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U.S. DEPARTMENT OF COMMERCE / National Bureau of Standards

A Guide to Methods and Standards for the Measurement of Water Flow

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A Guide to Methods and Standards for the Measurement of Water Flow

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PREFACE

Recent water pollution control legislation and the need for a closer accounting of the disposition of our water resources have produced a need for more and better field measurements of water flow. Wastewater flow criteria and requirements in particular are now involving many engineers and technicians who have had little past experience in flow measurement. Getting information into the hands of those who need and use it is one of the major thrusts of the measurements programs at the National Bureau of Standards. To that end this guidebook has been compiled to provide convenient access to useful information on good flow measurement practice and error sources for those people who have direct responsibility for field flow measurements.

The guidebook is organized in chapters according to measuring instruments or methods. The general information sources listed in each chapter combined with the technical references cited at the end of each chapter are in no sense intended to form an exhaustive bibliography on each subject. Other publications exist to fill that need.

It is intended that this document be frequently updated and improved to increase its usefulness to readers. For that purpose readers' comments and suggestions are invited.

GUIDE TO METHODS AND STANDARDS FOR THE
MEASUREMENT OF WATER FLOW

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GUIDE TO METHODS AND STANDARDS FOR THE MEASUREMENT OF WATER FLOW

Gershon Kulin and Philip R. Compton

Selected information sources on methods and standards for making measurements of water and wastewater flow in the field are listed and described. Both closed conduit and free surface flows are treated, but emphasis is on open channel flow measurements needed in water resource engineering and in water pollution control. Instruments and methods covered include weirs, flumes, current meters (and velocity traverse methods), dilution techniques, pipe flow instruments, acoustic meters and others. In addition to summarizing the basic properties of each instrument or method and referring users to the best available sources of detailed information on performance and field application, potential sources of error are described and quantified where possible.

Key words: Flow measurement, water; instruments, flow measurement; open channel flow measurement; pipe flow measurement; standards, flow measurement.

1. INTRODUCTION

Population density and advanced technology continue to place increasing demands upon the nation to conserve its supply of water and to control the quality of its water. Recent legislation and a heightened public interest in conservation and environmental matters have emphasized the importance of water velocity and flowrate measurements conducted in the field. Uniform and reliable measurement data are needed in order to identify the resource levels and quality of bodies of water; determine the results of conservation and quality control efforts; and enforce water conservation and quality regulatory requirements.

1.1. Purpose

The information presented in this guidebook is intended to assist measurement supervisors and their personnel engaged in establishing and operating water and wastewater flow measurement stations in the field. It is geared to technical people who are not necessarily fluid mechanics specialists.

Several good flow measurement handbooks are now available, including the Bureau of Reclamation's Water Measurement Manual and the U. S. Geological Survey series on Techniques of Water Resources Investigations. However, no single source provides all of the information needed by workers in the field, and the major purpose of this guidebook is to identify and describe the most useful sources in one document. Also, flow measurement references frequently do not point out potential errors and pitfalls nor attempt to quantify them. It is expected that the comments given here will prove useful to readers in that regard.

1.2. Scope

Information on flow measurements in pressure conduits as well as in open channels is included in this publication. However, the open channel measurements have been emphasized for two reasons. First, information on the traditional pipe flow measuring instruments, e.g., orifice and venturi meters, generally appears to be more widely available in popular sources than corresponding open channel information; and second, free surface flows are so frequently encountered in water pollution and water resource flow measurement. Within the context of open channel measurements, the traditional instruments and methods have been emphasized rather than newer, more sophisticated devices, because the majority of users will have only the traditional means available to work with, at least in the near future.

Although the information provided herein will be useful for many water flow applications, some emphasis has been given to problems peculiar to wastewater measurements. The reason is that recent legislation in this field is bringing about the involvement of many workers who are relative newcomers to flow measurement activity.

1.3. Approach

Each instrument or method is introduced with a brief description of its operation or utilization. In some cases this description contains what appears to be all the information needed to make measurements. For example, it is difficult to describe thin-plate weirs without listing some of the "standard" equations and geometries in use. Nevertheless, this document is intended not as a self-contained measurement manual but as a guidebook to sources of detailed information. Good measurements (or, if "good" measurements are impossible, an appreciation of the errors involved in the best possible measurements under the circumstances) depend upon attention to published detail, appreciation for the subtleties of the flow and measurement process and the experience of others.

The general information sources listed in each section of this guidebook are intended to provide these details. The references at the end of each chapter, on the other hand, provide technical backup material which, although useful, is not necessarily of as much immediate importance to the user as that contained in the general information sources.

The compilers of this guidebook have tried to provide enough ordering information to make the general information sources as accessible as possible. We have assumed that most field users will not have rapid or convenient access to a large technical library. No foreign language sources are used, and no foreign English-language general information sources are cited unless they can be conveniently obtained from domestic sources without undue delay.

1.4. Evaluation of the Uncertainties in Practical Measurements

Section 1.4 has been contributed by J. M. Cameron, Chief, Office of Measurement Services, Institute for Basic Standards, National Bureau of Standards.

1.4.1. General Remarks

A flow measuring device, once installed, will be called upon to make a sequence of measurements throughout its useful life. It is the measurements in this sequence for which one needs assurance that errors are suitably small. What is needed is not merely a description of the properties of the instrument, but rather of the measurement process generated by that instrument. The properties of this process will depend on many factors in addition to the instrument - errors in installation, properties of the fluid involved, in some cases on operator technique, etc. One also needs continuing evidence that the process is "in control."

These questions could easily be answered if one had a much more accurate measuring method available and could periodically check the measurements. One could study the differences between the two methods and make statements about the offset of one relative to the other and describe the distribution of the observations about that offset point. The size of the systematic offset and the scatter of the values might turn out to be a function of flow rate, contaminants in the fluid, and a host of other factors. However, the ability to "sample" the output enables one to arrive at whatever degree of knowledge of the process is needed or is economically feasible.

Next best would be the presence of redundancy in the form of a second instrument, but most often one has to rely on indirect evidence such as that given by the day-to-day consistency of the flow being measured. In this latter case one might be satisfied with his knowledge of the performance of the device with respect to random variations but one does not have any direct measure of the offset due to systematic errors in the system. One therefore desires to set bounds to the possible extent of these systematic effects as well as having an error limit for those statistical variations that appear to be random in nature.

1.4.2. Evaluation of Systematic Offsets in the Measurements

For each measurement method a number of possible sources of systematic error can be listed. These fall into several general classes such as: (a) those related to departures from ideal geometry or correct installation; (b) those related to the approximating equation or calibration used; (c) those related to the velocity and flow characteristics of the fluid; (d) those related to impurities and contamination in the fluid; and (e) those related to wear, aging or other changes in instrument response.

The ideal method of setting bounds for the possible bias remaining after known corrections for the factor have been made, is that of measuring the deviation resulting from introducing a change corresponding to the maximum expected in practice. For example, the geometry can be varied, the properties of the liquid changed or procedural changes introduced as a part of the regular measurement. The resultant changes constitute bounds to the effect from the source being varied.

In many cases one knows only that a certain parameter is correct to within some percentage value. Often one can compute the effect such an uncertainty will have on the results. This sort of bound is of unknown quality because the formula by which it is computed does not take account of all the possible effects of the variable involved.

One has the problem of how to combine the various limits of error. Unless one has experiment or theory to determine the effect of joint variation of two or more factors, the correct method of combination will not be known. Until the bounds can be verified by experiment, arguments over whether a sum of absolute values or a root sum of squares is better cannot be settled. One should list all error bounds and the rule of combination used to arrive at the bound to be used for the limit to the offset or systematic error of measurements made by the process.

1.4.3. Random Variation

Even if one could reproduce the same flow again and again it is characteristic of measurement processes that they would not give the same results. The random variation so exhibited is a characteristic of the instrument and flow condition and may depend on rate, fluid involved, etc. An umpire method, or a second instrument enables one to estimate this random variation directly. However in most cases one cannot reproduce the flow exactly so that the random variation includes a component from variations in the flow itself.

For a proper characterization of instrument performance one needs a bound for the effect of all the components of random error in the process. This variation is usually expressed by a standard deviation which can be associated with the values from the instrument, and the limit is taken to be three (some people use two) times this standard deviation.

1.4.4. Total Uncertainty in the Measurement

A single value reported out by the process will be uncertain by an amount that depends on the sum of the systematic offset and the random error which occurs. The average of several measurements will reduce the effect of the random errors by a factor depending on the square root of the number of measurements, but the systematic error in the average will not be reduced at all. For this reason, one should state both elements of the total uncertainty and report the sum as the possible uncertainty.

1.4.5. Units

Conventional units have been used throughout this publication, mainly because these are still the units most often used by those concerned with water and wastewater flow measurements. However, because of the increasing importance of International Standard (SI) units, the factors for conversion from conventional to SI units are listed in Appendix A for those units used in this document.

2. PARSHALL FLUMES

2.1. Background

The Parshall flume was developed in the late 1920's primarily to measure irrigation water but it is now frequently used in sewers and sewage treatment plants as well. The flume shape and standard sizes are shown in figure 2.1. Although it is operable with the crest overfall either free or submerged*, it is highly desirable to select and place the flume so that the overfall is free. The reason is that in free flow the flowrate can be deduced from a single upstream depth measurement, while in submerged flow the downstream depth has to be obtained also.

The head-discharge equations for free flow are of the form

$$Q = CH^n \qquad [2.1]$$

where Q is the discharge in cubic feet per second and H is the depth in feet measured at the location "a" in figure 2.1. Values of C and n are given in table 2.1. The free flow discharges computed from eq. 2.1 and table 2.1 can be found in tabular or graphical form in many of the references cited in this chapter.

The flow becomes submerged when the downstream depth (measured at "b" in figure 2.1) attains a certain critical percentage of the upstream depth as given in table 2.2. In the 1, 2 and 3 inch flumes it has been found that a more reliable downstream head measurement can be made at "c" (see figure 2.1) than at "b." The relationship between H_c and H_b is given in a curve in ref. 2.3.1.

Curves for submerged flow, for which eq. 2.1 and table 2.1 are not valid, are available in refs. 2.3.1, 2.3.2, 2.3.4 and 2.3.5. Reference 2.3.3 has submerged flow curves for small flumes only. The effect of submergence on the discharge is shown approximately in figure 2.2. Actually the percentage flow reduction varies slightly over the discharge range for each flume size. (See also section 2.4.7.) Figure 2.2 is included mainly to give users an idea of submergence effects; to achieve accuracy the more detailed submerged flow curves in the general information sources of section 2.3 should be consulted.

2.2 Uses

The principal advantages of the Parshall flume are its capability for self-cleansing (particularly when compared with sharp-edged weirs) and its relatively low head loss. These characteristics of the Parshall flume make it particularly suitable for flow measurement in irrigation canals, in certain natural channels, and in sewers. An important disadvantage is that it is not amenable to theoretical treatment, so that the dimensions of figure 2.1 must be closely adhered to if the empirical information of table 2.1 is to be valid. See also section 2.4.4.

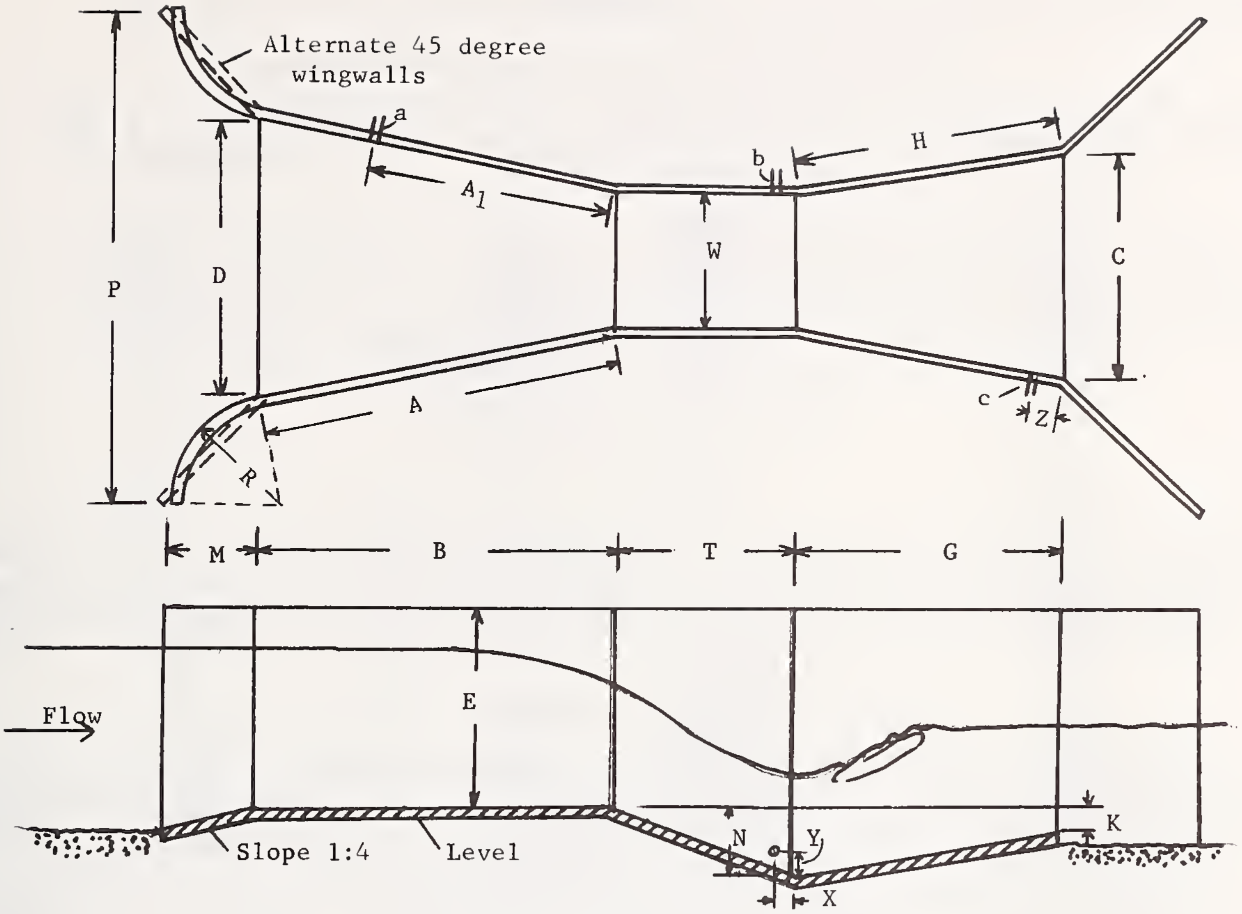
2.3. General Information Sources

2.3.1. "Water Measurement Manual," Bureau of Reclamation, U. S. Dept. Interior, Second Ed., Revised Reprint, 1974, Ch. 3, pp. 43-88. Order from U. S. Govt. Printing Office, Washington, D. C. 20402. $5.80. Cat. No. I27.19/2:W29/2, Stock No. S/N 2403-0027.

This chapter is a fairly comprehensive treatment of the Parshall flume and is particularly strong on flume selection and placement to avoid submergence. Head loss data is also given.

2.3.2. "Measurement of Irrigation Water," Section 15, Chap. 9 of National Engineering Handbook, U. S. Dept. Agriculture, Soil Conservation Service, 1962. Order from

*See Appendix for definition of terms.



W		A		A_1		B		C		D		E		T		G		H		K		M		N		P		R		X		Y		Z	
FT.	IN.	FT.	IN.	FT.	IN.	FT	IN	FT.	IN.	FT.	IN.	FT.	IN.	FT.	IN.	FT	IN	FT.	IN	FT.	IN	FT.	IN	FT.	IN.	FT.	IN.	FT.	IN	FT.	IN.	FT	IN.	FT.	IN
0	1"	1	2 9/32	0	9 17/32	1	2	0	3 21/32	0	6 19/32	0	6 to 9	0	3	0	8	0	8 1/8	0	3/4	–	–	0	1 1/8		–	–		0	5/16	0	1/2	0	1/8
	2"	1	4 5/16		10 7/8	1	4		5 5/16		8 13/32		6 to 10		4 1/2		10		10 1/8		7/8		–		1 11/16		–	–			5/8		1		1/4
	3"	1	6 3/8	1	1/4	1	6		7		10 3/16	1 to 1 1/2			6	1	0	1	5/32		1	–			2 1/4		–	–			1		1 1/2		1/2
0	6	2	7/16	1	4 5/16	2	0	1	3 1/2	1	3 5/8	2	0	1	0	2	0	–		0	3	1	0	0	4 1/2	2	11 1/2	1	4	0	2	0	3	–	
	9	2	10 5/8	1	11 1/8	2	10	1	3	1	10 5/8	2	6	1	0	1	6	–			3	1	0		4 1/2	3	6 1/2	1	4		2		3	–	
1	0	4	6	3	0	4	4 7/8	2	0	2	9 1/4	3	0	2	0	3	0	–			3	1	3		9	4	10 3/4	1	8		2		3	–	
1	6	4	9	3	2	4	7 7/8	2	6	3	4 3/8	3	0	2	0	3	0	–			3	1	3		9	5	6	1	8		2		3	–	
2	0	5	0	3	4	4	10 7/8	3	0	3	11 1/2	3	0	2	0	3	0	–			3	1	3		9	6	1	1	8		2		3	–	
3	0	5	6	3	8	5	4 3/4	4	0	5	1 7/8	3	0	2	0	3	0	–			3	1	3		9	7	3 1/2	1	8		2		3	–	
4	0	6	0	4	0	5	10 5/8	5	0	6	4 1/4	3	0	2	0	3	0	–			3	1	6		9	8	10 3/4	2	0		2		3	–	
5	0	6	6	4	4	6	4 1/2	6	0	7	6 5/8	3	0	2	0	3	0	–			3	1	6		9	10	1 1/4	2	0		2		3	–	
6	0	7	0	4	8	6	10 3/8	7	0	8	9	3	0	2	0	3	0	–			3	1	6		9	11	3 1/2	2	0		2		3	–	
7	0	7	6	5	0	7	4 1/4	8	0	9	11 3/8	3	0	2	0	3	0	–			3	1	6		9	12	6	2	0		2		3	–	
8	0	8	0	5	4	7	10 1/8	9	0	11	1 3/4	3	0	2	0	3	0	–			3	1	6		9	13	8 1/4	2	0		2		3	–	
10	0	–		6	0	14	0	12	0	15	7 1/4	4	0	3	0	6	0	–		0	6	–		1	1 1/2	–		–		1	0	0	9	–	
12	0	–		6	8	16	0	14	8	18	4 3/4	5	0	3	0	8	0	–			6	–		1	1 1/2	–		–		1	0		9	–	
15	0	–		7	8	25	0	18	4	25	0	6	0	4	0	10	0	–			9	–		1	6	–		–		1	0		9	–	
20	0	–		9	4	25	0	24	0	30	0	7	0	6	0	12	0	–		1	0	–		2	3	–		–		1	0		9	–	
25	0	–		11	0	25	0	29	4	35	0	7	0	6	0	13	0	–		1	0	–		2	3	–		–		1	0		9	–	
30	0	–		12	8	26	0	34	8	40	4 3/4	7	0	6	0	14	0	–		1	0	–		2	3	–		–		1	0		9	–	
40	0	–		16	0	27	0	45	4	50	9 1/2	7	0	6	0	16	0	–		1	0	–		2	3	–		–		1	0		9	–	
50	0	–		19	4	27	0	56	8	60	9 1/2	7	0	6	0	20	0	–		1	0	–		2	3	–		–		1	0		9	–	

Figure 2.1. Parshall flume dimensions, from ref. 2.3.1.

Table 2.1

Free Flow Values of C and n for Parshall Flumes
(See Equation 2.1)

Flume Throat, W	C	n	Max. Q, cfs
1 in	0.338	1.55	0.2
2 in	0.676	1.55	0.5
3 in	0.992	1.55	1.1
6 in	2.06	1.58	3.9
9 in	3.07	1.53	8.9
1 ft	4W (*)	$1.522W^{0.026}$	16.1
1.5 ft	"	"	24.6
2 ft	"	"	33.1
3 ft	"	"	50.4
4 ft	"	"	67.9
5 ft	"	"	85.6
6 ft	"	"	103.5
7 ft	"	"	121.4
8 ft	"	"	139.5
10 ft	39.38	1.6	200
12 ft	46.75	1.6	350
15 ft	57.81	1.6	600
20 ft	76.25	1.6	1000
25 ft	94.69	1.6	1200
30 ft	113.13	1.6	1500
40 ft	150.00	1.6	2000
50 ft	186.88	1.6	3000

(*) W in feet

U. S. Govt. Printing Office, Wash., D. C. 20402. $1.10. Cat. No. A57.6/2:En3/sec 15, Ch. 9, S/N 0107-00708.

The section of this document dealing with Parshall flumes covers roughly the same material as ref. 2.3.1 above.

2.3.3. ASTM Standard D1941-67, "Standard Method for Open Channel Flow Measurement of Industrial Water and Industrial Waste Water by the Parshall Flume." Order from American Society for Testing and Materials, 1916 Race St., Philadelphia, Pa. 19103. $1.50.

This standard gives a brief treatment of the Parshall flume. Useful details on stilling well design for head measurement are included. Information is given only on flumes up to 8 ft in throat width.

2.3.4. "Design and Calibration of Submerged Open Channel Flow Measurement Structures, Part 2 -- Parshall Flumes," G. V. Skogerboe, M. L. Hyatt, J. D. England and J. R. Johnson, Rep. No. WG31-3, Utah Water Research Lab., 1967. Order from Utah Water Res. Lab., Utah State Univ., Logan, Utah 84321. $0.25.

This report has details on flume use under submerged flow.

2.3.5. "Stevens Water Resources Data Book," First Edition, Leupold & Stevens, Incorporated, P. O. Box 688, Beaverton, Ore. 97005. $4.00.

This source is particularly useful for evaluating error sources in head measurements for flumes and weirs using float actuated gages. Errors related to internal friction, float size, humidity effects on chart paper, etc., are covered.

2.3.6. Davis, S., "Unification of Parshall Flume Data," Proc. Amer. Soc. Civ. Eng., 87, IR4, Dec. 1961, pp. 13-26. Order from Engineering Societies Library, 345 E. 47th St., New York, N. Y. 10017. $0.25 per page plus $3.00.

2.3.7. In addition to the sources cited above, several other well-known hydraulic source books, e.g., reference 2.8.1, list the "standard" flume dimensions and give the head-discharge relations of section 2.1 above in either tabular or graphical form. The original sources of virtually all of the data are the publications of Parshall (2.8.2, 3, 4) with data on the smallest flumes added by Robinson (2.8.5).

2.4. Error Sources in Flume Use

2.4.1. General

As is true for all flow measuring devices, the accuracy of the result depends upon the performance of the instrument treated as a system -- in this case (a) the primary element (flume), (b) the secondary element (depth sensor and recorder), and finally (c) the manner in which these elements are installed and maintained. The following sections deal with these three aspects.

2.4.2. The "Standard" Head-Discharge Relations

The published head-discharge relations which are given in section 2.1 above and in sources such as refs. 2.3.1, 2, 3, 4 are apparently all based on the original experiments of Parshall (2.8.2, 3, 4). However, there might be uncertainties in these relations of up to 5 percent; this is recognized by the ASTM standard (2.3.3). For example, extensive data by Blaisdell (2.8.6) indicate differences of up to 6 percent from the "standard" relations. It is therefore imperative that flumes be calibrated in place if accuracy of the order of 2 percent in the head-discharge relation is desired.

2.4.3. Errors in Head Measurement

From eq. 1.1 and table 2.1 it is seen that the percentage error in head measurement is multiplied by approximately 1.5 (the value of n) to determine its contribution to the percentage error in flowrate. Accurate head measurements are therefore particularly important to the overall accuracy of flow rates determined with flumes and weirs.

Some errors associated with operation of float actuated gages and recorders are treated in ref. 2.3.5. As an illustration to alert users to the potential for these errors, a system with a 6-inch diameter float and requiring a 2-ounce force to move it can have a float-lag error as high as 0.02 ft of head. At low heads this error is significant. For example, the flowrate error in a flume operating at a head of 0.5 ft would be about 6 percent. Errors from this source can be reduced by using larger floats and by minimizing internal friction. No error information on scow float performance is available.

Small systematic errors in head measurement caused by failure to accurately reference the gage zero to the crest elevation can be similarly damaging to accuracy. See also comments on level in section 2.4.4.

The foregoing errors in head measurements do not include all of the systematic and random errors which could actually occur on site. See Introduction and section 2.4.8 for further comments on this subject.

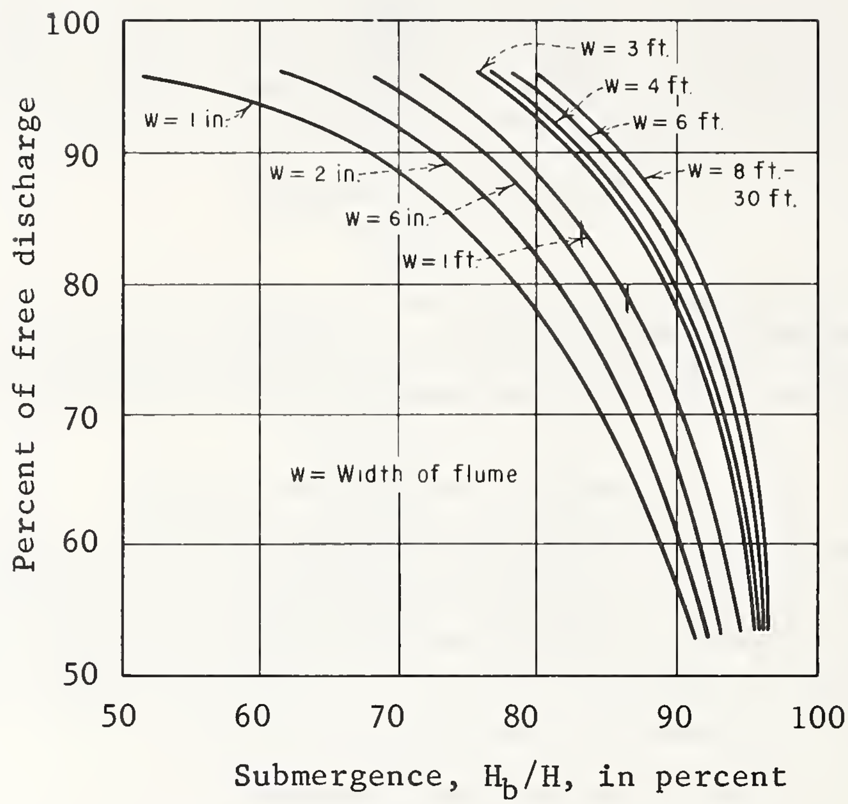


Figure 2.2. Approximate submergence effect on Parshall flume discharge, from ref. 2.3.1.

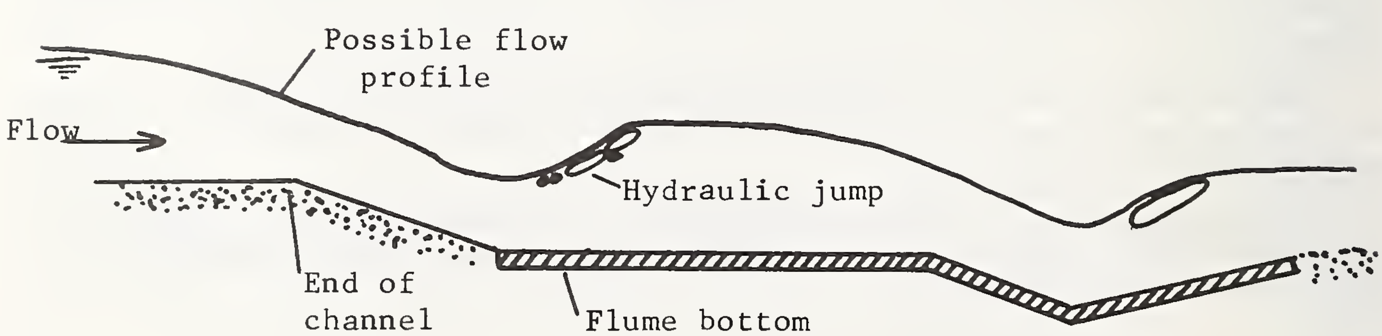


Figure 2.3. Potential hydraulic jump development in a Parshall flume.

2.4.4. Departures from "Standard" Geometry

If the standard head-discharge relations are to be used, the standard flume dimensions must be strictly adhered to. For the smallest flume sizes, tolerances of 1/64 inch have been suggested for the throat, with 1/32 inch tolerance elsewhere (2.3.1). In a 1-inch flume a throat width difference of even 1/64 inch will cause a 1.5 percent error unless corrected for as in the following paragraph.

Width. If the throat width differs slightly from the standard dimension, it has been recommended (2.3.1) that the standard flowrate be changed proportionately, e.g., if the throat of a 1 ft flume is 1/8" too narrow, the flowrates in the standard tables should be reduced by 1 percent at each value of H. This correction should be applied with discretion, because there are no guidelines yet as to how large the departure from the standard throat dimension can be before this correction procedure becomes invalid. Probably it should be restricted to throat width changes of a few percent only.

No guidelines presently exist for adjusting the discharge when the width at the measuring section differs from the recommended width. Accuracy of this dimension is usually not as critical as that of the throat width.

For flumes which are not of standard dimensions but which have the correct sidewall convergence angle, and for flumes in which only the location of the depth measurement is incorrect, corrections can be made by the method of Davis (2.3.6).

Level. No guidelines exist for adjustment of head-discharge curves of flumes which are not level longitudinally. If the flume is slightly out of level transversely, we can only use the standard H-Q curves provided an average H is used. Caution: A staff gage mounted on the flume sidewall at the measuring station will give an incorrect head reading if the flume tilts laterally, with the direction of the error depending on whether the gage is mounted on the "deep" or "shallow" side.

Length. The converging section of the flume (length B in figure 2.1) cannot be shortened without affecting the performance (2.8.7). There is no discernible effect if it is lengthened, but the head-measuring station should remain at the standard distance from the brink. There is reportedly no difference in performance between the curved and straight entrance wingwalls of figure 2.1. However, the curved entrance appears to insure smooth flow at the head-measuring station.

2.4.5. Velocity Distribution

The entrance transitions and converging sidewalls of the Parshall flume provide a small favorable pressure gradient which tends to straighten the velocity distribution. However, this gradient is too small to adjust grossly distorted velocity profiles in the approach flow and if these are permitted to exist, the flume will not produce the standard performance (2.3.1). Therefore situations which distort the entering flow should be avoided. Examples are:

- Placement of flume immediately downstream of a bend, junction or turnout without allowing sufficient flow straightening length.

- Discharging from a narrower conduit into a wider flume entrance without providing enough entrance length for the flow to become evenly distributed across the section. Construction of a transition section (see, e.g., ref. 2.8.1) can be helpful here if available length is restricted.

Comparatively small changes in velocity distribution, such as those associated with change in roughness of an artificial channel, probably do not affect measurements significantly, at least in small flumes. However, more research is needed on this subject to provide a basis for a more definitive statement.

The velocity distribution is usually expressed in terms of a velocity distribution coefficient, α, (see Appendix) which would have a value of unity for a completely uniform distribution. For turbulent flow in relatively smooth channels, α is close to unity, typically around 1.05. Typical values for various field situations can be found in other references, such as 2.8.1.

In the absence of substantial experimental evidence on the ability of the Parshall flumes to flatten nonuniform velocity distributions, it is best to avoid high values of α by providing a long and regular entrance channel where possible. Unfortunately no published recommendations on minimum lengths are available. Pending further research, only in-place calibrations can inspire confidence in H-Q curves where conditions are such that distorted velocity distributions are expected in the approach flow.

2.4.6. Other Entrance Effects

Supercritical Flow. The Parshall flume should not be placed at a lower elevation than that of the channel being measured, because of the danger of inducing supercritical flow and a hydraulic jump within the flume. See figure 2.3. One flume so installed, in which a diagonal jump formed near the float gage, was found to be under-registering by amounts up to 40 percent at low flows.

When the channel is steep and flow is always supercritical, a check should be made using hydraulic jump principles (see, e.g., ref. 2.8.1) to insure that the jump occurs sufficiently far upstream of the flume to permit smoothing of the flow. Reference 2.8.8 recommends a minimum distance of 20 channel widths upstream.

Weak hydraulic jumps with upstream Froude numbers between 1.2 and 1.7 should in particular be avoided near the flume. These jumps are characterized by long-lasting standing waves downstream and the flume depth measurement will be erroneous. Stilling wells cannot average out the depths associated with stationary waves.

2.4.7. Submerged Flow

It is highly desirable that flumes be installed so that they operate in free flow. The ratio between downstream depth, H_b, (see figure 2.1) to upstream depth H, at which submergence begins to affect the upstream conditions is given in table 2.2 below for the various flume sizes.

Table 2.2

Critical Submergence for Parshall Flumes

(Data from Ref. 2.3.4)

Flume Size	Critical Submergence	Flume Size	Critical Submergence
1 in	0.56	3 ft	0.68
2 in	0.61	4 ft	0.70
3 in	0.64	5 ft	0.72
6 in	0.55	6 ft	0.74
9 in	0.63	7 ft	0.76
1.5 ft	0.64	10 to 50 ft	0.80
2 ft	0.66		

Even submergence ratios slightly in excess of the critical ratios of table 2.2 can introduce significant errors if they are not taken into account by use of submergence curves in, e.g., ref. 2.3.1. This is shown in figure 2.4 for a 1 ft flume where errors as large as 10 percent occur at low heads for uncorrected submergence of 70 percent. Even at the higher flows the error exceeds 3 percent for 70 percent submergence. At high submergence the error becomes intolerable for most applications.

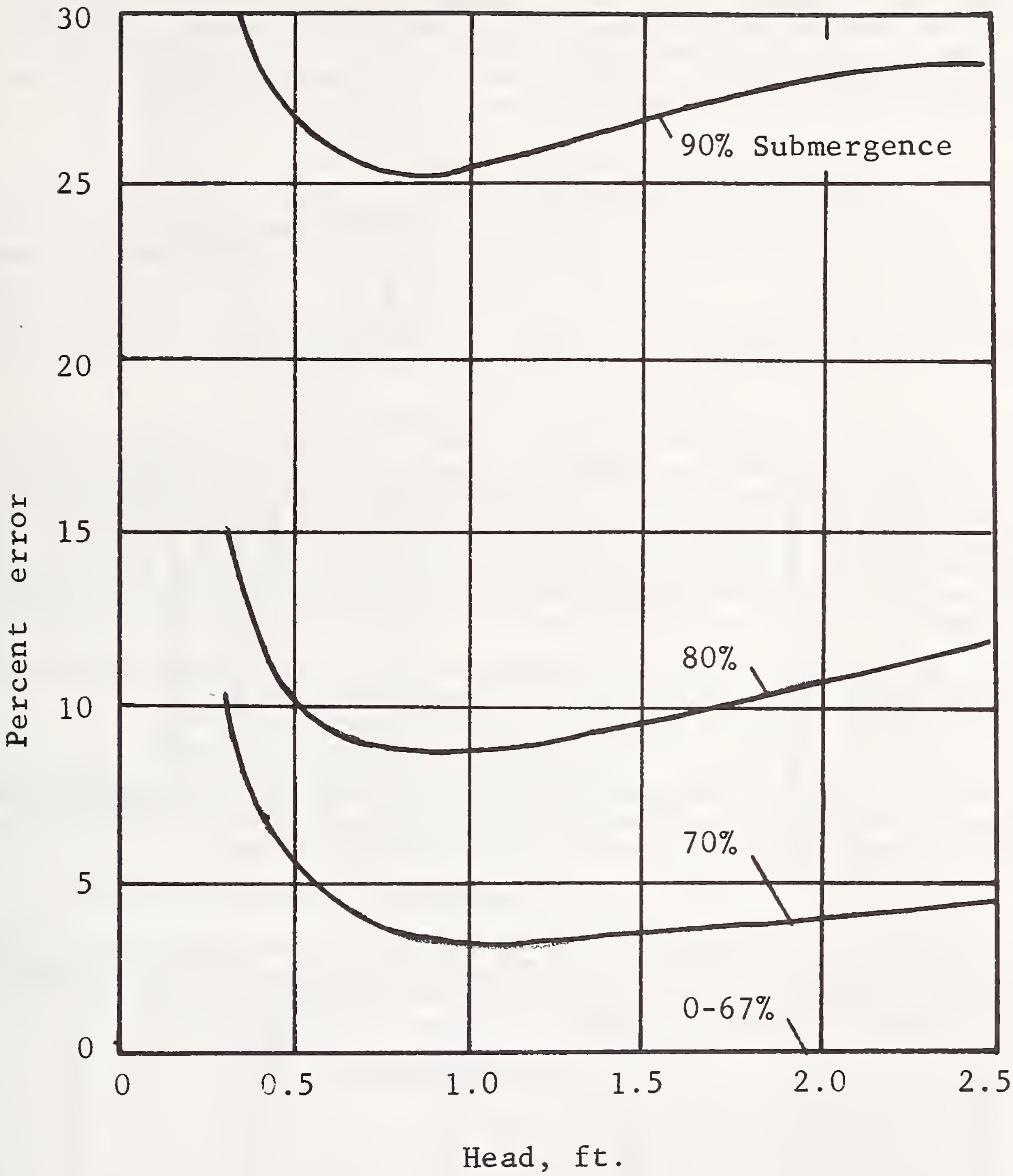


Figure 2.4. Errors in 1-ft Parshall flume measurements if uncorrected for submergence.

