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Code: 293

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

WATER MEASUREMENT PROCEDURES IRRIGATION OPERATORS' WORKSHOP-1967

Report No. Hyd-577

HYDRAULICS BRANCH
DIVISION OF RESEARCH

OFFICE OF CHIEF ENGINEER
DENVER, COLORADO

OCTOBER 15, 1967

Where approximate or nominal English units are used to express a value or range of values, the converted metric units in parentheses are also approximate or nominal. Where precise English units are used, the converted metric units are expressed as equally significant values.

ABSTRACT

"Water Measurement Procedures" was written primarily for use in the Irrigation Operators' Workshop classes as a teaching aid in presenting the fundamentals of water measurement to field personnel engaged in irrigation work. Technical material has been simplified to provide a clear understanding of water measurement devices and procedures. Basic hydraulics presented includes the discharge equation, velocity head concept, orifice and weir relationships, and the effect of submergence. Factors affecting the accuracy of measurement such as worn equipment, infrequent head measurement, use of wrong measuring device, and others are analyzed. Commonly used devices and methods are discussed including orifices, weirs, Venturi meters, Parshall and Venturi flumes, meter gates, constant head turnouts and propeller meters; new devices and methods include vane deflection meters, acoustic and magnetic meters, and the dilution and radioisotope methods of measurement. Hints for troubleshooting poorly operating devices, suggestions to operators on how to do a good job, and a selected reading list for operators are given. This edition supersedes the previous 1965 and 1966 editions numbered Hyd 552 and 565. Has 34 references.

DESCRIPTORS-- *hydraulics/ *hydraulic structures/ *discharge measurement/ *irrigation/ water delivery/ *water measurement/ open channel flow/ errors/ closed conduit flow/ discharge coefficients/ weirs/ orifices/ water meters/ current meters/ water metering/ irrigation O&M/ turbulent flow/ Venturi meters/ instruction/ Venturi flumes/ Parshall flumes/ submerged orifices/ radioactive isotopes/ velocity/ field investigations/ flow meters/ laser/ bibliographies/ submergence/ instrumentation
IDENTIFIERS-- textbooks/ Irrigation Oper Workshop/ dilution method/ vane flow meters/ ultrasonic flow measurement/ acoustic flow meters/ magnetic flow meters/ constant-head orifices

UNITED STATES
Department of the Interior
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Special Investigations Section
Denver, Colorado
October 15, 1967

Report No. Hyd-577
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Water Measurement Procedures
Irrigation Operators' Workshop--1967

This report contains accumulated lecture notes "Water Measurement Procedures" presented during 7 consecutive years of "Irrigation Operators Workshop" meetings conducted by the Bureau of Reclamation, Chief Engineer's Office, in the Denver Federal Center, Denver, Colorado. In previous years, the notes were made available only to the Workshop participants. However, because of increasing general interest and numerous requests for information on water measurement, these notes have been reproduced as a Hydraulics Branch Report to make them more accessible to a greater number of readers. This report supersedes Hyd-552 and -565.

Bureau of Reclamation
Denver Federal Center

IRRIGATION OPERATORS' WORKSHOP - 1967

WATER MEASUREMENT PROCEDURES

by

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Hydraulics Branch
Division of Research

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FOREWORD

These notes contain the essential parts of six previous workshop sessions presented between 1961 and 1966. The purpose of this writing is to fulfill the needs of operators in making meaningful and accurate measurements of irrigation water.

After a discussion of the need for improved measuring devices and techniques, standard and nonstandard devices are defined and their implications in regard to water measurement are discussed.

Water-measuring devices and methods are classified under three categories: (1) the velocity device, (2) the head device, and (3) miscellaneous devices, including chemical and dye dilution methods, total count radioisotope methods, magnetic methods and sonic methods.

Under "Some Basic Hydraulics," the concepts of the discharge equation and velocity head are developed; these two concepts are used to derive the basic equations for both orifice and weir discharge using the simplest methods possible. Several of the general aspects of water measurement accuracy are also discussed.

The procedures for operating flumes and weirs, probably the most common devices, are used to furnish examples of good and poor water measurement practices because the effects of good and bad measurement practices are often not visible on some of the more sophisticated measuring devices.

The more complicated devices and techniques such as the submerged orifice meter, Venturi meter, metergate, and constant-head orifice turnout measuring device, and propeller meters are then described.

Hints are given for troubleshooting metering devices suspected of being inaccurate and instructions are given for selecting the proper size and obtaining proper installation of metergates, constant-head orifice meters, and Parshall flumes.

Progress in water measuring techniques, including the chemical dilution and radioisotope methods is reported, and an evaluation of a commercially available open channel deflection meter is given. Progress in the development of magnetic and acoustic meters is reported.

The section on Parshall flumes brings together under one cover the essentials needed to understand the size selection, vertical placement, discharge determination procedures for free and submerged flows, and the theory of flume operation and performance. Sample problems are used to illustrate proper procedures.

A list of reference material and a chart showing the head required to operate certain measuring devices are presented.

NEED FOR IMPROVING WATER-MEASURING DEVICES AND TECHNIQUES

It is impossible at the moment to overstate the need for an improved water measuring program, not only in your district, project, or region, but nationwide, and even worldwide. The population explosion and the shifting of the population center of the United States toward the West is causing concern in that water needs may eventually limit this movement. According to the latest figures, if we account for births, deaths, immigrants, and emigrants, a new person arrives on the scene every 10 seconds; 8,000 a day; 3,000,000 a year. By the end of the century the population will have doubled. Twice as many people--the same amount of water. Within the next decade, every water supply, not only those in the West but throughout the United States, will be critically examined to determine quantity, use, and waste. Plans will then be formulated for extending the use of the water.

One way to increase the quantity of available water, of course, is to find new water sources. This is not always possible and is usually costly. The other way is to conserve and equitably distribute the water now developed and available. This latter course is usually possible and less expensive. We are interested in the latter course because this is our business and, as all of you know, more extensive use of the available water can be made if all measuring devices are accurate and dependable at all times and the best water management procedures are followed at all times. It is, therefore, important that every attempt be made to upgrade existing water-measuring devices, not only in ditches, laterals, and canals, but in the supply systems as well. Every cubic foot of water "saved" as a result of improving the measuring devices is more valuable than a similar amount obtained from a new source because the saved water can be produced at considerably less cost.

Accuracy in water measurement is therefore of prime importance in the operation of any water distribution system. Even though a surplus of water may now exist on your project, it is time to begin accurate accounting of what happens to your water. An experience on a project actively engaged in upgrading water-measurement devices and procedures will serve as a good example. A water user entitled to 1 cfs (cubic feet per second) had been receiving up to 5 cfs because water had been plentiful and no accurate measuring device had been installed at his turnout. When a new meter was installed and 1 cfs was delivered, he complained that his new concrete field distribution system would not operate properly and much of his acreage would have to go without water until he could change the elevation and slope of his ditches.

This actual occurrence illustrates another point which is important in water measurement. Practically all measuring devices, when in rundown condition or when improperly installed, deliver more water than they indicate they are delivering. The very nature of most measuring devices makes it impossible for a device to deliver less water than it indicates. For this reason, water accounting records may not show a proper division between water used and water lost through seepage or waste. Proper evaluation of losses is necessary to establish the economic advisability of providing canal linings. Canal linings obviously cannot help to recover water lost through poor measuring equipment or procedures.

The purpose of this presentation is to discuss the factors in flow measurement devices which affect the accuracy of discharge measurement. To accomplish this, it is necessary to understand the basic flow principles involved and to know how each flow factor can influence the flow quantity indicated. By upgrading existing measurement devices or by properly installing and maintaining new devices, considerable quantities of water can be made available for new uses. This "new" water can be produced at considerably less cost than can a similar quantity be developed from a new source.

STANDARD AND NONSTANDARD DEVICES

It has been said that a waterlogged boot which is partially blocking the flow in a ditch can be a measuring device--if it is properly calibrated. Certainly the boot would be a nonstandard device because no discharge tables or curves are available from which to determine the discharge. Many other devices including certain weirs, flumes, etc., are also nonstandard because they have not been installed correctly and, therefore, do not produce standard discharges. Although these commonly used devices may appear to be standard devices, closer inspection often reveals that they are not, and like the boot, must be calibrated to provide accurate measurements.

A truly standard device is one which has been fully described, accurately calibrated, correctly installed, and sufficiently maintained fo fulfill the original requirements. Standard discharge tables or curves may then be relied upon to provide accurate water measurements.

Any measuring device, therefore, is nonstandard if it has been installed improperly, is poorly maintained, is operated above or below the prescribed limits, or has poor approach (or getaway) flow conditions. Accurate discharges from nonstandard structures can be obtained only from specially prepared curves or tables based on calibration tests as current meter ratings.

Calibration tests can be quite costly when properly performed. Ratings must be made at fairly close discharge intervals over the complete operating range, and curves and/or tables prepared. It is, therefore, less costly and usually not too difficult to install standard devices and maintain them in good condition. Standard discharge tables may then be used with full confidence.

In maintaining a standard structure it is only necessary to visually check a few specified items or dimensions to be sure that the measuring device has not departed from the standard. In maintaining a nonstandard device it is difficult to determine by visual inspection whether accuracy is being maintained except by recalibration, an expensive procedure.

BASIC PRINCIPLES OF WATER MEASUREMENTS

To upgrade existing water-measuring devices and improve the quality of installation of new devices it is necessary to understand some of the basic principles which influence the quantity of water passed by a measuring structure. Most devices measure discharge indirectly, i.e., velocity or head is measured directly and prepared tables or an indicator are used to obtain the discharge. Measuring devices may be classified, therefore, in two groups: (1) velocity type and (2) head type. Those using the velocity principle include:

- Float and stopwatch
- Current meters
- Propeller meters
- Flow boxes
- Vane deflection meters
- Magnetic and acoustic meters
- Salt velocity method
- Color velocity method

When the velocity (V) principle is used, the area of the stream cross section (A) must be measured and the discharge (Q) computed from $Q = AV$.

Devices using the head principle include:

- Pitot tubes
- Rectangular weirs
- V-notch or multiple-notched weirs
- Cipoletti weir
- Parshall flume, Venturi flume
- Metergates
- Orifice or Venturi meters
- Constant head turnouts
- Clausen weir gage (or stick)

When the head (H) principle is used, the discharge (Q) may be computed from an equation such as the one used for a sharp-crested rectangular weir of length L,

$$Q = CLH^{3/2}$$

The area of the cross section (A) does not appear directly in the equation but C, a coefficient, does. C can vary over a wide range in a nonstandard installation but it is well defined for standard installations.

Special methods and devices may also be used and these include dilution methods which utilize chemicals or radioisotopes, acoustic or magnetic meters, tapered-tube-and-float devices, and many others which are not commonly used. In the dilution method the discharge is determined by calculating the quantity of water necessary to dilute a known quantity of concentrated chemical, dye, or radioisotope solution, injected into a flowing stream, to the strength obtained by sampling the stream after thorough mixing has taken place. Chemical analysis, color comparison, or gamma ray counting may be used to determine the degree of dilution of the injected sample. In the acoustic meter the variation in velocity of a sound pulse produced by the water velocity is used to determine the average water velocity of the flow. In the magnetic meter the flowing water disturbs the lines of force in a magnetic field to produce a voltage that can be related to the discharge. Pitot tubes and tapered-tube-and-float devices utilize the velocity head to indicate velocity.

SOME BASIC HYDRAULICS

DERIVATION OF DISCHARGE EQUATION

The volume of a cube, Figure 1, may be found by multiplying a times b times c

a times b gives the area A

2 x 2 = 4 sq ft

A times c gives the volume

4 x 2 = 8 cu ft

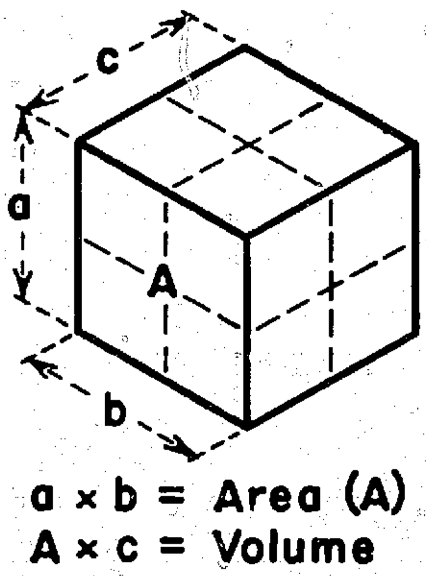
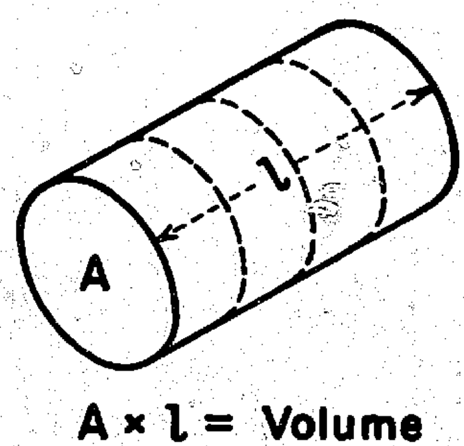


Figure 1

The volume of a cylinder (or pipe) may be similarly found. If the A for the cylinder is 2 sq ft and l is 4 ft then

area, A, times l gives the volume

2 x 4 = 8 cu ft



The measurement of discharge is actually a volume measurement and can be calculated in the same way using different terms

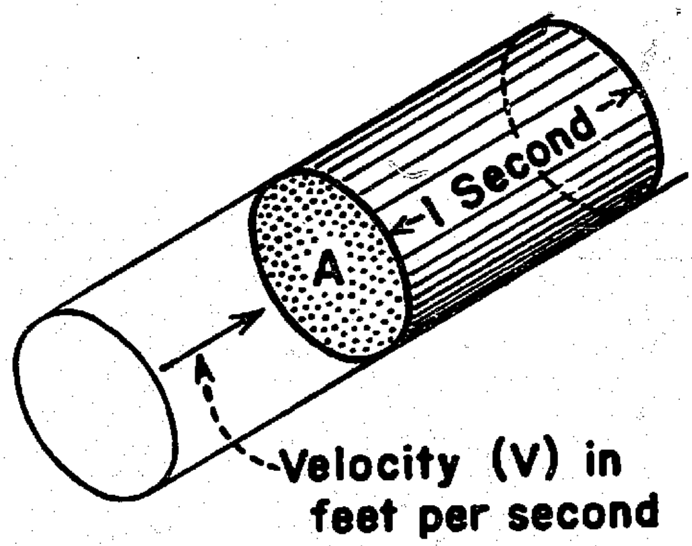


Figure 2

Instead of using 1, as before, substitute the velocity, V, Figure 2, and,

$$\text{Discharge} = \text{Area} \times \text{Velocity}$$

or

$$Q = AV$$

Since "Area" units are in square feet and "Velocity" units are in feet per second

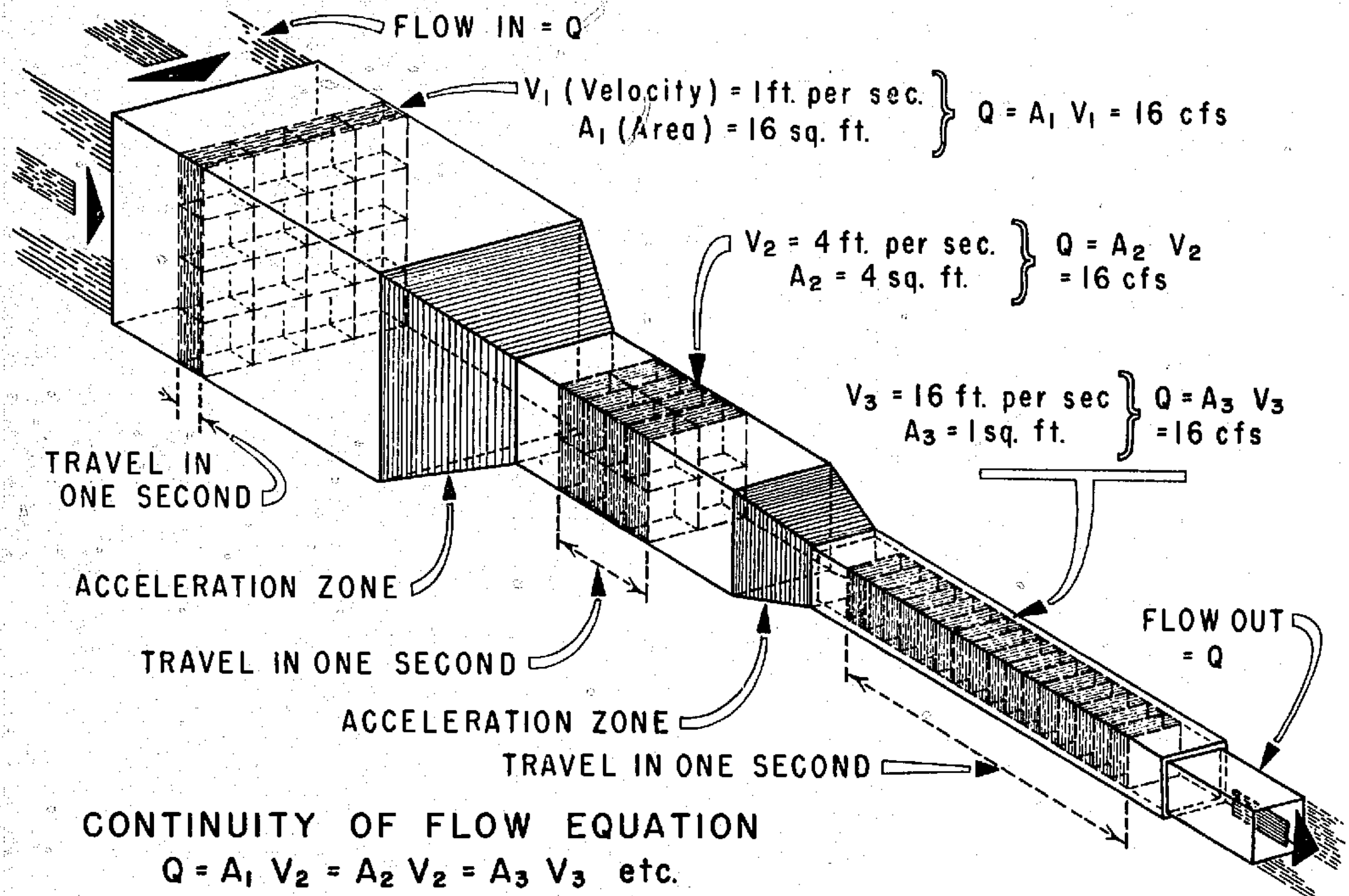
$$Q = \text{square feet times feet per second}$$

$$Q = \text{cubic feet per second}$$

Therefore, the discharge Q in cubic feet per second is equal to the Area of the flow cross section in square feet times the Velocity in feet per second. The relationships between A and V in a pipeline of varying cross section are shown in Figure 3.

The first basic equation is

$$Q = AV \qquad \ldots\ldots (1)$$



CONTINUITY OF FLOW EQUATION

$Q = A_1 V_2 = A_2 V_2 = A_3 V_3$ etc.

FIGURE 3

DERIVATION OF VELOCITY HEAD CONCEPT

It has been found by direct measurement that an object (including water) falling from rest will fall 16.1 feet in the first second. Because of the continuing acceleration caused by gravity (32.2 feet per second per second), the object will fall 64.4 feet in 2 seconds, etc., as shown in Table 1.

Table 1

Time in seconds	Vertical fall feet	Instantaneous velocity
1	16.1	32.2
2	64.4	64.4
3	144.9	96.6
4	257.6	128.8
5	400.0	161.0

At the end of the first second the velocity will be twice the vertical fall distance (because the object started from rest at zero velocity) or 32.2 feet per second, Table 1.

Since the acceleration caused by the steady pull of gravity, g, is constant, the velocity values in Table 1 increase 32.2 for each succeeding second.

An equation to express these facts is

$$V = \sqrt{2gH} = \sqrt{2g}\sqrt{H} \qquad \ldots\ldots (2)$$

where g is the acceleration due to gravity and H is the height of fall.

Substituting values from Table 1

$$V = 8.02\sqrt{16.1} = 32.2$$

$$V = 8.02\sqrt{64.4} = 64.4, \text{ etc.}$$

The equation is therefore seen to be valid.

After squaring both sides, Equation 2 becomes

$$V^2 = 2gH \text{ or } H = \frac{V^2}{2g} \qquad \ldots\ldots (3)$$

The latter expression is often used to express velocity head, i.e., the head necessary to produce a particular velocity. It is discussed further under "Velocity of Approach," page R9-23.

BASIC ORIFICE RELATIONSHIP

Equations 1 and 2 can be used to develop an equation for the flow through an orifice (a hole in the side or bottom of a container of water, Figure 4).

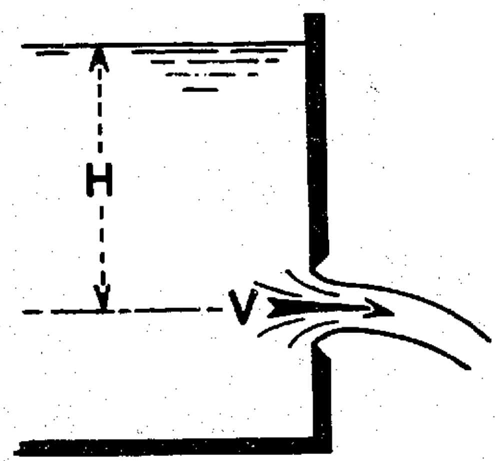


Figure 4

To find the velocity of flow in the orifice, use Equation 2, $V = \sqrt{2gH}$. Then using the size or area of the hole, the quantity of water being discharged, Q, can be determined using Equation 1, Q = AV. This can be accomplished in one step, however, if the two equations are combined.

Inserting the value $\sqrt{2gH}$ for V in Q = AV gives

$$Q = A\sqrt{2gH} \qquad \ldots\ldots (4)$$

the equation for the theoretical discharge through an orifice. This equation assumes that the water is frictionless and is an ideal fluid. Since water is not an ideal fluid, a correction must be made.

The diameter of the water jet continues to contract after passing through the orifice, and if the orifice edges are sharp, the jet will appear as shown in Figure 5.

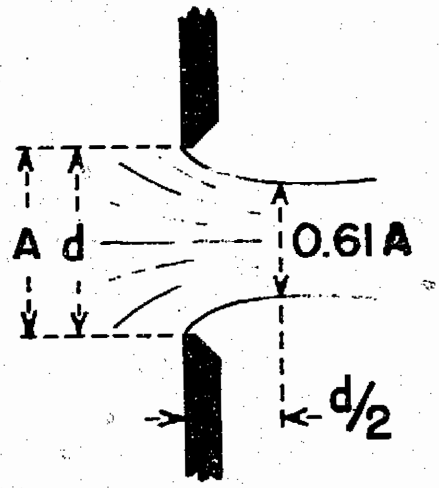


Figure 5

To explain, at one-half of the orifice diameter downstream from the sharp edge, d/2, the maximum jet contraction will occur and the cross-sectional area of the jet of water will be only about 6/10 of the area of the orifice, A. The maximum velocity also occurs at this point, and so Equation 4 must contain a coefficient "C" (0.61 for an orifice in a large container) to determine the quantity of water being discharged.

$$Q = CA\sqrt{2gH} \qquad \ldots\ldots (5)$$

For a coefficient of 0.61, the equation would become

$$\begin{aligned} Q &= 0.61\ A\sqrt{2g}\sqrt{H} \\ &= 0.61\ A\ 8.02\sqrt{H} \\ &= 4.89\ A\sqrt{H} \end{aligned}$$

The value 0.61 should not be used indiscriminately, however, calibration tests establish the proper value for a particular orifice under a given set of conditions.

BASIC WEIR RELATIONSHIP

Equations 1 and 2 can also be used to develop an equation for flow over a weir (a sharp-edged blade that measures discharge in terms of overflow depth or head, Figure 6).

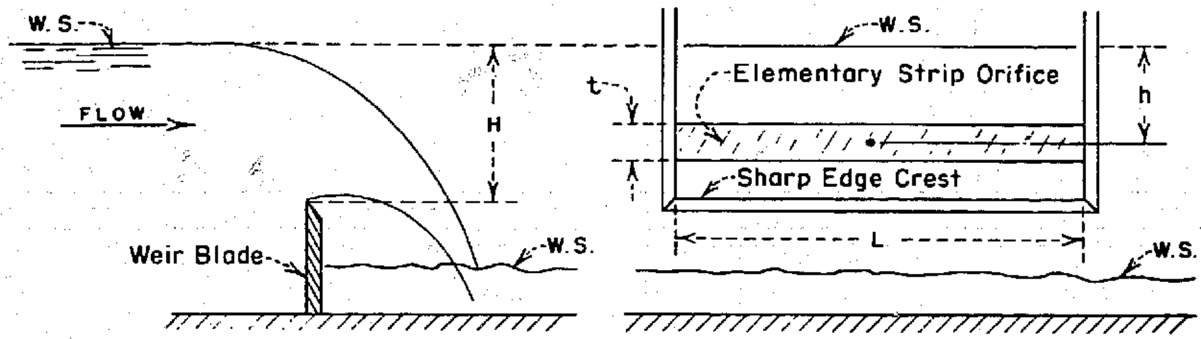


Figure 6

The weir flow is divided into small elementary horizontal strips and each is considered to be a long narrow rectangular orifice of length (L), height (t), and a flow producing head (h). The area (a) of each strip is

$$a = tL$$

The velocity (B) of each element is expressed by Equation 2:

$$V = \sqrt{2gh}$$

Since q = VA by Equation 1, the discharge (q) of each element is:

$$q = tL\sqrt{2gh}$$

To obtain the total discharge (Q) over the weir, the sum of all elementary discharges (q) must be taken. For example, the flow is divided into two elementary strip orifices, Figure 7.

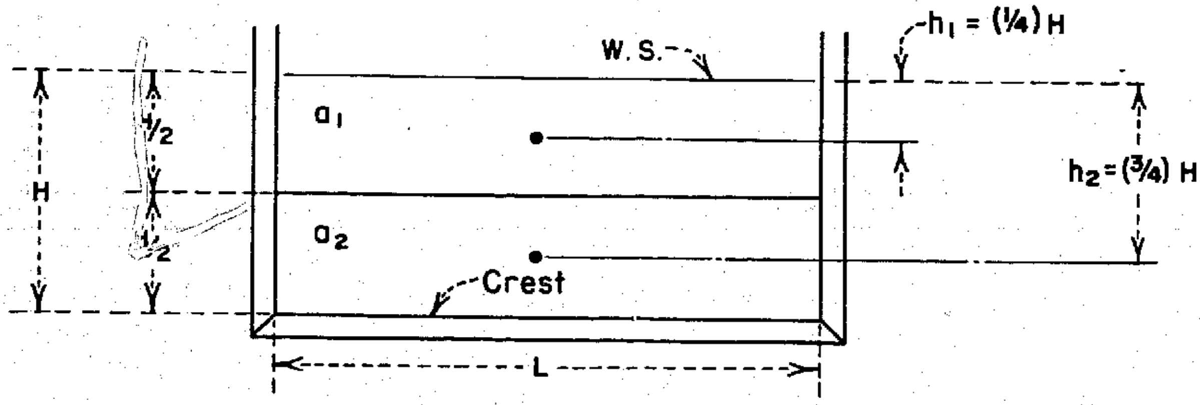


Figure 7

By Equation 2,

$$V_1 = \sqrt{2gh_1} = \sqrt{2g\left(\frac{1}{4}\right)H}$$

$$V_2 = \sqrt{2gh_2} = \sqrt{2g\left(\frac{3}{4}\right)H}$$

and the areas of the elements are equal and expressed by

$$a_1 = a_2 = L\left(\frac{H}{2}\right)$$

By Equation 1 and addition, the total discharge (Q) over the weir is given by

$$Q = a_1 V_1 + a_2 V_2$$

$$Q = L\left(\frac{H}{2}\right)\sqrt{2g\left(\frac{1}{4}\right)H} + L\left(\frac{H}{2}\right)\sqrt{2g\left(\frac{3}{4}\right)H}$$

Taking ($\sqrt{2g} \cdot LH\sqrt{H}$) out of each expression on the right-hand side:

$$Q = \left(\frac{1}{2}\sqrt{\frac{1}{4}} + \frac{1}{2}\sqrt{\frac{3}{4}}\right)(\sqrt{2g} \cdot LH\sqrt{H})$$

and

$$\left(\frac{1}{2}\sqrt{\frac{1}{4}} + \frac{1}{2}\sqrt{\frac{3}{4}}\right) = K = 0.683$$

As more and more strips are used in the analysis K approaches the value 0.667. Therefore,

$$Q = \frac{2}{3}\sqrt{2g} \cdot LH\sqrt{H}$$

Introducing a constant (C') to account for contraction and friction losses, a more accurate discharge formula is:

$$Q = \frac{2}{3}\sqrt{2g} \cdot C' LH\sqrt{H}$$

By regrouping and letting

$$C = \frac{2}{3}\sqrt{2g} \cdot C'$$

$$Q = CLH\sqrt{H}$$

or

$$Q = CLH^{3/2} \qquad \ldots\ldots (6)$$

This relationship is the basic weir formula and can be modified to account for weir blade shape and velocity of approach. However, C must be determined by calibration tests carefully conducted. C usually varies from about 3.3 for a broad-crested weir to about 3.8 or more for a sharp-crested weir.

The two examples given indicate that discharge determination methods are a mixture of rationalized thinking and coefficient evaluation. The equations are useful in making calibrations because they reduce the number of calibration points required to make up a discharge table. However, the equations alone will not suffice without sufficient testing to establish the value of C.

GENERAL ASPECTS OF WATER MEASUREMENT ACCURACY

In this portion of the text general aspects of water measurement accuracy will be considered. The performance of weirs and flumes will be used to illustrate flow and accuracy principles because the average irrigation operator is more familiar with their use. Also, many of the factors which adversely affect accuracy are visible on these devices and are invisible on closed system devices. Many of the facts and principles established for weirs and flumes also apply to other water-measuring devices. These principles will be mentioned and elaborated upon later when the more sophisticated meters and techniques are considered.

FLOW CHARACTERISTICS REDUCING ACCURACY OF MEASUREMENT

In inspecting a water measuring station to judge or evaluate its probable accuracy, it should be determined whether the device is a head or velocity measuring station. This is the key to the order and importance of other observations. In either case, the first observation should concern the visible flow conditions just upstream from the measuring device.

APPROACH FLOW

Extremely large errors in discharge indication can occur because of poor flow conditions in the area just upstream from the measuring device. In general, the approaching flow should be the same as tranquil flow in long straight canals (without obstructions) of the same size. Any deviation from a normal horizontal or vertical flow distribution, or the presence of water surface boils, eddies or local fast currents, is reason to suspect the accuracy of the measuring device. Errors of 20 percent are not uncommon and may be as large as 50 percent or more, if the approach flow conditions are very poor. Sand, gravel, or sediment bars submerged in the approach channel, weeds or riprap obstructions along the banks or in the flow area can cause unsymmetrical approach flow. Other causes may be too little distance downstream from a drop, check, turnout or other source of high velocity or concentrated flow, a bend or angle in the channel just upstream from the measuring device, a too rapid expansion in the flow section, or an eddy tending to concentrate the flow cross section.

Figure 8 shows an example of a poor approach to a weir. The high-velocity, turbulent stream is approaching the weir at a considerable angle. Head measurement is difficult because of the high-velocity approach flow and the waves on the surface. This weir will not discharge a "standard" quantity of water consistent with the measured head.

Standard weir installations for rectangular, Cipoletti, and 90° V-notch weirs are shown in Figure 9. The velocity of approach to a weir should be less than 0.5 foot per second. This value is obtained by dividing the maximum discharge by the product of channel width B and depth G measured at a point 4H to 6H upstream from the blade.

TURBULENCE

Turbulence is the result of relatively small volumes of water spinning in a random pattern within the flow mass as it moves downstream. It may be recognized as water surface boils or three-dimensional eddies which appear and disappear in a haphazard way. Because of this local motion within the general motion of the flow mass, any particle of water may at any given instant be moving forward, sideways, vertically, or even backward. In effect, then, the water is passing a given point with a start and stop motion rather than with a uniform velocity which is ideal. It may be said that turbulent water does not flow as a train of railroad cars on a level track, but rather as a train of cars coupled with elastic bands, traveling over a series of rises, dips, and horizontal curves. Thus, fewer or more cars may pass a given point over identical short periods of time, depending on the observation point chosen. Turbulent water flows in the same manner, Figure 8.

Excessive turbulence will adversely affect the accuracy of any measuring device but is particularly objectionable when using current meters or propeller meters of any kind. Turbulence can be objectionable even without the "white water," caused by air entrainment, which is often associated with it. Turbulence is usually caused by a stilling basin or other energy dissipator immediately upstream, by a sudden drop in water surface or by obstructions in the flow area such as operating or nonoperating turnouts having projections or indentations from the net area. Shallow flow passing over a rough or steep bottom can also be the cause. Weeds or riprap slumped into the flow area or along the banks, or sediment deposits upstream from the measuring device also can cause excessive turbulence.

