any one increment, then α may be treated as a constant. Therefore, equation 18 becomes

$$L_b \approx \cos \alpha \int V_V \, dt. \tag{19}$$

However,

$$\int V_V \, dt = L_V, \tag{20}$$

where L_V is the relative distance through the water between two consecutive observation points as represented by the output from the rate indicator and counter. Therefore,

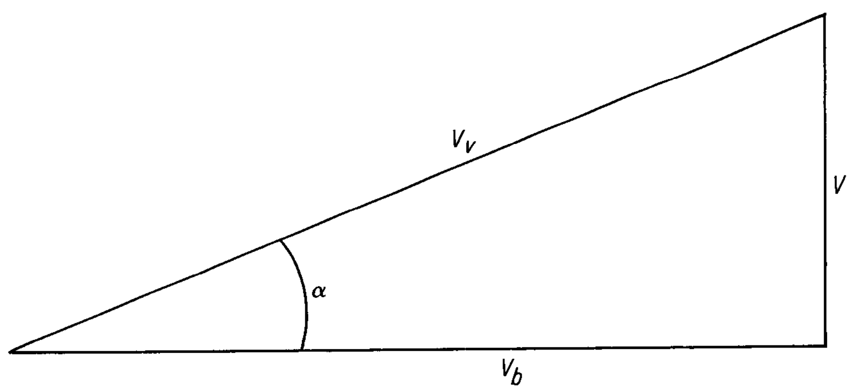


FIGURE 104.—Diagram of velocity vectors.

$$L_b \approx L_V \cos \alpha. \tag{21}$$

Finally, d, stream depth at each observation point, is obtained by adding the transducer depth to the depth obtained from the sonic-sounder chart. Upon determining V, L_b, and d for each vertical, the midsection method of computing a discharge measurement is used. (See p. 80–82.)

Much of the accuracy of a moving-boat discharge measurement depends upon the skill of the boat operator in maintaining a true course. Although even the most experienced pilot cannot be expected to keep the boat absolutely on course for an entire run, it is still extremely important that the measurement begin and end on line and that any deviations from the true course be kept as few in number and as small in magnitude as possible.

If velocity readings are taken while the boat is moving off course in an upstream direction, those readings will be greater than the true velocities; if the readings happen to be taken when the movement is toward the downstream direction, then the sampled velocity readings will be less than the true velocities. Thus, if one assumes the equal likelihood of overregistering or underregistering the stream velocities because of deviations from the true course, the errors can be considered compensating in nature. However, to further insure the reliability of the measurement, it is recommended that the results of at least six individual runs, each with from 30 to 40 observation points, be averaged to obtain the discharge when steady-flow conditions exist. This is practicable because of the ease and speed with which the extra runs can be made.

For unsteady-flow conditions on tidal streams, it will usually be desirable not to average the results from a series of runs but rather to keep them separate so as to better define the discharge cycle.

EQUIPMENT

Specialized instrumentation consisting of a sonic sounder, a vane with indicator, a special current meter with its associated electronic equipment, and an easily maneuverable small boat with some modifications, provide the capability needed for a moving-boat measurement.

VANE AND ANGLE INDICATOR

A vane with an indicating mechanism is mounted on the bow of the boat, with the vane centered approximately 3 to 4 ft (0.9 to 1.2 m) below the water surface (fig. 105). This assembly consists of a vertical, stainless-steel shaft with a pointer connected to its upper end and a thin vertical aluminum fin, 1-ft (0.3 m) high and 1½-ft (0.46m) long, attached to its lower end. The shaft is housed in an aluminum bearing tube and is mounted with ball bearings at the upper end and a teflon bearing (no lubrication needed) in the lower end of the tube so that the assembly (vane, shaft, pointer) is free to rotate as a unit. The vertical vane alines itself in a direction parallel to the movement of the water past it. The pointer is attached to the shaft so that it will be in line with the vane, pointing directly into the flow past the vane. The angle between the direction of the vane and the true course of the boat (the line of the cross section) is indicated on a dial by the pointer. The circular dial, calibrated in degrees on either side of an index point, swivels freely about the upper end of the vertical shaft, just below the pointer. A sighting device attached to the dial provides a means of alining the index point on the dial with the true course. In positive (downstream) streamflow the pointer above the dial will always point to the upstream side of the true course. Because the upstream side may be to the left or right side, depending on the direction in which the boat is traveling, and also because of possible negative velocities, the dial is calibrated in degrees (from 0 to 90) on both sides of its index point.

CURRENT METER

The current meter used by the U.S. Geological Survey is a component propeller type with a custom body made for mounting on the leading edge of the vane (fig. 105). The component propeller is less susceptible than are other types of meters to vertical components of velocity and was chosen to minimize errors created by the bobbing of the boat.

A 24-toothed gear passing in the proximity of a magnetic field is

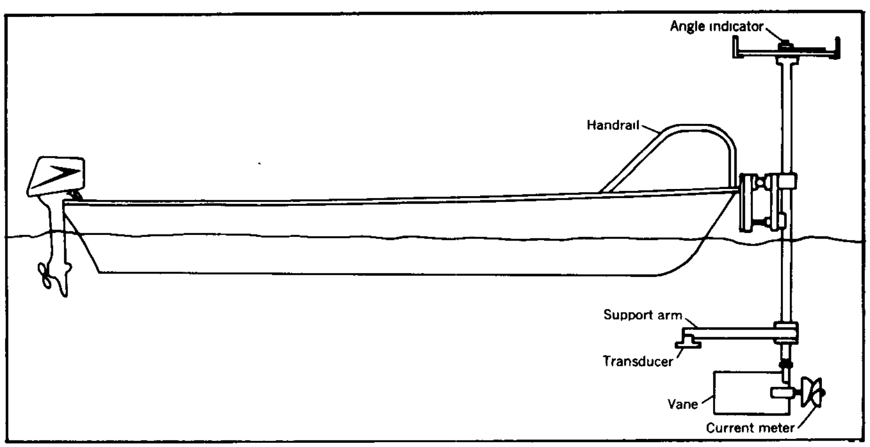


FIGURE 105.—Sketch of boat showing equipment.

used to generate 24 pulses per revolution of the propeller. The large number of pulses for each revolution facilitates the conversion of the pulse rate to an analog readout. An electronic pickup assembly registers these pulses and feeds them into a frequency-to-voltage converter, and they are then displayed as a reading on an electrical meter.

At one end of the meter cable is a metallic probe that is screwed into the meter body at the opening just behind the propeller nut (fig. 106). The probe is a permanent magnet that provides the magnetic field necessary for pulse generation. To function properly, the probe tip must be positioned within a few thousandths of an inch of the 24-toothed gear located within the meter. Adjustment of the probe position, which is seldom required, should be done with care to prevent damage to the probe tip.

Before the meter is used, the cup within the hub of the current-meter propeller should be filled with thin oil (Ott propeller oil) as shown in figure 106. The bearing assembly is then inserted into the hub, and the propeller nut is tightened. After the conclusion of a series of measurements, the cup should be emptied completely and cleaned before storage.

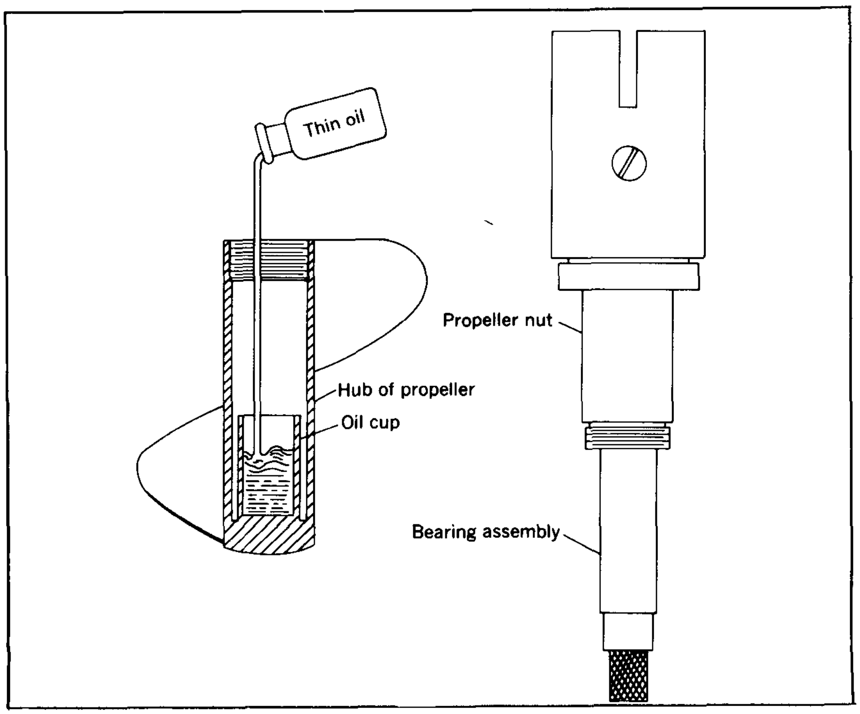


FIGURE 106.—Component propeller-type meter.

RATE INDICATOR AND COUNTER

One of the principal functions of the rate indicator and counter is to register the pulses received from the current meter, feed them into a frequency-to-voltage converter, and then display them as a reading on its electrical meter (fig. 107). These pulses are received through the current-meter cable which is plugged into the marked receptacle provided in the front panel of the unit. The current meter generating the pulses is calibrated so that the reading on the electrical meter in pulses per second can be converted to a particular velocity in feet per second through the use of a rating table (fig. 108). The value read from the electrical meter at any particular instant represents an instantaneous readout of velocity.

Two scale selections are available for the rate-indicator unit. If the switch is set at the "500" selection, the readout is taken from the lower scale of the panel meter; at the 1,000 setting, the upper scale is used. The 500 scale is the more sensitive of the two, and therefore its use is recommended during those measurements in which the velocity of the water past the meter is not great enough to give readings that exceed 500 pulses per second.

In addition to serving as a pulse-rate indicator from which velocity determinations can be made, this unit has also been designed to provide a method of automatically selecting measurement points in a section at regular intervals of travel distance. This design makes use of the fact that each revolution of the meter propeller generates 24

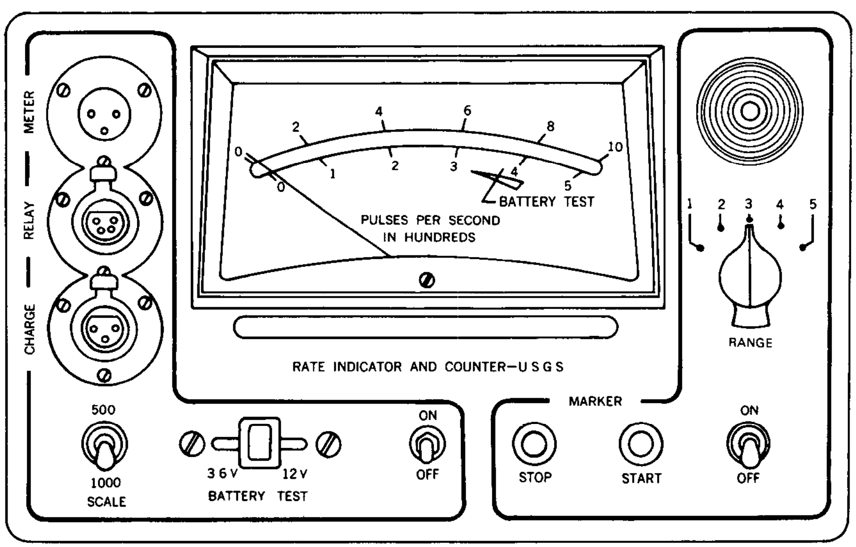


FIGURE 107.—Control panel of rate indicator and counter.

DEPARTMENT OF THE INTERIOR - GEOLOGICAL SURVEY
Water Resources Division
RATING TABLE FOR MOVING BOAT METER NO. . 2-4 . . . . . .
EQUATIONS: . V = 0.01768 N + 0.078 . . . . . . Limits of actual rating . . 1.0 . . to . . 10 . . . feet per second, Rated JULY 5, 1967

Counts per Second	VELOCITY IN FEET PER SECOND																				Counts per Second
	0	5	10	15	20	25	30	35	40	45	50	55	60	65	70	75	80	85	90	95	
0											0.96	1.05	1.14	1.23	1.32	1.40	1.49	1.58	1.67	1.76	0
100	1.85	1.93	2.02	2.11	2.20	2.29	2.38	2.46	2.55	2.64	2.73	2.82	2.91	3.00	3.08	3.17	3.26	3.35	3.44	3.53	100
200	3.61	3.70	3.79	3.88	3.97	4.06	4.14	4.23	4.32	4.41	4.50	4.59	4.67	4.76	4.85	4.94	5.03	5.12	5.21	5.29	200
300	5.38	5.47	5.56	5.65	5.74	5.82	5.91	6.00	6.09	6.18	6.27	6.35	6.44	6.53	6.62	6.71	6.80	6.88	6.97	7.06	300
400	7.15	7.24	7.33	7.42	7.50	7.59	7.68	7.77	7.86	7.95	8.03	8.12	8.21	8.30	8.39	8.48	8.56	8.65	8.74	8.83	400
500	8.92	9.01	9.09	9.18	9.27	9.36	9.45	9.54	9.63	9.71	9.80	9.89	9.98	10.07	10.16	10.24	10.33	10.42	10.51	10.60	500
600	10.69	10.77	10.86	10.95	11.04	11.13	11.22	11.30	11.39	11.48	11.57	11.66	11.75	11.84	11.92	12.01	12.10	12.19	12.28	12.37	600
700	12.45	12.54	12.63	12.72	12.81	12.90	12.98	13.07	13.16	13.25	13.34	13.43	13.51	13.60	13.69	13.78	13.87	13.96	14.05	14.13	700
800	14.22	14.31	14.40	14.49	14.58	14.66	14.75	14.84	14.93	15.02	15.11	15.19	15.28	15.37	15.46	15.55	15.64	15.72	15.81	15.90	800
900	15.99	16.08	16.17	16.26	16.34	16.43	16.52	16.61	16.70	16.79	16.87										900

TABLES OF L_b, IN FEET											
Range No.	Angle α in degrees	0	1	2	3	4	5	6	7	8	9
1	0	18.2	18.2	18.2	18.2	18.2	18.2	18.1	18.1	18.1	18.0
	10	18.0	17.9	17.9	17.8	17.7	17.6	17.5	17.5	17.4	17.3
	20	17.1	17.0	16.9	16.8	16.7	16.5	16.4	16.3	16.1	16.0
	30	15.8	15.6	15.5	15.3	15.1	14.9	14.8	14.6	14.4	14.2
	40	14.0	13.8	13.6	13.3	13.1	12.9	12.6	12.4	12.2	11.9
2	0	36.5	36.5	36.5	36.5	36.4	36.4	36.3	36.2	36.1	36.1
	10	35.9	35.8	35.7	35.6	35.4	35.3	35.1	34.9	34.7	34.5
	20	34.3	34.1	33.8	33.6	33.3	33.1	32.8	32.5	32.2	31.9
	30	31.6	31.3	31.0	30.6	30.3	29.9	29.5	29.2	28.8	28.4
	40	28.0	27.5	27.1	26.7	26.3	25.8	25.4	24.9	24.4	23.9
3	0	73.0	73.0	73.0	72.9	72.8	72.7	72.6	72.5	72.3	72.1
	10	71.9	71.6	71.4	71.1	70.8	70.5	70.2	69.8	69.4	69.0
	20	68.6	68.2	67.7	67.2	66.7	66.2	65.6	65.0	64.4	63.8
	30	63.2	62.6	61.9	61.2	60.5	59.8	59.0	58.3	57.5	56.7
	40	55.9	55.1	54.2	53.4	52.5	51.6	50.7	49.8	48.8	47.9
4	0	146	146	146	146	146	145	145	145	145	144
	10	144	143	143	142	142	141	140	140	139	138
	20	137	136	135	134	133	132	131	130	129	128
	30	126	125	124	122	121	120	118	117	115	114
	40	112	110	108	107	105	103	101	99.6	97.7	95.8
5	0	292	292	292	292	291	291	290	290	289	288
	10	288	287	286	284	283	282	281	279	278	276
	20	274	273	271	269	267	265	262	260	258	255
	30	253	250	248	245	242	239	236	233	230	227
	40	224	220	217	214	210	206	203	199	195	192

SIN of ANGLE α										
Angle α	0	1	2	3	4	5	6	7	8	9
0	.000	.017	.035	.052	.070	.087	.105	.122	.139	.156
10	.174	.191	.208	.225	.242	.259	.276	.292	.309	.326
20	.342	.358	.375	.391	.407	.423	.438	.454	.469	.485
30	.500	.515	.530	.545	.559	.574	.588	.602	.616	.629
40	.643	.656	.669	.682	.695	.707	.719	.731	.743	.755
50	.766	.777	.788	.799	.809	.819	.829	.839	.848	.857

FIGURE 108.—Sample tables of meter rating, L_b, and sine α.

evenly spaced pulses, and that from the calibration of the meter it can be determined that one pulse is equal to some fraction of a foot of meter travel through the water, or of water travel past the meter. By using a set of frequency-dividing modules, provision is made for these pulses to be electronically counted to a preset number, at which time an audible signal is generated and the sounder chart is automatically marked. The counter then automatically resets itself, and the process is repeated. The purpose of the audible signal is to let the boat crew know when a sampling location is reached. At this point they will take an angle reading from the pointer and a readout from the electrical meter. The markings on the sounder chart are automatically triggered by an electrical impulse transmitted to the depth-sounder unit by a relay in the meter electronics. The relay cable from the counter to the sounder should be plugged into the appropriately marked receptacle on the front panel of both units. The markings on the sounder chart locate observation points in the cross section and thus show where depth readings should be taken.

Preset intervals that are available on each unit are as follows:

Range selection	*Pulse counts*	*Distance, in feet*
1	1,024	18.75
2	2,048	37.5
3	4,096	75
4	8,192	150
5	16,384	300

The distances listed above are typical; exact ones depend upon the calibration of the particular current meter used. If possible, the pulse-selector switch should be set for a distance that will divide the measured width between the two floats into from 30 to 40 increments. For example, if the distance between floats is 500 ft (150 m), range 1 should be selected; for a distance of 1,000 ft (300 m), range 2 should be used, and so on. Each distance listed in the table above represents L_V, the relative distance through the water, and it will be somewhat larger than the corresponding L_b value, the distance along the true course that the boat has traveled. L_b is the distance one should use to determine the number of observation points that will be taken in a given cross section. However, the listed L_V values can be used to estimate roughly the number of observation points—the estimated number will always be less than the actual number of observation points.

The rate indicator and counter has two internal power supply packs, each consisting of a set of nickel-cadmium rechargeable batteries. A battery test switch located on the front panel of the unit can be used to test the condition of either the 3.6-volt or the 12-volt power supply (fig. 107). Testing should be done with both the main power and the marker switches off. A reading of the panel meter above the test switch will indicate the degree of charge. A needle deflection greater than the battery testline mark indicates a satisfactory charge level for most measurement requirements. A fully charged battery pack will operate satisfactorily from 12 to 16 hr. A marginal reading, with needle deflection just to the test mark, indicates approximately 4 to 8 hr of useful battery operation remaining. A reading below the mark serves as a warning that a battery charge is needed.

BATTERY CHARGER

The battery charger serves as a dual unit for charging either one of the two battery supply packs located within the rate indicator and counter unit. Its charge plug is inserted into the charge receptacle located on the panel of the rate indicator and counter. The other plug should be connected to a 115-volt, 60-cycle line power supply. With proper care the batteries should provide many years of service.

SONIC SOUNDER

A portable sonic sounder (fig. 109) is used to provide a continuous strip-chart record of the depth of the stream, that is, a profile of the cross section between the two floats. Its transducer releases bursts of ultrasonic energy at fixed intervals. The instrument measures the time required for these pulses of energy to travel to the streambed, to be reflected, and to return to the transducer. With a known propagation velocity of sound in water, the sounder computes and records the depth. Accuracy of the recording depth sounder is approximately ±0.5 ft (0.15 m). The sounder used in this application is a commercially available model with a minor modification for automatically marking the chart at each observation point in response to an electrical pulse from the meter electronics. The sounder is powered by a standard lead-acid battery of 6 or 12 volts, depending on the model of the sounder.

One minor modification to the sonic sounder is the installation of a receptacle on its front panel into which is plugged the relay cable from the rate indicator and counter. The purpose of this relay connection is to transmit the electrical pulses from the counter unit that will automatically trigger the vertical-line markings on the sounder chart. This provision for automatically marking the sounder chart at regular intervals of distance traveled eliminates the need for the manually operated "mark" switch, except in the event that the relay cable is damaged or missing. Then, as an expedient measure, the chart is marked at each observation point by manually pulling the switch at each tone signal sounded by the counter unit.

Three paper-feed speeds are provided with the unit. The small lever in the lower left-hand corner of the recorder chassis is the control. The set of speeds available will vary according to the model. Some models

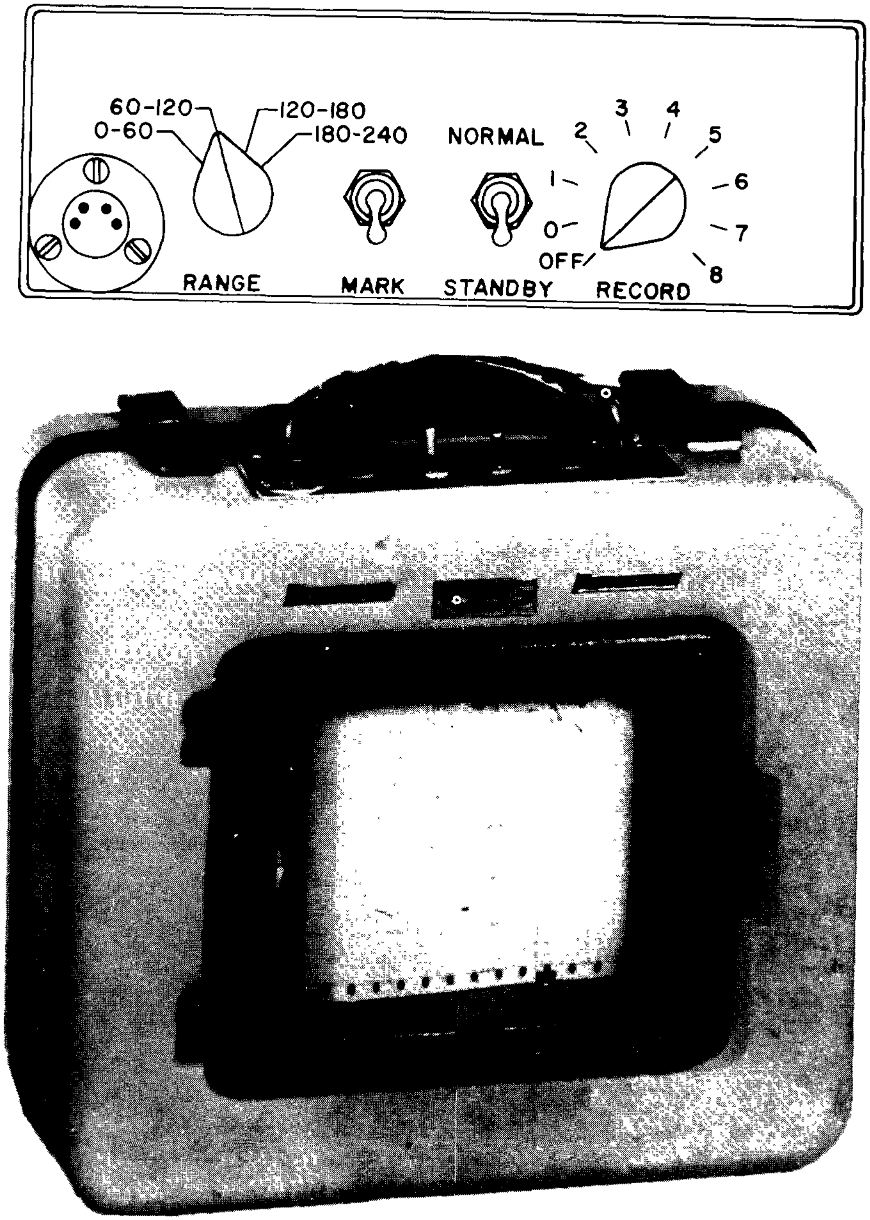


FIGURE 109.—Sonic sounder and control panel.

provide choices of 12, 30, and 60 in/hr (0.30, 0.76, and 1.52 m/hr), and others 36, 90, and 180 in/hr (0.91, 2.29, and 4.57 m/hr). A suitable chart speed is one that results in a spacing of approximately 0.25 in between the vertical-line markings that are set on the chart during the measurement. Spacing wider than that needlessly wastes chart paper, whereas much narrower spacing results in poor resolution of the streambed trace. Narrow spacing also causes difficulty in determining the fractional part of a full spacing that should be assigned to the final width increment. Width of spacing is dependent upon the range setting on the counter unit, the current-meter velocity, and the chart speed. Table 12, computed for a current-meter velocity of 5 ft/s (1.52m/s), provides an example of typical spacing distances expected for various combinations of chart speed and range distance.

BOAT

Any easily maneuverable small boat that is sufficiently stable for the stream on which it is to be used is adequate for a moving-boat measurement. A photograph of a boat with the vane assembly mounted is shown in figure 110.

Preparation of the boat.—A 12×12×¼-in steel plate should be so attached that it is centered on the bow of the boat, perpendicular to the centerline of the boat, and as nearly vertical as possible. This plate must be securely anchored because at high velocities great force will be exerted on it. It may be necessary, depending on the style of boat being used, to erect handrails on the forward part of the boat, similar to those shown in figure 105. This is done for the safety of the angle reader who must stand in the bow.

Mounting of the equipment.—The 12×12-in aluminum plate on the vane assembly is attached to the 12×12-in steel plate on the bow of the boat. It is necessary to clamp these two plates together and drill four holes for accepting bolts to permanently fasten the plates together. The general location of the bolt holes should be near the four corners, but exact placement is not critical.

The two cap screws in the depth adjustment clamp that hold the aluminum bearing tube of the assembly (fig. 111) can be loosened so

TABLE 12.—*Spacing of vertical-line markings, in inches, on the sonic-sounder chart for various combinations of chart speed and range distance*

[Computed for a current-meter velocity of 5 ft/s.]

Chart speed	Range distance				
	18 75 ft	37.5 ft	75 ft	150 ft	300 ft
36 in/hr	0.04	0.08	0.16	0.32	0.64
90 in/hr	.10	.20	.40	.80	1.6
180 in/hr	.20	.40	.80	1.6	3.2

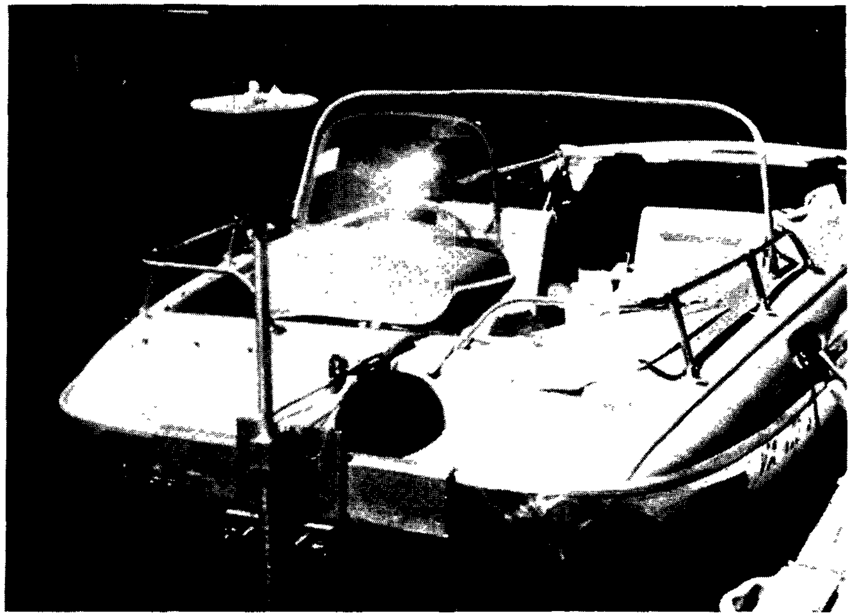


FIGURE 110.—Boat with mounted vane assembly.