Even when the submergence correction is properly made, a small error can be introduced, particularly at large submergence, if the depth measurements are in error. Consider a 1-ft flume with $H = 1.00$ ft and $H_b = 0.85$ ft, so that the nominal submergence is 85 percent. But if each of the two depth measurements is uncertain by as much as $\pm$ 1 percent, then the actual submergence ratio could be (although it is not likely to be) as high as 86.7 percent or as low as 83.3 percent. From figure 2.2 or the curves of, e.g., ref. 2.3.1, this corresponds to a discharge spread of 5 percent. This example (in which any uncertainty in the correction curves themselves has been ignored for lack of information) illustrates the desirability of designing flume installations for free flow.

2.4.8. Combined (System) Error

The foregoing discussions of error sources can help the user to estimate some, but not all, of the error components which contribute to the total error of measurement. Other errors, particularly those relating to installation, maintenance, aging and wear, cannot be estimated without some sort of in place check on the performance, as described in the Introduction. Nevertheless, in selecting or evaluating a new system that is to be, or has been, installed in accordance with good practice, the user can still try to make a useful estimate of the total system error in advance of installation or of in-place rating. However, it is important to recognize the limitations of such an estimate.

As an illustrative example, consider a 1-ft Parshall flume that is constructed and installed according to recommended practices. Taking into account the discussion of errors in section 2.4.2, the user might still want to assign a possible error of, say, 3 percent to the flowrates published in the Parshall flume tables. The user can also estimate how closely he was able to reference the head zero to the crest elevation. Suppose for the sake of illustration that this referencing is within 0.02 ft; how this will change with use cannot be estimated. Suppose also that from information in ref. 2.3.5 or other sources, the user estimates the head measuring instrument to be accurate within 0.01 ft; again the effects of use on this figure cannot be estimated in advance.

One way of computing a combined estimated error from the foregoing components is by the square root of the sum of the squares, so that in view of eq. 2.1

$H = 2.0$ ft: Error $= [(0.03)^2 + (1.522)^2[(0.01)^2 = (0.005)^2]]^{\frac{1}{2}} = 3.4$ percent

$H = 0.5$ ft: Error $= [(0.03)^2 + (1.522)^2[(0.04)^2 + (0.02)^2]]^{\frac{1}{2}} = 7.4$ percent

Again, the above illustrative results include only those sources of systemmatic error that can be reasonably estimated for conditions at the outset. Systemmatic errors that develop in place and random errors cannot be accounted for in this way.

2.5. Maintenance

Although the Parshall flume was originally developed to pass moderate sediment loads, heavy debris is known to settle out and affect the performance of the meter. Periodic cleanouts are therefore recommended for sewage meters. Periodic calibrations of float gage, scow float and recorder are needed, as well as checks on flume level.

2.6. Field Calibration

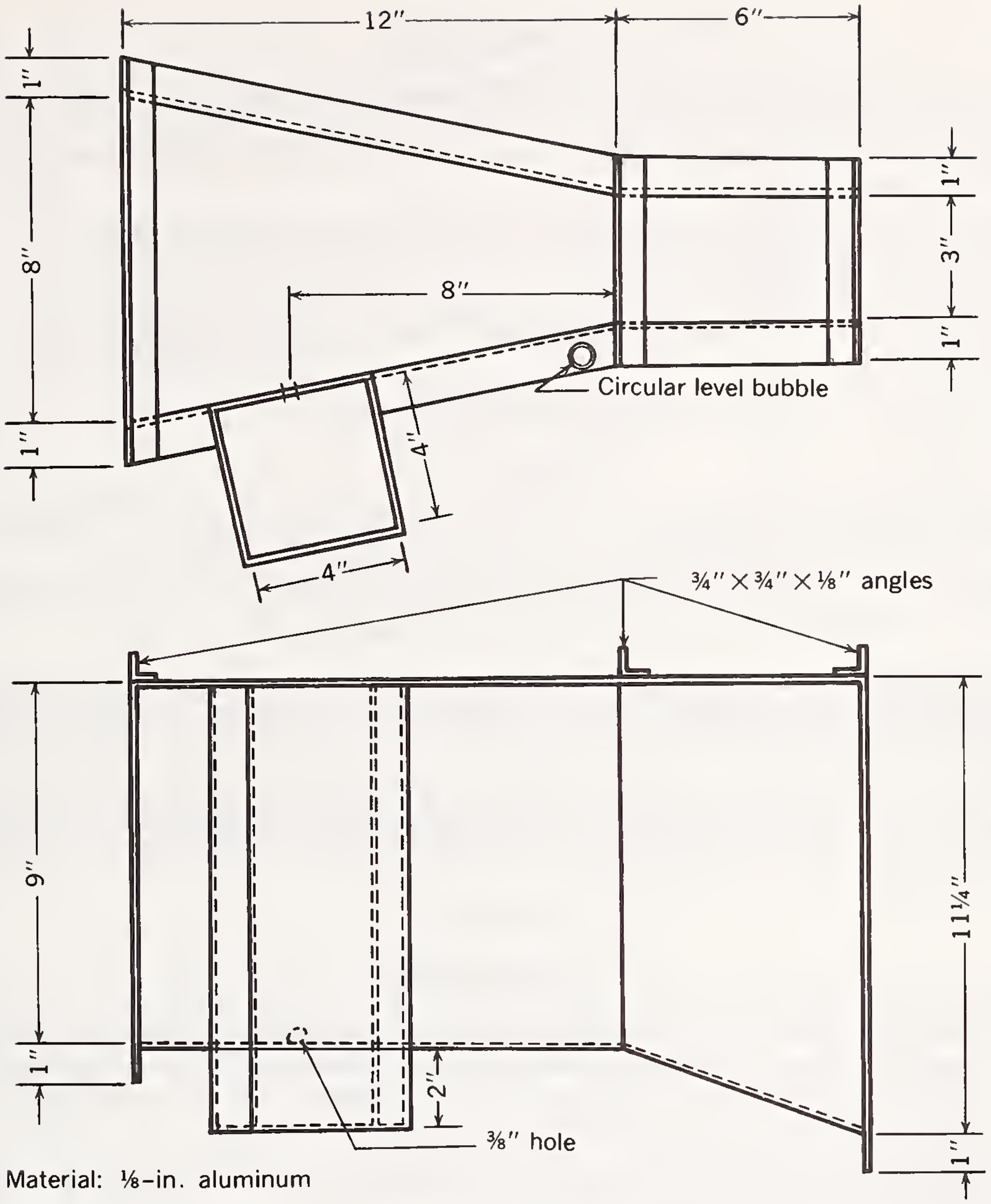
1. By current meter traverse, see Chapter 8.

2. By dilution methods, see Chapter 9.

3. By thin plate weir, see Chapter 4.

2.7. Portable Parshall Flume

A portable modified Parshall flume has been developed by the U. S. Geological Survey (2.8.9) in the 3-inch size. Figure 2.5 shows this flume and its rating table. It is intended for free flow only, so it has no diverging section. The converging section is shorter and the angle of convergence slightly larger than in the standard 3-inch flume. The discharge is higher than in the standard flume for each measured head.



Material: ⅛-in. aluminum

Gage height (ft)	Discharge (cfs)	Gage height (ft)	Discharge (cfs)	Gage height (ft)	Discharge (cfs)
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		

Figure 2.5. Portable 3-inch Parshall flume, from ref. 2.8.9.

2.8. References

2.8.1. Chow, V. T., "Open Channel Hydraulics," McGraw-Hill, 1959, pp. 75-81.

2.8.2. Parshall, R. L., "Parshall Flumes of Large Size," Colorado Agricultural College Bull. 386, May 1932.

2.8.3. Parshall, R. L., "The Improved Venturi Flume," Colorado Agricultural College Bull. 336, March 1928.

2.8.4. Parshall, R. L., "The Parshall Measuring Flume," Colorado State College Bull. 423, March 1936.

2.8.5. Robinson, A. R., "Parshall Measuring Flumes of Small Sizes," Colorado State Univ., Tech. Bull. 61, 1957.

2.8.6. Blaisdell, F. W., "Discussion of Model-Prototype Conformity," Trans. ASCE, 109, 1944, pp. 1944, pp. 157-167.

2.8.7. Hauser, V. L., "Relation Between the Flowrate Equation and Length of Converging Section of Parshall Flume," Trans. ASCE, 1960, pp. 86-88.

2.8.8. British Standards Institution, Standard No. 3680-4A, "Methods of Measurement of Liquid Flow in Open Channels, Part 4A: Thin Plate Weirs and Venturi Flumes," 1965.

2.8.9. Buchanan, T. J. and Somers, W. P., "Discharge Measurements at Gaging Stations," Chapter A8, Book 3 of Techniques of Water-Resources Investigations of the United States Geological Survey, 1969.

3. OTHER FLUMES

3.1. Introduction

Although Parshall flumes are the most frequently encountered flumes in water delivery and wastewater systems today, there are other types of flumes which are in use and warrant discussion. Also, there are flumes which have been developed only recently and which are likely to be used frequently in the future.

In the following, each flume is briefly described and selected information sources are cited. Proprietary flumes are not included.

3.2. Palmer-Bowlus Flumes

3.2.1. Background

Palmer-Bowlus flumes are formed by placing inserts into canals and sewers which raise the bottom and/or constrict the sidewalls. See figure 3.1 for a typical example.

Many Palmer-Bowlus flumes are in use in the United States. Their principal advantage lies in simplicity of construction and ease of installation through manholes. Also, because critical and nearly parallel flow is attained in the throat section, the flume is amenable to analytical treatment using critical flow principles. According to the experiments of Wells and Gotaas (3.2.2.1), performance of Palmer-Bowlus flumes inserted into U-shaped channels can be predicted theoretically within 3 percent, so long as the upstream depth does not exceed 0.9D, where D is the diameter of the circular conduit leading into U-channel. Users are cautioned that there appear to be no other published data on this flume to corroborate this conclusion.

3.2.2. General Information Sources

3.2.2.1. Wells, E. A. and Gotaas, H. B., "Design of Venturi Flumes in Circular Conduits," Amer. Soc. Civ. Eng., 82, Proc. Paper 928, April 1956, 23 pp. Order from Engineering Societies Library, 345 E. 47th St., New York 10017. $0.25 per page plus $3.00.

This paper gives the theoretical basis for the Palmer-Bowlus flume and compares theoretical predictions with the results of numerous laboratory experiments on various flume shapes, i.e., bottom-sidewall insert combinations. Limitations on its use are described based on the specific energy of the approach flow.

3.3. Cutthroat Flumes

3.3.1. Background

The cutthroat flume derives its name from the absence of a parallel-wall throat section. See figure 3.2. It is a flat-bottomed device whose main advantage is extreme simplicity of form and construction. The general form of the discharge equation for free flow is

$$Q = KW^{1.025}h_a^{n_1} = Ch_a^{n_1} \qquad [3.1]$$

where W is the throat width, h_a is the upstream depth measured at the location shown in figure 3.2, n_1 is the free-flow exponent, and C and K are free-flow coefficients. Free flow values needed for eq. 3.1 are given in Table 3.1 for a family of flume dimensions. Since K and n_1 depend only on flume length, interpolations can be made for coefficients of intermediate size flumes which are proportioned according to figure 3.2.

The discharge equation for submerged flow is of the form

$$Q = C_1(h_a - h_b)^{n_1}/(-\log S)^{n_2} \qquad [3.2]$$

where h_b is the downstream depth measured as in figure 3.2, submergence S is h_b/h_a, n_2 is the submerged-flow exponent, and $C_1 = K_1W^{1.025}$. Table 3.1 lists the values needed for use in eq. 3.2 as obtained experimentally for a family of flumes. Again interpolation for intermediate flume sizes is possible.

Note: There are no experimental data published to compare with the empirical eqs. 3.1 and 3.2, so no evaluation of their accuracy can be made here. The developer recommends (for increased precision) that throat-width to flume-length ratios between 0.1 and 0.4 be used.

A major error source can occur in all flumes installed in earth channels if there is downstream scour and the flume thereby acquires a downward longitudinal slope. The cutthroat flume is the only one for which some data exist on the effect of such slopes (3.3.2.3). These results are reproduced in table 3.2 for free flow only. Submerged flow data are available in ref. 3.3.2.3.

Reference 3.9.1 discusses the cutthroat flume from the viewpoint of its use in sewage treatment plants. For example, selection of a flume for installation immediately downstream of a grit chamber is covered.

3.3.2. General Information Sources

3.3.2.1. Skogerboe, G. V., Bennett, R. S., and Walker, W. R., "Generalized Discharge Relations for Cutthroat Flumes," Proc. Amer. Soc. Civ. Eng., 98, IR4, Dec. 1972, 569-583. Order from Engineering Societies Library, 345 E. 47th St., New York, N. Y. 10017. $0.25 per page plus $3.00.

This paper gives analytical and experimental background for table 3.1.

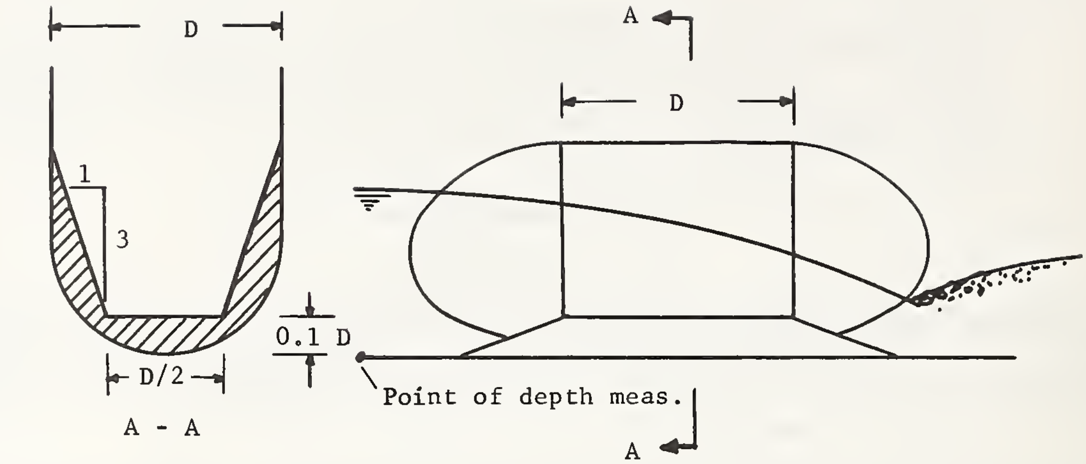
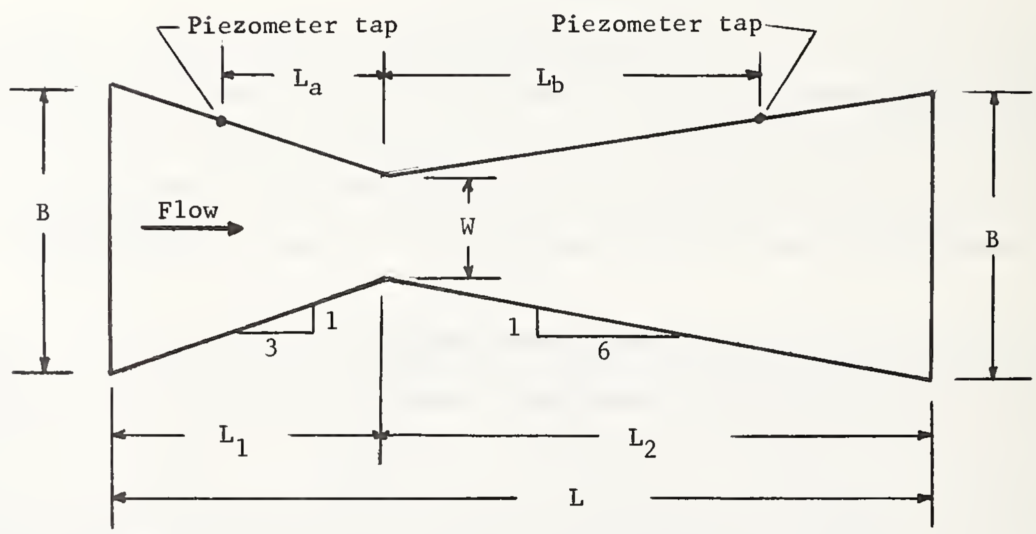


Figure 3.1. Example of a Palmer-Bowlus flume.



$$B = W + 2L_1/3 = W + L_2/3$$

$$L_a = 2L/9$$

$$L_b = 5L/9$$

Figure 3.2. Dimensions of rectangular cutthroat flumes.

3.3.2.2. Skogerboe, G. V., Hyatt, M. L., Anderson, R. K., and Eggleston, K. O., "Design and Calibration of Submerged Open Channel Flow Measurement Structures, Part 3 -- Cutthroat Flumes," Utah Water Research Lab. Rep. WG31-4, 1967. Obtain from Utah Water Research Lab., Utah State Univ., Logan, Utah 84321, $0.25.

Table 3.1

Coefficients and Exponents for Cutthroat Flumes
(See Eqs. 3.1 and 3.2)

Length, L ft	Width, W ft	Free Flow C	n_1	K	Submerged Flow C_1	n_2	K_1
9.00	1.000	3.50	1.560	3.500	1.688	1.390	1.700
9.00	2.000	7.11	1.560	3.500	3.430	1.390	1.700
9.00	4.000	14.49	1.560	3.500	6.970	1.390	1.700
9.00	6.000	22.0	1.560	3.500	10.600	1.390	1.700
4.50	0.250	0.96	1.720	3.980	0.548	1.410	2.250
4.50	0.500	1.96	1.720	3.980	1.120	1.410	2.250
4.50	1.000	3.98	1.720	3.980	2.275	1.410	2.250
4.50	2.000	8.01	1.720	3.980	4.575	1.410	2.250
3.00	0.167	0.719	1.840	4.500	0.413	1.480	2.580
3.00	0.333	1.459	1.840	4.500	0.837	1.480	2.580
3.00	0.667	2.970	1.840	4.500	1.705	1.480	2.580
3.00	1.333	6.040	1.840	4.500	3.465	1.480	2.580
1.50	0.083	0.494	2.150	6.100	0.261	1.741	3.250
1.50	0.167	0.974	2.150	6.100	0.516	1.741	3.250
1.50	0.333	1.975	2.150	6.100	1.048	1.741	3.250
1.50	0.667	4.030	2.150	6.100	2.140	1.741	3.250

Table 3.2

Effect of Floor Slope on Cutthroat Flumes
(See Equation 3.1)

Length, L ft	Width, W ft	Floor Slope ft/ft	Free Flow C	n_1
1.50	0.167	0.0000	0.974	2.150
1.50	0.167	0.0278	1.075	2.150
1.50	0.167	0.0556	1.220	2.150
1.50	0.167	0.0833	1.390	2.150
1.50	0.333	0.0000	1.975	2.150
1.50	0.333	0.0278	2.180	2.150
1.50	0.333	0.0556	2.470	2.150
1.50	0.333	0.0972	2.990	2.150
4.50	0.500	0.0000	1.960	1.720
4.50	0.500	0.0185	1.980	1.720
4.50	0.500	0.0556	2.140	1.720
4.50	0.500	0.1078	2.560	1.720
4.50	1.000	0.0000	3.980	1.720
4.50	1.000	0.0185	4.000	1.720
4.50	1.000	0.0556	4.350	1.720
4.50	1.000	0.1064	5.200	1.720

This report discusses only one size of the rectangular cutthroat flume, the results of which are already included in ref. 3.3.2.1. However, data on cutthroat flumes of trapezoidal and triangular section are also included in this report.

3.3.2.3 Skogerboe, G. V., Walker, W. R., Wu, T-Y, and Bennett, R. S., "Slope-Discharge Ratings for Cutthroat Flumes," Trans. Amer. Soc. Agric. Eng., 16, 1, 1973, 78-81. Copy available from American Society of Agricultural Engineers, P. O. Box 410, St. Joseph, Mich. 49085. $1.50.

3.4. Trapezoidal Flumes with Bottom Slope

The main advantage of flumes with trapezoidal section is that they have much larger capacities than rectangular flumes of the same bottom width. One trapezoidal flume -- the cutthroat type -- has already been alluded to in ref. 3.3.2.2. A trapezoidal flume recommended by the U. S. Geological Survey incorporates a 5 percent bottom slope to create supercritical flow, as shown in figure 3.3. Discharge curves are shown in figure 3.4, reproduced from ref. 3.9.2. Data for 1-ft and 3-ft flumes only are available. No accuracy statements are available.

3.5. Critical-Depth Flumes

3.5.1. Background

The trapezoidal critical-depth flume is shown in figure 3.5. In a sense it belongs in the same category as the Palmer-Bowlus flume, since the flow can be predicted analytically using critical-flow principles. In this case the long parallel throat permits the flow to closely approximate parallel critical flow. Friction effects, i.e., head loss along the flume and velocity distribution in the throat are accounted for in the theory (as closely as possible) and flume ratings have been computed (3.5.2.1) for a selected group of flume geometries covering a variety of channel conditions encountered in the field. Accuracy of $\pm$ 2 percent is claimed for these solutions.

Rectangular critical flow flumes are described in ref. 3.5.2.2. Again these in principle belong with the Palmer-Bowlus flumes, but the inlet and outlet geometry is more detailed and discharge coefficients are more closely determined analytically.

3.5.2. General Information Sources

3.5.2.1. Replogle, J. A., "Critical-Depth Flumes for Determining Flow in Canals and Natural Channels," Trans. Amer. Soc. Agric. Eng., 14, 3, 1971, pp. 428-433, 436. Copy available from American Society of Agricultural Engineers, P. O. Box 410, St. Joseph, Mich. 49085. $1.50.

3.5.2.2. British Standards Institution, Standard No. 3680-4A, "Methods of Measurement of Liquid Flow in Open Channels: Part 4A, Thin Plate Weirs and Venturi Flumes," 1965. Order from American National Standards Institute, 1430 Broadway, New York, N. Y. 10018. $9.50.

3.6. San Dimas Flume

3.6.1. Background

The San Dimas flume was developed specifically to pass large amounts of sediment and debris. The original design is shown in figure 3.6 along with rating curves. Later modifications are covered in ref. 3.6.2.1.

3.6.2. General Information Sources

3.6.2.1. Bermel, K. J., "Hydraulic Influence of Modifications to the San Dimas Critical-Depth Measuring Flume," Trans. Amer. Geophys. Union, 31, 5, Oct. 1950, 763-770. Engineering Societies Library, 345 E. 47th St., New York, N. Y. 10017. $0.25 plus $3.00.

Flume Size, W_T ft	Width of Approach Section, W_A ft.	ANGLES		LENGTHS			Flume Height h, ft.	Sloping Wall Length D, ft.	CAPACITIES		FLOOR SLOPES	
		Sloping Walls θ	Converging Walls ϕ	Approach Section L_A, ft.	Converging Section L_C, ft.	Throat Section L_T, ft.			Min. cfs	Max. cfs	Approach Section	Converging & Throat Sect.
1	5.0	30°	21.8°	5.0	5.0	5.0	4.0	8.0	0*	350	5 %**	5 %
3	9.0	30°	21.8°	Omitted	7.5	6.5	5.0	10.0	10	600	0 %	5 %

* With V-notch weir installed (see fig. 3.4)
** Should be level for very low flows

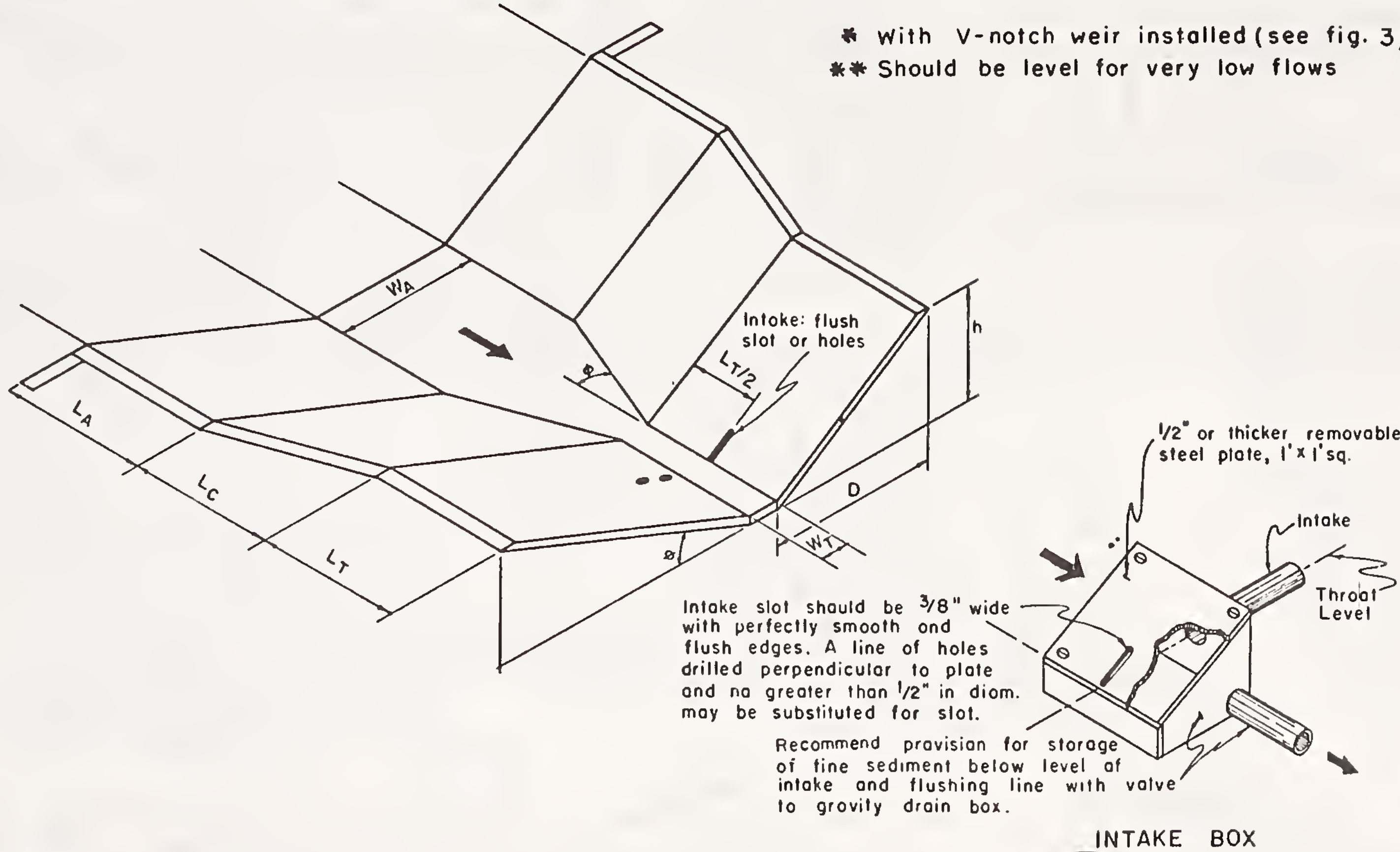


Figure 3.3. Trapezoidal supercritical flow flume, from ref. 3.9.2.

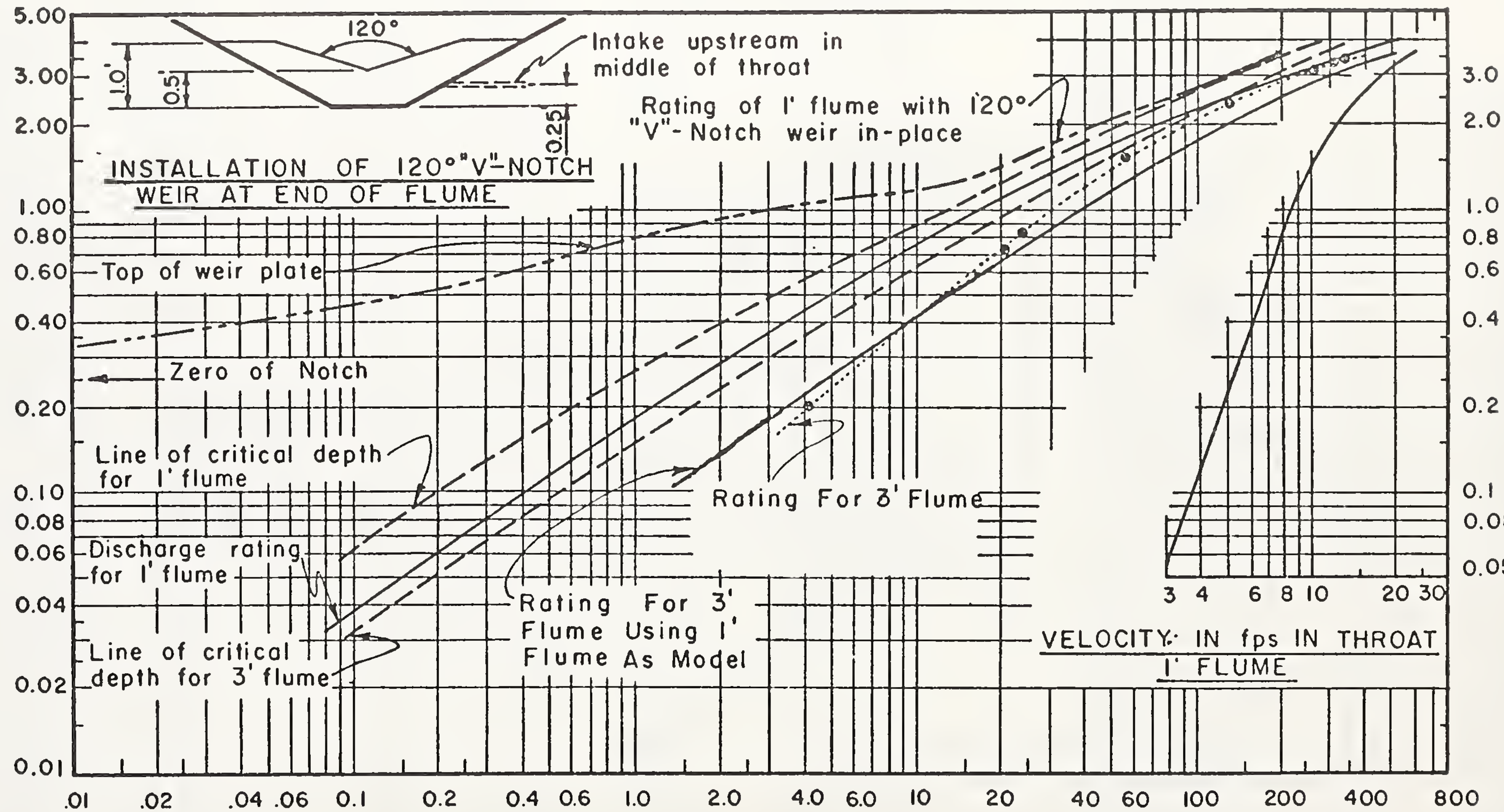


Figure 3.4. Discharge rating curves for trapezoidal super-
critical flow flumes of fig. 3.3, from ref. 3.9.2.

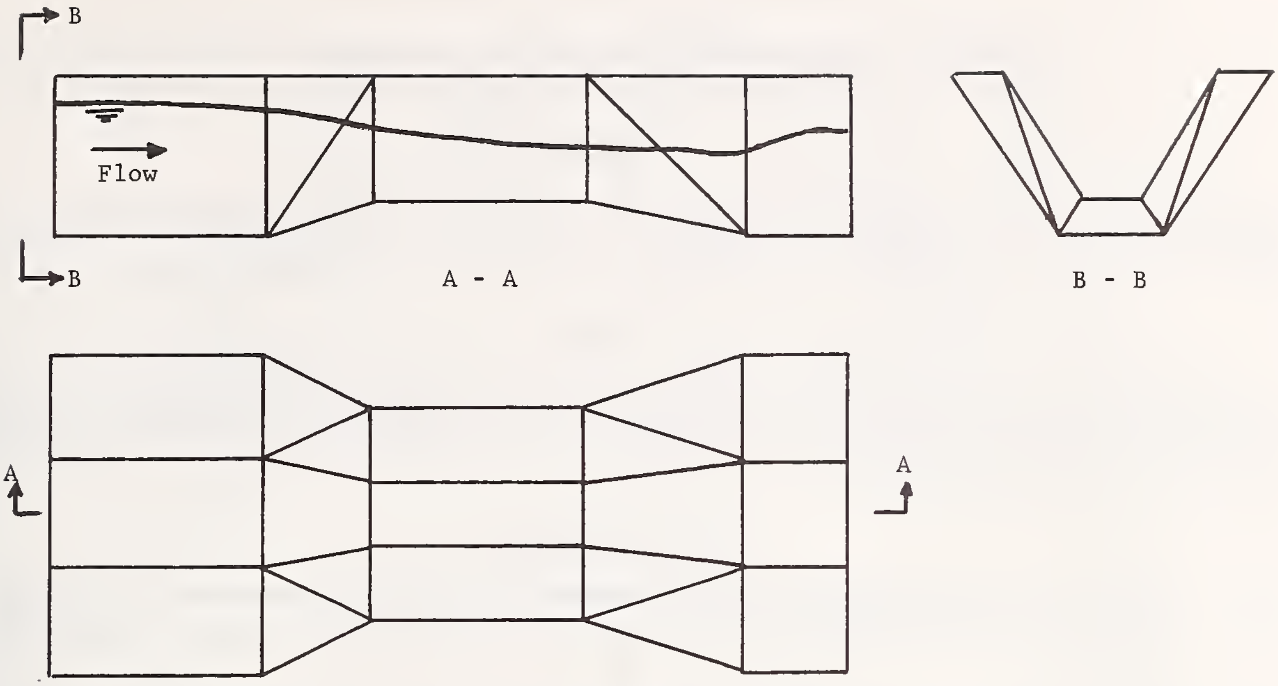


Figure 3.5. A trapezoidal critical depth flume.

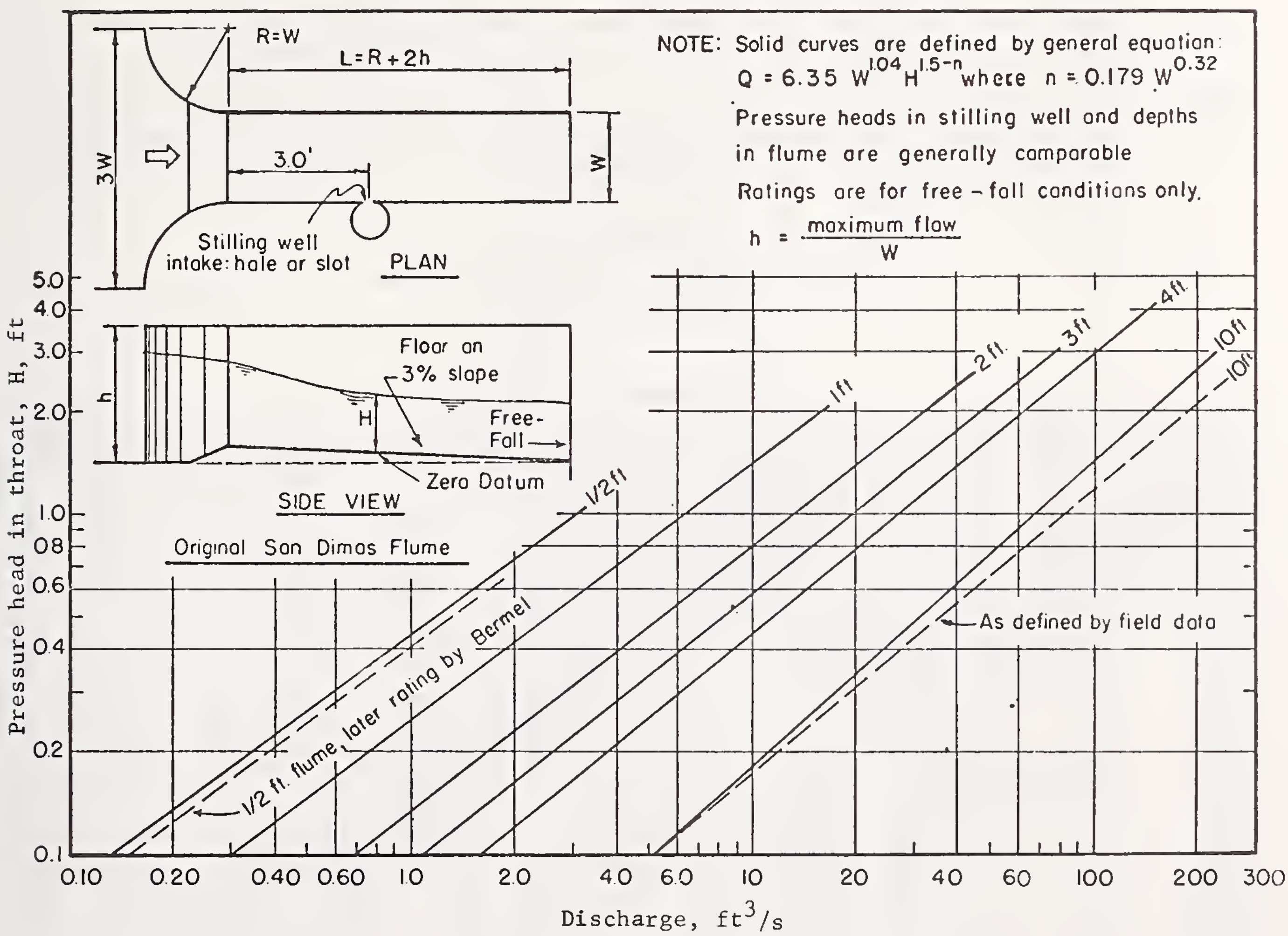


Figure 3.6. Original San Dimas flume, from ref. 3.9.2.