Measuring errors of up to 10 percent or more can be caused by excessive turbulence and it is absolutely necessary that all visible signs of turbulence be eliminated upstream from a measuring device.

Poor approach flow conditions upstream from weir. The high-velocity, turbulent stream is approaching the weir at a considerable angle. Head measurement is difficult, and the weir does not discharge a "standard" quantity.

Figure 8

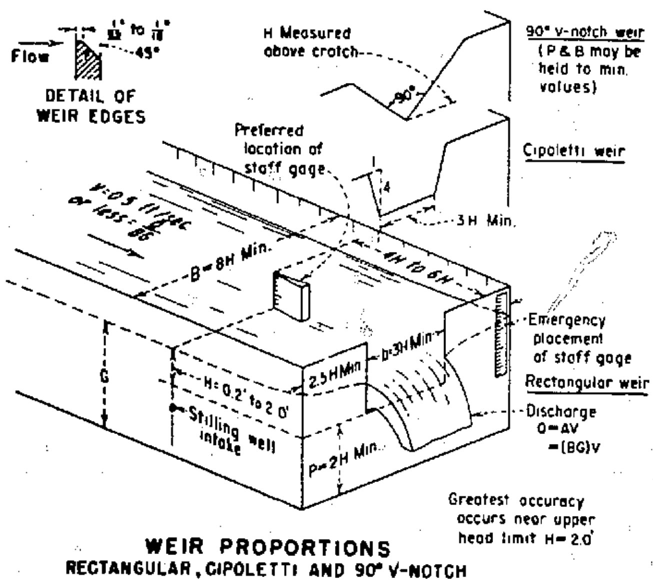
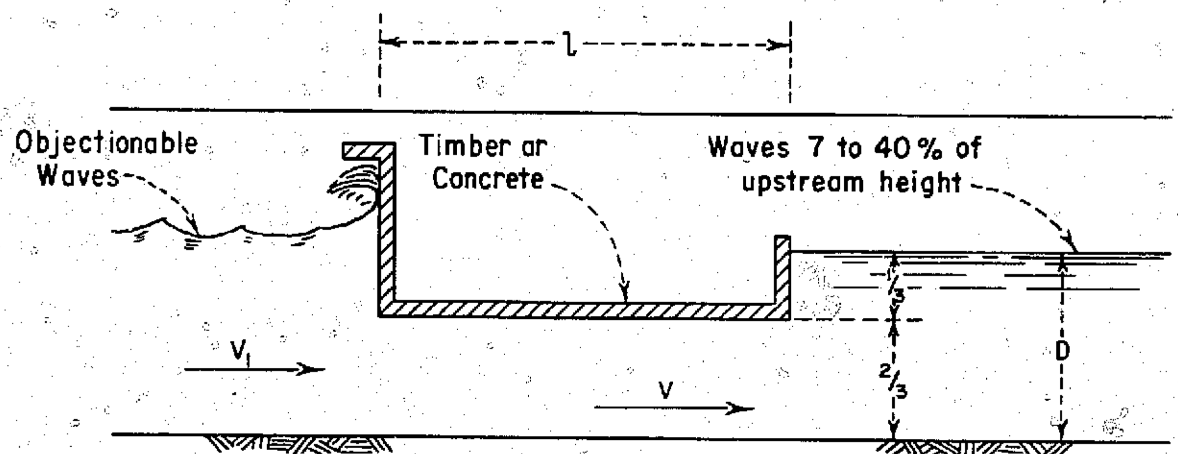


Figure 9

ROUGH WATER SURFACE

A rough water surface, other than wind generated waves, can usually be eliminated by reducing turbulence or improving the distribution of the approach flow. A rough water surface can cause errors in discharge measurements when it is necessary to (1) read a staff gage to determine head, or (2) determine the cross-sectional area of the flow. A stilling well will help to reduce errors in head measurement but every attempt should be made to reduce the water surface disturbances as much as possible before relying on the well.

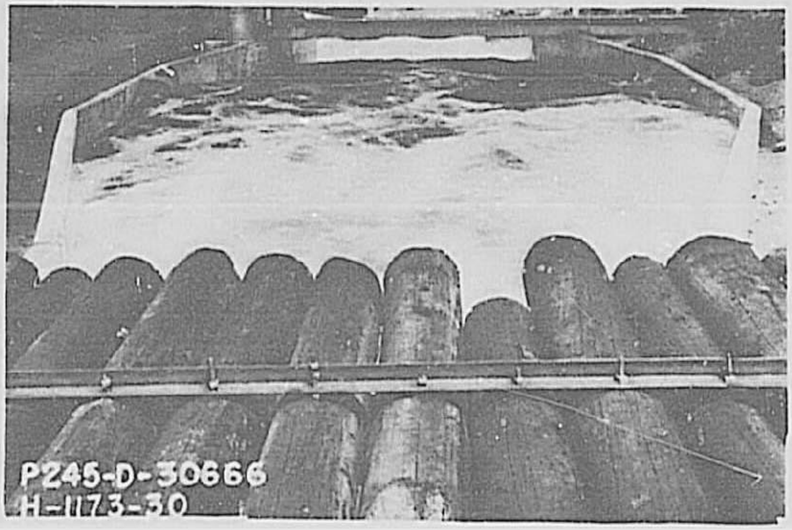
Errors of 10 to 20 percent are not uncommon if a choppy water surface makes it impossible to determine the head accurately. It is sometimes necessary to resort to specially constructed wave damping devices to obtain a smooth water surface. Figure 10 shows a schematic of an underpass type of wave suppressor successfully used in both large and small channels.



LENGTH l	PERCENT WAVE REDUCTION
1D TO 1.5D	60 TO 75
2D TO 2.5D	80 TO 88
3.5D TO 4.0D	90 TO 93

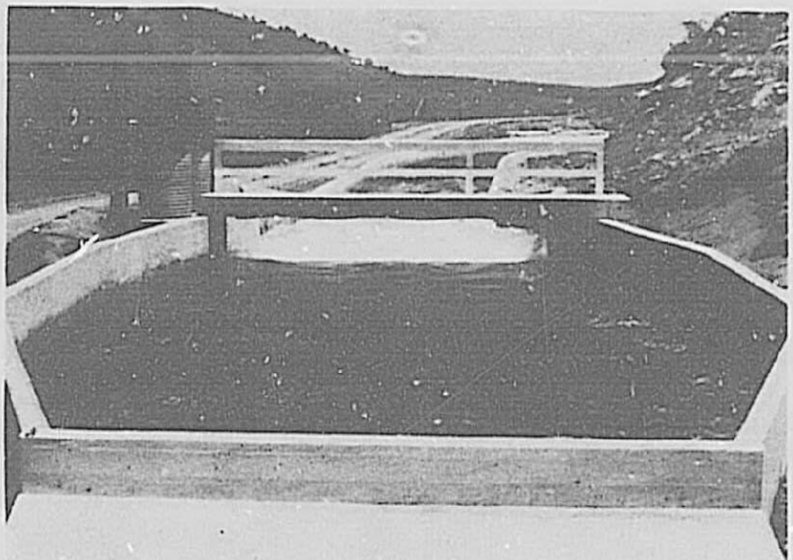
UNDERPASS WAVE SUPPRESSOR SECTION

Figure 10



Turbulence and waves in a Parshall flume produced by an outlet works stilling basin made accurate discharge determination impossible. Log raft in foreground, used in futile attempt to quiet the flow, is inoperative

Figure 11



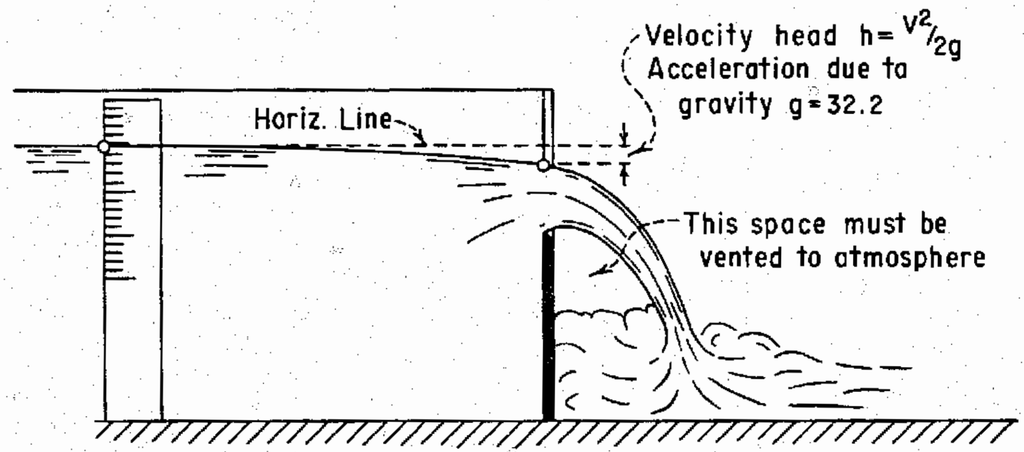
Underpass-type wave suppressor significantly reduces turbulence and waves in Parshall flume, making accurate discharge determination a routine matter. P245-D-30663

Figure 12

The channel may be either rectangular or trapezoidal in cross section. Waves may be reduced as much as 93 percent by constructing the suppressor four times as long as the flow is deep. A slight backwater effect is produced by the suppressor for the most effective vertical placement. The suppressor may be supported on piers, can be constructed of wood or concrete, and need not be watertight. The design of several other types of suppressors, along with sample problems, is covered in Engineering Monograph No. 25, available through the Chief Engineer's Office, Denver, Colorado. Figures 11 and 12 (before and after) show the effectiveness of an underpass wave suppressor at a Parshall flume measuring station.

VELOCITY OF APPROACH

It can be observed that as flow approaches a weir the water surface becomes lower on a gradually increasing curve, Figure 13.



SECTION ON LONGITUDINAL CENTERLINE

Figure 13

At the weir blade, the water surface is considerably lower than say, 5 feet upstream. The difference in elevation between the two circled points on the surface of the approach flow is called the velocity head and represents the potential required to produce the increase in velocity between the points. The relationship between head (h) and velocity (V) is expressed as

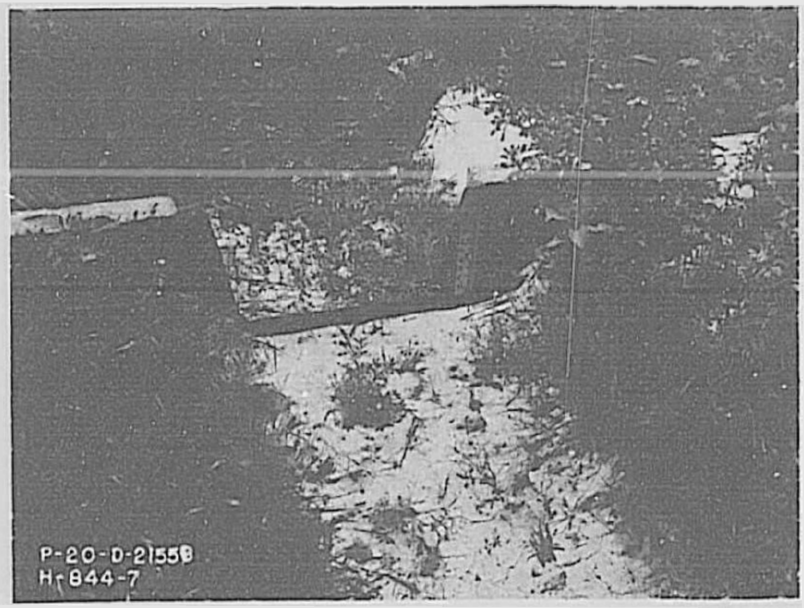
$$h = \frac{V^2}{2g}$$

g is the acceleration due to gravity, 32.2 (feet per second per second).

A drop in water surface of 0.1 foot is not uncommon just upstream from a weir and (from the equation above) represents an increase in velocity of 0.8 foot per second. If the head on the weir is measured too close to the weir, the head measurement can be 0.1 foot too small. For a weir 6 feet long and discharging 7 cubic feet per second, the corresponding error in discharge would be about 35 percent, based on an indicated or reported discharge of 5.1 second-feet.

Standard weir tables are based on the measured head on the weir (velocity head negligible) and do not compensate for excessive velocity head. Any increase in velocity above standard conditions, therefore, will result in measuring less than the true head on the weir and more water will be delivered than is measured.

Causes of excessive velocity head include (1) too shallow a pool upstream from the weir, (2) deposits in the upstream pool, Figure 14, and (3) poor lateral velocity distribution upstream from the weir, Figure 8.



Sediment deposits have reduced the depth of the weir pool sufficiently to increase the velocity of approach to well above the desirable level. The head gage should not be located close to the weir blade. The weeds should be removed and the "edge" of the weir should be sharp. Discharges over this weir will be larger than indicated in "standard" tables.

Figure 14

POOR FLOW PATTERNS

It is often found that the poor flow distribution which exists upstream from a measuring device cannot be resolved on the basis of any one of the above-discussed causes. The best solution then is to assume that several or more basic causes have together caused the difficulty. Starting with the easy factors, work through the list, improving each probable cause of poor flow patterns until the desired flow conditions are obtained.

Operating or nonoperating turnouts located just upstream from a measuring device may cause poor approach conditions as may bridge piers, channel curves, or a skewed measuring section. Relocating the measuring device may be the only remedy in these cases.

Submerged weeds or debris can cause excessive turbulence or local high-velocity currents. Eddies adjacent to the shoreline can cause the flow approaching a weir to contract into a narrow band. Sediment bars deposited from inflow or from sloughing banks can also produce undesirable flow conditions. More drastic remedial measures include deepening the approach area, widening the approach channel to make it symmetrical, or introducing baffles or other devices to spread the incoming flow over the entire width of the approach. Surface waves are usually very difficult to reduce or eliminate by ordinary procedures. These may require special treatment, as discussed under "Rough Water Surface."

EXIT FLOW CONDITIONS

Exit flow conditions can cause as much flow measurement error as some of the approach flow problems. However, in practice, these conditions are seldom encountered. In general, it is sufficient to be sure that backwater does not or tend to submerge a device designed for free flow. Occasionally, a Parshall flume is set too low and backwater submerge the throat excessively at high discharges. Extremely large errors in discharge measurement can be introduced in this manner. The only remedy is to raise the flume, unless some local obstruction downstream can be removed to reduce the backwater. Weirs should discharge freely rather than submerged, although a slight submergence (the backwater may rise above the crest up to 10 percent of the head) reduces the discharge a negligible amount (less than 1 percent). Whenever a weir operates at near submergence the operation should be checked. Submergence may not affect the discharge as much as the possible lack of nappe ventilation as a result of the rising backwater.

The underside of weir nappes should be ventilated sufficiently to provide near atmospheric pressure beneath the nappe; between the under nappe surface and the downstream face of the weir, Figure 13.

If the nappe clings to the downstream side of the weir (does not spring clear) the weir may discharge 25 percent more water than the head reading indicates. An easy test for sufficient ventilation is to part the nappe downstream from the blade for a moment with the hand or a shovel, to allow a full supply of air to enter beneath the nappe. After removing the hand or shovel the nappe should not gradually become depressed (over a period of several or more minutes) toward the weir blade. If the upper nappe profile remains the same as it was while fully ventilated, the weir has sufficient ventilation.

When the head on a straight weir is about an inch or less, the weir may not give reliable discharge values unless the weir has been calibrated under exactly similar flow conditions. On V-notch weirs, reliable results may not be obtainable for heads of 2 inches or even 3 inches.

Gates calibrated for free discharge at partial openings should not be submerged nor should eddies interfere with the jet of water issuing from the gate. Gaging stations should be kept free of deposited sediment bars or other obstructions to prevent backflow or eddies from interfering with the uniform flow conditions which should exist in the cross section being measured.

EQUIPMENT CHARACTERISTICS REDUCING ACCURACY OF MEASUREMENT

Measuring devices themselves may be at fault in producing measurement errors rather than the flow conditions discussed in the previous section. The faults may be divided into two types--those caused by normal wear and tear, and those resulting from poor installation.

WEATHERED AND WORN EQUIPMENT

An unwelcome but fairly common sight on older irrigation systems are weir blades which were once smooth and sharp, in a sad state of disrepair. Edges are dull and dented; the blade is pitted with large rust tubercles--weir plates are discontinuous with the bulkheads and are not vertical. Weir blades have sagged and are no longer level. Staff gages are worn and difficult to read. Stilling well intakes are buried in sediment or partly blocked by weeds or debris. Parshall flumes are frost heaved and out of level. Meter gates are partly clogged with sand or debris and the gate leaves are cracked and warped.

These and other forms of deterioration are often the causes of serious errors in discharge measurements. This type of deficiency is difficult to detect because normal wear and tear may occur for years before it is apparent to a person who sees the equipment frequently. On the other hand, it is readily apparent to an observer viewing the installation for the first time.

It is imperative, therefore, that the person responsible for the measuring devices inspect them with a critical eye. His attitude should be--I am looking for trouble--not, I will excuse the little things because they are no worse today than they were yesterday.

Measuring devices which are rundown are no longer a standard measuring device, and indicated discharges may be considerably in error. To be certain of the true discharge, they should be rehabilitated and/or calibrated.

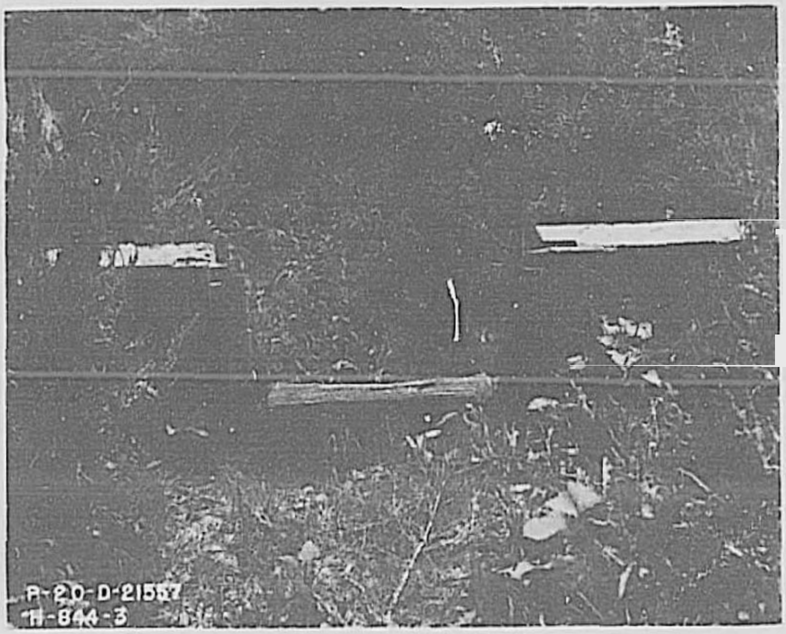
Repairing or refurbishing a rundown measuring device is sometimes a difficult or impossible task. Fixing the little things as they occur will prevent, in many cases, replacing the entire device at great cost at some later date. Regular and preventive maintenance will extend the useful life of measuring devices.

POOR WORKMANSHIP

Contrasting with the measurement devices which were once accurate and dependable and have deteriorated, are those which, because of poor workmanship, were never a standard device. These include devices which are installed out of level or out of plumb, those which are skewed or out of alinement, those which have leaking bulkheads with flow passing beneath or around them, and those which have been set too low or too high for the existing flow conditions. Inaccurate weir blade lengths or Parshall flume throat widths, insufficient or nonexistent weir nappe ventilation, or incorrect zero setting of the head or staff gage can also be the cause of measuring errors.

A transverse slope on a weir blade can result in errors, particularly if the gage zero is referenced to either end. The error can be minimized by determining the discharge based on the head at each end and using the average discharge. Errors in setting the gage zero are the same as misreading the head by the same amount. At low heads a relatively small zero setting error can result in errors up to 50 percent of the discharge or more. A head determination error of only 0.01 foot can cause a discharge error of from 5 percent on a 90° V-notch weir, to over 8 percent on a 48-inch Cipoletti weir (for a head of 0.20 foot). The same head error on 6- and 12-inch Parshall flumes can result in 12 and 6 percent errors, respectively, for low heads.

Weir blades which are not plumb or are skewed will show flow measurement inaccuracies of measurable magnitude if the weir is out of line by more than a few degrees. Rusted or pitted weir blades or those having projecting bolts or offsets on the upstream side can cause errors of 2 per cent or more depending on the severity of the roughness. Any form of roughness will cause the weir to discharge more water than indicated. Rounding of the sharp edge of a weir or reversing the face of the blade also tends to increase the discharge. On older wood crests a well-rounded edge can cause 15 to 25 percent or more increase in discharge, Figure 15.



The well-rounded edge on this once sharp-crested weir will increase the discharge well above "standard." The weeds are also undesirable as is the weir gage which projects into the flow area.

Figure 15

Pressure readings are needed to determine discharges through certain types of meters. Piezometers, or pressure taps as they are sometimes called, must be regarded with suspicion when considering accuracy of flow measurements.

Piezometers must be installed with care and with a knowledge of how they perform, otherwise the pressure values they indicate can be in error. For example as shown in Figure 16, the three piezometers will indicate different pressure readings (water levels) because of the manner in which flow passes the piezometer opening. Unless the piezometer is vertical as in Y, the water elevation will be drawn down as in X, or increased as in Z. Rough edges or surfaces in the vicinity of the piezometer can also result in erroneous indications in that they deflect the water into or away from the piezometer opening. The higher the pipe velocity, the greater the error will be.

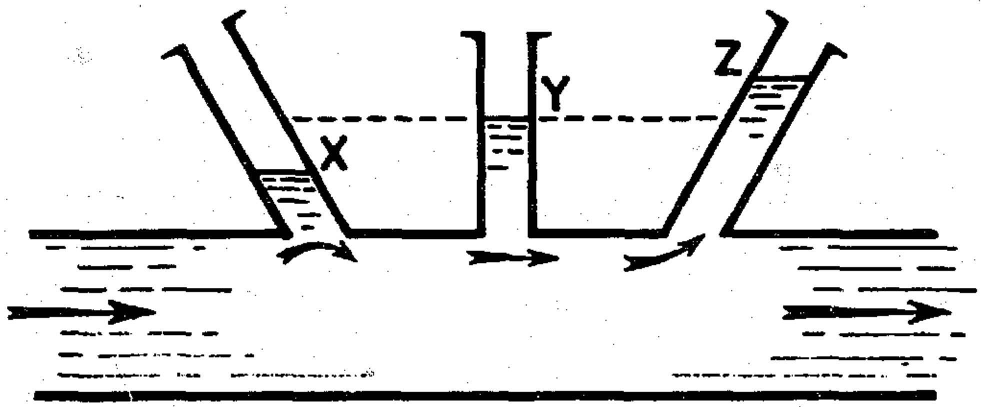


Figure 16

Note: Piezometer openings above are shown larger than they should be constructed in practice. Always use the smallest diameter opening consistent with the possibility of clogging by foreign material.

The effect of a few deficiencies often found in measuring devices has been given to illustrate the degree of error to be expected in making ordinary measurements under ordinary conditions. Other effects have not or cannot be stated as percent error without an exact definition of the degree of fault or deterioration. The examples given should be sufficient, however, to emphasize the importance of careful and exact installation practices, as well as regular and prompt repair or rehabilitation of the devices after they have been installed.

MEASURING TECHNIQUES REDUCING ACCURACY OF MEASUREMENT

It is possible to obtain inaccurate discharge measurements from regularly maintained equipment properly installed in an ideal location, if poor measuring techniques are used by the operator. Measurement of head is very important and some of the techniques now in use are not compatible with the relationships between head and discharge known to exist.

The frequency of head measurement is also important and may be the cause of inaccurate water measurement. These and other related miscellaneous techniques are discussed in the next paragraphs.

FAULTY HEAD MEASUREMENT

Measurement of the head on a weir seems to be a simple matter but can be difficult under all but ideal conditions. The head is the height of water above the blade edge (or crotch of a V-notch) measured at a point where the velocity head (or velocity of approach) is a negligible value, Figure 9. In practice this means a point located four to six times the head upstream from the center of the weir blade. If the head is measured farther upstream, the head necessary to produce flow in the approach channel (water surface slope) may be inadvertently included to give a larger head measurement. If the head is measured closer to the weir blade, some drawdown (caused by increased velocity near the weir) may occur and less than the true head may be measured. If the head is measured at the side of the approach channel, more or less than the true head may be measured depending on the geometry of the approach pool, Figures 9 and 13.

The practice of placing staff gages on weir bulkheads or on bankside structures should be investigated in each case to be sure that a true head reading can be obtained. Placing a rule or a Clausen-Pierce gage on the weir blade also gives an erroneous reading. The taking of head measurements when debris or sediment has a visible effect on the flow pattern can also result in faulty head determination, Figure 17. Measuring head, when the measuring device has obviously been damaged or altered, is also to be avoided.

Figure 18 shows a weir performing properly for the discharge shown. At larger discharges the unsymmetrical approach pool may produce undesirable conditions.

The principles described above also apply to head measurements on Parshall flumes, metergates or any other device dependent on a head measurement for discharge determination.

Improper gage location, or an error in head measurement in a Parshall flume can result in very large discharge errors. Throat width measurements (and weir lengths) can also produce errors although these errors are usually small because of the relative ease of making accurate length measurements. (Operators should measure lengths in the field and not rely on values stated or shown on drawings.) Readings obtained from stilling wells, whether they are visual or recorded, should be questioned unless the operator is certain that the well intake is not partially or fully clogged. Data from an overactive stilling well can also be misleading, particularly if long period surges are occurring in the head pool. In fact, all head determinations should be checked to be sure that the instantaneous reading is not part of a long period surge. Sufficient readings, say 10, should be taken at regular time intervals, say 15 seconds, and averaged to obtain the average head. More readings may be required if it is apparent the pool is continuing to rise or fall. If this is too time consuming the cause of the instability should be removed.

Weeds protruding through the opening and sediment in the approach pool will result in inaccurate discharge determinations.

Figure 17

Cipoletti weir operating with good flow conditions in the approach pool. Flow is well distributed across wide pool and shows no evidence of excessive turbulence. Accurate or "standard" discharges can be expecte under these conditions.

Figure 18

Readings from gages or staffs which may have slipped or heaved should be avoided. Periodic rough checks can sometimes be made with a carpenter's level or square from a reference point on another structure. A still water level at weir crest height is a valuable check on the staff gage zero.

In short, it is desirable that each operator understand the measurement he is trying to make, and then critically examine each operation to be sure he is measuring what he intends to measure. He should try to find fault with every step in making a head measurement and try to improve his technique wherever possible.

INFREQUENT MEASUREMENT

When a head or velocity measurement is made to determine discharge, it can be concluded that the measured discharge occurred only at the moment of the measurement. It cannot be concluded that the discharge was the same even 5 minutes later or 5 minutes earlier. Therefore, water deliveries can be accurate only if enough measurements are made to establish the fact that the discharge did or did not vary over the period of time that water was being delivered.

In many systems, measurements are made only once a day, or only when some mechanical change in supply or delivery has been made. Problems introduced by falling head, rising backwater, gate creep or hunting are often ignored when computing a water delivery. The problem is not a simple one, at times, and there are many factors to consider in determining the number of readings to be made per day or other unit of time. If the discharge in the supply system is increasing or decreasing, it will be necessary to take more than a single reading. If the rate of rise is uniform the average of two readings, morning and night, would be better than one. If the rate of change is erratic, frequent readings may be necessary. If a great many readings are known to be necessary, a recording device may be justifiable.

Sometimes when the discharge in the supply system remains constant, the water level or velocity reading change because of a change in control, or because checks have been placed in operation. Temporary changes in discharge in the main supply system may occur for example because water, in effect, is being placed in storage as a result of the rising water level. Conversely, the discharge may temporarily increase in parts of the system, if the operating level is being lowered. The changing water level may make it necessary to take more frequent head readings.

Here again, the operator should try to visualize the effect of any change in discharge in the supply system, upstream or downstream from a measuring device, and attempt to get more than enough readings to accurately compute the quantity of water delivered.

USE OF WRONG MEASURING DEVICE

Every water-measuring device has limitations of one kind or another and it is impossible to choose one device that can be used in all locations under all possible conditions. It is to be expected, therefore, that for a given set of conditions there may be several devices which would be suitable, but none could be considered entirely satisfactory. If flow conditions change or are changed by modified operations, an original device, which was marginal in suitability, may be found to be totally inadequate. It is possible, too, that the wrong device was selected in the first place. Whatever the reason, there are instances where accurate measurements are being attempted using a device which cannot, even with the greatest care, give the desired results. The operator should call attention to such a situation and attempt to have remedial measures taken.

For example, a weir cannot be expected to be accurate if the head is appreciably less than 0.2 foot, or greater than about one-third of the weir blade length. Large measurement errors can be expected (departure from standard), if these limits are exceeded appreciably. If a weir is submerged appreciably by backwater, large errors may be introduced depending on other factors. In view of uncertainties which cannot be explained satisfactorily, submerged weirs should be avoided wherever possible. Parshall flumes should not be operated at more than the critical degree of submergence (80 percent); in fact, they should not be submerged at all, unless provisions have been made in the flume for a downstream head measuring well, and the method of computing submerged discharges from the published tables is thoroughly understood. This is explained in detail in the section on "Parshall flumes."

Propeller meter devices should not be permanently installed where weeds, moving debris, or sediment are apt to foul the meter or grind the bearings. Submerged devices, such as metergates, should not be used where a moving bedload can partly block the openings.

In short, it is necessary to analyze the flow conditions to be encountered at a particular site, and only then, select the measuring device that can best cope with the unusual condition to be encountered.

ELABORATIONS ON THE BASIC WATER-MEASURING DEVICES AND TECHNIQUES

ORIFICES

Submerged Orifice

For a free-flow orifice, Figure 5, the discharge equation was shown to be approximately equal to $Q = 4.89\ A\sqrt{H}$.

If the head H on the orifice was 4 feet and the area of the orifice was 2 square feet, the discharge would be

$$Q = 4.89 \times 2 \times \sqrt{4} = 19.6 \text{ cfs}$$

If an orifice is discharging into a ditch, as shown in Figure 19, there may be some backwater to prevent free-flow conditions.

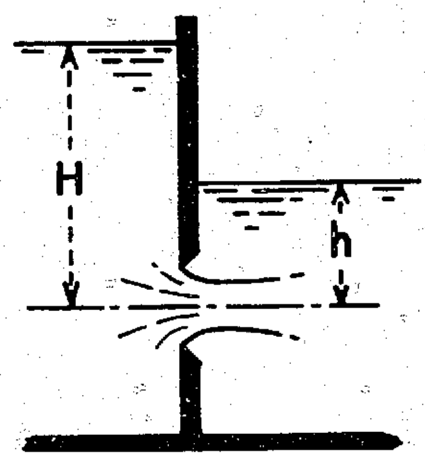


Figure 19

Since the head H is pushing water through the orifice and the head h is attempting to hold it back, h must be subtracted from H before using a head value in the equation

$$Q = CA\sqrt{2g(H - h)}$$

In the preceding problem, if h was 1 foot, the discharge would be

$$Q = 4.89 \times 2 \times \sqrt{3} = 17.1$$

or a reduction of 2.5 cfs from the 19.6 cfs computed without submergence.

Orifice in Pipeline

If an orifice is placed in a pipeline as shown in Figure 20, there is almost certain to be a backwater effect, and it will be necessary to measure both the upstream and downstream heads. Since there is no free water surface in a full pipe, piezometers or pressure taps must be used to obtain the necessary data.

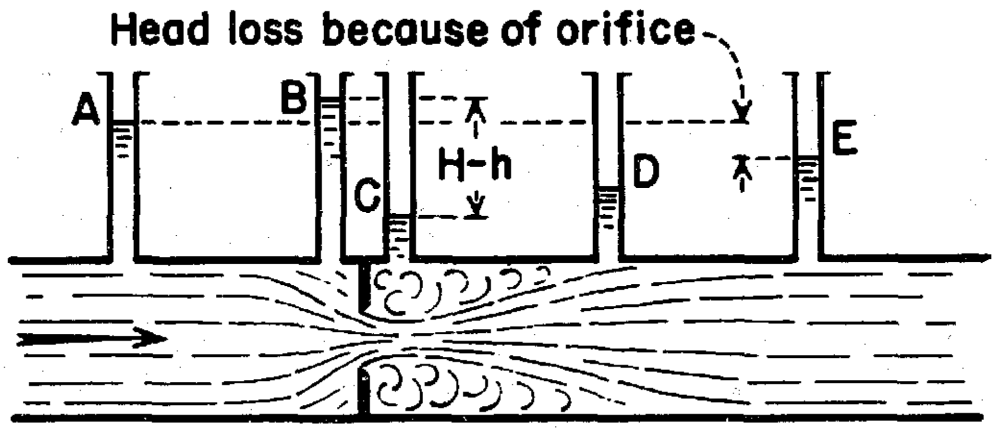


Figure 20

Piezometers are no more than small diameter standpipes in which water rises sufficiently to balance the pressure inside the pipe. The accuracy aspects of piezometers have been discussed, Figure 16.