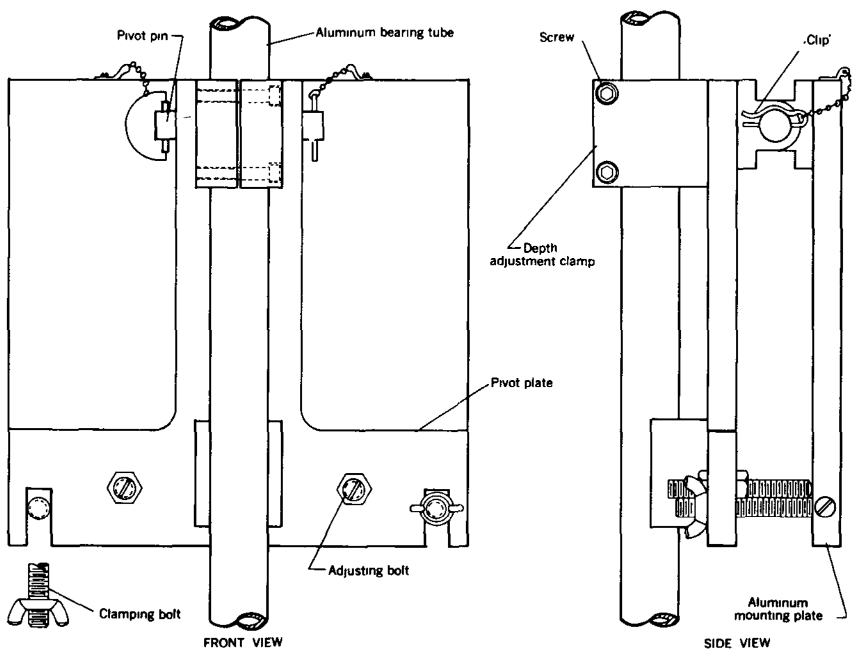


FIGURE 111.—Detailed view of vane-mounting assembly.

that the tube may be either raised or lowered in order to position the meter at the desired depth. This depth should preferably be at least 3 ft (0.91 m) to avoid the effect of surface disturbances and not greater than 4 ft (1.22 m) to avoid the danger of too great a torque being exerted at high velocities. Caution should be exercised to avoid high boat speeds, for the drag on the vane assembly is proportional to the square of the velocity of the water past the vane and therefore increases very rapidly with speed.

The sonic-sounder transducer is mounted on a support arm at a depth of either 2 or 3 ft (0.61 or 0.91m). It can be positioned by first loosening the two cap screws that secure the support arm to the aluminum bearing tube of the vane assembly and then moving the arm either up or down to the desired depth.

Because the vane assembly is mounted perpendicular to the centerline of the boat, it will change slightly from its original vertical position as a result of the raising of the bow of the boat during the moving-boat measurement. To offset this change and thus allow for vertical positioning of the assembly during normal boat operation, the mounting assembly provides for a compensating angle adjustment to be accomplished through the use of two adjusting bolts and a pivot plate (fig. 111). By screwing these bolts either inward or outward, the lower portion of the assembly can be pivoted toward or away from the boat. If the adjusting bolts are touching the aluminum plate when the assembly is in a vertical position, they must be screwed away from the plate so that the lower portion of the assembly can then be pivoted toward the boat and secured in position by use of two clamping bolts. The degree of adjustment would depend upon the operating velocity of the boat during the measurement.

Removal of the equipment.—After the measurements are completed, the vane assembly can be removed while leaving the aluminum plate bolted to the steel plate on the bow of the boat. This is accomplished by first loosening the wing nuts on the two clamping bolts of the plate and then removing the clip and sliding out the pivot pin that secures the assembly to the aluminum plate (fig. 111). Prior to doing this, the meter and the transducer cables should be disconnected from the rate indicator and the sonic sounder, respectively.

MEASUREMENT PROCEDURES

Procedures for a moving-boat measurement include selection and preparation of a suitable measuring site, preparation and assembly of the equipment used for the measurement, and a selection of settings for the instruments used to collect the data.

SELECTION AND PREPARATION OF THE MEASUREMENT SITE

Some preparation is required at the site prior to starting a series of moving-boat measurements. First, a path for the boat to travel is selected, it being as nearly perpendicular to the flow direction as possible. Then, two clearly visible range markers are placed on each bank in line with this path. The color of these markers should contrast sharply with the background. Spacing between the markers is dependent upon the length of the path, the longer paths requiring greater spacing. Approximately 100 ft (30 m) of spacing is needed for each 1,000 ft (300 m) of path length. Next, anchored floats are placed in the stream 40 to 50 ft (12 to 15 m) from each shore along the selected path (fig. 103). In making a traverse, this distance is needed for maneuvering the boat when entering or leaving the path. The floats should be placed so that the depth of water in their vicinity is always greater than 3 to 4 ft (0.9 to 1.2 m), which is vane depth. Large plastic bleach containers are suitable for use as floats; styrofoam cubes can also be used. It is preferable not to place the floats directly in the boat path but rather 10 to 20 ft (3 to 6 m) upstream. Their purpose is to mark the beginning and ending points of the boat measurement, and by offsetting them upstream they can serve that purpose without being in the way as the boat approaches along the selected path. Finally, the width of the stream is measured by triangulation, stadia, or other methods, and the exact locations of the floats are determined. Because the floats are close to the bank, a tape measure can be used to determine the distance from edge of water to each float. These distances should be recorded in the spaces provided on the front page of the discharge-measurement notes, for they will be used in the computation of the measurement.

If a station is to serve as a site for moving-boat discharge measurements on a continuing basis, it will probably be desirable to construct permanent range markers. Such markers would serve two useful purposes. First, determination of stream width would become a relatively simple procedure because of the availability of the constant distance between the markers once this distance has been established. A tape measure could be used to obtain the horizontal distance from the nearest marker to the water's edge on each bank. Subtracting these two distances from the established distance between the two streamward markers would provide the width of the stream. A second advantage would be that if the need arose, the markers could serve as permanent initial points from which cross-section profiles of the measurement section could be constructed.

PREPARATION OF THE EQUIPMENT

The special equipment and instruments necessary for a moving-boat measurement have been described in some detail on pages 187–197 under the general heading "Equipment." The purpose of this section of the manual is to summarize, for the convenience of the boat crew, the steps involved in the assembly of the equipment and in the selection of the instrument settings.

ASSEMBLY OF THE EQUIPMENT

The steps listed below should be followed in assembling the equipment:

1. Permanently mount a steel plate to the bow of the boat (p. 195) and then attach the aluminum plate of the vane assembly to it (p. 197). Both of these steps are "one time" operations that should be completed in advance of the trip.
2. Several days prior to the trip, check the batteries in the rate-indicator and counter unit and the storage battery for the sonic sounder to see that they are adequately charged. This will provide time to charge the batteries, if necessary, before the start of the trip.
3. Attach the sonic-sounder transducer to its support arm on the vane assembly (fig. 105).
4. Attach the current meter to the leading edge of the vane (fig. 105).
5. Use the pivot pin and clip and the two clamping bolts of the aluminum plate to secure the vane assembly to the plate (fig. 111).
6. Position the current meter at the desired depth (3 to 4 ft). This is done by loosening the two cap screws in the depth adjustment clamp and then raising or lowering the aluminum bearing tube to the proper position before retightening the screws. Measure the meter depth and record it in the measurement notes once positioning is completed.
7. Route the current-meter cable up the vane assembly and plug it into the marked receptacle on the rate indicator and counter unit. To prevent entanglement, the cable should be taped to the aluminum bearing tube in several places. It is necessary to provide some slack in the cable at its lower end to allow for the movement of the meter as the vane rotates during the measurement.
8. Position the sonic-sounder transducer at a depth of 2 or 3 ft. This is accomplished by first loosening the two cap screws that secure the support arm to the aluminum tube and then sliding the arm either up or down the tube to the proper position before tightening the screws. Measure the transducer depth and record it in the measurement notes at this time.
9. Route the transducer cable up the vane assembly, securing it by tape to the aluminum bearing tube; feed it through the hole

provided near the hinge of the sonic-sounder case; and then plug it into the marked receptacle at the back of the dividing plate.

10. Insert the battery cable of the sonic sounder into the marked receptacle at the back of the dividing plate of the unit. It can be fed through one of the holes provided near the hinge. A standard 6- or 12-volt storage battery, depending on the sonic-sounder model used, should be used as a power supply (p. 193).
11. Plug the relay cable from the counter unit to the sonic sounder into the marked receptacle on the front of each unit.
12. Use the two adjusting screws (fig. 111) to provide for the compensating angle adjustment of the vane assembly as described on page 197.

SELECTION OF THE INSTRUMENT SETTINGS

The notekeeper is responsible for the functioning of the rate indicator and counter and the recording depth sounder. To assure proper operation of the equipment, it is necessary that he make several preliminary instrument settings for each unit prior to the measurement. The following is a list of the steps involved in obtaining these settings:

Sonic sounder:

1. Check to see that the pulleys turn smoothly and that the stylus enters the track easily.
2. Set the operation switch to "stand by" position.
3. Turn on the unit by rotating the "record" switch clockwise to some low-numbered position and wait a few minutes for the tubes to warm up.
4. Set the depth-range selection at phase 1 (0–60 ft) and advance the gain control to obtain the "zero mark" near the top of the sounder chart.
5. Set the "zero mark" on the zero line of the recorder paper through use of the zero adjustment screw located behind the top pulley.
6. Continue to advance the gain until the "echo mark" appears somewhere below the "zero mark." If no echo appears when the gain is opened, switch the range control to the next phase (60–120 ft) and so on until the bottom is found.
7. Determine the optimum chart speed through use of table 12; then use the small lever in the lower left-hand corner of the recorder chassis to make this selection.
8. Change the operating switch from "stand by" to "normal" position several minutes before the measurement begins. This will start

the chart recording and provide an opportunity for a final check of instrument operation before the measurement begins.

Rate indicator and counter:

1. Select the desired scale for the rate-indicator meter located on the panel.
2. Check both battery packs with the battery test switch. This final check will have been preceded by a test several days before the measurement date in order to provide time to charge the batteries if needed (p. 199).
3. Set the rate-indicator switch to "on" position.
4. Set the marker switch to "on" position. (The actual marking will not begin until the "start" button is depressed.)
5. Select a range setting that will provide between 30 to 40 observation points. Because of the scale settings available, this may not always be possible, in which case choose that comprise setting that will come closest to meeting this provision.

After all the control settings are completed, the instruments are ready for use. The "start" and "stop" buttons on the panel of the counter unit are used to begin and end instrument operation at the precise moments the bow of the boat reaches the first and second floats, respectively. The operation of these button controls in regard to the starting and stopping procedure is described in the section of the manual that immediately follows.

FUNCTION OF THE CREW MEMBERS

Three crew members are necessary for making a moving-boat discharge measurement. They include a boat operator, an angle observer, and a notekeeper. Before crew members begin making discharge measurements by the moving-boat method, it is important that they develop a high degree of proficiency in all phases of the technique. This can be done by making practice measurements at a site where the discharge is known and then comparing the moving-boat discharge with the rated discharge. If there is no suitable site available for that purpose, the boat crew should make a series of moving-boat measurements at a single location and compare results for repeatability.

BOAT OPERATOR

Before the measurement begins, the boat operator should become thoroughly familiar with the sampling site. In tidal streams the operator should be familiar with conditions during all phases of the tidal cycle. This will help him avoid running the boat aground in shallow depths and damaging the submerged equipment. While maneuvering the boat, it is necessary to avoid sudden sharp turns that might result in damage to the meter cable by causing it to be wrapped around the vane assembly.

The operator should select an approach path for the boat that will allow it to be properly maneuvered into position prior to passing the first float. The path should begin from a downstream position as close to the riverbank as depth considerations permit. From such a starting point the boat can be accelerated to near its normal operating speed and the turn into the measuring section can be completed before the measurement begins. By attaining both the proper speed and alinement prior to reaching the float, the instrument readings will have time to stabilize before the initial sample is taken.

During a traverse the only function of the boat operator is to pilot the boat. He maintains course by "crabbing" into the direction of flow sufficiently to remain on line throughout the run. As varying stream velocities are encountered in the cross section, he should rely more upon steering adjustments to keep the proper alinement than upon acceleration or deceleration of the boat. Alinement is determined by sighting on the shore which is being approached. Much of the accuracy of the measurement depends on the skill of the boat operator in maintaining a true course with the boat.

Because the stream velocity is calculated as a sine function of angle alpha (fig. 104, eq. 17), small angles should be avoided whenever possible. It is desirable to maintain angle alpha at approximately 45°. The reason for this is that an error of several degrees in reading angle alpha would be more significant at small angle readings than the same error at larger angle readings. In order to maintain an angle of 45°, the velocity of the boat must be equal to that of the stream. This can be done if the stream velocity is greater than 2.5 ft/s (0.75 m/s); however, control of the boat is difficult to maintain below that velocity. For example, in tidal streams the stream velocity will often vary from 0 to several feet per second; therefore, it is not always possible to maintain an angle as large as 45°, and the measurement must be made using smaller angles.

ANGLE OBSERVER

A second man alines the dial of the vane indicator through its sighting device and, upon receiving the audible signal from the pulse counter, he reads the angle formed by the vane with respect to the true course. He reports the angle to the notekeeper who then records it. If the boat has strayed from the true path, the angle reader should sight parallel to the cross-section markers rather than at the markers themselves.

NOTEKEEPER

The notekeeper has several functions to perform. Prior to the measurement it is his responsibility to see that the preparation of all equipment pertaining to the rate indicator and counter and to the sonic sounder is completed satisfactorily. That includes not only equipment assembly but also selection of appropriate instrument settings.

The accuracy of the sonic sounder varies with changes in the velocity of sound in water, which in turn varies slightly with temperature, dissolved solids, and other variables. Thus sounder output should be compared to a known depth. Any significant error detected in the comparison can be expressed as a percentage of the known depth, and this percentage would be applicable to all depths determined by the sonic sounder. Consequently, this percentage correction should be applied to the total area and total discharge.

A check to determine that the current meter and rate indicator are functioning properly is also desirable. Such a check can be made by comparing the indicated velocity to a velocity determined by the Price current meter. This test is intended to determine proper operation only and is not intended for calibration purposes.

It is the notekeeper's responsibility to operate the controls provided on the equipment for starting and stopping the counter. It is important to the accuracy of the measurement that this unit promptly begin and end its operation at the first and second floats, respectively. This is accomplished by operating the "start" and "stop" buttons on the panel of the counter in the following manner:

1. Approximately 1 s before the boat reaches the first float, the "start" button is depressed; then it is released at the moment of passing. This marks the sounder chart, resets the counter, and starts it.

Marking of the sounder chart and sounding of the beeper (tone signal) will be automatic during the measurement. This will occur at regular intervals as determined by the setting of the range switch on the panel of the counter unit.

2. At the moment the bow of the boat reaches the second float, the "stop" button is depressed for 1 s and then released. This marks the sounder chart, signifying the end of the measurement, and stops the counter.

During the measurement the notekeeper records the angle reading at each signal as it is called out by the angle reader. He also reads and records the instantaneous "velocity" from the rate indicator meter at the same time. Readings are taken at all observation points as defined by the tone signals, including the two float positions.

If the time between consecutive measurements is short, it is desirable to leave the operating switch of the sonic sounder in the "normal" position until the measurement series is concluded. In this way the chart continues to advance between measurements, but there are no vertical line markings. This absence of vertical lines provides a gap on the chart that clearly sets off one measurement from another.

COMPUTATION OF THE DISCHARGE MEASUREMENT

COMPUTATION OF UNADJUSTED DISCHARGE

The method of computing discharge measured by the moving-boat technique is basically similar to the computation method used for conventional current-meter discharge measurements. (See p. 80–82). The discharge in a subsection, q, is the product of the subsection area of the stream cross section and the average velocity in the subsection. The midsection method of computation is used in which it is assumed that the average velocity at the observation vertical is the average velocity for a rectangular subsection. The rectangular subsection extends laterally from half the distance from the preceding observation vertical to half the distance to the following observation vertical and extends vertically from the water surface to the sounded depth (fig. 112). The summation of the discharges of the subsections, $q_1, q_2, q_3, \ldots q_n$, is the total unadjusted discharge of the stream. A step-by-step outline of the computation procedure, which refers to the sample measurement notes shown in figure 113, is given here as a guide to the hydrographer who computes the discharge from the field observations.

1. The data in the first column of figure 113 are the angle readings recorded by the notekeeper during the measurement. Because these readings begin and end at the float positions (there are no edge-of-water readings), they represent the values observed at locations 2, 3, 4, . . . $(n-1)$.
2. Each value in column 2 represents an incremental distance the boat has traveled along the cross-section path between two consecutive observation points. For example, α_x (col. 1) represents the angle reading at location x, and L_{b_x} (col. 2) is the incremental distance the boat has traveled along the true course, extending from the previous observation point, $x-1$, to the location x where the reading was taken.

The values in column 2 can be read directly from a table (fig. 108) by using the angle values recorded in column 1 and the range number as determined by the range selection on the counter unit. Two exceptions are the first and last values in the column, representing the distance to each float from its nearest edge of water. They are determined by direct measurement prior to the actual run. This is necessary because the actual boat run does not begin and end at the edge-of-water positions and thus is not set up to measure those distances. Therefore, the beginning angle reading, α_2, at the first float is not used for obtaining distance from the edge of water to that float. It should also be noted that L_{b_3} is always recorded as one-half of the value found in the table. This is necessary because the counter unit has been programed to signal at a "half-count" on its first count routine. All remaining values can be recorded directly from the table of L_b values, without any changes, with the possible exception of the one determined from the last angle reading which was made at the second float. This angle reading may or may not have been made at

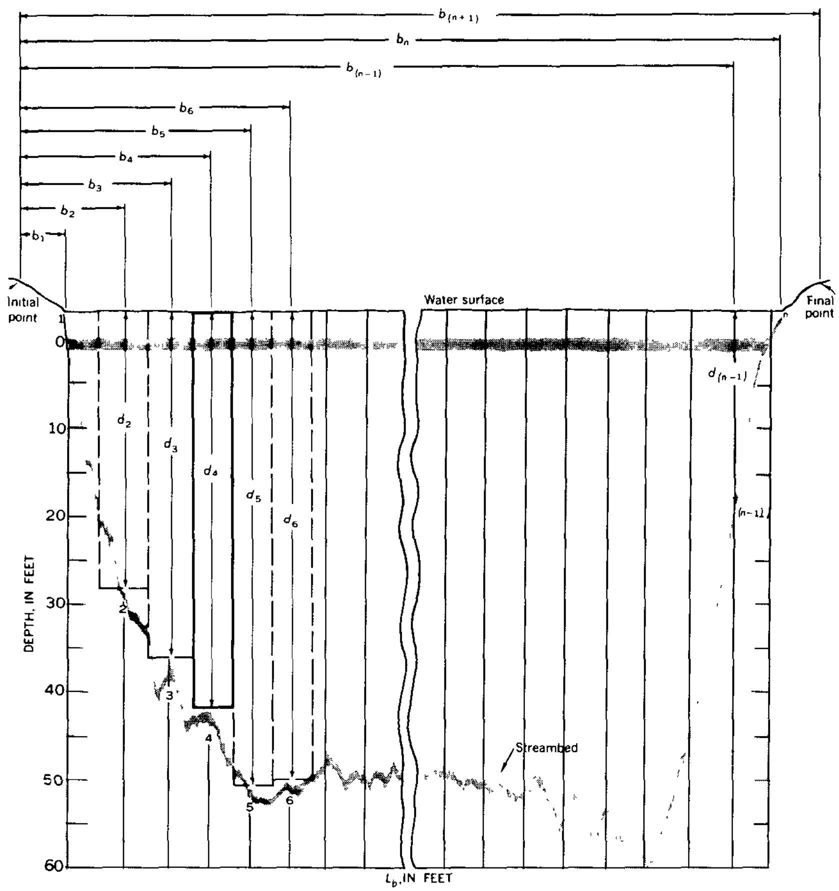


FIGURE 112.—Definition sketch of midsection method of computation superimposed on a facsimile of a sonic-sounder chart.

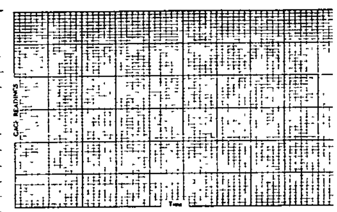
the end of a full count by the counter unit. In the sample measurement the value recorded was three-fourths of that in the table because the sounder chart distance between the last two vertical markings was approximately three-fourths of the normal spacing.

3. Each value recorded in column 3 represents the distance from the initial point (marker) to the observation point where the data were collected. The data in column 2, together with the "distance to float" and the "distance to marker" values recorded on the front sheet of the discharge measurement notes (fig. 113) are used to obtain these cumulative distances. For a moving-boat measurement, these distances are defined as follows:

b_1 = distance from initial point (marker) to edge of water
b_2 = b_1 + measured distance to float from edge of water
b_3 = $b_2 + L_{b_3}$
b_4 = $b_3 + L_{b_4}$

9-275-MB 1968 Temporary National River nr River City, Va. Run No 4-1 Range(L) 73.0 Time S 0752 F 0758
Meas'd Width 1579' Comp Width 1545' Width/Area Adj Coef 1.022 No secs 30

Angle α	L_b	Dist. from initial point	Width	Depth	Pulses per second	V_v	Sin α	$\frac{V_v}{\text{Sin } \alpha}$	Area	Discharge	Remarks
IP		0									IP to LEW = 28'
LEW		28 0	19.5	0					0		
20	39 0	67.0	36.0	9 0	250	4 50	.342	1 54	324	499	
25	33 1	100.1	48 2	39 0	340	6 09	423	2 58	1880	4850	(½ count)
30	63 2	163 3	63 5	38 0	370	6 62	.500	3 31	2410	7980	
29	63 8	227 1	60 2	37 5	340	6 09	485	2 95	2260	6670	
39	56 7	283 8	56 3	37 0	340	6.09	.629	3 83	2080	7970	
40	55 9	339 7	57.8	35 0	330	5 91	643	3 80	2020	7680	
35	59 8	399 5	57 8	35 5	330	5 91	.574	3 39	2050	6950	
40	55 9	455.4	52 4	33 0	330	5.91	643	3 80	1730	6570	
48	48.8	504 2	48 8	32 5	350	6 27	743	4.66	1590	7410	
48	48 8	553 0	50 6	32 0	340	6 09	743	4.52	1620	7320	
44	52 5	605 5	50.2	31.5	340	6 09	695	4 23	1580	6680	
49	47 9	653.4	48 4	31.0	320	5 74	755	4 33	1500	6500	
48	48.8	702 2	52 4	30 0	330	5 91	743	4 39	1570	6890	
40	55.9	758 1	53 8	28 5	320	5 74	643	3 69	1530	5650	
45	51.6	809 7	52.6	26.5	300	5 38	707	3 80	1390	5280	
43	53 4	863.1	54 2	27 0	330	5 91	.682	4 03	1460	5880	
41	55.1	918 2	50.5	27 0	350	6.27	.656	4.11	1360	5590	
51	45.9	964 1	51.3	26 0	330	5 91	777	4 59	1330	6100	
39	56.7	1020.8	55.0	25 0	320	5.74	.629	3.61	1380	4980	
43	53 4	1074.2	54.6	25 0	320	5 74	682	3 91	1360	5320	
40	55 9	1130 1	57 4	25.0	330	5 91	643	3.80	1440	5470	
36	59.0	1189 1	51 4	24 5	330	5 91	588	3 48	1260	4380	
53	43 9	1233 0	48.2	23 5	320	5 74	799	4 59	1130	5190	
44	52.5	1285 5	53 8	22 5	320	5.74	.695	3 99	1210	4830	
41	55 1	1340 6	58.8	22.5	330	5 91	656	3 88	1320	5120	
31	62.6	1403.2	60 8	22 0	330	5 91	515	3 04	1340	4070	
36	59 0	1462 2	55 4	22 5	320	5 74	588	3.38	1250	4220	
19	51 8	1514.0	55 4	12 0	340	6 09	326	1 99	665	1320	(¾ count)
REW	59 0	1573.0	29 5	0					0		
FP		1596.0									REW to FP = 23'
	1545 0		1544 7						42,039	157 369	
						1579			x 1 022	x 1 022	Width/Area Adj Coef
Width/Area Adjustment Coef.						1545					
									43,000	160 831	
					or	1.022				x 91	Vel. Corr Coef.
										146,000	



9-275-MB 1968 Temporary
UNITED STATES
DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY
WATER RESOURCES DIVISION
DISCHARGE MEASUREMENT NOTES
Meas. No. 4
Comp. by J H I
Checked by J H D

Sta. No 1-1665 8
National River nr River City, Va
Date Oct 1, 1967 Party Smith, Jones, Doe
G H change 000 in 010 hrs G H 45.94 Disch. 146,000
Vel Corr Coef. 91 Scale 1000 Meter no Z-4 Date rated 7-5-67

Time		Recorder	Inside	Outside
0730		45.94	45.94	45.94
0752	S	45.94		
0758	F	45.94		
0815		45.94	45.94	45.94
Weighted M. G. H.				
G. H correction				
Correct M. G. H		45.94	45.94	45.94

Meter 3.5 feet below surface
Transducer 2.5 feet below surface
Distance to float LEW 39.0' REW 59.0'
Distance to marker LEW 28.0' REW 23.0'
Meas plots % diff from rating
Meas'd 200 feet, ~~ , ~~ below gage and
Check-bar chain found —
changed to — at —
Correct —
Levels obtained —

Measurement rated excellent (2%), good (5%), fair (8%), poor (over 8%), based on following conditions: Cross section Fairly Regular: clay bottom
Flow Fairly Regular Weather Cloudy
Other Good Distribution Air °F at
Gage Inspected Water 65 °F at 0745
Record removed Yes Intake flushed
Observer None
Control
Remarks
G.H. of zero flow

FIGURE 113.—Sample computation notes of a moving-boat measurement.

. . . .

$b_{(n-1)} = b_{(n-2)} + L_{b_{(n-1)}}$

$b_n = b_{(n-1)}$ + measured distance from float to edge of water

$b_{(n+1)} = b_n$ + distance to final point (marker).

4. Each of the incremental widths in column 4 represents the distance that extends laterally from half the distance from the preceding meter location $(x-1)$, to half the distance to the next, $(x+1)$. These values are obtained by using the distances in column 3.

5. Each of the values in column 5 represents the stream depth at a sampling point in the cross section. These values are obtained by adding the transducer depth to each of the depth readings recorded on the sounder chart at the sampling locations.

6. The data in column 6 are the pulses-per-second readings recorded by the notekeeper during the measurement.

7. The values recorded in column 7 represent the instantaneous velocity of the water past the vane at each observation point. They are read directly from the meter-rating table (fig. 108), using the pulses-per-second values of column 6.

8. The data in column 8 are the sine function values of the angle readings in column 1. These values may be obtained from the sine table in figure 108, using the angle readings of the first column.

9. Each of the values in column 9 represents the stream velocity normal to the cross section at that particular sampling point.. To obtain these values, it is necessary to multiply each V_V value in column 7 by the corresponding sin α value in column 8.

10. The values in column 10 represent the individual subsection areas for the measurement. They are obtained by multiplying the widths of column 4 by their corresponding depths in column 5. The incremental areas are then summed to provide the total unadjusted area for the measurement.

11. Each quantity in column 11 represents the unadjusted discharge through one of the subsections of the discharge measurement. These values are summed to provide the total unadjusted discharge of the measurement.

12. This column is used for recording any descriptive remarks pertaining to the measurement.

ADJUSTMENT OF TOTAL WIDTH AND AREA

As explained on page 186, the relation expressed by the equation $L_b \approx L_V \cos \alpha$ is used to obtain the incremental widths across the stream. This equation is based on the assumption that a right-triangle relationship exists among the velocity vectors involved. If the flow is not normal to the cross section, that assumed situation does not exist and the use of the equation can result in a computed width that is too large or too small (fig. 114), depending on whether the vector quantity representing the oblique flow has a horizontal component that is opposed to, or in the direction of, that of the boat path. Thus in figure 114 the computed width would be *AB′* of the right triangle *AB′C* rather than the true width *AB* of oblique triangle *ABC*. In this case the computed width is too large, whereas the computed width *DE′* of right triangle *DE′F* is less than the actual width *DE* of oblique triangle *DEF*.

Ideally the correction for error in the computed width would be applied to that particular increment in the cross section where the error occurred. However, in practice only the overall width is directly measured, and thus only the total width is available for comparison with the computed quantities. Therefore, if the sum of the computed incremental widths does not equal the total measured width of the cross section, it is assumed that each increment requires a proportionate adjustment.

The moving-boat method uses the relation between the measured and computed widths of the cross section to determine a width/area adjustment factor. To obtain that coefficient, the measured width of the cross section is divided by its computed width, that is

$$k_B = \frac{B_m}{B_c} \qquad (22)$$

where

k_B = width/area adjustment factor,
B_m = measured width of cross section, and
B_c = computed width of cross section.

The coefficient (k_B) is then used to adjust both total area and total discharge of the measurement, on the basis of the previously mentioned assumption that the error in width is evenly distributed, on a percentage basis, across each width increment of the cross section.

The computation notes in figure 113 provide an example of the application of a width/area adjustment coefficient.