This paper covers changes in ratings due to changes in floor slope, roughness, and errors due to partial blockage of the approach channel by debris.

3.7. H-Flumes

The H-flume was developed by the Soil Conservation Service, U. S. Department of Agriculture (3.9.3). Its principal advantage is simplicity of construction, as can be seen in figure 3.7. Rating data are shown in table 3.3. No accuracy information is available.

3.8. Errors

For those flumes for which an estimate of the accuracy of the head discharge equations or curves are given, this error can be combined with the head measurement error (estimated by the user) to give a combined or system error. See section 2.4.8. for a discussion of this type of estimate.

3.9. References

3.9.1. Walker, W. R., Skogerboe, G. V., and Bennett, R. S., "Flow Measuring Flume for Wastewater Treatment Plants," Jour. Water Poll. Control Fed., 45, 3, March 1973, pp. 542-551.

3.9.2. Kilpatrick, F. A., "Use of Flumes in Measuring Discharge at Gaging Stations," Book 1, Chap. 16 of Surface Water Techniques, U. S. Geological Survey. Now under revision. Original version out of print.

3.9.3. "Field Manual for Research in Agricultural Hydrology," Agriculture Handbook No. 224, Agric. Res. Serv., Soil and Water Conserv. Res. Div., reprinted 1968.

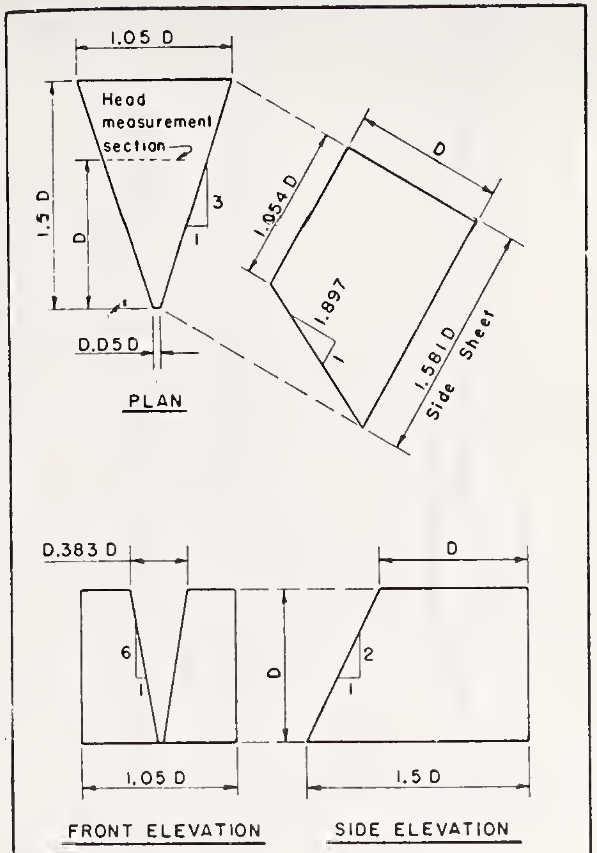
3.9.4. "Field Manual for Research in Agricultural Hydrology," Agriculture Handbook No. 224, reprinted 1968, Soil and Water Conservation Research Division, Agricultural Research Service, U. S. Dept. of Agriculture.

Table 3.3

Discharge (cfs) for Selected H-Flumes
from Reference 3.9.3

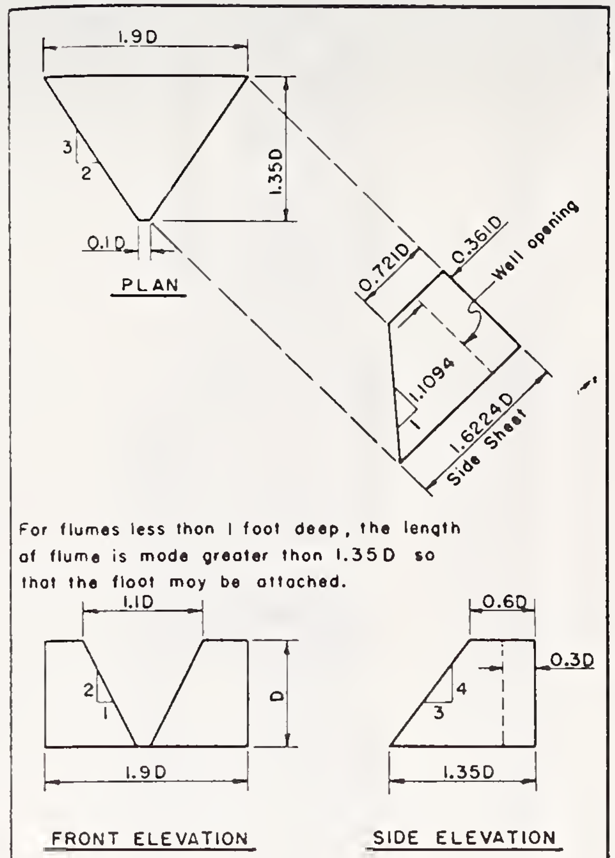
Flume Type and Size, D

Head	Type HS				Type H								Type HS
ft	0.4	0.6	0.8	1.0	0.5	0.75	1.0	1.5	2.0	2.5	3.0	4.5	4.0
0.05	0.0010	0.0014	0.0017	0.0021	0.002	0.003	0.004	0.006	0.007	0.009	0.010	0.015	0.03
0.10	0.004	0.005	0.006	0.007	0.010	0.013	0.015	0.020	0.025	0.030	0.035	0.050	0.09
0.20	0.018	0.021	0.024	0.027	0.04	0.05	0.06	0.07	0.08	0.10	0.11	0.16	0.28
0.30	0.044	0.049	0.054	0.060	0.11	0.12	0.13	0.16	0.18	0.21	0.23	0.31	0.56
0.40	0.085	0.092	0.10	0.11	0.20	0.22	0.24	0.28	0.32	0.36	0.40	0.52	0.94
0.50		0.15	0.16	0.18	0.35	0.37	0.40	0.45	0.51	0.56	0.62	0.78	1.42
0.60		0.23	0.24	0.26		0.57	0.60	0.67	0.74	0.82	0.89	1.11	2.01
0.80			0.47	0.50			1.16	1.27	1.38	1.49	1.60	1.94	3.53
1.00				0.82			1.96	2.09	2.25	2.41	2.57	3.04	5.56
1.20								3.20	3.38	3.59	3.80	4.42	8.06
1.40								4.60	4.82	5.06	5.33	6.11	11.2
1.60									6.58	6.84	7.16	8.12	14.9
1.80									8.67	8.98	9.33	10.50	19.2
2.00									11.1	11.5	11.9	13.2	24.3
2.50										19.4	19.9	26.6	39.9
3.00											31.0	32.7	60.3
3.50												46.8	85.9
4.00												63.9	117
4.50												84.5	



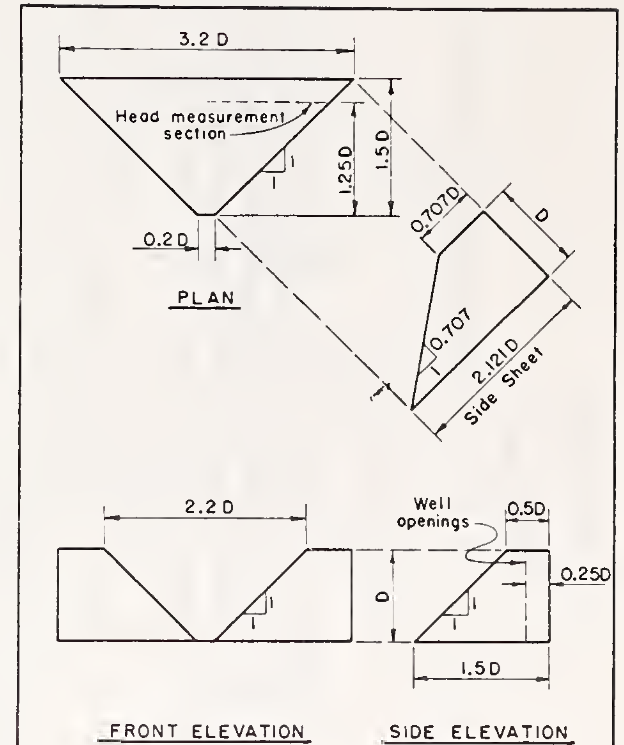
PRDPDRTIDNS OF THE TYPE HS FLUME

APPRDXIMATE CAPACITIES	
DEPTH-D Feet	CAPACITY C.f.s.
D.4	0.085
D.6	D.23
D.8	0.47
I.D	0.82



PROPORTIONS OF THE TYPE H FLUME

APPROXIMATE CAPACITIES	
DEPTH-D Feet	CAPACITY C.f.s.
0.5	0.3+
0.75	1-
1.0	2
1.5	5+
2.0	11
2.5	19
3.0	30+



PROPORTIONS OF THE TYPE HL FLUME

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

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

Figure 3.7. H-flume dimensions, from ref. 3.9.4.

4. THIN PLATE WEIRS

4.1. Background

The weir shapes in most common use are rectangular, 90-degree V-notch, and trapezoidal. These are shown in figure 4.1.

4.1.1. Rectangular Weirs

The rectangular weir crest can be in the form of a notch (figure 4.1a), i.e., a "contracted" weir, or it can span the entire channel width (figure 4.1b) in which case it is called a "suppressed" weir. For the weir to operate as a "standard" contracted weir, the side contractions should be at least twice the head and not less than 1 foot, and the crest height should be at least twice the head and not less than 1 foot. Heads less than 0.2 ft should not be used.

Francis equations. The Francis formulas below are frequently used for rectangular weir discharge.

$$\text{Suppressed weir:} \quad Q = 3.33H^{1.5}L \tag{4.1a}$$

$$\text{Fully contracted weir:} \quad Q = 3.33H^{1.5}(L - 0.2H) \tag{4.1b}$$

where Q is flowrate (cubic feet per second), H is the height of water above the crest elevation at the measuring station (ft) and L is the weir crest length (ft). In both equations the velocity of approach can be included by replacing $H^{1.5}$ with

$$(H + h)^{1.5} - h^{1.5}$$

where h is the approach velocity head $V^2/2g$. This correction usually is made by trial. Solutions to eqs. 4.1 in tabular form can be found in refs. 4.3.1 and 4.3.5, and in other sources. The Francis equations have the advantage of convenience even where tables are not available, because of the constant coefficient. They should not be used for $H > L/3$, nor should eq. 4.1b be used for contractions which are less than standard (figure 4.1).

Kindsvater-Carter equation. This equation is coming into more frequent use and is considered by some to be more accurate than the Francis formula.

$$Q = C_e L_e H_e^{1.5} \tag{4.2}$$

where $H_e = H + 0.003$.

$L_e = L + k_b$, with k_b as given in figure 4.2a.

C_e = a coefficient as given in figure 4.2b.

An advantage of eq. 4.2 is that the velocity of approach effect is incorporated in the curves and need not be included by trial. Also, contractions less than standard can be accounted for.

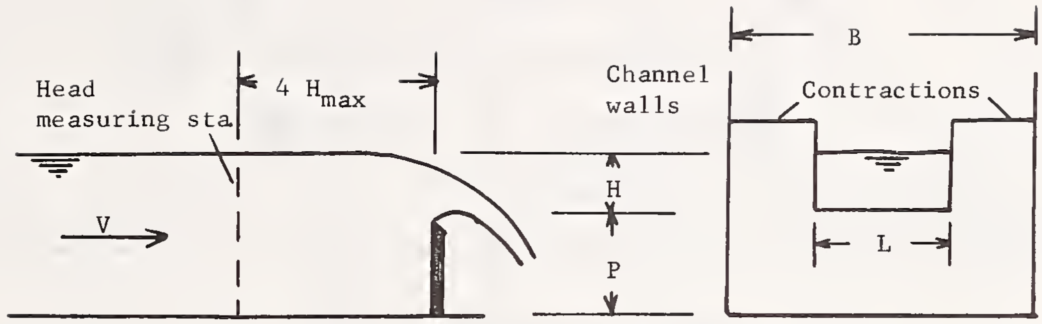
Eqs. 4.1 and 4.2 are compared in section 4.4.2.1.

4.1.2. V-Notch Weirs

Triangular or V-notch weirs (figure 4.1c) are particularly useful for low flows. Standard contraction requirements are the same as for rectangular weirs.

Cone formula. This formula is for 90-degree fully contracted V-notches only. Results are given in tables in ref. 4.3.1.

$$Q = 2.49H^{2.48} \tag{4.3}$$



Note: Contractions > 2H; P> 2H, for full contraction

Figure 4.1a. Fully contracted rectangular weir.

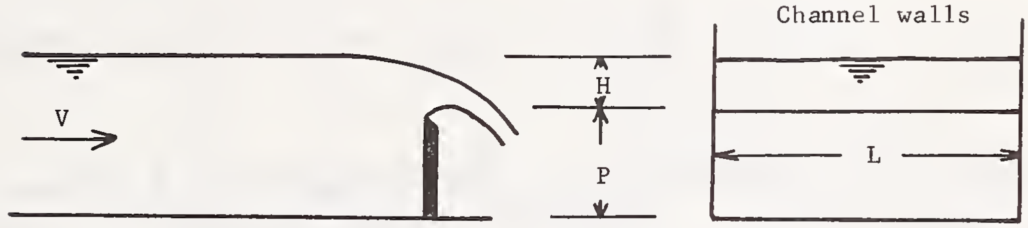


Figure 4.1b. Suppressed rectangular weir.

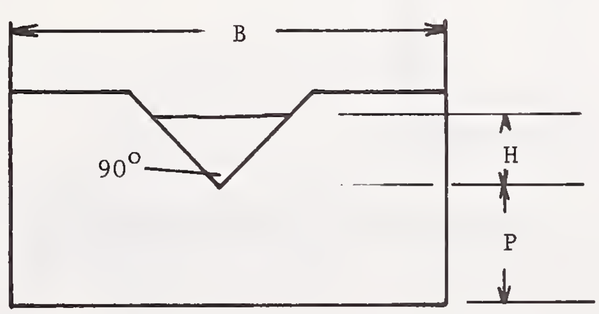


Figure 4.1c. V-notch 90 degree weir.

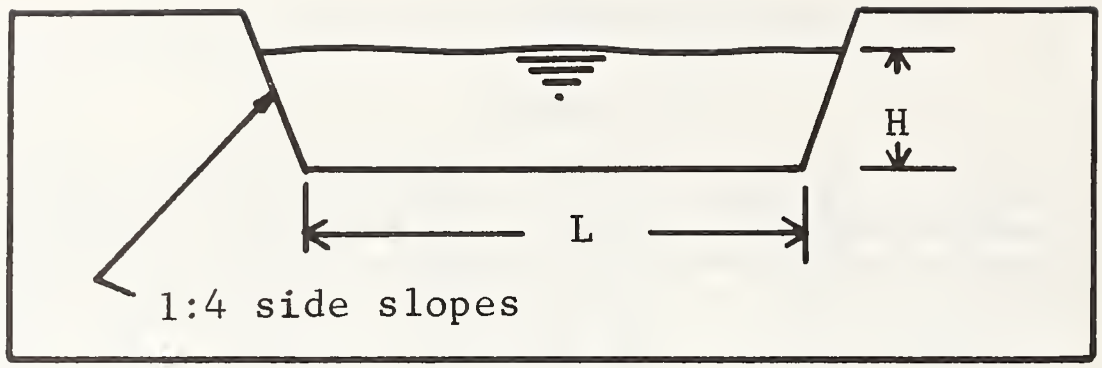


Figure 4.1d. Cipolletti weir.

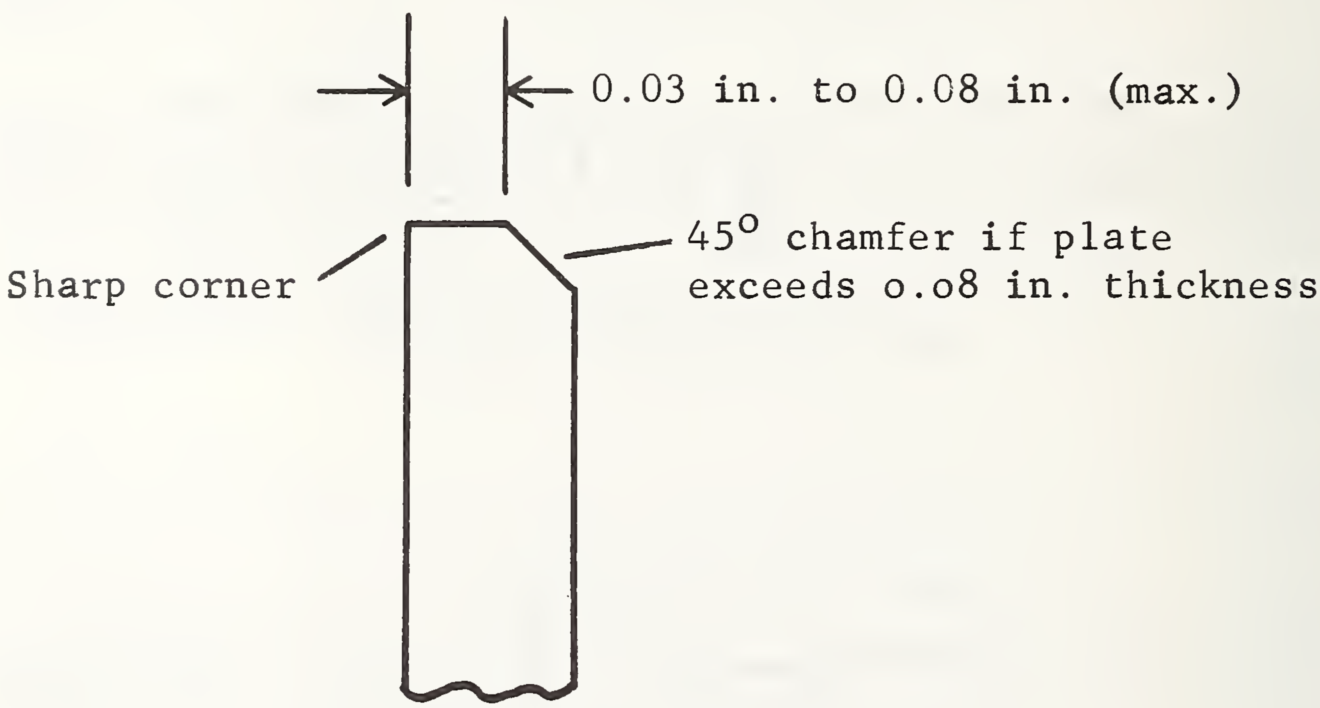


Figure 4.1e. Weir edge detail.

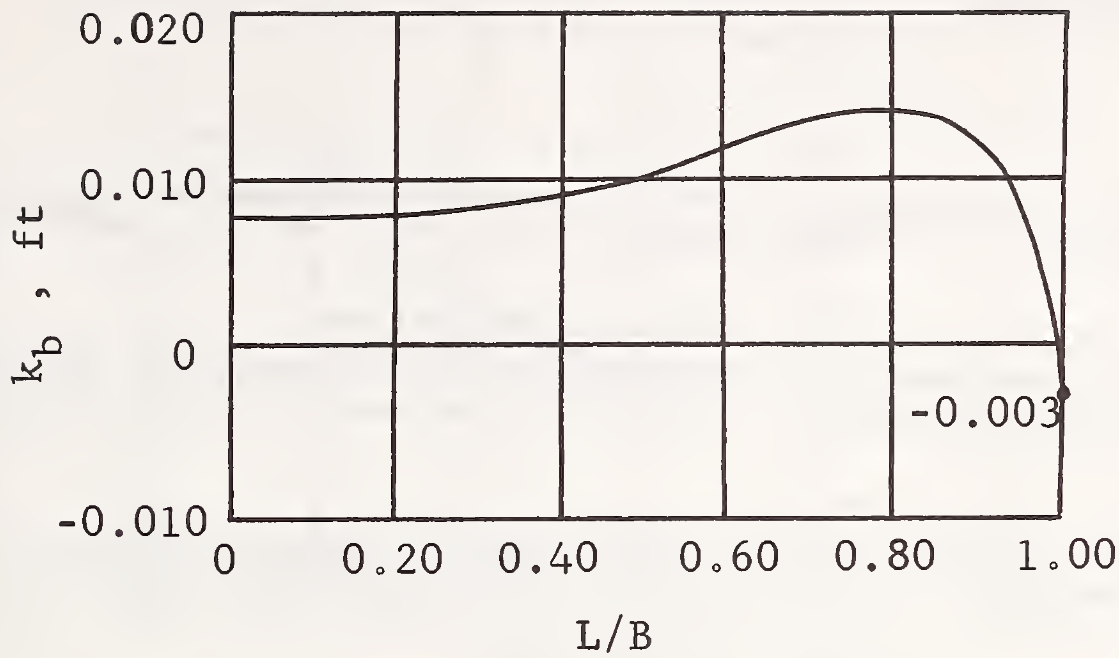


Figure 4.2a. Values of k_b for eq. 4.2, from ref. 4.3.1.

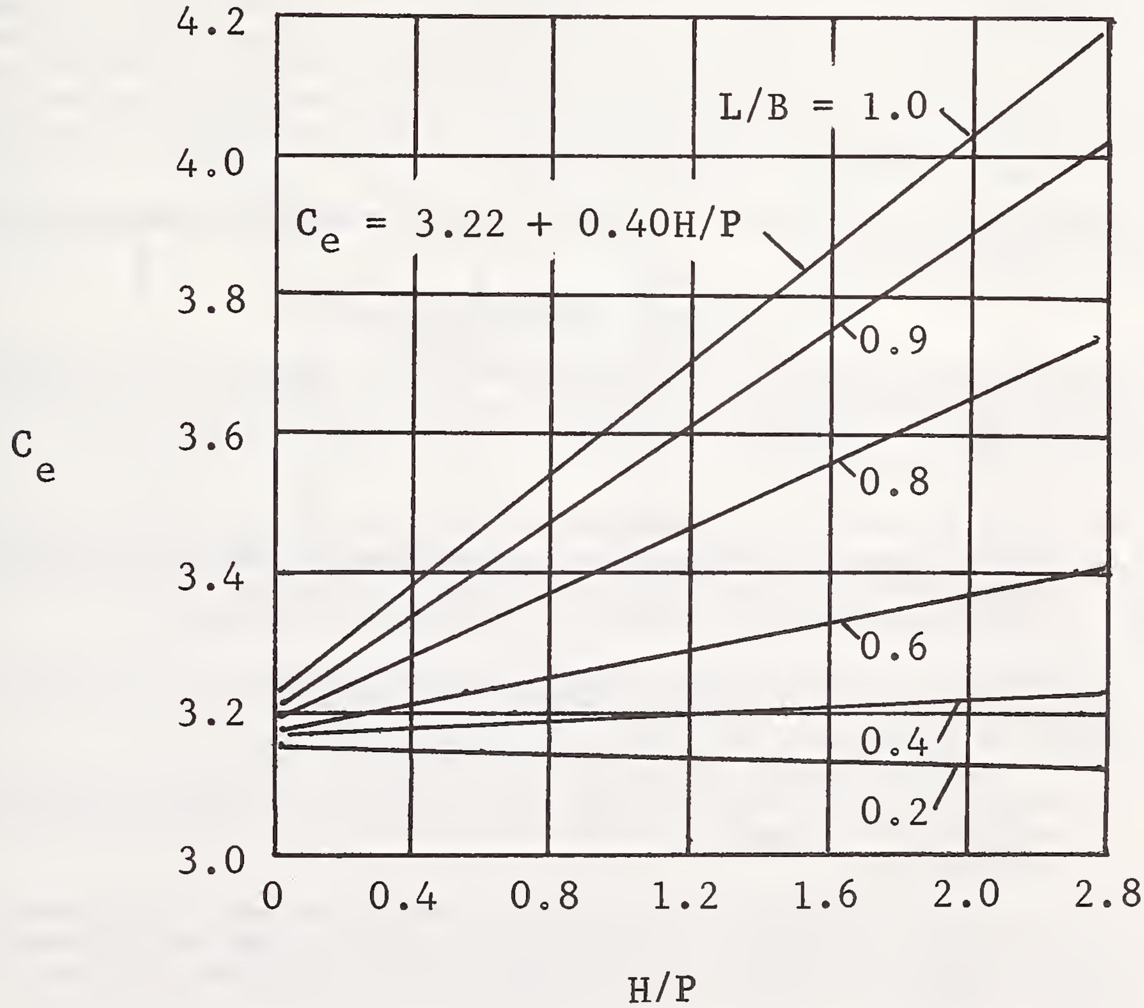


Figure 4.2b. Values of C_e for eq. 4.2, from ref. 4.3.1.

Kindsvater-Shen. The basic equation, which is valid for any notch angle is

$$Q = (8/15)\ (2g)^{1/2} C_e \tan(\theta/2)\ H_e^{3/2} \qquad [4.4]$$

where: θ is the notch angle.

H_e is measured head H plus K_h, a factor given in figure 4.3.

C_e is a coefficient given in figure 4.4. Values of C_e for incomplete contractions are given in figure 4.5 for 90-degree notches only. Equations 4.3 and 4.4 are compared in section 4.4.2.2.

4.1.3. Cipolletti (Trapezoidal) Weirs

The Cipolletti weir (figure 4.1d) is frequently found in field practice and has the same contraction requirements as rectangular weirs. Its equation is, without the velocity-of-approach factor,

$$Q = 3.367\ L\ H^{1.5} \qquad [4.5]$$

Tabular solutions to eq. 4.5 are given in ref. 4.3.1. Use of the Cipolletti weir with eq. 4.5 does not give results as accurate as do the rectangular and triangular weirs (4.3.1). Information is not available with which to estimate accuracy limits.

4.2. Uses

Weirs have the advantage of being relatively simple to construct and install, although it should be emphasized that this simplicity does not imply that lack of care and precision is tolerable. Another advantage is that a considerable data base exists, although not all results are in agreement. Their main disadvantage is the collection of solids behind the weir plate, which if allowed to continue, ultimately will affect the head-to-crest height ratio, H/P, and thus the discharge. They are nevertheless frequently used as temporary gaging stations in sewers, with apparent success if the installations are carefully monitored for crest condition and upstream deposits.

Thin plate weirs also impose an appreciable head loss, which is undesirable or intolerable in some applications.

4.3. General Information Sources

4.3.1. "Water Measurement Manual," Bureau of Reclamation, U. S. Dept. of the Interior, Second Ed., Revised reprint, 1974, Ch. 2, pp. 7-42. Order from U. S. Govt. Printing Office, Washington, D. C. 20402. $5.80. Stock No. SN2403-0027, Cat. No. I 27.19/2:W 29/2.

This reference contains considerable general background information on weirs, a limited amount of information on construction and maintenance, head-discharge relations in tabular form for eqs. 4.1a, 4.3 and 4.5, curves for eq. 4.2, recommended weir box sizes and other useful information.

4.3.2. "Fluid Meters -- Their Theory and Application," H. S. Bean, ed., ASME Research Committee on Fluid Meters, Sixth Edition 1971. Obtain from Amer. Soc. Mech. Engrs., 345 E. 47th St., New York, N. Y. 10017. $20.00.

This source gives curves for application of eqs. 4.2 and 4.4. Principles of weir flow are discussed, but there are no details on construction, head measurement, etc.

4.3.3. British Standards Institution, Standard No. 2680-4A, "Methods of Measurement of Liquid Flow in Open Channels: Part 4A, Thin Plate Weirs and Venturi Flumes," 1965. Order from American National Standards Institute, 1430 Broadway, New York, N. Y. 10018. $9.50.

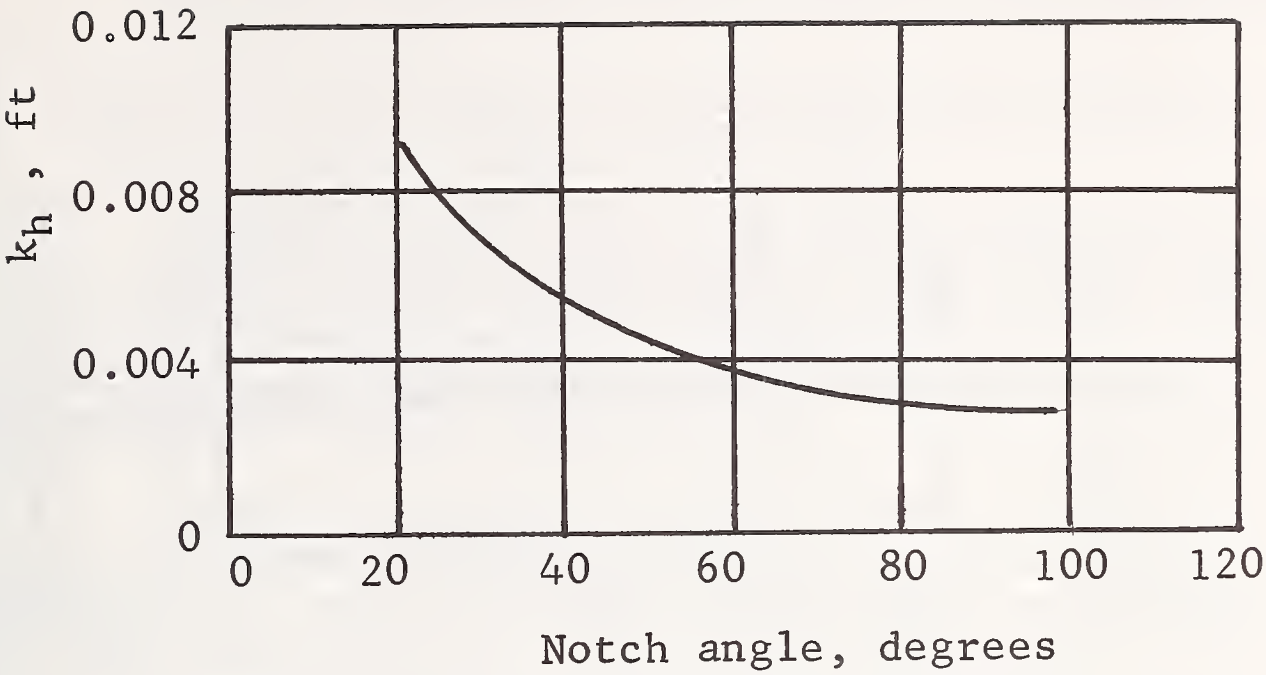


Figure 4.3. Values of K_h for eq. 4.4, from ref. 4.3.3.

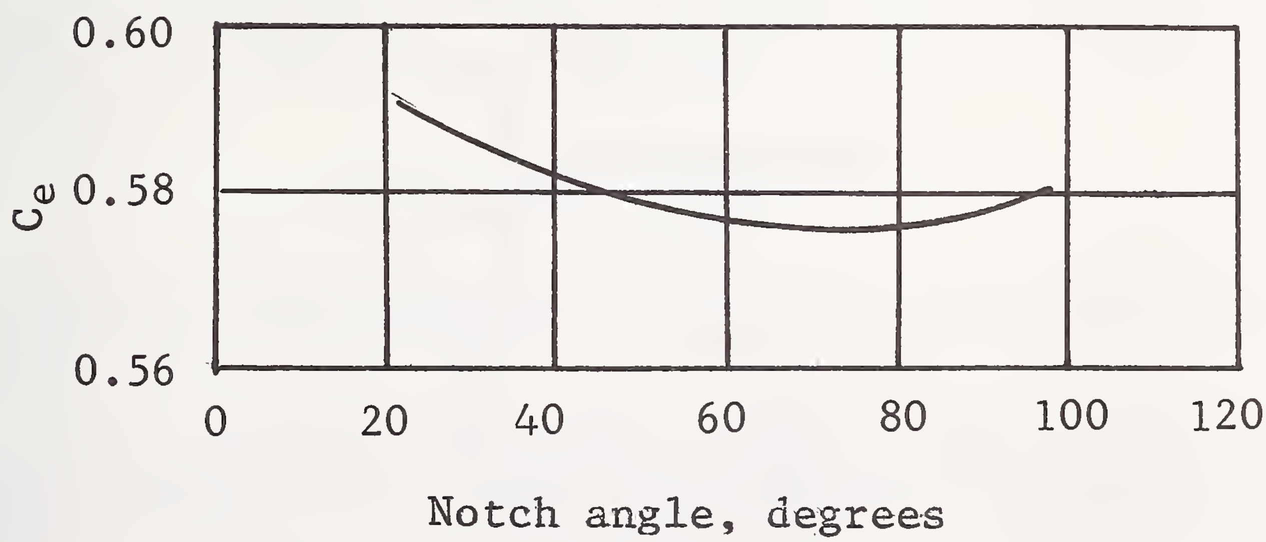


Figure 4.4. Values of C_e for eq. 4.4, from ref. 4.3.3.

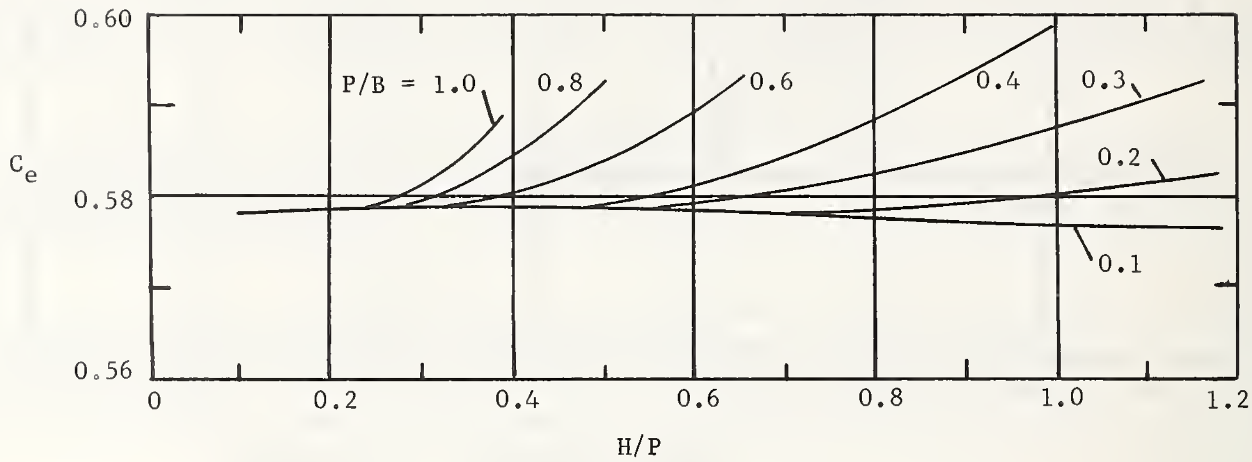


Figure 4.5. Values of C_e for eq. 4.4, incomplete contractions, from ref. 4.3.3.

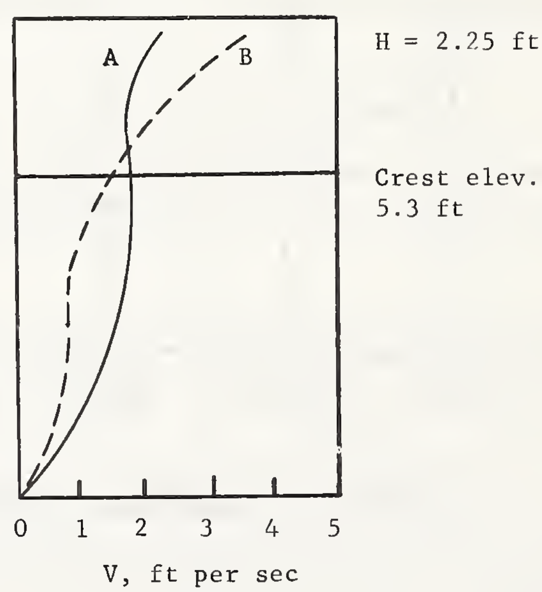


Figure 4.6. Upstream velocity distributions (see 4.4.6).

This standard gives curves for application of eqs. 4.2 and 4.4. Many details are given on requirements for accurate measurements, and on estimation of measurement errors.

4.3.4. ASTM Standard D2034-68, "Open Channel Flow Measurement of Industrial Water and Industrial Waste Water by Weirs." Order from American Society for Testing and Materials, 1916 Race St., Phila., Pa. 19103. $1.50.

This standard provides a concise treatment of weirs, but some of the recommended discharge formulas are more complex than those heretofore cited in this chapter. A convenient table is presented giving recommended weir and weir box geometries for specific ranges of discharge. Details on float well construction and on depth sensing and recording instruments are given.

4.3.5. "Planning and Making Industrial Waste Surveys," Ohio River Valley Water Sanitation Commission, Cincinnati, Ohio, 1952. $2.00.

This report gives a very basic and well-illustrated introduction to weir measurements. Flowrates are given in terms of gallons per minute in tables for rectangular and 90-degree notch weirs.

4.3.6. Thomas, C. W., "Errors in Measurement of Irrigation Water," Proc. ASCE, 83, IR3, Proc. Paper 1362, Sept. 1957, 14 pp. Obtain from Engineering Societies Library 345 E. 47th St., New York, N. Y. 10017. $0.25 per page plus $3.00.

Errors due to incorrect geometries and head measurement are given in curves for Parshall flumes and weirs.