If piezometers were placed in a pipe as shown in Figure 20, the differential pressure H - h (between B and C) would be the head producing flow to be used in the orifice discharge equation. The head at A would be lower than at B because some of the total head would have been used in producing the velocity in the pipe. At B the velocity of flow would be nearly zero, and so the true head would be indicated by the piezometer. The pressure at C would be very low because of the high velocity. At D some head recovery would occur because of the reduction in velocity caused by the spreading of the orifice jet.

At E normal pipe flow has been reestablished and the loss A - E represents the head lost because of the disturbances in the flow caused by the orifice. Energy in the flow was converted to heat as a result of turbulence in the flow and extra friction losses at the orifice plate and pipe boundaries.

Orifice discharges may be calculated with reasonable accuracy if all the factors affecting the flow are evaluated and the coefficient "C" is adjusted accordingly. For example, the graph in Figure 21 shows the variation in C to be expected for various combinations of pipe size and orifice diameter.

The orifice coefficient is seen to be 0.61 (in the solid line curve) when the orifice is 0.2 of the pipe diameter or less and increases to 1.0 when the orifice is 0.9 of the pipe diameter. It would therefore appear that large orifices would be preferable to small. This is not necessarily so, however, because large orifices give such a small differential that the error in reading the head is a large part of the differential. Also the head tends to fluctuate severely so that at times it may appear that there is a reverse differential.

Thus, orifice installations should provide sufficicient head (and/or differential) to make head reading errors negligible in terms of the differential head. In fact, it has been shown that the head on a freely discharging orifice should be at least twice the diameter of the orifice. For lower heads, the coefficient falls off rapidly and may be as low as 0.2.

Rounding of the sharp edge of a circular orifice may be the cause for considerable error in determining discharges. A 1-inch-diameter circular orifice rounded to a radius of 0.01 inch will discharge 3 percent more water than a sharp edge. This is because the contraction is not as great with a rounded edge as with a sharp edge. (Note that this is a very slight degree of rounding.)

In general, the percent increase in C (or discharge) due to rounding, equals three times the percent that the radius of rounding is of the diameter of the orifice.

The dotted line curve shows coefficients (for $H - h = 3$ feet) obtained from a careful volumetric calibration of five orifices 1-1/2-inch, 2-3/8-inch, 3-7/8-inch, 6-inch, and 8-1/2-inch used in a 12-inch pipeline as a laboratory metering system. The departure of the coefficient from the generally accepted solid line curve is considerable.

The broken line curve shows coefficients for five orifices 1-1/4-, 1-3/4-, 2-3/8-, 3-3/8-, and 4-3/8-inch used in an 8-inch pipeline having an 8- to 5-1/2-inch reducer placed upstream from the orifice. The 8-inch pipe size was used to compute the ratio plotted as the abscissa in the sketch. Here, again, is a departure from the generally accepted coefficient curve and if a coefficient had been assumed from the solid line curve, serious discharge measuring errors of perhaps as much as 15 percent could have occurred.

Because of the many factors which affect orifice discharges, it is usually desirable to calibrate an installation by volumetric measurements, current meter, Pitot tube, or other primary means. This may not be possible, and it may then be necessary to improvise a calibration. Another objection to the use of orifice meters is that the head loss caused by the meter may be excessive. Losses may run as high as one or more velocity heads. One velocity head is equal to the head required to produce the velocity in the pipe upstream from the orifice determined from Equation 3.

Orifice meters are not generally available from commercial supply houses, and it is not ordinarily possible to buy a meter complete with piezometers and head or differential head gages.

When a submerged orifice is used in an open ditch, the area of the orifice should be no more than about one-sixth the ditch cross-sectional flow area to minimize velocity of approach effects. This is roughly equivalent to using an orifice-to-pipe-size ratio, Figure 21, of about 0.25; coefficient 0.62. A high velocity of approach means that some of the head (which is to be measured) has been converted to velocity and cannot be measured directly. To account for this head the velocity must be determined, converted to head, and added to the measured head.

The height of the rectangular orifice should be considerably less than the width to minimize the effect of variable head on the orifice coefficient. The submerged orifice equation (6) may be used along with a coefficient of 0.61-0.62.

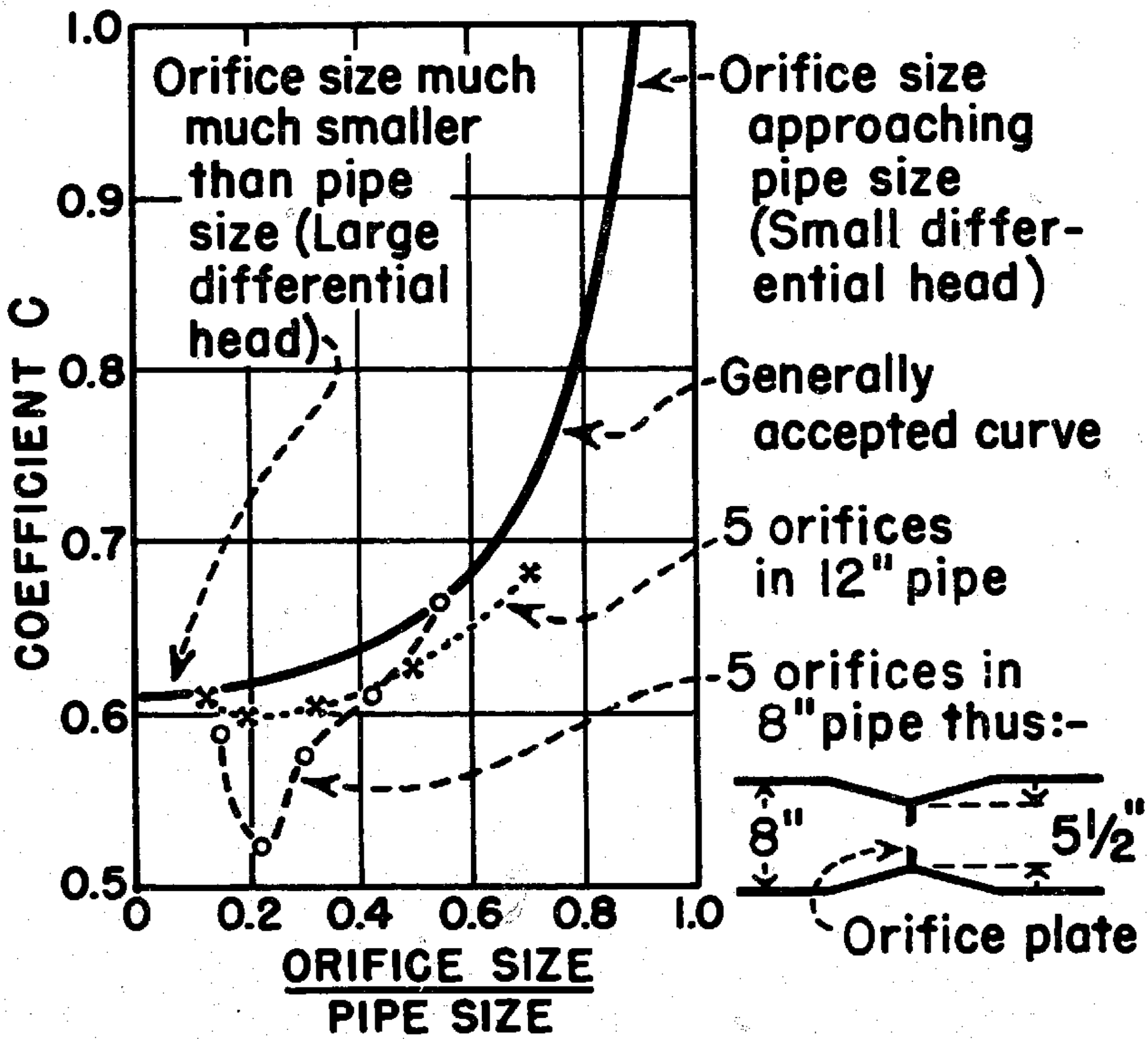
If the velocity of approach is excessive (head has been converted to velocity and cannot be read on the staff gage), the velocity head (use the average velocity in the ditch upstream from the orifice and convert to H by $\frac{V^2}{2g} = H$) must be added to the measured head.

If the orifice is suppressed (hindered by floor, walls, or other) from a normal approach flow pattern, use the equation

$$Q = 0.61\ (1 + 0.15r)\ A\sqrt{2gH} \qquad \ldots\ldots (7)$$

where r is the ratio, length of suppressed portion of perimeter of orifice divided by total perimeter.

Discharges for standard rectangular orifices are given in Table 29 and correct coefficients for suppressed orifices in Table 30 of the Water Measurement Manual. Other information on submerged orifices is given in Chapter IV of the Manual.



ORIFICE COEFFICIENTS

Figure 21

VENTURI METERS AND FLOW TUBES

The Venturi meter is basically a streamlined orifice meter and was devised to reduce the head loss produced by the orifice meter. The meter consists primarily of a constriction in a pipe with a curved approach to the constriction and a gradual expansion to the pipe diameter as shown in Figure 22.

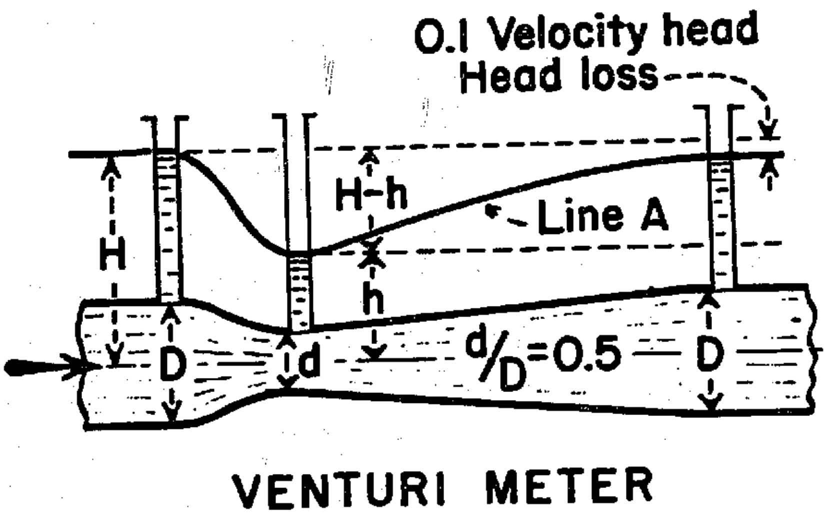
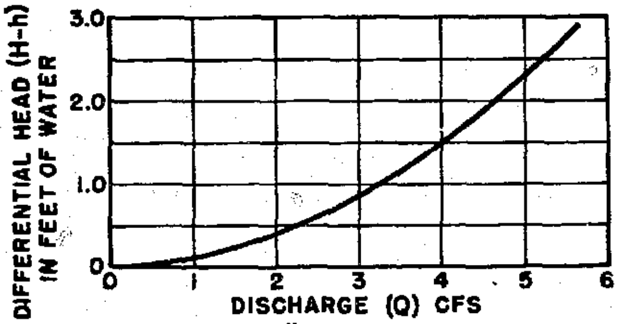


Figure 22

A typical Venturi meter is shown in the sketch for a constriction of one-half the pipe meter (d/D = 0.5). The piezometric heads are shown as H and h and the differential used to determine meter discharges as H - h. The head loss is shown at the downstream piezometer as being about 0.1 velocity head, considerably less than for an orifice of the same size. Line A (the hydraulic grade line) shows the elevation of the water surface which would be indicated if a large number of piezometers were installed in the meter to indicate the variation in pressure from point to point.

Although tables are usually used, the curve in Figure 23 shows a typical rating curve for a commercial 8-inch Venturi meter (approximately 4.5-inch throat and 8-inch pipe).



LABORATORY 8" VENTURI METER
(N.E. BANK)

Figure 23

Of particular interest and concern is the shape of the lower portion of the rating curve. A differential head of only 0.1 produces a discharge of 1 cfs. It would be difficult to measure 0.1 foot accurately because of the usual fluctuations; and, consequently, it would be difficult to say whether the discharge was 0.7, 1.0, or 1.2 cfs. The meter should not be used, therefore, for discharges less than 2.5 cfs where the differential head is about half a foot. For discharges of 3 to 5 cfs, the 8-inch meter could be expected to be extremely accurate. If a discharge of 1 cfs must be measured accurately, a smaller Venturi meter should be used. Because of the nature of the meter, the differential varies as the square of the rate of flow. This means that when the meter is discharging 50 percent of capacity the differential is 25 percent of maximum; 10 percent of capacity shows only 1 percent of the maximum differential; 5 percent of capacity shows only one-fourth percent of maximum differential. An orifice meter has the same characteristic rating curve, and the above statements apply to orifice meters as well.

Venturi meters are available commercially in a range of sizes and can be purchased with an accompanying set of discharge tables or curves. Venturi meters must always be calibrated because it is impossible to calculate discharges accurately. Calibrated meters are usually accurate to within 2 percent.

Venturi meters are usually machined castings and are relatively expensive, although cheaper cast concrete has been successfully used in some cases. Some success has been achieved in constructing meters from standard pipe fittings which can be screwed or bolted together. Two standard pipe reducers with a standard gate valve between them makes a satisfactory measuring device which has been found to be accurate to plus or minus percent. Some of the early work on this subject is contained in a Master of Science Thesis, 1942, "Hydraulic Characterists of Simplified

Venturi Meters," by R. A. Elder, Oregon State University, Corvallis, Oregon.

Venturi meters are usually of two basic types--the standard long form or the short form. The Dall tube is a commercial version of the short form tube, but is claimed by its manufacturers to have a low head loss. The long form tube usually has less head loss than the short form because more head is recovered in the long tapered expanding section downstream from the throat than in the more abrupt short section. The short form usually costs less and requires less space, however. Flow tubes constructed of fiberglass and epoxy are available for installation between flanges in a pipeline. These require little space because the flow tube is placed, essentially, inside the pipe.

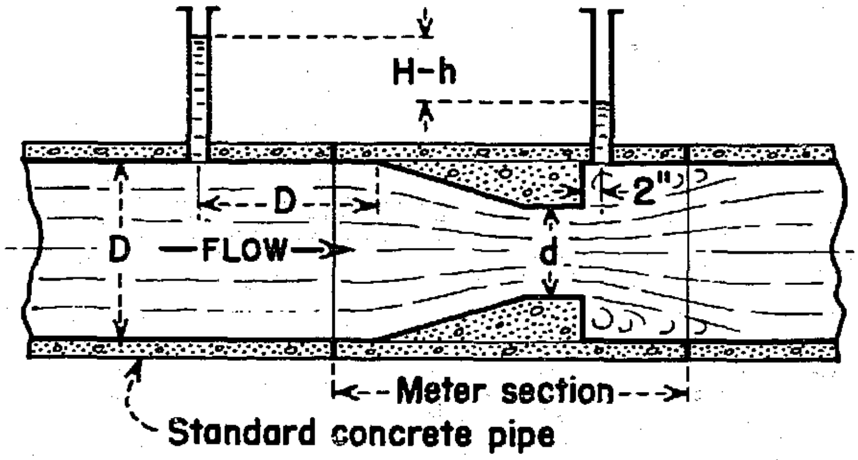
Head loss must be considered when selecting a meter because pump sizing may be affected and pumping costs may be a part of the daily operating cost. The head loss is governed chiefly by the length of the tube and the ratio of throat to inlet diameters, the loss being greater for short tubes and small throats. Attempts to reduce the head loss by increasing the throat diameter will result in smaller differential pressures for a given discharge. Too large a throat, therefore, may result in measurement inaccuracies.

Comparative head losses for several types of meters are given in column 2 of the table below, for a throat-to-inlet diameter ratio of 0.5.

Meter type	Head loss, feet	Yearly pumping cost, dollars
Flow tube	0.6	13
Long form Venturi	1.0	23
Short form Venturi	1.2	29
Orifice	6.3	140

The yearly cost of electricity for pumping 1 cfs against these heads is shown in column 3 (75 percent efficiency, power cost $0.02/kwh).

Concrete meters (Figure 24) have been constructed and used by the Fresno Irrigation District, Fresno, California. The Fresno meter consists of a length of standard concrete pipe into which has been formed a circular throat section to give a reduction in area so that the principle of the Venturi meter is applicable for the measurement of flow.



FRESNO IRRIGATION FLOWMETER

Figure 24

Meters are available in 8-, 10-, 12-, 14-, 16-, 18-, 20-, and 24-inch sizes (D). Accurate laboratory calibrations have been made for the 8-, 10-, 12-, and 18-inch sizes. Head loss versus discharge curves are also available for these sizes. The losses range from 0.2 foot for the 8-inch meter discharging 1 cfs, 0.4 foot for a 10-inch meter discharging 2 cfs, 0.7 foot for a 12-inch meter discharging 4 cfs, to 0.9 foot for an 18-inch meter discharging 10 cfs. A report, Restricted Hydraulic Laboratory Report No. Hyd-340, gives the results of extensive tests on these meters, and is available to Bureau employees from the Technical and Foreign Services Branch, Bureau of Reclamation, Denver Federal Center, Denver, Colorado.

VENTURI FLUMES

The familiar Parshall flume (Figure 12, Parshall flume section of this report) belongs to a large class of water-measuring devices known generally as Venturi flumes used in open channel installations. These devices depend upon contraction of the flow either by tapering the side walls of the flume, by changing the elevation of the flume bottom, or both. Parshall flumes incorporate a floor drop along with the converging of the side walls. Parshall flumes are discussed in a separate section in this report.

Venturi flumes are of two types, the "free-flow" type where a simple head reading is required to determine discharge and the "submerged" type which requires two head readings to account for the backwater depth effect and determine the discharge. The latter type is sometimes called a "critical depth flume" and/or a "standing wave flume." The former is sometimes called the "true Venturi" type. The Parshall flume is an example of a Venturi flume that is often used in either category or both.

When head must be conserved the flat-bottomed Venturi flumes are more desirable than flumes having vertical configurations in the floor. If the canal is trapezoidal, the flat-bottomed Venturi flume can also be made trapezoidal for convenience of construction of placement. But, like all measuring devices they should be either calibrated before use or be constructed exactly the same as an existing flume (standard device) that has been calibrated.

From studies made on Venturi flumes it has been found that for any given flume, each value of the discharge (Q) has a unique and corresponding head (H). The results of these studies indicate that the relation between discharge and head may be expressed in the general form:

$$Q = K H^{n}$$

where the coefficient (K) and the exponent (n) are predominantly dependent upon the geometry of the flume. When the values of (K) and (n) are determined from actual measurements, the device is said to be calibrated.

The Bureau of Reclamation has studied the flat-bottomed trapezoidal Venturi flume shown in Figure 25. This particular flume was studied for discharges ranging from 0.5 to 5 cfs; the discharge equation was found to be

$$Q = 3.48 H^{2.24}$$

Studies of other flumes of both larger and smaller sizes will be continued and will be directed toward standardizing the flumes in terms of geometry and in providing rating tables for general use.

Small flat-bottomed trapezoidal Venturi flumes were studied by A. R. Robinson and A. R. Chamberlain, "Trapezoidal Flumes for Open-Channel Flow Measurement," ASAE, Volume 3, No. 2, 1960. Their study presents the calibration test results on seven flumes with side slopes (θ) ranging from 30° to 60°, throat bottom width varying from 0 to 4 inches, and contraction angles (φ) varying between 8° to 22°. The discharge range covered by these flumes is from 0.02 to 2.0 cfs. If flumes of this type are to be built and used without field calibration the dimensions and limitations discussed in the article should be carefully followed.

Flat-bottomed Venturi flumes can be made of concrete, metal, or wood. However, the use of wood should be avoided wherever possible because the effect of swelling and warpage can be severe. Regardless of the material used for construction, the flumes should be sufficiently rigid to prevent bowing caused by earth pressure from backfilling.

For best results the flat-bottomed flume should be set flush with the bottom of the incoming canal. If possible, the cross sections of the canal and the start of the converging portion of flume should match. If matching is not possible, transitions to the flume can be made of concrete, metal, wood, or gravel large enough to resist movement with the flow.

The head measuring station should be located just upstream from the start of the convergence in the flume, Figure 25. If a stilling well and hook gage are used, the pressure tap or piezometer should be placed about 2 inches above the bottom of the flume to prevent sediment and other debris from plugging the lines to the stilling well. Staff gages may also be used to measure the head. To indicate the necessary accuracy for a head determination (in terms of discharge error) the following table may be used. This table is for the flume size shown in Figure 25.

Error in head reading feet	Discharge error range percent
0.005	1 to 2
0.010	2 to 6
0.020	4 to 10

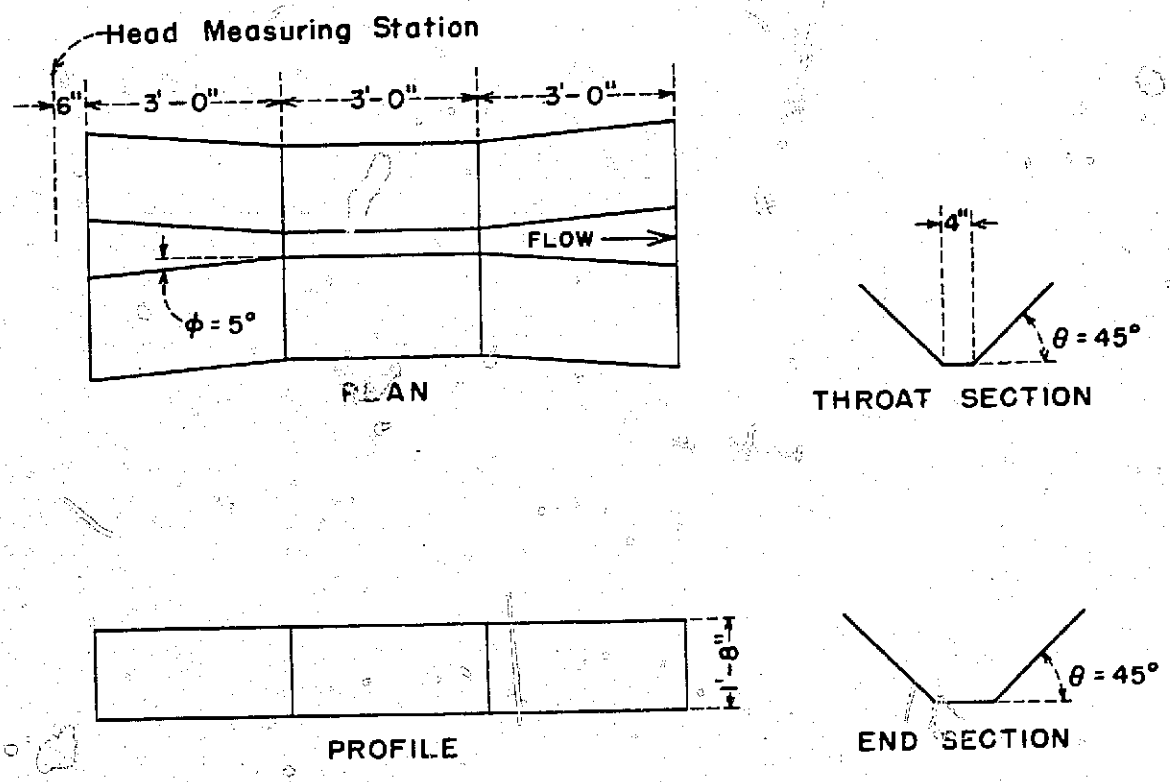


Figure 25

In an ordinary installation the velocity of approach to a Venturi flume would not have an effect on accuracy. Excessive flow velocity at the flume entrance can cause errors of up to 4 percent, however, and some care in installation and maintenance is required if the Venturi flume is to be considered an accurate measuring device.

Reasonable entrance velocities will result in no measurable discharge errors. If the water surface just upstream from the flume is smooth (shows no surface boils, waves, or high-velocity current concentrations), it may be concluded that the Venturi flume accuracy is not being affected by the approach flow.

Venturi flumes are calibrated in a channel having a horizontal bottom. Field installations should approximate this condition if accuracy is important.

METERGATES

A metergate is basically a modified submerged orifice arranged so that the orifice is adjustable in area, Figure 26.

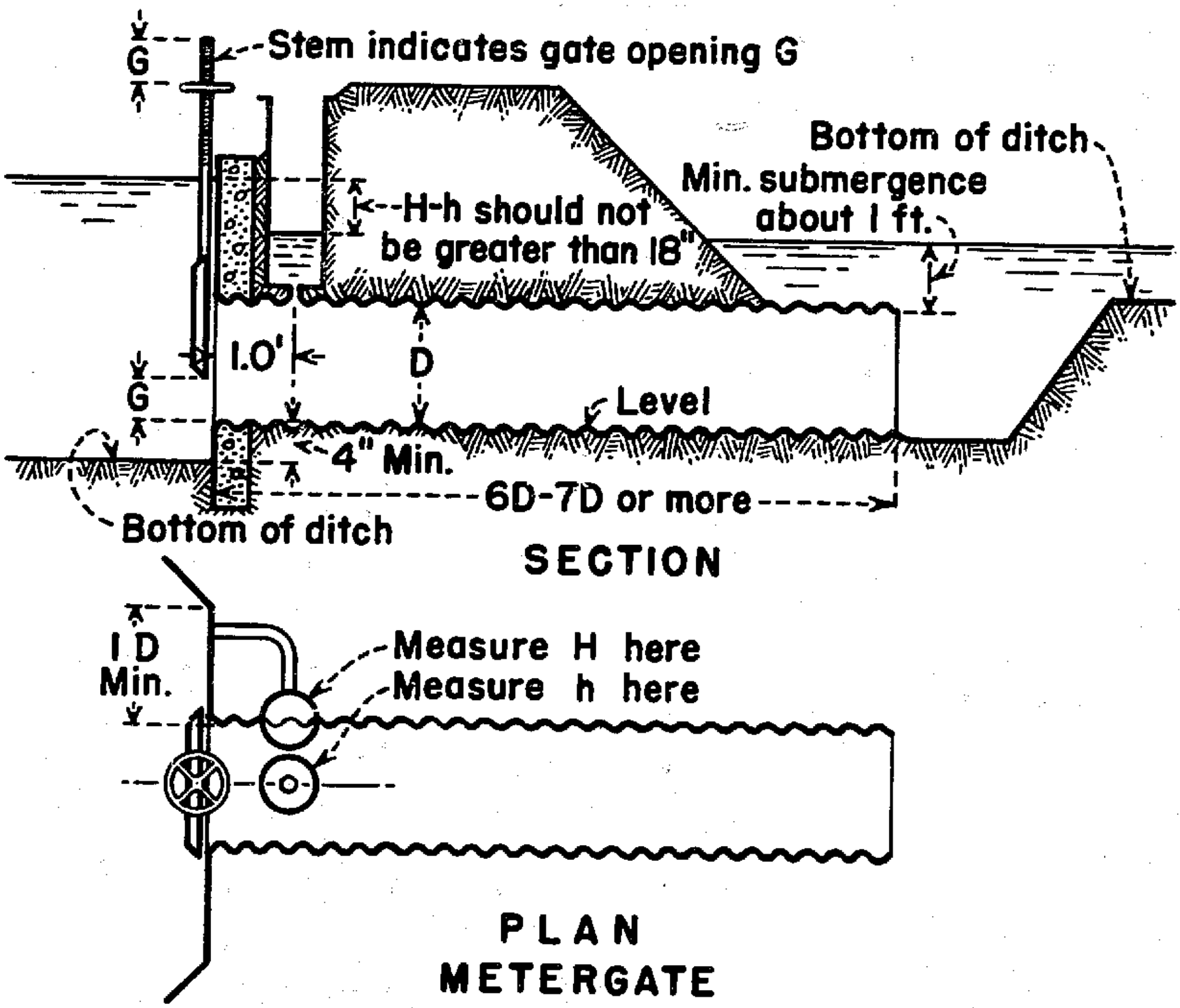


Figure 26

Although it should be possible to compute the discharge, this is rarely done because there are usually too many departures from standard definable conditions for which correction coefficients are not known. Metergates are usually purchased from a commercial supplier who supplies a discharge table. Ordinarily, the tables give a good accounting of the flow, but in some instances, errors of 13 percent or more have been found.

If a discharge error is suspected, the installation should be thoroughly checked to be sure that it complies with the essential conditions shown in the above sketch, particularly that there is no blockage of flow and that the outlet is sufficiently submerged to make the pipe flow full. The many factors affecting metergate performance and accuracy are described in detail in the following paragraphs. These suggestions apply particularly when the gate is operated at large openings (50 percent or more) and/or with small upstream submergence (1D or less).

SOURCES OF DISCHARGE INDICATION ERROR

Type of gate

The discharge table being used should be checked to be sure that it applies to the metergate in question. Tables for round bottom gates will not work with square bottom gates, or vice versa, except perhaps at the wide-open position. Be sure that the table being used is for the brand of gate, model number, or other identifying symbols.

Stilling well blockage

If there is no blockage of flow at the gate or in the pipe, make sure that the stilling wells are open. A bucket of water poured into the well should readily drain out or, if the gate is in operation, the water level in the well should rapidly return to the head indicated before the water was added. As a matter of general maintenance, it would be a good idea to flush the wells occasionally, push a probe through the piping, and flush again. Any difference in readings before and after cleaning might indicate the need for further flushing and cleaning. Staff gages or scales should also be checked to be sure they have been installed at the proper zero position and that they have not become displaced vertically.

Gate and gate opening indicator

Be certain that the gate opening indicator, whether it is the rising stem on the gate or some other device, has not become displaced to give a false gate opening indication. Check the installation of the gate on the end of the pipe. The gate must seal when closed. Too much clearance may allow an excess of water to flow between the gate or frame and the end of the pipe, changing the flow pattern and indicated head in the downstream stilling well.

Approach area

Weeds, trash, or sediment in the approach to the gate can change the pattern of flow sufficiently at the gate leaf to produce sizable discharge errors. The flow along the sidewalls (wingwalls) has more effect on discharge than the flow along the bottom. Be sure that flow can follow the sidewalls without interference. Large amounts of sediment deposited in the area just upstream from the gate can upset the normal flow patterns as can waterlogged trash, rocks, or other submerged material. The approach area should be cleaned and reshaped, if necessary, until no flow lines or velocity concentrations are visible on the water surface.

Submergence

The water level at the gate should be at least one pipe diameter (preferably two) above the crown of the pipe during operation (flow measurement). As previously shown for the orifice, considerable error results when the head is less than one diameter above the top of the pipe. The pipe outlet must also be sufficiently submerged to make the pipe run full. Usually, if the pipe length is standard, at least six or seven diameters (discussed later), the submergence need be only about 6 inches above the crown of the pipe. Unless the pipe runs full at the outlet, the downstream head measuring stilling well may not contain enough water to indicate the true differential pressure across the metergate, and serious discharge measuring errors can occur.

Small differential head

Large errors in discharge determination can be introduced if the differential head (difference in water surface elevation between the two stilling wells) is small. For example, in reading the two water surface elevations in the stilling wells, an error of 0.01 foot could be made in each reading, giving a differential of 0.10 foot instead of 0.08 foot. The difference in indicated discharges would be about 0.12 cfs for a discharge of 1.10 through an 18-inch metergate open 5 inches, an error of about 11 percent.

If the gate opening was reduced to 2 inches and the upstream pool could be allowed to rise to pass the same discharge, the differential head would be 0.40 foot and the same head reading error of 0.02 foot would indicate a change of only 0.03 cfs. The error in discharge determination would be reduced from about 11 percent to less than 3 percent.

If the pool level cannot be elevated as described and it is necessary to operate continually with small differential heads, it would be well to consider installation of a smaller gate. This would allow operation in the upper ranges of capacity where the differential head is larger. If a smaller gate cannot pass the required maximum flow, it might be necessary to use two small gates in place of one large one.

Aside from head reading errors, it is desirable to operate with larger differentials because (1) the flow is more stable and the water surface in the stilling wells does not surge as badly and (2) the higher velocity through the meter prevents a reduction in orifice coefficient (as discussed for the orifice meter).

Other methods of achieving a larger differential might include reducing the backwater level, if excessive, or reducing the pipe length, if it is considerably longer than six or seven diameters, to reduce the backwater effect; change the location of the downstream stilling well and recalibrate the meter (discussed later).

Location of stilling well intakes

Because the discharge is directly related to the difference in water levels in the two stilling wells, it is essential that the stilling well intakes (pressure taps or piezometers) be located exactly as they were when the meter was calibrated.

The upstream intake should be located in the headwall several inches (at least) from the gate frame, several inches (at least) from any change in headwall alinement in plan (see Figure 26), and at an elevation such that the intake will be covered at minimum operating level. The opening should be flush with the surface of the headwall and the piping arranged so that a cleaning probe may be pushed through for cleaning purposes. The pipe should slope continuously downward from well to headwall to prevent air locks in the system. If air is suspected in the piping, it may be flushed by pouring water into the well at a rapid rate to force the air out through the intake end, taking care not to entrain air in the pouring process.