ADJUSTMENT OF MEAN VELOCITY AND TOTAL DISCHARGE

During a moving-boat discharge measurement, the current meter is set at a predetermined fixed depth of from 3 to 4 ft (0.9 to 1.2 m) below the water surface. In other words, this technique uses the subsurface method of measuring velocity. (See page 136.) The measurement is computed by using constant-depth subsurface velocity observations without adjustment coefficients, as though each observed velocity were a mean in the vertical. In adjusting the computed discharge, each measured velocity should ideally be multiplied by a coefficient to adjust it to the mean velocity in its vertical. However, it

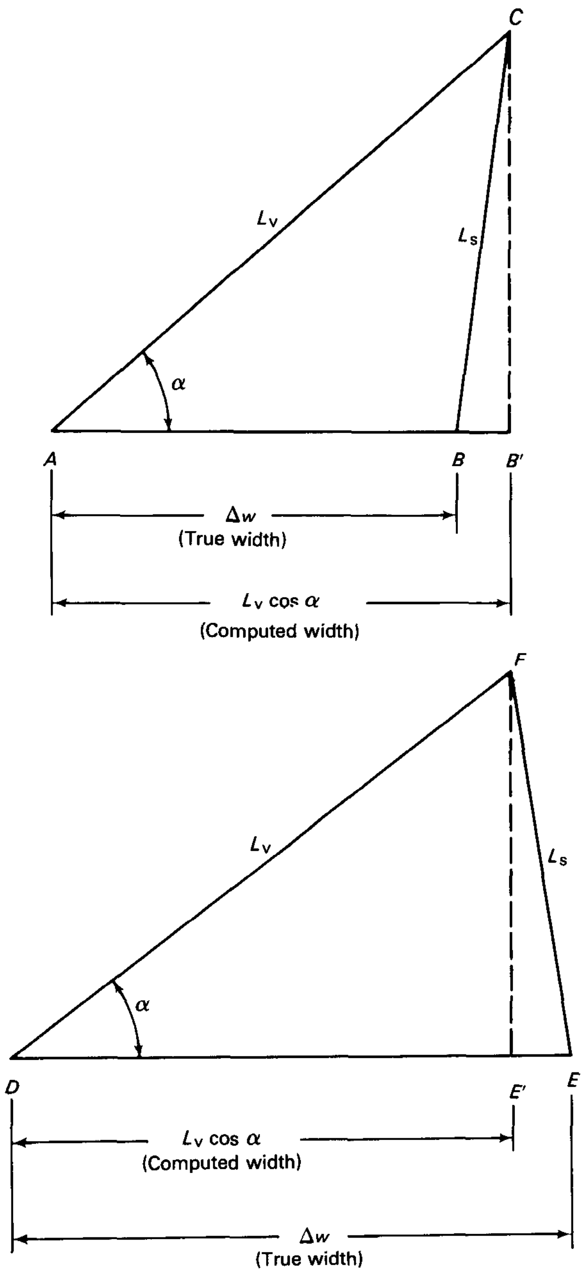


FIGURE 114.—Comparison of actual and computed values of incremental widths.

is assumed that in the larger streams where the moving-boat technique would be applicable, these coefficients would be fairly uniform across a section, thus permitting the application of an average velocity coefficient to the total discharge. Information obtained from several vertical-veolocity curves, well distributed across the measurement section, would be needed to determine a representative velocity coefficient for the total cross section.

DETERMINATION OF VERTICAL-VELOCITY ADJUSTMENT FACTOR

Vertical-velocity curves (p. 132–133) are constructed by plotting observed velocities against depth. The vertical-velocity curve method calls for a series of velocity observations (by conventional methods) at points well distributed between the water surface and the streambed. Normally these points are chosen at 0.1-depth increments between 0.1 and 0.9 of the depth. Observation should also be made at least 0.5 ft (0.15 m) from the water surface and 0.5 ft (0.15 m) from the streambed; for this particular application, a velocity reading should be made at the moving-boat sampling depth of from 3 to 4 ft (0.9 to 1.2 m) below the water surface. Once the velocity curve has been constructed, the mean velocity for the vertical can be obtained by measuring the area between the curve and the ordinate axis with a planimeter, or by other means, and then dividing this area by the length of the ordinate axis.

To obtain a velocity-correction coefficient at location x in the cross section, the mean velocity in the vertical is divided by the observed velocity at the measured depth, that is,

$$k_V = \frac{\overline{V}}{V}, \tag{23}$$

where

k_V = vertical-velocity adjustment factor,
$\overline{V}$ = mean velocity in the vertical, and
V = observed velocity (3- or 4-ft depth).

To arrive at a representative average coefficient, coefficients should be determined at several strategically located verticals that are representative of the main portion of the streamflow. Once an average coefficient has been determined, it should not be necessary to redetermine it each time when making future discharge measurements at the same site. However, it would be necessary to test its validity at several widely varying stages and, in estuaries, at widely different parts of the tidal cycle.

Investigations on the Mississippi River at both Vicksburg and St. Louis, on the Hudson River at Poughkeepsie, and on the Delaware

River at Delaware Memorial Bridge all indicated coefficients that lie in the narrow range of 0.90 to 0.92 for adjusting the subsurface velocity to the mean velocity. Carter and Anderson (1963) present a table of velocity ratios and standard deviations for various relative depths. That table indicates that for depths of 10 ft (3 m) or more, an average coefficient of 0.90 is satisfactory for adjusting velocities obtained 4 ft (1.2 m) below the surface to mean velocity. The sample from which the data in that table were obtained consisted of 100 stream sites at each of which 25 to 30 verticals had been used. Similar conclusions concerning subsurface velocity coefficients can be drawn from table 2 (p. 132).

APPLICATION OF VELOCITY ADJUSTMENT TO COMPUTED DISCHARGE

Application of the vertical-velocity adjustment factor is made immediately after the width-area adjustment has been applied to the computed discharge. In other words, after the computed discharge has been multiplied by the width-area adjustment factor, the resulting product is multiplied by the vertical-velocity adjustment factor. The final product is the adjusted, or "true," discharge for the measurement. (See fig. 113.)

SELECTED REFERENCES

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Carter, R. W., and Anderson, I. E., 1963, Accuracy of current-meter measurements: Am. Soc. Civil Engineers Jour., v. 89, no. HY4, p. 105–115.

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CHAPTER 7.—MEASUREMENT OF DISCHARGE BY TRACER DILUTION

GENERAL

The measurement of stream discharge by dilution methods depends on the determination of the degree to which an added tracer is diluted by the flowing water. Any substance can be used as a tracer if it meets the following criteria:

1. It dissolves readily in water at ordinary temperatures.
2. It is either absent in the water of the stream or present only in very low concentrations.
3. It is not decomposed in the water of the stream and is not retained or absorbed in significant quantity by sediments, plants, or other organisms.
4. It can be detected in extremely low concentrations by simple methods.

5. It is harmless to man and animals in the concentration it assumes in the stream.

Until recent years chemical salts, primarily common salt (NaCl), were usually used as the tracers injected into the streams. The use of salt tracers, on all but the smallest streams, was limited, however, to the sudden-injection method, because of the difficulty of handling the large quantities of salt solution required by the usually more accurate constant-rate-injection method. In recent years, and particularly in the U.S.A., the use of dye tracers in the constant-rate-injection method has become the most popular method of discharge measurement by tracer dilution. That has resulted from the development of fluorescent dyes and fluorometers that can detect those dyes at very low concentrations. The use of fluorescent dyes is not as popular in Europe as in the U.S.A. There, colorimetric analysis, using sodium dichromate as the tracer dye, is the most widely used means of measuring discharge by the constant-rate-injection method. However, where the sudden-injection method is to be used, the use of common salt as a tracer is still preferred, particularly on the smaller streams, because of the greater ease in handling salt and in determining concentrations. Radioactive elements, such as gold 198 and sodium 24, have also been used in recent years as the tracers in the sudden-injection method, but the use of such elements is still (1980) considered experimental.

The tracer-dilution methods of measuring discharge are more difficult to use than the conventional current-meter method, and under most conditions the results are less reliable. Dilution methods should therefore not be used when conditions are favorable for a current-meter measurement of discharge. Tracer-dilution methods of measuring discharge in open channels can be used advantageously in rough channels that carry highly turbulent flow.

THEORY OF TRACER-DILUTION METHODS

In the tracer-dilution methods of measuring discharge, a tracer solution is injected into the stream to be diluted by the discharge of the stream. From measurements of the rate of injection, the concentration of the tracer in the injected solution, and the concentrations of the tracer at a sampling cross section downstream from the injection site, the stream discharge can be computed.

Either of two methods may be used for determining the discharge of a stream by tracer dilution. The first method, the constant-rate-injection method, requires that the tracer solution be injected into the stream at a constant flow rate for a period sufficiently long to achieve a constant concentration of the tracer in the streamflow at the downstream sampling cross section. The second method, the sudden-injection method, requires the instantaneous injection of a slug of tracer solution and an accounting of the total mass of tracer at the sampling cross section. Another use is often made of the sudden-injection method that is not concerned with degree of dilution. Where the cross-sectional area of the flow is constant, as in pressure conduit, the sudden-injection method may be used to determine the velocity of flow; discharge can then be computed by using that velocity and the known cross-sectional area. (See p. 533.)

THEORY OF THE CONSTANT-RATE-INJECTION METHOD

A constant-rate-injection system is shown schematically in figure 115. If the tracer is injected for a sufficiently long period, sampling of the stream at the downstream sampling cross section will produce a concentration-time curve similar to that shown in figure 116. The stream discharge is computed from the equation for the conservation of mass, which follows:

$$QC_b + qC_1 = (Q + q)C_2 \tag{24}$$

or

$$Q = \left[\frac{C_1 - C_2}{C_2 - C_b}\right] q,$$

where

- q is the rate of flow of the injected tracer solution,
- Q is the discharge of the stream,
- C_b is the background concentration of the stream,
- C_1 is the concentration of the tracer solution injected into the stream, and
- C_2 is the measured concentration of the plateau of the concentration-time curve (fig. 116).

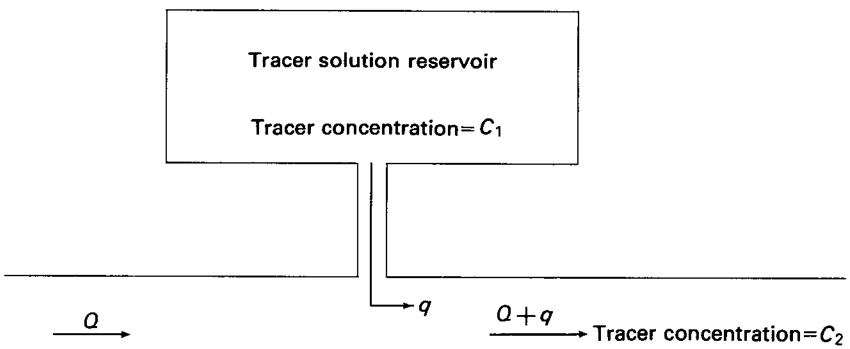


FIGURE 115.—Constant-rate-injection system.

THEORY OF THE SUDDEN-INJECTION METHOD

If a slug of tracer solution is instantaneously injected into a stream, sampling of the stream at the downstream sampling cross section will produce a concentration-time curve similar to that shown in figure 117. The equation for computing stream discharge, which is again based on the principle of the conservation of mass, is

$$Q = \frac{V_1 C_1}{\int_0^\infty (C - C_b)dt}, \tag{25}$$

where

Q is the discharge of the stream,
V_1 is the volume of the tracer solution introduced into the stream,
C_1 is the concentration of the tracer solution injected into the stream,
C is the measured tracer concentration at a given time at the downstream sampling site,
C_b is the background concentration of the stream, and
t is time.

The term $\int_0^\infty (C - C_b)\,dt$ is the total area under the concentration-time curve. In practice the term $\int_0^\infty (C - C_b)dt$ can be approximated by the term

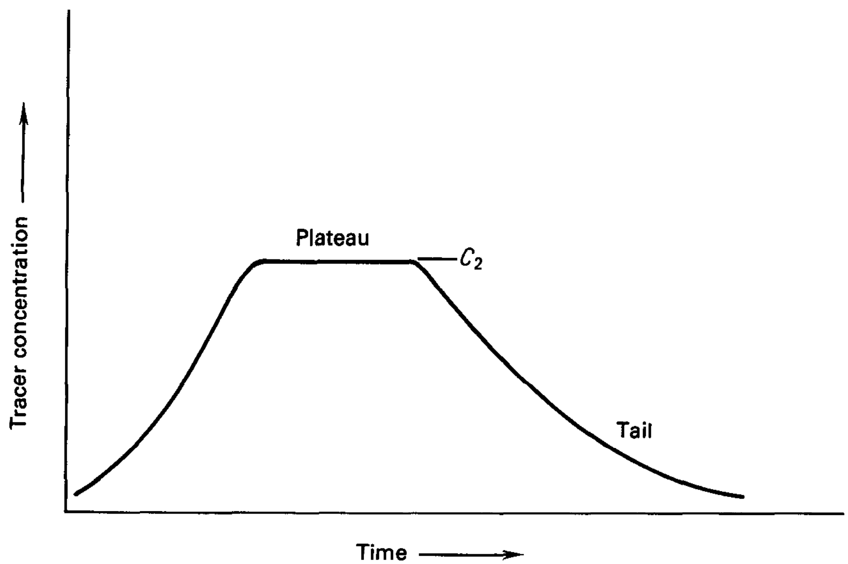


FIGURE 116.—Concentration-time curve at downstream sampling site for constant-rate injection.

$$\sum_{i=1}^{N} (C_i - C_b)(t_{i+1}-t_{i-1})/2,$$

where

i is the sequence number of a sample,
N is the total number of samples, and
t_i is time when a sample, C_i, is obtained.

FACTORS AFFECTING THE ACCURACY OF TRACER-DILUTION METHODS

Even if it is assumed that measurements of concentrations and of the injection rate are error free, there still remain three factors that affect the accuracy of tracer-dilution methods of measuring discharge. Those factors are stream turbidity, loss of tracer between the injection site and the downstream sampling site, and incomplete mixing throughout the stream cross section before the downstream sampling section is reached.

TURBIDITY

Turbidity may either increase or decrease the recorded tracer fluorescence depending upon the relative concentrations of tracer and turbidity (Feuerstein and Selleck, 1963). To minimize the effect of turbidity, samples should be permitted to stand long enough to allow

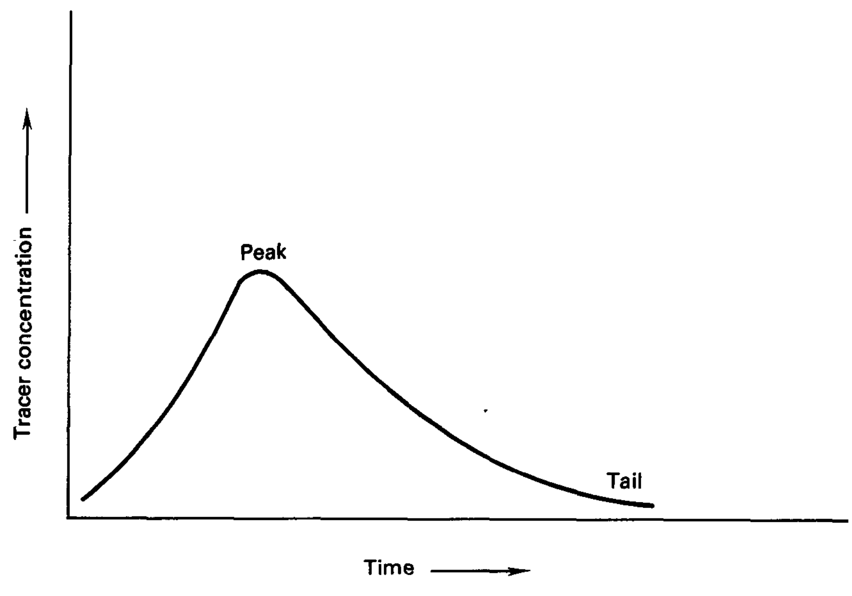


FIGURE 117.—Concentration-time curve at downstream sampling site for sudden injection.

suspended solids to settle prior to fluorometric analysis (p. 231, 240). Use of a centrifuge for the purpose of separating the suspended solids is even more effective, but a centrifuge is seldom used for that purpose in stream-gaging work.

LOSS OF TRACER

The computation of stream discharge, as mentioned earlier, is based on equations for the conservation of mass. Consequently, the accuracy of the computed discharge will be adversely affected if some of the tracer is lost in the reach of channel between the injection site and the downstream sampling site. Tracer losses are primarily due to sorption and chemical reaction between the tracer and one or more of the following: streambed material, suspended sediments, dissolved material in the river water, plants, and other organisms. For most of the tracers used, chemical reaction is a minor factor in comparison with sorption. The degree of tracer loss by sorption for a particular tracer varies primarily with the type and concentration of suspended and dissolved solids in the water. In routine stream gaging by the tracer-dilution method no attempt is made to quantify tracer loss by sorption; instead, the "best" tracer available from the standpoint of being least affected by sorption is used.

Photochemical decay is also a source of tracer loss which varies with the tracer material used and its residence time in direct sunlight. Loss from that source is usually negligible, even with fluorescent dyes, if the proper dye is used and if residence time in direct sunlight is limited to only a few hours, as it usually is in stream-gaging work.

Equations 24 and 25 show that loss of tracer will result in a computed discharge that exceeds the true stream discharge.

CRITERIA FOR SATISFACTORY MIXING

Tracer-dilution measurements require complete vertical and lateral mixing at the sampling site. Vertical mixing is usually accomplished very rapidly compared to lateral mixing; therefore, the distance required for lateral mixing is the primary consideration. Frequently, long reaches are needed for complete lateral mixing of the tracer. The mixing distance will vary with the hydraulic characteristics of the reach. Engmann and Kellerhals (1974) have demonstrated that ice cover significantly reduces the mixing capacity of a reach of river.

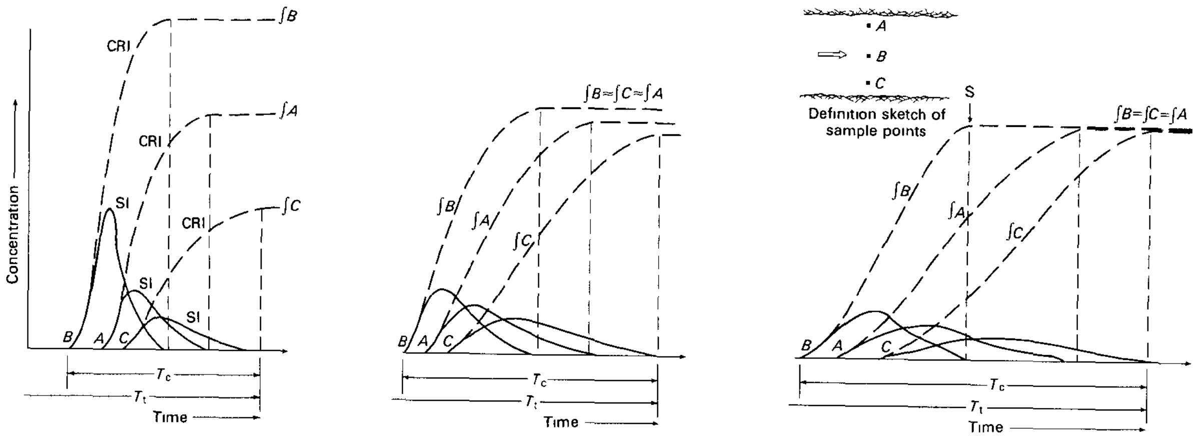
When the constant-rate-injection method is used, complete mixing is known to have occured when the concentration C_2, shown in figure 116, has the same value at all points in the downstream sampling cross section. When the sudden-injection method is used, complete mixing is considered to have occurred when the area under the concentration-time curve, shown in figure 117, has the same value at all points in the downstream sampling section.

For a reach of channel of given geometry and stream discharge, the length of reach required for adequate mixing of the tracer is the same for either of the two methods of tracer injection. Several formulas are available for estimating the required mixing length for a particular set of conditions, but these formulas, while useful as guides, are too simplistic to give adequate consideration to the degree of mixing desired. Perfect mixing is seldom the optimum goal (see below) because perfect mixing usually requires an extremely long reach of channel, along with a correspondingly long period of injection in the constant-rate-injection (CRI) method and a correspondingly long period of sampling in the sudden-injection (SI) method.

Figures 116 and 117 are only rudimentary illustrations of the two methods. For either method to be successful an understanding is needed of the interrelations among mixing length and injection and sampling times. Figure 118 attempts to illustrate those interrelations for both types of injection. It is important to realize that unless adequate mixing is known to exist at a given sampling site, the tracer cloud in the SI method must be sampled for its entire time of passage at several locations laterally in the channel, such as at A, B, and C in figure 118. Similarly, in the CRI method the plateau concentration must be sampled at several locations laterally in the channel. Experience indicates that regardless of method or stream size, at least three lateral sampling points should be used at each sampling site.

Figure 118 indicates that there is an optimum mixing length for a given stream reach and discharge. Use of too short a distance will result in an inaccurate accounting of the tracer mass passing the sampling site. Use of too great a distance will yield excellent results, but only *if it is feasible to inject the tracer for a long enough period* (CRI method) or to sample for a long enough period (SI method). An optimum mixing length is one that produces mixing adequate for an accurate discharge measurement but does not require an excessively long duration of injection or sampling.

As mentioned earlier, figure 118 shows that the tracer cloud resulting from a sudden injection must be sampled at the sampling site from the time of its first appearance there until the time (T_t) of its disappearance at all points in the sampling cross section. For the same mixing reach and discharge, if the CRI method is used, a plateau will first be reached at all points in the sampling cross section at time T_t after injection starts at the injection site. Thus, it is seen that for the CRI method the duration of injection must at least be equal to T_t, and injection should continue long enough thereafter to



I. Short Sampling Reach

1 Areas under curves differ considerably in the SI method, lateral mixing incomplete

2 Rapid buildup to concentration plateaus of different levels in the CRI method

3 Discharge results will usually be poor, even by the use of such expedients as velocity-integrated sampling or discharge-weighting of the concentration data.

II Optimum Sampling Reach

1 Areas under the curves nearly equal in the SI method, lateral mixing nearly complete

2 Fast buildup to plateaus of nearly equal concentration in the CRI method, lateral mixing adequate

3 Reliable discharge measurement can be made by the CRI method using a moderate injection time that exceeds T_c

III. Excessively Long Sampling Reach

1 Areas under the curves are identical in the SI method, perfect mixing

2 Slow buildup to plateaus of equal concentration in the CRI method but only after a long period of injection

3 Minimum duration of constant injection is equal to total time, T_c, that slug dye cloud is present

4 Reliable discharge measurement can be made by CRI method if injection period is long enough to exceed T_c

5 Results by CRI method will be poor if injection time is too short or sampling is discontinued too soon Samples taken at time S would give impression that mixing is poor, as well as suggesting loss of dye

FIGURE 118.—Comparison of concentration-time curves obtained by SI and CRI methods, observed at three points in sampling cross section, for sampling reaches of three different lengths.

insure adequate sampling of the plateau. It should be noted that a stable condition of mixing is usually first attained in midchannel. Tracer lag along the streambanks will generally prolong the time required for complete lateral mixing.

The CRI method has become increasingly popular because reliable injection apparatus is available and the sampling process is relatively simple. If an injection period well in excess of T_c is used, the sampling can be done leisurely at several points in the sampling cross section. In the SI method the injection is simple, but the sampling process is much more demanding. More reliable results are usually obtained by the CRI method.

Figure 118 indicates that a satisfactory discharge measurement requires "nearly equal" areas under the concentration-time curves for the sampling points in the SI method and "nearly equal" concentration plateaus for the sampling points in the CRI method. A numerical criterion—P_m, the percentage of mixing—is often used as a quantitative index of the consistency of sampling results for the sampling cross section. A satisfactory discharge measurement can usually be obtained if the value of P_m is 95 percent or greater. The computation of P_m is discussed in the paragraphs that follow.

Percentage of mixing, P_m, is defined in this manual by the equation

$$P_m = 100 - \{|X_A - X_m|Q_A + |X_B - X_m|Q_B + |X_C - X_m|Q_C + \ldots\}\frac{50}{X_m Q}, \tag{26}$$

where

$X_A, X_B, X_C, \ldots$ are the areas under the concentration-time curves or, for constant-rate injection, the C_2 concentrations at the points A, B, C, . . . across the sampling section;

X_m is the mean area under the concentration-time curves or, for constant-rate injection, the mean C_2 concentration for points A, B, C, . . . across the sampling section;

$Q_A, Q_B, Q_C, \ldots$ are the subsection discharges applicable to the points, A, B, C, . . .;

Q is the total stream discharge.

If the distribution of discharge is unknown at the sampling section, the cross-sectional subareas applicable to each point should be used in place of Q_A, Q_B, Q_C; total cross-sectional area should be used in place of Q. If both discharge and area distribution are unknown, as will often be the case, the appropriate widths may be used. Terms $|X_A - X_m|$, $|X_B - X_m|$, $|X_C - X_m|$, . . . are absolute values.

The percentage of mixing can also be determined graphically, as shown in figure 119. Values of C_2 or $\int_0^\infty (C - C_b)dt$, depending on the injection method used, are plotted for each sampling point against the distance from an initial point at water's edge. The mean value C_2 or $\int_0^\infty (C - C_b)dt$ is determined by dividing the area under the distribution curve for C_2 or $\int_0^\infty (C - C_b)dt$ by the total width of the section. The percentage of mixing, P_m, is then given as

$$P_m = \left(\frac{A}{A + B}\right) 100 \qquad (27)$$

where

A is the area under the C_2 distribution and mean C_2 curves, or the area under the $\int_0^\infty (C - C_b)dt$ distribution and mean $\int_0^\infty (C - C_b)dt$ curves;

B is the area above the C_2 distribution and under the mean C_2 curves or the area above the $\int_0^\infty (C - C_b)dt$ distribution and under the mean $\int_0^\infty (C - C_b)dt$ curves.

The graphical technique illustrated in figure 119 can also be used if the distribution of discharge or area is known. The only change in the procedure that is required is a change in the abscissa of figure 119. In place of "distance from edge of water" either cumulative discharge or cumulative area from edge of water is substituted.

CALIBRATION OF MEASUREMENT REACH

Several theoretical studies have been made by the U.S. Geological Survey (for example, Yotsukura and Cobb, 1972) in which equations have been derived for determining the length of measurement reach required for satisfactory or complete mixing. However, the derived equations cannot be applied in the field without detailed information

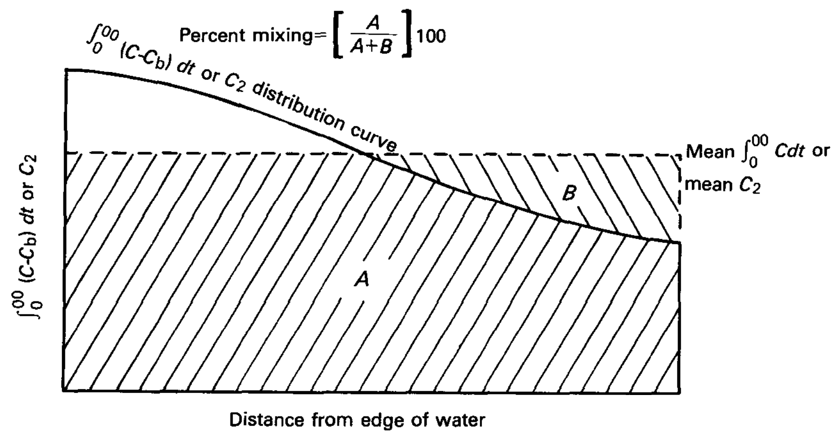


FIGURE 119.—Concentration-distribution curve illustrating the graphical method of determining the percentage of mixing.

on channel geometry and velocity distribution for the reach of channel. Consequently several gross formulas have been developed from the theoretical equations strictly for use as guides in determining a satisfactory length of measurement reach. Some of the simplified formulas follow:

a. Point injection at midchannel (English units)—

1. $$L = 0.25 \frac{V}{(gDS)^{1/2}} \frac{W^2}{D}, \tag{28}$$

where

L is distance downstream from the injection point,
V is mean velocity,
g is acceleration of gravity,
D is mean depth,
S is gradient of water surface or streambed, and
W is stream width.

2. $$L = 0.6 \frac{R^{1/6}}{ng^{1/2}} \frac{W^2}{D}, \tag{29}$$

where R is the hydraulic radius and n is the Manning roughness coefficient.

3. $$L = 2V \frac{(W^2)}{D} \cdot \tag{30}$$

b. Point injection at bankside (English units)—

$$L = 8V \frac{W^2}{D} \cdot \tag{31}$$

Studies indicate that if more than one injection point is used, the mixing distance will vary inversely with the square of the number of injection points, provided that the injection points are located so that the dye will disperse equal distances both to the left and right of the injection points. This statement may seldom hold true for natural stream conditions, but it does indicate that the mixing length may be shortened considerably by using additional injection points.

In Europe the formula commonly used as a guide for determining the required length, L, between the injection site and sampling section is

$$L = 0.13C \frac{(0.7C+6)}{g} \frac{b^2}{h} \text{(metric units)}, \tag{32}$$

where

b is the average width of the wetted cross section,
h is the average depth of flow,
C is the Chezy coefficient for the reach ($15<C<50$), and
g is acceleration of gravity.

Equation 32 is intended for use where injection is made at a single point in midstream.