4.4. Error Sources in Weir Use

4.4.1. General

As is true for all flow measuring devices, the accuracy of the result depends upon the performance of the instrument treated as a system -- in this case (a) the primary element (weir), (b) the secondary element (depth sensor and recorder), and finally (c) the manner in which these elements are installed and maintained. The following sections deal with these three aspects.

4.4.2. The Head-Discharge Relations

4.4.2.1. Rectangular Weirs

Of the many published rectangular weir formulas, only eqs. 4.1 and 4.2 are included in this chapter, the first because it is so popular currently, and the second because it appears to be acquiring more acceptance and is generally considered to be very accurate. The two equations can best be compared by using numerical examples.

If there should be significant differences between the results of the two equations, one can consider the range of the experiments from which the equations were derived. Francis' experiments were on long crests, mainly 10 ft, with heads not exceeding 2 ft. Kindsvater and Carter, on the other hand, used crest lengths of 0.1 to 2.7 ft and heads from 0.1 to 0.75 ft.

Example 1. Fully contracted weir; head, 0.50 ft; length 2.00 ft; channel width, 5.00 ft; crest height, 2.00 ft.

From eq. 4.1b: $Q = 2.24$ cfs

From eq. 4.2: $Q = 2.26$ cfs

Example 2. Suppressed weir; head, 0.50 ft; length, 2.00 ft; crest height, 2.00 ft.

From eq. 4.1a: Q = 2.35 cfs

From eq. 4.2: Q = 2.36 cfs

Example 3. Suppressed weir; head, 1.00 ft; length 10.00 ft; crest height = 2.00 ft.

From eq. 4.1a: Q = 34.2 cfs (including velocity-head correction).

From eq. 4.2: Q = 34.4 cfs

These results for weirs of reasonable dimensions agree within 1 percent. To illustrate the error that can be involved in using the Francis contracted weir formula (eq. 4.1b) on a weir which is not fully contracted, consider the following example.

Example 4. Partially contracted weir; head, 0.50 ft; length, 2.00 ft; channel width, 3.00 ft; crest height, 0.50 ft.

From eq. 4.1b: Q = 2.30 cfs (including velocity-head correction)

From eq. 4.2: Q = 2.39 cfs

The difference here is approximately 4 percent.

Note. Do not use eqs. 4.1 for heads larger than one-third the crest length, even if such results are included in published tables. Data reported in ref. 4.3.1 show that, at least for short weirs, significant errors can result. For example, a fully contracted weir 2 ft long with a 1 ft head had a measured discharge of 6.31 cfs. Equation 4.1 gives 6.00 cfs (an error of -5 percent), while eq. 4.2 results in about 6.35 cfs (an error of less than 1 percent).

Errors due to failure to correct eqs. 4.1 for the velocity of approach at the head measurement section can be computed by trial. They are tabulated in ref. 4.3.6 and are abstracted in table 4.1 below for purposes of illustration. Reference 4.3.1 contains tables of three-halves powers of numbers to facilitate making this correction.

Table 4.1

Percentage Error in Q When
Francis Equations are Uncorrected
for Velocity of Approach

V, ft/sec	Measured Head, H, ft: 0.2	0.6	1.0
	Error in Q, %		
0.5	2.7	0.9	0.6
1.0	9.8	3.4	2.2
1.5	20.8	7.5	4.7

It is seen that approach velocities have to be well below 0.5 ft per second before the error becomes negligible at low heads. The use of eq. 4.2 obviates the need for the velocity-of-approach correction.

4.4.2.2. Triangular Weirs

Contraction errors. Equations 4.3 and 4.4 give results within about 0.5 percent of each other for 90-degree fully contracted weirs. However, if contractions are incomplete, errors can result from use of eq. 4.3.

Example: 90 degree notch, H = 1.00 ft, P = 1.00 ft, B = 2.50 ft (see figure 4.1c). In this case the contractions are only 0.25 ft at the water surface.

From eq. 4.3: Q = 2.49 cfs

From eq. 4.4: Q = 2.57 cfs

The difference is more than 3 percent.

For angles other than 90 degrees, use eq. 4.4. Corrections for incomplete contractions are available only for 90 degrees, however.

Angle measurement errors. A change of only 1 degree for a nominal 90 degree notch would introduce an error of almost 2 percent in flowrate, through its effect on the tan $\theta/2$ term in eq. 4.4, even the effect on C_e from figure 4.4 would be negligible. Therefore the notch angle must be accurately determined.

4.4.3. Errors Due to Weir Plate Imperfections

Edge Sharpness. Rounding the upstream edge of the weir increases the flowrate. Small amounts of rounding can occur in an initially "sharp" edge as a result of erosion and wear. Schoder and Turner (4.8.2) presented data on roundness effects. An example is given in Table 4.2 below.

Table 4.2

Approximate Percentage Change in Q Due to Edge Rounding

Radius of Round in.	Head ft	Discharge Change %
0.016	0.2	0.8
0.016	0.6	0.6
0.016	1.0	0.3
0.042	0.2	2.2
0.042	0.6	1.2
0.042	1.0	0.5
0.125	0.2	6.0

Roughness. According to experiments cited in ref. 4.8.2, roughening the upstream face of a rectangular weir to the texture of a rough file can increase flowrate by about 1.5 percent at 0.5 ft head and about 1 percent at a 1.25 ft head. Another set of experiments cited in the same reference indicated that a V-notch weir (angle not stated) with a 3-inch head would have the flowrate increased by 1.8 percent if the initially smooth brass weir plate were coated with a medium emery, and by 2.4 percent if coated with coarse emery.

The effects of edge rounding and plate roughening are in the same direction, increasing the flow for the same head. Therefore possible effects of aging and wear on weir plates must be closely monitored.

4.4.4. Aeration of Suppressed Rectangular Weirs

Failure to provide sufficiently large air vents beneath the nappe of a suppressed rectangular weir causes an increase in discharge. As an indication of the magnitude of the potential error, a pressure of 1.2 in. of water (below atmosphere) beneath the nappe caused a 5 percent discharge increase at 0.5 ft head (4.3.6). A method of computing the air requirement can be found in ref. 4.8.1.

4.4.5 Errors Due to Weir Slope

If there is transverse slope to a rectangular weir, a systematic error in head can be introduced, depending upon what part of the crest the head gage has been referenced to. Apparently weirs are not very sensitive to small upstream or downstream slopes of the plate (4.3.6).

4.4.6 Upstream Velocity Distribution Effect

Examples of acceptable upstream velocity distributions are given in ref. 4.3.3. In figure 4.6 are shown two velocity distributions which cause a discharge difference of about 3.5 percent for the same head on a suppressed weir as presented by Schoder and Turner (4.8.2). Distribution B was deliberately created with artificial barriers. The α factors are about 1.2 and 2.1 for A and B, respectively. However, the velocity distribution error cannot be generalized, except to say that it can be minimal if the velocity distributions are in the range found in approach channels which are reasonably smooth, long and regular. Additional examples of velocity distribution effect can be found in ref. 4.8.2. In general the weir performance becomes more sensitive to velocity distribution as the head-weir height ratio increases.

4.4.7. Supercritical Flow Channels

If the channel flow is supercritical, the weir has to be high enough to cause a hydraulic jump upstream. Reference 4.3.3 recommends that the jump be upstream a distance of at least 30 times the maximum head.

4.4.8. Errors in Head Measurement

The comments in section 2.4.3 on head measurements for Parshall flumes are also valid for weirs. An additional factor here is that for triangular weirs, in which discharge varies with head to the five-halves power, errors in head measurement are correspondingly more important in determining the total system error.

4.4.9. Combined (System) Error

The foregoing sections have called attention to a number of possible error sources as an aid to the user in identifying some of the error components which contribute to the total error of the measurement. However, as discussed in the Introduction and in section 2.4.8, the estimates given usually will not account for all the factors which enter into <u>field</u> performance, particularly those associated with maintenance, aging and wear. Readers are referred to section 2.4.8 for discussion and an illustrative example (for Parshall flumes) which can be adapted to thin plate weirs.

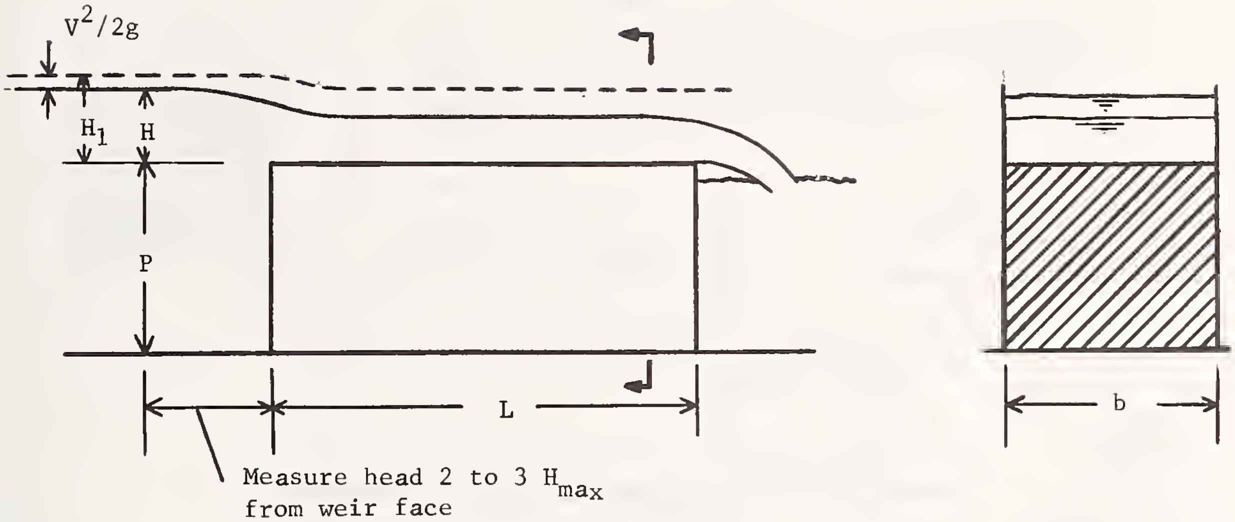
4.5. Weirs in Circular Channels

V-notch and contracted rectangular weirs as described in section 4.1 are frequently installed in sewers or other circular conduits. For the sake of additional information, work by Diskin (4.8.3) is cited here, in which several types of thin plate weirs were installed at the end of a circular pipe. These weirs included a "suppressed" (horizontal crest) weir across the conduit, vertical slots, V-notches with apex at the pipe invert, and trapezoids starting at the pipe bottom. Diskin's experiments included only one pipe diameter, 10 inches.

In connection with weirs for sewer gaging, Mort (4.8.4) describes a few practical items, such as a method for shaping bulkheads to fit a sewer and a device to help set the head zero.

4.6. Maintenance

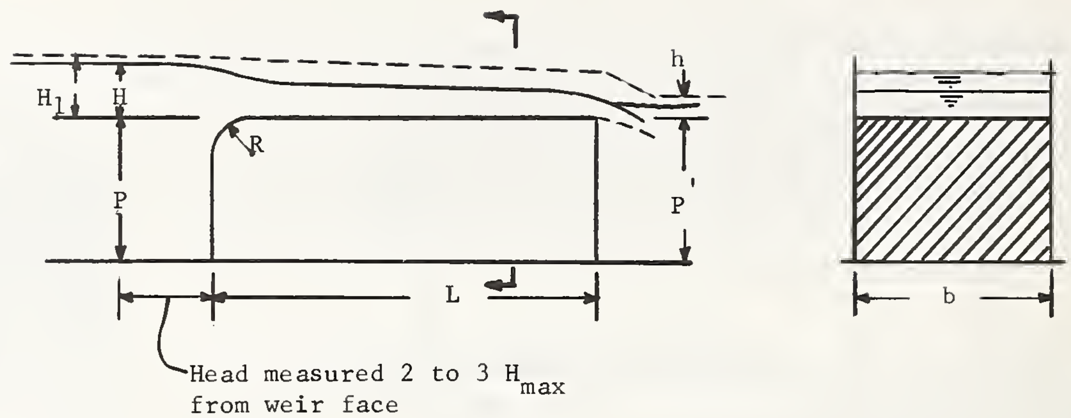
Frequent cleanout of debris collected behind a weir and on the crest edge is imperative. Solids accumulated behind the weir in effect raise the bottom and introduce errors associated with increased values of H/P.



<u>Recommended limits on geometry (ref. 5.3.2.)</u>

$H > 0.2$ ft
$b > 1$ ft
$P > 0.5$ ft
$0.08 < H/L < 0.85$
$0.18 < H/(H + P) < 0.6$
Tailwater energy level above crest should not exceed $2H_1/3$ for free flow.

Figure 5.1a. The square-edge broad-crested weir.



Recommended limits on geometry (ref. 5.3.2.)

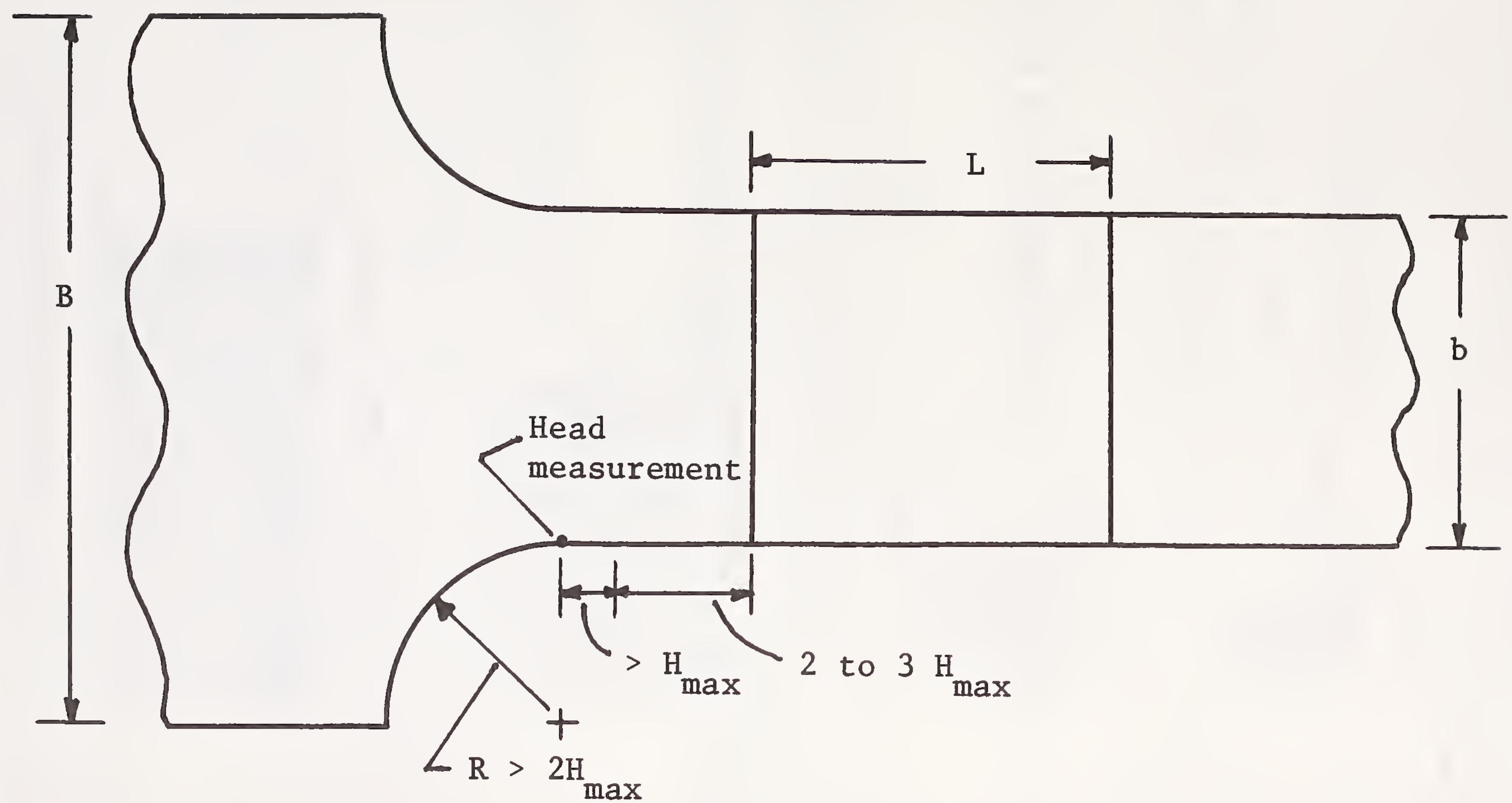
$R > 0.2\ H_{max}$
$H > 0.2$ ft (or 0.03L, whichever larger)
$H_1/P < 1.5$
$H_1/L < 0.57$
$P > 0.5$ ft
B_{min} is largest of H_{1max}, L/5 or 1 ft

Maximum submergences, h/H_1, for free flow

Vertical downstream face:
$H_1/P' < 0.5$, $h/H_1 \approx 0.66$
$H_1/P' \approx 0.5$, $h/H_1 \approx 0.75$
$H_1/P' > 1.0$, $h/H_1 \approx 0.80$

Sloping downstream face:
Add 0.05 to above values of h/H_1

Figure 5.1b. The rounded-edge broad-crested weir.



TOP VIEW

Figure 5.1c. Contraction from wider channel to broad-crested weir, as recommended in ref. 5.3.2.

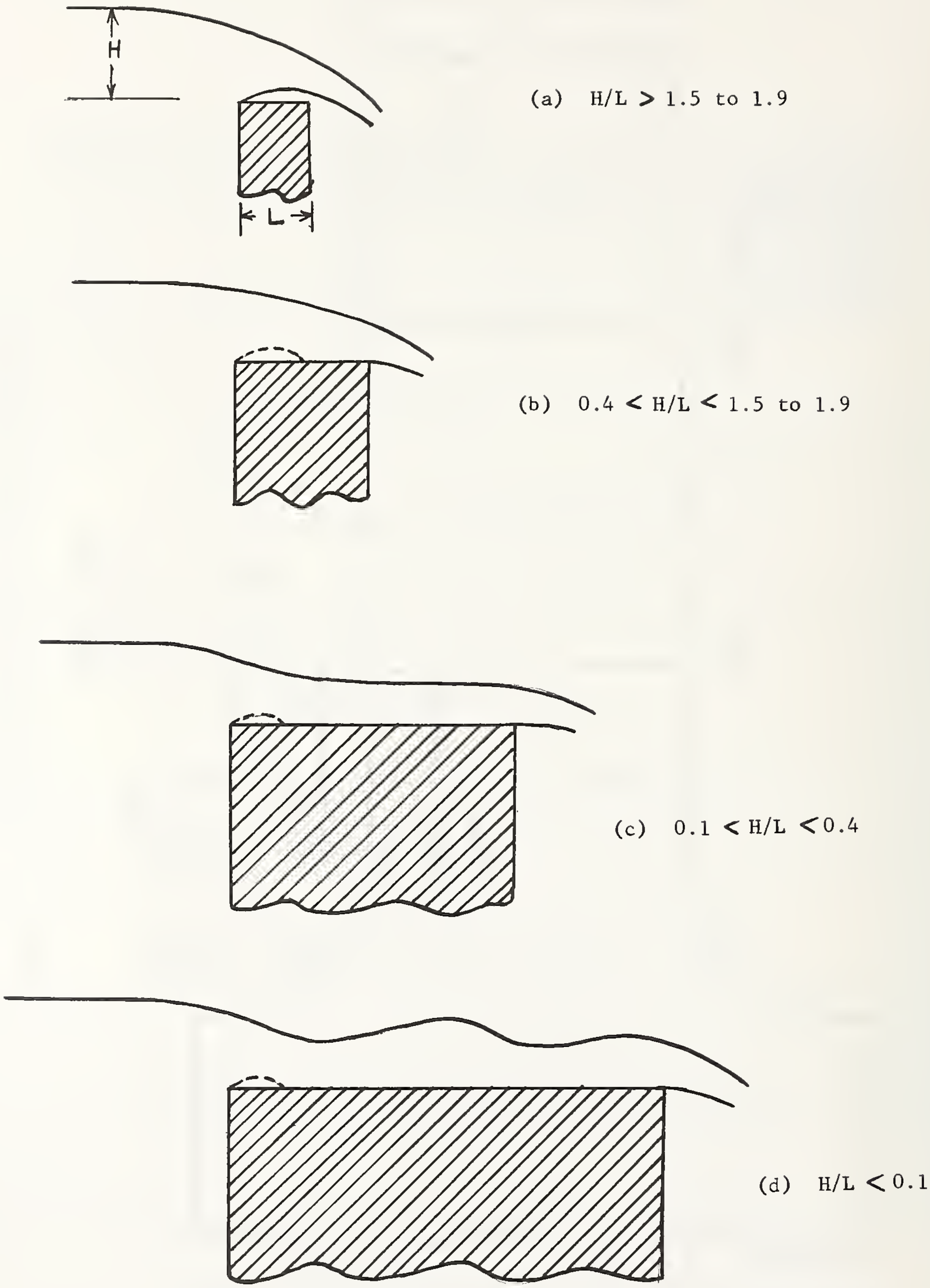


Figure 5.2. Flow regimes for square-edge broad-crested weirs.

4.7. Field Calibration

1. By current meter (see Chapter 8). Often the velocities in the weir box immediately upstream of the weir will be too low for accurate calibration by velocity traverse. In example 1 of section 4.4.2.1, the approach velocity is 0.18 ft per second. Therefore any velocity traverse would have to be made in a higher velocity section of the main channel upstream or downstream.

2. By dilution methods (see Chapter 9).

4.8. References

4.8.1. Johnson, J. W., "The Aeration of Sharp Crested Weirs," Civil Engineering, 5, 3, 1935, pp. 177-178.

4.8.2. Schoder, E. W. and Turner, K. B., "Precise Weir Measurements," Trans. Amer. Soc. Civ. Eng., 93, paper 1711, 1929, pp. 999-1190.

4.8.3. Diskin, M. H., "Sharp Crested Weirs for Circular Channels," Water and Water Eng., 75, Aug. 1971, pp. 309-313.

4.8.4. Mort, S. F., "The Practical Gaging of Dirty Water and Its Application to Sewer Design," Proc. Inst. Civ. Eng., 1955, Part 3, No. 1, pp. 81-113.

5. BROAD-CRESTED WEIRS

5.1. Background

Broad-crested weirs have not been used for flow measurement in this country as extensively as thin plate weirs and Parshall flumes. Nevertheless they have certain advantages and applications (see section 5.2) which warrant discussion.

The broad-crested weir can be rectangular, triangular or trapezoidal in lateral section. Only the rectangular sections will be considered here. A cross-section in the flow direction can be either a rectangle, i.e., a square-edge weir, or a rectangle with rounded upstream corner. See figure 5.1.

5.1.1. Square-Edge Broad-Crested Weirs

The flow over square-edge broad-crested weirs can occur in different regimes (5.1), depending upon the weir head-to-length ratio, H/L. See figure 5.2.

The usual "broad-crested" weir range for square edges is shown in figure 5.2c and prevails for H/L between about 0.1 and 0.4. In this range there exists essentially parallel flow over most of the crest. However, the upstream sharp edge is the control point. The flow passes through a critical condition in that region and remains supercritical everywhere downstream. If H/L becomes smaller than about 0.1, the flow has the length needed to change back to subcritical, and the control shifts to the downstream overfall. This range is sometimes called the "long-crested weir." If H/L is larger than 0.4, the flow becomes curvilinear throughout. The limiting H/L values given in figure 5.2 are approximate and will vary somewhat with flow and weir conditions.

The basic broad-crested weir equation is

$$Q = (2/3)\ (2g/3)^{1/2} C_D b H_1^{\ 3/2} = C_1 b H_1^{\ 3/2} \qquad [5.1a]$$

or

$$Q = (2/3)\ (2g/3)^{1/2} C_D C_v b H^{3/2} = CbH^{3/2} \qquad [5.1b]$$

$$C_v = (1 + \alpha V^2/2gh)^{3/2}$$

where b is the weir width (transverse to the flow direction), H_1 is the total head (measured head H plus the effective approach velocity head). The coefficient C_D is empirical in the case of the square edge weir and is often combined into the overall coefficients C and C_1; however, in rounded edge weirs, C_D accounts for frictional effects and can be estimated analytically (see section 5.1.2).

The user is cautioned, when using published coefficients, to note whether they are actually C or C_1. In this section all values are based on total head. A successive approximation procedure is required to convert from measured head, H, when approach velocity is appreciable, but use of H_1 is somewhat more frequent in publications.

In the broad-crest range ($0.1 < H/L < 0.4$, approximately) C_1 is a constant at 2.62 ± 0.08, provided H/P is between 0.22 and 0.56 (5.3.2). P is the crest height above the channel floor. Some supporting data are shown in figure 5.3. This C_1 is for ft-sec units only.

In the range $H/L > 0.4$, recommended coefficients are given in figure 5.4 (5.3.2). For the most part, available data represent tests for H/P less than about 1.0, and application should be restricted accordingly.

In the long-crest range ($H/L < 0.1$) published results differ, and the square-edge weir is in reality not well-suited for this range. Readers can find results in refs. 5.9.1 and 5.9.3.

The fact that there is considerable scatter and discrepancy in published results for the square-edge weir is not surprising. For $H/L > 0.1$ the upstream corner is the control. Therefore the results will be subject to effects due to lack of sharpness, roughness, etc., just as for thin plate weirs. At $H/L < 0.1$ the sharpness of the edge comes into play as an energy loss mechanism, as does the roughness of the weir surface. The user should consider these factors when applying published results.

5.1.2. Rounded-Edge Broad-Crested Weirs

In the rounded-edge broad-crested weir (figure 5.1b) the nose radius should be at least $0.2H_{max}$ (5.3.2) in order to prevent separation. Then the control point is at the overfall and hydrostatic critical flow exists near the overfall in the nearly parallel part of the flow, and it can be shown theoretically that

$$C_1 = (2/3)^{3/2} g^{1/2} (1 - 2\delta_*/b)\ (1 - \delta_*/H)^{3/2} \qquad [5.2]$$

Here δ_* is the boundary layer displacement thickness, which is described in all fluids textbooks and which is also discussed in ref. 5.3.1. A good analytical determination of C_1 therefore requires an estimate of the weir roughness and of the location of transition between laminar and turbulent flow in the boundary layer (5.3.1). Reference 5.3.2 suggests a method for estimating δ_* in terms of crest length so that the user does not have to resort to the boundary layer equations.

5.2. Uses

The square-edge broad-crested weir is simple in configuration and easy to build. It has a certain structural stability which thin plate weirs lack and permits a higher downstream water level without submergence effects than does the thin plate weir. On the other hand, it also possesses the main disadvantages of the thin plate weirs -- trapping of debris, sensitivity of discharge to edge and crest condition, and susceptibility to leading-edge damage.

Therefore, although much of the past research was concerned with the square-edge weir, there appears little reason to favor it over the rounded-edge broad-crested weir, particularly when one adds the consideration that the latter is amenable to analytical treatment over a considerable flow range ($H/L < 0.4$). See ref. 5.3.1. A rounded-edge broad-crested weir is similar to a rectangular flume with raised floor.

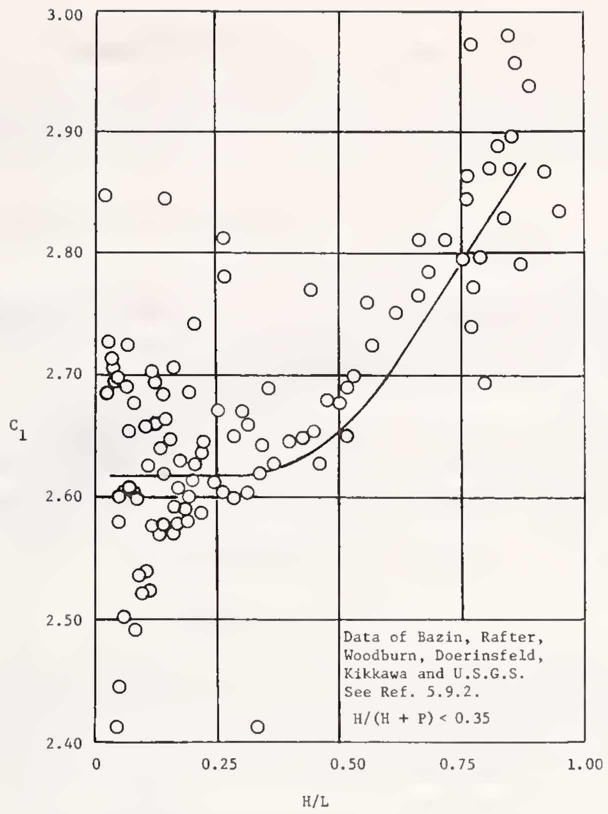


Figure 5.3. Discharge coefficient data for square-edge broad-crested weirs, ft-sec units.

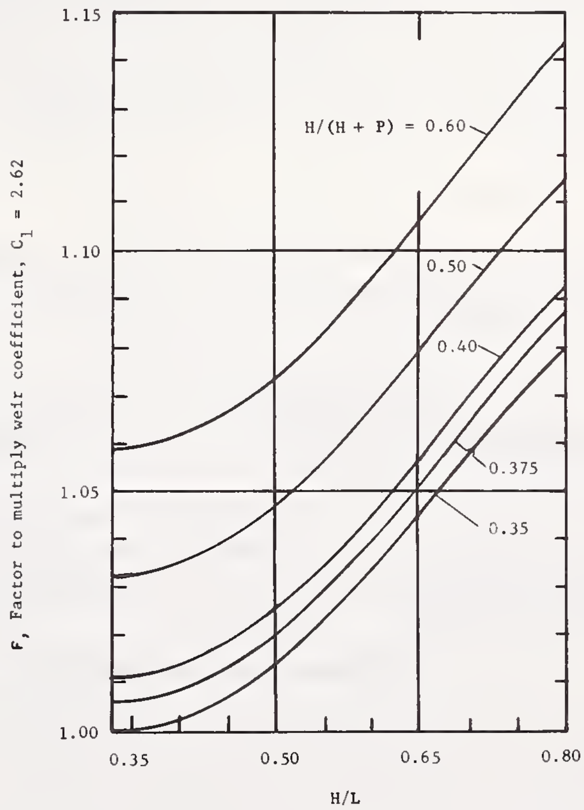


Figure 5.4. Discharge coefficients for square-edge broad-crested weirs including head-to-weir height ratio, from ref. 5.3.2.

5.3. General Information Sources

5.3.1. Harrison, A. J. M., "The Streamlined Broad-Crested Weir," Proc. Inst. Civ. Engrs., 38, 1967, Paper No. 7028, pp. 657-678. Obtain from Engineering Societies Library, 345 E. 47th St., New York, N. Y. 10017. \$0.25 per page plus \$3.00.

This paper gives an analytical treatment and reviews available data for weirs with rounded upstream corner.

5.3.2. British Standards Institution, Standard No. 3680-4B, "Methods of Measurement of Liquid Flow in Open Channels, 4B: Long-Base Weirs," 1969. Order from American National Standards Institute, 1430 Broadway, New York, N. Y. 10018. \$7.00.

This standard gives many details on the installation, use and determination of coefficients for broad-crested weirs. It also includes the triangular (Crump) weir, which has not yet been widely used in the United States.

5.4. Error Sources

5.4.1. General

The accuracy of the final measurement depends upon the performance of the instrument as a system -- (a) the primary element (weir), (b) the secondary elements (depth sensor and recorder), and finally (c) the manner in which these elements are installed and maintained. This section and the following sections deal with these aspects.

5.4.2. Head-Discharge Relations

Square-edge weirs. Some of the available results for square-edge weirs are shown in figure 5.3 and these data afford some idea of the expected precision. The British Standards Institution (5.3.2) suggests using the curves of figure 5.4 to extend the constant coefficient $C_1 = 2.62$ to higher values of H/L. The error in head-discharge relation for a carefully fabricated and installed square-edge weir can be estimated from

$$\text{Error} = \pm\ (10F - 8) \text{ percent} \qquad [5.3]$$

where F is as defined in figure 5.4.

Rounded-edge weirs. Some published results for the round-edge weir are shown in figure 5.5. Reference 5.3.2 suggests an estimated error in the head-discharge relation of a carefully fabricated round-edge weir of

$$\text{Error} = \pm\ 2(21 - 20C_D) \text{ percent} \qquad [5.4]$$

where $$C_D = (1 - 2\delta_*/b)(1 - \delta_*/H)^{1.5} \qquad [5.5]$$

The data of figure 5.5 substantiate that in regions of high C_D, i.e., H/L between about 0.2 and 0.5, the scatter is not large. It is emphasized that the error estimates of eqs. 5.3 and 5.4 should be applied only to carefully fabricated weirs which are new. There is no way to assess the effects of aging and wear without direct field measurement.

5.4.3. Departures from "Standard" Geometry

Dimensions should be adhered to within 1 part in 400 (5.3.2). Of course differences in weir width b are accounted for directly in the discharge equation, provided the sidewalls remain parallel.

Level. Downward slopes are often caused by settlement due to downstream scour in earth channels. Rounded-edge weirs with downward slope will have the control point (critical depth) shifted from the overfall to the upstream end of the weir at some value of this slope. This value of weir slope depends upon geometry and roughness but it can be as small as 0.002.

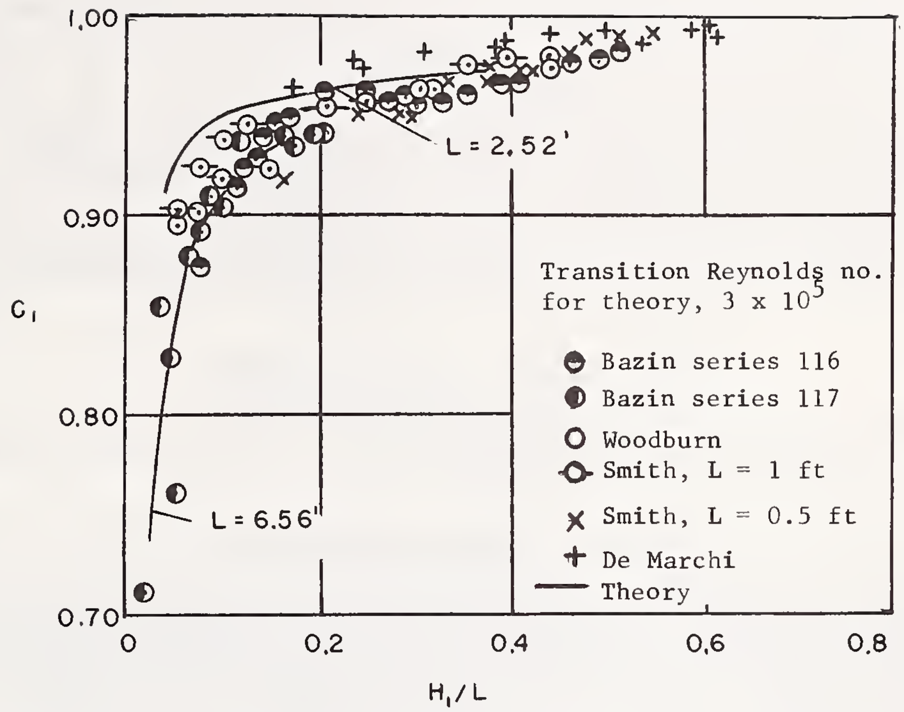


Figure 5.5. Discharge coefficient data for rounded-edge broad-crested weirs.

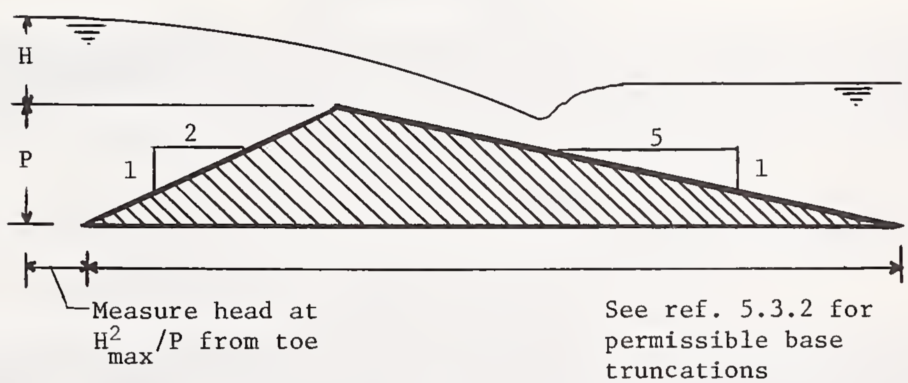


Figure 5.6. The triangular weir.

As an illustration of the error so introduced, in ref. 5.9.3 rounded-edge weirs with low values of H/L were tested level and at slopes of 0.004 and 0.026. Comparing results for H/L = 0.10 shows that discharge coefficients are increased over the level condition by about 1.5 percent and 5.5 percent for the 0.004 and 0.026 slopes, respectively. No information is available for higher values of H/L.

5.4.4. Velocity Distribution Upstream

The effect of the upstream velocity distribution can be estimated analytically only for those weir flows in which the control point (critical depth) is at the downstream brink, i.e., mainly rounded-edge weirs and, to some extent, "long-crested" square-edge weirs. If the approach velocity head is very small relative to the measured head, modest changes in approach velocity distribution will not have a significant effect. For example, if $\alpha V^2/2g$ (see section 2.4.5) is less than .02H, even a 25 percent change in approach α will cause only a 0.5 percent effect on the head. For large values of velocity head, a rational correction procedure for increased α is as follows. Assume that for the same Q, the overfall conditions and therefore the total head at the overfall remain the same. Therefore, the upstream total head, H_1, should remain the same, ignoring small differences in energy losses. However, the measured head, H, will decrease by an amount $\alpha V^2/2g$ as α is increased.