The downstream piezometer (pressure tap) should be located on the centerline of the top of the pipe, exactly 1 foot downstream from the downstream face of the gate. The intake pipe must be flush with the inside surface of the pipe (grind off any projections beyond corrugated or smooth surface) and absolutely vertical (the effect of tilted piezometers is illustrated in Figure 16).

As shown in Figure 27, the rate of change in pressure is very rapid in the region of the downstream pressure tap and any displacement of the tap from the location used during calibration will result in large discharge determination errors.

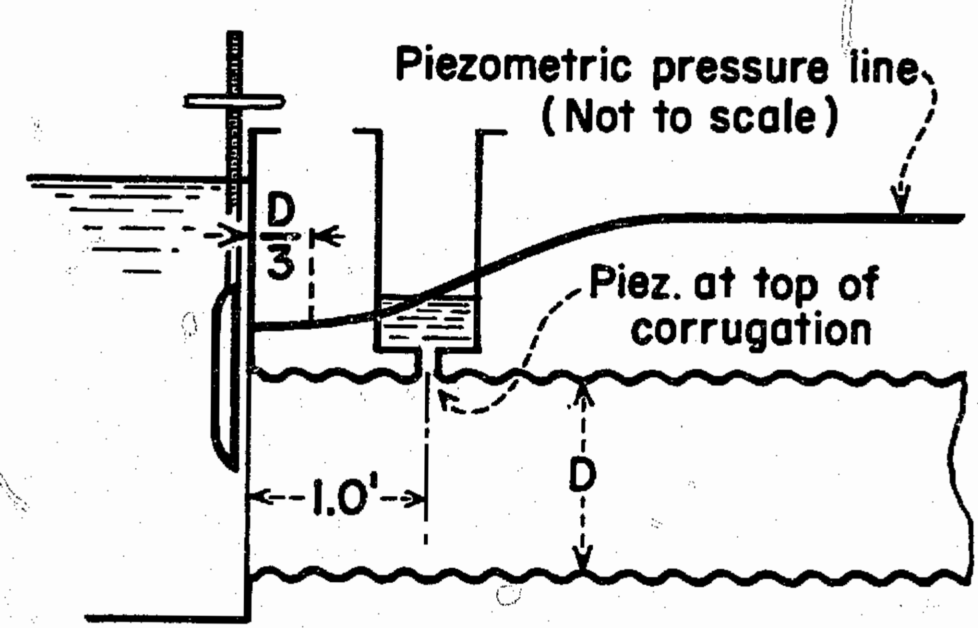


Figure 27

A better location for the downstream piezometer would have been D/3, measured from the downstream face of the gate. The pressure gradeline here is lower and flatter. Minor variations in piezometer locations would not result in major measuring errors. However, if the piezometer is moved to this point (to increase differential head), the meter must be recalibrated because the published tables will not apply.

Laboratory tests have been conducted on metergates to determine the coefficient of discharge C_d for a pressure tap located at D/3, as discussed. This curve shown in Figure 28 is valid for all sizes of metergates under certain standard conditions. These include:

1. Approach channel floor sloping upward, 2:1, toward gate with downstream end of floor 0.17D below pipe entrance invert.

2. Flaring entrance walls, 8:1, starting D/4 distance from edges of gate frame.

3. Zero gate opening set when bottom of leaf is at invert of entrance.

4. Upstream submergence is greater than D.

5. Downstream end of pipe is submerged to make pipe flow full.

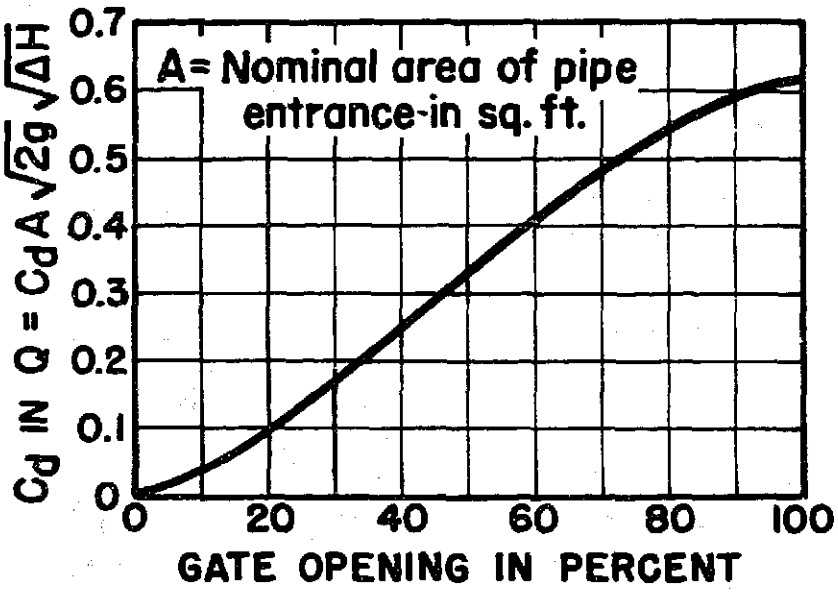


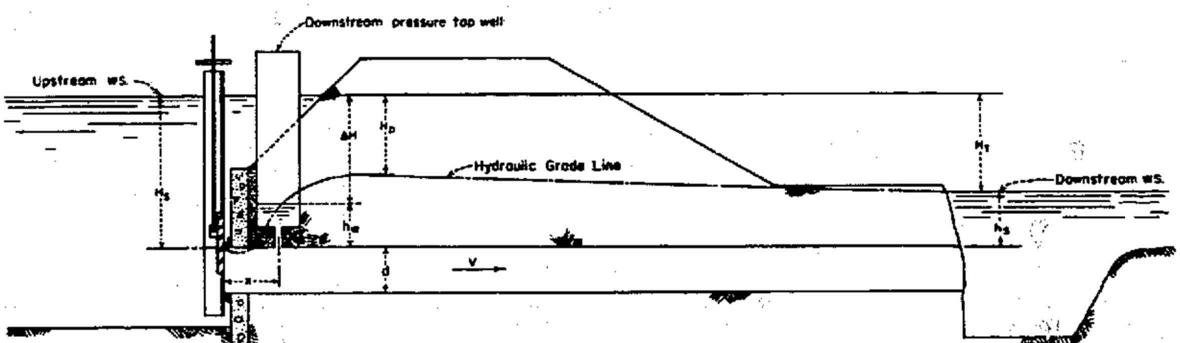
Figure 28

It should be noted that the coefficient C_d *is a different* coefficient than the C used in the orifice equation. C_d is used with A which in this case is the area of the pipe and not the gate opening. Discharges may be computed from this equation with an accuracy of plus or minus 2-1/2 percent. The degree of downstream submergence does not affect the accuracy of the meter if water rises sufficiently in the downstream well to obtain an accurate reading and the pipe runs full at the outlet.

METERGATE INSTALLATION

Metergates have been found to be set too low, too high, or the wrong size of gate was employed. To aid in the proper selection of gate size and the elevation at which the gate should be placed, the following suggestions are given in the drawing in Figure 29. The metergate entrance structure should be as described in the preceding discussion.

An analysis of other factors that influence metergate performance and accuracy, in cases where the installation is not standard, is given in Hydraulic Laboratory Report No. Hyd-471, dated March 15, 1961, "Flow Characteristics and Limitations of Screw Lift Vertical Metergates." This report covers various entrance problems effects of submergence velocity and gate design, and gives rating curves for 18-, 24-, and 30-inch gates for both confined and unconfined approaches.



SECTION THROUGH INSTALLATION

DETERMINATION OF METERGATE INSTALLATION

GIVEN

1. Upstream water surface El. 100.0.
2. Downstream water surface El. 99.0.
3. Turnout discharge, Q, = 8 cfs.
4. Depth of water in downstream measuring well, h_w, should be 6 inches above crown of pipe.
5. Length of metergate pipe, 50 feet.
6. Submergence of metergate inlet, H_s, should be equal to or greater than d above the crown of the pipe.

FIND

1. SIZE OF METERGATE (One of two methods may be used)
   a. Where downstream scour may be a problem.
      Select exit velocity that will not cause objectionable scour, say 4 feet per sec.
      From $A = \frac{Q}{V} = \frac{8}{4} = 2.00$, $d = 19\frac{1}{8}$ inches.
      Requires 20-inch metergate.
   b. Where scour downstream is not a problem.
      Assume metergate to be operated at openings up to 75 percent. (The influences of entrance design, upstream submergence and downstream pressure tap location are minor for these openings.)
      For 75 percent gate opening coefficient of discharge, $C_d \approx 0.5$, and maximum $\Delta H \approx 1.85\ H_D$
      $\Delta H \approx 1.85\ (1.0) \approx 1.85$ ft.
      From $Q = C_d A\sqrt{2g\Delta H}$
      Area of pipe, $A = \frac{8}{0.5\ (8.02)(1.36)} = 1.47$ sq. ft.
      $d = 16\frac{3}{8}$ inches.
      Requires 18-inch metergate, d = 18 inches.
   c. Check capacity of gate using 18 inch metergate.
      $H_D = H_T - H_f$ (H_f is friction loss from pressure recovery point to pipe exit).
      $H_f = f\frac{L}{d}\frac{v^2}{2g}$
      Where f is coefficient of friction, L is length of pipe, d is pipe diameter and v is velocity in pipe.
      From $v = \frac{Q}{A} = \frac{8}{1.767} = 4.53$
      Assume f for concrete or steel pipe as 0.025.
      $H_f = \frac{0.025\ (26.3)\ 20.5}{64.4} = 0.21$ feet.
      $H_D = 1.0 - 0.21 = 0.79$ feet.
      In order to have a measurable water surface in the downstream well for all gate openings and downstream tap positons the installation should be designed for maximum ΔH,
      $\frac{\Delta H}{H_D}$ (maximum) ≈ 1.85
      $\Delta H \approx 1.85 H_D \approx 1.85(0.79) \approx 1.46$
      Using this adjusted value of ΔH, turnout capacity at 75 percent gate opening, $Q \approx 0.5(1.767)(8.02)(1.21) \approx 8.57$ cfs.
      18-inch metergate is adequate.

2. ELEVATION AT WHICH METERGATE SHOULD BE PLACED.
   a. To meet upstream submergence requirement, H_s, of 1.0d, crown of pipe entrance should be set at El. 100.0 − d = 98.5.
   d. To meet requirement of water surface 6 inches above crown of pipe in downstream well, elevation of crown of entrance would be set at El. 100.0 $- \Delta H - h_w = 100.0 - 1.46 - 0.50 = 98.04$, say El. 98.0.
   Depth requirement for measurable water surface in downstream well is governing factor and gate should be set with crown of entrance not higher than El. 98.0.

3. MAXIMUM CAPACITY OF METERGATE (Full open)
   C_d for full gate opening with downstream pressure tap at $x = 2d/3$ (12 inches from entrance on 18-inch gate) is about 0.75
   $\frac{\Delta H}{H_d}$ for $x = 2d/3 \approx 1.1$
   $\Delta H = 1.1\ (0.79) \approx 0.87$
   From $Q = CA\sqrt{2g\Delta H}$
   $\approx 0.75\ (1.767)\ (8.02)\ 0.93$
   ≈ 9.9 cfs.

SCREW LIFT VERTICAL METERGATE

INSTALLATION CRITERIA AND EXAMPLE

FIGURE 29

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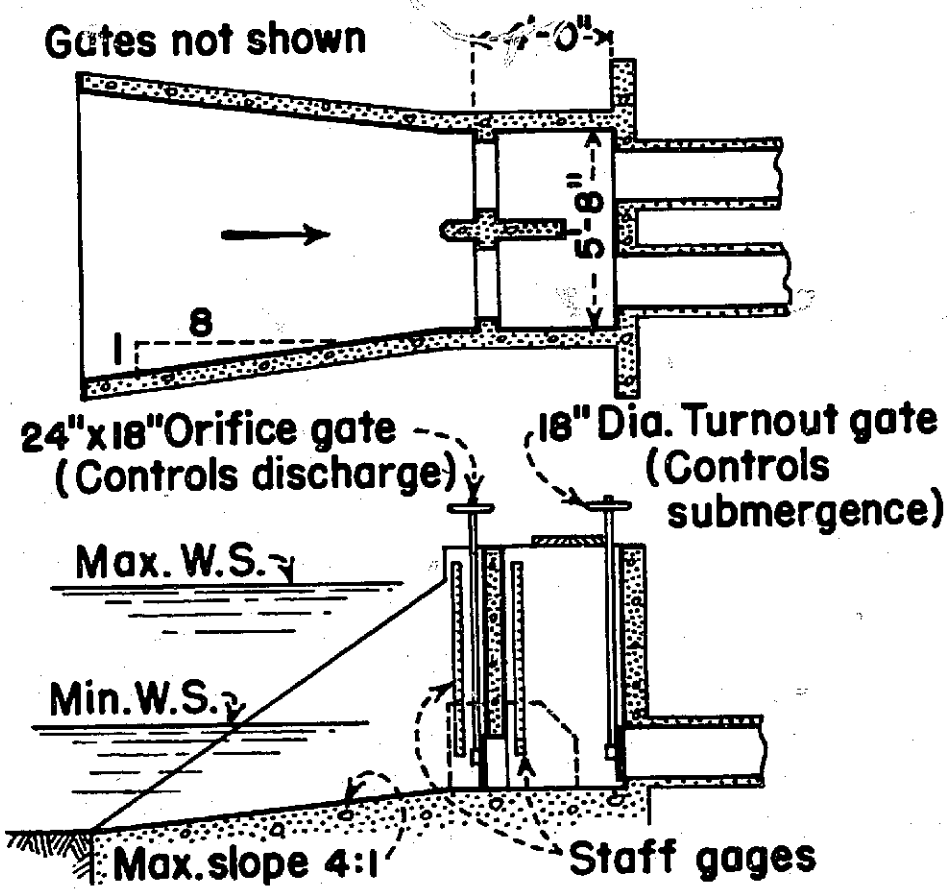
CONSTANT-HEAD ORIFICE TURNOUT (CHO)

The constant-head orifice turnout, Figure 30, is essentially a submerged orifice-meter type of measuring device. The upstream or orifice gate controls the discharge while the downstream or turnout gate controls the submergence on the upstream gate.



PLAN I

CONSTANT HEAD ORIFICE TURNOUT

Figure 30

As a means of standardizing the device, it was arbitrarily decided to always submerge the orifice gate sufficiently to produce a 0.2-foot difference in water surface elevation (differential head) across the upstream or orifice gate.

The constant-head orifice is usually operated as follows: The orifice gate opening for the desired discharge is obtained from the discharge table and set. The turnout gate is adjusted until the differential head across orifice gate is at the required constant head of 0.2 foot. The discharge will then be at the desired value. Two standard sizes of constant-head orifice meters have been calibrated and the discharge values are given in Tables 32 and 33 of the Water Measurement Manual.

The 10-second-foot capacity turnout is designed to operate with the canal water surface from 21 inches to 6 feet above the orifice gate seat. Minimum operating depth is 18 inches. This turnout uses a rectangular 24- by 18-inch screw lift vertical gate for the orifice gate and an 18-inch-diameter screw lift vertical gate for a turnout gate; two sets of gates are used side by side in the turnout structure which employs 18-inch-diameter pipe.

The 20-second-foot-capacity turnout is designed to operate with the canal water surface from 27 inches to 6 feet above the orifice gate seat. Minimum practical operating depth is about 24 inches. This turnout uses a rectangular 30- by 24-inch screw lift vertical gate for the orifice gate, a 24-inch-diameter screw lift vertical gate for the turnout gate; two sets of gates are used side by side and discharge into 24-inch-diameter precast concrete pipe.

DISCHARGE CHARACTERISTICS

The discharge through a constant-head orifice turnout may be computed from the orifice equation

$$Q = CA\sqrt{2gH}$$

where

- Q = discharge in cfs
- H = differential head on orifice gate (0.2)
- A = area of orifice gate opening in square feet
- C = coefficient of discharge
- g = acceleration due to gravity (32 ft/sec/sec)

The coefficient "C" determined in 98 tests on 6 different designs of turnout, for a complete range of gate openings and canal water surface elevations is shown in Figure 31.

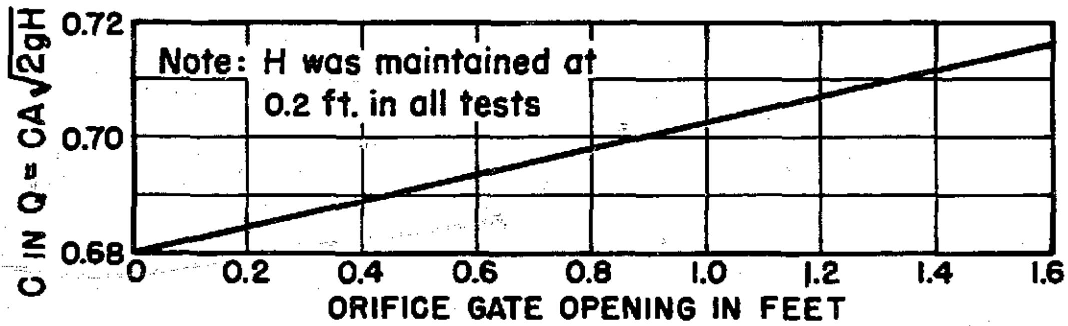


Figure 31

The discharge tables (referred to above) were prepared from this curve. Single barrel and double barrel tests gave the same discharge coefficients.

When only one of the two orifice gates is open, it is desirable to open the turnout gate directly downstream from the opened orifice gate. The head should be read on the sidewall of the pool next to the open gates. An incorrect head reading will be obtained if the gages on the sidewall opposite the open gate is used. If both turnout gates are opened with only one orifice gate open, an incorrect head reading will be obtained on all gages.

More consistent results will be obtained if the downstream gage is relocated adjacent to the orifice gate instead of adjacent to the turnout gate. Any arrangement of open and closed gates that produces a tilted water surface between the orifice and turnout gates should be avoided because of the difficulty in determining the head by any means.

DISCHARGE DETERMINATION ERRORS

Since the principle of operation of the constant-head orifice turnout is to maintain a constant differential of 0.2 foot across the orifice gate, it is extremely important that this differential be determined accurately in the field if accurate discharge determinations are to be expected. The equation for discharge may be written

$$Q = CA\sqrt{2g}\sqrt{H}$$

where H is 0.2 foot.

If an error of 0.01 is made in reading each gage, H could be as small as 0.18 foot or as large as 0.22 foot. The error in discharge would be proportional to the square root of the head or

$\sqrt{0.18} = 0.4243$

.... Difference

0.0229   0.0224 average

$\sqrt{0.20} = 0.4472$

.... 0.02.8

$\sqrt{0.22} = 0.4690$

and

$$\frac{0.0224}{0.4472} \times 100 = \pm 5 \text{ percent}$$

For an error of 0.02 foot in reading each gage the discharge determination error would be plus or minus 10 percent.

It is, therefore, apparent that accurate discharge measurements can be obtained only if great care is used in determining the differential head. Some operators have complained that it is next to impossible to read a staff gage accurately when looking downward at a steep angle into a dark hole at a choppy water surface which may also be surging. Since there is a good bit of truth in this statement, two suggestions are given to help obtain a better differential head reading.

The staff gages could be mounted in stilling wells made from a suitable length of commercially available transparent plastic pipe. This would prevent the choppy water surface from interfering with making an accurate reading. The wells should be quickly removable for cleaning. If surges are causing the water level in the well to rise and fall, a wood bottom having a 3/8-inch hole drilled in it could be fastened in the well. This would allow the well to average the surges and provide a more dependable head determination.

A second method would make use of a portable manometer constructed to be temporarily mounted on the transverse concrete wall between the staff gages. The portable manometer would be constructed as shown in Figure 32.

Flexible transparent plastic tubing is slipped over a metal tee (soldered copper tubing) so that both legs are interconnected at the top and lead to a stem fitted with a stopcock. Water is sucked up into the manometer to the desired height for ease of reading and the stopcock tightened to hold the water columns up. The partial vacuum, applied equally to each water column, does not change the differential head. The metal discs with small holes (plus or minus 1/8-inch) drilled in them may not be necessary but will help to stabilize the water columns if surges are a problem. The discs

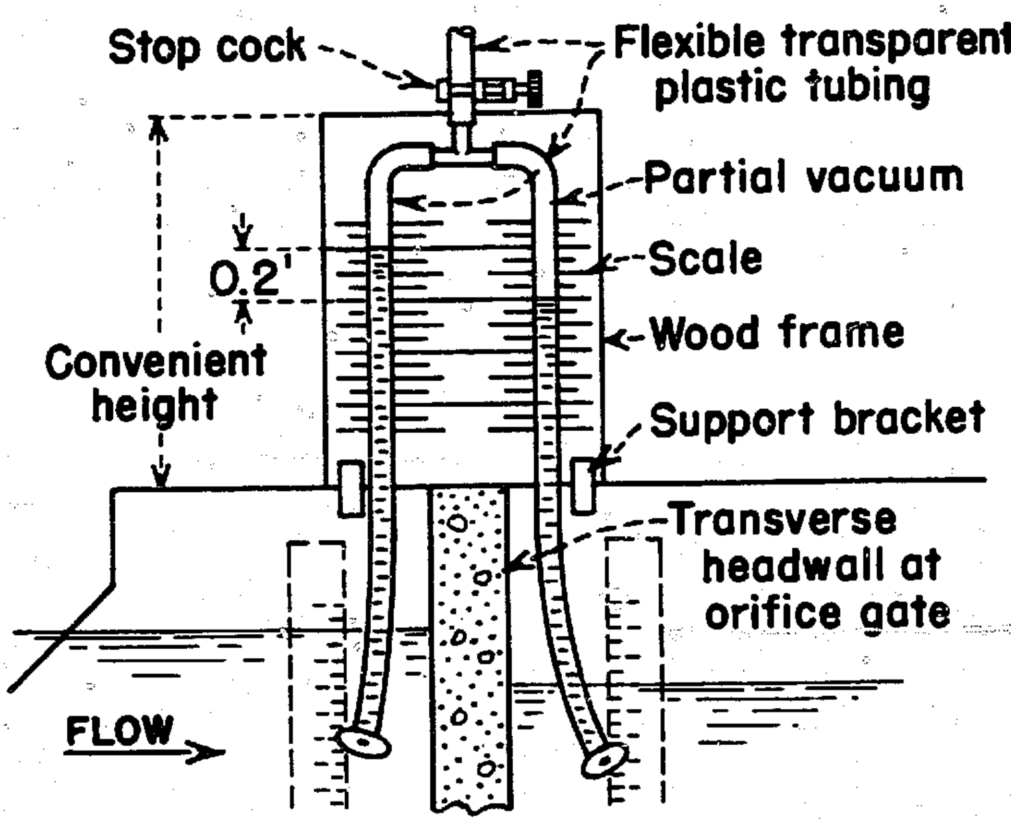


Figure 32

should be thin (1/16 inch or less) and several inches in diameter. Rubber stoppers with a preformed hole might be used in place of the discs if the velocity past the hole is not too great. The inside diameter of the tubing should be several times greater than the diameter of the hole in the rubber stopper or metal disc to obtain significant damping action on the water columns in the tubing.

The differential head for normal operation may be increased if difficulty still exists in setting, reading, or maintaining the 0.2-foot standard differential. Discharges may be calculated using the coefficient for the orifice gate opening actually used and the differential head actually measured. Turbulent flow conditions or a reduced submergence at the turnout gate will not affect the discharge if it is possible to obtain a true downstream head. To be certain of the accuracy of these higher differential head discharges, it would be desirable to check several gate settings using a current meter or other calibration method to measure the discharge. This displacement, if any, of the coefficient curve from the values given for the 0.2-foot differential could be determined from calculations, and a new coefficient curve drawn parallel to the one shown. Only a few accurate check points would be required because the curve shape would necessarily be the same as for the 0.2-foot curve.

Errors in discharge measurement might be caused by factors other than the head measurement as discussed for metergates and orifices.

EFFECT OF ENTRANCE STRUCTURE GEOMETRY

The preceding sketch of the constant-head orifice turnout indicates 8:1 flaring walls in plan on the entrance or approach structure and a 4:1 sloping floor. The floor slopes downward away from the orifice gate, Plan 1, Figure 30. Other common installations are shown in Figure 33; each has 8:1 flaring walls in plan.

Selection of the type of entrance for a new installation will usually be limited by the relative elevations of the canal bottom and the pipe invert. If there is any choice in the matter, however, it should be noted that Plans 1 and 2 provide the best operating structures. Flow conditions with these entrances are steady and smooth and the differential head is not difficult to read. When the entrance is partially constricted as in Plans 3 and 4, by an adversely sloping floor, the flow pulsates and the surface is rough. Surges and boils upstream from the turnout gates tilt the water surface and make the head difficult to read.

It has been noticed that some structures in the field are now of the Plan 4 type, even though they were Plan 2 type when installed. Sediment and debris have collected near the entrance to the turnout and maintenance crews have cleaned for only a short distance upstream, producing the sharp downward slope to the orifice gate. More extensive pool cleaning would improve the ease of obtaining head readings and might improve the accuracy of the measurement.

CURRENT METER GAGINGS

The current meter uses the velocity principle to obtain discharge. Velocity is measured in a small area at one time and therefore, enough readings must be taken to insure obtaining accurate values for the average velocity and for the area of the flow section.

In selecting a site for a gaging station or a location for a meter or any other propeller device, it is important that smooth uniform flow exist upstream (to some degree downstream) from the location at all times. The approach to the site should be straight for several hundred feet or more and, to the eye, the surface velocity should be the same across the entire width of the section. The cross section of the site should be typical of the sections upstream and downstream and should be in stable material. Locations where banks or bottom can erode or where sediment

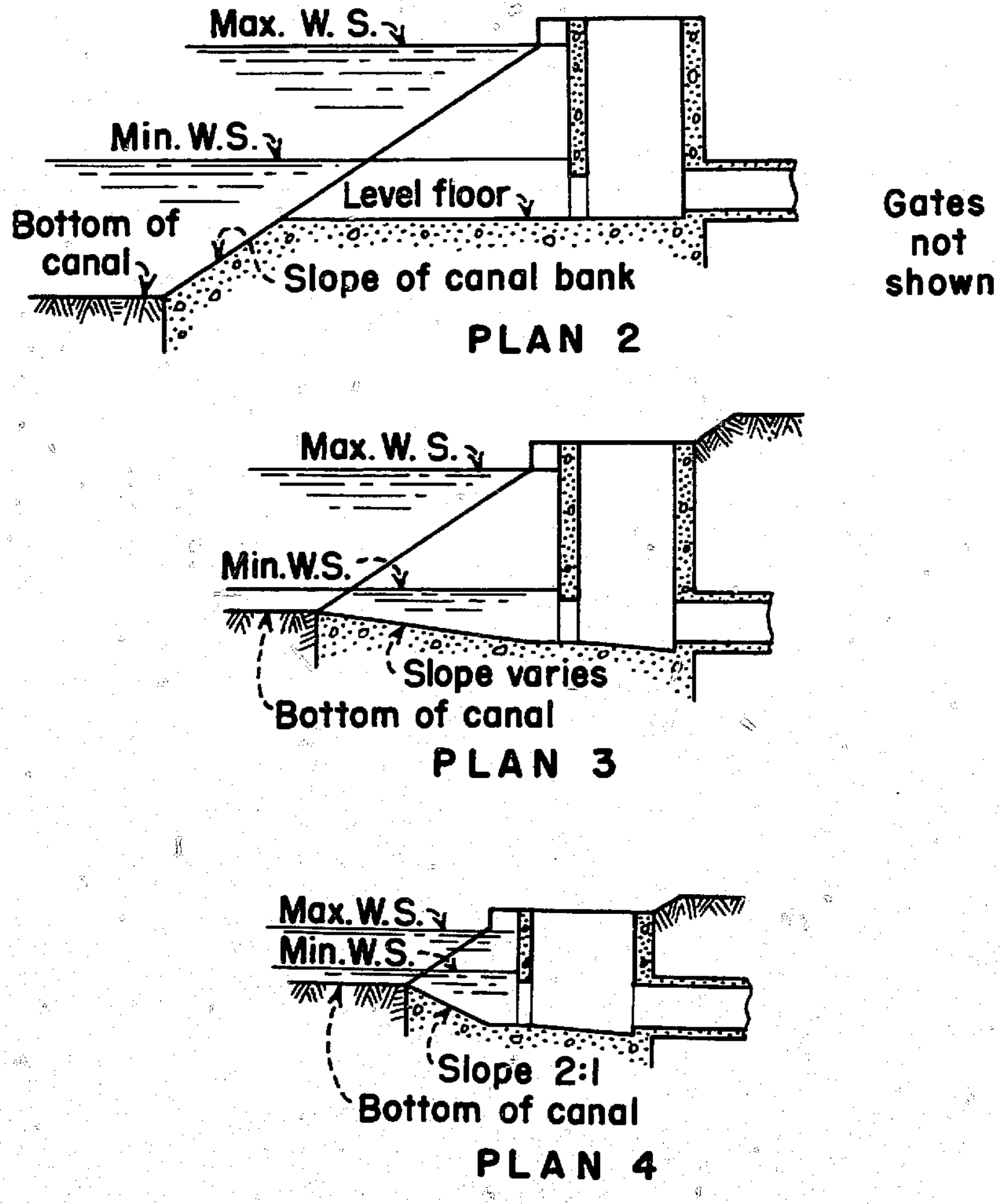

Max. W. S.
Min. W.S.
Level floor
Bottom of canal
Slope of canal bank
Gates not shown
PLAN 2
Max. W. S.
Min. W.S.
Slope varies
Bottom of canal
PLAN 3
Max. W.S.
Min. W.S.
Slope 2:1
Bottom of canal
PLAN 4

Figure 33

is known to deposit, should not be used. A site where meters can be operated from the upstream side of a bridge is desirable because a cableway or other crossing need not be constructed. The depth at minimum flow should be sufficient to use a current meter in its usual velocity range. If this is not possible, choose one site for low flows and another one for high flows.

The sensitivity of the station, in general, should be suitable--small errors in stage reading should not result in large measurement errors. Conversely, a significant change in discharge should be accompanied by a significant change in stage.

Sites affected by variable depth backwater should be avoided as should those having seasonal growths of aquatic weeds, or those having a confluence with a sizable tributary downstream.

The selected site should be close to a benchmark for easy zeroing of the staff and other gages. The station should be suitable for the installation of a water stage recorder and an intake to the stilling gage well. These should be closely grouped because it is imperative that the recorder and staff gage indicate the same water level.

Reference or staff gages should be firmly anchored and in an inclined or vertical position. Vertical gages should be vertical in all respects. Inclined gages should be graduated on the site by precise leveling after the gage is installed. Clear, accurate markings (0.01 foot) should extend to above and below the anticipated levels. Stilling wells should be vertical and have sufficient depth and height to allow the float to travel the entire range of water levels. The float diameter should be large enough to overcome friction and actuate the recorder as necessary. The intake piping to the well should be large enough to allow the water in the well to rise and fall with the river or canal stage without undue delay. All pipe joints should be watertight. Elbows should be made up of plugged tees to allow rodding if clogged.

Recorder chart records should be legible to read plus or minus 0.02 foot or better. If waves are causing the recorder to blur, use a restriction (perhaps a partially open gate valve) in the stilling well piping. A steel tape (electric indicator) should be installed in the well to set and check the recorder.

Recorders should be of such a design and type that a change in the chart record can be produced only by a change in water level. The recorder should be sensitive to changes of 0.02 foot or less. Clocks should be reliable and keep good time.

The paper chart scale chosen for recording should permit readings to be made which are within 1 percent of the depth of the water (above zero flow level) or within 0.02 foot, whichever is greater.

Price-type current meters should not ordinarily be used where velocities are less than 0.5 foot per second. The upper range should not exceed the calibrated range of the meter. Meters should not be operated in shallow water when the horizontal axis of the meter is closer to the surface than 1-1/2 times the vertical dimension of the rotor. Similar bottom clearance should be provided, measured from the top of any obstruction such as a rock or ledge. Meters should be re-rated after about 100 hours of use or at least checked against some known standard. They should be re-rated immediately if dropped, bumped, or used extensively in sediment laden water.

In making a gaging measurement, choose a time when the stage will remain constant throughout the measurement. Discharge corrections for a changing stage are never completely satisfactory, even when all of the factors are known.

Current meter measurements are usually made on "verticals; i.e., vertical lines on the cross-section," chosen so that they provide an adequate sample of the velocity distribution in the cross section. These verticals should be chosen so that (1) the error in computing the area of the segment between two verticals does not exceed 3 percent if the portion of the bed profile between the verticals is treated as a straight line, and (2) the difference between the mean velocities on adjacent verticals does not exceed 20 percent by reference to the lower two (except close to the banks). In general, this means that the intervals between verticals should not be greater than 1/15 of the cross-section width (when the bottom is smooth), or 1/20 of the width when the bottom is irregular. Verticals need not be closer than 1 foot in any case; the number may be reduced when working in small lined channels having a regular geometric profile.