Before making a discharge measurement by the tracer-dilution method, the proposed measurement reach should be calibrated to determine the length required for adequate mixing and the possibility of significant tracer loss. If repeated tracer-dilution measurements of the stream are to be made, it will be especially advantageous to make the calibration study comprehensive. One of the equations, 28 to 32, may be used as a guide in selecting a length of reach for the calibration tests; the quantity of tracer to be used for any given discharge is discussed later on pages 235–237. Instructions for calibration testing follow.

Select an injection site on the stream, and on the basis of one of the equations, 28 to 32, select several downstream sampling sites. Inject the tracer and then sample the water at three or more points in the cross section at each of the selected sampling sites. Take adequate samples at each point to define the plateau of the concentration-time curve if the constant-rate-injection method is used, or to define the entire concentration-time curve if the sudden-injection method is used. Analyze the samples and determine the percentage of mixing by use of equation 26. As mentioned earlier, for a satisfactory measurement of discharge the percentage of mixing should be at least 95. Mixing distances and losses may vary with discharge, and the reach may therefore have to be calibrated at more than one discharge. The quantity of tracer lost, if any, between successive sampling sites can be determined by a comparison of the values of C_2 (constant-rate-injection method) or of $\int_0^\infty (C - C_b)dt$ (sudden-injection method) obtained at each site. That determination provides a basis for the final selection of a sampling site from the several sites tested.

EFFECT OF INFLOW OR OUTFLOW BETWEEN INJECTION AND SAMPLING SITES

A satisfactory tracer-dilution measurement of discharge made in a reach that has no inflow or outflow between the injection and sampling sites will give the discharge occurring at all cross sections in the reach.

If tributary inflow enters the reach and if the tributary inflow is well mixed with the water in the main stream, the discharge measured will be that at the sampling site. If the tributary inflow is not well mixed, that fact may be evident from the difference in concentrations at the various sampling points in the sampling cross section; a low percentage of mixing would indicate that the particular sampling site could not be used for a satisfactory measurement of discharge.

If outflow occurs between the injection and sampling sites and if the tracer becomes completely mixed anywhere upstream from the outflow channel, the discharge measured at the sampling site will be that for the reach upstream from the outflow channel. If the tracer does not become completely mixed upstream from the outflow channel, the discharge measured at the sampling site will be indeterminate; the magnitude of the "measured" discharge will be dependent on the quantities of both tracer and water that are carried off in the outflow.

MEASUREMENT OF DISCHARGE BY FLUORESCENT DYE DILUTION

As mentioned earlier, the most popular type of tracer for the measurement of streamflow in the U.S.A. is fluorescent dye. The cost of the dye is relatively small because the quantity of dye needed for a discharge measurement is small; modern fluorometers are capable of accurately measuring dye concentrations of less than 1 μg/L (microgram per liter) and can detect concentrations as low as 0.02 μg/L.

FLUORESCENT DYES

Fluorescence occurs when a substance absorbs light at one wavelength and emits it at another, and usually longer, wavelength. The dyes commonly used as tracers are strongly fluorescent. They are organic dyes of the rhodamine family and are commercially available.

Dyes selected for use as tracers should (1) have a high detectability range, (2) have little effect on flora or fauna, (3) have a low sorption tendency, (4) have a low photochemical decay rate, (5) be soluble and disperse readily in water, (6) be chemically stable, (7) be inexpensive, (8) be easily separated from common background fluorescence, and (9) be easy to handle.

On the basis of recent studies in the U.S.A. on the adsorption potential and detectability of dyes, Rhodamine WT dye is recommended as the best dye available for use in dye-dilution measurements of discharge. Formerly used at times, but no longer recommended are Rhodamine B, BA, and Fluorescein.

FLUOROMETER

The fluorometer is an instrument that gives a measure of the strength of the light emitted by a fluorescent substance. Figure 120 is a schematic diagram of a fluorometer. The flurometer briefly described in this manual is the Turner model 111 fluorometer, and it is discussed because it is the instrument that is in general use by the U.S. Geological Survey. However, there are many satisfactory fluorometers that are commercially available, and for detailed instructions concerning any particular fluorometer it is necessary that the hydrographer consult the service manual prepared by the manufacturer of that instrument. The fluorometry techniques described in this manual are oriented toward use of the Turner fluorometer, but those techniques are applicable to most types of fluorometer. (Note.—The use of brand names in this manual is for identification purposes only and does not imply endorsement by the U.S. Geological Survey.)

DESCRIPTION OF FLUOROMETER

The principle of the operation of the Turner model 111 fluorometer is described in the operating manual (1963, p. 12) as follows:

This fluorometer is basically an optical bridge which is analogous to the accurate Wheatstone Bridge used in measuring electrical resistance. The optical bridge measures the difference between light emitted by the sample and that from a calibrated rear light path. A single photomultiplier surrounded by a mechanical light interrupter sees light alternately from the sample and the rear light path. Photomultiplier output is alternating current, permitting a drift-free A–C amplifier to be used for the first electronic stages. The second stage is a phase-sensitive detector whose output is either positive or negative, depending on whether there is an excess of light in the forward (sample) or rear light path, respectively. Output of the phase detector drives a servo amplifier which is in turn connected to a servo motor. The servo motor drives the light cam (and the "fluorescence" dial) until equal amounts of light reach the photomultiplier from the sample and from the rear light path. The quantity of light required in the rear path to balance that from the sample is indicated by the "fluorescence" dial. Each of this dial's 100 divisions add equal increments of light to the rear path by means of a light cam.

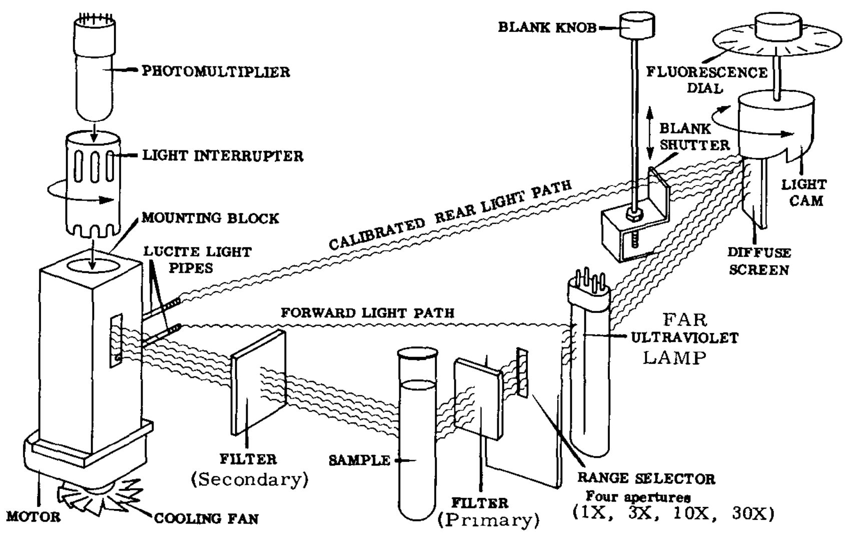


FIGURE 120.—Schematic diagram of the fluorometer (from G. K. Turner Associates, 1963, p.13).

Lamps.—At least three different lamps can be used as a light source for the fluorometer when the fluorescence of rhodamine dyes is tested. They are the general purpose ultraviolet lamp, the far ultraviolet lamp, and the green T–5 lamp. Although all three lamps work well, the far ultraviolet and green T–5 lamps are recommended as a light source because their outputs are compatible with the fluorescent properties of the dyes. Sensitivity of the fluorometer is increased approximately twofold to tenfold when the far ultraviolet and green T–5 lamps, respectively, are substituted for the general purpose ultraviolet lamp.

Filters.—The sensitivity of the fluorometer is directly related to the filter system employed. Light having undesirable wavelengths may be screened out by placing filters at two points in the fluorometer: (1) between the source light and the water sample and (2) between the water sample and the photomultiplier.

The filter (primary) recommended for use with rhodamine dyes for the absorbed light has a peak color specification of 546 mμ (millimicrons); that is, the greatest amount of light passed through the filter is at a wavelength of 546 mμ. Light at other wavelengths is subdued or eliminated. The recommended filter (secondary) for the emitted light has a peak color specification of 590 mμ. The peak color specifications for these filters are near the peak excitation and emittance wavelengths of the rhodamine dyes and will eliminate most natural background fluorescence. The filters may be used regardless of the type of lamp that is installed.

Doors.—Three main types of doors available for use with the fluorometer are the standard-cuvette (test tube) door, the temperature-stabilizing door, and flow-through door. All three doors are easily interchanged.

The standard-cuvette door is the easiest to use in the field for intermittent sampling and should be used with the green T–5 lamp. If the far ultraviolet lamp is used, a high-sensitivity kit should be installed on this door because it will increase the overall sensitivity of the fluorometer about tenfold. It is important to note that excessive sensitivity will be obtained if the high-sensitivity kit and green T–5 lamp are used together.

The temperature-stabilizing door is recommended for the final testing of samples. This door is similar to the standard-cuvette door with the high sensitivity kit but has, in addition, a water-cooled, copper block that surrounds the cuvette. Cooling water may be pumped or tap water of constant, or near-constant, temperature may be run through the door to stablize the sample temperature. Only round 12×75-mm cuvettes can be used with this door.

The flow through door permits continuous sampling which can be recorded. A pump is used to circulate water from the stream through the cuvette in the door. The results may be recorded on any recorder with a 0 to 1 milliamp or 0 to 10 millivolt readout. The intake hose used with the flow-through door should be made of plastic or other nonabsorptive material. Different fittings and cuvette sizes are available for this door. The sensitivity of the flow-through door is about the same as that of the standard-cuvette door with the high sensitivity kit. That is so because of the larger cuvette used in the flow-through door.

Normally dye concentrations are not determined in the field because it is difficult to attain sufficient accuracy under such conditions. The flow-through door arrangement may be used for preliminary calibration of a reach, however. As a rule, bottle samples are collected and transferred to the laboratory for accurate fluorometric analysis.

EFFECT OF TEMPERATURE ON FLUOROMETRY

Accurate dilution discharge measurements require accurate fluorometry, and accurate fluorometry can only be attained in the laboratory where operating conditions are favorable. Temperature has a significant effect upon the fluorescence intensity of dyes. Fluorescence decreases with increasing temperature. This characteristic of the dyes has been investigated by several researchers, among them Feuerstein and Selleck (1963). For best results, all samples, including background and standard solutions, should be placed in a laboratory temperature bath and kept at constant temperature prior to fluorometric analysis. If the same temperature is used for all samples, no temperature corrections will be needed. If temperatures cannot be held constant, temperature corrections, as given in table 13 for Rhodamine WT dye, should be applied to dial readings or to concentrations.

Dunn and Vaupel (1965) point out the need for corrections to fluorometer dial readings as a result of changing compartment temperatures. However, tests by the U.S. Geological Survey showed that these corrections were needed only during the warmup period. A 1½-to 2-hr warmup will usually eliminate the need for this type of correction, although minor changes may still be observed thereafter. The warmup characteristics of each fluorometer should be determined. If possible, the fluorometer should not be operated where large temperature changes can occur rapidly.

CALIBRATION CHARACTERISTICS OF THE FLUOROMETER

Most fluorometers have a linear calibration ratio, meaning that dye concentration is directly proportional to the dial reading on the fluorometer and is related by the equation

TABLE 13.—*Temperature-correction coefficients for Rhodamine WT dye*

Temperature difference $(T_s - T)$[1] F°	Temperature difference $(T_s - T)$[1] C°	Temperature-correction coefficient
−20	−11.1	1.36
−15	−8.3	1.25
−10	−5.6	1.16
−8	−4.4	1.13
−6	−3.3	1.09
−5	−2.8	1.08
−4	−2.2	1.06
−3	−1.7	1.05
−2	−1.1	1.03
−1	−.6	1.02
0	0	1.00
+1	+.6	.99
2	1.1	.97
3	1.7	.96
4	2.2	.94
5	2.8	.93
6	3.3	.91
8	4.4	.89
10	5.6	.86
15	8.3	.80
20	11.1	.74

[1] T_s is the standard cuvette-sample temperature and T is the cuvette-sample temperature at the time the sample was tested in the fluorometer.

$$C = aD, \tag{33}$$

where

- C is the dye concentration of the sample tested,
- a is a coefficient, and
- D is the fluorometer dial reading for the sample tested.

Solutions of known concentration are tested with the fluorometer, and the relation between concentration and dial reading is determined from equation 33.

Some fluorometers, however, do not have linear calibration ratios, and it is therefore necessary to determine the nature of the relation between concentration and dial reading by testing with standard solutions over a wide range of concentrations. A plot of concentration versus fluorometer-dial reading will give the shape of the calibration relation for the particular fluorometer. Experience to date indicates that nonlinear response occurs more frequently and to a greater degree at the low end of the curve; special effort should be made, therefore, to define that part of the curve.

If a fluorometer is found to have a nonlinear calibration relation, the instrument should be returned to the manufacturer for replacement of parts or adjustment. Before an instrument is returned, one should be certain that the relation is truly nonlinear and not the result of an error in calibration. Inadequate instrument warmup is frequently the cause of a nonlinear calibration. If the instrument has to be used and the calibration is nonlinear, the derived nonlinear calibration curve should be used.

It is recommended that, if possible, dial readings of less than 10 be avoided, because the normal instrument error inherent in fluorometer operation can result in relatively large percentage errors for low dial readings. Low dial readings can be avoided by use of the proper fluorometer aperture. For example, the Turner fluorometer has four apertures, each designed to allow the passage of a different amount of light (fig. 120). The amount of light passed at each scale is approximately proportional to scale number; that is, the 30× scale will pass about 30 times as much light as the 1× scale. The relation between scales, for any fluorometer used, should be determined by testing each of several samples of different dye concentrations on more than one scale. A plot of dial readings for one scale against corresponding dial readings on another scale will give the desired relation. It should be mentioned here that low dial readings are unavoidable when testing the background (natural) fluorescence of river water. (See p. 231.)

PREPARATION OF STANDARD DYE SOLUTIONS FOR FLUOROMETER CALIBRATION

In calibrating the fluorometer it is necessary to first prepare samples of dye solution of known relative concentration, then test the samples in the fluorometer, and finally relate the readings on the fluorometer dial to the known relative concentrations. This matter of relative concentrations should be explained at this point. The accurate determination of stream discharge depends on the measurement of concentrations relative to each other; the absolute values of the concentrations are of no importance as long as the values of the concentrations are all determined in the same manner and all bear the same ratio to the absolute or true concentrations. Where the term "concentration" is used in this manual, it refers to "relative concentration"; the two terms are used here interchangeably. The calibration characteristics of the fluorometer were discussed in the preceding section of this manual. This section explains the preparation of standard dye solutions of various concentrations for use in the calibration process.

Standard dye solutions are prepared by diluting, with known amounts of water, the dye solution furnished by the manufacturer of the dye. The relative concentration of the furnished solution is the value given by the manufacturer. For example, if the manufacturer's

20-percent dye solution is used, its relative concentration is 200,000,000 $\mu g/l$. If the relative concentration of that solution differs from the true or absolute concentration by some small percentage, all diluted samples that are prepared from that batch of dye solution will have relative concentrations that differ from their absolute values by that same small percentage. Chlorinated tapwater should not be used in the dilution, because the chlorine present in the water quenches the fluorescence of rhodamine dyes.

The equipment and materials needed to make the dilutions are listed below. Where glass is specified for the equipment, other nonabsorptive materials may also be used.

1. Volumetric flasks (glass) of 100, 250, 500, 1,000, and 2,000 mL capacity.
2. Volumetric pipettes (glass) in an assortment of sizes.
3. Beakers (glass), or 1-gal (3.75 mL) glass jars.
4. Wash bottles (polyethylene).
5. Sample bottles (glass).
6. Masking tape and pen.
7. Distilled water. River or chlorinated tap water should not be used; however, tap water exposed to the air for 24 hr will lose its chlorine.

The dilution process is performed in steps, as illustrated in the example shown below. The relative concentration at each step is computed from the equation

$$C_n = \frac{C_i V_i}{V_a + V_i} , \tag{34}$$

where

C_n is the new concentration,
C_i is the initial concentration,
V_i is the volume of initial concentration, and
V_a is the added volume of water.

A flow chart similar to that shown in the example below is recommended when making the dilute solutions.

Example.—The original dye solution, as obtained from the manufacturer, is a 20-percent solution of Rhodamine WT. In making the solutions of known relative concentration, the process will be checked by duplicating the procedure for each of two samples. In other words, two samples of equal size of the original solution, are diluted in precisely the same manner. The resulting two dilute solutions are each diluted further in the same manner, and so on. As a result we will have two independent sets of standard solutions for use in calibrating the fluorometer.

To get back to the dilution process, 20 mL of the original 20-percent solution is diluted to 2,000 mL with pure water. The concentration of the resulting solution is computed by use of equation 34. The process is repeated twice, each time using the solution last obtained, as indicated in the following flow chart:

$$C_n = \frac{\begin{array}{c}20 \text{ mL of } (200\times10^6)\\ \mu\text{g/L dye solution}\end{array}}{2{,}000 \text{ mL total solution}} = \frac{1}{100}\,(200\times10^6) = 2\times10^6\mu\text{g/L}$$

$$\downarrow$$

$$\frac{\begin{array}{c}20 \text{ mL of } (2\times10^6)\\ \mu\text{g/L solution}\end{array}}{2{,}000 \text{ mL total solution}} = \frac{1}{100}(2\times10^6) = 20{,}000\ \mu\text{g/L}$$

$$\downarrow$$

$$\frac{\begin{array}{c}20 \text{ mL of } 20{,}000\\ \mu\text{g/L solution}\end{array}}{2{,}000 \text{ mL total solution}} = \frac{1}{100}(20{,}000) = 200\ \mu\text{g/L}$$

The solution obtained, with concentration equal to 200 μg/L, is referred to in this manual as a working standard solution because any lesser concentration that may be desired can be prepared from it by one additional serial dilution. For example, a dilution ratio of 1/200 will provide a solution with a concentration as low as 1 μg/L. No dilution ratios of less than 1/200 should ever be used. The working standard solution should be stored for future use. Experience has indicated that standard solutions of 200 or more μg/L when properly stored in hard glass dark bottles out of direct light will keep for periods of at least 6 months. There is evidence, though not conclusive, to indicate that very weak standard solutions tend to deteriorate or to adhere slightly to the bottles in which they are stored.

The stored working standard solution may be used for periodically checking the fluorometer calibration to determine if changes in the calibration have occurred, and enough of the standard solution should be retained for future use if the same dye lot from the manufacturer is to be used for discharge measurements. New sets of standard solutions should be prepared after 6 months to avoid the risk of decay of fluorescence in the stored standard solutions. In addition, new sets should also be prepared when new batches of dye are to be used for discharge measurements because the dyes received from the manufacturer may vary from batch to batch.

If the fluorometer calibration is known to be linear, from previous tests using standard solutions made with distilled or unchlorinated tap water, only a limited few concentrations need be prepared and tested for the discharge-measurement calibration of the fluorometer. The concentrations prepared should approximate the plateau concentration expected in stream samples that will be taken during the discharge measurement.

OPERATION OF THE FLUOROMETER

The U.S. Geological Survey uses the Turner model 111 fluorometer. The fluorometer is operated in accordance with the following instructions.

Allow a warmup period of 1½ to 2 hr before any tests are made. This warmup period will permit the instrument temperature to stabilize. If possible, have all samples, including standard and background solutions, at the same temperature. Establish the zero point on the fluorometer by inserting the dummy cuvette in the cuvette holder, after which the dial is adjusted to zero with the blank knob. To test a standard solution, fill the cuvette with that solution, but before putting the cuvette into the fluorometer, wipe the outside of the cuvette dry with laboratory-grade paper tissue. That prevents distortion from droplets on the glass or contamination of the fluorometer, and possible erroneous readings when testing other samples. After wiping, insert the cuvette in the holder and close the fluorometer door. After the fluorometer dial reaches a stable reading, record the reading. As mentioned earlier, test each pair of several standard solutions of different concentrations on more than one fluorometer scale for calibration of the instrument. An alternate, and perhaps more desirable, course of action is to first test all field samples on one fluorometer scale. If that scale accommodates all field samples, calibrate only that one scale in the range found pertinent.

If a water sample from a discharge measurement is being tested, first rinse the cuvette with tap water and then, after shaking out excess droplets, rinse the cuvette with the sample being tested. The cuvette may then be filled. It is important that any sediment in the sample be allowed to settle and that any oxygen bubbles be removed. (Cold samples taken from streams with a high dissolved-oxygen content will often have bubbles form on the sides of the cuvette.) If the samples have been allowed to sit in a temperature bath overnight, the sediment will have settled and the oxygen bubbles will usually have been released. If the oxygen bubbles have not been eliminated by allowing the samples to warm up before testing, they can usually be removed by tapping the cuvette before testing.

The blank and background samples should be tested at intervals during the fluorometer operation as a check against possible instrument malfunction or change in instrument calibration. Background samples are water samples used to define the natural fluorescence of the stream. Natural fluorescence should be determined for any dye-dilution measurement. Always correct water-sample concentrations for this background effect.

The frequency of testing known relative concentrations will depend primarily on the number of stream samples. Known relative concentrations are ordinarily tested before and after the stream samples, but more frequent testing is desirable where numerous samples are involved

Sample bottles made of polyethylene, glass, or similar nonabsorptive material can be used repeatedly for sampling. After each measurement all sample bottles should be thoroughly rinsed and drained. The bottles, if stored, should be well separated from any dye-contaminated equipment.

Whatever the equipment used, care should be taken at every step in the discharge determination to prevent contamination of stream samples, standard solutions, and the equipment used for measurement and analysis. A sample can be contaminated, from the standpoint of concentration, by the addition of pure water as well as by the addition of minute quantities of dye from the analyst's hands or other source.

DYE-INJECTION APPARATUS

Apparatus for the injection of dye solution at a constant rate is of three types: mariotte vessel, floating siphon, and pressure tank.

MARIOTTE VESSEL

The features of the mariotte vessel, shown in figure 121 are (1) an

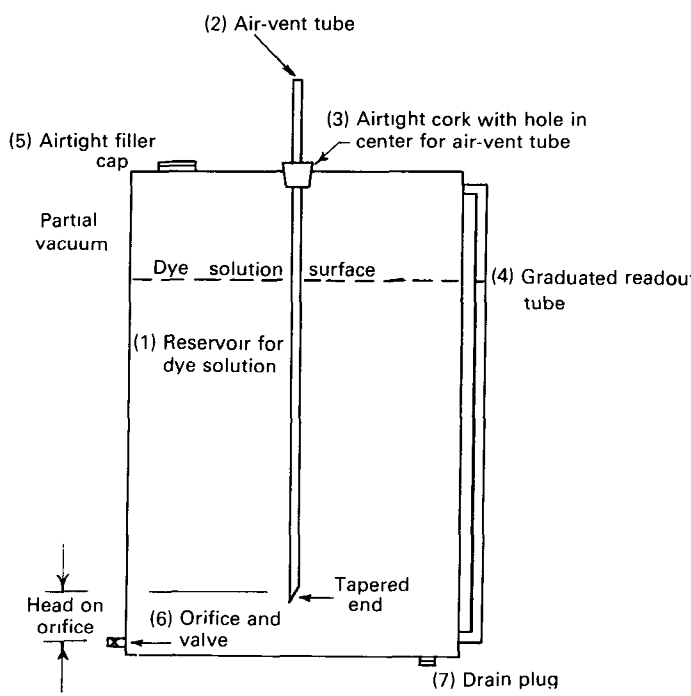


FIGURE 121.—Mariotte vessel (constant-head device).

airtight tank that holds the dye solution, (2) an air-vent pipe inserted into the tank through (3) a rubber stopper that will form an airtight seal both on the tank and the vent pipe, (4) a plastic or glass readout tube for determining the level of the liquid in the tank, (5) a filling plug that must be airtight, (6) a discharge orifice and valve, and (7) a flushing orifice and valve.

The lower tip of the air-vent pipe should be tapered to prevent the tip from being placed directly on the bottom of the tank and thereby obstructing the air flow. Discharge at more than one point may be obtained by installing the desired number of orifices in the tank.

The operation of the vessel is as follows. When the discharge valve (6) is opened, the level of the dye solution in the tank (1) will drop, creating a partial vacuum above the liquid (the top of the tank must be airtight). As the level of the liquid continues to drop, the vacuum increases causing the level of liquid in the air vent (2), which is open to the atmosphere, to drop until it reaches the bottom of the vent. As the solution continues to be discharged, air will enter the tank through the vent causing an equilibrium between the partial vacuum formed above the liquid surface and the weight of the liquid above the bottom of the air vent. When this equilibrium has been reached, a constant discharge will have been attained and will continue until the liquid in the tank drops to the bottom of the air vent. The acting head is the different in elevation between the bottom of the air vent and the orifice.

FLOATING SIPHON

Constant injection of a dye solution is possible through the use of the floating siphon device shown in figure 122. The device is designed to discharge at a constant rate by maintaining a fixed head on the orifice. The features of the floating siphon are (1) a tank that holds the solution, (2) a float with siphon tube and guide assembly, and (3) priming valves that can be used in activating the siphon. The discharge tube with orifice is permanently attached to the float in the position shown. Note that regardless of the position of the float within the reservoir, the operating head on the orifice remains the same. Stable, uniform movement of the float and siphon assembly is possible through the use of guides and a balance counterweight. When the siphon is effective, the float and assembly will drop with the water surface but at the same time maintain a constant discharge because of the steady head operating on the orifice.

The mariotte vessel and the floating siphon can both be equipped with readout gages that are used to calibrate the reservoirs volumetrically. These gages can be read at selected time intervals during the injection period to determine the constant-flow rate. The flow rate can also be determined by volumetrically measuring the discharge from the orifice for a given period of time.

PRESSURE TANK

A constant-rate injection of dye solution may be obtained by use of a pressure tank in combination with a flow regulator (fig. 123). The flow regulator maintains a constant pressure differential across a valve in the regulator, thus giving a constant flow rate at any setting of the regulator. A pressure differential of a few pounds per square inch must be maintained for the flow regulator to function properly.

The solution to be injected is poured into the pressure tank, and the tank is sealed. Air is then pumped into the tank, providing the necessary pressure for the regulator. The solution is forced from the tank, by the pressure, through the regulator and to the injection lines.

Most commercially available flow regulators have a purge meter

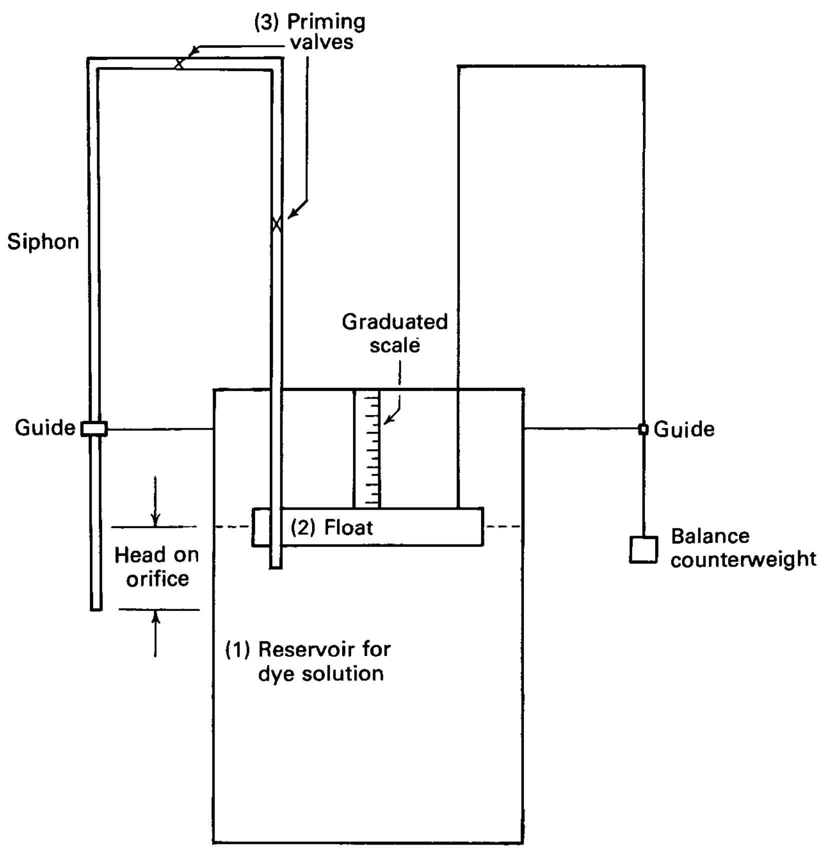


FIGURE 122.—Floating siphon (constant-head device).

attached to them. These meters may be used to obtain the approximate desired flow rate. The flow rate used in the computations should be determined from a volumetric measurement of the tank discharge.

Pressure tank and flow regulator systems are commercially available. These systems are as light as 8 lb (3.6 kg), when empty, for an operating-pressure tank capacity of about 2 gal (7.5 L). Flow regulators may be obtained for flow rates between 0.012 and 4 gal/hr or more. Flow rates once established will be maintained within ±2 percent.