5.4.5. Errors in Head Measurement

These errors are the same as those encountered in flumes and thin plate weirs. See section 2.4.3. See also ref. 5.3.2 for general instructions on head-measurement installations.

5.4.6. Combined (System) Error

The foregoing sections have indicated some of the error sources which contribute to the total error of measurement. Estimates of combined error may be meaningful in evaluating the performance of new weirs which have been fabricated and installed in accordance with recommended practices. However, errors due to aging, wear or less-than-optimum field and installation conditions can best be determined through an independent and more accurate measurement on site. Users are referred to the Introduction and to section 2.4.8 for additional discussion and an illustrative example of combined errors in Parshall flumes which can be adapted to broad-crested weirs.

5.5. Triangular Weirs

Triangular weirs (figure 5.6) warrant mention in this chapter even though they are not, strictly speaking, weirs with finite crest width. This type of weir is able to pass some debris and still retain some of the advantages of a sharp edge. Information on this weir is available in ref. 5.3.2. The weir coefficient is

$$H < 0.5 \text{ ft}: \quad C_D = 1.150\ (1 - 0.001/H)^{3/2} \qquad [5.6]$$

$$H > 0.5 \text{ ft}: \quad C_D = 1.150 \qquad [5.7]$$

5.6. Installation

Broad-crested weirs should be installed so that they never become submerged, i.e., so that they operate independently of tailwater elevation change. Reference 5.3.2 gives details on the limiting downstream heads for achieving this condition.

Upstream of the weir, the channel should be straight and reasonably uniform for a distance of at least 10 b. If straightening vanes can be used, they should not come closer than 10 H_{max} to the measuring station (5.3.2). If the channel is steep and flow is supercritical, the weir should be designed so that the hydraulic jump occurs at least 30 H_{max} upstream (5.3.2).

A recommended convergence for installing weirs in wider channels is shown in figure 5.1c.

5.7. Maintenance of Weir Structure

The channel section upstream of the weir should be cleaned periodically of silt and debris, and the weir itself must be checked for corner and surface damage. See 3.4.2.

5.8. Field Calibration

1. By current meter traverse (see Chapter 8).

2. By dilution methods (see Chapter 9).

5.9. References

5.9.1. Govinda Rao, N. S. and Muralidhar, D., "Discharge Characteristics of Weirs of Finite Crest Width," La Houille Blanche, Aug. - Sept. 1963, pp. 537-545.

5.9.2. Harrison, A. J. M., "Some Comments on the Square-Edged Broad-Crested Weir," Water and Water Engineering, London, Nov. 1964.

5.9.3. Woodburn, J. G., "Tests of Broad-Crested Weirs," Trans. Amer. Soc. Civ. Eng., 96, 1932, pp. 387-453.

6. PRICE AND PYGMY CURRENT METERS

6.1. Background

Vertical axis meters with rotating bucket wheels are popular for field use in this country, so a separate chapter is being devoted to them. The principal instrument in this group is the Price meter (figure 6.1). The Pygmy meter (figure 6.2) is, roughly speaking, a two-fifth scale replica of the Price meter and was developed later for use in small channels. Many of the investigations of these meters have been performed by personnel of the U. S. Geological Survey and are thoroughly described in the publications of that agency, some of which are referred to here.

Price meters can be rod mounted and used by a wader (or in the case of very small channels, by an operator on a bank or bridging plank) or they can be suspended by a cable and weight (see figure 6.3). The Pygmy meter is used only with rod mounting.

6.2. Uses

The most frequent application of this class of current meter is in the determination of volumetric flowrate through a section by velocity-area integration. This procedure is covered in Chapter 8; in this chapter only the performance characteristics of the current meter itself are considered.

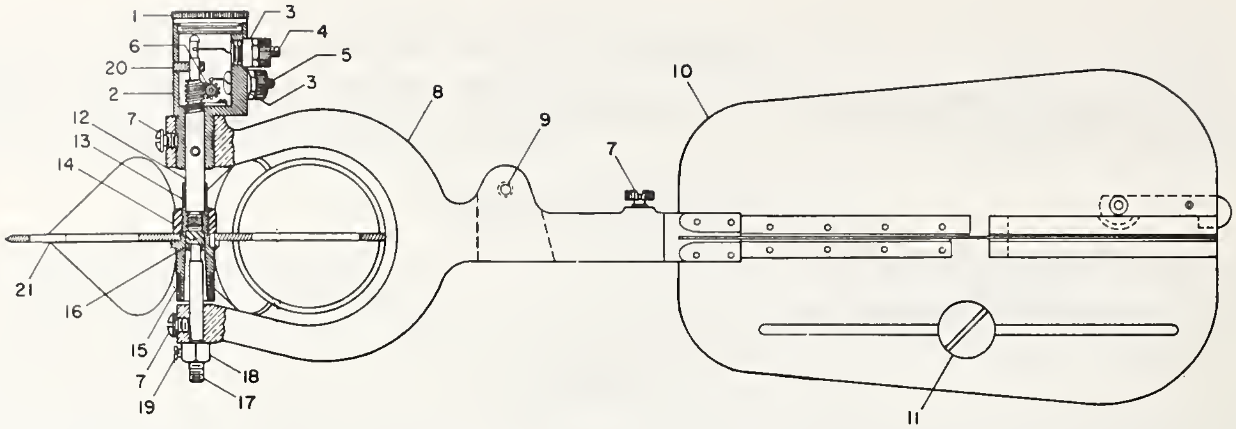
In comparison with horizontal-axis (propeller) meters the Price and Pygmy meters have the following advantages:

1. Their threshold velocities are usually lower;

2. The lower pivot bearing operates in an air pocket, so the likelihood of silt intrusion is reduced;

3. The Price meter in particular has earned a reputation for sturdiness and reliability under field use.

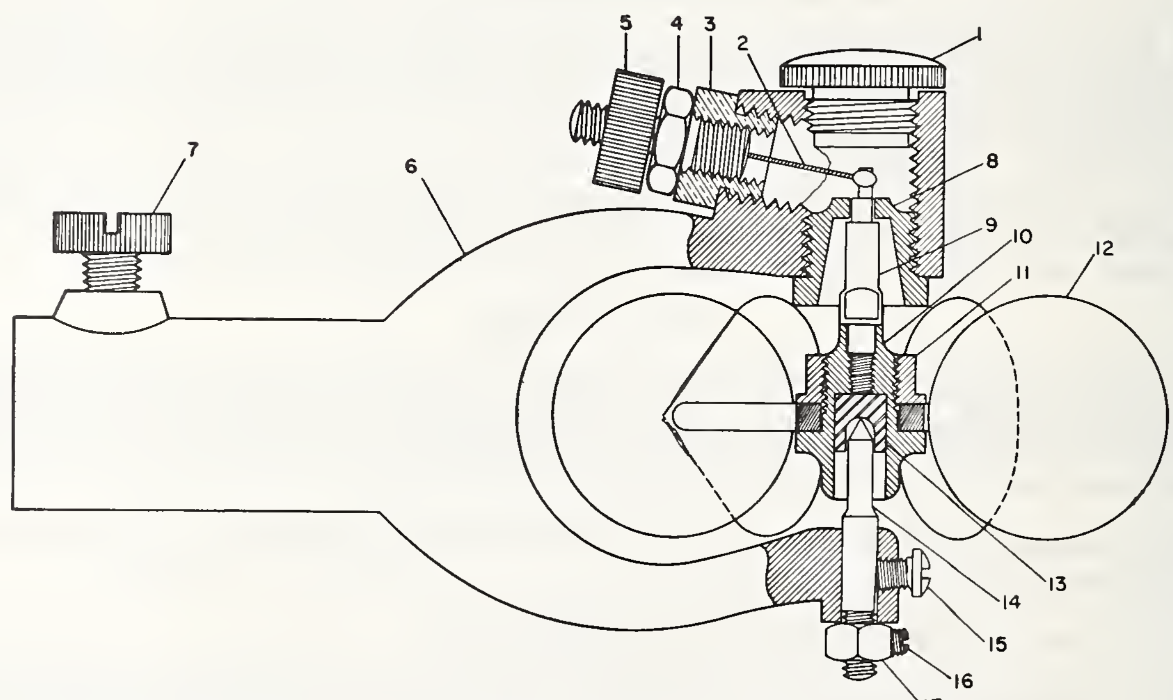
The main disadvantage of the Price and Pygmy meters is that they are not sensitive to the angle of approach of flow in the horizontal plane. As an extreme example, these meters rotate in the same direction and at almost the same rate for a downstream velocity, V, and for an upstream velocity, -V, of the same magnitude. Also, the meters respond to vertical components of velocity with the same direction of rotation, so that vertically oscillating platform motions appear in the record as an apparent positive velocity. See section 6.5.4 and ref. 6.6.2.



Wheel diam. = 5 in

1. Cop for contact chomber
2. Contoct chomber
3. Insuloting bushing for contoct binding post
4. Single—contact binding post
5. Pento—contact binding post
6. Pento geor
7. Set screw
8. Yoke
9. Hole for honger screw
10. Toilpiece
11. Balance weight
12. Shoft
13. Bucket—wheel hub
14. Bucket—wheel hub nut
15. Raising nut
16. Pivot bearing
17. Pivot
18. Pivot-odjusting nut
19. Keeper screw for pivat-adjusting nut
20. Beoring lug
21. Bucket wheel

Figure 6.1. Price current meter, from ref. 6.3.3.



1. Cop far cantact chomber
2. Binding-post beaded wire
3. Binding-past insulating bushing
4. Binding-past body
5. Binding-post nut
6. Yake
7. Yoke set screw
8. Upper bearing
9. Shaft
10. Bucket-wheel hub
11. Bucket-wheel hub nut
12. Bucket wheel
13. Pivat bearing
14. Pivot
15. Pivat set screw
16. Pivat-adjusting nut keeper screw
17. Pivat-odjusting nut

Wheel diam. = 2 in.

Figure 6.2. Pygmy current meter, from ref. 6.3.3.

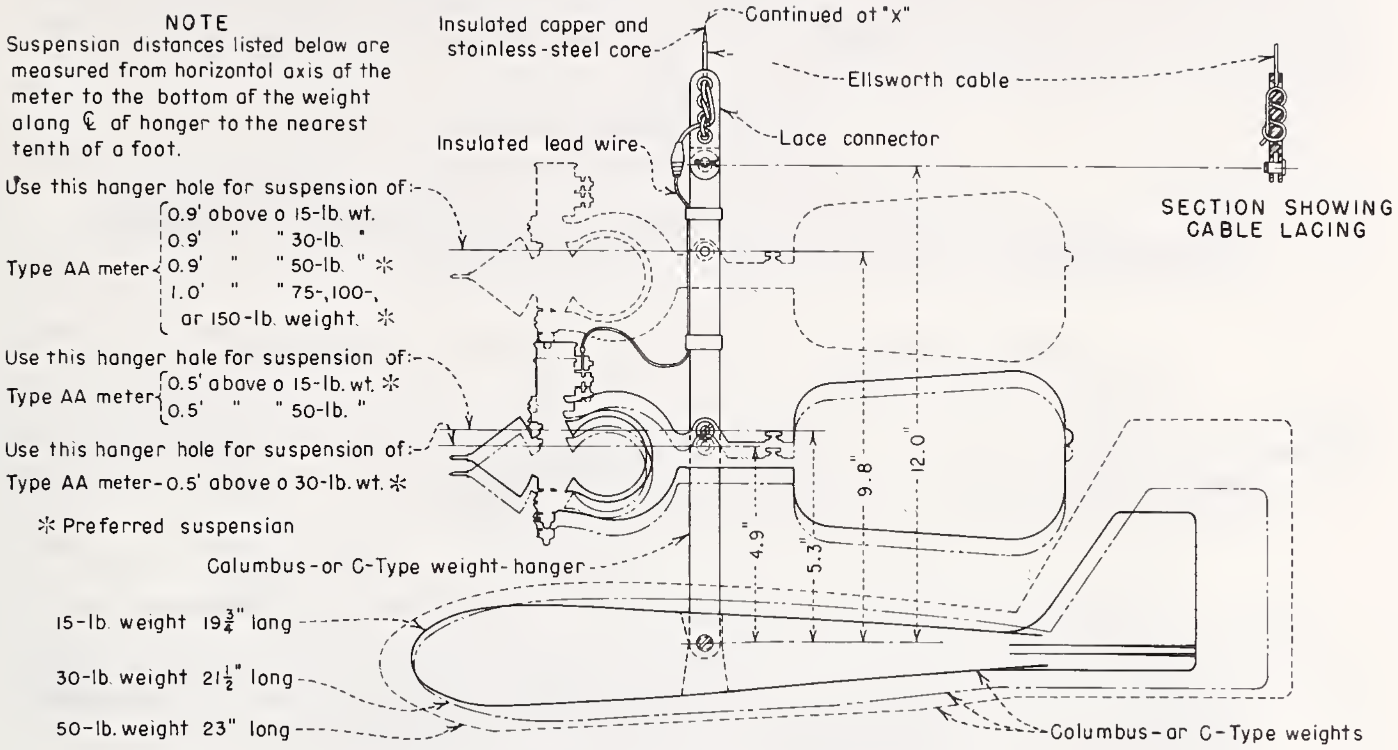


Figure 6.3a. Price meter with Columbus weight, from ref. 6.3.2.

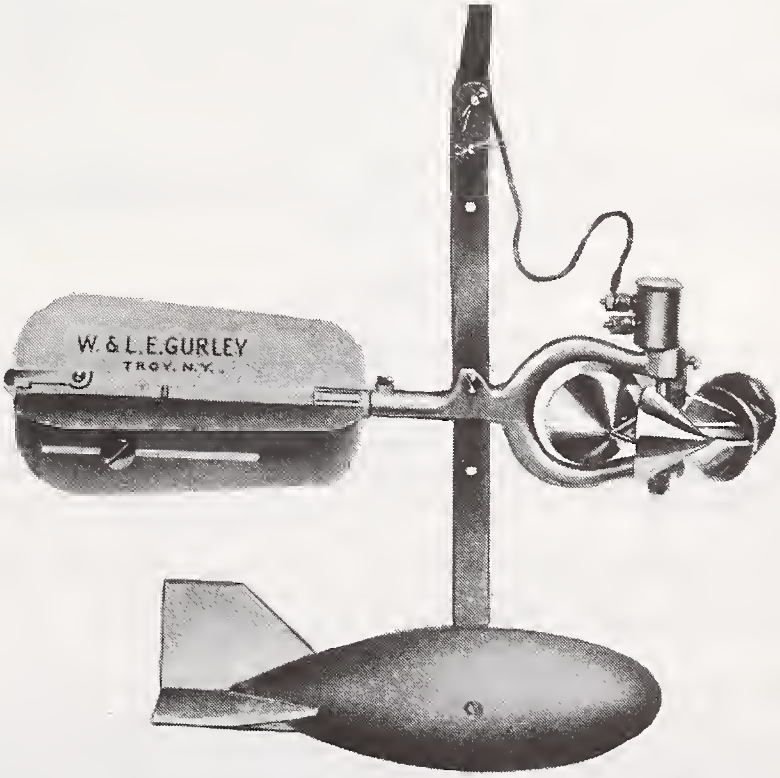


Figure 6.3b. Price meter with Elliptic weight.

The Price meter is well-suited for use in natural streams and in artificial open channels (even those containing some silt) provided the measuring station and methods are chosen to minimize the disadvantages cited in the previous paragraph. Price meters have even reportedly been used in domestic sewage for very brief periods of immersion, although they are generally considered more susceptible than propeller meters to fouling by large debris. Pygmy meters are advantageous in small and shallow channels.

6.3. General Information Sources

6.3.1. Buchanan, T. J. and W. P. Somers, "Discharge Measurements at Gaging Stations," Book 3, Chap. A8 of Techniques of Water Resources Investigations of the U. S. Geological Survey, 1969. U. S. Govt. Printing Office, Wash., D. C. 20402, $0.70. Cat. No. I.19.15/5:bk 3, Chap. A8, S/N 2401-0498.

This document gives a good general overview of current meters and auxiliary flow measuring equipment, their proper use and their application to streamgaging.

6.3.2. "Water Measurement Manual," Bureau of Reclamation, U. S. Dept. Interior, Second Ed., Revised Reprint, 1974, Chap. 5, U. S. Govt. Printing Office, Wash., D. C. 20402. $5.80. Cat. No. I27.19/2:W29/2, S/N 2403-0027.

This reference reviews current meter application to streamgaging, and for this purpose it could be used as an alternate to ref. 6.3.1 above.

6.3.3. Smoot, G. F. and Novak, C. E., "Calibration and Maintenance of Vertical-Axis Type Current Meters," Book 8, Chap. B2 of Techniques of Water Resources Investigations of the U. S. Geological Survey, 1968. U. S. Govt. Printing Office, Wash., D. C. 20402. $0.40. Cat. No. I19.15/5:bk.8, Chap. B2, S/N 2401-0504. Note: This document is being reprinted. Check price before ordering.

This document gives the most detailed information available on the actual handling of meters for maintenance purposes, e.g., assembly, lubrication, etc.

6.3.4. Smoot, G. F. and R. W. Carter, "Are Individual Current Meter Ratings Necessary?", Proc. Amer. Soc. Civ. Eng., 94, HY2, Mar. 1968, 391-397. Obtain from Engineering Societies Library, 345 E. 47th Street, New York, N. Y. 10017. $0.25 per page plus $3.00.

This paper gives useful information on the effect of bearing wear on Price meter performance and on the variability of calibrations of meters from the same and from different manufacturers.

6.4. Calibration

6.4.1. Calibration Equations

Price meters. Price meters are usually calibrated over the range 0.25 to 8.0 ft. per second. The calibration curve of velocity, V, versus revolutions per second, N, can be closely approximated by two straight lines intersecting at N = 1. For example, typical calibration equations might be:

$$N < 1, \; V = 2.198N + 0.017$$

$$N > 1, \; V = 2.182N + 0.033$$

These results are sometimes closely approximated by a tabular chart for convenience.

A calibration equation is supplied by the manufacturer at the time of purchase. This could be either a group average calibration equation or an individual calibration. Individual calibration curves (not the separate calibration points) which are run repetitively are repeatable to well within 1 percent over practically the entire velocity range. In ref. 6.3.4 it is shown that a group or batch calibration equation can adequately represent the performance of individual Price meters from the same manufacturer. However, using an average or "class" curve for instruments of three manufacturers resulted in differences as high as 2 percent even for velocities greater than 0.5 ft per second. If accuracy of better than about $\pm$ 3 or 4 percent is desired at the low end of the calibration range (V near .25 ft per second) individual calibrations are desirable. The need for individual calibrations is increased if data are to be taken at velocities less than 0.25 ft. per second, in which case the calibration velocities should be extended to below 0.25 ft per second. Note: The U. S. Geological Survey does not recommend use of these meters below 0.2 ft per second unless absolutely necessary. See also section 6.5.6.

Pygmy meters. Pygmy meters are generally rated over the range 0.25 to 3.0 ft per second, and the calibration curve is closely approximated by a single straight line equation, e.g.,

$$V = 0.970N + 0.030$$

Pygmy meters are sometimes used with a class calibration of V = N, where V is velocity in ft per second and N is rate of rotation in revolutions per second. Although it appears (based on limited data) that this might be accurate within about 3 percent or so for velocities higher than 1 ft per second, the velocities so obtained at 0.25 ft per second are likely to be considerably more in error and individual calibrations are recommended if greater accuracy is needed.

Although conflicting opinions are reported in the literature, it appears that these meters (Price and Pygmy) are not subject to significant temperature effects.

6.4.2. Maintenance of Calibrations

No quantitative guidelines for frequency of recalibration of current meters have been published. According to Smoot and Carter (6.3.4) Price meters are not particularly sensitive to bearing condition, particularly at velocities higher than 0.25 ft per second. See also section 6.5.1. One can assume that, if the meter passes the 4-minute spin described in references such as 6.3.1, at least the meter assembly, bearing and contact friction are within tolerable limits. Pivot pins can be readily replaced in the field when necessary to reduce bearing friction.

On the other hand, Price meters are sensitive to dented cups (6.3.4). See also section 6.5.1. A meter with dented cups should be returned to the shop for rotor replacement, after which it is preferable that the meter be recalibrated.

Field spin test recommendations are also available for Pygmy meters (6.3.1).

6.4.3. Method of Support

According to refs. 6.3.1 and 6.3.3, the calibration equations are the same for the Price meter rod-mounted or suspended properly with a Columbus weight (figure 6.3). This conclusion is essentially supported by limited comparative tests made by NBS (unpublished) in which the differences in rod and cable (15 and 30 lb Columbus weights) ratings were less than 1 percent provided that the velocity was greater than 0.75 ft per second. A larger difference was indicated at the lowest velocities, but it is not certain yet whether this was a valid hydrodynamic effect or a spurious result of the calibration procedure.

When the Elliptic weight series are used (figure 6.3) one meter manufacturer recommends a 2 percent velocity increase over that indicated by rod ratings.

6.4.4. Calibration Method

Meters are usually calibrated by towing them through still water in a towing tank. Generally, two passes along the tank (in opposite directions) are made to cancel the effects of drift currents. The stilling time between runs needed to limit the effects of residual inertial currents has not been quantified.

There has always been a question as to whether a rating done in a towing tank properly reproduces conditions in turbulent flowing water. Very intense and large scale turbulence was generated by Yarnell and Nagler (6.6.1) by placing obstructions and paddles in a channel. The Price meter over-registered significantly under these conditions, as would be expected from the properties described in section 6.2. However, any station selected for current meter traversing should be free of this kind of "turbulence," if there is any choice. Because of the shape of the cups, there is probably little effect of very small scale turbulence, although there is no experimental verification for this statement.

Reference 6.6.4 suggests an over-registration for cup-type meters of about $0.2\ (\bar{u}/V)^2$ where $\bar{u}$ is the rms longitudinal turbulence and V is the time average velocity. This result pertains to low frequency turbulence, between 0.08 and 0.8 Hz, and for $\bar{u}$ small compared to V. Application of this guideline would result in a predicted over-registration of about 0.5 percent for a 15 percent turbulence level. Pending a systematic and thorough investigation of this problem by comparative calibrations in water tunnels and towing tanks, the safest policy is to select measuring stations which, at the least, have no obvious severe eddying motions or other evidence of intense, large scale turbulence. See also section 7.5.4.

6.5. Error Sources

6.5.1. Meter Condition Effects

Meter condition was mentioned in section 6.4.2 and quantitative information on Price meter condition effects is available in ref. 6.3.4. To illustrate the size of these effects, a worn pivot point can cause a Price meter rating change of 4 percent at 0.25 ft per second. However, at velocities greater than 1 ft per second, the change is not more than a few tenths of a percent. On the other hand, even "slightly dented" cups can cause an error of close to 1 percent at high velocities, increasing to more than 3 percent at 0.25 ft per second (6.3.4). Corresponding data for Pygmy meters are not available to the writers.

6.5.2. Turbulence Effects

Intense, large-scale eddying motions in the measuring section must be avoided because the bucket-type meters will over-register in such flows (6.6.1). See section 6.4.4 for additional discussion.

6.5.3. Rod Not Vertical

Data in ref. 6.6.1 indicate that the Price meter will not show any effects of rod tilt in the vertical plane for vertical angles up to 5 degrees, at least for velocities over 2 ft per second. For larger angles, the effect depends upon whether the effective vertical component of current is upward or downward on the meter. Curves can be found in ref. 6.6.1.

6.5.4. Platform Motion Errors

Vertical motions. Because the bucket-type current meters will rotate slowly in response to vertical relative motions, it is especially important to avoid such motions. An example is the rocking of the platform in waves when cable-suspension measurements have to be made from a boat. Data on vertical relative motions for Price meters are given by Kallio (6.6.2). Errors from this source become more important as the horizontal velocity decreases and the vertical velocities increase. For example, if the horizontal velocity (to be measured) is 0.6 ft per second, there is a negligible error for vertical velocities up to about 0.3 ft per second, but the error approaches + 30 percent if the vertical motions increase to 0.6 ft per second.

Horizontal motions. If a flow has horizontal axial oscillations superposed on it, the meter can properly integrate the result provided that the oscillation frequency is sufficiently low and provided also that the oscillation amplitude is small enough so that negative resultant velocities are not created. Some indication of "sufficiently low" frequency is available from limited data in ref. 6.6.1. A Price meter oscillating with 2 ft stroke in a 2 ft per second flow began to over-register measurably at frequencies higher than about 0.2 Hz.

However, transverse oscillations in the horizontal plane will cause over-registration even at very low frequency. For example, a Price meter immersed in a 2 ft per second flow but sinusoidally oscillating laterally in a ± 0.5 ft excursion with a 10-second period would record an average velocity 2.4 percent too high.

These platform motions can be considered to approximate the effects of large scale, low frequency turbulence.

6.5.5. Boundary Effects

Price and Pygmy meters should not be used within 0.5 ft and 0.3 ft, respectively, of the bottom. If used closer than these distances from the water surface, they will under-register. Also, there are limited data to suggest that they should not be used closer than 0.5 ft to the sidewalls. In general, the data available to provide corrections, should these proscriptions be violated, are too limited for reliable application.

6.5.6. Very Low Velocities

If a Price meter has to be used at velocities less than 0.25 ft per second (and such use should be avoided where possible), the calibration data should cover that range and the meter condition should be carefully and frequently monitored with spin tests thereafter. A routine calibration (to 0.25 ft per second) should never be extrapolated to 0.1 ft per second unless the meter has unequivocally passed a 4-minute spin test. For example, in unpublished NBS tests one Price meter with a 3-minute spin rotated 25 percent more slowly at 0.1 ft per second than the same meter with a spin test close to 5 minutes.

6.5.7. Oblique Currents

If for some reason the flow at a measuring station is oblique (in a horizontal plane) to the Price or Pygmy meter orientation, and the velocity directly in line with the meter is desired, the meter angle of approach θ, should be measured. For small θ, a correction can be made by multiplying the registered velocity by cosine θ. For larger θ, where yoke orientation would have an effect, see ref. 6.6.1.

6.5.8. Wading

Although procedures are recommended for minimizing body interference on wading measurements, e.g., refs. 6.3.1 and 6.3.2, there are limited data which suggest that velocities registered by waders might be about 2.5 percent too low (6.6.3).

6.6. References

6.6.1. Yarnell, D. L. and F. A. Nagler, "Effect of Turbulence on the Registration of Current Meters," Trans. Amer. Soc. Civ. Eng., 95, 1931, 766-860.

6.6.2. Kallio, N. A., "Effect of Vertical Motion on Current Meters," U. S. Geol. Survey Water Supply Paper 1869-B, 1966. U. S. Govt. Printing Office, Wash., D. C. 20404, $0.15.

6.6.3 Schoof, R. R. and F. R. Crow, Discussion of "Are Individual Current Meter Ratings Necessary?", Proc. Amer. Soc. Civ. Eng., 94, HY6, Nov. 1968, 1596-1601.

6.6.4. "Fluid Meters -- Their Theory and Application," Amer. Soc. Mech. Eng., 6th ed., 1971.

7. PROPELLER CURRENT METERS

7.1. Background

The most frequently encountered horizontal-axis current meters are the Neyrpic and Ott type meters, both of which employ helical-bladed propellers. See figures 7.1 and 7.2.

These meters are very widely used in Europe and most standardization efforts by international groups are devoted to these or similar meters. In the U. S. they are used to some extent for streamgaging but more frequently for determinations of volumetric flowrates through turbines and pumps.

The Haskell meter and the Hoff meter are propeller types which are occasionally used in this country, the latter frequently for measuring pipe flow. However, there is little recent technical information on them available in the open literature. See figure 7.3. They are discussed very briefly in ref. 7.6.1 and some information on their behavior in turbulence is given in ref. 6.6.1.

Propeller meters of the type commonly used in oceanographic studies are not included in this chapter.

7.2. Uses

The principal advantage of the propeller current meters (relative to Price and Pygmy meters) is that, with properly selected rotor, they can be relatively insensitive to obliqueness of the approaching current, i.e., for small angles of approach such a meter responds only to the component of velocity in line with the meter axis. (See section 7.5.5 for typical limitations.) This is the feature that makes them more suitable for velocity traverses at turbine intakes where oblique velocities are often encountered.

The main disadvantage of the Ott and Neyrpic meters is that, at least in the opinion of some users (ref. 7.6.1), they are not as sturdy under harsh field conditions as the Price meter.

7.3. General Information Sources

7.3.1. I. S. O. Standard No. 748, "Liquid Flow Measurement in Open Channels -- Velocity-Area Methods," 1973. Order from American National Standards Institute, 1430 Broadway, New York, N. Y. 10018. $12.95.

This standard is concerned with the total problem of flowrate determination, and it therefore has some sections on current meter use and the associated probable errors.

7.3.2. British Standards Institution, Standard No. 3680-3, "Methods of Measurement of Liquid Flow in Open Channels, Part 3: Velocity Area Methods," 1964. Order from American National Standards Institute, 1430 Broadway, New York, N. Y. 10018. $7.00.

This standard contains information similar to that of I. S. O. Standard No. 748, ref. 7.3.1.

7.3.3. Jepson, P., "Currentmeter Errors Under Pulsating Flow Conditions," J. Mech. Eng. Sci., 9, 1, 1967, pp. 45-54. Obtain from Engineering Societies Library, 345 E. 47th St., New York, N. Y. 10017. $0.25 per page plus $3.00.

7.3.4. Johnson, R. L., "Laboratory Determination of Current Meter Performance," Tech. Rep. 843-1, Division Hydraulic Lab., U. S. Army Eng. Div., North Pacific, 1966. Available at no charge (while supply lasts) from Division Hydraulic Laboratory, U. S. Army Engineer Division, North Pacific, Corps of Engineers, Bonneville, Ore. 97008.

This report is a thorough documentation of one agency's experience in determining propeller meter performance. Many factors affecting the calibration equations are discussed in detail.

7.4. Calibration

7.4.1. Calibration Equations

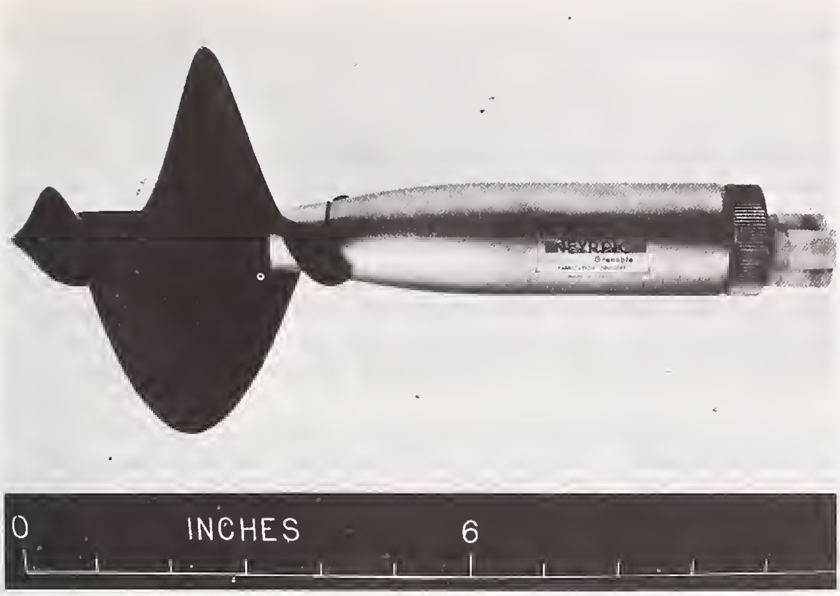


Figure 7.1. A Neyrpic current meter.

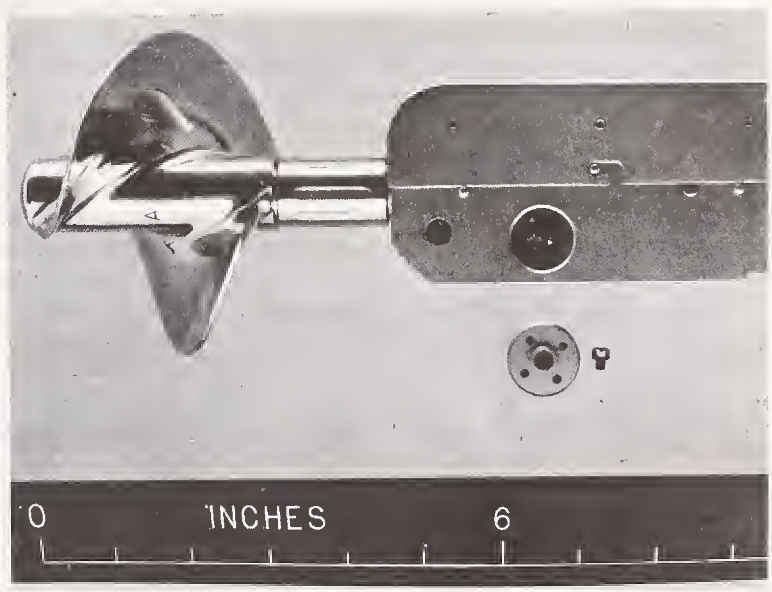


Figure 7.2. An Ott current meter.

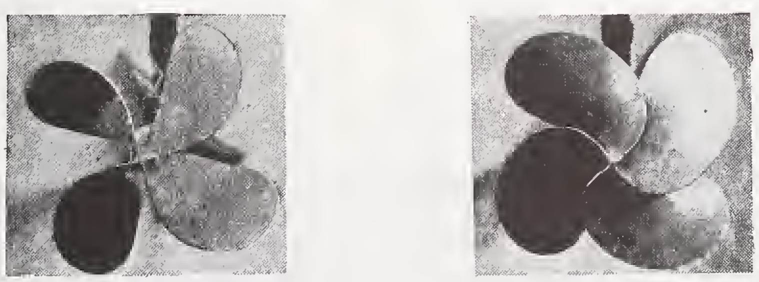


Figure 7.3a. Haskell current meter.

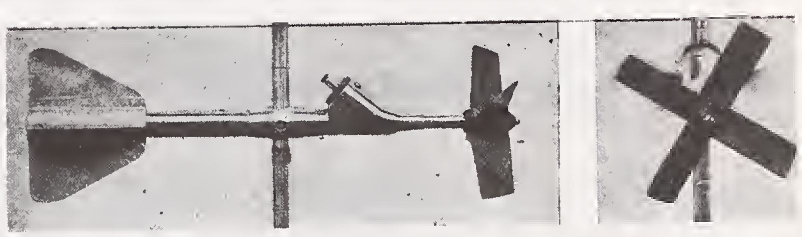


Figure 7.3b. Hoff current meter.

The calibration curves for Neyrpic and Ott meters are usually approximated by two (and sometimes three) straight-line segments. For example, an Ott rating equation will have the form

$$V = 0.4211N + 0.130, \qquad 1.00 < N < 12.08$$

$$V = 0.4264N + 0.066, \qquad 12.08 < N < 16.00$$

where V is velocity in ft per second, and N is rotation speed in revolutions per second. The above is only illustrative; propeller current meters frequently are supplied with two or more propellers of different pitch for use on the same bearings, and of course the numerical constants in the rating equations will change drastically with geometry.

Use of an average or group calibration is probably accurate within about 2 percent, at least for velocities greater than 1 ft per second. However, analysis of data presented by Johnson (7.3.4) for a group of Neyrpic meters suggests that the accuracy is less at 0.5 ft per second. Therefore, individual calibrations are recommended for low velocities. The manufacturer claims 1 percent accuracy if meters are calibrated individually.

7.4.2. Maintenance of Calibrations

There is little in the literature concerning recommended frequency of recalibration. Reference 7.3.2 recommends that individually rated meters be re-rated after 100 hours of use. If they are not re-rated then, the group calibration should be used thereafter. Manufacturers' maintenance instructions should be followed.

7.4.3. Method of Support

Although propeller meters can be used with both rod and cable suspension, there is no information in the literature about cable-suspended calibrations. Manufacturers' instructions on cable/weight as well as rod configurations should be followed closely. For example, for the particular vertical strut used in ref. 7.3.4 the Neyrpic meter rotated about 1 percent slower when the length of the horizontal sting was reduced from 34 inches to 13 inches.

7.4.4. Calibration Method

Propeller current meters are usually calibrated in relatively still water in a towing tank, but they are used in flowing water containing turbulent fluctuations of various scales and intensities. The turbulence effect on propeller meters is the subject of continuing research, and some indication of the upper limit of this effect is given in section 7.5.4.

7.5. Error Sources

7.5.1. Temperature

Many propeller meters have bearings which are immersed in oil. Temperature can affect meter performance by changing the viscosity of this lubricating oil. Therefore, it is important that the temperature of meter calibration be known and taken into account. Similarly, manufacturers' recommended lubricating oils should be used. If this is not possible, the substitute oil should have a viscosity close to that of the recommended oil (7.3.4).

For example, one particular meter in ref. 7.3.4 slowed about 1 percent at high velocities when the temperature was reduced "..... from 70°F to 40°F." At the lowest velocity, however, the corresponding change was almost 4 percent.

Additional data on temperature effect are available in refs. 7.3.4 and 7.6.3.

7.5.2. Calibration Error

Calibrations made in different towing tanks, i.e., in different calibration laboratories, can yield different results. For example, one Neyrpic meter in ref. 7.3.4 was rated in three laboratories at the same towing tank temperature. The results were within about 1 percent, i.e., $\pm$ 0.5 percent about an average, for velocities between 1 and 11 ft per second. However, at 0.5 ft per second this spread increased to more than 5 percent. The only accommodation that can be made with this state of affairs, apart from trying to avoid low-velocity measuring sections, is to consider these deviations when estimating the total uncertainty of a measurement.