Provisions should be made to operate the meter from a cable or rod suspended in such a way that the performance of the meter is not affected by disturbances in the flow caused by the observer or the suspension equipment. The meter should be held in a given position, after allowing operation to become stabilized for 40 seconds or more. Total operation of the meter at each vertical should be not less than two consecutive periods of at least 40 seconds. If significant differences are apparent, more readings should be taken. The mean of all the readings at that point should be used for the velocity, unless there is an obvious reason for eliminating one or more readings. The meter should be removed from the water between readings to be sure that its rotation is not being impeded by debris or any other cause.

Errors will arise if the meter:

(1) is used to measure velocities less than 0.5 foot per second or beyond the calibrated range

(2) is not held steady in the same location during the timing sequence, or if the meter is held in an unsteady flow area such as an eddy

(3) is used when there is a significant water surface disturbance by wind

(4) is used in flow which is not parallel to the axis of a propeller-type meter or is oblique to the plane of the cup-type meter

If only one or two velocity measuring points on each vertical are obtained, an arithmetic solution to obtain the discharge is appropriate. If only one velocity on a vertical has been determined, the mean velocity is (1) the value observed at 0.6 depth used without modification; (2) the value observed at 0.5 depth multiplied by 0.96. If two velocity points, such as the 0.2 and 0.8 depth have been determined the average of the two points should be used.

To compute the discharge the cross section should be regarded as being made up of a number of segments, each bounded by the two adjacent verticals. If V_1 is the mean velocity at the first vertical and V_2 the mean velocity at the second vertical and, if D_1 and D_2 are the depths measured at the respective vertical, and if b is the horizontal distance between these verticals, the discharge of the segment is:

$$Q = \frac{(V_1 + V_2)}{2} \times \frac{(D_1 + D_2)}{2} \times b$$

This calculation is repeated for each full segment. Segments adjacent to the banks may be handled by assuming zero depth and velocity at the water's edge. The total discharge is obtained by adding together the discharges from all the segments.

Careful plotting of a stage-discharge relationship curve for each gaging station will help to evaluate the accuracy of each gaging measurement as it is made, and will help to establish confidence in the station. After the station is put into operation, the cross section should be checked periodically and maintained in its original condition. Sediment bars should be removed from the bottom and corrections to the net section made, if erosion occurs on the banks or bottom. If the water surface is raised or lowered by checking, careful time records should be kept to determine when the staff gage or water stage records are an indication of the discharge.

WEIRS

Since weirs were frequently used as examples in "General Aspects of Water Measurement Accuracy," they will not be elaborated upon in this portion of the text. They have been specifically referred to under headings of: Approach Flow, Turbulence, Velocity of Approach, Exit Flow Conditions, Weathered and Worn Equipment, Poor Workmanship, Faulty Head Measurement, and Use of Wrong Measuring Device.

PROPELLER METERS

Propeller meters have been in use since about 1913 and are of many kinds, shapes, and sizes. They are used submerged near the ends of pipes or conduits or "in-line" in pressurized pipe systems. Many meter designs and modifications for special conditions and purposes are available from manufacturers and it is therefore impractical to try to discuss all makes and models. All propeller meters have certain common features, faults, and advantages, however, which can be analyzed to provide a better understanding of meter operation. A thorough comprehension of meter principles, their inherent limitations and advantages, and the operating experiences of many users may be beneficial when purchasing, installing, operating, and maintaining meters for a field installation. The information presented herein has been gathered from project and water district personnel, letters, reports, inspections, complaints, and from laboratory and field tests conducted specifically to evaluate propeller meter performance. Much information, some good and some doubtful has been sifted to emphasize basic material and eliminate incorrect or conflicting statements. An attempt has been made to eliminate material which applies only, or particularly to, one make of meter or to one specific installation. The material presented applies to all meters, in general, and to all installations, except as noted.

Propeller meters utilize a multibladed propeller (two to six blades) made of metal, plastic or rubber, rotating in a vertical plane and geared to a totalizer in such a manner that a numerical counter can totalize the flow in cubic feet (perhaps to within 10 cubic feet), acre-feet, or any other desired volumetric units, and/or an indicator to show the instantaneous discharge, in cubic feet per second, acre-feet per day, or any other desired units. The propeller, designed and calibrated for operation in a pipe or conduit, should always be fully submerged, that is, the pipe or conduit must be flowing full. The propeller diameter is always a fraction of the pipe diameter; usually varies between 0.5 to 0.8 of the diameter of the pipe. Compound flowmeters have more than one propeller and are more complicated in design. Meters are available for a range of pipe sizes from 2 to 72 inches in diameter.

The measurement range of the meter is usually about 1 to 10; that is, the maximum discharge the meter can indicate or totalize is about 10 times the minimum that the meter can handle. The meter is ordinarily designed for use in water flowing at from 0.5 to 17 feet per second although inaccurate registration may occur for the lower velocities in the 0.5 to 1.5 ft/sec range.

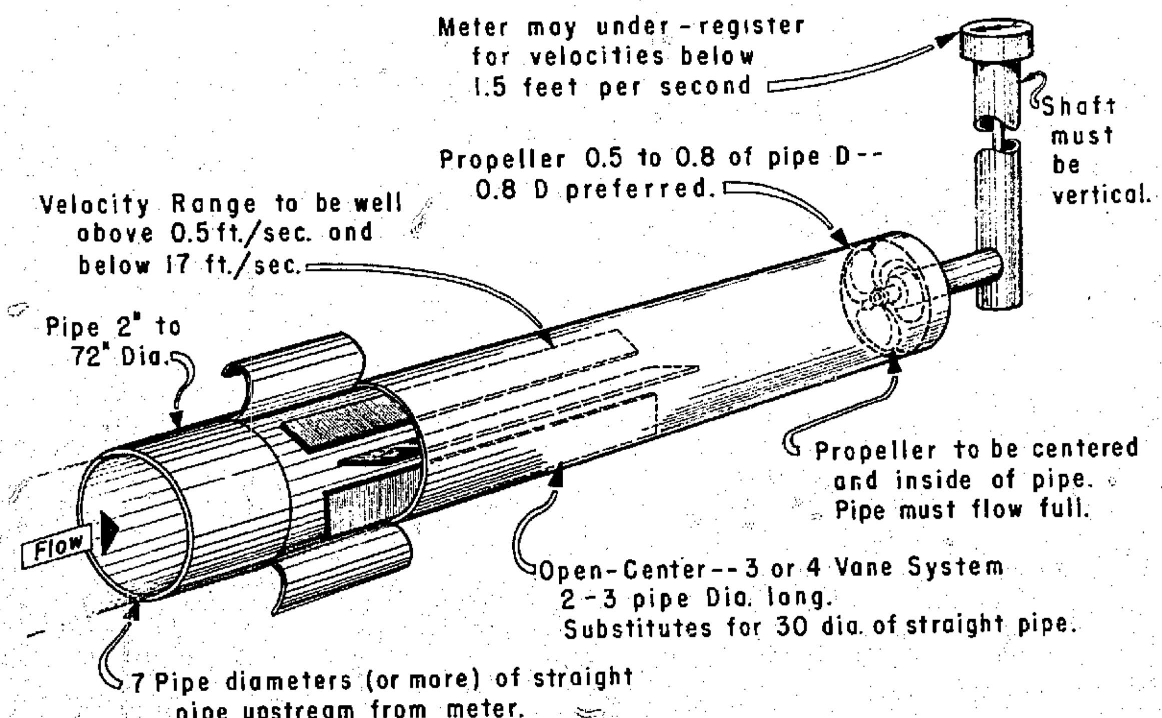
The meter size is usually stated as the diameter of the pipe in which it is to be used. The propeller size may vary for a given size of meter. For example, a 24-inch meter might have a 12-inch-diameter propeller for use in a 24-inch-diameter pipe. Thus, the principle involved in measuring discharges is not a displacement principle as in certain municipal water meters or indicating devices used on gasoline service station pumps, but rather a simple counting of the revolutions of the propeller as the flow passes the propeller and causes it to rotate. Anything that changes the frictional resistance of the propeller, the number of revolutions in a given time, or the relationship between pipe and propeller areas, therefore, affects the registration or accuracy of the meter. The many factors affecting propeller rotation are discussed in the following pages.

FLOW PATTERNS

The accuracy of a propeller meter (in new condition) is primarily dependent upon the similarity of the flow patterns in the vicinity of the propeller during calibration and during regular use. Factors which change the flow pattern approaching or leaving the propeller will change the accuracy or registration of the meter.

SPIRAL FLOW

A poor entrance to a turnout pipe, elbows, fittings, unsymmetrical approach flows and many other factors can produce spriral flow in a pipe. The propeller meter, because it has a hub at the center of the pipe and a revolving propeller, is therefore very sensitive to water flowing in a spiral pattern. Significant errors in registration can result when the meter is used in spiral flow. Depending on the direction of rotation of the flow with respect to the pitch and direction of rotation of the propeller, the meter will over or under register. Flow straightening vanes inserted in the pipe upstream from the meter will help to eliminate errors resulting from this cause. The meter manufacturer usually has specific instructions regarding the size and placement of vanes and these should be followed when installing a propeller meter. It is usually suggested that vanes be several pipe diameters in length and that they be located in a straight, horizontal piece of pipe just upstream from the propeller. The horizontal pipe should be seven diameters or more in length. Vanes for clean waterflow are usually made in the shape of a + sign to divide the pipe into equal quarters. Laboratory experiments have shown that vanes of this type (used where no spiral flow exists) may reduce registration by 1 to 2 percent compared to readings made with the vanes removed. This is because the area taken up by the vanes tends to reduce the velocity near the center of the propeller. Some manufacturers have suggested the use of vanes which do not meet in the middle but divide the pipe into thirds as shown in Figure 34. A variation of these two configurations, that leaves the central part of the pipe open and provides four vanes, is shown in Figure 35.



DESIRABLE FEATURES FOR PROPELLER METERS
FIGURE 34

Figure 34

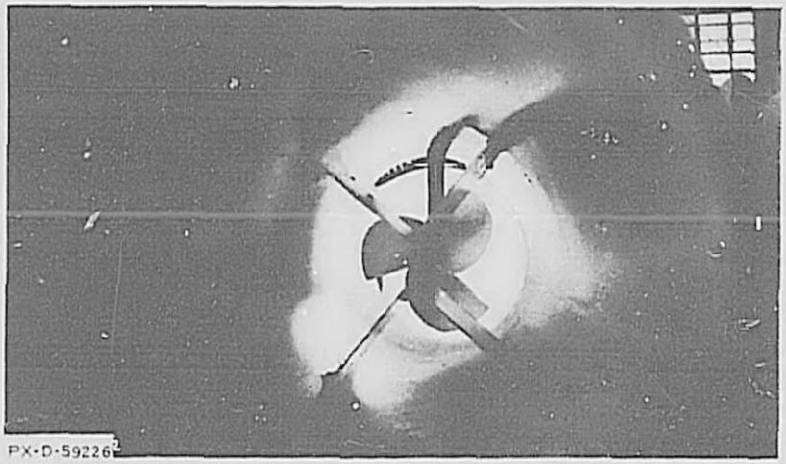


Figure 35

Because of the open flow area in the center of the pipe there is less disturbance to the flow pattern in the center of the pipe. One or more diameters of clear space between the downstream end of the vanes and the propeller helps to nullify any adverse effects caused by either type of vane.

If straightening vanes are not used, a long length of straight, horizontal pipe (30 or more diameters long) may be required to reduce registration errors. Venting the pipe to the atmosphere just downstream from the control gate, if this is possible, may help to reduce spiral flow.

VELOCITY PROFILES

In any pipe--even a very short one--the friction between the inside flow surface of the pipe and the water is greater than the internal friction of the water. This results in the water in the center area of the pipe having a higher velocity than the water near the boundary.

In a short pipe the velocity profile would be similar to Case A, Figure 36; in a longer pipe the profile would look as shown in Case B. In the latter fully developed velocity profile the difference between the center and edge velocities is quite large but is stable and does not increase further. It is obvious therefore that the propeller, which receives its impetus from the central area of the pipe, will receive different total forces for Cases A and B above, and that a greater number of revolutions will occur in the long pipe, Case B. On the other hand, less force, and fewer revolutions (under registration), will occur for Case A. Laboratory tests have shown that long turnouts or pipes (30 or more diameters long) have fully developed

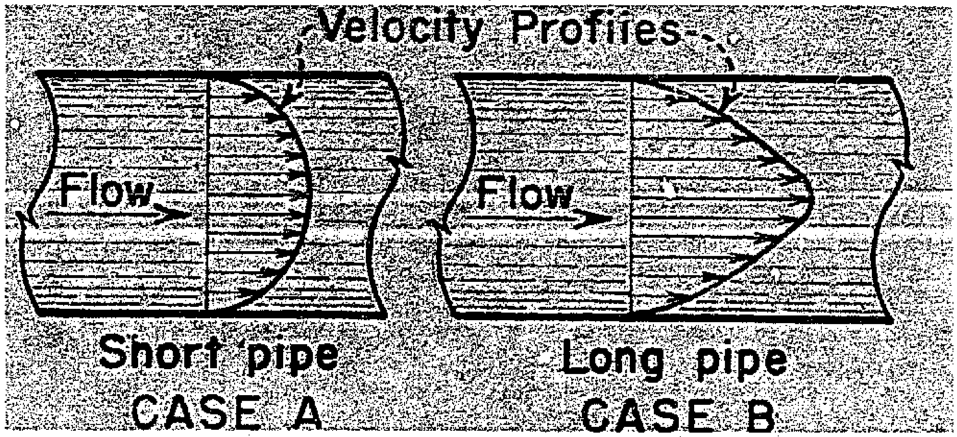


Figure 36

velocity profiles and give 3-4 percent greater registration than short pipes, 6-10 diameters long. Rough or corrugated pipes tend to produce the Case B velocity profile in shorter lengths than smooth wall pipes. No exact data are available to define every situation, however.

Control gates, such as the slide gate often used at a turnout entrance, may affect the flow pattern and/or the velocity profile in a short pipe. For example, tests have shown that for a 24-inch pipe turnout 23 feet long from gate to meter, in the 5-cfs range, a full gate opening resulted in an indication of 96 percent of the true discharge. When the same discharge was put through a 6-inch gate opening, the meter indicated 100 percent of the actual discharge. The 4 percent difference in discharge was the maximum effect noted in these tests.

It is apparent, therefore, that changes in velocity distribution in the pipe cross section, that make the distribution significantly different from that used during the meter calibration, will cause a change in meter registration. Changes may be either plus or minus with respect to the original discharge calibration. Inspection and analysis of the flow conditions upstream from the meter may prove beneficial in trouble shooting a field installation suspected of giving incorrect meter registrations. Checking the flow distribution in a cross section just upstream from the meter with a Pitot tube would conclusively establish whether a poor velocity pattern was present. Meters are never calibrated with poor velocity patterns in the pipe.

PROPELLER AND PIPE SIZE RELATIONSHIPS

Meters should always be used in pipes of the proper or recommended diameter. The meter manufacturer can supply this information. However, a discussion of the relative effects of propeller diameter will be helpful in understanding the trends in over and under registration where a meter is used in a pipe larger or smaller than the recommended size. Propeller diameters vary between 0.5 and 0.8 of the pipe diameter. It is therefore not always possible to determine the proper pipe size by measuring the propeller diameter.

In visualizing the effects of a larger or smaller pipe on the accuracy of a given propeller meter, Cases A and B of Figure 36 will be helpful. Depending on the type of velocity profile in the proposed larger or smaller pipe, the propeller will tend to intercept higher velocities (overregister) or lower velocities (underregister) and will be in error depending on the change in the average velocities intercepted. No numerical values can be given because different meter manufacturers have different propeller designs. For example, propeller tips are affected differently by varying velocity values; also, different clearance requirements between the propeller and the pipe wall produce different register values. Items that are important for one propeller shape may not affect another. Tests have shown, however, that the larger the pipe diameter with respect to the propeller diameter, the more a change in velocity profile will affect the meter registration. Thus, meters having propellers nearly the diameter of the pipe they are to be used in should provide most accurate results. Conversely, propellers that are half or less than the pipe diameter will give the least accurate results.

It has been established in laboratory tests that changes in registration for two pipe sizes will be minor if, for both pipe sizes, the propeller diameter is 75 percent or more of the pipe diameter. In all cases, registration errors will be less in rough wall pipe because the rough wall helps to establish quickly a fully developed velocity profile.

Even when it is established that the differences in velocity profiles intercepted by the propeller will be negligible, the meter must be corrected (change in gear ratio or other means) to account for the change in pipe diameter.

PROPELLER MOTION

Since the meter head, in effect, counts the number of revolutions of the propeller to indicate the discharge, any factor that influences the rate of propeller turning can affect the meter registration. Practically all propeller effects are negative; that is, they reduce the number of propeller revolutions which would otherwise occur and result in under registration. More water is therefore delivered than is indicated or paid for.

Propellers are usually designed to turn on one or more bearings. The bearings are contained in a hub and are protected from direct contact with objects in the flow. However, water often can and does enter the bearing. Some hubs trap sediment, silt, or other foreign particles, and after these work into the bearing a definite added resistance to turning of the propeller becomes apparent. Some propellers are therefore designed for flow-through cleaning action so that particles do not permanently lodge in the bearings.

Silt has been found to be particularly damaging to bearings. It is present in many flows and as a result many otherwise satisfactory meters have been rendered unfit for service. Figure 37 shows the wear in the worm gear teeth of an inline meter laboratory tested for 2,200 hours in fairly clear water.

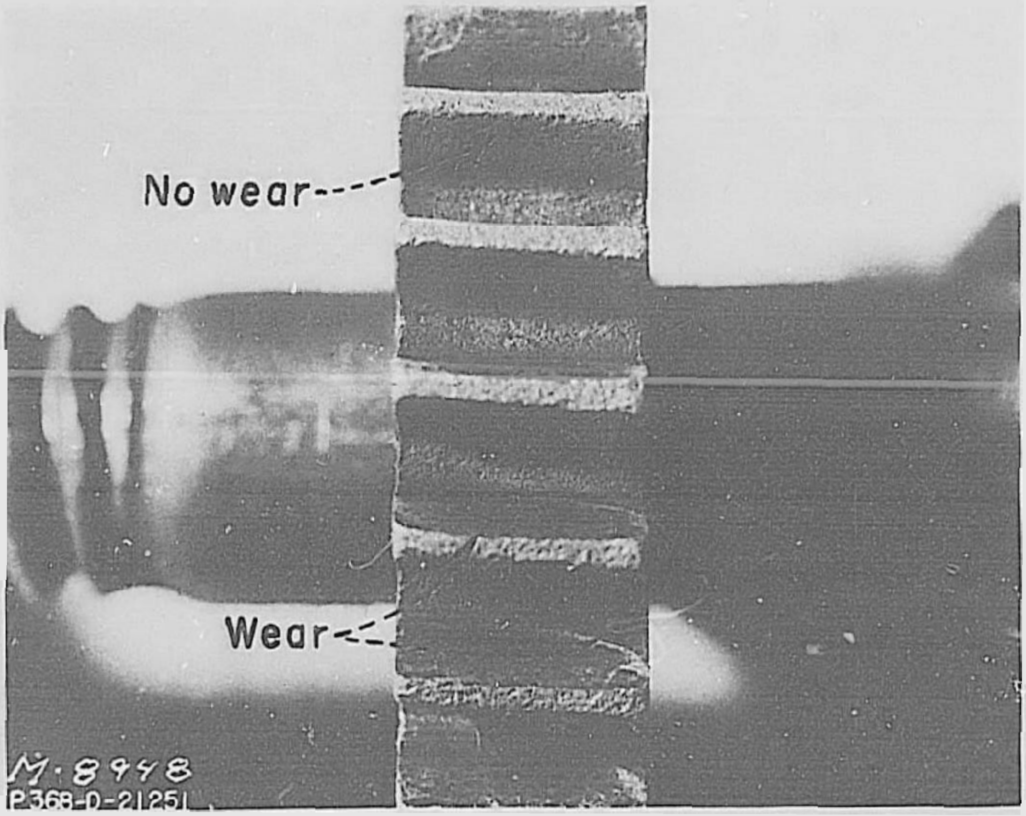


Figure 37. Flowmeter gear teeth worn on one face--
2,200 hours of operation

In another laboratory-controlled test a flow containing 5,000 parts per million of silt and very fine sand (0.005-0.15 mm) was used to test a medium size open-flow propeller meter. In less than 2 hours of operating time silt particles had collected in the bearing and produced propeller binding. After 5 acre-feet of water had passed the meter, 22 grams of silt (dry weight) were found in the propeller hub and bearing housing. (A 50-cent piece weighs about 12 grams.) A check of the propeller showed that it turned intermittently and slower than when new, in water flowing at 0.5 ft per sec and less.

After 45 acre-feet of water had passed the meter the unit was disassembled; bearing wear was easily visible. Holes were drilled through the bearing assembly. Less silt was then found to accumulate. In effect, when the hub could be drained, less bearing damage was evident but bearing wear was not eliminated. Sand traps may be necessary in field installations to reduce the amount of sediment (bedload) reaching the meter.

Care should be taken in lubricating meter bearings. Use of the wrong lubricant (perhaps none should be used) can increase the resistance to propeller motion, particularly in cold water. It should also be established that the lubricant is reaching the desired bearing or other surfaces after it is injected. For some meters, the manufacturers did not recommend lubrication of the bearings.

Although propellers are designed to pass (to some degree) weeds, moss, and other debris there is a limit to the amount of foreign material that can be tolerated in the flow. Even moderate amounts of floating moss and/or weeds can foul a propeller unless it is protected by screens. Heavy objects can break the propeller. With larger amounts, or certain kinds, of foreign material in the water even screens may not solve the problem.

METER SCREENS, SAND TRAPS

Screens to protect meter propellers are usually designed for a particular type of turnout on a particular canal, and to handle a particular type and size of debris; however, screens have certain common features which seem to be universally desirable and which help to prevent head losses across the meter and improve the quality of water measurement.

Screens usually consist of a metal frame, covered with wire mesh, which fit into a slot at the upstream end of the turnout. Double screens (set a foot or more apart in small turnouts) are usually desirable so that protection is provided while one screen is being removed and cleaned. The wire mesh usually varies from 1- by 1-inch (No. 9 wire) to 1/4-inch galvanized hardware cloth. Openings of one-half inch seem to be most successful and popular. Another common size is 5 by 5, or 5 by 4 mesh, and 19-20 gage wire.

Small screens may be set on a slope but larger screens should be set in the vertical position so that a winch (sometimes portable) can be used to raise and lower the screen for cleaning. Cleaning may be done by broom, wire brush, or water jet. Reverse flow through the screens may also be used but provisions must then be made for wasting the cleaning water. In large turnouts from canals, traveling screens may be used to remove debris and reduce the trash problems for several meters in the turnout.

Screen area should be a minimum of 8-10 times the area of the flow cross section; in many installations the screen area is 15-20 times the flow area and this has not been found excessive. Where sizable head losses cannot be tolerated the screen area should be large, the cleaning frequent, or both.

Sand traps, to catch the bedload (sand and gravel that moves along the bottom), should be arranged so that the trapped material can be flushed along the main canal--not into the turnout. Settling basins to trap the larger particles of suspended sediment (suspended in the flowing water) may be helpful at a meter installation. To remove suspended sediment the velocity of the approaching flow must be reduced to allow sediment to settle out. To accomplish this, fairly large and relatively costly settling basins are required. The advice of an expert should be obtained before considering a facility of this type.

HEAD LOSSES

The head loss across a propeller meter is usually considered to be negligible, although there is evidence that losses for open-flow meters may run as high as two velocity heads. This is equivalent to 0.6 foot of lost head in a 24-inch-diameter pipe carrying 8 cfs. The losses for certain inline compound and other type meters may be as high as 6 to 8 feet of head. In general, however, losses are low but measurements of loss for all meters have not been made.

In many cases, turnout losses including losses through the pipe entrance, screens, sand trap, pipe, etc., are large enough to make the losses at the meter seem negligible. Some allowance for meter losses should be made during turnout design, however, and the meter manufacturer can usually supply the necessary information.

METER ACCURACY

The accuracy of most propeller meters, stated in broad terms, is within plus or minus 2 to 5 percent of the actual flow. Greater accuracy is sometimes claimed for certain meters and this may at times be justified. On the other hand, it is sometimes difficult to repeat calibration tests under controlled conditions in a laboratory within plus or minus 2 percent. A change in lubricating practice or lubricant, along with a change in water temperature can cause errors of this magnitude. A change in line pressure (the head on the turnout entrance) can cause errors of from 1 to 2 percent.

EFFECT OF METER SETTING

The setting of the meter in the turnout may be responsible for sizable errors if the meter is not carefully positioned. A meter (24-inch-diameter pipe, 12-inch-diameter propeller, 8-cfs discharge) set with the hub center 1 inch off the center of the pipe showed an error of 1.2 percent. When the meter was rotated 11.5° in a horizontal plane (one-fourth inch measured on the surface of the 2-1/2-inch-diameter vertical meter shaft housing) the error was 4 percent; for 23° the error was 16 percent (under registration). Setting the meter (shaft housing) in a nonvertical position would introduce the same degree of error.

EFFECT OF INITIAL COUNTER SETTING

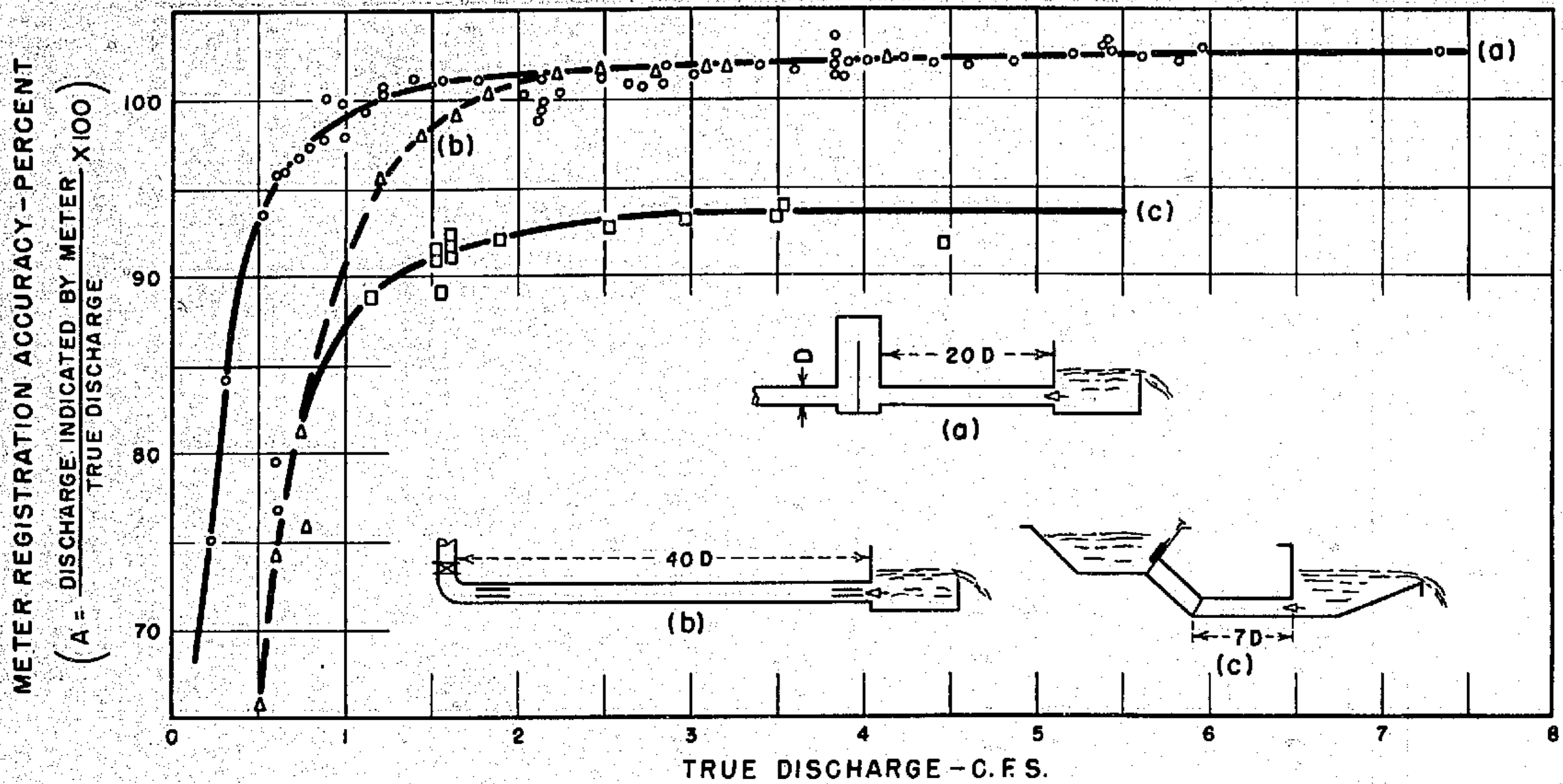
Meter manufacturers recognize that meters tend to underregister after they have been in use for a time and some meters are set to read 101.5 percent of the actual flow as their initial registration. This is done in anticipation that the meter will read correctly (100 percent of the actual flow) during the middle portion of the meter's life. Meters which are readjusted to record a particular flow (the lower end of the scale) with greater accuracy may cause registration errors of up to 10 to 15 percent at greater flows (the high end of the scale).

EFFECT OF RAPIDLY VARYING DISCHARGE

Meters are most accurate when a near constant discharge is to be measured. Considerable error can be introduced by varying the flow rate greatly or quickly. Registration accuracies may vary from 97 to 102 percent of true measurement as a result of continually varying the flow over a significant range during the measurement period. The greatest error always occurs for low flows (at the lower end of the meter capacity scale). Propeller meters are always most dependable and accurate when used in uniformly flowing clean water and a closed system.

EFFECT OF TURNOUT DESIGN

The exact position of the meter in the turnout and the arrangement of the turnout are responsible for sizable differences in meter registration. Since the relative location of the meter with respect to the entering flow can vary, the intercepted velocity profile can vary and different meter registrations can occur. Figure 38 shows the range of accuracies that exist for three different types of turnouts.



NOTES

- ∘ (a) Calibration with surge tank, straightening vanes, and 20 diameters pipe.
- Δ (b) Calibration with straightening vanes and 40 diameters pipe.
- □ (c) Calibration in model turnout, with 7 diameters pipe.

Meter with 6-inch propeller.