DETERMINATION OF QUANTITIES OF FLUORESCENT DYE FOR MEASURING DISCHARGE

QUANTITY OF DYE NEEDED FOR MEASUREMENT BY THE CONSTANT-RATE-INJECTION METHOD

The volume of dye required to make a discharge measurement

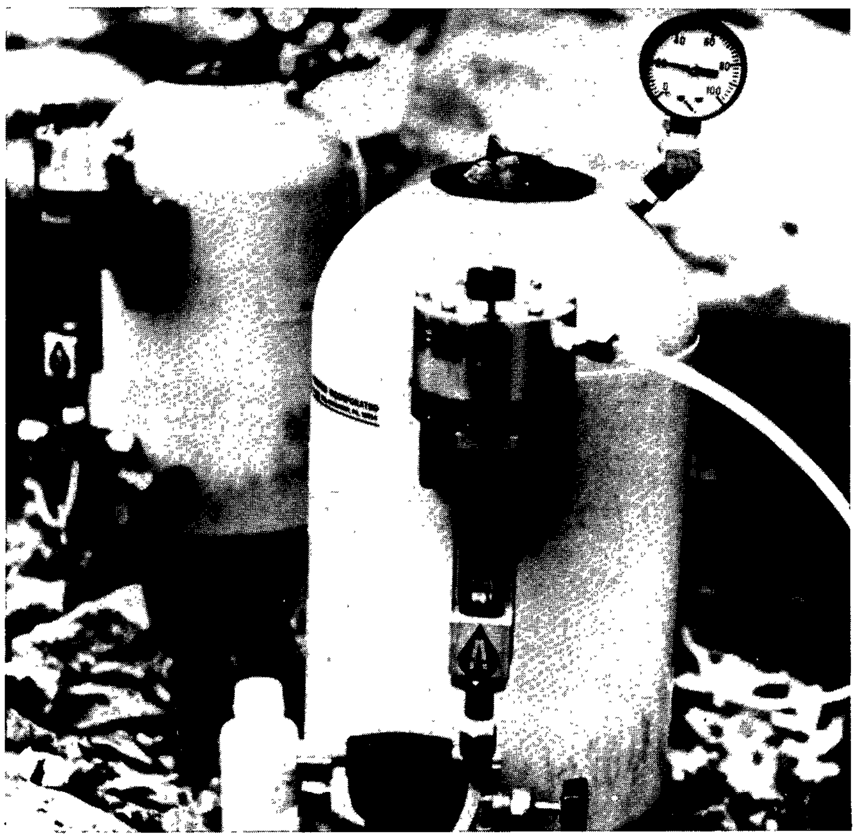


FIGURE 123.—Pressurized constant-rate injection tanks for injection of dye into streams.

using the constant-rate-injection method will depend on the discharge of the stream and on the total injection time. That volume can be estimated using the equation

$$V_d = 1.02 \times 10^8 \left(\frac{C_2}{C_d}\right) Q\, t_t, \tag{35}$$

where

V_d is the volume, in milliliters, of dye solution to be added to the water in the injection tank,

C_d is the concentration, in micrograms per liter, of the dye solution added to the water in the injection tank. If a 20 percent dye solution is added, the concentration of C_d is considered to be 200,000,000 μg/L. (See p. 228–229 for a discussion of relative and absolute concentration values.)

C_2 is the plateau concentration, in micrograms per liter, desired at the sampling site,

Q is the estimated stream discharge, in cubic feet per second, and

t_t is the total injection time, in hours.

The value of 1.02×10^8 is a combination of conversion factors. A concentration value for C_2 of 5 μg/L is recommended for Rhodamine WT.

The volume of tracer solution injected, which consists of the volume dye solution (V_d) plus added water, and the rate of injection are mutually dependent. Their limiting values are based on the size and characteristics of the dye-injection apparatus that is used. The capacity or volume of the apparatus divided by t_t gives the upper limiting value of the rate of injection, q. Any paired values of q and volume (qt_t) that are within the capability of the apparatus are satisfactory. However, it is recommended that no value of q less than 2×10^{-5} ft^3/s (0.57 mL/s) be used, because low injection rates are difficult to measure accurately. It is further recommended that V_d be diluted sufficiently to give the injection solution a concentration of no more than 25×10^6 μg/L, because more highly concentrated solutions tend to clog the injection apparatus.

QUANTITY OF DYE NEEDED FOR MEASUREMENT BY THE SUDDEN-INJECTION METHOD

If discharge is to be measured by the sudden-injection method, it is recommended that common salt be used as the tracer, rather than dye. (See p. 250–256.) However, as a matter of general interest, an equation is given here for computing the quantity of dye required for a discharge measurement of that type.

The volume of dye required to make a discharge measurement by the sudden-injection method can be estimated using the following empirical equation:

$$V_d' = 3\times10^7 \left(\frac{C_p}{C_d'}\right) Qt_p, \tag{36}$$

where

- V'_d is the volume, in milliliters, of dye solution of concentration C'_d to be introduced into the stream,
- C_p is the peak dye concentration, in micrograms per liter desired at the sampling site,
- Q is the estimated stream discharge, in cubic feet per second, and
- t_p is the estimated time, in hours, for the dye peak to travel from the injection site to the sampling site.

The value of 3×10^7 is a combination of empirical equation constants and conversion factors. A concentration value for C_p of 10 μg/L is recommended for Rhodamine WT.

PROCEDURES FOR MEASURING DISCHARGE BY THE DYE-DILUTION METHOD

The only dye-dilution method that will be discussed here is the constant-rate-injection method. The sudden-injection method is not considered here because not only is it usually a less accurate method, but it is extremely laborious when dye is used and it can be applied with less effort by using common salt as a tracer. As mentioned earlier, the sudden-injection method requires that the entire concentration-time curve be defined, and for that purpose it is desirable to sample the stream concentration at intervals of less than a minute during periods of rapidly changing concentration. If the dye-dilution technique is used, it means that within a minute or so, grab samples must be obtained at all three sampling points in the sampling cross section. Furthermore, because it is recommended that at least 20 samples be taken at each sampling point to define the concentration-time curve, it means that a minimum of 60 stream samples must be analyzed by fluorometer, along with calibration and background samples. In short, it is not practical to use dye as a tracer in the sudden-injection method. Although recording fluorometers with flow-through doors are available, they seldom provide the precision required for a discharge measurement. When salt is used as the tracer, conductivity readings can be obtained at a point in a matter of a few seconds, by using a temperature-compensated probe and portable meter. (See p. 252–255.)

FIELD PROCEDURES

The following field procedures are recommended for determining stream discharge by the dye-dilution method:

1. Select and calibrate the measurement reach (p. 220–222).

2. Select exact locations for the injection and sampling sites. Data obtained from the calibration tests are used in making the site selections, and consideration should be given to accessibility and convenience for conducting field operations. Bridges are often excellent for the purpose. The injection site for either single- or multiple-point injection should be one where the injection system can be readily observed and, if necessary, serviced. Select sampling sites where sampling is possible anywhere in the cross section. Locate a minimum of three sampling points in the cross section so that the section can be checked for lateral mixing.

The best location for a single-point injection is normally at the center of the cross section. A center injection generally produces the minimum possible mixing length if the dye is to be introduced at only one point. Space the injection points for a multiple-point injection so that each dye injection diffuses equal lateral distances.

3. Prepare the sample bottles. Show the following information on each bottle:

a. The sampling site (may be abbreviated).
b. Location of the sampling point in the cross section.
c. Sample number (if desired).
d. Sample time (use military time, that is, 24-hr clock system).
e. Date.

Make these notes with a ballpoint pen using masking tape for labels on the bottles. Note the time (sample time) when the sample is obtained.

4. Determine the amount of dye needed for injection from equation 35. The total injection time used in equation 35 includes the time required for the concentration plateau to have been reached at the sampling section and sufficient time to permit at least 15 min of sampling of the plateau.

5. Prepare the dye solution and injection equipment. Make certain that the solution is thoroughly mixed. Exercise caution in transporting the dye and do not carry excessive quantities of it on field trips. Dye in solution can be easily removed from containers by the use of pipettes. Adverse weather conditions, particularly high wind or rain, create serious problems in handling the dyes. Prepackaging of specific quantities of dye will facilitate operations in the field. River water taken at the site may be used in preparing the injection solution, but do not obtain river water immediately below a sewage outfall or at sites where chlorine is injected into the river system.

6. Obtain several background samples upstream from the injec-

tion site. Do this before handling the dye.

7. Obtain three samples of injection solution. These samples, each of about 2 oz (60 mL), may be taken immediately before or after the solution is injected into the stream.

8. Inject the diluted dye solution. Determine the actual rate of dye injection by timed observations of volume depletion in the injection tank. If a mariotte vessel is used for injection, check the fittings for air leakage before and during injection. Discharge will become constant within a few seconds after the orifice is opened if the tank is airtight. The sound of air bubbling from the air vent through the solution can be heard when the vessel is operating properly.

9. Do not inject the highly concentrated dye solution near the streambed; otherwise, the bed may be stained by the dye and excessive dye loss will result.

10. The time of arrival of the concentration plateau can be estimated from data obtained when the reach was calibrated. It is usually desirable to start sampling a little before the estimated time of arrival of the concentration plateau. Define the plateau with at least five samples at each of the three sampling points in the sampling cross section. If a fluorometer is available, the arrival and passage of the dye can be determined by using the flow-through door and recorder accessory, or by periodically testing individual samples using the standard door. Do not mistake a possible temporary leveling of the concentration-time curve as being the actual plateau. A short time should elapse between the taking of samples to minimize the effect of nonrepresentative surges of high or low concentration.

11. In obtaining stream samples at the sampling site, take samples just below the water surface and far enough from the banks to avoid areas of slack water. If possible, the sampling site should be at a contracted section of the stream or at one where velocities are fairly uniform across the entire section. The following equipment is needed for grab sampling:

a. Sampling device and line (for sampling from a structure).

b. Boots or waders (for sampling by wading).

c. Boat, motor, paddle, gasoline can, life jacket (for sampling by boat).

d. Bottles (glass or plastic, 4-oz, wide-mouth bottles are recommended).

e. Masking tape (for labeling samples).

f. Ballpoint pens (for making identifying notations on sample labels).

g. Watch.

12. Contamination of stream samples is always a possibility. When handling any dye, keep hands and clothes clean as a safeguard

against accidental contamination. Rinse bottles at least three times in stream water immediately before sampling. This precaution is necessary because bottles are used repeatedly and some may have had high concentrations of dye in them previously. Dye from stained hands can contaminate samples, resulting in erroneous concentration determinations; therefore, keep hands away from the uncapped mouths of bottles and out of the sample water.

13. Clean the injection and mixing equipment.

14. Store and carry sample bottles in an upright position to prevent leakage and to allow sediment to settle.

ANALYSIS AND COMPUTATIONS

Fluorometer analysis should not begin until the standard concentration solutions have been prepared. Place all samples, including background samples and standard concentration solutions, in a constant-temperature bath. By keeping the samples in the bath overnight, any sediment present will have an opportunity to settle, and any oxygen bubbles present will usually be released. After fluorometer warmup, all samples may be analyzed. Use of a constant-temperature bath will eliminate the need for temperature corrections. However, if for any reason a temperature bath was not used, the first step in the analysis procedure that follows is to make temperature corrections.

1. Correct each fluorometer reading for the effect of temperature by the following method. A standard cuvette-sample temperature is selected. This may be the mean, median, mode, or any other convenient cuvette-sample temperature recorded while testing the samples. The difference between each recorded cuvette-sample temperature and the standard temperature is used to select the appropriate coefficient from table 13. Temperature-correction coefficients are usually prorated with respect to time between temperature observations. Multiply the fluorometer dial-readings by the coefficients to obtain adjusted dial readings.

2. Next correct the dial readings, or temperature-adjusted dial readings if such adjustment had been necessary, for the effect of background fluorescence, as determined from the background samples. This background correction is applied to each water sample and to each sample of standard concentration.

Usually all fluorometer testing will be done using a single fluorometer aperture. If more than one aperature is used, obtain a background reading for each aperature and use a mean background value for correcting the dial readings. Do not use a mean background value if a definite change in background has been noted. A change in background fluorescence may result from a variety of causes such as unstable fluorometer operation, increased suspended-sediment load, and the presence of sewage or industrial wastes. Enter the background readings in the notes. Subtract the background readings from the adjusted dial readings from step 1 above to obtain the final corrected dial readings.

3. Use the final corrected dial readings for the standard solution of known relative concentration to compute the fluorometer coefficient, a, in equation 33 (p. 227). When this standard solution is tested several times during a period of sample testing, the value of the coefficient is normally prorated, with respect to time, between testings.

4. Determine the relative concentrations of samples by multiplying the corrected dial readings, from step 2 above, by the coefficient, a.

5. Compute the stream discharge by use of equation 24. Because the background concentrations have already been subtracted from the stream sample concentrations, equation 24 becomes

$$Q = \left[\frac{C_1}{C_2} - 1 \right] q,$$

which can be simplified, without significant loss of precision, to

$$Q = \left(\frac{C_1}{C_2} \right) q. \tag{37}$$

An average value of C_2 is used in the computations. If the individual C_2 values show little variation among them, use the arithmetic mean. If the variation is significant, obtain the mean value of C_2 by weighting the mean plateau concentrations for each sampling point by the percentage of discharge associated with the sampling point. If the discharge distribution is unknown, but the area distribution is known, obtain the mean value of C_2 by weighting the plateau concentrations for each sampling point by the percentage of area associated with each sampling point. If both the discharge and area distributions are unknown, as will usually be the case, compute the mean value of C_2 by using percentage of width as the weighting factor.

6. Determine the percentage of mixing by using equation 26, or by use of the graphical procedure described on page 219–220. Reliable results are generally obtained when the percentage of mixing is 95 or more. Reliable results are also possible for flow conditions where the percentage of mixing is less than 95 if the discharge distribution at the sampling section is uniform or known.

SAMPLE COMPUTATION—CONSTANT-RATE-INJECTION METHOD

A sample computation of the constant-rate-injection method is presented here based on data obtained on March 25, 1965, at Little Seneca Creek near Boyds, Md., in the course of a detailed investigation of dye-dilution methods of measuring discharge. The average width of Little Seneca Creek at the time of the measurement was 30–35 ft (9–10 m). Depths averaged between 1 and 2 ft (0.3 and 0.6 m). The streambed was composed mostly of gravel, with some rock outcrops. Flow in the measuring reach passed through several bends.

Calibration of the measurement reach at about the stage to be measured had indicated that a measuring reach of 1,800 ft (550 m) was satisfactory for adequate mixing when the dye was injected at three points across the stream. The calibration further indicated that the time required for the concentration plateau to arrive at the sampling site was about 45 min. Three sampling points, designated A, B, and C, respectively, were to be used at the sampling site. Grab-samples were taken immediately before and during the passage of the concentration plateau.

1. The required volume of dye (V_d), to be supplied from a mariotte vessel, was computed from equation 35,

$$V_d = (1.02\times10^8)\ \frac{C_2}{C_d}\, Qt_t\ .$$

The approximate value of the plateau concentration (C_2) desired at the sampling site was 5 μg/L. The estimated value of stream discharge (Q) was 50 ft^3/s. The total injection time (t_t) was estimated to be 80 min (1.33 hr)—45 min for traveltime and 35 min to insure a concentration plateau of adequate duration. The dye to be used was Rhodamine WT, 20-percent solution, and C_d was therefore equal to 2×10^8 μg/L. Consequently,

$$V_d = (1.02\times10^8)\ \left(\frac{5}{2\times10^8}\right) (50)\ (1.33) = 170 \text{ mL}.$$

2. The discharge rate of diluted dye solution (q) from the injection vessel that was selected was 1.50×10^{-4} ft^3/s. The volume of solution needed in the injection tank was equal to qt_t, where t_t was 1.33 hr, or 4,800 s. That volume was therefore

$$(1.50\times10^{-4})\ (4800) = 0.72 \text{ ft}^3, \text{ or approximately } 20{,}000 \text{ mL}.$$

The 170 mL of dye solution was mixed with sufficient stream water to give an injection solution of 20,000 mL.

3. The concentration of the solution in the injection tank (C_u) was computed from equation 34,

$$C_n = \frac{C_t V_t}{V_a + V_i}$$

$$= \frac{(200{,}000{,}000)\ (170)}{20{,}000}$$

$$= 1{,}700{,}000\ \mu g/L.$$

4. Timed observations of the change in the level of the dye solution in the injection tank showed the actual rate of outflow to be 1.51×10^{-4} ft^3/s.

The steps that follow were taken in the office to complete the measurement.

1. A working standard solution of concentration 200 μg/L had previously been prepared (p. 229–230). A dilution ratio of 1/40 was applied to that solution to obtain two standard sample solutions whose concentration was 5.00 μg/L.

Standard solutions of 5.00 μg/L were used for calibration of the fluorometer because that was the expected value of the plateau concentration of the stream. (The concentration of the standard solutions should approximate the concentrations of the stream samples that will be used to compute stream discharge.)

2. A form for keeping notes was prepared as shown in figure 124. Column 1 of figure 124 shows the time when the sample was obtained in the field, and column 2 shows the time when the sample was tested in the fluorometer.

3. The samples had not been placed in a constant-temperature bath, and consequently temperature corrections were necessary. The samples, including those for background testing (stream water untouched by dye) and for fluorometer calibration (5.00 μg/L), were tested in the fluorometer. Columns 3 and 4 of figure 124 show dial readings on the fluorometer aperture that was used. Column 5 shows two temperature readings. The upper reading is the temperature of the water being circulated through the fluorometer door. The lower reading is the temperature of the fluorometer compartment. Obtain that temperature from a thermometer that can be taped to the compartment side at the left of the secondary filter. Column 6 shows the temperature of the cuvette sample. That temperature is taken immediately after the fluorometer dial is read by removing part of the water from the cuvette, inserting the thermometer in the cuvette, and replacing the cuvette in the door. Allow the thermometer to stabilize before reading. Compute temperature-correction coefficients from the observed cuvette-sample temperatures.

4. A standard cuvette sample temperature of 82°F (27.8°C) was selected for use. Temperature coefficients from table 13, correspond-

ing to the differences between 82°F and the observed cuvette-sample temperatures in column 6 of figure 124, were then obtained and recorded in column 7.

Little Seneca Creek near Boyds, Maryland. *March 25, 1965*

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
Time		Dial reading		Temperature		Temp coef.	Adj. dial reading	Background reading	corrected dial reading	Q = C/D	Concentration (ppm)	Remarks
Sample	Fluor. test	30X	10X	circ./comp.	cuvette							
					Sample Point A							
Blank	1459	0										
1a	1500	65.8				1.02	67.1	1.2	65.9	.0759	5.00	*
1b	1501	67.7		79.5/93.5	83	1.02	69.1	1.2	67.9	.0736	5.00	*
										.0748		Mean
1250	1503	1.2				1.02	1.2	1.2	—	—	—	**
1313	1505	64.4				1.02	65.7	1.2	64.5	.0748	4.82	
1320	05	72.0				1.02	73.4	1.2	72.2	.0748	5.40	
1327	06	74.5				1.02	76.0	1.2	74.8	.0748	5.60	***
1335	07	75.8				1.02	77.3	1.2	76.1	.0748	5.69	***
1343	08	75.2				1.02	76.7	1.2	75.5	.0748	5.65	***
1350	08	75.2				1.01	76.0	1.2	74.8	.0748	5.60	***
1356	1509	74.6				1.01	75.3	1.2	74.1	.0748	5.54	***
					Sample Point B						5.62	Mean
Blank	1523	0										
1251	24	1.3				1.00	1.3	1.2	—	—	—	**
1322	27	71.0		80.0/93.5	81.5	1.00	71.0	1.2	69.8	.0747	5.21	
1328	27	74.6				1.00	74.6	1.2	73.4	0747	5.48	***
1336	29	75.8				1.00	75.8	1.2	74.6	.0747	5.57	***
1344	30	76.2				1.00	76.2	1.2	75.0	.0747	5.60	***
1351	30	75.1				1.00	75.1	1.2	73.9	.0747	5.52	***
1357	1531	72.8				1.00	72.8	1.2	71.6	0747	5.35	***
											5.50	Mean
					Sample Point C							
Blank	1546	0										
1252	1546	1.2				1.00	1.2	1.2	—	—	—	**
1315	48	65.8		80.0/94.0	82	1.00	65.8	1.2	64.6	.0746	4.82	
1321	50	73.3				1.00	73.3	1.2	72.1	.0746	5.38	***
1329	51	72.9				1.00	72.9	1.2	71.7	.0746	5.35	***
1337	52	72.0				1.00	72.0	1.2	70.8	.0746	5.28	***
1345	52	72.9				1.00	72.9	1.2	71.7	.0746	5.35	***
1352	53	72.0				1.00	72.0	1.2	70.8	.0746	5.28	***
											5.33	Mean
1a	1608	67.7				1.00	67.7	1.2	66.5	.0752	5.00	*
1b	1609	68.9				1.00	68.9	1.2	67.7	.0738	5.00	*
										.0745		Mean

Turner III Fluorometer Filters (546 mμ, 590 mμ), Far ultraviolet lamp. Temperature-stabilized door. Standard cuvette-sample temperature, 82°F. Tested 3-29-65
** Standard solution of known relative concentration. ** Background sample.*
**** Sample used to compute C_2.*

FIGURE 124.—Sample analysis sheet used for computing discharge by the constant-rate-injection method of dye dilution.

5. The adjusted dial readings in column 8 were obtained by multiplying the values in either column 3 or column 4 by the temperature coefficients in column 7.

6. The average temperature-adjusted fluorometer reading for background fluorescence, obtained by testing the unaffected stream water, was recorded in column 9. Its value, 1.2 in this test, was then subtracted from all temperature-adjusted readings in column 8, and the corrected dial readings were recorded in column 10.

7. The fluorometer coefficient, a, in column 11, was obtained as the average of the ratios of the known concentration of the two background samples (5.00 μg/L) to their corrected dial readings. The background samples were tested at the beginning and end of the fluorometer analysis, and the fluorometer coefficients obtained were prorated with time for use in the analysis.

8. The concentrations of the samples listed in column 12 were obtained by multiplying the fluorometer coefficients in column 11 by the corrected dial readings in column 10.

9. The concentrations obtained for each sampling point are normally plotted against sampling time to determine the average concentration of the plateau C_2 at each of the three sampling points, A, B, and C. In this example, no plots were made; instead, C_2 for each sampling point was computed, in column 12 of figure 124, as the mean of the last five values obtained, it being evident that those values were plateau values.

10. Because the three sampling points, A, B, and C, were evenly spaced across the stream, each of the three values of C_2 from step 9 were given equal weight in computing the mean value of C_2 for the entire sampling section:

$$\text{mean } C_2 = \frac{5.62+5.50+5.33}{3} = 5.48 \ \mu\text{g/L}.$$

11. The discharge was computed from the simplified form of equation 24 that is shown as equation 37 on page 241.

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

$$= \left(\frac{170\times 10^4}{5.48}\right)(1.51\times 10^{-4}) = 46.8 \text{ ft}^3/\text{s}.$$

12. Mixing percentage (P_m) was computed, using appropriate widths in equation 26:

$$P_m = 100 - \frac{[|X_A-X_m|W_A+|X_B-X_m|W_B+|X_C-X_m|W_C]\times 50}{(W_{total})(X_m)}$$

Because the sampling points were evenly spaced, W_A, W_B, and W_C may each be replaced by 1, and W_{total} may be replaced by 3. Then

$$P_m = 100 - \frac{[|5.62-5.48| + |5.50-5.48| + |5.33-5.48|] \times 50}{3(5.48)}$$

$$= 100 - \frac{(0.31)\times 50}{16.44}$$

$$= 99 \text{ percent (satisfactory)}$$

In the original study of Little Seneca Creek, the discharge had been measured by current meter for a comparison with the discharge obtained by the dye-dilution method. The current-meter discharge was 47.3 ft^3/s, whereas the dye-dilution discharge was 46.8 ft^3/s. The two values differ by 1 percent.

SIMPLIFIED PROCEDURES FOR MAKING NUMEROUS DYE-DILUTION MEASUREMENTS OF DISCHARGE

The preceding description of the method of measuring discharge by the constant-rate-injection method was highly detailed to give complete understanding of the procedure. It should be apparent that there are many details to be considered in collecting and analyzing the samples. If only an occasional dye-dilution measurement is to be made, the procedures described above should be followed. If numerous dye-dilution measurements are to be made on a routine basis, "mass production" methods may be instituted to simplify and speed the procedure.

The first simplifying step is the laboratory production of two batches of solution, each of which represents a working standard concentration. Bulk production of the two concentrations will considerably reduce the amount of time that would otherwise be required later for serial dilutions and for calibrating the fluorometer. The standard concentration that is needed for fluorometer calibration is one that closely approximates the C_2 (plateau) concentrations that will be obtained in the discharge measurements. The value of C_2 will usually approximate 5 μg/L. Consequently one working standard solution of 200μg/L should be produced in adequate quantity from the stock of 20-percent Rhodamine WT dye. The concentration of 200 μg/L is high enough to permit storage of the solution for at least 6 months with little danger of deterioration and is still low enough to produce, with a single dilution, concentrations that approximate any C_2 values that may be obtained in a discharge measurement.

The other solution of working standard concentration that should be produced in bulk is one that can be used as the injection solution in a discharge measurement without further dilution. By way of background explanation we examine figure 125, the basis of which is equation 37. Figure 125 shows the relation among Q, q, and C_1 for a constant value of C_2. The constant value of C_2 that is used is 5 μg/L, which is the optimum value of the plateau concentration in the constant-rate-injection method. Concentration values of C_1 ranging from 0.05 percent (0.5×10^6 μg/L) to 2.5 percent (25×10^6 μg/L) are shown in the diagonal lines of figure 125. To produce a C_1 solution of, say 0.4 percent, 8 L of water is added to the parenthetical quantity of 137.2 mL of 20-percent Rhodamine WT dye. Examination of figure 125 shows that with a given injection concentration (C_1) a wide range of discharges may be measured by merely changing the injection rate (q). For example, a C_1 concentration of 0.4 percent (4×10^6 μg/L) permits the measurement of stream discharges from 15 ft^3/s to almost 300 ft^3/s, while still creating a plateau concentration (C_2) of 5 μg/L. Even those discharge limits may be exceeded by allowing C_2 to vary from its value of 5 μg/L. By using figure 125 with a knowledge of the range of discharges to be measured, a suitable value of C_1 can be selected for use as a working standard concentration.

To recapitulate what has been said up to this point, two working standard concentrations should be used for laboratory production of bulk solutions.

1. A concentration (C_1) of 4×10^6 μg/L, or one more suitable for the range of discharges to be measured, should be used for one solution. That solution will be used, without further dilution, as the injection solution for discharge measurements.

2. A concentration of 200 μg/L should be used for the second solution. A single dilution of that solution will provide standard solutions for calibrating the fluorometer at concentration C_2 (usually about 5 μg/L). Both solutions must be made from the same batch of 20-percent Rhodamine WT dye that is received from the manufacturer.

The discharge measurements will be made in the manner described in the earlier sections of this chapter, but before analysis by fluorometer, all samples and standard solutions should be allowed to sit overnight in a laboratory temperature bath. That will eliminate the need for temperature corrections to fluorometer dial readings.

As the hydrographer gains experience with the dye-dilution method, he will find that he can dispense with the computation of percentage of mixing (P_m) at the conclusion of his fluorometer analysis. By visual inspection of the computed individual plateau concentrations, he will be able to judge whether or not satisfactory mixing had been attained at the site of his sampling

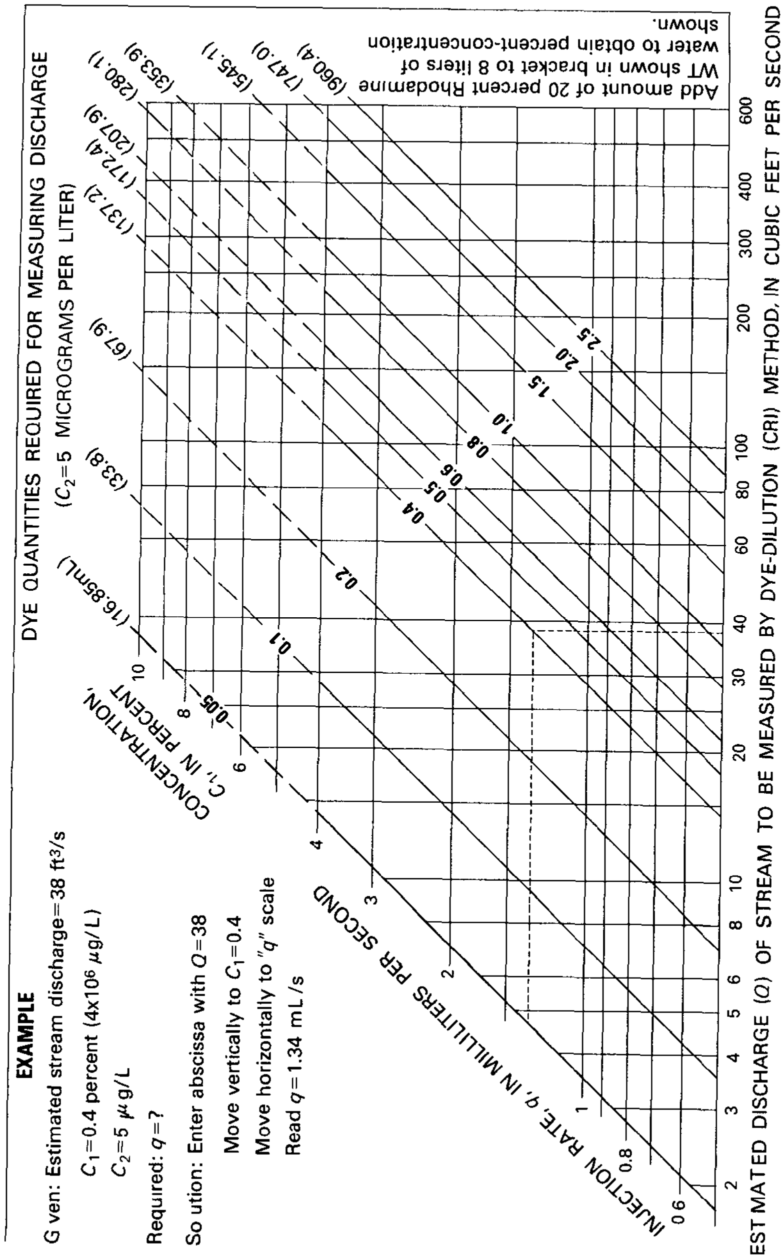


FIGURE 125.—Dye-injection rate of various stock dye concentrations related to estimated stream discharge.