For errors caused by use of a group (rather than an individual) calibration, see section 7.4.1 above.

7.5.3. Boundary Effects

Ott and Neyrpic meters are apparently quite insensitive to wall and bottom proximity, the effect generally being less than 0.5 percent even with the meter axis within one propeller diameter of the boundary (7.3.4, 7.6.4).

These results pertain only to geometric proximity effects at smooth walls and bottom. A bottom roughened by large geometric features, e.g., dunes; will shed local vortices which can cause meter registration errors. Therefore, close proximity to rough boundaries should be avoided in spite of the data cited herein. See section 7.5.4.

7.5.4. Turbulence and Velocity Pulsation Errors

In general, propeller meters over-register because of longitudinal fluctuations and under-register in the presence of transverse fluctuations. Natural turbulence includes both fluctuations, but the effects do not necessarily compensate.

Reference 7.6.5 assigns to turbulence errors an upper limit of $(0.5)\ (\Delta V/V)^2$ with $\Delta V = (u^2 + v^2 + w^2)^{1/2}$, where u, v and w are the turbulence fluctuations in three directions with respect to the time average velocity, V. If the turbulence intensity is 15 percent, there would be a velocity error (relative to a still-water calibration in a towing tank) not exceeding 1 percent, and the error would likely be negative. To give the reader an example of one set of conditions which produce a 15 percent turbulence level, approximately this intensity (in u^2 alone) was measured by Jonsson (7.6.7) near the bottom of a laboratory channel 4 ft wide and 0.5 ft deep which had been artificially roughened with transverse triangular ridges 0.5 cm high. McQuivey (7.6.8) lists turbulence intensities found in several field situations. It is clearly important to select measuring stations where the turbulence is not obviously intense.

Velocity pulsations. In ref. 7.3.3 the effects of velocity fluctuations were observed in a series of experiments in which Ott meters were oscillated both axially and transversely in flowing water. This type of experiment can be considered to approximate the effects of turbulence of low frequency and of a scale larger than the meter size.

The results can be described briefly as follows. For axial fluctuations, over-registration depended greatly on impeller geometry. For the same geometry it increased with increasing amplitude of the fluctuation, with increasing frequency, with increasing rotor inertia and with decreasing average velocity. The effect of transverse fluctuations depended mainly on meter geometry, but also increased slightly as the average velocity was reduced. However, a propeller geometry could be selected (in this case, a three-bladed propeller with 5 cm pitch and 3 cm diameter) which was almost insensitive to transverse fluctuations over the range of variables covered in ref. 7.3.3, i.e., frequencies 2.6 to 10.5 Hz, average velocities 2 to 4 ft per second, and amplitudes of the velocity perturbations from zero to the average velocity.

For the foregoing range of variables the errors for all the Ott propellers tested in ref. 7.3.3 were generally less than 1 percent provided the amplitude of the velocity perturbation did not exceed 0.2 V. This result is in agreement with the upper limit criterion cited in the second paragraph of this section (7.5.4).

7.5.5. Oblique Currents

Meters should be calibrated to check response to oblique approach flow, if this information has not been supplied by the manufacturer and if accuracy is desired. Even the so-called "component runners," which are intended to register only axial components, deviate from this condition as the approach angle increases. For example, Neyrpic meters tested by Johnson (7.3.4) indicated a 2.5 percent deviation at the high velocities for a 30° angle. However, for a 10° angle, the deviation was within 0.5 percent.

7.6. References

7.6.1. Buchanan, T. J. and W. P. Somers, "Discharge Measurements at Gaging Stations," Chapter A8, Book 3, of Techniques of Water Resources Investigations of the U. S. Geological Survey, 1969. U. S. Govt. Printing Office, Wash., D. C. 20402, $0.70.

7.6.2. British Standards Institution, Standard No. 3680, "Methods of Measurement of Liquid Flow in Open Channels: Part 3, Velocity Area Methods," 1964.

7.6.3. Coffin, J., "First Results Concerning the Effect of Water Temperature in the Calibrating Tank," Paper A-3, Flow Measurements in Closed Conduits, H. M. Stationery Office, Edinburgh, U. K., 1962.

7.6.4. Benini, G., "Researches on Mutual Interference and Wall Proximity Effects on Current Meter Readings," Paper A-2, Flow Measurements in Closed Conduits, H. M. Stationery Office, Edinburgh, U. K., 1962.

7.6.5. "Fluid Meters -- Their Theory and Application," Amer. Soc. Mech. Eng., 6th ed., 1971.

7.6.6. O'Brien, M. P. and R. G. Folsom, "Notes on the Design of Current Meters," Trans. Amer. Geophys. Union, 29, 2, Apr. 1948. (This reference was not included in the General Information Sources, because it does not contain experimental information directly useful in field applications. However, it is recommended for background information because it presents in an easily followed style the analytical rationale behind propeller meter performance.)

7.6.7. Jonsson, I. G., "On Turbulence in Open Channel Flow," Acta Polytechnica Scandinavica, No. Ci 31, 1965.

7.6.8. McQuivey, R. S., "Summary of Turbulence Data from Rivers, Conveyance Channels, and Laboratory Flumes," U. S. Geol. Surv. Prof. Paper 802-B, 1973, U. S. Govt. Printing Office, S/N 2401-02429, $1.55.

8. VELOCITY-AREA METHODS

8.1. Background

The velocity traverse is perhaps the most common field method for determining volumetric flowrate. It is also the most time-consuming method and is often unwieldy as well. Nevertheless, it is resorted to in the following situations:

1. Streamgaging of rivers and large artificial waterways. In this application, velocity traverses are often made upstream of a control point so that a stage-discharge relation for the stream can be established and periodically checked.

2. In small artificial free-surface conduits, including sewers, often for the purpose of making a field calibration of a flume, weir or other measuring device.

3. In large closed conduits, often to determine flowrate through turbines for field acceptance tests.

Procedures for the first application above, i.e., for streams and large artificial channels which generally have large width-depth ratios and reasonably gradual depth changes, are well established, and estimates of the errors involved are available, as in refs. 8.2.2 and 8.8.2. The third application has also received considerable attention, e.g., ref. 8.2.3. However, the second application can involve small rectangular sections of various width-depth ratios or part-full circular conduits, both with non-uniform roughness distribution. Also, immersion of velocity measuring instruments is severely restricted in some types of wastewater. Therefore, procedures and error estimates for this application are not as well formulated as in the other cases. In this chapter, we call attention to possible error sources.

8.2. General Information Sources

8.2.1. Buchanan, T. J. and W. P. Somers, "Discharge Measurements at Gaging Stations," Book 3, Chap. A8 of Techniques of Water Resources Investigations of the U. S. Geological Survey, 1969. U. S. Govt. Printing Office, Wash., D. C. 20402. $0.70. Cat. No. I19.15/5:bk3, Chap. A8, S/N 2401-0498.

> This reference gives considerable detail on the use of current meters for velocity traverses. In addition to covering the technical base for discharge measurements, it is strong on descriptions of field equipment including current meters, cables and weights, winches, sounding equipment, etc.

8.2.2. I. S. O. Standard No. 748, "Liquid Flow Measurement in Open Channels -- Velocity Area Methods," 1973. Order from American National Standards Institute, 1430 Broadway, New York, N. Y. 10018. $12.95.

> This document is particularly strong on estimation of errors associated with the velocity measurements and with the various methods of velocity-area traversing.

8.2.3. International Electrotechnical Commission, Publ. No. 41, "International Code for the Field Acceptance Tests of Hydraulic Turbines," 1963. Order from American National Standards Institute, 1430 Broadway, New York, N. Y. 10018. $36.00.

> This code describes several methods for measuring flowrate in open and closed conduits for turbine acceptance tests. The section on velocity-area current meter procedure is written for propeller meters only.

8.2.4. "Water Measurement Manual," Bureau of Reclamation, U. S. Dept. Interior, Second Ed., Revised Reprint, 1974, Chapters 5 and 8. United States Govt. Printing Office, Wash., D. C. 20402. $5.80. Catalog No. I27.19/2:W29/2, S/N 2403-0027.

> These two chapters give brief overviews on current-meter methods in open channels, although not with the detail of ref. 8.2.1, and of Pitot-tube traverses in pressure conduits.

8.2.5. British Standards Institution, Standard No. 1042-2A, "Methods for the Measurement of Fluid Flow in Pipes, Part 2: Pitot Tubes -- 2A, Class A Accuracy," 1973. Order from American National Standards Institute, 1430 Broadway, New York, N. Y. 10018. $13.00.

> This standard gives details on Pitot-static tubes and on traversing procedures which should result in an overall accuracy of 1 percent in pipe flow.

8.3. Errors in Streamgaging

8.3.1. Background

Streamgaging by velocity-area integration is normally accomplished by dividing the stream into vertical strips and obtaining an "average" velocity within each strip by measuring the velocity either (1) at many points throughout the depth (full profile), (2) at 0.2 and 0.8 depth from the surface, or (3) at 0.6 depth from the surface. (The three options were listed in order of decreasing accuracy.)

One method of estimating an overall measurement error is to use the square root of the sum of the squares of the error components. This was the method used in ref. 8.2.2 as is shown in the following:

The overall random standard error in discharge, X_{QR}, is (8.2.2)

$$X_{QR} = \pm \sqrt{X_m^2 + \frac{1}{m}\left(X_b^2 + X_d^2 + X_v^2\right)} \qquad [8.1]$$

where: (1) X_m is the error associated with the number of vertical sections. Typical values for a wide river with irregular bed profile are $\pm$ 5, 3 and 1 percent for 8, 15 and 50 vertical sections, respectively (8.2.2).

(2) X_b is the random error due to width measurement. This can be neglected if the number of vertical sections is large.

(3) X_d is the random error due to depth measurement. This has to be estimated for the individual case, as it clearly depends on the nature of the bottom, i.e., rigid or movable, the regularity of the bottom and the total depth.

(4) X_v is the average error of the mean velocity in all the verticals. It is defined as

$$X_v = \sqrt{\frac{X_f^2}{p} + X_o^2} \qquad [8.2]$$

where X_f is the standard deviation of a point measurement, p is the number of points in a vertical, and X_o is the error associated with the number of points. As a guide, X_f can be taken as 6 percent for 40-second readings (8.2.2). Also, as a guide only, X_o can be taken as $\pm$ 3.5, 3.0 and 0.5 percent for one-point (0.6 depth), two-point (0.2 and 0.8 depth) and full velocity profile methods, respectively (8.2.2). These are values obtained from river gaging.

(5) m is the number of vertical sections.

Different values for the various error components can be found in ref. 8.8.2. Collection of river flow data to improve these component error estimates is described in ref. 8.8.9.

In addition to the errors covered by eq. 8.1, systematic errors X_{QS} must also be considered. The major source here is the current meter calibration, and the error can usually be considered to be 1 percent at a maximum, provided very low velocity sections are avoided in accordance with the recommendation of refs. 8.2.1 and 8.2.2, and provided that the meter is individually calibrated. The overall error X_Q is then

$$X_Q = \pm \sqrt{X_{QR}^2 + X_{QS}^2} \qquad [8.3]$$

8.3.2. Numerical Example

Consider a stream in which 10 vertical sections and the 0.2 - 0.8 depth method are used with 40-second velocity readings. From the foregoing we select $X_f = 0.6$, $X_o = .03$, $X_d = .01$, $X_m = .04$, and a current meter error of 0.01. The overall standard error is then computed to be $\pm$ 4.4 percent. Double this value, $\pm$ 8.8 percent is the "tolerance," i.e., 95 percent of the measured discharges will be within these limits. The greatest improvement would be brought about by increasing the number of vertical sections. If 50 sections are used, the overall standard error is reduced to $\pm$ 1.6 percent.

When conditions differ substantially from those for which the error component estimates of ref. 8.2.2 or 8.8.2 were made, users have to assign their own best estimate of errors.

8.4. Errors in Small Conduit and Sewer Measurements

8.4.1. General

In this section we are concerned with flowrate measurements which are often made in a conduit in order to field check the performance of an open-channel metering device located downstream. In these cases the X_m term of eq. 8.1 is not so meaningful, because there is often little flexibility in the choice of number of vertical sections. Shallow flows often preclude any procedure but the one-point method in any vertical. Durations of velocity recording sometimes must be short to avoid fouling of the current meter. One element on the positive side is that depth measurement errors should be minimal owing to the fixed boundaries. In short, it is difficult to generalize limits of estimated errors and each case should be considered individually.

In the following, we attempt to obtain rough estimates of the errors caused by obtaining only a few point velocities in a small conduit and by short sampling times at each point.

8.4.2. Numerical Example -- Rectangular Section

Suppose figure 8.1 represents the "true" velocity distribution in an approach channel to a Parshall flume. The width is 4.0 ft, depth is 2.4 ft and 12 cfs are flowing, so the average velocity is 1.25 ft per second, high enough for reliable current meter behavior.

Divide the section into four vertical strips 1 ft wide. Assume, for the moment, that true point velocities are obtained. Then the two-point method (velocities measured at 0.2 and 0.8 depth) gives an average velocity 2.2 percent too high, an error which is due only to the spatial distribution of the velocity sampling. If, additionally, we allow 1.0 percent for meter error and 6 percent error for short sampling time at each of the eight points, the overall standard error is about $\pm$ 3.2 percent and the "tolerance" is $\pm$ 6.4 percent. The result could be improved most effectively by using more sampling points.

On the other hand, if the one-point (0.6 depth) method were used in each vertical strip, the error in average velocity alone would be 5.0 percent in this example, and the combined standard error would be $\pm$ 5.9 percent.

Note that if this traverse were being made with a Price meter, the meter positions would be just about at the allowable limits for wall proximity (see chapter 6). Therefore, if this same velocity distribution were found in a channel half the size, 2.0 x 1.2 ft, and only a Price meter were available, the one-depth method with two vertical sections would have to be used, and the overall error would be close to $\pm$ 10 percent. In this case, it would probably be better to violate the wall proximity rules and use four vertical sections. An alternative (not possible in sewage) would be to go to a Pygmy or propeller meter.

Table 8.1 summarizes the errors due to unrepresentative or incomplete spatial sampling for the cross-sections of figures 8.1 and 8.2.

Table 8.1. Examples of Percentage Errors in Spatial Sampling

Flow	No. of Verticals	0.6 Depth	0.2-0.8 Depth
Fig. 8.1a	4	+5.0	+2.2
Fig. 8.1a	2	+9.0	+5.0
Fig. 8.1b	4	+4.5	0
Fig. 8.1b	2	+6.0	+2.0

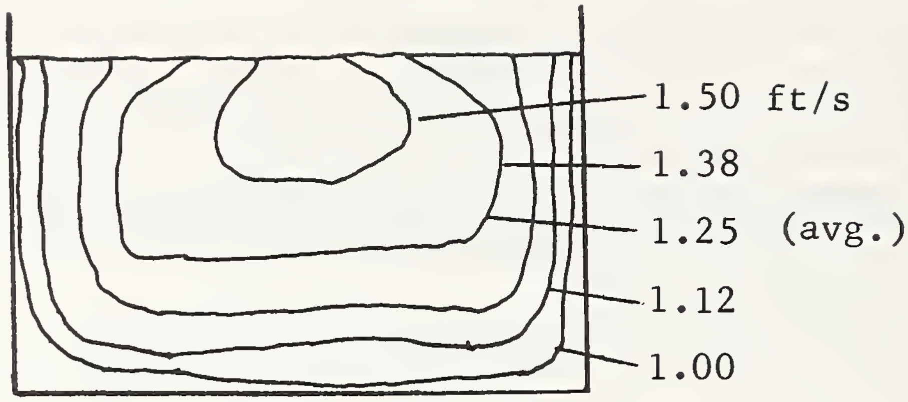


Figure 8.1. Example of velocity contours in a rectangular section.

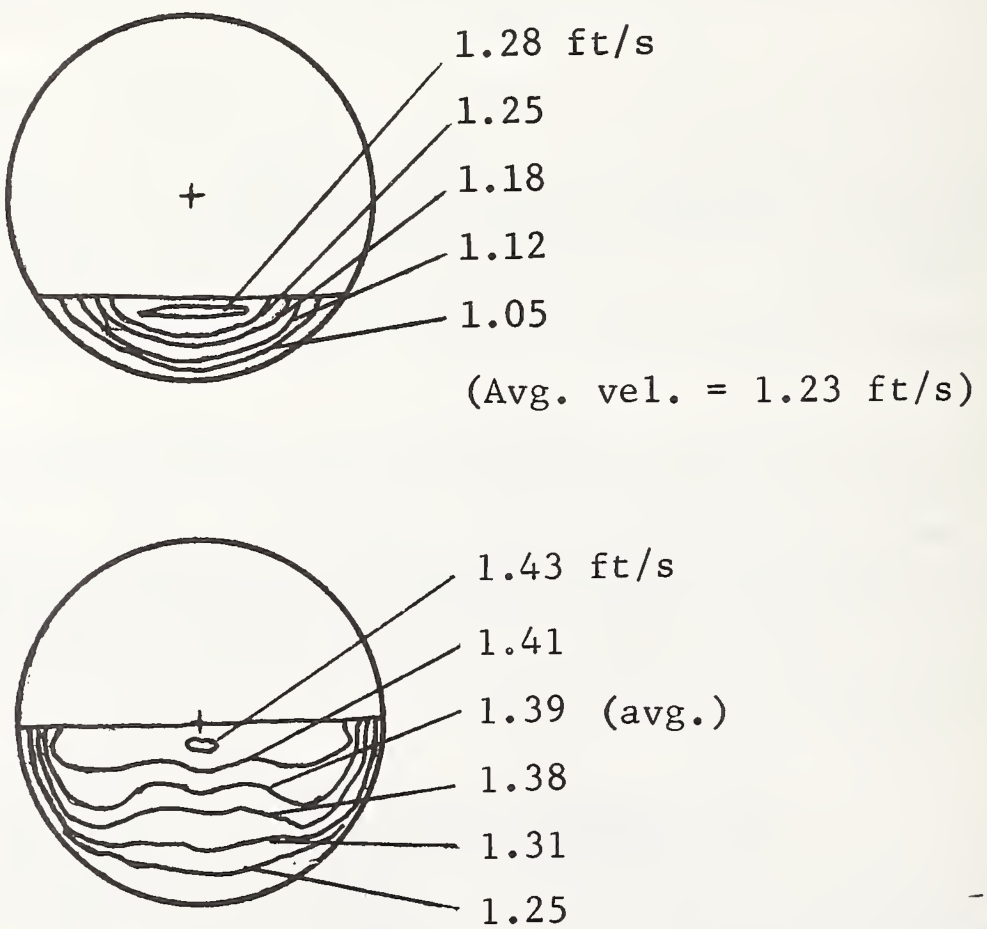


Figure 8.2. Examples of velocity contours in a part-full circular section.

The benefits of increasing the number of velocity measuring points are apparent. An added benefit is that the error due to finite sampling time is, thereby, also reduced (eq. 8.2). This numerical example is presented only for illustration, in order to acquaint users with the magnitude of errors that can be involved. It is not presented as a typical situation. Many more representative flows would have to be investigated before these results could be generalized.

Probably the most accurate traverse would result if one line of velocity points were taken with a propeller meter as close as permitted to the walls and bottom, with the interior points of the traverse spread accordingly. Then the flowrate could be determined graphically as recommended for closed conduits, e.g., ref. 8.2.3, with the velocity distribution between the boundaries and the outermost velocity points being estimated by an appropriate power law curve.

8.4.3. Numerical Example -- Circular Section

The sketches of figure 8.2 represent velocity contours obtained in a smooth laboratory pipe at Reynolds numbers of approximately 10^5 (8.8.7). They could also represent contours for larger, rougher pipes at higher Reynolds numbers.

In the case of larger sewers where access is only available from street level through a manhole, for practical purposes current meter measurements are obtainable only along the vertical centerline. In the shallow-flow cases of figure 8.2a, a single measurement at 0.6 depth will record (if registration of true velocities is assumed for this discussion) a velocity about 5 percent less than the average velocity. Using the 0.2 and 0.8 depth method improves this to about a 3 percent error.

When the pipe flows half-full (figure 8.2b) the error is about 3.3 percent for both the one-point method and the two-point method.

Caution: The errors in the foregoing appear deceptively small. Users are cautioned that the data available on velocity distribution are extremely limited for part-full flow in pipes, so that figure 8.2 is not necessarily representative. Also, the distribution will change with Reynolds number and roughness.

8.4.4. Error Due to Finite Sampling Time

The errors due to finite sampling time at a point as given in refs. 8.2.1 and 8.8.2 were obtained from numerous measurements in rivers and are not necessarily valid for small channels and sewers.

The following equation for estimating sampling time on the basis of turbulence parameters is given in ref. 8.8.5.

$$t = 2\bar{u}^2 T_x / X_f^2 \qquad [8.4]$$

Here t is the measuring time (seconds), $\bar{u}$ is the r.m.s. turbulence level, T_x is the macro time scale of the turbulence, and X_f is the sampling time error as in eq. 8.2. We assume that $T_x = L_x/V$, where V is the average velocity and where L_x is the turbulence macro scale, which we can further assume to be of the order of the depth of flow (sometimes the width is used).

Then, in the first illustration of section 8.4.2, if we want to attain $X_f = 0.06$ at each sampling point, and if we assume that the flow is fairly smooth with a turbulence level of 10 percent (see section 7.5.4), then the required sampling time is about 13 seconds.

8.5. Floats

The use of floats is actually a rudimentary velocity-area procedure and is, therefore, discussed in this chapter. Factors for multiplying float velocities to obtain average velocities according to ref. 8.8.1 are given in the curves of figure 8.3 for floats of various vertical lengths. These factors really refer to individual vertical strips of a wide section rather than to relatively narrow rectangular channels. For example, applying these factors to a centerline surface float in figure 8.1a gives an average velocity 5 percent too high, while use of two vertical strips yields an average velocity about 7 percent too low. A value of n = 7 was used in this illustration. Similarly, centerline surface floats in part-full circular sections of figure 8.2 in conjunction with figure 8.3 give average velocities about 10 percent too low.

The table of coefficients in figure 8.3 is intended for large dish-shaped sections where only a centerline surface float measurement is available. If several float measurements are made across the section, the coefficients from the curve of figure 8.3 should be used.

8.6. Optical Current Meter

The optical current meter developed by the U. S. Geological Survey (refs. 8.2.1, 8.8.3, 8.8.4) should be mentioned along with surface floats, since it obtains a surface velocity measurement by "stopping" the surface motion with a rotating prism arrangement. It was developed for use in heavily debris-laden flows, which would be destructive to intrusive meters. Limitations on its use are discussed in ref. 8.8.4.

8.7. Closed (Pressure) Conduits

The use of velocity-area methods in closed conduits has been well-documented in refs. 8.2.3 and 8.8.6 and will not be discussed in detail here. Both current meters and Pitot tubes can be used in this application. The largest uncertainty is associated with the blockage effect of current meter arrays. This effect has not been fully evaluated yet, but the most recent developments are summarized in ref. 8.8.8. The Pitot tube method is capable of good accuracy if done according to recommendations. Reference 8.2.5 is the most complete source of information on this method.

8.8. References

8.8.1. British Standards Institution, Standard No. 3680-3, "Methods of Measurement of Liquid Flow in Open Channels: Part 3, Velocity Area Methods," 1964.

8.8.2. Carter, R. W. and I. E. Anderson, "Accuracy of Current Meter Measurements," Proc. Amer. Soc. Civ. Eng., 89, HY4, July 1963, 105-115.

8.8.3. Chandler, T. S., and W. Smith, "Optical Current Meter Use in Southern California," Proc. Amer. Soc. Civ. Eng., 97, HY9, Sept. 1971, 1461-69.

8.8.4. Smith, W., and G. F. Bailey, "Optical Current Meter," Proc. Amer. Soc. Civ. Eng., 88, HY5, Sept. 1962, 13-22.

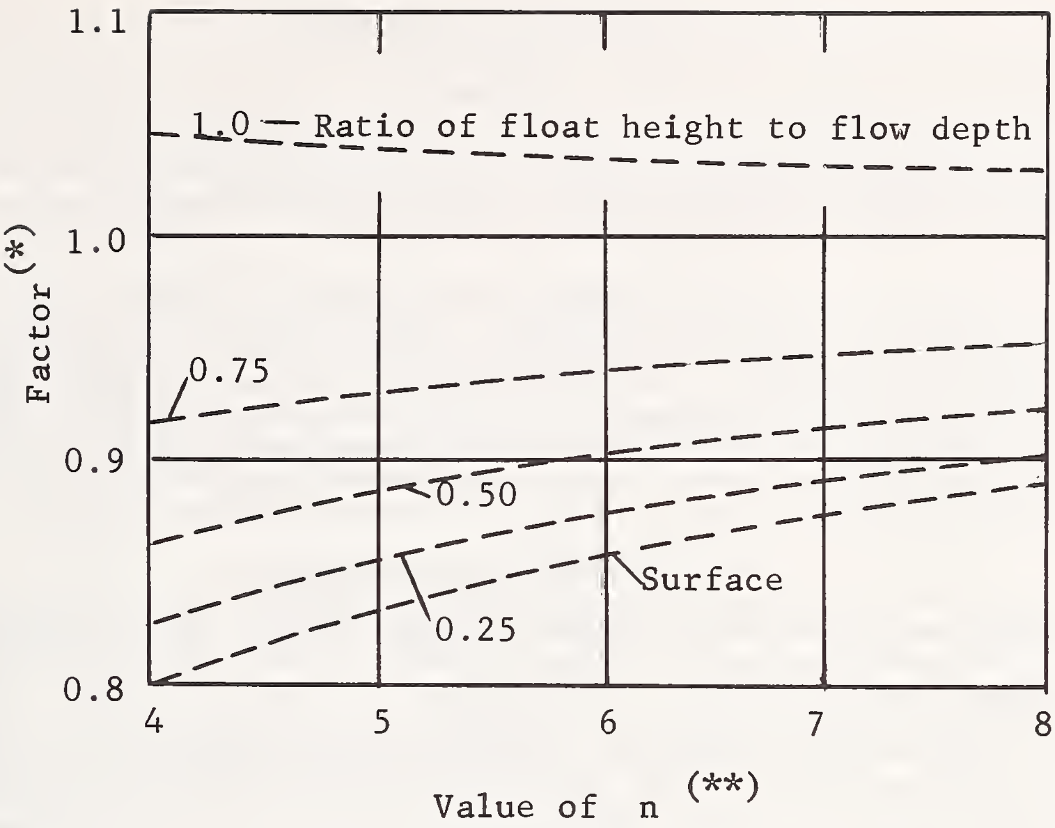
8.8.5. Jonsson, I. G., "On Turbulence in Open Channel Flow," Acta Polytechnica Scandinavica, No. Ci 31, 1965.

8.8.6. International Standards Organization, "Draft Proposal for an ISO Standard: Measurement of Clean Water Flow in Closed Conduits by the Velocity Area Method Using Current Meters," ISO/TC 30/SC3:67E, 1972.

8.8.7. Nalluri, C. and P. Novak, "Turbulence Characteristics in a Smooth Open Channel of Circular Cross-Section," Jour. Hydraulic Res., 11, 4, 1973, pp. 343-368.

8.8.8. Kinghorn, F. C., "Twelfth Meeting of the International Current Meter Group," Water Power, Nov. 1973, pp. 432-435.

8.8.9. I.S.O. Standard No. 1088, "Liquid Flow Measurement in Open Channels, Velocity-Area Methods, Collection of Data for Determination of Errors in Measurement," 1973, ANSI. $7.70.



(*) Float velocities are multiplied by this factor to obtain average velocity in a vertical strip.

(**) Exponent in power law velocity profile:

$$v/V_s = (y/D)^{1/n}$$

where v is velocity at elevation y, D is depth of vertical strip, and V_s is surface velocity. n = 7 corresponds to a smooth bed and n = 5 to a rough bed, approximately.

Coefficients to Apply to Centerline Surface Velocity of Dish-shaped Sections to Obtain Average Velocity

Avg. Depth in reach, ft.	Coefficient
1	0.66
2	0.68
3	0.70
5	0.74
9	0.77
15	0.79
>20	0.80

Figure 8.3. Surface float coefficients, from ref. 8.8.1.

9. CHEMICAL ADDITION METHODS

9.1. Introduction

Chemical addition techniques for volumetric flowrate measurement include "tracer" procedures, e.g., the salt-velocity method, and dilution methods. The latter can be based on the color, conductivity, fluorescence (and sometimes radioactivity) of the material injected into the flow.

These methods have in common requirements for specialized equipment, scrupulous care and often considerable time for their execution. They are therefore likely to be expensive in terms of instruments, special setups and labor, particularly when compared with current meter traverses. On the other hand there are situations -- small but highly turbulent streams and raw sewage flows, for example -- where current meters are often impractical but chemical methods are useful. For that reason, and also because of their potential for high accuracy, the dilution methods comprise an important flow measurement tool, both for direct streamgaging and for in situ calibration of flowrate measuring devices.

9.2. General Information Sources

9.2.1. International Electrotechnical Commission, Publ. No. 41, "International Code for the Field Acceptance Tests of Hydraulic Turbines," 1963. Order from American National Standards Institute, 1430 Broadway, New York, N. Y. 10018. $36.00.

This code gives considerable detail on the salt-velocity test, including features of the salt tank, pop valves, injection arrangement and electrodes.

9.2.2. I. S. O. Standard No. 2975/1, "Measurement of Water Flow in Closed Conduits--Tracer Methods--Part I: General," 1974. Order from American National Standards Institute, 1430 Broadway, New York, N. Y. 10018. $7.90.

This is the first of a projected series of seven I. S. O. standards on tracer methods in closed conduits. Advantages and disadvantages of the chemical addition methods and of the various tracers are listed in this introductory document. Information on necessary mixing lengths and on estimating errors is also given.

9.2.3. I. S. O. Standard No. 555, "Liquid Flow Measurement in Open Channels---Constant Rate Injection Method," 1966. Order from American National Standards Institute, 1430 Broadway, New York, N. Y. 10018. $10.15.

This standard given details on colorimetric, conductivimetric and volumetric chemical analysis methods of dilution analysis, as well as site and injection equipment requirements for the constant rate dilution method. Examples of error estimation are also given.

9.2.4. "Fluid Meters -- Their Theory and Application," Amer. Soc. Mech. Eng., 6th ed., 1971, Chapter I-0. Order from ASME, 345 E. 47th St., New York, N. Y. 10017. $20.00. ($16.00 to ASME members).

Details on the salt-velocity method and brief overviews of other methods.

9.2.5. Replogle, J. A., L. E. Meyers and K. J. Brust, "Flow Measurements With Fluorescent Tracers," Proc. ASCE, 92, HY5, pp. 1-15, Sept. 1966. Also see discussion of this paper by F. A. Kilpatrick, W. W. Sayre and E. V. Richardson, Proc. ASCE, 93, HY4, pp. 298-308, July 1967. Obtain from Engineering Societies Library, 345 E. 47th St., New York, N. Y. 10017. $0.25 per page plus $3.00 for each volume handled.

These papers, in addition to covering the principles of the methods and the comparative advantages of continuous vs. slug injection, give some details on the handling of equipment and tracers.

9.2.6. Blakey, A. W., "Flow Measurement in Sewers Using a Constant Rate of Injection Dilution Technique," Jour. Inst. Municipal Engrs., 96, Feb. 1969, pp. 44-52. Obtain from Engineering Societies Library, 345 E. 47th St., New York, N. Y. 10017. $0.25 per page plus $3.00.

Lithium chloride was used as the additive in these measurements. However, the interest in this paper lies in the detail with which the sewer measurement planning, operation and costs are described.

9.3. Salt Velocity Method

In the salt velocity method, brine is injected suddenly at an upstream station in such a manner that it becomes well distributed across the section very rapidly. The time of passage of the salt pulse between two downstream stations is measured by means of electrodes which detect the increased conductivity associated with the passage of the brine. The flowrate can then be determined provided that the volume of the conduit between the electrode stations is accurately known.

The salt velocity method has a potential for better than 1 percent accuracy when properly used. Major error sources are improper determination of elapsed time between the centers of gravity of the recorded conductivity pulses and inaccurate determination of conduit volume.

References 9.2.1 and 9.2.4 are recommended for information on this method. Although these documents are mainly written and illustrated in terms of circular closed conduits, the procedures can be adapted to open channel flows. Because velocities in open conduits are often low, users are cautioned to note the minimum velocity vs. brine-density criteria cited in ref. 9.2.1.

The method as described in ref. 9.2.1 and elsewhere requires an array of injection points at the upstream injection station, and an array of immersed electrodes at the downstream stations. Therefore, it would have to be used with caution in raw domestic sewage. Lengths of regular conduit geometry are desirable for this test.

In some situations it is possible to obtain an average velocity between stations by visually determining the time of passage of a dye cloud (9.5.9). This method could be refined by using fluorometry to determine time of passage.

9.4. Dilution Methods

Dilution methods can employ either constant rate (continuous) injection (9.2.3) or slug injection. The progressive dye dilution is shown in figure 9.1 for the two methods as illustrated in ref. 9.2.5. Accuracy in both cases depends, among other factors, on adequate lateral mixing at the sampling station and on accurate concentration measurement. Dilution methods are further subdivided by the type of material or dye added and/or by the property which is analyzed to determine the dilution. These are briefly mentioned in the following paragraphs with references to the best available literature on the subject.

9.4.1. Constant Rate Injection

The basic principle of the constant rate method is expressed in the following continuity equation

$$Q = q(C_1 - C_2)/(C_2 - C_0) \qquad [9.1]$$

where Q is the flowrate to be measured, q is the injected constant flowrate of additive concentration C_1, C_2 is the concentration at the sampling station, and C_0 is the background concentration of the particular additive in the water. The main advantages of this method are that the sampling can be done during a concentration plateau (figure 9.1) and that the long injection period reduces the adverse effect of slack water areas in natural streams (9.5.3). It is also more suitable for unsteady (time varying) flow than is slug injection.

9.4.2. Constant Rate Injection -- Error Sources

Equation [9.1] assumes that no dye is lost between injection and sampling stations by either physical or chemical action. See refs. 9.2.2, 9.2.5 and 9.5.2 for comments on dye stability. Any dye loss will be reflected as apparent additional dilution and hence apparent greater flowrate. For example, in sewer measurements it is important to stay upstream of chlorination stations (9.5.1).

The main disadvantage of this method is shown in eq. 9.1, namely that the injection rate has to be accurately measured as well as the injected concentration C_1. Errors in measuring C_2 are also important but are common (to varying degrees) to all dilution methods.

9.4.3. Length Required for Transverse Mixing

Another error source common to all dilution methods is failure to achieve sufficient lateral mixing at the sampling station. Several criteria are available, e.g., refs. 9.2.2, 9.2.5 and 9.5.3, but this factor is still in doubt. It is best to be conservative in length, but to check to be certain that the length is not so great as to cause dye loss.

As a numerical example, consider an 8-ft diameter trunk sewer flowing half full on a slope of 0.00020 with an average velocity of 3 ft per second. Most criteria for mixing length have been developed for rivers or wide, shallow canals. Nevertheless, using the 100 x top width criterion of ref. 9.2.5 results in an 800 ft required length. Using a rationally based criterion of ref. 9.5.3, the required distance is about 480 ft. Adapting the closed-conduit recommendation of ref. 9.2.2, the measuring length should be about 1000 ft for the concentration across measuring section to be within one percent of uniform. To be conservative, one is inclined to select the largest distance (or the closest manhole spacing increment) provided one can find such a length without sewer junctions.

9.4.4. Slug Injection

The principle of the slug injection method is expressed by the equation

$$Q_0\int(C_2 - C_0)dt = C_1(\mathrm{Vol.}) \qquad [9.2]$$

where t is time, (Vol.) is the volume of the injected slug of dye, and Q, C_0, and C_1 and C_2 are as defined earlier.

It is clear from eq. 9.2 that the main disadvantage of this method is that the tracer wave has to be sampled over its entire time of passage. See figure 9.1. (The previous requirement for adequate lateral mixing of course still prevails.) On the other hand, the main advantage (relative to constant rate injection) is that the injection apparatus need not be so sophisticated, since only the total injection load of the dye need be known accurately. Slug injection is best-suited for regular, artificial conduits, because they have no slack water areas to entrap or delay dye.

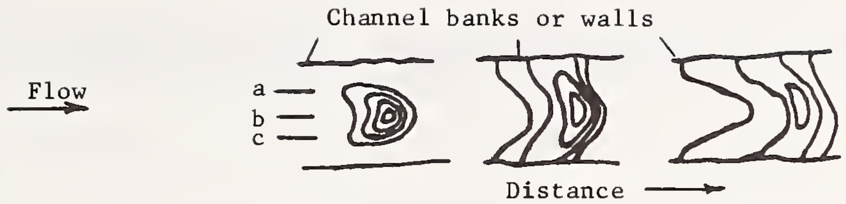
References 9.2.5 and 9.5.7 give additional information on the slug injection method.