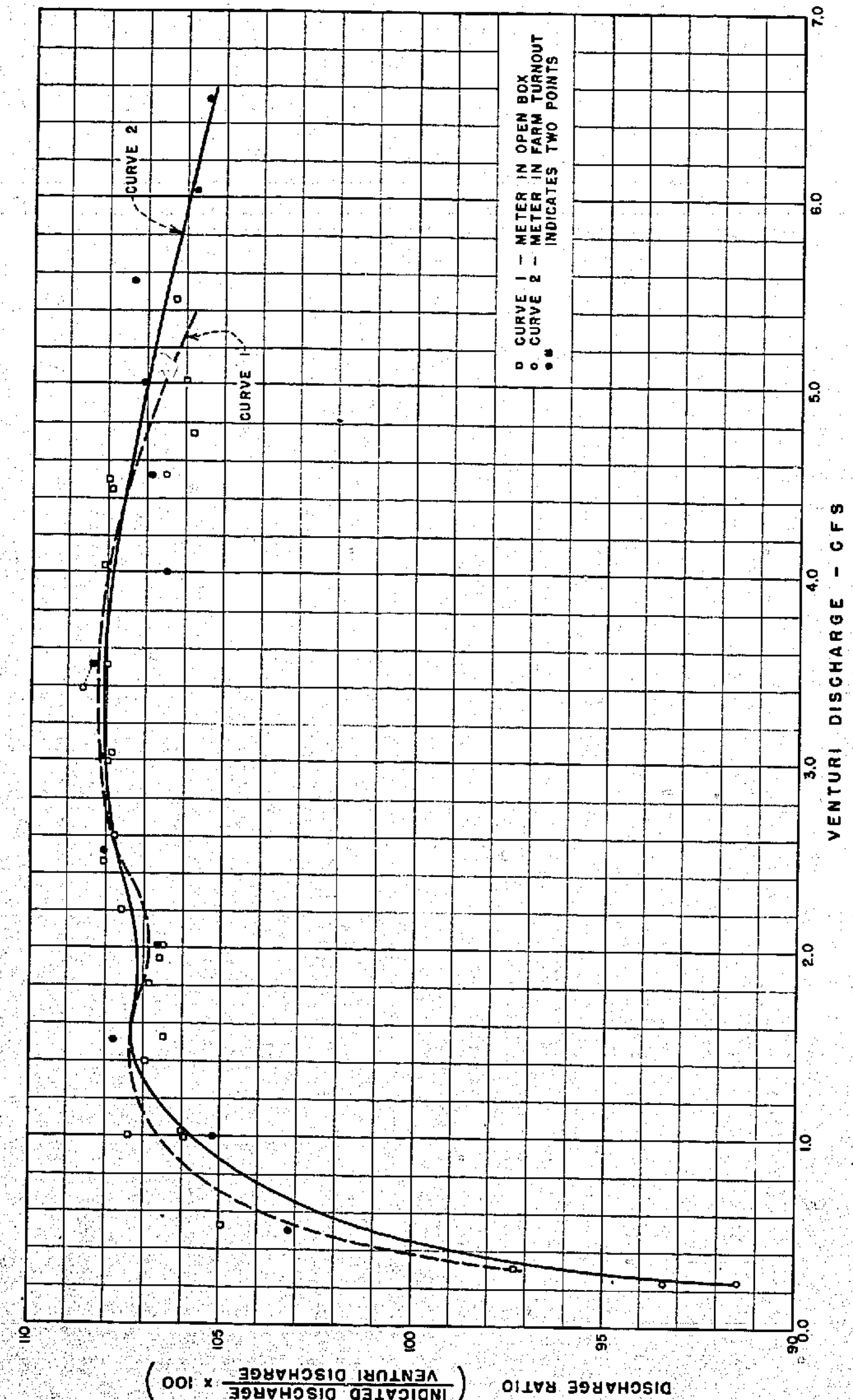
STUDY OF IRRIGATION TURNOUTS

EFFECT OF METER SETTING ON REGISTRATION ACCURACY

R9-73

Figure 38

Figure 39

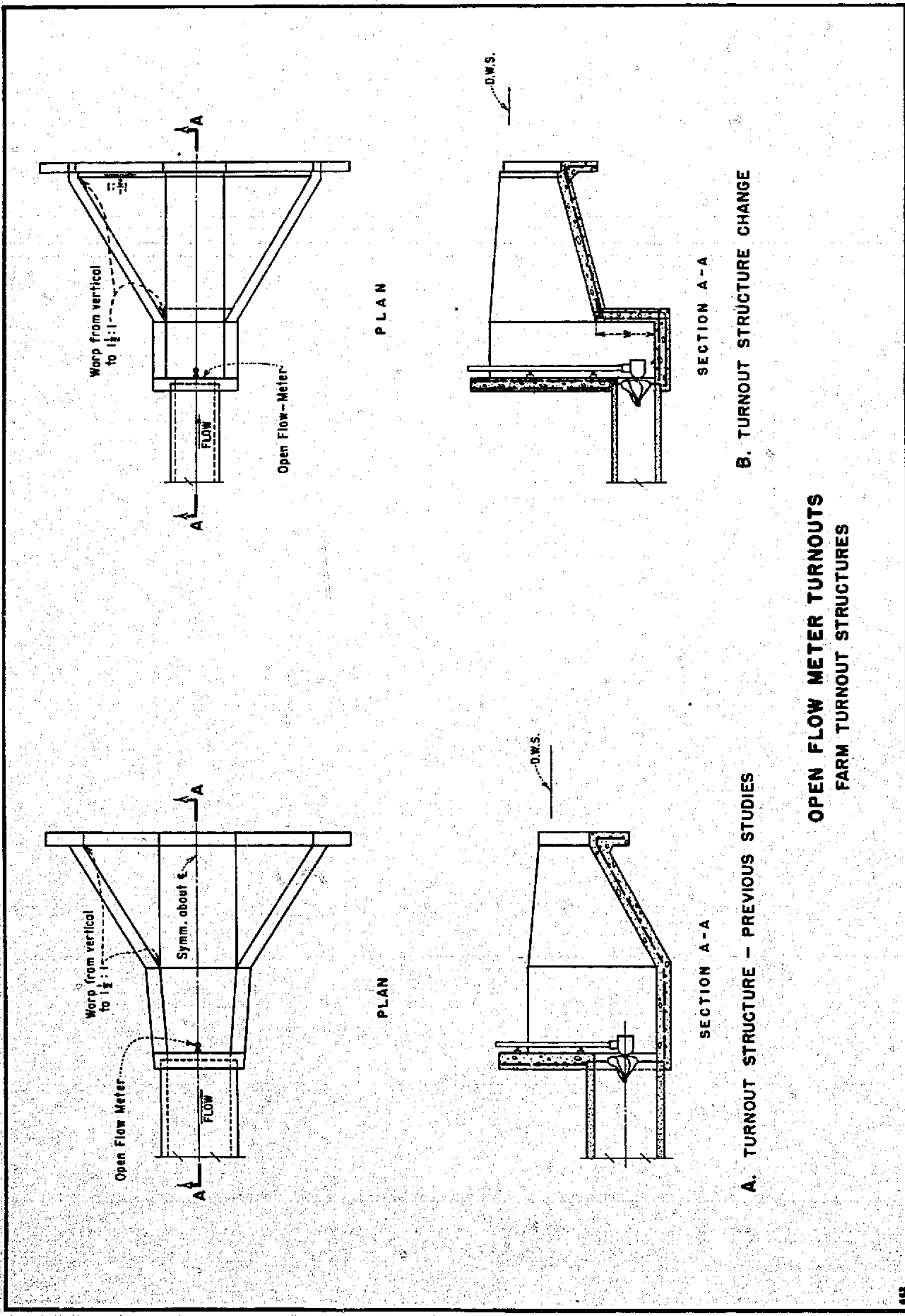


OPEN FLOW METER TURNOUTS

METER CALIBRATION CURVES – OPEN BOX AND FARM TURNOUT

462

Figure 40



OPEN FLOW METER TURNOUTS
FARM TURNOUT STRUCTURES

The geometry of the outlet box downstream from the flowmeter may also affect meter accuracy. If the outlet is sufficiently constricted to cause turbulence, boils and/or white water the meter registration may be affected.

Curve 1 of Figure 39 shows a meter calibration where the pipe discharges into a large open box that has no backwater or other effect on the meter except to keep it submerged. Curve 2 shows the calibration for the same meter using the outlet structure shown in the sketch of Figure 40. This outlet structure (shown to scale) is believed to be the smallest that can be built without significantly affecting the meter calibration. The vertical step is as close to the meter as is desirable. Larger outlet structures--those providing more clearance between the meter and the vertical step--would probably have less effect on the registration. More rapidly diverging walls (in plan) should be avoided since they tend to produce eddies over the meter and/or surging flow through the turnout. This has been observed as a continuously swinging indicator hand which follows the changing discharge through the meter. The surging may often be heard as well as seen. As previously discussed, large registration errors can occur when rapidly or continually changing discharges are being measured.

METER COSTS, MAINTENANCE

Propeller meter and maintenance costs are difficult to state in terms of dollars but some relative figures may be of value in making rough estimates. A propeller meter installation may cost two or three times more than a weir, depending on labor rates, and be somewhat less costly than, say, a Venturi meter. Two-thirds of the cost usually is for the meter (and other equipment such as screens, etc.) and one-third is for installation. The propeller meter will require more maintenance than a weir or a Venturi meter. Propeller meters may require continuous maintenance which may amount to as little as $10 to $25 per year or several times more. In some meters a single bearing may cost up to $75. To offset these costs, meters have paid for themselves in as short a time as 2 months, based on the value of the water they have saved. In other areas where water is relatively plentiful they have never paid out.

Hundreds of propeller meters bought for regular use have been taken out of service and stored in warehouses because of various troubles, either because the meter did not serve the purpose for which it was purchased, or because it became unreliable or inoperable after a period of service. Propeller bearing trouble is the most common problem and may be difficult to overcome except by means of a well-planned maintenance program. In districts where maintenance is accepted as inevitable, and where bearings and spare parts are stocked for immediate use, the maintenance costs and problems seem to be minimum.

In other districts where personnel are unfamiliar with meter mechanisms, and where spare parts are ordered on a one-at-a-time basis, the maintenance costs are high. In some cases users expect to replace bearings--they do not consider the need for bearing replacement to be a defect in the meter; in other areas a bearing failure is cause for permanently removing the meter from service. Experience has shown that maintenance costs can be reduced by establishing a regular maintenance program which includes lubrication and repair of meters; screen cleaning, repair, and replacement (about every 2 years); sand trap cleaning; and general maintenance of the turnout and its approaches. In a regular program many low-cost preventive measures can be made routine and thereby reduce the number of higher cost curative measures to be faced at a later time.

CHOICE OF METER SIZE

Many meters have been retired from service without ever having accomplished their original purpose, simply because a larger than necessary meter was purchased and the meter was not able to record the usual smaller daily flows. In attempting to use an existing turnout pipe (pipe sizes may not have any relationship to the discharge to be measured) a large meter was purchased to fit the existing pipe. The meter could then handle the maximum possible flow through the turnout but was too large to handle the small flows that were the usual daily requirement. In some districts it has been necessary to state the minimum flow that can be delivered and measured; the user is then expected to arrange his water use so that smaller discharges are not necessary. Care should be taken not only to match the meter to the pipe size but to match the meter to the proper discharge range. It may be necessary to reduce the turnout pipe size as a result, but the savings in purchasing a smaller meter might help to offset this cost. If possible, the meter size should be selected so that usual operation will occur in the midrange of the meter.

The velocity in the pipe should be above 1.5 feet per second for best performance. If sediment is present in the water, the velocity should be even higher to minimize the added friction effect produced by worn bearings. A meter that operates continually in the lowest range (or highest) will not be as accurate as one that operates in midrange.

NEW MEASURING DEVICES AND TECHNIQUES

VANE DEFLECTION METER

A portable vane flowmeter is on the market and, according to the manufacturer's claims, the meter is accurate and useful. The meter has been evaluated from comprehensive tests made under simulated field conditions in the Hydraulic Laboratory and the claims of the manufacturer were found to be quite truthful. The meter is indeed an accurate and useful device.

The portable deflector vane rests in permanent brackets mounted in a 6-foot-long ditch liner, either rectangular or trapezoidal in cross section, set in an earth ditch. Therefore, one meter head will service any number of ditches of the same general flow capacity having liners and brackets permanently installed. About 30 sizes of meter and ditch liner are available. Each meter handles a wide range of flows in a given size of ditch and automatically compensates for different combinations of velocity and depth. There is negligible loss of head caused by the ditch liner or meter. Installation is simple and the cost is reasonable, especially if several or more ditches can be served with one meter. Instantaneous discharges may be read and if an available special recorder is purchased, the total delivery may be recorded.

Since the meter works on the deflection principle, wind effects on the exposed portion of the meter can cause serious measurement errors unless precautions are taken. A wind break made from a piece of plywood was found to be effective in minimizing wind-caused errors.

Under ideal conditions the meter was found to be accurate to 1.6 percent, and to about 3 percent under less favorable conditions. Wind produced errors of up to 100 percent but simple precautions eliminated practically all of this error.

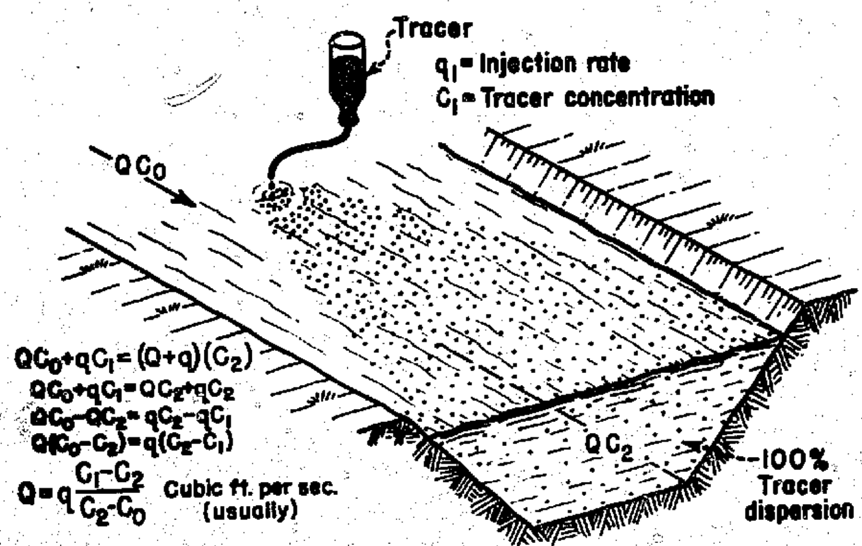
The meter is durable, well constructed, and should retain its original factory calibration indefinitely. Interchangeable calibrated scales are available from the manufacturer to indicate cfs, gpm, miners inches, acre-feet per day, etc.

The complete evaluation of the meter is available to Bureau employees in a restricted report, Hydraulic Laboratory Report No. R-Hyd-10, dated July 20, 1962. This may be obtained by writing to Technical and Foreign Services Branch, Bureau of Reclamation, Denver Federal Center, Denver, Colorado.

DILUTION METHOD

In making the usual water measurement it is necessary to measure head, velocity, cross-sectional area, depth, meter revolutions or some factor(s) that may be difficult to measure, because the measurement must be made in, on, over, or beneath the flowing water. One method of determining discharge that circumvents the need for making difficult mechanical measurements is the dilution method. In this method a substance in concentrated form is introduced into the flowing water and allowed to thoroughly mix. At a downstream station a sample is taken, and from the degree of dilution of the concentrate, the discharge is computed. Since only the quantity of water necessary to accomplish the dilution is involved, there is no need to measure velocity, depth, head, cross-sectional area or any of the other hydraulic factors usually considered in a discharge measurement.

Figure 41, illustrates schematically the use of the dilution method. A relatively large quantity of chemical or dye, called a tracer, is dissolved in a small quantity of water and placed in a bottle so that the tracer solution can be discharged into the flowing water at a known rate. A canal is illustrated but a pipe or pressure penstock could also have been shown. In either case the discharge to be measured is referred to as Q.



DILUTION METHOD OF DETERMINING DISCHARGE

Figure 41 PX-D-43065

Dilution method of measuring discharge

The concentration $\left(\frac{\text{weight of tracer}}{\text{weight of water}}\right)$ of tracer in the bottle is a ratio C_1. The rate of injection (cubic feet per second) is q_1. To account for any tracer which might already be in the upstream flow, the original tracer concentration will be called C_o. At the downstream station shown in the figure it will be assumed that thorough transverse mixing of the tracer with the flow has taken place. Since no flow has entered the canal during the mixing of the tracer and since no loss of flow has occurred, including seepage losses, the upstream Q is the same as the downstream Q plus the quantity of added tracer. The concentration of tracer at the downstream station is C_2.

The above conditions may be stated in a simple relationship--the canal discharge Q multiplied by the concentration of upstream tracer C_o, plus the injection rate q_1 multiplied by C_1 the tracer concentration in the bottle, are equal to the quantity Q plus q_1 multiplied by the concentration C_2 at the downstream station, or

$$QC_o + q_1C_1 = (Q + q_1)\ C_2$$

or

$$QC_o + q_1C_1 = QC_2 + q_1C_2$$

Rearranging terms

$$QC_o - QC_2 = q_1C_2 - q_1C_1$$

Simplifying

$$Q(C_o - C_2) = q_1(C_2 - C_1)$$

or

$$Q = q_1 \frac{(C_2 - C_1)}{(C_o - C_2)} \qquad \ldots\ldots (8)$$

In effect, Equation (8) states that the discharge in the canal (pipe conduit or other) may be obtained by multiplying the injection rate of the tracer by a ratio obtained from three concentration values. There is no need to know or measure canal velocity, depth, cross section or any of the usual hydraulic elements. A thorough understanding and realization of these facts is the key to understanding the dilution method of measuring discharges.

Many different substances have been used in dilution tests. Chemicals, including ordinary salt, have been used and the dilution has been evaluated by the change in the ability of the flowing water to conduct an electric current. Or chemicals such as sodium dichromate have been utilized and the quantity in a sample determined by means of chemical analysis or flame photometer determination. Sodium dichromate has been recommended for use in many waters because it is stable and chromium ions do not ordinarily appear in natural waters. Dyes have been used to color the water and colorimeters and comparators have established the dilutions by comparing the samples with standard concentrated or dilute solutions.

Fluoresein, Rhodamine B, or Pontacil Pink dyes have been used. The latter two are very stable, do not occur in nature, and can be simply and accurately detected by means of a fluorometer in very low concentrations (several parts per billion). New dyes are being developed to fill the needs of dilution testing. Rhodamine WT (water tracer) is one of the newer dyes found to give better response in a fluorometer.

Radioisotopes are also being used in dilution tests and the degree of dilution is determined by counting the gamma ray emissions from the diluted isotope solution (the downstream flow) using Geiger counters or scintillation counters. Since the counters can at best account for only the emissions in the sphere of influence surrounding the counter device, and since the understanding of the radioisotope method involves complete comprehension of the laws of probability and of mathematical and physical derivations beyond the scope of this paper, no attempt will be made here to explain directly the theory behind the radioisotope method of making discharge measurements. However, considerable comprehension of the method may be had by understanding the principles involved in making a dilution discharge determination.

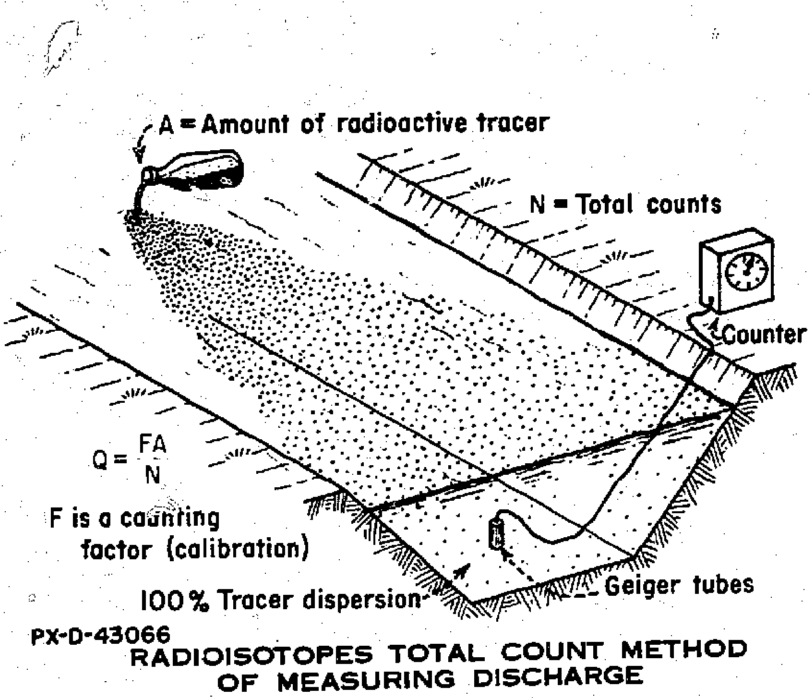


Figure 42 PX-D-43066

Radioisotope method of making a discharge measurement

RADIOISOTOPE METHOD

Radioisotopes may be used in place of a chemical or dye tracer and the same general procedures followed. Figure 42, "Radioisotopes Total Count Method of Measuring Discharge," shows all the elements necessary to make a radiosotope discharge measurement. The following explanation may help to explain the procedures. In the "pulse" or "total count" method a known amount of radioisotope A (corresponding to $C_1 v_1$ where v_1 is the volume of injected tracer) is introduced into the flow in a relatively short time. At the measuring station downstream where the radioisotope is throughly mixed, the concentration of the radioisotope tracer is determined from the gamma ray emissions detected and counted by Geiger-Muller or scintillation detectors. Where C_2 was a constant of concentration in the chemical dilution, Equation (8), the concentration of radioactivity in the pulse is a variable with respect to time. Thus, C_2 in the latter case must be measured with respect to time. In mathematical terms this may be written

$$Q = \frac{C_1 v_1}{\int C_2 dt} \div \frac{A}{\int C_2 dt} \quad \ldots\ldots (9)$$

In preparing for a discharge determination test, the Geiger or other counters must be calibrated, taking into account the exact conditions under which the counters will be used. The counters are submerged in a large container filled with a mixture of water and radioisotope of known concentration. The container must be large enough that any further increase in volume would not change the counting rate. Determination of the counting rate in this manner simulates the action of the counter in a canal where the container is, in effect, infinite in size.

Since the total number of gamma rays (counts) N, counted during the passage of a pulse of tracer is

$$N = F \int C_2 dt \qquad \ldots\ldots (10)$$

where F is the calibration or correction coefficient for a specific counting system in a specific location substitution of (10) into (9) gives

$$Q = \frac{FA}{N}$$

in which

Q = volume per unit time (cubic feet per second)
F = counts per unit of radioactivity per unit volume per unit of time (counts per second) (millicuries per cubic feet)
A = total units of radioactivity to be introduced for each discharge measurement (millicuries)
N = total counts (number).

In preparing A, a sufficient quantity of isotope for the desired measurements (Gold-198--salts dissolved in aqua regia) is divided into parts known as aliquots, using a portable field laboratory containing a standardized counting system. The individual parts, A, are usually contained in 1-pint plastic bottles and are transported to the test site in numbers sufficient for the discharge measurements.

In making a measurement in a canal the bottles are handled with tongs about 3 feet long and the isotope is introduced into the canal by pouring onto the water surface. Sometimes the isotope is contained in glass bottles which are smashed in an impact device below the surface of the water. When possible the isotope is introduced at a point where turbulence can aid in the mixing, Figure 43. Turbulent water aids dispersion and thorough mixing of the radioisotope with the flowing water.

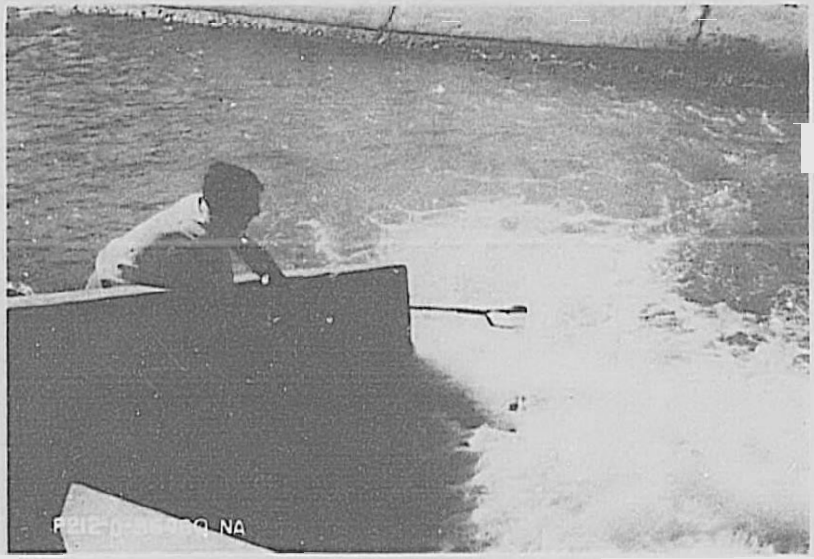


Figure 43

Turbulent water aids dispersion and mixing of the radioisotopes and water

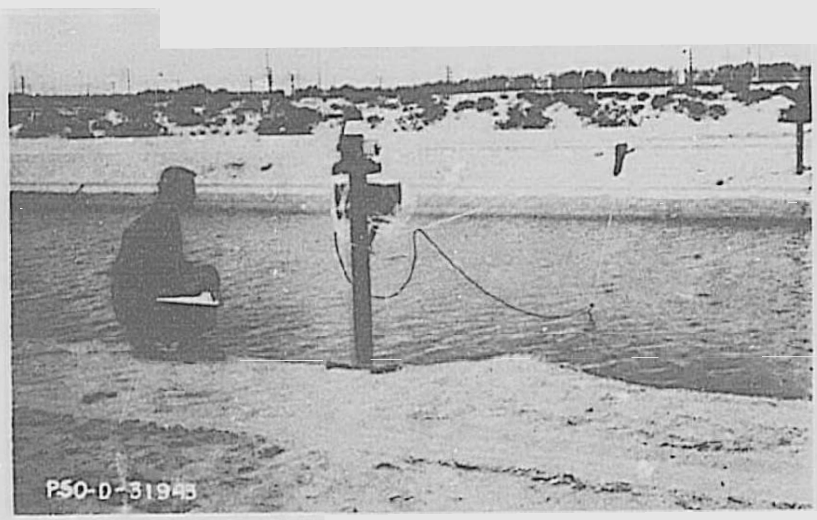


Figure 44

Radioisotope detector and counter installed for canal discharge measurement

Geiger or scintillation counters and portable decade scalers for totalizing the gamma ray emissions are used in the measurements, Figures 44 and 45.

To obtain a total count, N, the counts received from natural sources such as cosmic rays (usually called background) must be measured before and after a test and the background count subtracted from the overall totals indicated. Therefore, counting is usually started well before the time of arrival of the diluted isotope and is continued long after the isotope has departed. Recording of the counting is not usually stopped until the background has receded to pretest levels.

Thorough mixing of the isotope with the flowing water is of primary importance in obtaining accurate measurements. In canals it is very difficult to obtain sufficient mixing because of the lack of turbulence, a characteristic of a well-designed canal. The isotope tends to string out in a long line rather than extend itself laterally. Studies are presently being made (throughout the world) to determine the length of canal or river needed to obtain satisfactory mixing. Equations are being derived and investigated to determine whether it is possible to predict the lengths required for a given set of initial conditions. Less difficulty in obtaining satisfactory mixing has been experienced in natural streams and in pipelines because of the higher velocities which usually prevail and the correspondingly greater intensity of turbulence. It may be necessary to use turbulence inducers when working in canals, particularly if they are checked up, or are discharging at less than maximum capacity. In a typical canal discharging 300 to 600 cfs the required mixing length in a recent set of tests was found to be less than 2,000 feet. In other canals insufficient mixing has occurred in a length of 5,000 feet. It would be desirable to reduce the required length as much as possible in order to increase the number of possible measurement sites and also because absorption of isotope by materials such as earth, concrete or sediment might adversely affect the accuracy of the measurement.

The radioisotope used in Bureau of Reclamation tests is Gold-198. This isotope was selected because it has a half-life of 64 hours, can be detected in weak concentrations, is cheap, and is easy to obtain. In the concentrations used, the isotope is practically harmless after it is introduced into the flow. In case of an accident only a few days would be required to dissipate half of its radioactivity (half-life) and in another few days half of the remaining activity would be dissipated. No danger to human, animal, or fish life exists, however, since the radioactive material is handled in such a way (only by licensed personnel) that only small quantities are concentrated in any one area. Concurrence of all public health agencies is always obtained before any tests are run.

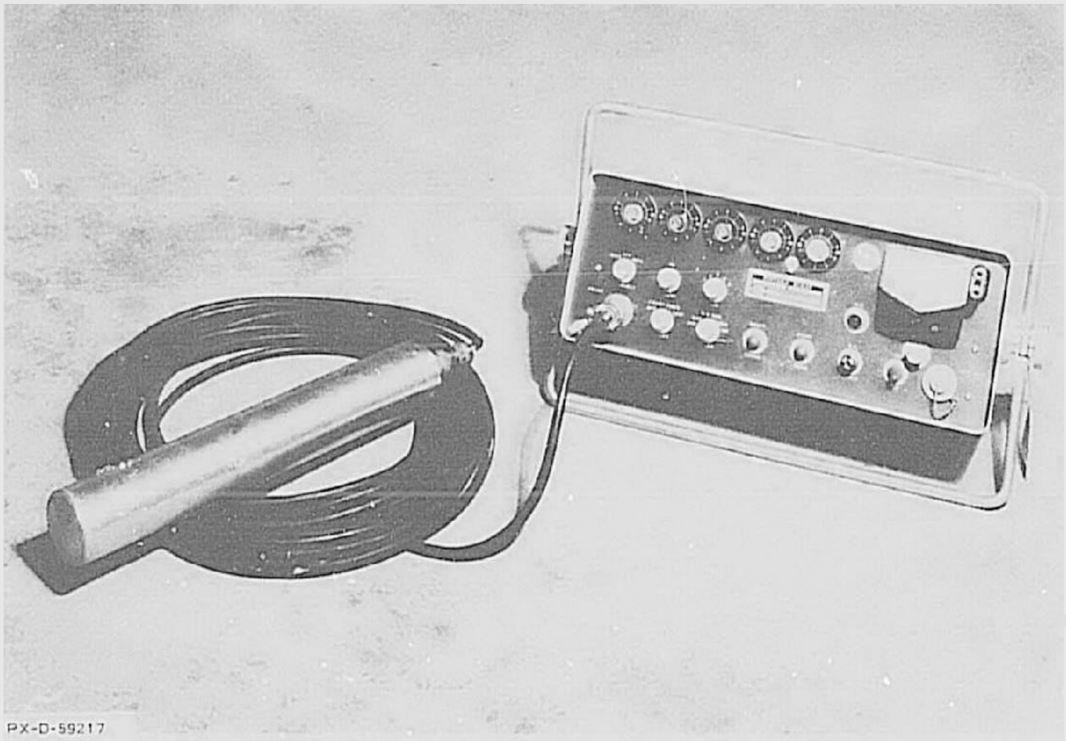


Figure 45

Radioisotope detection and counting system used in Bureau of Reclamation tests

Other isotopes which have been used by other investigators and which could be used in discharge determination tests are: iodine-131 (15¢ per m/c), sodium-24 (12¢ per m/c), and bromine-82 (12¢ per m/c). These have not been used by the Bureau because of their short half-life or their less desirable health effects. The minimum cost of Gold-198 is about 10 cents per millicurie. In terms of a field test, the cost of Gold-198 is about $1.00 per 1,000 cfs to be measured. The quantity of isotope used is based on obtaining statistical counting accuracy within about plus or minus 1 percent. (If there are too few emissions to count, the background count becomes a significant part of the total count; also, the lower the total count the greater the chance for error.)

Greater possible statistical accuracy, which may or may not always be justified, could raise the cost to as much as $5 per 1,000 cfs of discharge. Considering practical limits, very little gain in possible statistical accuracy can be expected when more than $2 worth of Gold-198 is used per 1,000 cfs.

The accuracy of the results achieved in using isotopes to measure canal discharges in over 100 tests has, in general, been encouraging. If thorough transverse mixing of the isotope with the flow can be assured, measurement accuracy to within plus or minus 1 percent can be achieved. Comparisons of isotope discharge determinations with "operational" discharges, current meter measurements, or other means, have been used to evaluate the isotope tests. It is believed that the isotope method shows real promise of becoming a routine method having guaranteed accuracy limits which are comparable to other devices and methods. Figure 44 shows the detection and counting equipment in operation on a medium-size canal.

The greatest deterrent to obtaining consistently accurate measurements has been the inability to obtain thorough transverse mixing. Work on this phase of the problem is continuing, both as a research subject in the laboratory and as a test problem in the field. Tests on devices to produce or introduce turbulence into canal flows have not been made, mainly because every attempt is being made to keep the measuring procedure and equipment as simple as possible. However, it may be necessary to provide turbulence-inducing injectors which utilize an external source of energy, such as an air blast, to start the eddy action which promotes transverse mixing. If this can be done with simple and reliable equipment, the problem of adequate mixing will be greatly simplified.

Considering all the factors involved in making any type of discharge determination, it may be seen that the radioisotope method shows promise of being developed into a simple procedure, requiring a minimum of time to execute, and providing maximum possibilities for being accurate. Eventually the radioisotope discharge determination development program will be expanded and injection equipment will be developed to introduce radioisotopes into pipelines or penstocks flowing under high heads. Pipe capacities, or turbine or pump efficiencies could be determined as an ultimate goal of the radioisotope discharge method development program.

ACOUSTIC FLOWMETER

Commercial development of acoustic flowmeters has progressed rapidly since the practical nature of the method was realized about 1950. Systems for small pipes (up to 10 inches) have been available but in 1964 a system was installed and experimentally evaluated in a 24-foot-diameter steel penstock at Oahe Dam on the Missouri River with satisfactory results. An open channel acoustic flowmeter has been installed in the Delta-Mendota Canal of the Central Valley Project by Government agencies. Research and development of the system shows promise that the principle utilized in the pipe systems may be satisfactorily applied to most open channel discharge measurements. Comprehensive studies of both open-channel and closed-conduit types of acoustic meters are continuing with the expectation that the method can be applied to measuring large discharges in both open-channel and closed-conduit systems.

Acoustic flowmeter systems are expected to offer, eventually, a relatively inexpensive, rapid, and reliable method of measuring discharges in large pressure conduits and in large canals. The system uses the principle that the difference in time of arrival of two acoustic (pressure) pulses traveling in opposite directions through the water can be related to the water velocity. Figure 46 shows a schematic arrangement of an acoustic meter installation in a pipeline. In one direction the waterflow velocity increases the speed of the acoustic pulse $(C + V_w)$ while in the other direction the flow velocity delays the arrival of the pulse $(C - V_w)$. Acoustic transducers are used to transmit and receive the acoustic pressure energy along oblique upstream and downstream paths in the channel or conduit. The accuracy of the system depends on positioning the transducers to obtain a true average velocity in the pipe or canal. Several pairs of transducers may therefore be required to obtain the true average velocity if the velocity profile in the conduit is not symmetrical about the centerline of the conduit. In an open channel the transducers must be raised or lowered as the flow depth changes to compensate for a changing velocity profile.

Acoustic systems are being developed that can operate over a range of conduit diameters or open-channel path lengths. The power supply, transducers, and velocity read-out components can be made compact for ease of installation in the structure. Power requirements have been reduced so that a 110-volt supply source can be used. There are indications that velocities can be measured with 1 percent accuracy. Consequently, discharges may be determined with considerable accuracy depending on the velocity profile at the measuring station.