MEASUREMENT OF DISCHARGE BY SODIUM DICHROMATE DILUTION

GENERAL

In Europe colorimetric analysis, using sodium dichromate ($Na_2Cr_2O_7 \cdot 2H_2O$) as the tracer, is the most widely used means of measuring discharge by the constant-rate-injection method. Because colorimetric analysis is rarely used in the U.S.A., the method will be described here only in brief; detailed descriptions are available in a report by the International Standards Organization (1973) and in a paper by Hosegood, Sanderson, and Bridle (1969).

The solubility of sodium dichromate is relatively high—the stock solution commonly used has a concentration of 600 g/L—and the salt satisfies most of the requirements for a tracer that are listed on page 211. Complications resulting from the presence of chromium ions in the water being gaged seldom occur, because natural waters generally contain few such ions. However, the nature and quantity of suspended sediment in the natural water can seriously affect the accuracy of analysis because of the possibility of sorption of sodium dichromate by the sediment. Another consideration in the use of sodium dichromate as a tracer is its potential toxicity to aquatic life, particularly in localized areas of high concentration; drinking-water standards in the U.S.A., for example, recommend a limiting concentration of 0.05 mg/L for chromium ions (Environmental Protection Agency, Environmental Studies Board, 1972, p. 62).

The factors that enter into the selection of a reach of channel for a measurement of discharge by sodium dichromate dilution are identical with those discussed earlier on pages 215–223. The types of apparatus for constant-rate-injection—mariotte vessel, floating siphon, and pressure tank—were described on pages 232–235.

The method of sampling for a discharge measurement by the constant-rate-injection method is similar to that described for fluorescent dyes (p. 237–240).

PRINCIPLE OF COLORIMETRIC ANALYSIS

Colorimetric analysis permits the measurement of extremely low concentrations of sodium dichromate. In that type of analysis a colorimeter is used to compare (1) the natural dilution of stream samples after tracer injection with (2) the known dilution of a stock solution of the salt. The comparison is based on differences in the absorption of light by the solutions after a reagent has been added to each of them.

The colorimeter is calibrated by the use of standard solutions, and it is recommended that the colorimeter used be one that gives a linear relation between color intensity (concentration) and meter reading. The colorimeter coefficient is then equal to the known concentration of the standard solutions divided by the average of the colorimeter readings.

METHOD OF ANALYSIS BY COLORIMETER

As mentioned earlier, the stock solution commonly used for discharge measurements contains 600 g of sodium dichromate in each liter of solution. The injection rate of this stock solution depends on the estimated river discharge and the desired increase or rise in concentration of chromium ions (Cr^6) corresponding to the plateau concentration. That rise in concentration, $(C_2 - C_b)$ in the equation that follows, is generally in the range of 0.04 to 0.08 mg/L.

Discharge is computed by use of the equation

$$Q = \left[\frac{C_1}{C_2 - C_b}\right] q, \tag{38}$$

where

Q is stream discharge (m^3/s),
q is rate of flow of injected solution (m^3/s or $mL/s \times 10^{-6}$),
C_1 is concentration of the injected solution (mg/L),
C_2 is concentration of the plateau of the concentration-time curve shown in fig. 116 (mg/L), and
C_b is background concentration of the stream (mg/L).

MEASUREMENT OF DISCHARGE BY SALT DILUTION

GENERAL

As mentioned earlier, when salt is the tracer used in measuring discharge by the tracer-dilution technique, the sudden-injection method is generally used. Although the sudden-injection method is usually less accurate than the constant-rate-injection method, the difficulty of handling the large quantities of salt solution required by the constant-rate-injection method make the method impractical. Common salt (NaCl) is the salt tracer generally used because it is relatively cheap and it meets all the criteria for a tracer that are listed on page 211.

The basic principles behind the salt-dilution method are as follows. The ion concentration of a dilute salt solution, such as natural river water, increases as the salt content increases. Consequently, the easily measured electrical conductivity (conductance) of the solution is an index of the salt concentration. Furthermore, over a wide range of concentrations the conductance is directly proportional to salt concentration. Therefore, after injection of a concentrated salt solution into a stream, the discharge can be computed by use of the ratios of the conductances of the injected and sampled salt solutions, instead of the ratios of the salt concentrations themselves. The procedure followed in the salt-dilution method is analogous to that used in the dye-dilution method in that relative conductance values are used rather than absolute values.

The factors that enter into the selection of the reach of channel for a salt-dilution measurement of discharge are identical with those discussed earlier on pages 215–223. Although loss of salt in the measurement reach by sorption is a negligible concern, reaches that have areas of slack water should be avoided, if possible, because the storage and slow release of tracer from those areas greatly prolongs the time required for the entire salt cloud to pass the sampling site. The sampling site should be free of excessive turbulence because the operation of the electrode cell that is placed in the stream to measure relative conductance is adversely affected by the presence of air bubbles.

The salt-dilution method of measuring discharge in open channels is littled used in the U.S.A. but is popular elsewhere, as in the U.S.S.R. The description of the salt-dilution technique that follows, and the sample computation that concludes the discussion, have been taken from a description of salt-dilution technique that follows, and the sample computation that concludes the discussion, have been taken from a description of salt-dilution stream gaging in the U.S.S.R. (World Meteorological Organization, 1962, p. 47–49). The subject is discussed under the following three headings:

1. Preparation and injection of the concentrated salt solution.
2. Measurement of relative conductance at the sampling site.
3. Computation of discharge.

PREPARATION AND INJECTION OF THE CONCENTRATED SALT SOLUTION

The following equipment is needed for the preparation and injection of the concentrated salt solution at the injection site:

1. Two tanks, each having a capacity of 60–70 L; one tank is used for preparation of the solution and the other for injection of the solution.
2. A measuring vessel of 10-L capacity, having a mark showing the exact level for 10 L of liquid.
3. A wooden or aluminum paddle for mixing the solution.
4. A vessel having a capacity of 0.3–0.5 L for adding small volumes of solution to the 10-L measuring tank.
5. A glass flask with a ground glass stopper, having a capacity of 20–30 mL, in which a sample of the solution is retained.

The quantity of salt (NaCl) required for injection depends on the discharge to be measured; 1–2 kg of salt is needed for each cubic meter per second of stream discharge. The smaller unit quantity of salt is used for streams having a low natural concentration of dissolved material. In estimating the quantity of salt to be brought to the injection site, consideration should also be given to the possibility that the discharge measurement will have to be repeated.

The preparation and measurement of the injection solution takes place at the injection site, but at a distance from the river to avoid any accidental spill of salt or salt solution into the river. An accident of that kind could affect the results of the discharge measurement. To prepare the injection solution, the quantity of salt needed is first determined from an estimate of the discharge and the natural concentration of dissolved materials in the stream. The computed quantity of salt is then placed in one of the large tanks, and river water is added to the tank at the rate of 10 L to each 3 kg of salt. The mixture is then stirred to obtain a saturated salt solution. After the larger undissolved crystals have settled to the bottom of the tank, the solution is transferred to the second large tank, and its volume is carefully measured in the process, the transfer being made by means of the 10-L measuring vessel. An additional amount of river water sufficient to fill the small glass flask (20–30 mL) to its brim is then added to the measured solution in the second large tank. The flask and stopper are carefully rinsed in that tank, after which the contents of the tank are thoroughly stirred once more. The small glass flask is then filled with the solution, tightly stoppered, and is delivered to the sampling site downstream for later use there. The large tank now holds the volume of salt solution that was originally measured into it. The large tank is then moved to the river, and at a signal from the sampling site that all is in readiness there, the tank is overturned to spill its contents almost instantaneously into midstream. That act completes the work to be done at the injection site.

MEASUREMENT OF RELATIVE CONDUCTANCE AT THE SAMPLING SITE

A conductance meter (fig. 126) that is a modification of a Wheatstone bridge is set up at the sampling site. The principal elements of the meter are a rheochord, a 10,000-ohm resistance chamber, and an electrode cell immersed in the stream, well away from the bank. The resistance chamber is used to compensate for the natural conductance of the water in the stream so that the needle of the galvanometer reads zero. The indicator on the rheochord is also set at zero. When this has been done, the signal to inject the salt solution is sent to the injection site.

After the solution has been poured into the stream at the injection

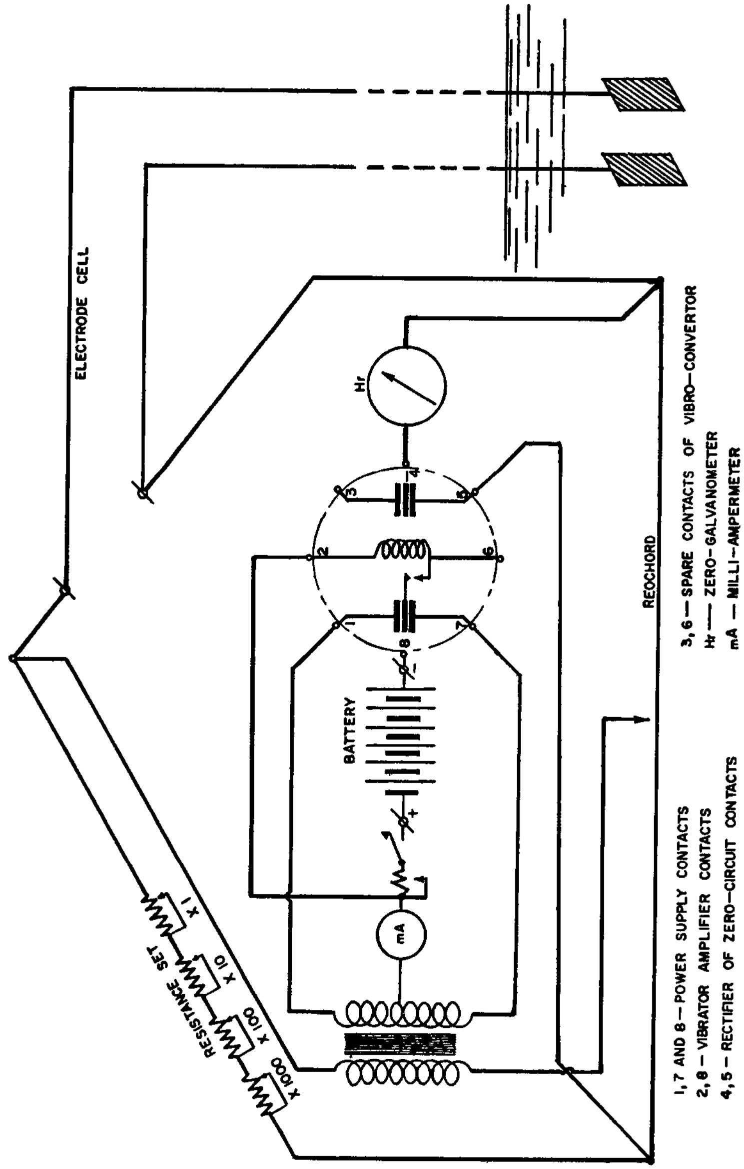


FIGURE 126.—Schematic diagram of conductance meter used in salt-dilution method of measuring discharge. (From World Meteorological Organization, 1962, p. 47.)

site, the current is switched on in the conductance meter, and the stability of the compensation for natural conductance is checked. If the galvanometer needle has deviated from zero, it is returned to zero by changing the resistance. Constant observation of the needle then begins. When the needle begins a perceptible movement to the right, it is a sign that the solution, or ionic wave, is approaching, and the stopwatch is started. From that time on, the galvanometer needle is maintained in a position approximating the zero reading by turning the rheochord handle. Every 15 s the needle is returned exactly to zero and the indicator position on the rheochord scale is read. That reading represents relative conductance (P) expressed in tenths of a percent (0/00):

$$P_W = 1{,}000 \left[\frac{(EC)_W - (EC)_o}{(EC)_o} \right], \qquad (39)$$

where EC is conductance (electrical conductivity), subscript W refers to the ionic wave, and subscript o refers to the natural water in the stream. (Note.—Conductance is the reciprocal of resistance.)

As the ionic wave passes the sampling site, relative conductance increases rapidly at first, reaches a peak, and then recedes slowly to zero or to a constant value slightly above or below zero. When the rheochord readings remain constant for 5 or 6 readings (1½ min) near the zero value, the measurement is terminated. Immediately thereafter the determination of the relative conductance of the injection solution begins, using the sample in the small flask obtained at the injection site just before the measurement began. For this determination the following equipment is needed:

1. A 10-L measuring vessel.
2. A jug for adding water to the measuring vessel.
3. A wooden or aluminum paddle for mixing the solution.
4. A 1-mL glass pipette.

The measuring vessel is rinsed three or four times with river water. It is then filled with river water to the 10-L mark, and the electrode cell of the conductance meter is immersed in it. The rheochord needle is set to zero, the current is switched on, and the resistance is adjusted to set the galvanometer needle to zero. The pipette is next rinsed in the injection solution that had been delivered in the small flask, and the pipette is used to add 1 mL of that solution to the 10 L of river water in the measuring vessel. The pipette is then rinsed three times in the vessel to insure that all of the extremely small volume of salt in the 1 mL of solution is introduced into the vessel. The contents of the vessel are then thoroughly mixed, and the relative conductance of the greatly diluted injection solution is measured with the meter, using the technique described for measuring relative conductance in the ionic wave. The ratio of dilution for the sample of injection solution is 1:10,000, because 1 mL of injection solution was mixed with 10 L of river water.

COMPUTATION OF DISCHARGE

The simplest way to describe the computation of discharge from salt-dilution data is by means of an example. Assume that in the preceding discussion of methodology, the stream discharge had been estimated to be about 4.5 m^3/s. The unit quantity of salt to be used was 2 $kg/m^3/s$ of discharge; consequently about 9 kg of salt was prepared. Water was added to the salt at the rate of 10 L per 3 kg of salt. As a result, 30 L (0.03 m^3) of injection solution was prepared. This value is shown on line 1 on the left-hand side of the computation sheet in figure 127. At the sampling site, 10 L of river water (line 2) was mixed with 1 mL of injection solution (line 3), giving a dilution ratio of 10,000 (line 4). The resistance of the river water was found to be 969 ohms (lines 5 and 6), and the resistance of the sample of diluted (1:10,000) injection solution was found to be 691 ohms (line 7). The relative conductance of that sample, as read on the rheochord in tenths of a percent, was 402 (line 8). On line 9, the rheochord reading for the dilute sample of injection solution is checked mathematically by use of the measured resistances. The equation used on line 9 is identical with equation 39. On line 10 the relative conductance of the injected solution is computed by multiplying the relative conductance of the diluted injection sample by the dilution ratio.

On the right-hand side of figure 127 are shown the relative conductance values read on the rheochord scale at 15-s intervals during the 15-min passage of the ionic wave. Subtotals are shown for the three columns of relative conductance values; the grand total is 1,933. The data are presented graphically in figure 128, where relative conductance is plotted against time.

Discharge is computed from equation 40, the denominator of which is the area under the curve of relative conductance versus time:

$$Q = \frac{V_1 P_p}{F \pm \Delta F}, \qquad (40)$$

where

Q is discharge, in cubic meters per second,

V_I is volume of the injected solution, in cubic meters,

P_p is relative conductance of the injected solution, in tenths of a percent,

F is the product of the sampling interval, in seconds, multiplied by the sum of the observed values of relative conductance, and

ΔF is a correction factor for any change in reading of the relative conductance of the natural river water between the start and end of the measurement:

$$\Delta F = \frac{P_{\text{start}} - P_{\text{end}}}{2}(t_n - t_1), \text{ where } t_n - t_1 \text{ is the time, in seconds, that elapsed during the measurement.}$$

The discharge is computed in lines 11–16 on the left-hand side of figure 127. Line 11 shows the sampling interval—15 s; line 12 shows the sum of the observed relative conductances of the ionic wave—1,933; line 13 shows the computation of F—28,995; line 14 shows the computation of ΔF—there was no change in natural relative conductance during the 900 s that elapsed during the measurement. The value of the denominator of equation 40 is shown on line 15. The discharge, computed in accordance with equation 40, is shown on line 16.

MEASUREMENT OF DISCHARGE BY DILUTION OF RADIOACTIVE TRACERS

GENERAL

The use of radioactive material for measuring discharge by tracer

Basic measurements data			
Item	Symbol	Value	Units
1 Volume of initial solution	V_1	0,0300	m^3
2 Volume of river water for dilution	Wo	10000	ml
3 Volume of initial solution for dilution	W_1	1	ml
4 Dilution ratio $\frac{Wo}{W_1}$	n	10000	—
5 Resistance of river water flow	R_o	969	ohms
6 Resistance of river water in tank	R	969	ohms
7 Resistance of mixture	R_t	691	ohms
8 Relative electrical conductivity of mixture on rheochord	P_t	402	‰
9 Or from formula $P_t = (\frac{R}{R_t} - 1)\ 1000$ ‰	P_t	402	‰
10 Relative electrical conductivity of initial solution $P_p = P_t\ n$	P_p	4020000	‰
Discharge calculation			
11 Time between observations	Δt	15	sec
12 Sum of ordinates of ionic wave graph	ΣP	1933	‰
13 Product $F = \Delta t(\Sigma P)$	F	28,995	‰ sec
14 Correction $\Delta F = \frac{P_1 - P_n}{2}(t_n - t_1) = 0$			‰ sec
15 Area of ionic wave graph $F \pm \Delta F$		28995	‰ sec
16 Water discharge $Q = \frac{V_1 \cdot P_p}{F \pm \Delta F} = \frac{(0,03)(4020000)}{28995}$			4.16 m³/sec

Observations of ionic wave					
Time t	‰ P	Time t	‰ P	Time t	‰ P
0'00"	0	6'15"	52	12'30"	3
15"	1	30"	48	45"	2
30"	3	45"	44	13'00"	2
45"	7	7'00"	40	15"	1
1'00"	13	15"	37	30"	1
15"	20	30"	33	45"	1
30"	32	45"	29	14'00"	1
45"	44	8'00"	26	15"	1
2'00"	56	15"	24	30"	1
15"	66	30"	21	45"	1
30"	75	45"	19	15'00"	0
45"	81	9'00"	18	15"	0
3'00"	85	15"	15	30"	0
15"	88	30"	13	45"	0
30"	90	45"	12	16'00"	0
45"	91	10'00"	10	15"	0
4'00"	90	15"	9	30"	0
15"	88	30"	8	45"	0
30"	85	45"	7	17'00"	0
45"	81	11'00"	6	15"	
5'00"	77	15"	5	30"	
15"	72	30"	5	45"	
30"	66	45"	4	18'00"	
45"	61	12'00"	3		
6'00"	56	15"	3		

$\Sigma P =$	1 428	$\Sigma P =$	491	$\Sigma P =$	14

FIGURE 127.—Sample computation of discharge using salt dilution in the sudden injection method. (From World Meteorological Organization, 1962, p. 49.)

dilution will not be described in detail, because the method is still (1980) considered to be in the experimental stage. Radioactive tracers should be ideal for the dilution method because concentrations as low as 10^{-9} ci/L may be accurately determined with a counter or count-rate meter whose sensing probe is immersed in the stream.

The radioactive element that has been most commonly used is gold 198, which has a half-life of 2.7 d. Tests have also been made using sodium 24, which has a half-life of 14.9 hr. It is necessary that the radioactive tracer have a short half-life so that the radioactivity introduced into the stream will decay to an insignificant level in a short

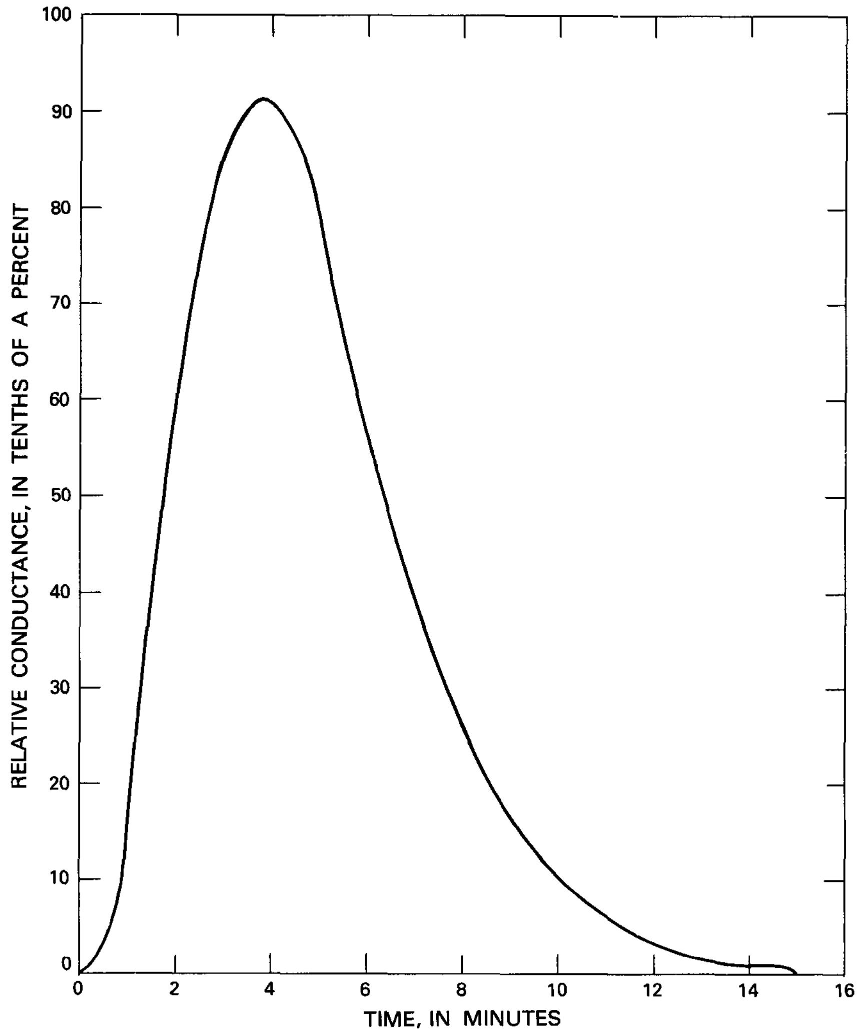


FIGURE 128.—Curve of relative conductance versus time for the ionic wave in the sample problem. (From World Meteorological Organization, 1962, p. 48.)

time. That requirement introduces a problem in logistics, in that the radioactive element must arrive from the nuclear plant and be used at the measurement site at the scheduled time, so that excessive decay of the radiation does not occur before the discharge measurement is made. Another factor that tends to discourage the use of radioactive tracers is the requirement of a government license merely to handle the material; radiation shielding must be provided while shipping and handling the material because of the potential threat to public health. In addition, approval must usually be obtained at all government levels, including the local level, before the radioisotope can be used in a stream, and local fear of radioactive pollution is often difficult to overcome.

METHODOLOGY

A measurement reach of channel is selected, using one of the equations, 28 to 32 (p. 221–222), as a guide in determining the length of reach needed for satisfactory mixing. Before the measurement is started, the instrument for counting gamma-ray emissions—Geiger or scintillation counter—is calibrated, taking into account the exact conditions under which the counter will be used.

A measured quantity of the radioisotope is introduced into the stream at the injection site by being quickly poured out of a bottle; or more frequently, a glass bottle containing the radioisotope is placed in a wire-screen container that is then lowered into the stream, after which the glass bottle is shattered to release the isotope for mixing in the streamflow. At the sampling site downstream, where the probe of the Geiger or scintillation counter is immersed in the stream, gamma-ray emissions are continuously counted. The counters are started well before the arrival of the isotope-stream mixture so that the counts received from natural (background) radiation sources may be measured. The counters continue to count throughout the passage of the isotope-stream mixture until the count returns to the background level. The total net count (N) is equal to the gross count obtained minus the background count accumulated during the period of the gross count.

The discharge is computed from the equation

$$Q = \frac{FA}{N}, \tag{41}$$

where

Q = stream discharge (volume per unit time),
F = a calibration factor for the probe and counting system,
A = total quantity of radioactivity introduced into the stream, and
N = total net count recorded by the counter.

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CHAPTER 8. MEASUREMENT OF DISCHARGE BY MISCELLANEOUS METHODS

GENERAL

This chapter deals with the measurement of discharge when conditions are such that it is not feasible to use either a velocity meter or tracer-dilution equipment. The situations and methodologies discussed include the following:

A. High flow
   1. Timed observation of floats

B. Low flow
   1. Volumetric measurement
   2. Use of a calibrated portable weir plate
   3. Use of a calibrated portable Parshall flume

C. Unstable flow—roll waves or slug flow
   1. Use of photographic techniques

Not discussed here is the practice followed in many countries, other than the U.S.A., in which discharge is measured by observing the head on gaging-station controls that are built in conformity with laboratory-rated weirs or flumes. The transfer of a laboratory discharge rating to a structure in the field requires the existence, and maintenance, of similitude between laboratory model and prototype, not only with regard to the structure, but also with regard to the approach channel. For example, scour and (or) fill in the approach channel will change the head-discharge relation, as will algal growth on the control structure. Both the structure and the approach channel must be kept free from accumulations of debris, sediment, and vegetal growth. Flow conditions downstream from the structure are significant only to the extent that they control the tailwater elevation, which may influence the operation of structures designed for free-flow conditions.

The existence or development of conditions that differ from laboratory conditions will necessitate in place calibration of the control to establish the extent of departure from the laboratory discharge ratings. In place calibration requires the measurement of discharge by current meter or by other means, as described in chapters 5 through 9. Because experience in the U.S.A. has indicated that departure from laboratory conditions is the norm, rather than the exception, gaging-station controls are always calibrated in place by the U.S. Geological Survey.

The reader who is interested in the measurement of discharge by the use of precalibrated controls is referred to publications by the World Meteorological Organization (1971) and the International Standards Organization (1969).

FLOATS

Floats are seldom used in stream gaging but are useful in an emergency for measuring high discharges under the following circumstances:

1. No conventional or optical current meter is available.
2. A current meter is available but the measurement structure—bridge or cableway—has been destroyed, and equipment for measuring from a boat is unavailable.
3. A conventional current meter is available, but floating ice or drift make it impossible to use the meter.

Surface floats are used in those situations, and they may be almost any distinguishable article that floats, such as wooden disks; bottles partly filled with either water, soil, or stones; or oranges. Floating ice cakes or distinguishable pieces of drift may be used if they are present in the stream.

Two cross sections are selected along a reach of straight channel for a float measurement. The cross sections should be far enough apart so that the time the float takes to pass from one cross section to the other can be measured accurately. A traveltime of at least 20 s is recommended, but a shorter time may be used for streams with such high velocities that it is not possible to find a straight reach of channel having adequate length. The water-surface elevation should be referenced to stakes along the bank at each cross section and at one or more intermediate sites. Those elevations will be used at a later date, when conditions permit, to survey cross sections of the measurement reach, and the end stakes will be used to obtain the length of the reach. The surveyed cross sections will then be used to derive an average cross section for the reach.

In making a float measurement a number of floats are distributed uniformly across the stream width, and the position of each with respect to distance from the bank is noted. The floats should be introduced a short distance upstream from the upstream cross section so that they will be traveling at the speed of the current when they reach the upstream section. A stopwatch is used to time their travel between the end cross sections of the reach. The estimated position of each float with respect to the bank is also noted at the downstream cross section.

If there is no bridge or cableway from which to introduce the floats in the stream, the floats will have to be tossed in from the streambank. If that is the situation that exists at a wide stream, it may be impossible to position any floats in the central core of the stream where most of the flow occurs. A float measurement of discharge made under those conditions would be meaningless. However, the difficulty of introducing floats at intervals across the entire width of a wide stream can be overcome if a boat can be obtained for the purpose.