9.4.5. Slug Injection -- Error Sources

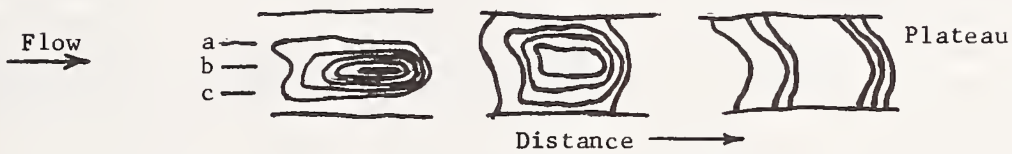
As is the case for constant rate injection, there can be no dye loss between injection and sampling, and dilution analysis must be carefully made. Additionally, the observer must be careful that the <u>entire</u> dye pulse at the measuring station is sampled. Failure to do this always results in too high a flowrate determination.

9.4.6. Selection of Additive

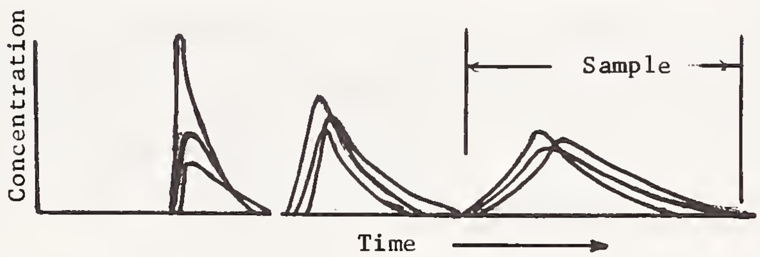
Florescent dyes have been studied in detail by the U. S. Geological Survey (9.5.2). Rhodamine WT dye has shown extremely small dye loss to channel boundaries and sediment and is therefore recommended for dilution studies. It has been used successfully in raw sewage flows (9.5.1). Existing fluorometers properly used appear to be capable of 1 percent accuracy.



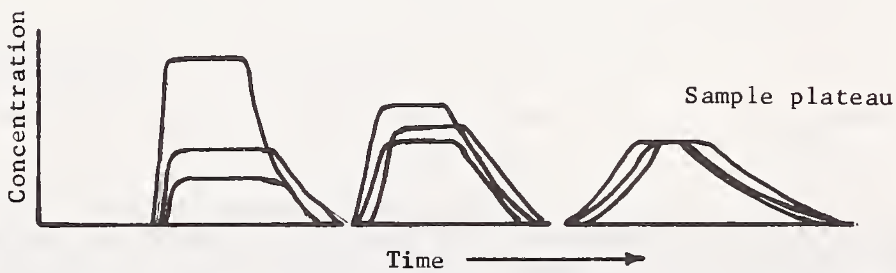
a. Spatial distribution of tracer in slug injection.



b. Spatial distribution of tracer in continuous injection.



c. Time-concentration curves in slug injection.



d. Time-concentration curves in continuous injection.

Figure 9.1. Dye dispersion for slug injection and constant rate of injection, after Kilpatrick, et al., (9.2.5).

Sodium dichromate is the most common dye used when dilution is measured by colorimetric analysis. Accuracy of colorimetric analysis can be seriously affected by suspended matter in the flow or by chromium ions in wastewater (9.2.3). Sodium chloride is frequently used when the dilution determination is to be based on conductivity. Large quantities might be required for continuous injection, particularly if the background level of salt in the flow is high. Minimum concentrations for 1 percent accuracy of analysis are given in ref. 9.2.3.

Details on analysis by colorimetry and conductivity as well as chemical analysis by titration are given in ref. 9.2.3. All three methods are potentially capable of 1 percent accuracy. See ref. 9.2.5 for some details on fluorometric analysis.

Lithium chloride is apparently used frequently in the U. K., with analysis by flame photometry (9.2.6).

9.4.7. Radioisotope Methods

The use of radioisotopes for dilution discharge measurements has been studied extensively by the U. S. Bureau of Reclamation in collaboration with the U. S. Atomic Energy Commission (9.5.4, 9.5.5). The counting procedures used for dilution analysis here are more accurate than the fluorometric, colorimetric or conductivity analyses of the previous sections. Although the dosages required for discharge measurements are well below accepted safety limits, radioisotope use in flow measurement has been handicapped by handling problems and by general public apprehension (9.5.2). It is therefore unlikely that its use will ever become widespread in local water and sewerage districts in spite of its potential for high accuracy. It is not further discussed here, but interested readers can note that a British standard is available for this method (9.5.6).

9.4.8. Overall Accuracy

It is difficult to pinpoint from available information the overall accuracy of carefully made dilution discharge measurements. In ref. 9.5.1 Kilpatrick achieved slightly better than ± 2 percent with continuous injection in the laboratory (although for only two tests). It can be assumed that similar accuracy could be attained in artificial conduits under good conditions in the field, with somewhat poorer accuracy achievable in natural streams. The same accuracy would be achievable in dilution tests using colorimetry, conductivity and volumetric titration, provided that the respective analyses were made with the same degree of accuracy as fluorometric analyses.

Note that the capability for this accuracy makes dilution methods suitable for in situ calibration of flowmeters which are not usually accurate to 2 percent uncalibrated. This is particularly true, for example, for Parshall flumes, which are usually considered ± 3 to 5 percent instruments under the best of circumstances. However, for the procedure used in ref. 9.2.6, accuracies of only 3 to 5 percent were claimed.

9.5. References

9.5.1. Kilpatrick, F. A., "Flow Calibration by Dye-Dilution Measurement," Civ. Eng., Feb. 1968, 74-76.

9.5.2. Wilson, J. F., "Fluorometric Procedures for Dye Tracing," Chap. A 12, Book 3 of Techniques of Water-Resources Investigations of the U. S. Geological Survey, 1968. (Out of print at U. S. Govt. Printing Office).

9.5.3. Discussion of ref. 9.2.5 by Fischer, H. B., Proc. Amer. Soc. Civ. Eng., 93, HY1, Jan. 1967, 139-140.

9.5.4. Schuster, J. C., "Canal Discharge Measurements with Radioisotopes," Proc. Amer. Soc. Civ. Eng., 91, HY2, March 1965, 101-124.

9.5.5. Schuster, J. S. and Hansen, R. L., "Radioisotopes and Turbine Flow Measurements," Proc. Amer. Soc. Civ. Eng., 98, PO1, June 1972, 1-9.

9.5.6. British Standard No. 3680-2C, "Methods of Measurement of Liquid Flow in Open Channels, Part 2: Dilution Methods -- C. Radioisotope Techniques," 1967. British Standards Institution (British standards are distributed in the U. S. by the American National Standards Institute, 1430 Broadway, New York, N. Y. 10018).

9.5.7. Kilpatrick, F. A., "Dosage Requirements for Slug Injections of Rhodamine BA and WT Dyes," U. S. Geol. Survey Prof. Paper 700-B, in Geological Survey Research, 1970.

9.5.8. "Water Measurement Manual," Bureau of Reclamation, U. S. Dept. of Interior, Second Ed., Revised Reprint, 1974. See p. 164. See ref. 2.3.1 for ordering information.

Table 9.1

Tabular Summary of Chemical Addition Methods

Application/Property	Salt Velocity	Dilution
Streamgaging	Not suitable (9.2.1)	Suitable, continuous injection only (9.5.3)
Open conduits (canals)	Suitable, regular planform conduits only (9.5.4)	Suitable
Sewers	Suitable with caution	Suitable with caution
Unsteady flow	Not suitable	Suitable, continuous injection only (9.2.5)
Silty water	Suitable	Suitable with caution
Accuracy (potential)	High (1 percent)	Good (2 percent)
Cost/Labor	High	High
Calibration of other devices	Suitable	Suitable (9.5.1)

10. OTHER OPEN CHANNEL METHODS

10.1. Introduction

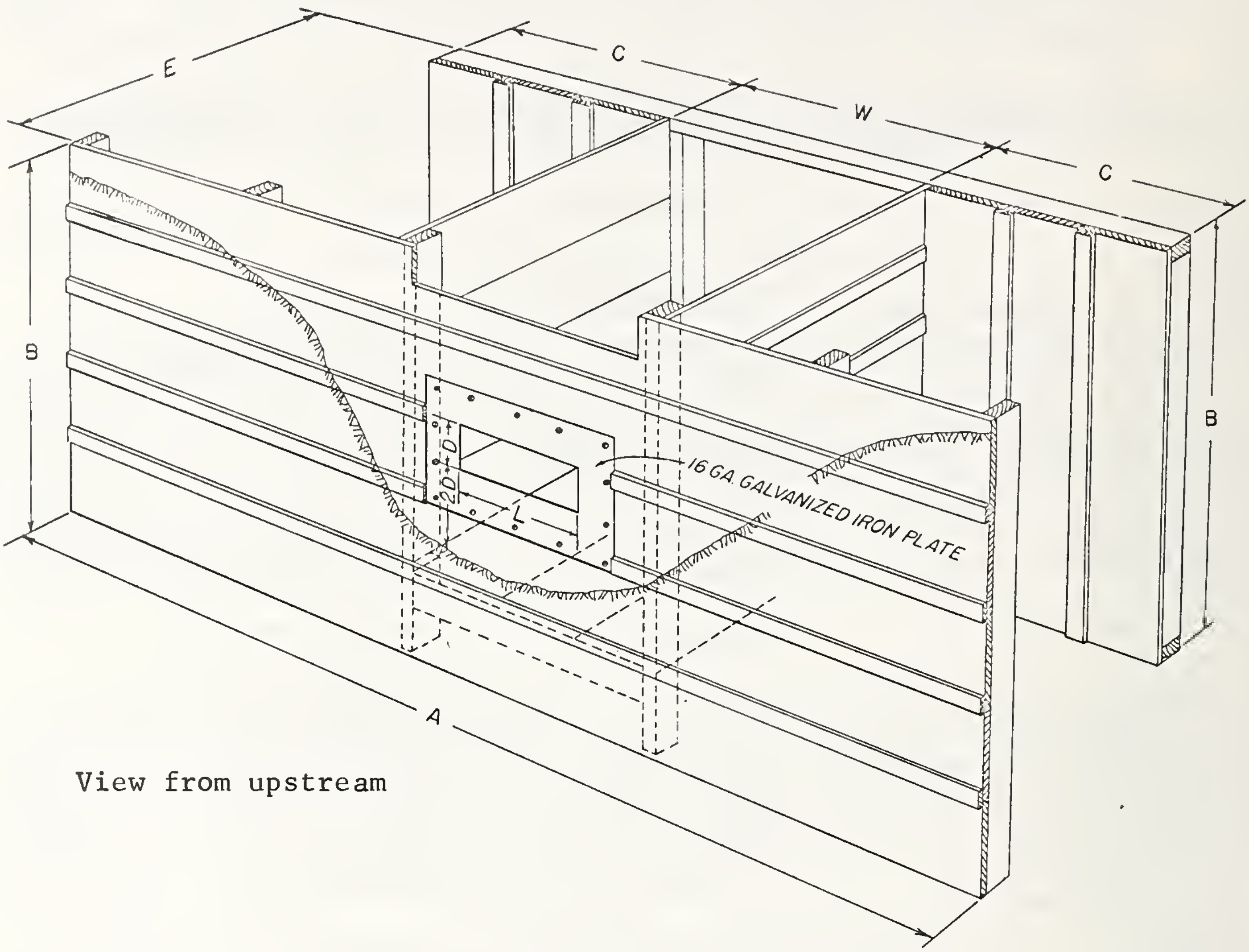
This chapter covers a few additional open channel methods which did not appear to belong in the previous chapters. In general, these methods are not as accurate as the traditional methods already described.

10.2. Submerged Orifice

10.2.1. Introduction

The submerged orifice (see figure 10.1) is a rectangular opening which can either have contractions on all four sides or have the contractions suppressed on the bottom and/or sides. In this respect, it is analogous to the thin-plate weirs (chapter 4). Unfortunately, the data base for submerged orifices does not appear to be as complete as that for the weirs.

Although the submerged orifice is used mainly for irrigation flows, it could also be used in specific situations in wastewater flows. The suppressed-contraction form would be required to pass solids, and only flows providing self-cleansing upstream velocities (after installation of the obstruction) could be considered.



View from upstream

RECOMMENDED DIMENSIONS

D (ins)	L (ins)	B (ft)	A (ft)	E (ft)	W (ft)	C (ft)
3	12	4.0	10.0	3.0	2.5	2.0
3	24	4.0	12.0	3.0	3.5	2.0
6	12	5.0	12.0	3.5	2.5	3.0
6	18	5.0	14.0	3.5	3.0	3.0
6	24	5.0	14.0	3.5	3.5	3.0
9	16	6.0	14.0	3.5	3.0	3.0
9	24	6.0	16.0	3.5	3.5	3.0

Figure 10.1. Submerged orifice structure, from ref. 10.2.2.1.

The equation for discharge through a fully contracted (distances from orifice edges to stream boundaries greater than twice the least dimension of the orifice) submerged orifice is

$$Q = 0.61A[2g(H + h)]^{1/2} \qquad [10.1]$$

where Q is the flowrate in ft^3/s, g is the acceleration due to gravity (32.2 ft per sec^2), A is the area of the orifice in ft^2, H is the difference between water surface elevations upstream and downstream of the orifice, in ft, and h is the approach velocity head, $V^2/2g$, in ft, based on the average upstream velocity, V. No published data are available on the reliability of the coefficient, 0.61, but it appears to be consistent with the (circular) orifice coefficients of chapter 11. It also follows that edge sharpness must be maintained for the coefficient to remain valid.

If the bottom and/or sidewall contractions are suppressed (see figure 10.1), eq. 10.1 can be replaced with

$$Q = 0.61(1 + 0.15r)A[2g(H + h)]^{1/2} \qquad [10.2]$$

where r is the ratio of the suppressed part of the orifice perimeter to the total perimeter. According to ref. 10.2.2.1, eq. 10.2 is approximate, but no quantitative information on its accuracy is available.

10.2.2. General Information Sources

10.2.2.1. "Water Measurement Manual," Bureau of Reclamation, U. S. Dept. of the Interior, Second Ed., Revised Reprint, 1974. Order from U. S. Government Printing Office, Wash., D. C. 20402. $5.80. Cat. No. I27.19/2:W29/2, S/N 2403-0027.

This reference gives solutions to eqs. 10.1 and 10.2 in tabular form. Recommended sizes of inlet sections are given, along with information on combination regulating/measuring devices for use at irrigation turnouts.

10.3. Free Overfall

10.3.1. Background

When an open channel discharges with a free fall in air, as in figure 10.2, the ratio of the brink depth, y_e, to the critical depth, y_c, is a function of channel shape, slope and roughness. Because critical depth can be related to flowrate, brink depth can be used as a measure of flowrate.

For example, some experimental results for y_e/y_c as a function of channel slope-to-critical slope ratio, S_o/S_c, are shown in figure 10.3 for rectangular channels. In rectangular channels, according to the principles of critical flow

$$Q = Bg^{1/2}y_c^{3/2} \qquad [10.3]$$

where B is the channel width. Consider a channel at zero slope. Using $y_e/y_c = 0.715$ from figure 10.3,

$$Q = 1.654\ Bg^{1/2}y_e^{3/2} \text{ (Horizontal rect. channel)} \qquad [10.4]$$

If the channel is sloped, a trial procedure must be used, since we cannot know y_e/y_c from figure 10.3 without knowing S_o/S_c. But S_c in turn cannot be computed (from Manning's equation, see eq. 10.5) without knowing Q to begin with. One procedure for small slopes is as follows.

The slope and y_e are measured.

(a) Use eq. 10.3 or 10.4 to estimate Q from measured y_e, i.e., use $y_e/y_c = 0.715$ as a first approximation.

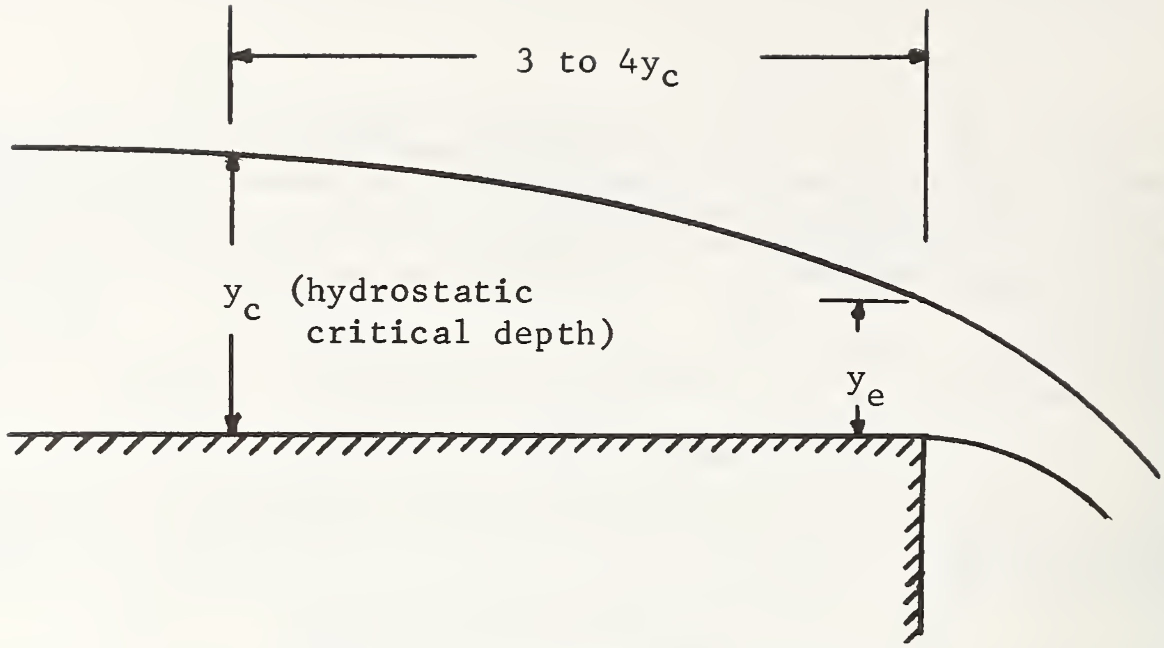


Figure 10.2. Free overfall into atmosphere.

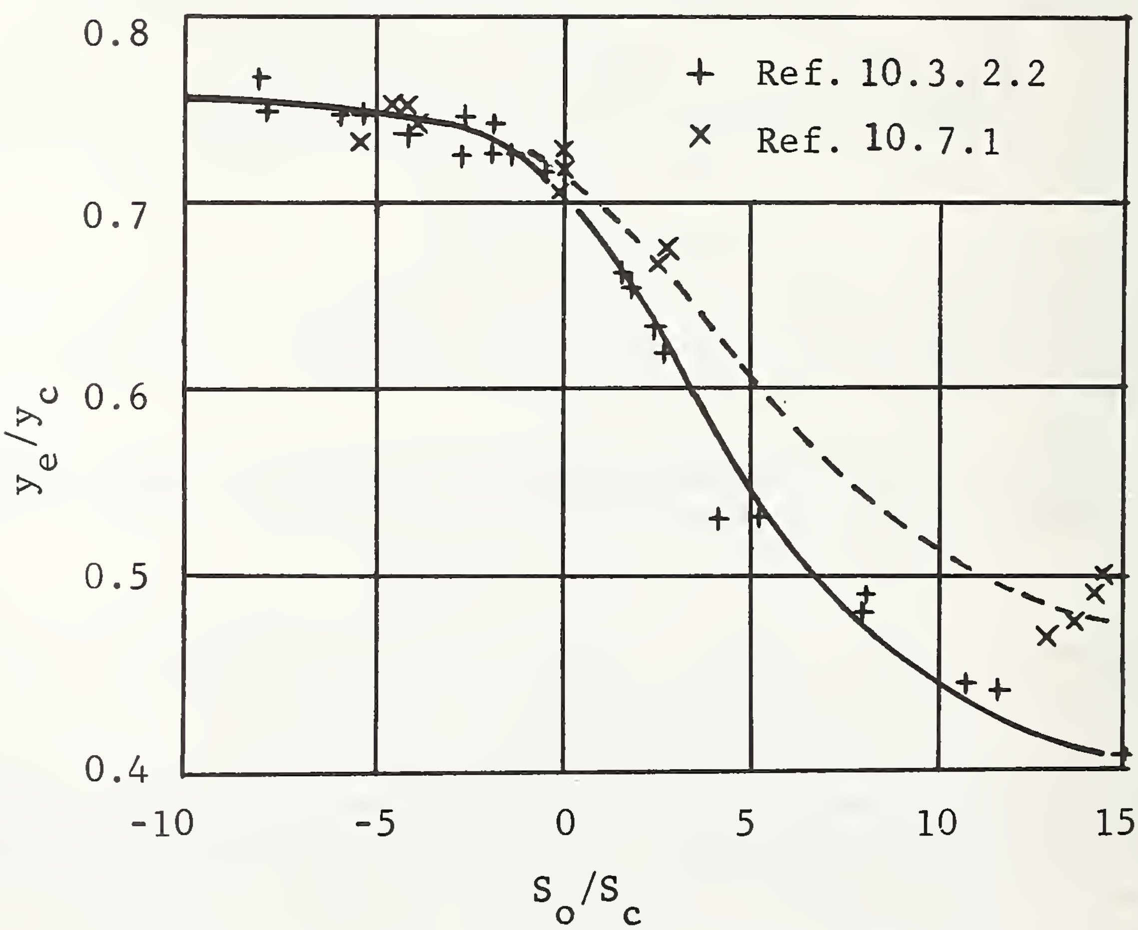


Figure 10.3. Rectangular free overfall data.

(b) Use this Q and corresponding y_c to determine S_c from the Manning equation, and then S_o/S_c.

(c) Using the second approximation value of y_c from step (c), get second approximation Q from eq. 10.3, and repeat procedure until convergence.

For circular channels the relations are more complex. An experimental Q vs. y_e curve is given in figure 10.4 from ref. 10.3.2.1 for horizontal channels only. Also, from ref. 10.3.2.1, curves are reproduced in figure 10.5 for sloping channels to obviate the need for the foregoing successive-approximation procedure.

Caution. The data for circular channels from ref. 10.3.2.1 are based on experiments on one pipe size only (8 inches), and should be treated cautiously until corroborating data are available.

Caution. Because the high curvature of the flow will cause stilling wells to give erroneous results at and near the brink, y_e can be measured only by some type of surface probe.

The ratio y_e/y_c has been found to be essentially unaffected by modest increases in channel roughness, e.g., ref. 10.3.2.2.

10.3.2. General Information Sources

10.3.2.1. Rajaratnam, N. and Muralidhar, D., "End Depth for Circular Channels," Proc. Amer. Soc. Civ. Eng., 90, HY4, March 1964, pp. 99-119. Order from Engineering Societies Library, 345 E. 47th St., New York, N. Y. 10017. $0.25 per page plus $3.00.

10.3.2.2. Delleur, J. W., Dooge, J. C. I., and Gent, K. W., "Influence of Slope and Roughness on the Free Overfall," Proc. Amer. Soc. Civil Eng., 82, HY4, Aug. 1956, pp. 1038-30 to 1038-35. Order as above.

These references give basic analysis and experiments on the free overfall. However, they are concerned with the mechanics of the overfall and not primarily with the overfall as a measuring device.

10.3.3. Error Sources and Comments

The foregoing results are valid only for free discharge such that the nappe is surrounded by atmospheric pressure. In a rectangular overfall with the sidewalls continued past the brink, sub-atmospheric pressures reduce the average pressure in the nappe at the brink section and consequently reduce y_e/y_c.

An example of the effect of reduced pressure is shown in figure 10.6 from ref. 10.7.2. In this case, an abrupt slope change is being used as an overfall. The difference in y_e/y_c from the completely free discharge case is about -10 percent for this example, corresponding to an error in Q of -15 percent.

Even if the lower nappe of a rectangular overfall is aerated, the presence of extended sidewalls can modify y_e/y_c slightly from the completely free case. With sidewalls present $y_e/y_c = 0.715$ is a generally accepted value for the horizontal rectangular channel. If there are no sidewalls to restrain the nappe, y_e/y_c is probably reduced by about 1 percent.

A survey of available experimental results on the horizontal rectangular overfall suggests that the value $y_e/y_c = 0.715$ should probably be assigned an accuracy of about ± 3 percent. Also, problems in measuring the depth of water that is moving rapidly and in a curved path along with increased uncertainty in y_e/y_c for slopes combine to make the overfall method less accurate in general than weir or flume methods.

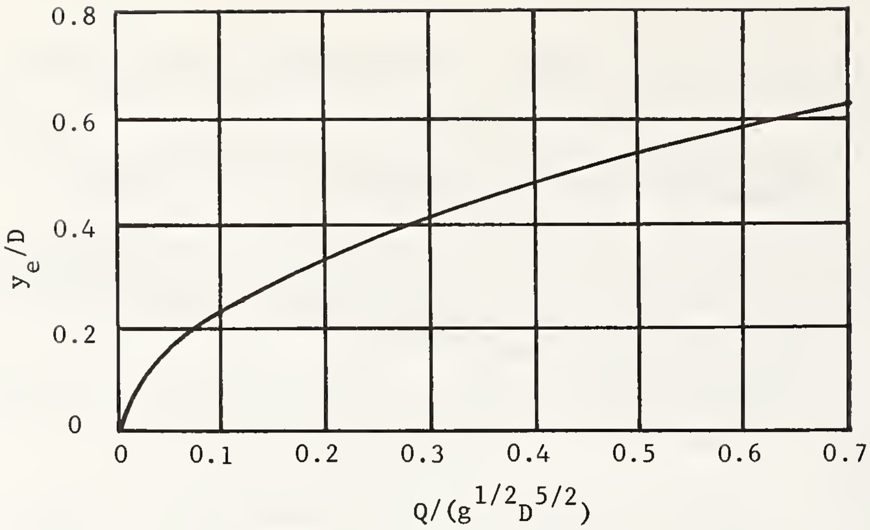


Figure 10.4. Discharge vs. brink depth for horizontal circular channels, from ref. 10.3.2.1.

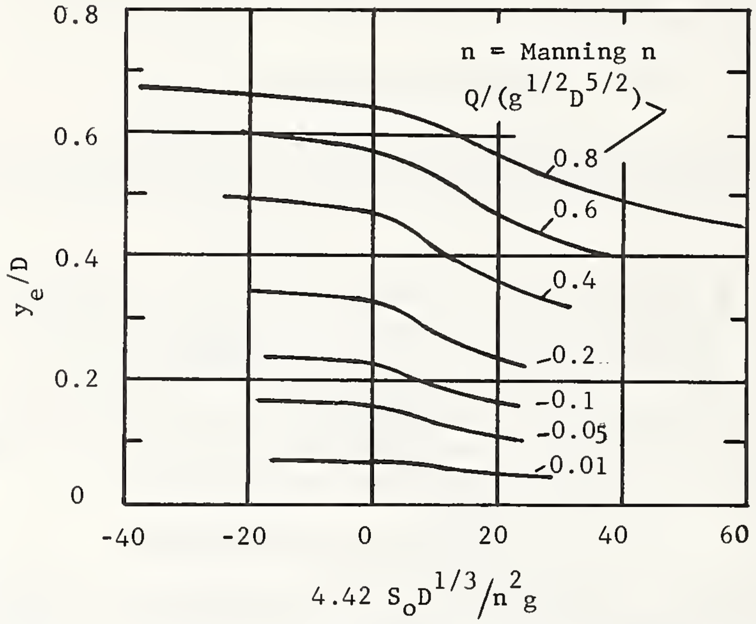


Figure 10.5. Discharge of sloping circular channels with free overfall, from ref. 10.3.2.1. Abscissa ft-lb-sec units only.

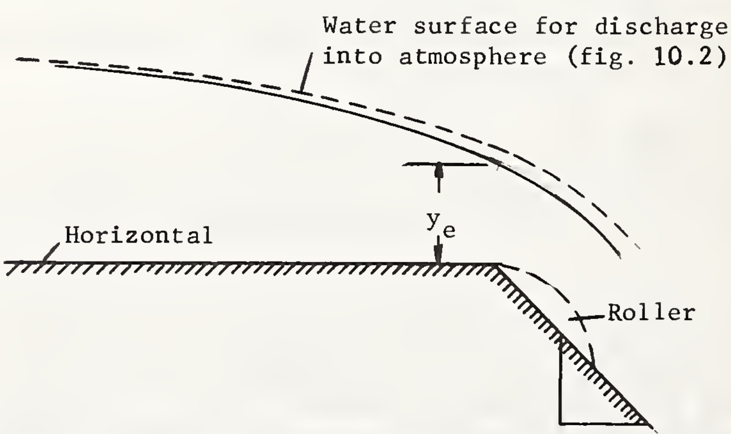


Figure 10.6. Effect of sub-atmospheric pressures beneath overfall.

There is insufficient data to permit assessment of the accuracy of the circular overfall information.

10.4. Acoustic Flowmeters

Acoustic meters can be applied to open channels and users are referred to section 11.12.

10.5. Slope-Area Method

In the slope-area method, the depth is measured, preferably in a long reach of uniform flow, and the Manning equation is used to estimate the flowrate. The Manning equation is

$$Q = (1.49/n)AR^{2/3}S^{1/2} \qquad [10.5]$$

where A is the sectional area of flow, R is the hydraulic radius, S is the bottom (and water surface) slope in uniform flow, and n is an empirical roughness coefficient. The numerical coefficient in eq. 10.5 is valid for foot-second units only. Estimated values of n for various types of channels can be found in most hydraulic textbooks and handbooks, e.g., ref. 2.8.1. Area and hydraulic radius can be determined from the depth measurement for known channel shapes. The slope can be measured, but it will be correct for use in eq. 10.5 only to the extent that the flow is close to uniform. By far the largest uncertainty in this method is due to the estimate of the Manning n. Unless there is some independent method for determining this coefficient (including the possibility of its variability with depth), this method has to be regarded as an estimate only. See ref. 10.2.2.1.

10.6. Other Methods

Other methods and devices for open channel flowrate measurements, such as deflection meters, propeller meters and others, can be found in ref. 10.2.2.1. Some indirect methods for determining peak streamflow are described in refs. 10.7.3, 10.7.4 and 10.7.5.

10.7. References

10.7.1. Rajaratnam, N. and Muralidhar, D., "Characteristics of the Rectangular Free Overfall," Jour. Hydraul. Res., 6, 3, 1968, pp. 233-258.

10.7.2. Rouse, H., "The Distribution of Hydraulic Energy in Weir Flow with Relation to Spillway Design," M. S. Thesis, M.I.T., 1932.

10.7.3. Bodhaine, G. L., "Measurement of Peak Discharge at Culverts by Indirect Methods," Book 3, Ch. A3 of Techniques of Water-Resources Investigations of the U. S. Geological Survey, 1968.

10.7.4. Matthai, H. F., "Measurement of Peak Discharge at Width Contractions by Indirect Methods," Book 3, Ch. A4 of Techniques of Water-Resources Investigations of the U. S. Geological Survey, 1967.

10.7.5. Hulsing, H., "Measurement of Peak Discharge at Dams by Indirect Methods," Book 3, Ch. A5 of Techniques of Water-Resources Investigations of the U. S. Geological Survey, 1967.

11. INSTRUMENTS AND METHODS FOR CLOSED CONDUIT FLOWS

11.1. Introduction

Three flow measurement methods used in closed conduits, i.e., the salt-velocity method, dilution techniques, and velocity-area integrations were covered separately in chapters 8 and 9. The scope of instruments and methods covered in this chapter ranges from rudimentary (and often inexact) methods for situations where approximate and ad hoc measurements are needed, through the traditional differential-head meters, to sophisticated modern meters based on magnetic or sonic principles. Instruments and methods will be taken up in the following sections in approximately increasing order of sophistication.

11.2. Trajectory Method (Purdue)

11.2.1. Background

If flow discharges into air from the end of a horizontal pipe, the flowrate can be determined from trajectory measurements as shown in figure 11.1. This is a situation frequently encountered in industrial wastewater outfalls. The trajectory measurements are used in conjunction with curves derived from Purdue University experiments in 1928 on pipes 2, 3, 4, 5 and 6 inches in diameter. See figure 11.2. Note that this method can be used for part-full pipe discharge (use the curves for X = 0) provided the brink depth is less than 0.8 diameter.

11.2.2. General Information Sources

11.2.2.1. "Water Measurement Manual," Bureau of Reclamation, U. S. Dept. of the Interior, Second Ed., revised reprint, 1974, Ch. 8, pp. 199-203. U. S. Govt. Printing Office, Washington, D. C. 20402. $5.80. Cat. No. I27.19/2:W29/2, S/N 2403-0027.

11.2.3. Comments

The results of using figure 11.2 will not agree with results obtained analytically by treating the issuing water as a freely falling body, as described in section 11.4. Because figure 11.2 is based on experiment, its use is recommended over the use of the theoretical parabolic trajectory, eq. 11.2, when standard commercial steel pipe (of the sizes covered in figure 11.2) is involved.

11.3. California Pipe Method

11.3.1. Background

The California Pipe Method involves the measurement of the overfall depth at the end of a short length of horizontal pipe flowing part full. See figure 11.3. It is a closed conduit method only to the extent that part-full flow can be achieved in the horizontal stub. The discharge, Q, in cubic feet per second is given by

$$Q = 8.69(1 - a/d)^{1.88}\, d^{2.48} \qquad [11.1]$$

where a is the distance in feet from the inside top of the pipe to the water surface, measured at the end section (figure 11.3), and d is the inside diameter of the pipe in feet. It is recommended that a/d be restricted to values greater than 0.5. The experiments from which eq. 11.1 is derived used pipe diameters from 3 to 10 inches.

11.3.2. General Information Source

11.3.2.1. "Water Measurement Manual," see (11.2.2.1) for ordering information. See Ch. 8, pp. 196-198 for California Pipe Method.

11.3.3. Comments

The California Pipe Method is related to the free overfall method of flow estimation, which is really an open channel method and which has more general application. See chapter 10.

The Purdue trajectory method, with X = 0 (see figure 11.1), should be approximately comparable to the California Pipe Method when the pipe is part full, and it is interesting to compare the results for two typical cases.

Case 1: 6-in. pipe, depth at end section is 3 inches. Using trajectory data (11.2.1), flow is 200 gpm. Using eq. 11.1 and converting to gpm, flow is 190 gpm, a difference of about 5 percent.

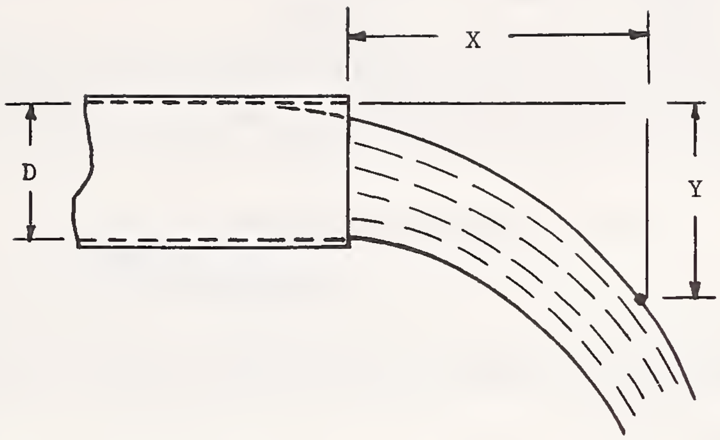
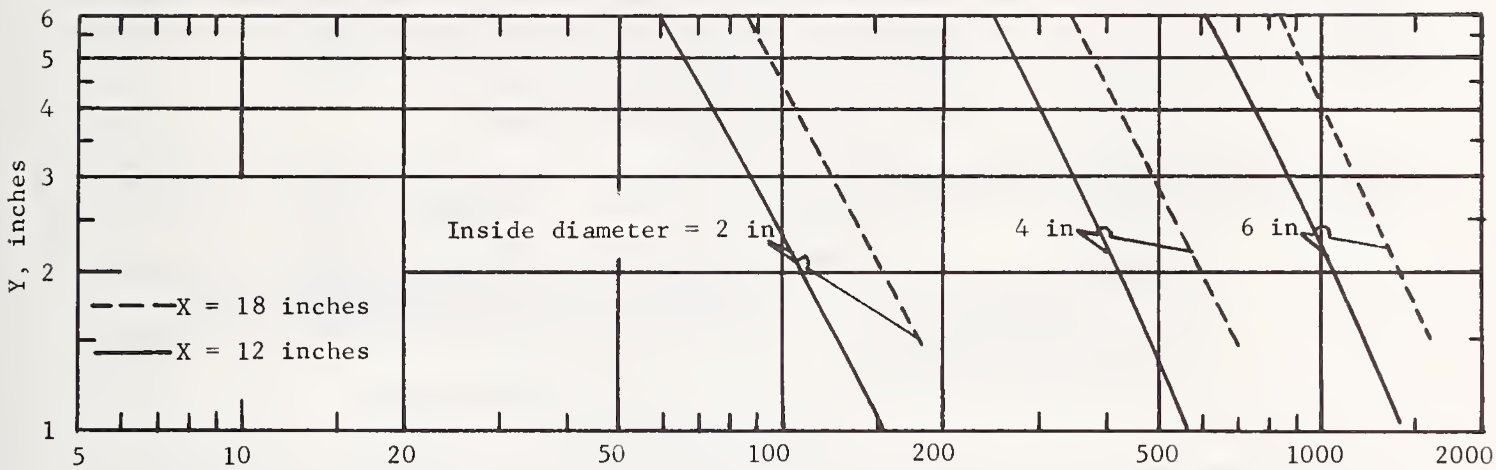


Figure 11.1. Trajectory measurement for Purdue method.