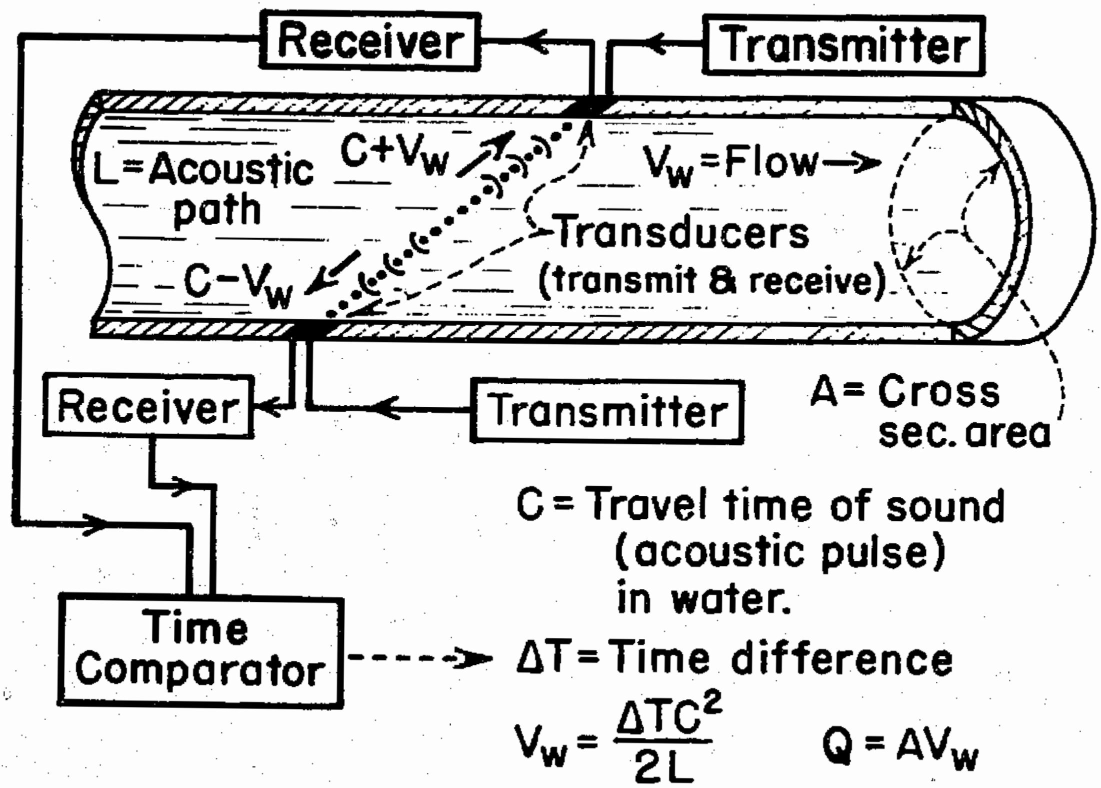


Figure 46

LASER FLOWMETER

Laser (light amplification by stimulated emission radiation) beams have been used for studying the turbulent characteristics of flowing liquids and for determining the velocity of fluid flow. As in the acoustic meter the Doppler principle is used in comparing the shift in frequencies. The flowing water scatters part of a beam of light (laser) directed through it. By comparing the frequencies of the scattered and unscattered rays, collected in receiving lenses on the opposite side of the stream, the velocity of the water (hence the discharge) can be calculated. In laboratory experiments the instrument has measured fluid flows as slow as a fraction of an inch per second and as fast as 5,000 feet per second. The device is a valuable research tool but should be considered a possible future water-measuring device.

MAGNETIC FLOWMETER

Magnetic flowmeters are used commercially in chemical and process industries to measure and proportion liquids used in manufactured products. Recently with the increased cost of irrigation, municipal and industrial water, magnetic flowmeters are being considered for use as turnout measuring devices.

The operating principle of the meter is based on Faraday's law of induction, in that the voltage induced across any conductor moving at right angles through a magnetic field is proportional to the speed of the conductor. The principle is the same as used in direct- and alternating-current generators.

In the flowmeter, water flows through a nonmagnetic tube which is installed as a portion of the pipeline, and surrounded by a magnetic field, produced by electronic circuitry. The flowing water induces an alternating-current voltage between two electrodes located in the tube walls. The voltage produced is proportional to the speed of the water in the tube and is used to indicate and record the rate or volume of flow. These meters are capable of measuring velocities of up to 30 feet per second through the tube.

Magnetic flowmeters are supplied in sizes ranging from an inch or less to several feet (diameter of the nonmagnetic flow tube). Development of the meter is continuing and considerations are being given to installing electromagnetic coils and electrodes on the inside surfaces of existing concrete pipes to form large flowmeters for use in pumping plants or distribution systems.

Accuracy of the smaller flowmeters has been shown by calibration to be relatively high. For velocities less than 1-fps accuracy is plus or minus 3-4 percent; 1-3 fps, plus or minus 2 percent; 3-30 fps, plus or minus 1 percent. Calculations indicate that the accuracy of large meters should also be high and attempts are being made to establish the accuracy of meters having diameters of 10 to 15 feet. The discharge measurement range of any particular meter can be extended by suitable switching in the electronic controls, to as much as 30 to 1, a definite advantage in most installations. Other types of meters usually have a 10 to 1 range. A minimum velocity of about 0.5 fps through the flow tube is desirable for producing a sufficiently high voltage for discharge measurement. The meter will operate below the 0.5-fps range but with decreased accuracy.

Contrary to most meters little effect on the accuracy of the flowmeter is caused by elbows, gates or valves located within a few pipe diameters upstream from the entrance to the meter flow tube, or near the outlet of the tube. The head loss in the system caused by the meter is negligible; is no more than the loss produced by the same length of pipe of similar roughness.

MEASURING, INDICATING, AND FLOW CONTROLLING METERS

Rising labor rates and the increasing value of water has accelerated the trend toward automated distribution systems and has helped make pipe delivery systems competitive with open ditches in some areas. Pipe systems often operated under high head and need a pressure-reducing device, control valve, and flowmeter to make them practical. Automatic and/or remote control, along with indicating and recording devices on the flowmeter are also desirable. Devices that eliminate, reduce, or combine these essential functions will reduce the cost of a turnout and its maintenance, and will find ready acceptance if they are dependable.

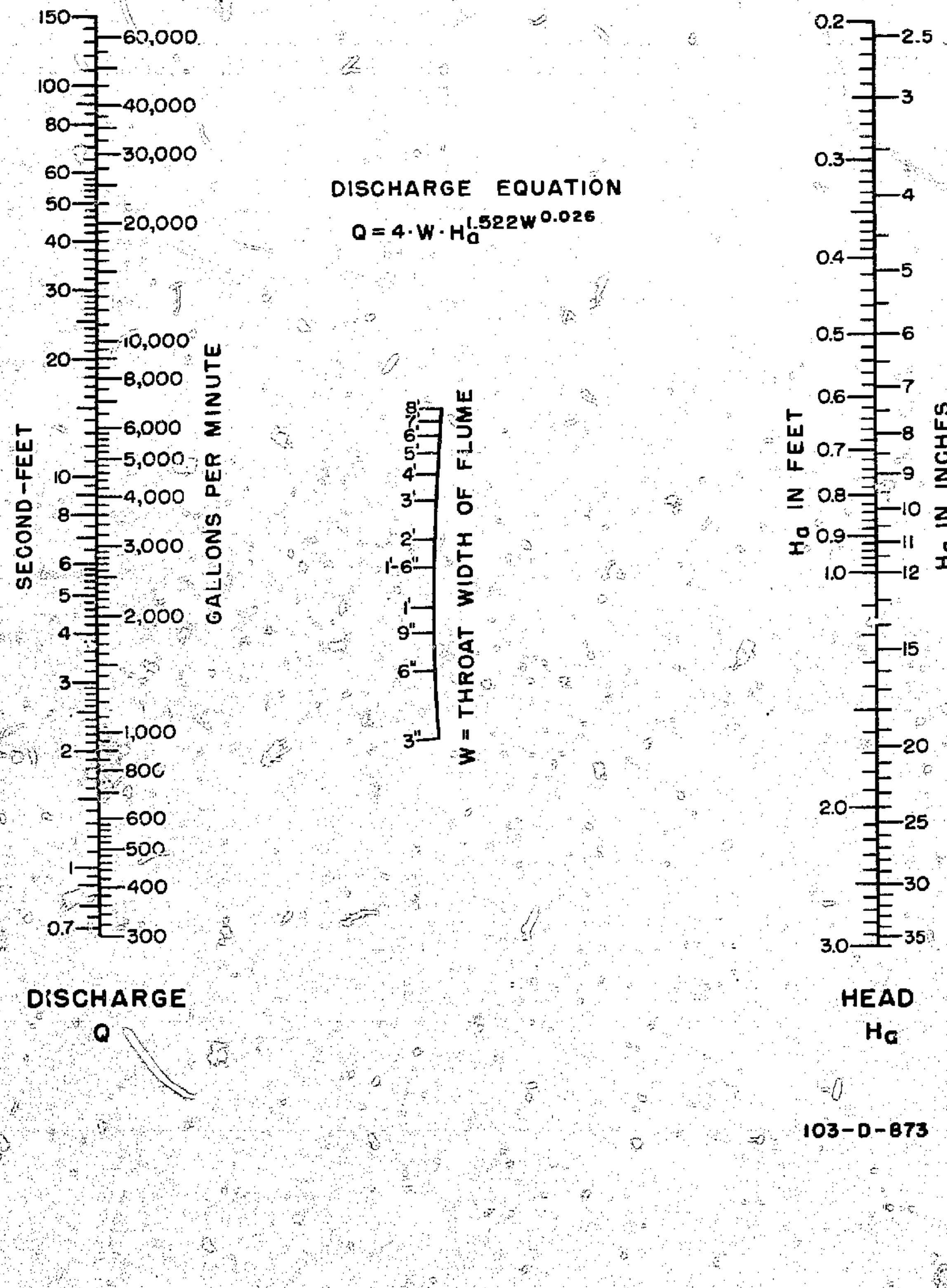
Two devices, utilizing the tapered-tube variable area principle, have been investigated in the laboratory and one has undergone short duration field tests in poor quality water (dissolved salts and silt). The meters indicated a greater than usual head loss and some technical difficulties that limited their expected performance. Both had a so-called "float" (actually a weight) which moves in the tapered tube to provide more flow area when the discharge is increased. The position of the float in the tube is used to indicate the quantity of water flowing. In one meter a rod attached to the float was used to actuate a punch which punched holes in paper tape to record the flow. The tape could be put through decoding and billing machines to simplify customer paying procedures.

In the other meter a pilot device controlled a large main valve which maintained a constant discharge (limited to 1 to 3 cfs) for a head range of from 7 to 175 feet, while indicating and totalizing the discharge. Pilot control could be manual or by remote wire or wireless signal.

These valves, and others expected to be designed and promoted, are expensive by ordinary standards but are certain to find a market in thriving agricultural areas. Although they are new and relatively untested under adverse field conditions or for long periods of time, they are being installed in some areas. More and more of these devices are certain to be developed as automation continues. There is no technical reason why these devices cannot be made to perform satisfactorily.

PARSHALL FLUMES

NOMOGRAPH FOR FREE FLOW DISCHARGE
THROUGH 3-INCH TO 8-FOOT
PARSHALL FLUMES



PARSHALL FLUMES

The Parshall flume is an open-channel-type measuring device containing a specially shaped constricted throat section, and was developed for use in a stream, canal, ditch, or other open flow way, to measure the rate of flow of water. Standard flumes are available over a wide range of sizes and each will measure discharges accurately within the limits of general irrigation requirements; usually within plus or minus 2 percent accuracy for free flows and plus or minus 5 percent for submerged flows.

Each flume is capable of measuring a wide range of discharges:

(a) without excessive loss of head (less head loss than an orifice or weir)

(b) without excessive amounts of sediment depositing in the structure

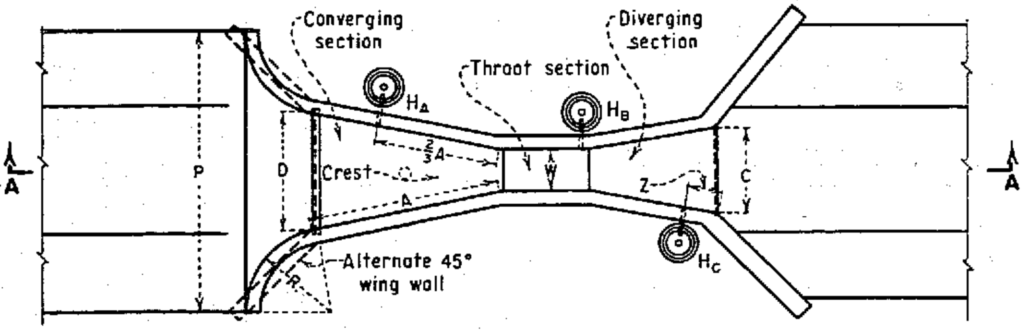
(c) without backwater or submergence effects nullifying the accuracy of measurement

(d) without a deep and wide upstream pool to reduce the velocity of approach

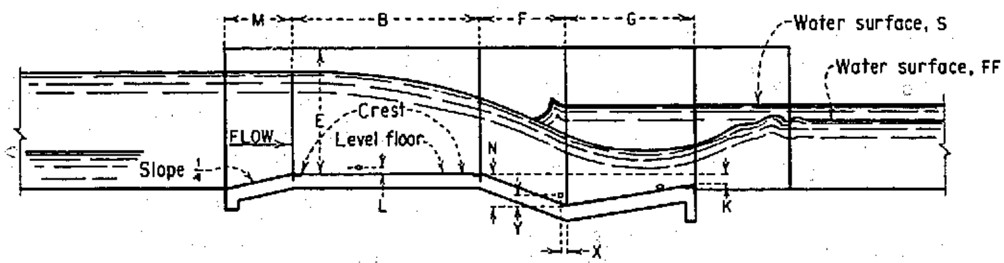
The constricted throat of the flume, a modified form of Venturi profile, produces a differential head, between the upstream and downstream water surfaces, that can be related to discharge, Figure 1. The raised crest and the other configurations of the flume bottom give the Parshall flume the ability to withstand a relatively high degree of submergence without reducing the rate of flow through the flume. In other words, the tailwater (H_b) may rise to at least 50 percent of the upstream depth (H_a), shown in Figure 1 without causing a reduction in discharge. (The 50 percent figure assumes that both H_a and H_b are measured from the crest elevation to the water surface as in Figure 3.)

The converging upstream portion of the flume accelerates the entering flow and almost eliminates the depositing of sediment in critical parts of the flume. Sediment deposits tend to reduce the accuracy of flow measurements. High velocity of approach, which often is a detrimental factor in the operation of weirs, has little, if any, effect on the rate of discharge of the flume. The approaching flow should, however, be well distributed across the channel and should be relatively free of turbulence, eddies, and waves for accuracy of measurement.

Discharge through a Parshall flume can occur for two conditions of flow. The first, called free flow, occurs when there is insufficient backwater depth to reduce the discharge rate; water surface line FF in Section A-A of Figure 1 would not reduce the discharge.



PLAN



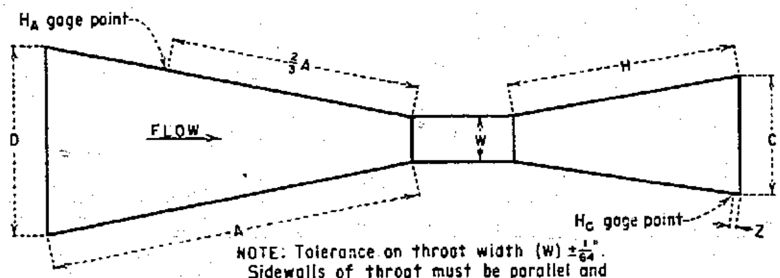
SECTION A-A

Plan and elevation of a concrete Parshall measuring flume. W, size of flume, in inches or feet; A, length of side wall of converging section; $\frac{2}{3}$A, distance back from end of crest to gage point; B, axial length of converging section; C, width of downstream end of flume; D, width of upstream end of flume; E, depth of flume; F, length of throat; G, length of diverging section; K, difference in elevation between lower end of flume and crest; M, length of approach floor; N, depth of depression in throat below crest; P, width between ends of curved wing walls; R, radius of curved wing wall; X, horizontal distance to H_B gage point from low point in throat; Y, vertical distance to H_B gage point from low point in throat. Water surface FF is free flow profile, S is submerged flow profile; Z, applies to 1 to 3-inch flumes, L, $1\frac{1}{2}$ inches, flume sizes 1-0 to 8-0 feet.

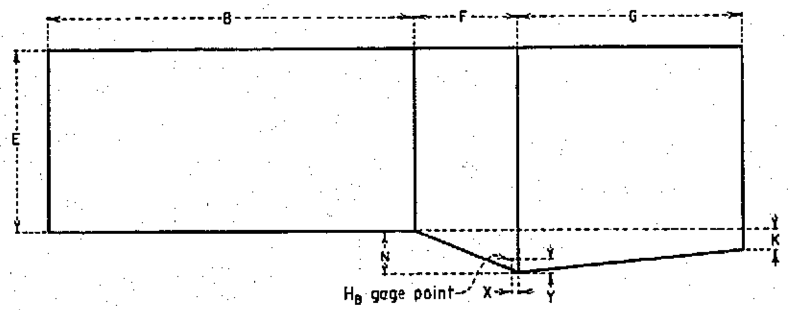
DIMENSIONS AND CAPACITIES OF THE PARSHALL MEASURING FLUME, FOR THROAT WIDTHS, W, FROM 3 INCHES TO 8 FEET

W	A	$\frac{2}{3}$A	B	C	D	E	F	G	K	N	R	M	P	X	Y	FREE-FLOW CAPACITY MINIMUM	FREE-FLOW CAPACITY MAXIMUM
Ft. in.	Ft. in.	Ft. in.	Ft. in.	Ft. in.	Ft. in.	Ft. in.	Ft. in.	Ft. in.	In.	In.	Ft. in.	Ft. in.	Ft. in.	In.	In.	Sec.-ft.	Sec.-ft.
0 3	1 $6\frac{3}{8}$	1 $\frac{1}{4}$	1 6	0 7	0 $10\frac{3}{16}$	2 0	0 6	1 0	1	$2\frac{1}{4}$	1 4	1 0	2 $6\frac{1}{4}$	1	$1\frac{1}{2}$	0.03	1.9
0 6	2 $\frac{7}{16}$	1 $4\frac{5}{16}$	2 0	1 $3\frac{1}{2}$	1 $3\frac{5}{8}$	2 0	1 0	2 0	3	$4\frac{1}{2}$	1 4	1 0	2 $11\frac{1}{2}$	2	3	.05	3.9
0 9	2 $10\frac{5}{8}$	1 $11\frac{1}{8}$	2 10	1 3	1 $10\frac{5}{8}$	2 6	1 0	1 6	3	$4\frac{1}{2}$	1 4	1 0	3 $6\frac{1}{2}$	2	3	.09	8.9
1 0	4 6	3 0	4 $4\frac{7}{8}$	2 0	2 $9\frac{1}{4}$	3 0	2 0	3 0	3	9	1 8	1 3	4 $10\frac{3}{4}$	2	3	.11	16.1
1 6	4 9	3 2	4 $7\frac{7}{8}$	2 6	3 $4\frac{3}{8}$	3 0	2 0	3 0	3	9	1 8	1 3	5 6	2	3	.15	24.6
2 0	5 0	3 4	4 $10\frac{7}{8}$	3 0	3 $11\frac{1}{2}$	3 0	2 0	3 0	3	9	1 8	1 3	6 1	2	3	.42	33.1
3 0	5 6	3 8	5 $4\frac{3}{4}$	4 0	5 $1\frac{7}{8}$	3 0	2 0	3 0	3	9	1 8	1 3	7 $3\frac{1}{2}$	2	3	.61	50.4
4 0	6 0	4 0	5 $10\frac{5}{8}$	5 0	6 $4\frac{1}{4}$	3 0	2 0	3 0	3	9	2 0	1 6	8 $10\frac{3}{4}$	2	3	1.3	67.9
5 0	6 6	4 4	6 $4\frac{1}{2}$	6 0	7 $6\frac{5}{8}$	3 0	2 0	3 0	3	9	2 0	1 6	10 $1\frac{1}{4}$	2	3	1.6	85.6
6 0	7 0	4 8	6 $10\frac{3}{8}$	7 0	8 9	3 0	2 0	3 0	3	9	2 0	1 6	11 $3\frac{1}{2}$	2	3	2.6	103.5
7 0	7 6	5 0	7 $4\frac{1}{4}$	8 0	9 $11\frac{3}{8}$	3 0	2 0	3 0	3	9	2 0	1 6	12 6	2	3	3.0	121.4
8 0	8 0	5 4	7 $10\frac{1}{8}$	9 0	11 $1\frac{3}{4}$	3 0	2 0	3 0	3	9	2 0	1 6	13 $8\frac{1}{4}$	2	3	3.5	139.5

PARSHALL FLUMES OF SMALL SIZE



PLAN



ELEVATION

DIMENSIONS OF 1, 2 AND 3-INCH STANDARD PARSHALL MEASURING FLUME, INCHES

W	A	$\frac{2}{3}A$	B	C	D	E	F	G	H	K	N	X	Y	Z
1	$14\frac{9}{32}$	$9\frac{17}{32}$	14	$3\frac{21}{32}$	$6\frac{19}{32}$	6–9	3	8	$8\frac{1}{8}$	$\frac{3}{4}$	$1\frac{1}{8}$	$\frac{5}{16}$	$\frac{1}{2}$	$\frac{1}{8}$
2	$16\frac{5}{16}$	$10\frac{7}{8}$	16	$5\frac{5}{16}$	$8\frac{13}{32}$	6–10	$4\frac{1}{2}$	10	$10\frac{1}{8}$	$\frac{7}{8}$	$1\frac{11}{16}$	$\frac{5}{8}$	1	$\frac{1}{4}$
3	$18\frac{3}{8}$	$12\frac{1}{4}$	18	7	$10\frac{3}{16}$	12–18	6	12	$12\frac{3}{32}$	1	$2\frac{1}{4}$	1	$1\frac{1}{2}$	$\frac{1}{2}$

DIMENSIONS FOR PARSHALL MEASURING FLUMES OF LARGE SIZE

SIZE (THROAT WIDTH)	FREE-FLOW CAPACITY		AXIAL LENGTH			WIDTH		WALL DEPTH CONVERGING SECTION	VERTICAL DISTANCE BELOW CREST		H_A GAGE DISTANCE (NOT AXIAL)*
	MAX.	MIN.	CONVERG-ING	THROAT	DIVERGING	UPSTREAM END	DOWNSTREAM END		DIP AT THROAT	LOWER END FLUME	
Feet	Sec. ft.	Sec. ft.	Feet	Feet	Feet	Feet	Feet	Feet	Feet	Inches	Feet
10	200	6	14	3	6	15' 7.25"	12' 0"	4	1' 1.5"	6	6'0"
12	350	8	16	3	8	18' 4.75"	14' 8"	5	1' 1.5"	6	6'8"
15	600	8	25	4	10	25' 0"	18' 4"	6	1'6"	9	7'8"
20	1000	10	25	6	12	30' 0"	24' 0"	7	2'3"	12	9'4"
25	1200	15	25	6	13	35' 0"	29' 4"	7	2'3"	12	11'0"
30	1500	15	26	6	14	40' 4.75"	34' 8"	7	2'3"	12	12'8"
40	2000	20	27	6	16	50' 9.5"	45' 4"	7	2'3"	12	16'0"
50	3000	25	27	6	20	60' 9.5"	56' 8"	7	2'3"	12	19'4"

NOTE: For all large sizes the H_B gage is located 12 inches upstream from, and 9 inches above, the floor at the downstream edge of throat.

* H_A gage distance is measured along flume wall, upstream from the crest line.

PARSHALL FLUMES OF "VERY SMALL" AND LARGE SIZE

The second condition, called submerged flow, occurs when the water surface downstream from the flume is sufficiently above the elevation of the crest of the flume to reduce the discharge. Water surface line 5 in Figure 1 would reduce the free flow discharge rate.

For free flow only the head, H_a, at the upstream gage location is needed to determine the discharge from the standard tables. The free flow range includes some of the range which might at first be considered the submerged flow range, because Parshall flumes can tolerate 50 to 80 percent submergence before the free flow rate is reduced to a measurable degree. For submerged flows (when submergence is greater than 50 to 80 percent, depending on flume size), both upstream and downstream heads, H_a and H_b, are needed to determine the discharge.

A distinct advantage of the Parshall flume is its ability to function as a single head device over a wide operating range with minimum loss of head. This loss is only about one-fourth of that needed to operate a weir having the same crest length. Another advantage is that the velocity of approach is automatically controlled if the correct size of flume is chosen and the flume is used as it should be as an "in-line" structure. Parshall flumes are widely used in irrigation systems because there is no easy way for the unscrupulous to alter the flume dimensions or change the flow channel to obtain an unfair proportion of the water. Also, it is easy for one water user to check his own water delivery against those of neighboring users.

The main disadvantages of a Parshall flume are (1) it may not be an accurate measuring device when used in a close-coupled combination structure consisting of turnout, control, and measuring device, and (2) it is (usually) more expensive than a weir or submerged orifice.

Parshall flume sizes are stated in terms of the throat width, W, and are available from the 1-inch size for discharges as small as 0.01 cfs* to the 50-foot size for discharges up to 3,000 cfs, Figure 1.

Parshall flumes may be constructed of wood, concrete, galvanized sheet metal, or other materials. Large flumes may be constructed piece by piece on the site, whereas smaller flumes may be purchased as prefabricated structures to be installed in one piece. Construction methods and materials are discussed in Items 1, 2, and 3 listed in "Acknowledgments" at the end of this section. Some flumes are available as light weight shells which are made rigid and immobile by placing concrete against the outside of the walls and beneath the bottom. The larger sizes are used in rivers and large streams; the smaller ones for measuring farm deliveries or for row requirements in the farmer's field.

*0.01 cfs is approximately 5 gallons per minute.

DEVELOPMENT OF FLUME

Experiments on a device called the Venturi flume were started in 1915 at the Colorado Agricultural Station, Colorado State University, and further experimentation was continued through 1920. The flume had a converging entrance and diverging outlet, joined by a parallel-walled throat. The walls were either vertical or inclined outward, and the floor was flat and level. In 1922 the late Mr. Ralph Parshall proposed some radical changes--the angles of convergence and divergence were altered, and the lengths of the various parts were changed. Also, the crest was elevated, and the floor in the throat was sloped downward to form a fixed control in the structure. The walls were made vertical and the floor of the diverging section was inclined upward. (The dip in the floor is the reason that discharges are not reduced by mild backwater effects.) The flumes were developed in various sizes with the larger sizes resulting from field tests on flumes constructed and calibrated in the Arkansas River Valley between the years 1926 and 1930. This perfected device was named the Parshall Measuring Flume by the Irrigation Committee of the American Society of Civil Engineers. The flumes are not patented nor are the discharge tables copyrighted.

Care must be taken to construct the flumes according to the structural dimensions given for each flume because the flumes are not geometrically similar. For example, it cannot be assumed that a length dimension in the 4-foot flume will be three times as great in the 12-foot flume.

The various flume sizes have been classified (in this discourse) according to throat width, W, into three main groups for convenience in discussing, selecting sizes, setting in the field, and determining discharges. These groupings are the "very small," "small," and "large," as shown in Figure 1. The flumes cover a wide range of discharges and have overlapping capacities to provide wide latitude in the selection of sizes when considering submergence effects. The flumes are capable of handling a range of discharges from 0.01 to 3,000 cfs with submergence effects up to 95 percent. Each of the flumes in Figure 1 is a "standard" device and has been calibrated for the entire range of discharges shown in the curves and tables. The flumes can be relied upon to provide reasonably accurate measurements if the standard dimensions are attained during construction, and if the flume is "set" (placed in the proper vertical position) and operated according to the recommended procedures.

PRINCIPLES OF OPERATION

FREE FLOW

In free flow the quantity of water a flume of width W will pass is solely dependent on the depth of water at the gaging point, H_a, in the converging section, Figure 2. Free flow conditions in the flume are similar to those that occur at a weir or spillway crest. Water passing over the crest is not impeded or otherwise affected by downstream conditions.

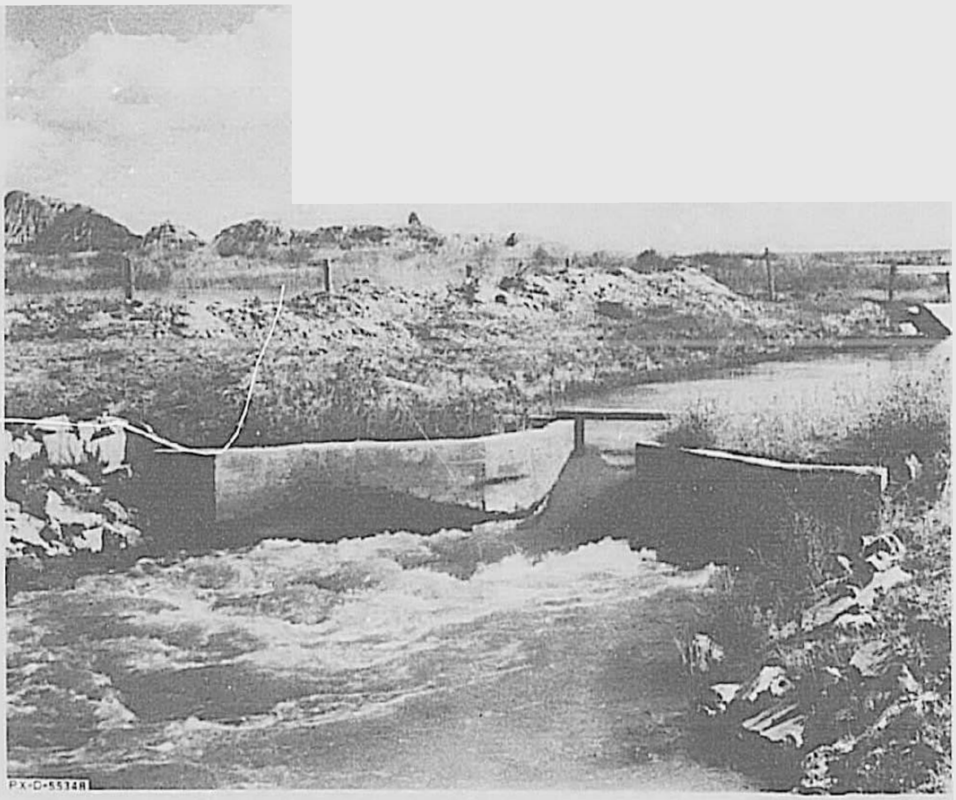
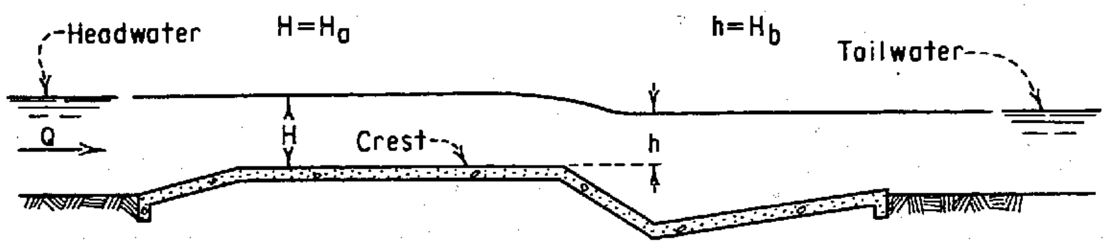


Figure 2. Four-foot Parshall flume discharging 62 cfs under free flow conditions. Downstream scour protection is required with this height of fall.

The lower curve, labeled free flow in Figure 3 shows a typical relationship between discharge, Q, and head H_a, for a Parshall flume discharging in the free flow range.



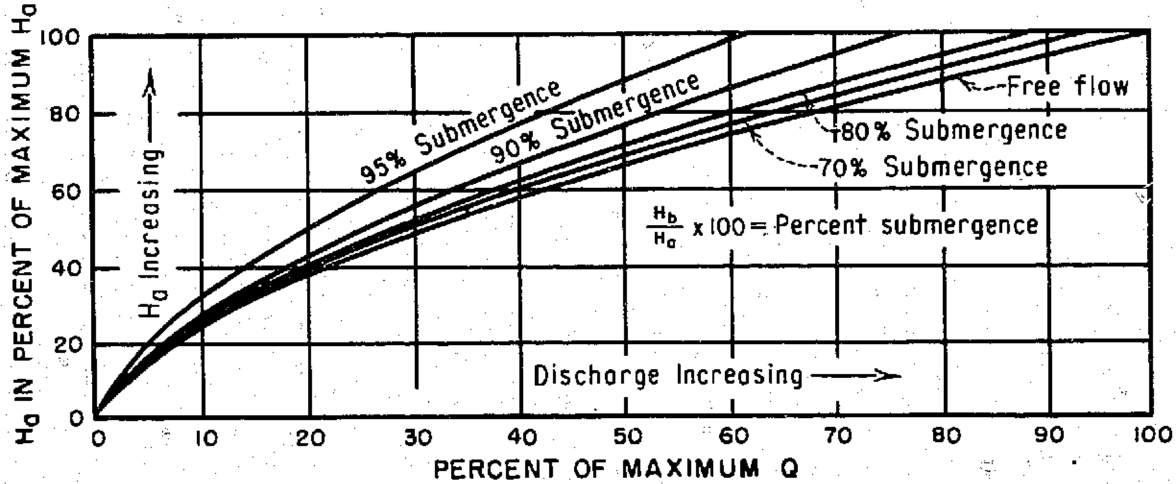


Figure 3

Typical discharge curves for Parshall flumes with free flow and with submerged conditions

SUBMERGED FLOW

In most installations when the discharge is increased above a certain critical value the resistance to flow (produced by the downstream reaches of the channel) is sufficient to reduce the velocity, increase the flow depth, and cause a backwater effect (sometimes called "flooding") at the Parshall flume. The effect is similar in some respects to that explained for the submerged orifice where (H - h) was the flow producing head.