The velocity of a float is equal to the distance between the end cross sections divided by the time of travel. The mean velocity in the vertical is equal to the float velocity multiplied by a coefficient whose value is dependent on the shape of the vertical-velocity profile of the stream and on the depth of immersion of the float with respect to stream depth. A coefficient of 0.85 is commonly used to convert the velocity of a surface float to mean velocity in the vertical.

The procedure for computing the discharge is similar to that used in computing the discharge for a conventional current-meter measurement. (See chapter 5.) The discharge in each subsection of the average cross section is computed by multiplying the area of the subsection by the mean vertical velocity for that subsection. The total discharge is equal to the sum of the discharges for all subsections.

Float measurements of discharge that are carefully made under favorable conditions may be accurate to within ±10 percent. Wind may adversely affect the accuracy of the computed discharge by its effect on the velocity of the floats. If a nonuniform reach is selected and few floats are used in the cross section, measurement results may be in error by as much as 25 percent.

VOLUMETRIC MEASUREMENT

The volumetric measurement of discharge is only applicable to small discharges, but it is the most accurate method of measuring such flows. In that method the hydrographer observes the time required to fill a container of known capacity, or the time required to partly fill a calibrated container to a known volume. The only equipment required, other than the calibrated container, is a stopwatch.

The container is calibrated in either of two ways. In the first method, water is added to the container by known increments of volume, and the depth of water in the container is noted after the addition of each increment. In the second method, the empty container is placed on a weighing scales, and its weight is noted. Water is added to the container in increments, and after each addition the total weight of container and water is noted, along with the depth of water in the container. The equation used to determine the volume corresponding to a depth that was read is

$$V = \frac{W_2 - W_1}{w}, \tag{42}$$

where

V = volume of water in container, in cubic feet or cubic meters,
W_2 = weight of water and container, in pounds or kilograms,

W_1 = weight of empty container, in pounds or kilograms, and
w = unit weight of water, 62.4 lb/ft^3 or 1,000 kg/m^3.

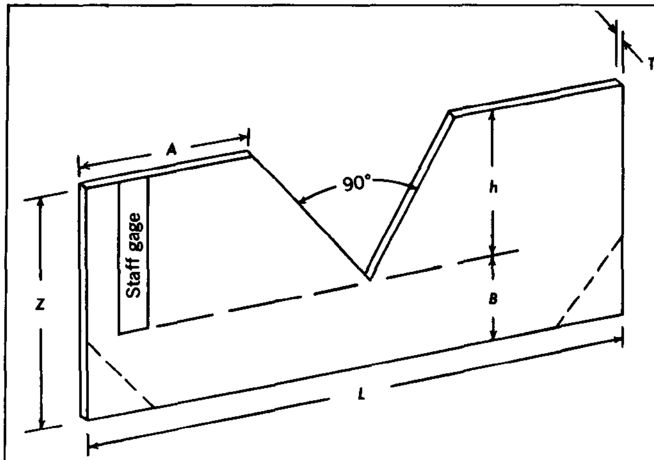
Volumetric measurements are usually made where the flow is concentrated in a narrow stream, or can be so concentrated, so that all the flow may be diverted into a container. Examples of sites presenting the opportunity for volumetric measurement of discharge are a V-notch weir; an artificial control where all the flow is confined to a notch or to a narrow width of catenary-shaped weir crest; and a cross section of natural channel where a temporary earth dam can be built over a pipe of small diameter, through which the entire flow is directed. Sometimes it is necessary to place a trough against the artificial control to carry the water from the control to the calibrated container. If a small temporary dam is built, the stage behind the dam should be allowed to stabilize before the measurement is begun. The measurement is made three or four times to be certain no errors have been made and to be sure the results are consistent.

Volumetric measurements have also been made under particular circumstances when no other type of measurement was feasible. One such circumstance involved a small stream that was in actuality a series of deep pools behind broad-crested weirs that acted as drop structures to dissipate the energy of the stream. At low flows the depth of water on the weir crest was too shallow to be measured by current meter, and the velocity in the pools was too slow for such measurement. To measure the discharge a large container of known volume was placed on a raft held close to the downstream weir face by ropes operated from the banks. A sharp-edged rectangular spout of known width was held so that one end butted tightly against the downstream face of the weir, the base of the spout being held just below the weir crest. The other end of the spout led to the container of known volume. Timed samples of the flow, sufficient to fill the container, were taken at a number of locations along the downstream face of the weir, the raft being moved laterally across the stream, from location to location, by the ropes. The procedure was analogous to making a conventional current-meter discharge measurement. Instead of measuring depth and velocity at a series of observation sites in the cross section, as is done in a current-meter measurement, the discharge per width of spout opening was measured at a series of observation sites. The discharge measured at each site was multiplied by the ratio of subsection width to spout width to obtain the discharge for the subsection. The total discharge of the stream was the summation of the discharges computed for each subsection.

PORTABLE WEIR PLATE

A portable weir plate is a useful device for determining discharge when depths are too shallow and velocities too low for a reliable current-meter measurement of discharge. A 90° V-notch weir is particularly suitable because of its sensitivity at low flows. Three different sizes of weir plate are commonly used by the U.S. Geological Survey; their recommended dimensions are given in figure 129.

The weir plate is made of galvanized sheet iron, using 10- to 16-gauge metal. The 90° V-notch that is cut in the plate is not beveled but is left with flat, even edges. The larger weir plates are made of thinner material than the smaller weir plates, but the medium and large plates are given additional rigidity by being framed with small angle irons fastened to the downstream face. A staff gage, attached to the upstream side of the weir plate with its zero at the elevation of the bottom of the notch, is used to read the head on the weir. The staff gage should be installed far enough from the notch to be outside the region of drawdown of water going through the notch. Drawdown becomes negligible at a distance from the vertex of the notch that is equal to twice the head on the notch. Consequently, if the weir plate has the dimensions recommended in figure 129, the staff gage should be installed near one end of the plate.



Weir	Z	h	B	L	A	T	Weight (lb)
Large	1.75	1.00	0.75	4.0	1.0	16 ga.	24
Medium	1.25	.80	.45	3.0	.7	14 ga.	17
Small	.75	.47	.28	2.0	.53	10 ga.	8

All dimensions, other than T, are in feet

FIGURE 129.—Portable weir-plate sizes.

To install the weir, the weir plate is pushed into the streambed. A pick or shovel may be necessary to remove stones or rocks that prevent even penetration of the plate. A carpenter's level is usually used to insure that the top of the plate is horizontal and that the face of the plate is vertical. Another means of leveling the weir plate is by the use of a staff gage or level bubble attached at each end of the plate. The weir plate is then leveled by adjusting it until both staff gages give identical readings of the water surface or until the level bubbles are centered. Through eyebolts that are attached to the plate, rods are driven into the streambed to maintain the weir in a vertical position. Soil or streambed material is packed around the weir plate to prevent leakage under and around it. Canvas is placed immediately downstream from the weir to prevent undercutting of the bed by the falling jet. It ordinarily requires only one man to make the installation.

A large weir plate of the dimensions shown in figure 129 can measure discharges in the range from 0.02 to 2.0 ft^3/s (0.00057 to 0.057 m^3/s) with an accuracy of ± 3 percent, if the weir is not submerged. A weir is not submerged when there is free circulation of air on all sides of the nappe. The general equation for flow over a sharp-edged 90° V-notch weir is

$$Q = Ch^{5/2}, \tag{43}$$

where

Q = discharge, in cubic feet per second or cubic meters per second,
h = static head above the bottom of the notch, in feet or meters, and
C = coefficient of discharge.

Each weir should be rated by volumetrically measuring the discharge corresponding to various values of head. In the absence of such a rating, a value of 2.47 may be used for C in equation 43 when English units are used, or 1.36 when metric units are used.

When the weir is installed it will cause a pool to form on the upstream side of the plate. No readings of head on the notch should be recorded until the pool has risen to a stable elevation. The head should then be read at half-minute intervals for about 3 min, and the mean value of those readings should be the head used in equation 43 to compute discharge. After the completion of the measurement the weir plate is removed.

PORTABLE PARSHALL FLUME

A portable Parshall flume is another device for determining discharge when depths are too shallow and velocities too low for a current-meter measurement of discharge. The portable flume used by the U.S. Geological Survey is a modified form of the standard Parshall flume (p.314–319) having a 3-in (0.076 m) throat. The modification consists, primarily, of the removal of the downstream diverging section of the standard flume. The purpose of the modification is to reduce the weight of the flume and to make it easier to install. Because the portable Parshall flume has no downstream diverging section, it cannot be used for measuring flows when the submergence ratio exceeds 0.6. The submergence ratio is the ratio of the downstream head on the throat to the upstream head on the throat. Although a submergence ratio of 0.6 can be tolerated without affecting the rating of the portable flume, in practice the flume is usually installed so that the flow passing the throat has virtually free fall. That is usually accomplished by building up the streambed a couple of inches under the level converging floor of the flume. (See fig. 130.)

Figure 130 shows the plan and elevation of the portable Parshall flume. The gage height or upstream head on the throat is read in the small stilling well that is hydraulically connected to the flow by a ⅜-in hole. The rating for the flume is given in table 14.

When the flume is installed in the channel, the floor of the converging section is set in a level position by using the level bubble that is attached to one of the braces (fig. 130). A carpenter's level can be used for that purpose if the flume is not equipped with a level bubble. Soil or streambed material is then packed around the flume to prevent leakage under and around it. Figure 131 shows a typical field installation. After the flume is installed, water will pool upstream from structure. No gage-height readings should be recorded until the pool has risen to a stable level. As with the portable weir, after stabilization of the pool level, gage-height readings should be taken at half-

TABLE 14.—*Rating table for 3-in modified Parshall flume*

Gage height (ft)	Discharge (ft^3/s)	Gage height (ft)	Discharge (ft^3/s)	Gage height (ft)	Discharge (ft^3/s)
0.01	0.0008	0.21	0.097	0.41	0.280
.02	.0024	.22	.104	.42	.290
.03	.0045	.23	.111	.43	.301
.04	.0070	.24	.119	.44	.312
.05	.010	.25	.127	.45	.323
.06	.013	.26	.135	.46	.334
.07	.017	.27	.144	.47	.345
.08	.021	.28	.153	.48	.357
.09	.025	.29	.162	.49	.368
.10	.030	.30	.170	.50	.380
.11	.035	.31	.179	.51	.392
.12	.040	.32	.188	.52	.404
.13	.045	.33	.198	.53	.417
.14	.051	.34	.208	.54	.430
.15	.057	.35	.218	.55	.443
.16	.063	.36	.228	.56	.456
.17	.069	.37	.238	.57	.470
.18	.076	.38	.248	.58	.483
.19	.083	.39	.259	.59	.497
.20	.090	.40	.269		

minute intervals for about 3 min. The mean value of those readings is the stage used in table 14 to obtain the discharge. A carefully made measurement should have an accuracy of ±2 or 3 percent. After completion of the measurement, the portable flume is removed.

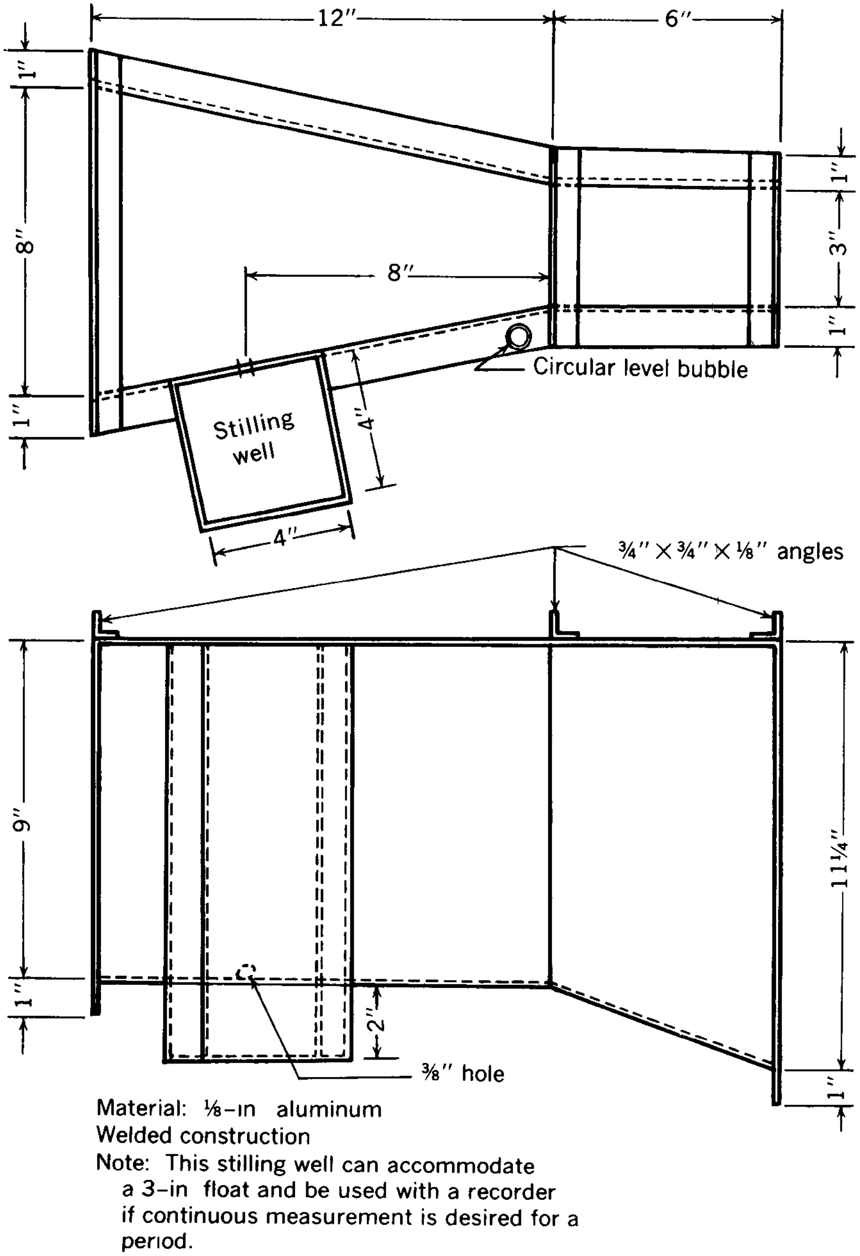


FIGURE 130.—Working drawing of modified 3-in Parshall flume.

MEASUREMENT OF UNSTABLE FLOW—ROLL WAVES OR SLUG FLOW

CHARACTERISTICS OF UNSTABLE FLOW

Unstable or pulsating flow often occurs during flash floods in arid areas. In pulsating flow the longitudinal profile is marked by a series of abrupt translatory waves (fig. 132) that rapidly move downstream. Translatory waves, commonly called roll waves or slug flow, can only develop in steep channels of supercritical slope and, therefore, are a matter of concern to the designer of steep-gradient channels. Channels are usually designed for stable flow. Should pulsating flow occur at high stages in a channel so designed, the channel capacity may be inadequate at a discharge much smaller than the design flow. Furthermore, if the overriding translatory wave carries an appreciable part of the total flow, conventional stream-gaging methods cannot be used to determine the discharge. Conventional water-stage recorders

FIGURE 131.—Modified 3-in Parshall flume installed for measuring discharge.

of either the float or pressure-sensing type do not react quickly enough to record the rapidly fluctuating stage; depths and velocities change too rapidly to permit discharge measurement by current meter; no stage-discharge relation exists for pulsating flow; and the commonly used formulas for computing stable open-channel discharge are not applicable.

DETERMINATION OF DISCHARGE

In brief, the method of determining the discharge of pulsating flow requires (1) computation of the discharge (Q_w) in the overriding waves and (2) computation of the discharge (Q_1) in the shallow-depth, or overrun, part of the flow. The sum of the two discharges is the total discharge at the time of observation.

To compute the discharge (Q_w) in an overriding wave, which is usually wedge shaped (fig. 132), the dimensions of the wave are observed, and the volume of the wave is divided by the elapsed time between the arrival of waves. For example, if the wedge-shaped wave in a train of waves has a volume of 200 ft^3 (5.67 m^3) and a wave arrives every 10 s, the discharge in the overriding wave is 200/10, or 20 ft^3/s (0.57 m^3/s). Average values are usually used in the computation—for example, the average volumes of five consecutive waves and the average time interval between the arrival of those consecutive waves. It should be mentioned at this point that the longitudinal profile of the wave is actually slightly concave upward and the wave front, while extremely steep, is not vertical. However, to simplify the computation of discharge, the waves are assumed to have a simple wedge shape.

To compute the discharge (Q_1) in the shallow-depth, or overrun, part of the flow, the cross-sectional area of the shallow-depth flow is

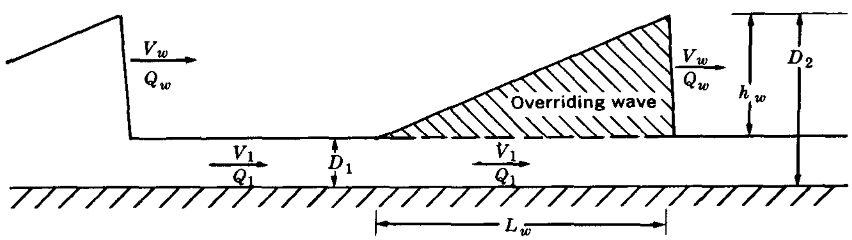


FIGURE 132.—Schematic sketch of longitudinal water-surface profile during pulsating flow.

observed (D_1 × channel width), and that area is multiplied by its velocity, V_1. Seldom will there be time enough between waves to obtain velocity observations of V_1 with a conventional current meter. V_1 may be computed by some stable-flow equation, such as the Manning equation, but preferably the surface velocity of shallow-depth flow should be measured by optical current meter. (See p. 91–93.) The surface velocity can then be multiplied by an appropriate coefficient—0.9 or 0.85, for example—to give the mean velocity, V_1. The final step is to compute the total discharge at the time of observation by adding Q_w and Q_1.

EXAMPLES OF DISCHARGE DETERMINATION

This section briefly describes two examples of discharge determinations made under conditions of pulsating flow in southern California.

Holmes (1936) obtained photographic documentation of a train of translatory waves in a steep stormflow channel. The rectangular channel was 43 ft (13.10 m) wide, 8 ft (2.44 m) high, and had a slope of 0.02. The waves themselves lapped at the top of the manmade channel giving them a height (D_2) of 8 ft, and their length (L_w) was about 600 ft (180 m). The average distance between wave crests was about 1,200 ft (360 m), and the average time interval between arrival of the waves (T_p) was 51 s. The channel between waves was dry or nearly so (D_1=0), meaning that the entire discharge was transported in the waves. The average discharge over the time T_p, computed from the equation

$$Q = (\text{volume})/T_p = \tfrac{1}{2}\,(W)\,(h_w)\,(L_w)/(T_p), \tag{44}$$

was therefore about 2,000 ft³/s (56 m³/s). The channel had been designed for stable-flow conditions and, according to the Manning equation, had a capacity of about 16,000 ft³/s (405 m³/s). We see then, that under the observed conditions of unstable flow the channel could accommodate only one-eighth of the design discharge.

Thompson (1968) described the experimental measurement of pulsating flow in the rectangular stormflow channel of Santa Anita Wash in Arcadia, California. The concrete channel was 28 ft (8.5 m) wide and had a slope of 0.0251. On the infrequent occasions when the channel had carried storm runoff in the past, the flow had been observed to be pulsating. For the test, water was released into the channel from an upstream reservoir at controlled rates of approximately 100 ft³/s (2.8 m³/s), 200 ft³/s (5.6 m³/s), 300 ft³/s (8.5 m³/s). Unstable flow did not develop until the flow came out of a bend in the steep storm channel and entered a straight reach of the channel. In other words, the released flow was stable upstream from the bend and pulsating downstream from the bend. During the release of water, discharge measurements were made continuously by current meter in the stable flow upstream from the bend. The discharge hydrograph based on those measurements is shown by solid line in figure 133. Downstream from the bend a simultaneous attempt was made to measure the unstable flow by the method described in this paper, for the purpose of verifying the method.

At the test site, about 3,300 ft (1,000 m) downstream from the bend, the equivalent of a series of staff gages, in the form of a grid, was painted on one of the vertical channel walls so that water-surface

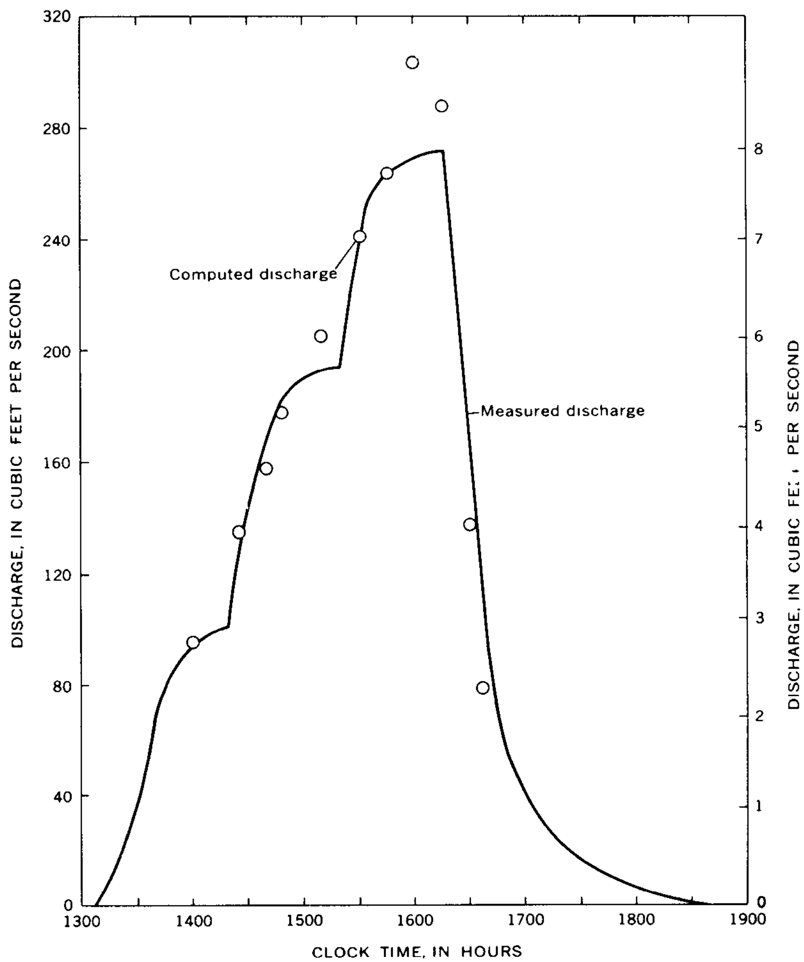


FIGURE 133.—Discharge hydrograph at Santa Anita Wash above Sierra Madre Wash, Calif., April 16, 1965, and plot of discharges computed from observations of flow.

elevations could easily be read by a crew of observers. Some of the observers were equipped with both still- and motion-picture cameras to document the observations; others were equipped with stopwatches to time wave velocity over a measured course of 102 ft (31 m) and to obtain the elapsed time between the arrival of waves. The waves were not evenly spaced and occasionally one wave would overtake another, but in general the waves were fairly uniform in size, maintained their spacing, and underwent little attenuation in the 3,300-ft reach above the test site. For computation purposes the average volumes of five consecutive waves and their average elapsed time between arrivals (T_p) were used, and discharges were computed at 15 min intervals using the procedure described earlier. For example, at the time of greatest discharge, the average value of T_p was 9.6 s, and the average wave dimensions were D_1 = 0.42 ft (0.13 m), h_w = 0.66 ft (0.20 m), and L_w = 158 ft (48.2 m). The computed value of Q_w equaled 152 ft^3/s (4.30 m^3/s). No optical current meter was available at the time, and Q_1 was therefore computed by the Manning equation. The computed value of Q_1 also equaled 152 ft^3/s (4.30 m^3/s), giving a computed total discharge 304 ft^3/s (8.60 m^3/s) at the time of observation. The values of discharge that were computed at 15-min intervals are plotted as open circles in figure 133 and show satisfactory agreement with the "true" discharge hydrograph. The field test of the method was therefore considered a success.

PROPOSED INSTRUMENTATION

There is as yet no instrumentation that is operational for automatically recording the data required to compute discharge under conditions of pulsating flow. Thompson concluded the above-cited report (1968) by describing three types of automatic instrumentation—photographic, depth sensing, and dye dilution—that might be developed for that purpose.

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CHAPTER 9.—INDIRECT DETERMINATION OF PEAK DISCHARGE

INTRODUCTION

During floods, it is frequently impossible or impractical to measure the peak discharges when they occur because of conditions beyond control. Roads may be impassable; structures from which current-meter measurements might have been made may be nonexistent, not suitably located, or destroyed; knowledge of the flood rise may not be available sufficiently in advance to permit reaching the site near the time of the peak; the peak may be so sharp that a satisfactory current-meter measurement could not be made even with an engineer present at the time; the flow of debris or ice may be such as to prevent use of a current meter; or limitations of personnel might make it impossible to obtain direct measurements of high-stage discharge at numerous locations during a short flood period. Consequently, many peak discharges must be determined after the passage of the flood by indirect methods such as slope-area, contracted opening, flow-over-dam, or flow-through-culvert.

Indirect determinations of discharge make use of the energy equation for computing streamflow. The specific equations differ for different types of flow, such as unobstructed open-channel flow, flow over dams, and flow through culverts. However, all the methods involve these general factors:

1. Physical characteristics of the channel; that is dimensions and conformation of the channel within the reach used and boundary conditions.
2. Water-surface elevations at time of peak stage to define the upper limit of the cross-sectional areas and the difference in elevation between two or more significant cross sections.
3. Hydraulic factors based on physical characteristics, water-surface elevations, and discharge, such as roughness coefficients and discharge coefficients.

This chapter provides only a brief general discussion of the procedures used in collecting field data and in computing discharge by the various indirect methods. That highly specialized subject is treated in detail in the several manuals of the series "Techniques of Water-Resources Investigations of the United States Geological Survey" that are listed as references at the end of this chapter.

It should be remembered that the discharge that is determined by either direct measurement or by indirect methods includes not only the water but also any substances suspended or dissolved in the water (p. 79).

COLLECTION OF FIELD DATA

The data required for the computation of discharge by indirect methods are obtained in a field survey of a reach of channel. The survey includes the elevation and location of high-water marks corresponding to the peak stage; cross sections of the channel along the reach; selection of a roughness coefficient; and description of the geometry of dams, culverts, or bridges, depending on the type of peak-discharge determination to be made. The selection of a suitable site is a most important element in the application of the indirect method of discharge determination.

It is recommended that a transit be used to make a "transit-stadia" survey of the selected site. That method combines vertical and horizontal control surveys in one operation and is accurate, simple, and speedy.

Selection of a roughness coefficient remains essentially an "art" that is developed through experience. The factors that exert the greatest influence on the coefficient of roughness are the character of the streambed material, cross section irregularity, the presence of vegetation, and the alinement of the channel. In the Manning equation the roughness coefficient, n, ranges from as low as 0.012 for a concrete-lined channel in excellent condition or for a smooth sand channel of regular geometry to more than 0.1 for overbank areas having a heavy cover of brush.

SLOPE-AREA METHOD

The slope-area method is the most commonly used technique of indirect discharge determination. In the slope-area method, discharge is computed on the basis of a uniform-flow equation involving channel characteristics, water-surface profiles, and a roughness or retardation coefficient. The drop in water-surface profile for a uniform reach of channel represents energy losses caused by bed and bank roughness.

In applying the slope-area method, any one of the well-known variations of the Chezy equation may be used. However the Manning equation is preferred in most countries, including the U.S.A., because it is simple to apply, and the many years of experience in its use have shown that it produces reliable results.

The Manning equation, written in terms of discharge, is

$$Q = \frac{1.486}{n} AR^{2/3}S^{1/2}, \quad \text{(in English units), or} \tag{45}$$

$$Q = \frac{AR^{2/3}S^{1/2}}{n} \quad \text{(in metric units),} \tag{45a}$$

where
- Q = discharge,
- A = cross-sectional area,
- R = hydraulic radius,
- S = friction slope, and
- n = roughness coefficient.

The Manning equation was developed for conditions of uniform flow in which the water-surface profile and energy gradient are parallel to the streambed and the area, hydraulic radius, and depth remain constant throughout the reach. For lack of a better solution, it is assumed that the equation is also valid for the nonuniform reaches that are invariably encountered in natural channels, if the water-surface gradient is modified by the difference in velocity-head between cross sections. The energy equation for a reach of nonuniform channel between cross section 1 and cross section 2 shown in figure 134 is

$$(h_1 + h_{v_1}) = (h_2 + h_{v_2}) + (h_f)_{1-2} + k(\Delta h_v)_{1-2}, \tag{46}$$

where
- h = elevation of the water surface at the respective cross sections above a common datum,
- h_v = velocity head at the respective cross sections = $\alpha V^2/2g$, where α = velocity-head coefficient,
- h_f = energy loss due to boundary friction in the reach,
- Δh_v = upstream velocity head minus the downstream velocity head, used as a criterion for expansion or contraction of reach, and
- $k(\Delta h_v)$ = energy loss due to acceleration or deceleration in a contracting or expanding reach, where
- k = energy loss coefficient.