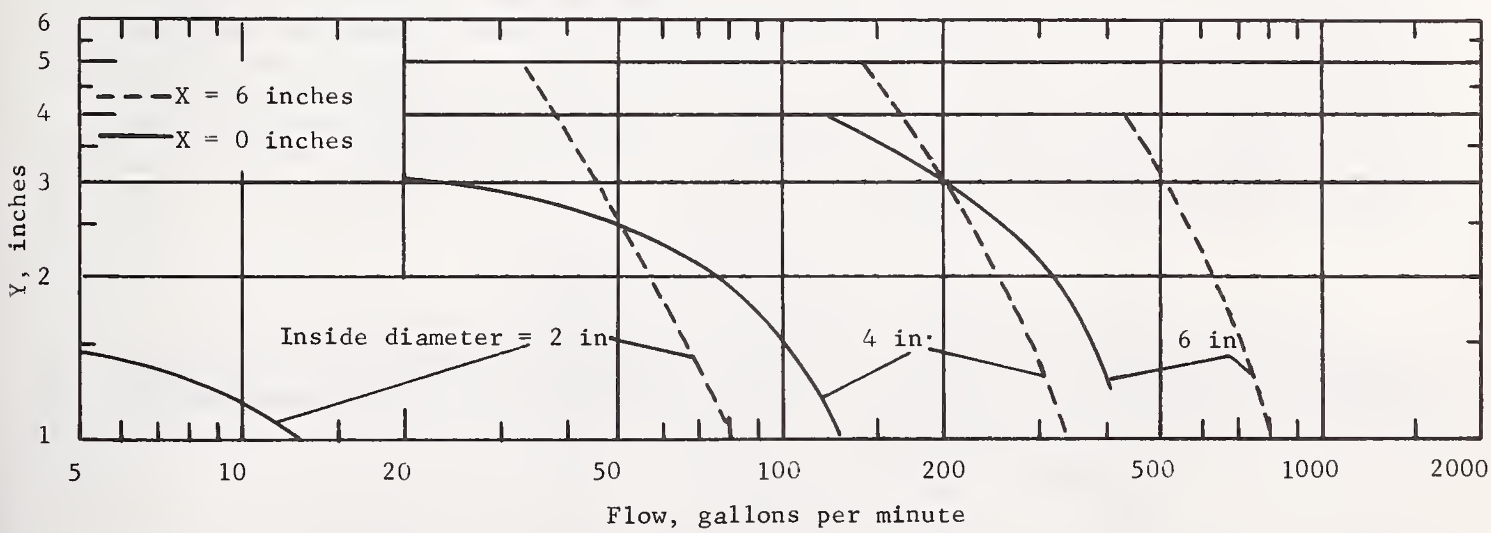


Figure 11.2. Discharge data for Purdue trajectory method, from ref. 11.2.2.1.

Case 2: 6-in. pipe, depth at end section is 2 inches. Using trajectory data (11.2.2), flow is 110 gpm. Using eq. 11.1, flow is 88 gpm. (The end depth method of chapter 10 gives 95 gpm.)

Some of this difference is caused by comparing exact 6-inch diameter pipe with commercial 6-inch pipe and by the fact that entrance conditions are not the same for the two methods. Nevertheless the results suggest that we should allow at least a few percent uncertainty when measuring the partly full condition, since we cannot assess which method warrants more confidence.

11.4. Trajectory Method (Theoretical)

11.4.1. Background

Treating the water jet as a freely falling body with a constant horizontal velocity component (equal to the issuing velocity) results in the equation

$$Q = A(g/2Y)^{0.5}X \qquad [11.2]$$

where Q is the discharge in cfs, A is the sectional area of the issuing water, and X and Y are trajectory coordinates measured as shown in figure 10.1, except that for this purpose Y should be referenced to the top of the issuing stream (if this is not the same as the top inside of the pipe). Some improvement in results would be obtained if Y could be determined conveniently for the jet centerline, particularly for large X.

11.4.2. General Information Sources

11.4.2.1. "Planning and Making Industrial Waste Surveys," Ohio River Valley Water Sanitation Commission, 414 Walnut Street, Cincinnati, Ohio 45202, 1952, $2.00.

In addition to illustrating the trajectory method, this publication covers other measuring methods -- Parshall flumes, weirs, California Pipe -- all in readily readable and understandable fashion, with numerical illustrations.

11.4.3. Comments

As was mentioned in section 11.2.3, the Purdue experimental results do not agree closely with eq. 11.2. Best overall agreement between the two methods was achieved for the 2-inch pipe at X = 18 inches, for which the differences were from 4 to 6 percent, depending on flow. This suggests that the theoretical trajectory computation will give better results if the measuring station is not too close (in terms of pipe diameters) to the pipe end, and it is recommended that X be taken as large as possible consistent with the trajectory remaining coherent and parabolic. There is insufficient experimental evidence at present to quantify this recommendation, but there is sufficient evidence to indicate that shorter distances give erroneous results. A point in favor of the theoretical trajectory method is that adjustments for pipe slope can be made analytically.

11.5. Orifice At End of Pipe

An orifice plate at the end of a pipe, with a trajectory issuing into air, is a simple device often used in irrigation systems. It could have application in certain industrial waste discharges as well. The setup is shown in figure 11.4, reproduced from ref. 11.15.1. Also shown are discharge coefficients, C, for use in

$$Q = CA\sqrt{2gh} \qquad [11.3]$$

where A is the orifice area in square feet, h is the head in feet measured as shown in figure 11.4, and Q is the flowrate in cubic feet per second (multiply by 449 to obtain gpm).

Figure 11.4 also describes how to make an orifice in a standard pipe cap fitting.

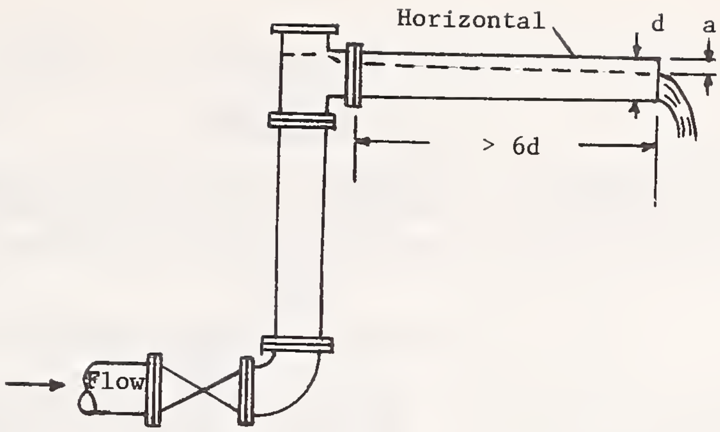


Figure 11.3. California pipe method, from ref. 11.3.2.1.

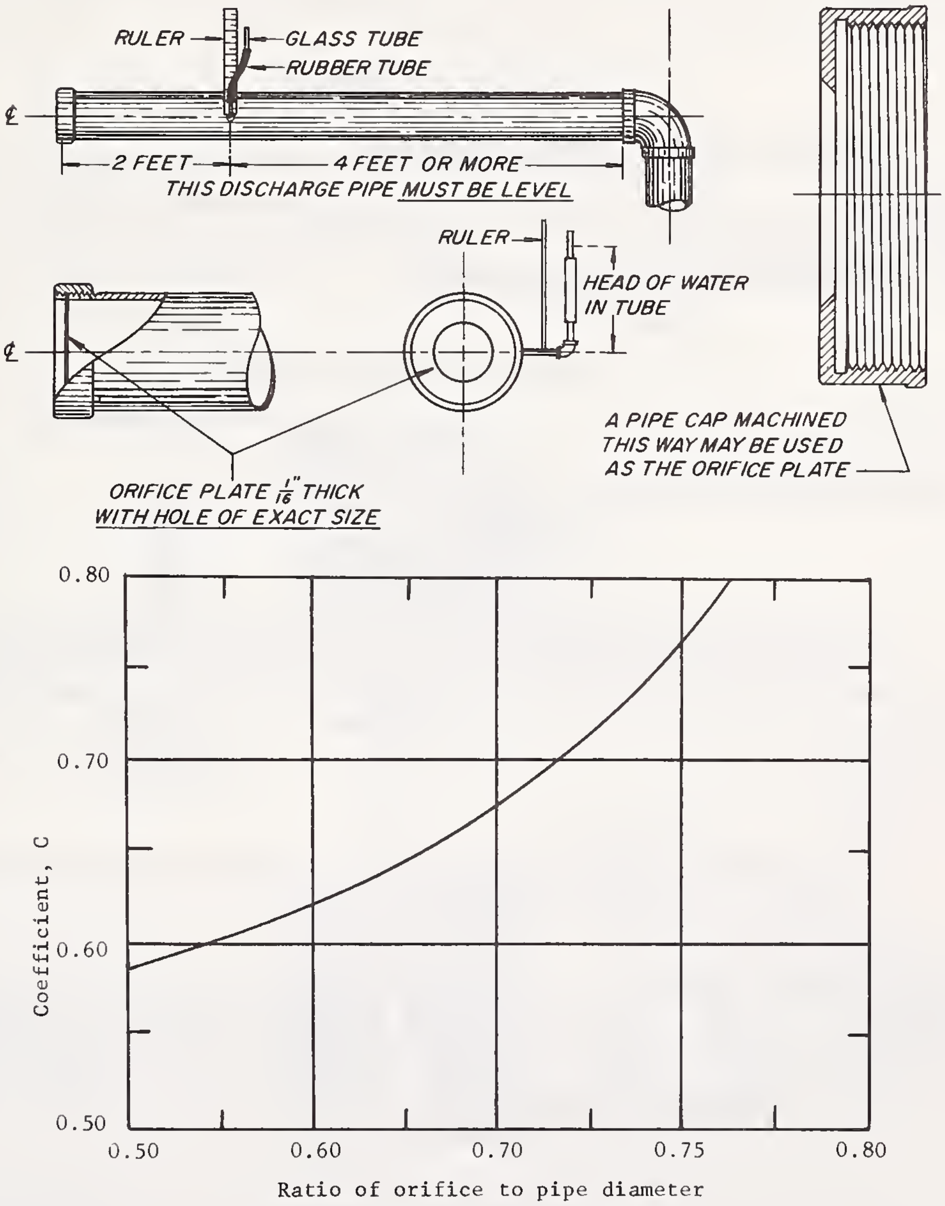


Figure 11.4. Orifice at end of pipe, from ref. 11.15.1.

11.6. Elbow Meters

11.6.1. Background

Centrifugal forces generated in flow around a pipe bend can be measured and used to determine flowrate. See figure 11.5. Certain commercial pipe elbows can be used (without in-place calibration) as ± 5 percent flowmeters using the procedures and data compiled in ref. 11.6.2.1.

11.6.2. General Information Sources

11.6.2.1. Replogle, J. A., Myers, L. E. and Brust, K. J., "Evaluation of Pipe Elbows as Flow Meters," Proc. Amer. Soc. Civ. Eng., 92, IR3, Sept. 1966, pp. 17-34. Obtain copy from Engineering Societies Library, 345 E. 47th St., New York, N. Y. 10017. $0.25 per page plus $3.00 handling.

11.6.2.2. "Fluid Meters -- Their Theory and Application," 6th edition, 1971, ASME. Obtain from American Society of Mechanical Engineers, 345 E. 47th St., New York, N. Y. 10017. $20 ($16 to ASME members).

11.7. Segmental Orifices

11.7.1. Background

A segmental orifice is shown in figure 11.6 and is used mainly for pipe flow which is heavily debris laden. Segmental orifice coefficients have not been completely characterized with regard to Reynolds number effects. Available information is given in the reference below.

11.7.2. General Information Sources

11.7.2.1. "Fluid Meters," ASME, see ref. 11.6.2.2 for ordering information.

See Chapter II-III, pp. 210-212 for meter coefficients.

11.8. Orifice Meters

11.8.1. Background

This section is concerned with orifice plates within a pipe, as shown in figure 11.7. This differential head device has long been used, along with the nozzle and Venturi meters of sections 11.9 and 11.10, for accurate flow measurements. Consequently there is a substantial body of literature on it, including standards and recommended practices, some of which are cited below.

The basic equation for discharge Q is

$$Q = CA_o(2gh)^{1/2}/(1 - \beta^4)^{1/2} \qquad [11.4]$$

where A_o is the area of the orifice, h is the head difference in height of water between upstream and downstream taps (which must be located exactly according to recommendations), β is the orifice-to-pipe diameter ratio, and C is a coefficient which depends upon geometry, Reynolds number and pipe relative roughness. Methods of estimating total tolerance on Q are given in refs. 11.8.2.2 and 11.8.2.3 below. In order to use standard values and tolerances for C, the installation conditions cited in the references must be adhered to strictly.

Because of the large head loss caused by orifice plates, they are not used in pumping systems where head is an important engineering or economic factor. Also their geometry makes them unsuitable for some solids-bearing flows, e.g., raw domestic sewage, as is also the case for thin-plate weirs.

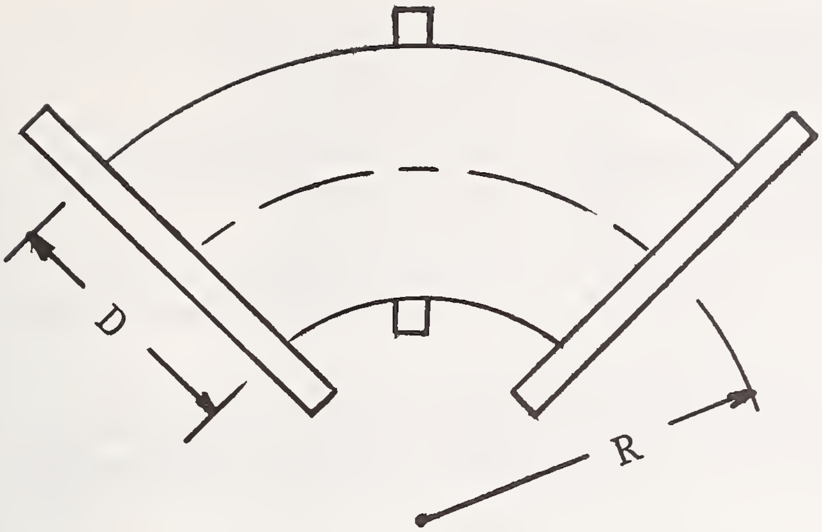
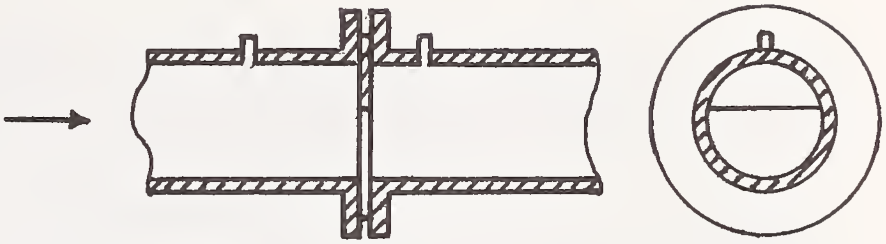


Figure 11.5. Elbow meter.

Figure 11.6. Segmental orifice.



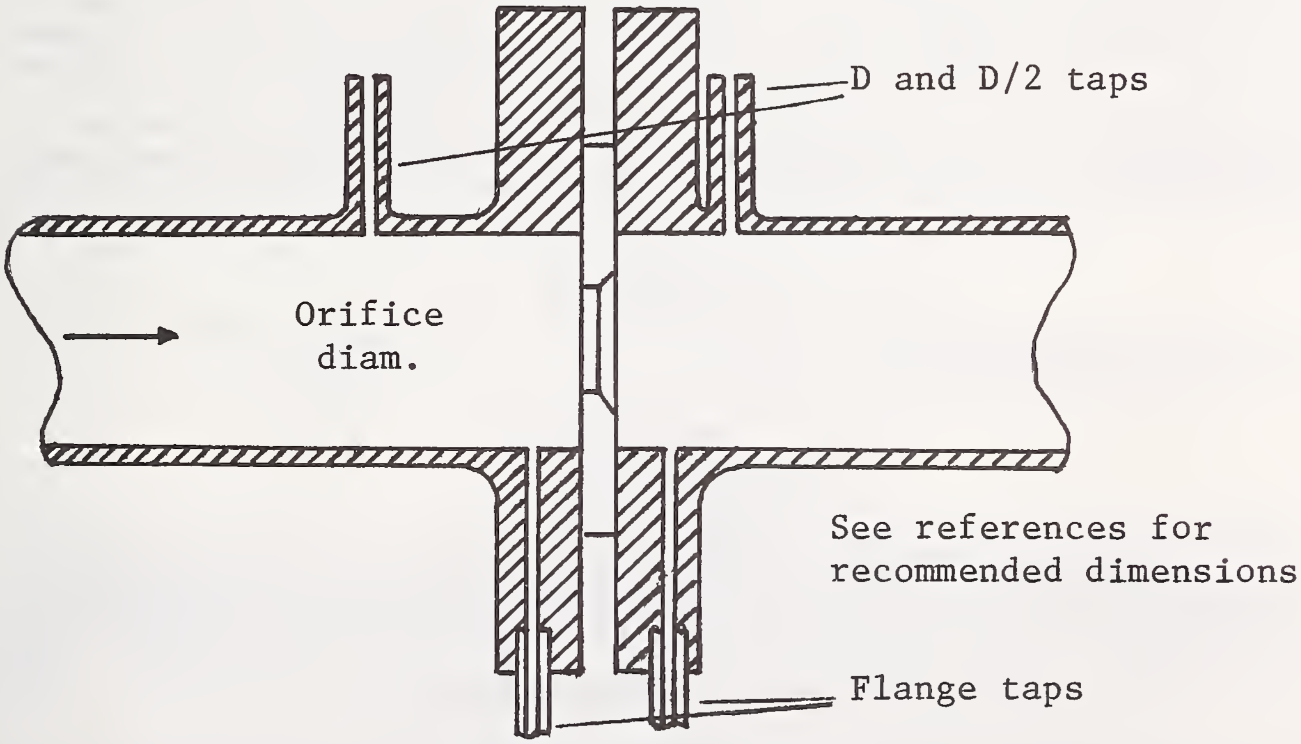


Figure 11.7. Orifice meter.

11.8.2. General Information Sources

11.8.2.1. "Fluid Meters," ASME, see ref. 11.6.2.2 for ordering information.

Chapter II-II of this book gives complete details on installation and pressure-measurement tubing layouts to minimize errors. Chapter II-III gives data on flow coefficients.

11.8.2.2. I. S. O. Recommendation R541, "Measurement of Fluid Flow by Means of Orifice Plates and Nozzles," 1967. Order from American National Standards Institute, 1430 Broadway, New York, N. Y. 10018, $15.75.

11.8.2.3. British Standard 1042: Part 1, "Methods for the Measurement of Fluid Flow in Pipes, 1. Orifice Plates, Nozzles and Venturi Tubes," 1964. Order from American National Standards Institute, 1430 Broadway, New York, N. Y. 10018. $13.00.

11.8.2.4. British Standard 1042: Part 3, "Methods for the Measurement of Fluid Flow in Pipes, 3. Guide to the Effects of Departure from the Methods in Part 1," 1965. Order from American National Standards Institute, 1430 Broadway, New York, N. Y. 10018. $5.40.

References 11.8.2.2 and 11.8.2.3 give detailed information on orifice geometry, installation and coefficients, and are particularly useful in estimating tolerances. Reference 11.8.2.4 gives information only on orifices and nozzles and not on Venturi tubes.

11.8.3. Errors

Published standards and recommendations give values for the meter coefficient C based on geometry, pipe size and roughness, and Reynolds number and attempt to assign tolerances for "standard" conditions and departures therefrom. The uncertainty of these published coefficients for standard installations is generally considered to be as high as 2 percent (e.g., ref. 11.8.2.1). However, errors in the differential head measurement can also contribute significantly to the system error. The user must estimate these on his own. It should be noted as a cautionary example, that a chart recorder (for head) which has an advertised accuracy of 0.5 percent full scale, can show a 5 percent error at 1/10th capacity. Therefore, it is important to try to design the measurement systems so that all components will operate as close as possible to rated capacity.

11.9. Nozzle Meters

A nozzle meter is illustrated in figure 10.8. Use the same general information sources as cited for orifice meters in section 10.8.

11.10. Venturi Meters

11.10.1. Background

Venturi meters have the advantage of reduced head loss relative to orifice and nozzle meters as well as capability for self cleansing. See figure 11.9. The basic equation is the same form as eq. 11.4, but with A_o now referring to the throat area. The coefficient C is a function of geometry, Reynolds number, roughness and installation conditions.

11.10.2. General Information Sources

11.10.2.1. I. S. O. Recommendation R781, "Measurement of Fluid Flow by Means of Venturi Tubes," 1968; and Appendix to R781, "Classical Venturi Tubes Used Outside the Scope Covered by I. S. O. Recommendation R781," 1969. Order from American National Standards Institute, 1430 Broadway, New York, N. Y. 10018. $14.70.

11.10.2.2. ASTM Standard D2458-69, "Standard Method of Flow Measurement of Water by the Venturi Meter Tube." American Society for Testing and Materials, 1916 Race Street, Philadelphia, Pa. 19103. $1.50.

11.10.2.3. "Fluid Meters," ASME. See ref. 11.6.2.2 for ordering information.

Chapter II-II is a good source for details on pipe layouts, pressure taps and pressure tubing layout, etc. See chapter III-III for meter coefficients.

11.10.2.4. See also ref. 11.8.2.3.

11.10.3. Errors

The uncertainty in published coefficients for "standard" meter and installation conditions ranges up to about 1.5 percent (e.g., ref. 11.8.2.1). See also orifice meter comments in section 10.8.3.

11.11. Magnetic Flow Meters

11.11.1. Background

The magnetic flowmeter operates on the same principle as an electric generator, i.e., an electromotive force is induced in a conductor. In this case the conductor is the flowing water, and insulated electrodes in the pipe (in a diametric plane normal to the magnetic field) in conjunction with a voltmeter measure the induced emf. Obvious advantages of this meter are (1) it is nonintrusive and can therefore be used in raw sewage, and (2) there is no added head loss. A disadvantage is its relatively high cost. The effect of variation in water conductivity is minimal provided there is a certain threshold conductivity. Velocity-distribution effects are still uncertain and caution should be observed in that regard.

Build-up of deposit or scale can be an error source depending upon the conductivity of the deposit. Regular cleaning is necessary. See ref. 11.15.2 for discussion of electrode maintenance.

Accuracies of the combined flowmeter-receiver systems depend upon the components selected and the method of calibration, but are generally within about 1 percent of full scale. To this date no standards or recommended practices have been published for this meter.

11.11.2. General Information Source

11.11.2.1. "Fluid Meters," ASME, see ref.11.6.2.2 for ordering information.

See Chapter I-0 of this reference, pp. 125-128; see also p. 255.

11.12. Acoustic Flowmeters

11.12.1. Background

Acoustic flowmeters usually employ the difference in transit time of upstream and downstream directed sonic pulses, this difference being caused by the velocity of the water in the conduit. See figure 11.10. Various manufacturers employ different electronic methods to take advantage of this principle. Another approach is to measure point velocities using the acoustic Doppler principle and to convert these to flow rates.

For the most part, the advantages and disadvantages previously listed for the magnetic flowmeter (section 11.11) prevail also for acoustic meters. Accuracy depends upon (1) the measurement capability of the electronic circuitry, (2) accuracy of measurement of sound velocity (in some methods the sound velocity cancels out of the equations), and (3) the accuracy with which the measured chordal velocities are integrated over the flow section.

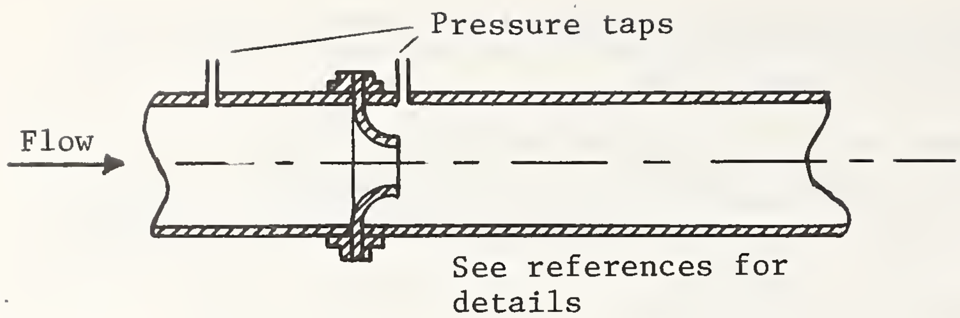


Figure 11.8. Nozzle meter.

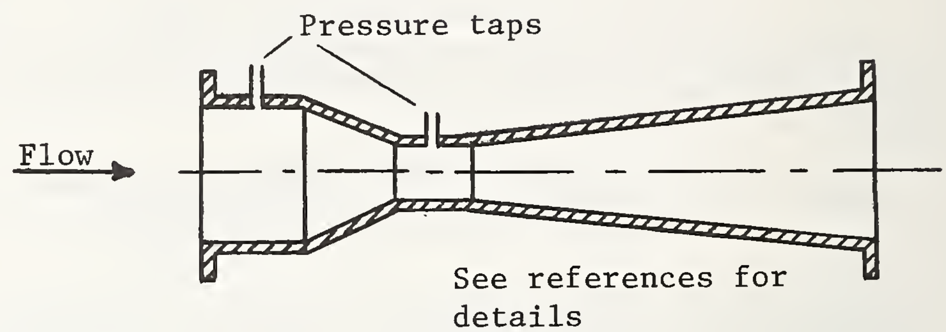
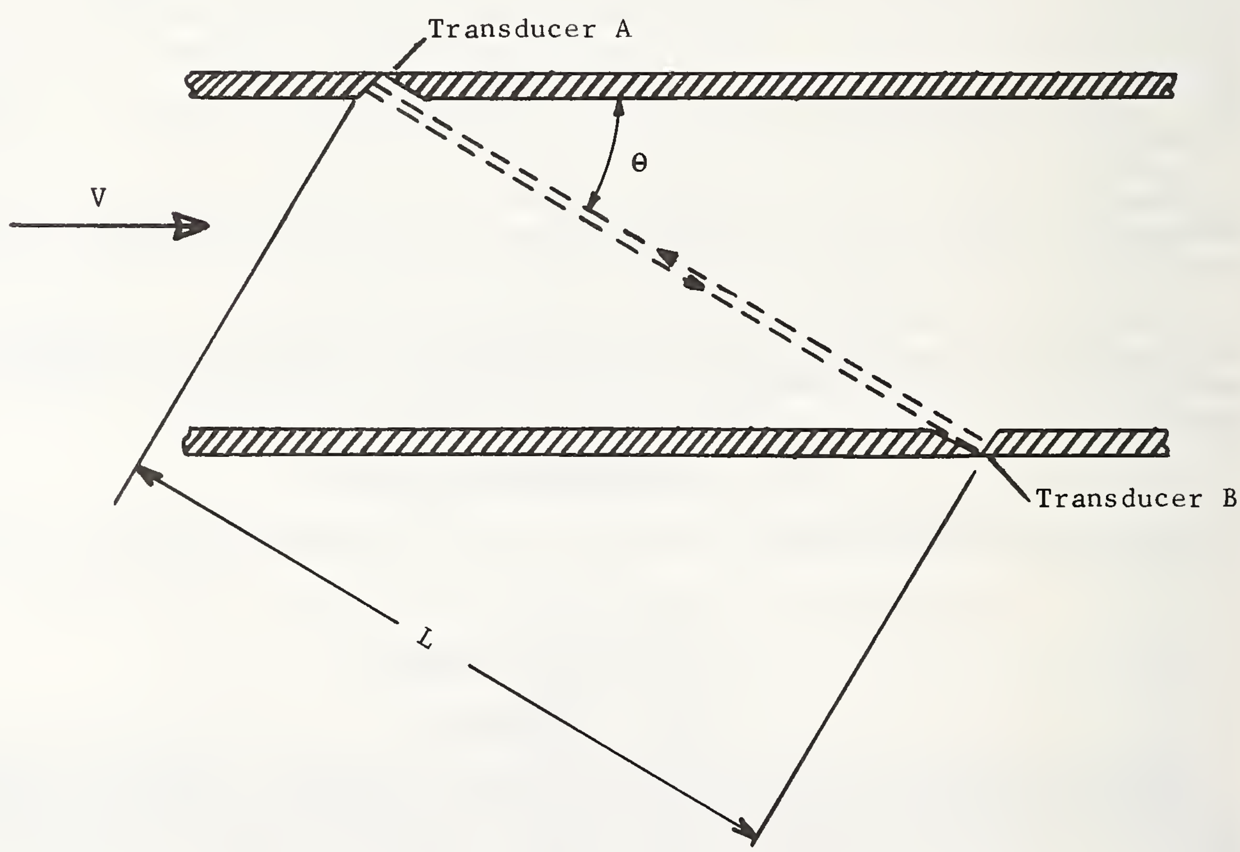


Figure 11.9. Venturi meter.



$t_{AB} = L/(C + V \cos\theta)$

$t_{BA} = L/(C - V \cos\theta)$

t is travel time, L is path length, C is sound speed

Figure 11.10. Acoustic meter principles.

According to manufacturers, at least 1 percent accuracy is obtainable. Information on extended field performance is minimal. No standards or recommended practices have been promulgated for acoustic meters.

11.12.2. General Information Sources

11.12.2.1. Hastings, C. R., "The LE Acoustic Flowmeter: An Application to Discharge Measurement," Jour. New England Water Works Assoc., 84, 2, June 1970, 127-151. Obtain from New England Water Works Association, 727 Statler Office Bldg., Boston, Mass. 02116. $0.50.

This paper describes the "leading edge" meter, in which the time difference between the upstream and downstream travel of the leading edge of sonic pulses is measured directly.

11.12.2.2. Genthe, W. K. and Yamamoto, M., "A New Ultrasonic Flowmeter for Flows in Large Conduits and Open Channels," Paper 2-10-57, AIP/ASME/ISA/NBS Symposium on Flow, Pittsburgh, Pa. 1971. This paper will appear in Vol. 1 of the symposium proceedings. Order from Instrument Society of America, 400 Stanwix St., Pittsburgh, Pa. 15222. $40.00 (three-volume proceedings, $110).

This paper describes a meter using the "sing-around" frequency counting method, i.e., a sound pulse arriving at an upstream transducer triggers a downstream pulse, etc. The reciprocals of the upstream and downstream travel times are frequencies and their differences can be measured.

11.12.3. Comments

The foregoing two references describe commercially available meters which use two different methods of effecting the difference measurement described in the first paragraph of this section. They were cited partly because they are recent but mainly because they are available in the open literature. They are not intended to comprise an all-inclusive list. Readers are reminded that other sonic meters are described in manufacturers' literature.

Acoustic flowmeters are also used in open channel flow, and sometimes another acoustic channel is added to measure flow depth. Schuster (11.15.3) details some of the problems encountered in acoustic flowrate measurements in a laboratory open channel.

11.13. Chemical Addition Methods

See chapter 9 for the salt velocity method and for dilution methods.

11.14. Velocity-Area Methods

See chapter 8, particularly section 8.7.

11.15. References

11.15.1. "Measurement of Irrigation Water," Section 15, Chapter 9 of National Engineering Handbook, Soil Conservation Service, U. S. Dept. of Agriculture, 1962. U. S. Government Printing Office, Wash., D. C. 20402. $0.45.

11.15.2. Grey, S., "Electrodes for Magnetic Flowmeters," Water and Sewage Works, 119, Reference Number, Aug. 1972, pp. R93-R97.

11.15.3. Schuster, J. C., "Measuring Water Velocity With an Ultrasonic Flowmeter," Bur. Reclamation, Eng. Res. Center, Rep. REC-ERC-72-32, Sept. 1972.

APPENDIX A

SI Conversion Factors

1 inch = 0.0254 metre

1 foot = 0.3048 metre

1 (U. S. liquid) gallon = 0.003785 $metres^3$ = 3785 $centimetres^3$

1 (U. S. liquid) gallon per minute = 63.09×10^{-6} $metres^3$ per second
= 63.09 $centimetres^3$ per second

APPENDIX B

Definition of Terms

α	See velocity distribution coefficient.
Aeration	In the context of this publication, the supplying of air to the underside of water overfalls, e.g., at weirs, to relieve the subatmospheric pressures which would otherwise be developed underneath the nappe.
Approach velocity	See velocity of approach.
Blockage	An effect on the registration of a velocity measuring instrument due to the presence of the instrument and its support in a conduit which is not of sufficient size relative to that of the instrument and support.
Boundary layer	The region of a flowing fluid which experiences the frictional effect of the solid boundary. In pipe or channel flow far downstream of any entrance effect, the boundary layer grows to occupy the entire flow. However, in developing flow, such as that over the crest of a rounded-edge weir (fig. 5.1b), the boundary layer thickness remains only a fraction of the depth.
Control (hydraulic)	In the context of this publication, control points occur at locations of critical flow. See critical flow.
Critical flow (or depth)	In open channels, a condition of maximum flow for available energy. For the same flowrate, flows which are faster (and shallower) than critical are called supercritical, and flows which are slower (and deeper) than critical are subcritical. A control point separates regions of subcritical (upstream) and supercritical (downstream) flow.
Displacement thickness	The amount by which streamlines are shifted away from the wall as a result of boundary layer formation. The displacement thickness times the velocity outside of the boundary layer equals the amount by which flowrate is

	reduced owing to the existence of the boundary layer.
Errors, random	Errors whose effect can be reduced by increasing the number of measurements.
Errors, systematic	Errors which result in an offset between the measured and "true" flowrate which cannot be reduced by repetition of the measurement.
Free overfall	See <u>overfall</u>.
Froude number	A dimensionless number describing the ratio between inertial and gravity forces, used in flow systems (such as open channel structures) where gravity is the important force producing motion. It is defined as $F = V/(gL)^{1/2}$ where g is the acceleration due to gravity and V and L are a characteristic velocity and length, respectively. In parallel open channel flow, a Froude number (based on depth and average velocity) of 1.0 corresponds to critical flow.
Hydraulic jump	A stationary and often extremely turbulent "shock" front which usually accompanies the passage of an open channel flow from an upstream supercritical condition to a downstream subcritical state.
Hydraulic radius	A geometric parameter used in determining flow resistance properties of a cross-section, defined as the cross-sectional area of flow divided by the <u>wetted</u> perimeter.
Jump, hydraulic	See <u>hydraulic jump</u>.
Kinetic energy coefficient	See <u>velocity distribution coefficient</u>.
Macro scale	See <u>turbulence</u>.
Overfall, free	A condition in which water flows over an overfall or brink in such a manner that the flow is unaffected by conditions downstream of the brink. "Free" overfalls usually refer to those in which the falling sheet of water is surrounded on all sides by atmospheric pressure.
Overfall, submerged	See <u>submerged flow</u>.
Reynolds number	A dimensionless number describing the ratio between inertial and viscous forces in a flow system, used chiefly (but not exclusively) in closed conduits. It is defined as $R = VL\rho/\mu$, where ρ is liquid density, μ is the coefficient of dynamic viscosity, V is an average velocity and L is a characteristic length, e.g., the diameter in pipe flow.
Scow float	A depth-measuring float used in the mainstream flow (rather than in a stilling well) of a weir or flume. It is often scow-shaped, and is

	connected to an arm which pivots about an upstream pin above the water surface.
Subcritical flow	Open channel flow in which the velocity is lower and the depth higher than for the critical condition (see critical flow). The Froude numbers for subcritical flow are less than 1.0.
Submerged flow, or overfall	A condition in which flow over a brink or weir is affected by the downstream water level. See also free overfall.
Supercritical flow	Open channel flow in which the velocity is higher and the depth lower than for the critical condition (see critical flow). The Froude numbers for supercritical flow are higher than 1.0. Physically speaking, water waves cannot propagate upstream against a supercritical flow; hence, the hydraulic jump, for example.
Turbulence, macroscale	If turbulence is considered to consist of a spectrum of turbulent eddy sizes, the macroscale is the size of the largest eddies in the flow and is of the same order as the conduit size.
Turbulence, r.m.s.	If one measures at a point the instantaneous velocities in, say, the direction of flow or x-direction, subtracts from them the time-average velocity at that point, squares the differences and takes the square root of the mean of these squares, the result is the x-component of the r.m.s. turbulence. The turbulence intensity is obtained by dividing this figure by the time-average velocity and expressing the result as a percentage.
Turnout	A structure for diverting water from a main irrigation canal to a subsidiary canal.
Uniform flow	Parallel flow in which velocities are the same from point to point along the conduit at a given time. This can occur only in a uniform conduit, i.e., a conduit whose cross-section is constant with length.
Velocity distribution coefficient, α	A factor larger than unity which is applied to $V^2/2g$ (where V is the average velocity in a section area, A) to give the true velocity head, thus allowing for non-uniform velocity distribution. $$\alpha = \int_o^A v^3 dA/V^3 A$$ where v is the local velocity over an incremental area dA.
Velocity head	$V^2/2g$ or $\alpha V^2/2g$

Velocity of approach	The average velocity in the measuring section immediately upstream of a weir, flume or differential head measuring device.

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Selected information sources on methods and standards for making measurements of water and wastewater flow in the field are listed and described. Both closed conduit and free surface flows are treated, but emphasis is on open channel flow measurements needed in water resource engineering and in water pollution control. Instruments and methods covered include weirs, flumes, current meters (and velocity traverse methods), dilution techniques, pipe flow instruments, acoustic meters and others. In addition to summarizing the basic properties of each instrument or method and referring users to the best available sources of detailed information on performance and field application, potential sources of error are described and quantified where possible.

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