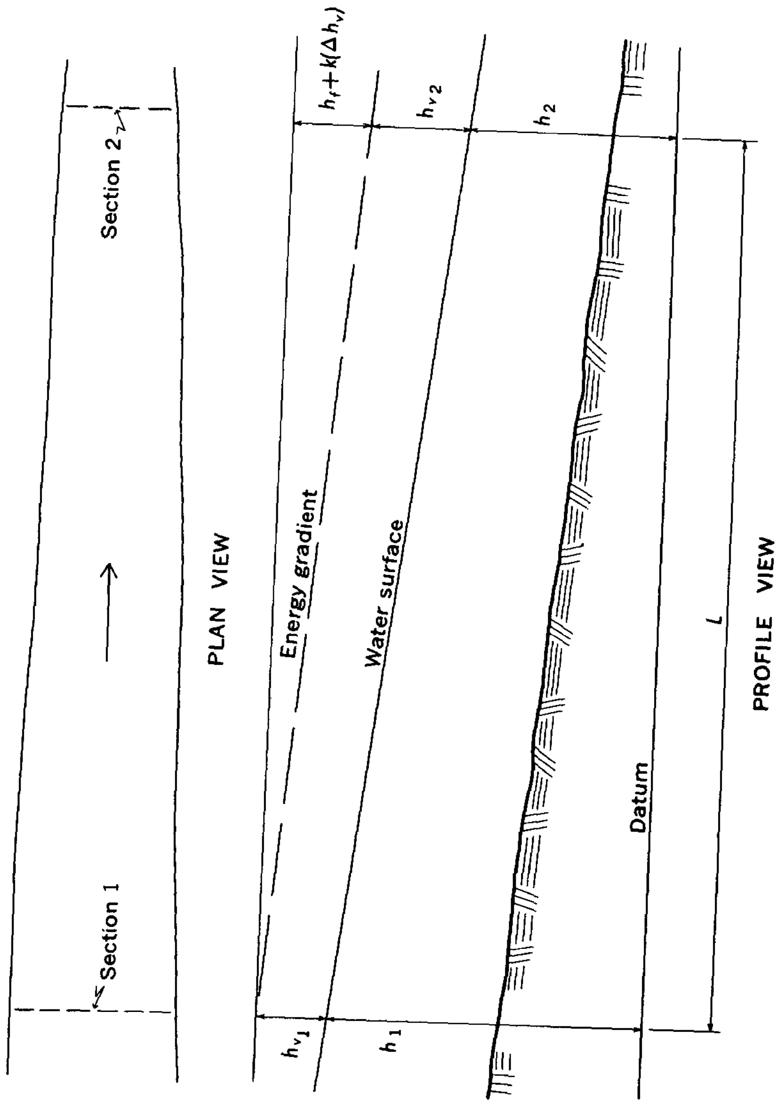

FIGURE 134.—Definition sketch of a slope-area reach.

The friction slope (S) to be used in the Manning equation is thus defined as

$$S = \frac{h_f}{L} = \frac{\Delta h + \Delta h_v - k(\Delta h_v)}{L}, \tag{47}$$

where Δh is the difference in water-surface elevation at the two sections and L is the length of the reach.

In using the Manning equation the conveyance, K, is computed for each cross section as $(1.486/n)AR^{2/3}$ in English units, or $(1/n)AR^{2/3}$ in metric units. The mean conveyance in the reach is then computed as the geometric mean of the conveyance at the two sections. This procedure is based on the assumption that the conveyance varies uniformly between sections. The discharge is computed by use of the equation

$$Q = \sqrt{K_1 K_2 S}, \tag{48}$$

where S is the friction slope as previously defined.

CONTRACTED-OPENING METHOD

The contraction of a stream channel by a roadway crossing creates an abrupt drop in water-surface elevation between an approach section and the contracted section under the bridge. The contracted section framed by the bridge abutments and the channel bed is in a sense a discharge meter that can be utilized to compute floodflows. The head on the contracted section is defined by high-water marks, and the geometry of the channel and bridge is defined by field surveys.

In computations of peak discharge at a contraction, the drop in water-surface level between an upstream section and a contracted section is related to the corresponding change in velocity. The discharge equation results from writing the energy and continuity equations for the reach between these two sections, designated as sections 1 and 3 in figure 135.

The discharge equation is

$$Q = CA_3\sqrt{2g\left(\Delta h + \alpha_1 \frac{V_1^2}{2g} - h_f\right)}, \tag{49}$$

in which

Q = discharge,
g = acceleration of gravity,
C = coefficient of discharge based on the geometry of the bridge and embankment,
A_3 = gross area of section 3; this is the minimum section parallel to the constriction between the abutments, and it is not

necessarily located at the downstream side of the bridge,

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

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

h_f = the head loss caused by friction between sections 1 and 3.

The friction loss, h_f, as computed by the Manning equation, is only

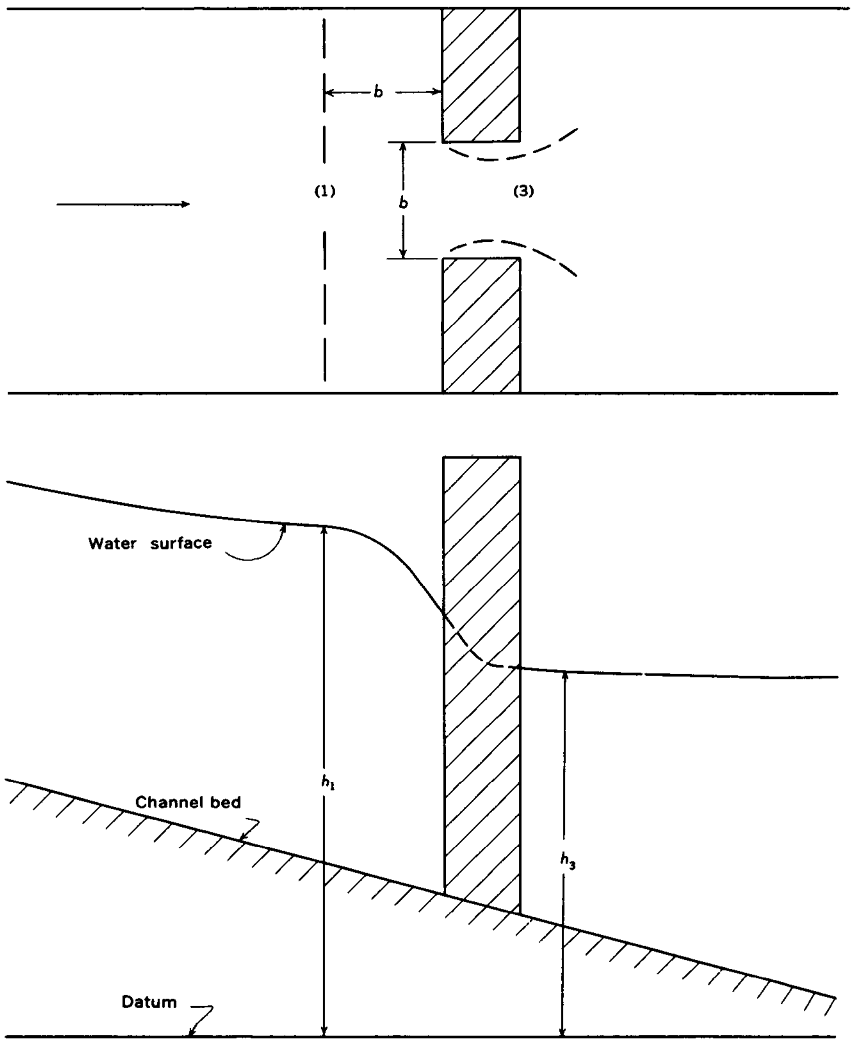

FIGURE 135.—Definition sketch of an open-channel contraction.

an approximation of the actual loss because of the rapid change in velocity from section 1 to section 3. Therefore, satisfactory results are attainable only if the term "h_f" is small relative to the difference in head, Δh.

FLOW OVER DAMS AND WEIRS

The term "dams," as used here, also includes highway and railway embankments that act as broad-crested dams during floods. The peak discharge over a dam or weir can be determined on the basis of a field survey of high-water marks and the geometry of the particular structure. (The terms "dam" and "weir" are used interchangeably.)

The basic equation for flow over a dam is

$$Q = CbH^{3/2}, \tag{50}$$

where

Q = discharge,
C = a coefficient of discharge having the dimensions of the square root of the acceleration of gravity,
b = width of the dam normal to the flow, excluding the width of piers, if any, and
H = total energy head ($h + V_a^2/2g$) referred to the crest of the dam, where h = static head, and V_a = mean velocity at the approach section to the dam.

It is apparent from equation 50 that the reliability of a computation of flow over a dam is dependent primarily on using the correct dam coefficient, C. Values of C vary with the geometry of the dam and with the degree of submergence of the dam crest by tailwater. One of the manuals referred to on page 273 (Hulsing, 1967) treats in detail the coefficients associated with sharp-crested (thin-plate) weirs, broad-crested weirs, round-crested weirs, and weirs of unusual shape. Because the technical details in that manual cannot be readily summarized here, the reader is referred to the Hulsing report.

FLOW THROUGH CULVERTS

The peak discharge through culverts can be determined from high-water marks that define the headwater and tailwater elevations. This indirect method is used extensively to measure flood discharges from small drainage areas.

The placement of a roadway fill and culvert in a stream channel causes an abrupt change in the character of flow. This channel transition results in rapidly varied flow in which acceleration, rather than boundary friction, plays the primary role. The flow in the approach channel to the culvert is usually tranquil and fairly uniform. However, within the culvert the flow may be tranquil, critical, or rapid if the culvert is partially filled, or the culvert may flow full under pressure.

The physical features associated with culvert flow are illustrated in figure 136. They are the cross section in the approach channel located upstream from the culvert entrance at a distance that is equivalent to the width of the culvert opening; the culvert entrance; the culvert barrel; the culvert outlet; the farthest downstream section of the barrel; and the tailwater downstream from the culvert barrel.

The change in the water-surface profile in the approach channel reflects the effect of acceleration that results from the contraction of cross sectional area. Loss of energy near the entrance is related to the sudden contraction and subsequent expansion of the live stream within the barrel, and entrance geometry has an important influence on this loss. The important features that control the stage-discharge relation at the approach section can be the occurrence of critical depth in the culvert, the elevation of the tailwater, the entrance or barrel geometry, or a combination of these elements.

The peak discharge through a culvert is determined by application of the continuity equation and the energy equation between the approach section and a section within the culvert barrel. The location of the downstream section depends on the state of flow in the culvert barrel. For example, if critical flow occurs at the culvert entrance, the headwater elevation is not a function of either the barrel friction loss or the tailwater elevation, and the terminal section is located at the upstream end of the culvert.

Information obtained in the field survey includes the peak elevation of the water surface upstream and downstream from the culvert and the geometry of the culvert and approach channel. Reliable high-water marks can rarely be found in the culvert barrel; therefore, the type of flow that occurred during the peak flow cannot always be determined directly from field data, and classification becomes a trial-and-error procedure.

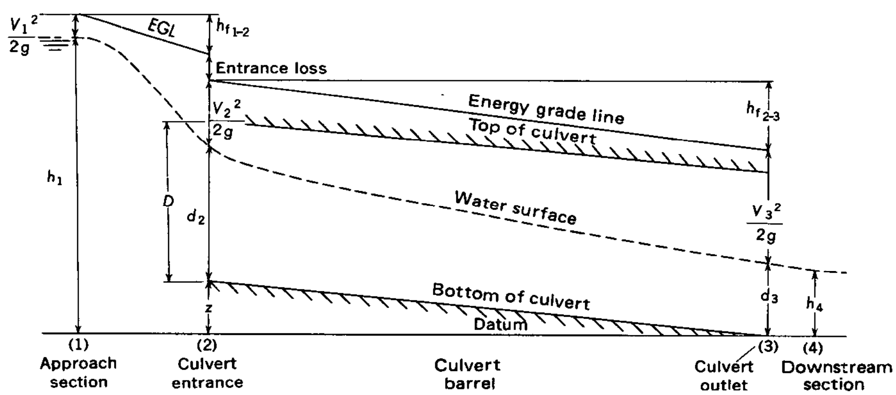

FIGURE 136.—Graphical presentation of the Bernoulli equation in culvert flow.

GENERAL CLASSIFICATION OF FLOW

For convenience in computation, culvert flow has been classified into six types on the basis of the location of the control section and the relative heights of the headwater and tailwater elevations. The six types of flow are illustrated in figure 137, and pertinent characteristics of each type are given in table 15. From that information the general classification of types of flow can be made.

1. If h_4/D is equal to or less than 1.0 and $(h_1 - z)/D$ is less than 1.5, only types I, II, and III flow are possible.
2. If h_4/D is greater than 1.0, only type IV flow is possible.
3. If h_4/D is equal to or less than 1.0 and $(h_1 - z)/D$ is equal to or greater than 1.5, only types V and VI flow are possible.

A manual by Bodhaine (1968) discusses trial-and-error procedures for further identification of the type of culvert flow and for the computation of discharge.

ESTIMATING DISCHARGE FROM SUPERELEVATION IN BENDS

Situations exist where none of the four methods previously described for determining peak discharge can be reliably applied. For example, in a highly sinuous, steep canyon stream, there may be no straight reaches of sufficient length for reliable application of the slope-area method; no abrupt area contractions may exist; and no manmade structures, such as dams or culverts, may have been built. In the above situation the peak discharge may sometimes be estimated from the superelevation in a bend of the stream (Apmann, 1973). The discharge should be estimated at each of several bends, after which the several estimated discharges are averaged.

A fundamental characteristic of open channel flow is the deformation of the free surface in a bend because of the action of centrifugal force. The water surface rises on the concave or outside bank of the

TABLE 15.—*Characteristics of types of culvert flow*

[D = maximum vertical height of barrel and diameter of circular culverts]

Flow type	Barrel flow	Location of terminal section	Kind of control	Culvert slope	$\frac{h_1-z}{D}$	$\frac{h_4}{h_c}$	$\frac{h_4}{D}$
I	Partly full	Inlet	Critical depth.	Steep	<1.5	<1.0	≤1.0
II	do	Outlet	do	Mild	<1.5	<1.0	≤1.0
III	do	do	Backwater	do	<1.5	>1.0	≤1.0
IV	Full	do	do	Any	>1.0	–	>1.0
V	Partly full	Inlet	Entrance geometry.	do	≥1.5	–	≤1.0
VI	Full	Outlet	Entrance and barrel geometry.	do	≥1.5	–	≤1.0

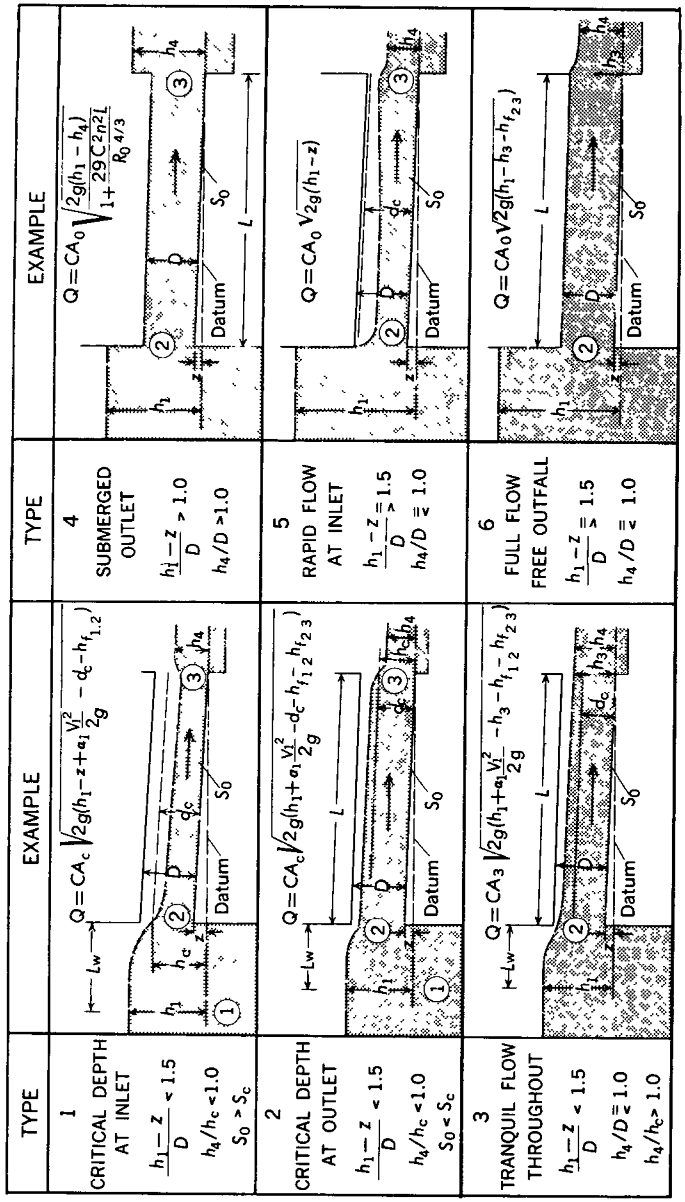

FIGURE 137.—Classification of culvert flow.

bend and lowers along the convex or inside bank of the bend. The difference in water-surface elevation between the banks is the superelevation. Superelevation varies with angular distance in the bend because of acceleration of the fluid entering and leaving the curve and because of the varying curvature of streamlines within the bend.

The discharge equation given by Apmann (1973) is

$$Q = A\sqrt{\frac{gh}{K}} \tag{51}$$

where

Q = discharge,
A = average radial cross section in the bend,
g = acceleration of gravity,
h = superelevation, that is, the maximum difference in water-surface elevation, measured along a radius of the bend, between inner and outer banks of the bend, and
K = superelevation coefficient.

The value of K is determined from the equation

$$K = \frac{5}{4}\tanh\left(\frac{r_c\Theta}{b}\right)\ln\left(\frac{r_o}{r_i}\right). \tag{52}$$

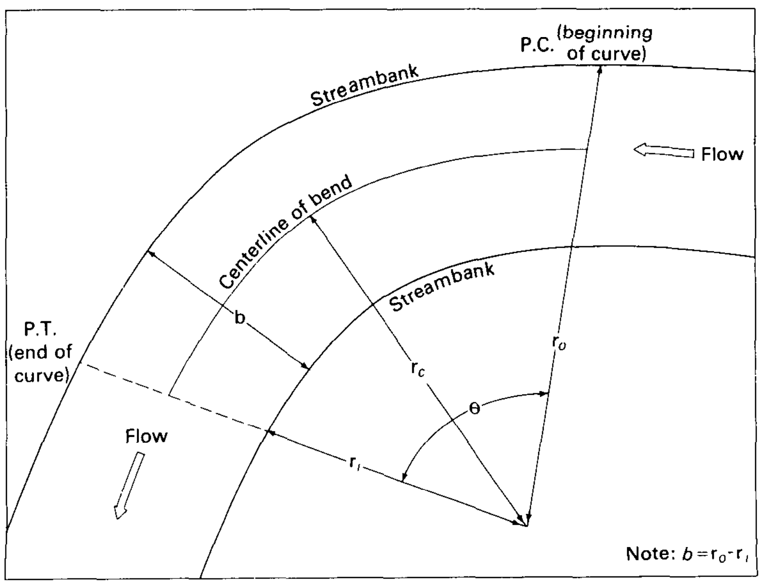

FIGURE 138.—Idealized sketch of a bend (plan view).

(NOTE.—tanh is the hyperbolic tangent)
The symbols in equation 52 are shown in the sketch in figure 138.

The method described applies only to bends having no overbank flow. The limited amount of work that has been done with the method indicates that it should not be applied where superelevations are less than 0.25 ft (0.076 m) because of uncertainties regarding the elevations of high-water marks at the banks. These uncertainties result from wave action and from the thickness of the bank-deposited debris that is often used as a high-water mark.

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INDEX

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Water Supply Paper 2175, Vol. I, Table 8 on page 165 should be replaced by the following table:

TABLE 8.—*Wet-line table, giving difference, in meters, between wet-line length and vertical depth for selected vertical angles*

Wet-line length (m)	*Vertical angle of sounding line at protractor (degrees)*																									
	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35
1	.00	.01	.01	.01	.01	.01	.01	.01	.02	.02	.02	.02	.02	.03	.03	.03	.03	.04	.04	.04	.05	.05	.05	.06	.06	.07
2	.01	.01	.01	.02	.02	.02	.03	.03	.03	.04	.04	.05	.05	.05	.06	.06	.07	.08	.08	.09	.09	.10	.11	.12	.12	.13
3	.01	.02	.02	.03	.03	.03	.04	.04	.05	.06	.06	.07	.07	.08	.09	.10	.10	.11	.12	.13	.14	.15	.16	.17	.19	.20
4	.02	.02	.03	.03	.04	05	.05	.06	.07	.07	.08	.09	.10	.11	.12	.13	.14	.15	.16	.18	.19	.20	.22	.23	.25	.26
5	.02	.03	.04	.04	.05	.06	.06	.07	.08	.09	.10	.11	.12	.14	.15	.16	.17	.19	.20	.22	.24	.25	.27	.29	.31	.33
6	.03	.04	.04	.05	.06	.07	.08	.09	.10	.11	.12	.14	.15	.16	.18	.19	.21	.23	.24	.26	.28	.30	.33	.35	.37	.40
7	.03	.04	.05	.06	.07	.08	.09	.10	.11	.13	.14	.16	.17	.19	.21	.23	.24	.27	.29	.31	.33	.36	.38	.41	.43	46
8	.04	.05	.06	.07	.08	.09	.10	.12	.13	.15	.16	.18	.20	.22	.24	.26	.28	.30	.33	.35	.38	.41	.44	.47	.50	.53
9	.04	.05	.06	.08	.09	.10	.12	.13	.15	.17	.18	.20	.22	.24	.27	.29	.31	.34	.37	.40	.42	.46	.49	.52	.56	.59
10	.05	.06	.07	.08	.10	.11	.13	.15	.16	.18	.20	.23	25	.27	.30	.32	.35	.38	.41	.44	.47	.51	.54	.58	.62	.66
11	.05	.07	.08	.09	.11	.12	.14	.16	.18	.20	.22	.25	.27	.30	.33	.36	.38	.42	.45	.48	.52	.56	.60	.64	.68	.72
12	.06	.07	.09	.10	.12	.14	.15	.18	.20	.22	.24	.27	.30	.33	.36	.39	.42	.45	.49	.53	.57	.61	.65	.70	.74	.79
13	.06	.08	.09	.11	.13	.15	.17	.19	.21	.24	.27	.29	.32	.35	.38	.42	.45	.49	.53	.57	.61	.66	.71	.76	.81	.86
14	.07	.09	.10	.12	.14	.16	.18	.20	.23	.26	.29	.32	.35	.38	.41	.45	.49	.53	.57	.62	.66	.71	.76	.81	.87	.92
15	.07	.09	.11	.13	.15	.17	.19	.22	.25	.28	.31	.34	.37	.41	.44	.48	.52	.57	.61	.66	.71	.76	.82	.87	.93	.99
16	.08	.10	.12	.14	.16	.18	.20	.23	.26	.29	.33	.36	.40	.44	.47	.52	.56	.61	.65	.70	.76	.81	.87	.93	.99	1.05
17	.08	.10	.12	.14	.17	.19	.22	.25	.28	.31	.35	.38	.42	.46	.50	.55	.59	.64	.69	.75	.80	.86	.92	.99	1.05	1.12
18	.09	.11	.13	.15	.18	.20	.23	.26	.30	.33	.37	.41	.45	.49	.53	.58	.63	.68	.73	.79	.85	.91	.98	1.05	1.12	1.19
19	.09	.12	.14	.16	.19	.21	.24	.28	.31	.35	.39	.43	.47	.52	.56	.61	66	72	.78	.84	.90	.97	1.03	1.11	1.18	1.25
20	.10	.12	.14	.17	.20	.23	.26	.29	.33	.37	.41	.45	.50	.54	.59	.65	.70	.76	.82	.88	94	1.02	1.09	1.16	1.24	1.32

Water Supply Paper 2175, Vol. I, Table 6 on page 162 should be replaced by the following table:

TABLE 6.—*Air-correction table, giving difference, in meters, between vertical length and slant length of sounding line above water surface for selected vertical angles*

Vertical length (m)	Vertical angle of sounding line at protractor (degrees)																											
	5	8	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35
1	.00	.01	.02	.02	.02	.03	.03	.04	.04	.05	.05	.06	.06	.07	.08	.09	.09	.10	.11	.12	.13	.14	.15	.17	.18	.19	.21	.22
2	.01	.02	.03	.04	.04	.05	.06	.07	.08	.09	.10	.12	.13	.14	.16	.17	.19	21	.23	.24	.27	.29	.31	.33	.36	.38	.41	.44
3	.01	.03	.05	.06	.07	.08	.09	.11	.12	.14	.15	.17	.19	.21	.24	.26	.28	.31	.34	.37	.40	.43	.46	.50	.54	.58	.62	.66
4	.02	.04	.06	.07	.09	.11	.12	.14	.16	.18	.21	.23	.26	.28	.31	.35	.38	.41	.45	.49	.53	.57	.62	.67	.72	.77	.82	.88
5	.02	.05	.08	.09	.11	.13	.15	.18	.20	.23	.26	.29	.32	.36	.39	.43	47	.52	.56	.61	.66	.72	.77	.83	.90	.96	1.03	1.10
6	.02	.06	.09	.11	.13	.16	.18	.21	.24	.27	.31	.35	.39	.43	.47	.52	.57	.62	.68	.73	.80	.86	.93	1.00	1.08	1.15	1.24	1.32
7	.03	.07	.11	.13	.16	.18	.21	.25	.28	.32	.36	.40	.45	.50	55	.60	.66	.72	.79	.86	.93	1.00	1.08	1.17	1 25	1.35	1.44	1.55
8	.03	.08	.12	.15	.18	.21	.24	.28	.32	.37	.41	.46	.51	.57	.63	69	.76	.83	.90	.98	1.06	1.15	1.24	1.33	1.43	1.54	1.65	1.77
9	.03	.09	.14	.17	.20	.24	.28	.32	.36	.41	.46	.52	.58	.64	.71	.78	.85	.93	1.01	1.10	1.19	1.29	1.39	1.50	1.61	1.73	1.86	1.99
10	.04	.10	.15	.19	.22	.26	.31	.35	.40	.46	51	.58	.64	.71	.79	.86	.95	1.03	1.13	1.22	1.33	1.43	1.55	1.67	1.79	1.92	2.06	2.21
11	.04	.11	.17	.21	.25	.29	.34	.39	.44	.50	.57	.63	.71	.78	.86	.95	1.04	1.14	1.24	1.35	1.46	1.58	1.70	1.83	1.97	2.12	2.27	2.43
12	.05	.12	.19	.22	.27	.32	.37	.42	.48	.55	.62	.69	.77	.85	.94	1.04	1.14	1.24	1.35	1.47	1.59	1.72	1.86	2.00	2.15	2.31	2.47	2.65
13	.05	.13	.20	.24	.29	.34	.40	.46	.52	.59	.67	.75	.83	.92	1.02	1.12	1.23	1.34	1.46	1.59	1.72	1.86	2.01	2.17	2.33	2.50	2 68	2.87
14	.05	.14	.22	.26	.31	.37	.43	.49	.56	.64	.72	.81	.90	1.00	1.10	1.21	1.32	1.45	1.58	1.71	1.86	2.01	2.17	2.33	2.51	2.69	2.89	3.09
15	.06	.15	.23	.28	.34	.39	.46	.53	.60	.69	.77	.86	.96	1.07	1.18	1.30	1.42	1.55	1.69	1.83	1.99	2.15	2.32	2.50	2.69	2.89	3 09	3.31
16	.06	.16	.25	.30	.36	.42	.49	.56	.64	.73	.82	.92	1.03	1.14	1.26	1.38	1.51	1.65	1.80	1.96	2.12	2.29	2.48	2.67	2.87	3.08	3 30	3.53
17	.06	.17	.26	.32	.38	.45	.52	.60	.69	.78	.87	.98	1.09	1.21	1.34	1.47	1.61	1.76	1.91	2.08	2.25	2.44	2.63	2.83	3.05	3.27	3.51	3.75
18	.07	.18	.28	.34	.40	.47	.55	.63	.73	.82	.93	1.04	1.16	1.28	1.41	1.55	1.70	1.86	2.03	2.20	2.39	2.58	2.78	3.00	3.23	3.46	3.71	3.97
19	.07	.19	.29	.36	.42	.50	.58	.67	.77	.87	.98	1.09	1.22	1.35	1.49	1.64	1.80	1.96	2.14	2.32	2.52	2.72	2.94	3.17	3.40	3.65	3.92	4.19
20	.08	.20	.31	.37	.45	.53	.61	.71	.81	.91	1.03	1.15	1.28	1.42	1.57	1.73	1.89	2.07	2.25	2.45	2.65	2.87	3.09	3.33	3.58	3.85	4.12	4.42