It might be expected, therefore, that the discharge would begin to be reduced as soon as the tailwater level exceeded the elevation of the flume crest. This is not the case, however. Calibration tests show that the discharge is not reduced until the submergence, in percent, exceeds the following values:

$\frac{h}{H}$ = 0.50 = 50 percent for flumes 1 to 9 inches wide

$\frac{h}{H}$ = 0.70 = 70 percent for flumes 12 inches to 8 feet wide

$\frac{h}{H}$ = 0.80 = 80 percent for flumes 8 to 30 feet wide

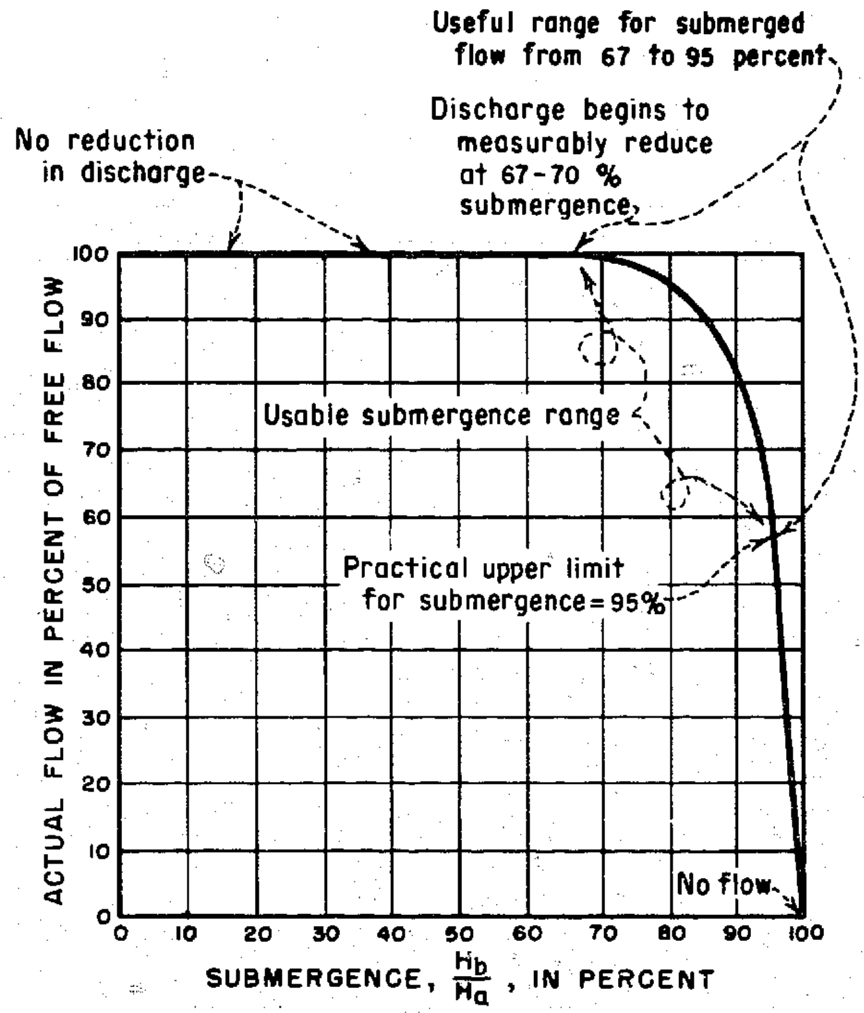
In the terminology used with Parshall flumes H is known as H_a and h is known as H_b.

A typical submergence curve which illustrates the effect of submergence in reducing the discharge through an open channel control structure such as a weir or small Parshall flume is shown in Figure 4.

The submergence curve shown is schematic, but characteristic, and indicates that the downstream tailwater surface can rise to 67-70 percent of the height of the headwater above the crest before the reduction in discharge becomes significant. Then with each additional percent increase in submergence the discharge is reduced at a more rapid rate, until, when H_b equals H_a there is no flow. This ultimate condition is purely theoretical, however, and would occur in nature only when the flow was stopped.

The 95 percent mark on the curve also represents a point of practical interest. In this range the Parshall flume ceases to be an accurate device. In other words a small differential between H_a and H_b is difficult to measure and any slight error in differential determination results in a large error in discharge.

The submergence curve of Figure 4 helps to explain the relative positions of the typical discharge curves shown in Figure 3 and indicates why the 70 percent submergence curve lies so close to the free flow curve. Also, Figure 3 shows that 60 percent of the maximum discharge occurs for 95 percent submergence; therefore, the remaining 40 percent of the maximum flow must be reduced to zero in the next 5 percent of submergence. Prohibitively accurate measurements of H_a and H_b would be required to make accurate discharge measurements in this range.



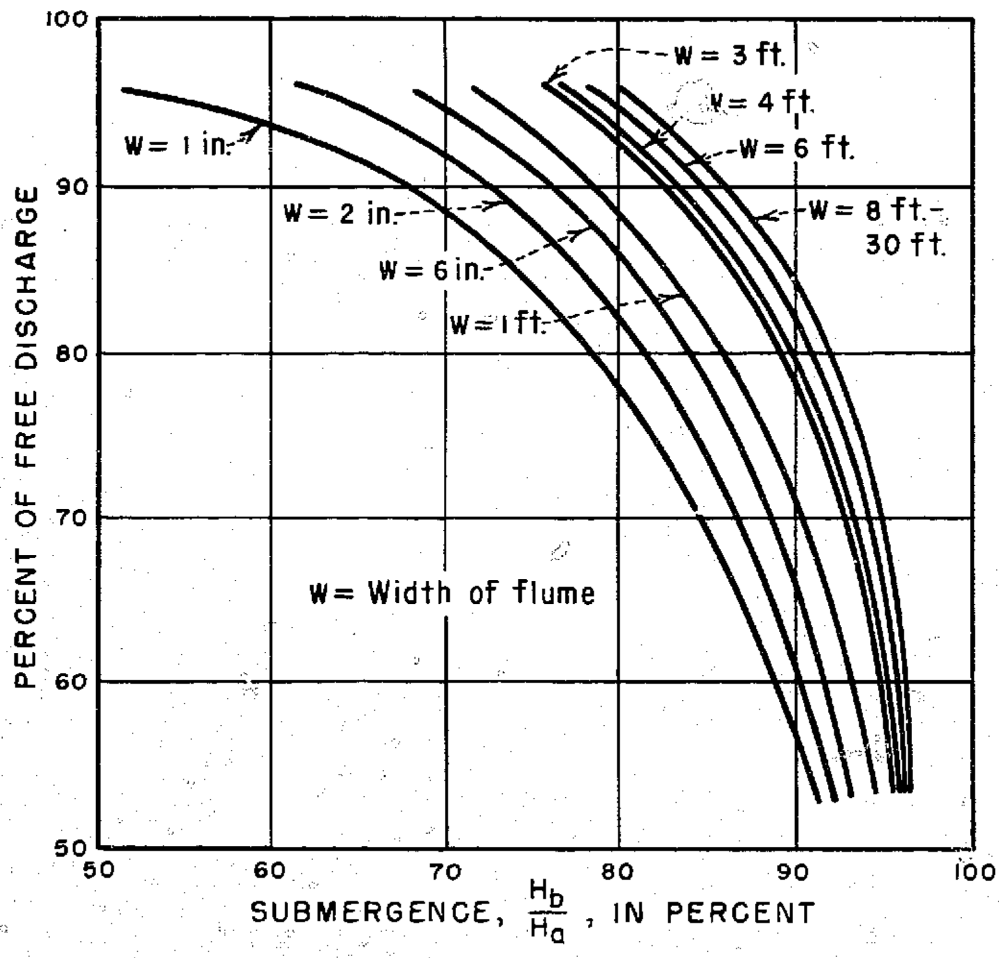
TYPICAL DISCHARGE REDUCTION CAUSED BY SUBMERGENCE IN 12-INCH TO 8 FOOT FLUMES

103-D-868

Figure 4

APPROXIMATION OF DISCHARGE RATE--SUBMERGED FLOW

It is difficult to visualize or estimate the change in discharge rate caused by changing the degree of submergence in various sizes of flumes. The curves in Figure 5 can be used to estimate the effect of increasing or decreasing the submergence, or increasing or decreasing the flume size without the need for knowing or considering the upstream head, H_a. For example, Figure 5 shows that at 60 percent submergence only the 1-inch flume would be affected; the discharge would be reduced to about 93 percent of the free discharge rate At 70 percent submergence only the 1-foot and smaller flumes would



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APPROXIMATE DISCHARGE AFTER SUBMERGENCE
FLUME WIDTHS 1 INCH TO 30 FEET

Figure 5

be affected; the 1-inch flume would discharge 89 percent of the free discharge rate and the 1-foot flume would discharge 94 percent of the free discharge rate. Also, at a submergence of 80 percent the discharge from all flumes will be affected to some degree. The 8- to 30-foot sizes will discharge 96 percent of the free discharge while the 1-inch flume will discharge only about 79 percent of the free discharge. The nomograph in the frontispiece can be used to obtain most free discharge values.

It may be seen, therefore, that submergence effects on discharge rates occur at the lower submergence ratios (percentages) on the small flumes, and that the greatest percentage reduction in discharge also occurs on the small flumes.

Although the curves in Figure 5 are approximate, because they do not include the small changes in discharge which also occur as a function of the magnitude of the discharge, the values are reasonably accurate and are useful for most estimating purposes. The graph may also be useful in making rough calculations for determining the size of flume required, and the best vertical placement in the channel. The curves represent observed data (obtained during calibration runs and checks) with a maximum deviation of plus or minus 7 percent. In most of the usually used regions of the curves the overall accuracy is somewhat better. Methods for determining exact submerged discharge values are discussed later.

APPROACH FLOW CONDITIONS

In the original writings on Parshall flumes only brief mention is made of the importance of good or bad approach flow conditions. A few statements are made such as "* * * the accuracy is not affected by silt or slow water" * * * "nor by changing velocity of the stream * * *," "the velocity of approach is automatically controlled * * *" "the angles of convergence and divergence are such as to eliminate the effect of switching of the current in the diverging section * * *." Over the years, these and similar phrases in the original writings have been loosely interpreted to mean that the Parshall flume is able to overcome poor flow conditions in the approach to the flume and still maintain a good degree of accuracy. Nothing could be further from the truth. It should be remembered that these flumes were intended for use as "in-line" structures in a stream or canal where reasonably smooth flow, uniformly distributed across the width and depth of the cross section was the normal condition. It was not expected that the flumes would be placed in turnouts where the main flow lines are at right angles to the flume, or below control gates where turbulence and flow concentrations are in evidence.

The converging section of the flume is designed to accelerate the flow and smooth out small differences in velocity and flow distribution so that the flow passes through the flume throat in a "standard" smooth pattern. Each lineal inch of throat width is expected to pass the same quantity of water as every other inch. Only under these conditions can the flume pass a standard discharge.

It is obvious, then, that any upstream flow condition which results in changing the standard pattern at the flume throat will cause a departure from the standard discharge indicated in the tables. Considering the length of the approach section and the geometry of the flume, it is understandable that the relatively short converging section can exert only a limited influence on redistributing flow concentrations, or in correcting excessively high velocities. Therefore, if accuracy is of importance, the approach conditions to the flume should be obviously good.

Experience has shown that the flume should not be placed at right angles to the flowing stream--such as in a canal turnout--unless the flow is straightened and redistributed to form a uniform velocity and flow pattern before entering the flume. Surges in the flow, which sometimes persist through the flume, should be eliminated as should surface waves* of any appreciable size. The water should enter the converging section reasonably well distributed across the entrance width and the flow streamlines should be essentially parallel to the flume centerline. Also, the flow at the flume entrance should be free of "white" water and turbulence in the form of visible surface boils. Only then can the flume be considered as being capable of fulfilling all of the claims made for it.

Experience has also shown that it is better to provide "standard" conditions of approach and getaway than to try to estimate the effect of nonstandard conditions on accuracy. Nonstandard flow conditions are impossible to describe and evaluate in terms of measurement accuracy. Poor approach flow conditions should therefore be eliminated by deepening, widening, or straightening the flow channel; or by resetting or rearranging the measuring station. These procedures are preferred over attempting to correct the indicated discharges.

In locations where inequalities in the approach flow have resulted in flow measurement difficulties and no upstream wingwalls have been included in the original construction, the curved wingwalls shown in Figure 1 should be considered for installation. If it appears that a more gradual acceleration of the approaching flow would improve the flow patterns in the flume, the wingwalls should be constructed (perhaps on a temporary trial basis at first). Curved wingwalls are preferred over straight 45° walls, although any arrangement of walls, channel banks, or other, that improves the uniformity and smoothness of the approaching flow is acceptable.

FLUME SIZE SELECTION AND SETTING

FACTORS GOVERNING SIZE OF FLUME

Because it is possible to pass a given discharge through any of several sizes of flume, the choice of the proper size requires the full consideration of other factors. For example, different throat widths W will be

*A wave suppressor may be required. See Reference 6 in Acknowledgments.

required if 20 cfs is to be discharged with 2.5 feet of depth rather than with 1 foot of depth. In the interests of economy, the smallest practical size should be selected.

In selecting a flume size it is necessary to use the trial and error system on several sizes which are believed to be minimum or adequate. The final selection is then made, keeping in mind the original channel dimensions in terms of the requirements of the various flumes. In other words, if a 2-foot flume will handle the discharge without producing an overtopping of the upstream channel banks it would ordinarily be preferred over a 3- or 4-foot flume. However, when the width of the channel is also considered in the problem, it may be just as economical to use a 3- or 4-foot flume because longer and more costly wingwalls may be required to span the channel when using the narrower flumes. Also, the narrower flumes will avoid submergence for a greater range of flows because the greater constriction of the narrower flumes causes a greater rise in the water level upstream from the flume.* The exact placement site of the flume should therefore be selected before the flume size is finally determined.

FACTORS GOVERNING SELECTION OF SITE

Proper location of the flume is very important from the standpoints of accuracy and ease of operation. For convenience it should be located near the diversion point and near the regulating gates used to change the discharge. On the other hand, the flume should not be located in turbulent, surging, or unbalanced flow, or in a velocity pattern poorly distributed across the channel. The flume should be in a straight section of channel; no bends upstream, particularly. Finally, the site should be readily accessible by car or truck for both installation and maintenance purposes. (See also remarks on the selection of site in "Current Meter Gaging Stations.") After tentatively selecting the flume location, hydraulic data should be obtained for the site, if available, and the channel dimensions should be measured (surveyed). Width, depth, height of banks upstream (to contain the increased depth caused by the flume installation) should be recorded. A rating curve for the channel would be helpful although this may be difficult to obtain, particularly for the smaller channels. However, the elevation of the water surface for the maximum and minimum flows should be obtained as a minimum data requirement. (The high waterline on the ditch banks may be all that is needed in certain cases.)

SELECTION OF FLUME SIZE AND CREST ELEVATION

Selection of flume size and the vertical placement in the channel, as required in irrigation practice, are best described by examples.

*This fact is in direct opposition to a statement made in a manufacturer's brochure.

Problem: Determine the flume size and setting to measure flows up to 20 second-feet in a channel of moderate grade where the water depth is 2.5 feet and the channel banks are about 10 feet apart; both for a free flow and submerged setting.

Free flow

Twenty second-feet can be measured in several sizes of flume as indicated by the throat width W in Table I. A flume 8 feet wide at 0.75 foot of head, H_a; 7 feet wide at 0.81 H_a; 6 feet at 0.89 H_a; 5 feet at 1.00 H_a; 4 feet at 1.15 H_a; 3 feet at 1.38 H_a; 2 feet at 1.80 H_a; and a flume 1.5 feet wide at 2.19 H_a are all capable of handling a discharge of 20 cfs. Considering the site, the 4-foot flume seems to be most practical but the 3- and 2-foot flumes will also be investigated.

For the 4-foot flume, H_a is 1.15 feet. For free flow the maximum submergence can be 70 percent; or the ratio H_b/H_a = 0.7 = 70 percent. Therefore,

$$\frac{H_b}{1.15} = 0.7$$

and

$$H_b = (1.15) \times (0.7) = 0.805 \text{ foot}$$

The depth downstream from the proposed flume, after the flume has been installed will be unchanged and will be 2.5 feet as assumed in the problem.

Referring to Figure 6, this 2.5 feet of depth will occur at D.

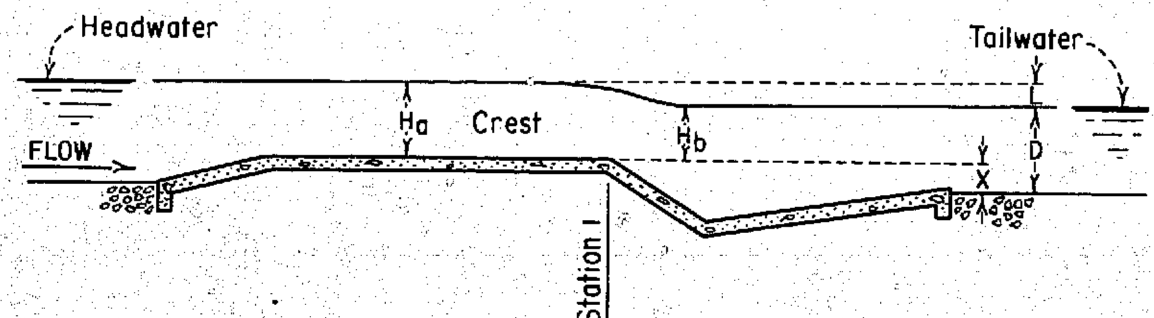


Figure 6. Relationship of flow depths to the crest elevation

Table 1

Free-flow discharge table for Parshall measuring flumes--3 inches to 8 feet

Head, H_a (feet)	Discharge, Q, in second feet, for throat widths, W, of--											
	3 inches	6 inches	9 inches	1 foot	1.5 feet	2 feet	3 feet	4 feet	5 feet	6 feet	7 feet	8 feet
0.10	0.028	0.05	0.09	0.11	0.15	-	-	-	-	-	-	-
.11	.033	.06	.10	.12	.18	-	-	-	-	-	-	-
.12	.037	.07	.12	.14	.21	-	-	-	-	-	-	-
.13	.042	.08	.14	.16	.24	-	-	-	-	-	-	-
.14	.047	.09	.15	.18	.27	-	-	-	-	-	-	-
.15	.053	.10	.17	.20	.30	0.42	0.61	-	-	-	-	-
.16	.058	.11	.19	.23	.34	.47	.68	-	-	-	-	-
.17	.064	.12	.20	.26	.38	.51	.75	-	-	-	-	-
.18	.070	.14	.22	.29	.42	.56	.82	-	-	-	-	-
.19	.076	.15	.24	.32	.46	.61	.89	-	-	-	-	-
.20	.082	.16	.26	.35	.51	.66	.97	1.26	1.55	-	-	-
.21	.089	.18	.28	.37	.55	.71	1.04	1.36	1.68	-	-	-
.22	.095	.19	.30	.40	.59	.77	1.12	1.47	1.81	-	-	-
.23	.102	.20	.32	.43	.63	.82	1.20	1.58	1.94	-	-	-
.24	.109	.22	.35	.46	.67	.88	1.28	1.69	2.08	-	-	-
.25	.117	.23	.37	.49	.71	.93	1.37	1.80	2.22	2.63	3.02	3.46
.26	.124	.25	.39	.51	.76	.99	1.46	1.91	2.36	2.80	3.25	3.68
.27	.131	.26	.41	.54	.80	1.05	1.55	2.03	2.50	2.97	3.44	3.90
.28	.138	.28	.44	.58	.85	1.11	1.64	2.15	2.65	3.15	3.65	4.13
.29	.146	.29	.46	.61	.90	1.18	1.73	2.27	2.80	3.33	3.85	4.37
.30	.154	.31	.49	.64	.94	1.24	1.82	2.39	2.96	3.52	4.08	4.62
.31	.162	.32	.51	.68	.99	1.30	1.92	2.52	3.12	3.71	4.30	4.88
.32	.170	.34	.54	.71	1.04	1.37	2.02	2.65	3.28	3.90	4.52	5.13
.33	.179	.36	.56	.74	1.09	1.44	2.12	2.78	3.44	4.10	4.75	5.39
.34	.187	.38	.59	.77	1.14	1.50	2.22	2.92	3.61	4.30	4.98	5.66
.35	.196	.39	.62	.80	1.19	1.57	2.32	3.06	3.78	4.50	5.22	5.93
.36	.205	.41	.64	.84	1.25	1.64	2.42	3.20	3.95	4.71	5.46	6.20
.37	.213	.43	.67	.88	1.30	1.72	2.53	3.34	4.13	4.92	5.70	6.48
.38	.222	.45	.70	.92	1.36	1.79	2.64	3.48	4.31	5.13	5.95	6.76
.39	.231	.47	.73	.95	1.41	1.86	2.75	3.62	4.49	5.35	6.20	7.05
.40	.241	.48	.76	.99	1.47	1.93	2.86	3.77	4.68	5.57	6.46	7.34
.41	.250	.50	.78	1.03	1.53	2.01	2.97	3.92	4.86	5.80	6.72	7.64
.42	.260	.52	.81	1.07	1.58	2.09	3.08	4.07	5.05	6.02	6.98	7.94
.43	.269	.54	.84	1.11	1.64	2.16	3.29	4.22	5.24	6.25	7.25	8.24
.44	.279	.56	.87	1.15	1.70	2.24	3.32	4.38	5.43	6.48	7.52	8.55
.45	.289	.58	.90	1.19	1.76	2.32	3.44	4.54	5.63	6.72	7.80	8.87
.46	.299	.61	.94	1.23	1.82	2.40	3.56	4.70	5.83	6.96	8.08	9.19
.47	.309	.63	.97	1.27	1.88	2.48	3.68	4.86	6.03	7.20	8.36	9.51
.48	.319	.65	1.00	1.31	1.94	2.57	3.80	5.03	6.24	7.44	8.65	9.84
.49	.329	.67	1.03	1.35	2.00	2.65	3.92	5.20	6.45	7.69	8.94	10.2
.50	.339	.69	1.06	1.39	2.06	2.73	4.05	5.36	6.66	7.94	9.23	10.5
.51	.350	.71	1.10	1.44	2.13	2.82	4.18	5.53	6.87	8.20	9.53	10.9
.52	.361	.73	1.13	1.48	2.19	2.90	4.31	5.70	7.09	8.46	9.83	11.2
.53	.371	.76	1.16	1.52	2.25	2.99	4.44	5.88	7.30	8.72	10.1	11.5
.54	.382	.78	1.20	1.57	2.32	3.08	4.57	6.05	7.52	8.98	10.5	11.9
.55	.393	.80	1.23	1.62	2.39	3.17	4.70	6.23	7.74	9.25	10.8	12.2
.56	.404	.82	1.26	1.66	2.45	3.26	4.84	6.41	7.97	9.52	11.1	12.6
.57	.415	.85	1.30	1.70	2.52	3.35	4.98	6.59	8.20	9.79	11.4	13.0
.58	.427	.87	1.33	1.75	2.59	3.44	5.11	6.77	8.43	10.1	11.7	13.3
.59	.438	.89	1.37	1.80	2.66	3.53	5.25	6.96	8.66	10.4	12.0	13.7

Table 1--Continued

Free-flow discharge table for Parshall measuring flumes--3 inches to 8 feet

Head, H_a (feet)	Discharge, Q, in second feet, for throat widths, W, of--											
	3 inches	6 inches	9 inches	1 foot	1.5 feet	2 feet	3 feet	4 feet	5 feet	6 feet	7 feet	8 feet
0.60	0.450	0.92	1.40	1.84	2.73	3.62	5.39	7.15	8.89	10.6	12.4	14.1
.61	.462	.94	1.44	1.88	2.80	3.72	5.53	7.34	9.13	10.9	12.7	14.5
.62	.474	.97	1.48	1.93	2.87	3.81	5.68	7.53	9.37	11.2	13.0	14.8
.63	.485	.99	1.51	1.98	2.95	3.91	5.82	7.72	9.61	11.5	13.4	15.2
.64	.497	1.02	1.55	2.03	3.02	4.01	5.97	7.91	9.85	11.8	13.7	15.6
.65	.509	1.04	1.59	2.08	3.09	4.11	6.12	8.11	10.1	12.1	14.1	16.0
.66	.522	1.07	1.63	2.13	3.17	4.20	6.26	8.31	10.3	12.4	14.4	16.4
.67	.534	1.10	1.66	2.18	3.24	4.30	6.41	8.51	10.6	12.7	14.8	16.8
.68	.546	1.12	1.70	2.23	3.31	4.40	6.56	8.71	10.8	13.0	15.1	17.2
.69	.558	1.15	1.74	2.28	3.39	4.50	6.71	8.91	11.1	13.3	15.5	17.6
.70	.571	1.17	1.78	2.33	3.46	4.60	6.86	9.11	11.4	13.6	15.8	18.0
.71	.584	1.20	1.82	2.38	3.54	4.70	7.02	9.32	11.6	13.9	16.2	18.5
.72	.597	1.23	1.86	2.43	3.62	4.81	7.17	9.53	11.9	14.2	16.6	18.9
.73	.610	1.26	1.90	2.48	3.69	4.91	7.33	9.74	12.1	14.5	16.9	19.3
.74	.623	1.28	1.94	2.53	3.77	5.02	7.49	9.95	12.4	14.9	17.3	19.7
.75	.636	1.31	1.98	2.58	3.85	5.12	7.65	10.2	12.7	15.2	17.7	20.1
.76	.649	1.34	2.02	2.63	3.93	5.23	7.81	10.4	12.9	15.5	18.0	20.6
.77	.662	1.36	2.06	2.68	4.01	5.34	7.97	10.6	13.2	15.8	18.4	21.0
.78	.675	1.39	2.10	2.74	4.09	5.44	8.13	10.8	13.5	16.2	18.8	21.5
.79	.689	1.42	2.14	2.80	4.17	5.55	8.30	11.0	13.8	16.5	19.2	21.9
.80	.702	1.45	2.18	2.85	4.26	5.66	8.46	11.3	14.0	16.8	19.6	22.4
.81	.716	1.48	2.22	2.90	4.34	5.77	8.63	11.5	14.3	17.2	20.0	22.8
.82	.730	1.50	2.27	2.96	4.42	5.88	8.79	11.7	14.6	17.5	20.4	23.3
.83	.744	1.53	2.31	3.02	4.50	6.00	8.96	11.9	14.9	17.8	20.8	23.7
.84	.757	1.56	2.35	3.07	4.59	6.11	9.13	12.2	15.2	18.2	21.2	24.2
0.85	.771	1.59	2.39	3.12	4.67	6.22	9.30	12.4	15.5	18.5	21.6	24.6
.86	.786	1.62	2.44	3.18	4.76	6.33	9.48	12.6	15.8	18.9	22.0	25.1
.87	.800	1.65	2.48	3.24	4.84	6.44	9.65	12.8	16.0	19.2	22.4	25.6
.88	.814	1.68	2.52	3.29	4.93	6.56	9.82	13.1	16.3	19.6	22.8	26.1
.89	.828	1.71	2.57	3.35	5.01	6.68	10.0	13.3	16.6	19.9	23.2	26.5
.90	.843	1.74	2.61	3.41	5.10	6.80	10.2	13.6	16.9	20.3	23.7	27.0
.91	.858	1.77	2.66	3.46	5.19	6.92	10.4	13.8	17.2	20.7	24.1	27.5
.92	.872	1.81	2.70	3.52	5.28	7.03	10.5	14.0	17.5	21.0	24.5	28.0
.93	.887	1.84	2.75	3.58	5.37	7.15	10.7	14.3	17.8	21.4	24.9	28.5
.94	.902	1.87	2.79	3.64	5.46	7.27	10.9	14.5	18.1	21.8	25.4	29.0
.95	.916	1.90	2.84	3.70	5.55	7.39	11.1	14.8	18.4	22.1	25.8	29.5
.96	.931	1.93	2.88	3.76	5.64	7.51	11.3	15.0	18.8	22.5	26.2	30.0
.97	.946	1.97	2.93	3.82	5.73	7.63	11.4	15.3	19.1	22.9	26.7	30.5
.98	.961	2.00	2.98	3.88	5.82	7.75	11.6	15.5	19.4	23.2	27.1	31.0
.99	.977	2.03	3.02	3.94	5.91	7.88	11.8	15.8	19.7	23.6	27.6	31.5
1.00	.992	2.06	3.07	4.00	6.00	8.00	12.0	16.0	20.0	24.0	28.0	32.0
1.01	1.01	2.09	3.12	4.06	6.09	8.12	12.2	16.3	20.3	24.4	28.5	32.5
1.02	1.02	2.12	3.17	4.12	6.19	8.25	12.4	16.5	20.6	24.8	28.9	33.0
1.03	1.04	2.16	3.21	4.18	6.28	8.38	12.6	16.8	21.0	25.2	29.4	33.6
1.04	1.05	2.19	3.26	4.25	6.37	8.50	12.8	17.0	21.3	25.6	29.8	34.1
1.05	1.07	2.22	3.31	4.31	6.47	8.63	13.0	17.3	21.6	25.9	30.3	34.6
1.06	1.09	2.26	3.36	4.37	6.56	8.76	13.2	17.5	21.9	26.3	30.7	35.1
1.07	1.10	2.29	3.40	4.43	6.66	8.88	13.3	17.8	22.3	26.7	31.2	35.7
1.08	1.12	2.32	3.45	4.50	6.75	9.01	13.5	18.1	22.6	27.1	31.7	36.2
1.09	1.13	2.36	3.50	4.56	6.85	9.14	13.7	18.3	22.9	27.5	32.1	36.8

Table 1--Continued

Free-flow discharge table for Parshall measuring flumes--3 inches to 8 feet

Head, H_a (feet)	Discharge, Q, in second feet, for throat widths, W, of--											
	3 inches	6 inches	9 inches	1 foot	1.5 feet	2 feet	3 feet	4 feet	5 feet	6 feet	7 feet	8 feet
1.10	1.15	2.40	3.55	4.62	6.95	9.27	13.9	18.6	23.3	27.9	32.6	37.3
1.11	1.16	2.43	3.60	4.68	7.04	9.40	14.1	18.9	23.6	28.4	33.1	37.8
1.12	1.18	2.46	3.65	4.75	7.14	9.54	14.3	19.1	23.9	28.8	33.6	38.4
1.13	1.20	2.50	3.70	4.82	7.24	9.67	14.5	19.4	24.3	29.2	34.1	38.9
1.14	1.21	2.53	3.75	4.88	7.34	9.80	14.7	19.7	24.6	29.6	34.5	39.5
1.15	1.22	2.57	3.80	4.94	7.44	9.94	14.9	19.9	25.0	30.0	35.0	40.1
1.16	1.25	2.60	3.85	5.01	7.54	10.1	15.1	20.2	25.3	30.4	35.5	40.6
1.17	1.26	2.64	3.90	5.08	7.64	10.2	15.3	20.5	25.7	30.8	36.0	41.2
1.18	1.28	2.68	3.95	5.15	7.74	10.3	15.6	20.8	26.0	31.3	36.5	41.8
1.19	1.30	2.71	4.01	5.21	7.84	10.5	15.8	21.1	26.4	31.7	37.0	42.3
1.20	1.32	2.75	4.06	5.28	7.94	10.6	16.0	21.3	26.7	32.1	37.5	42.9
1.21	1.33	2.78	4.11	5.34	8.05	10.8	16.2	21.6	27.1	32.5	38.0	43.5
1.22	1.35	2.82	4.16	5.41	8.15	10.9	16.4	21.9	27.4	33.0	38.5	44.1
1.23	1.37	2.86	4.22	5.48	8.25	11.0	16.6	22.2	27.8	33.4	39.0	44.6
1.24	1.38	2.89	4.27	5.55	8.36	11.2	16.8	22.5	28.1	33.8	39.5	45.2
1.25	1.40	2.93	4.32	5.62	8.46	11.3	17.0	22.8	28.5	34.3	40.0	45.8
1.26	1.42	2.97	4.37	5.69	8.56	11.5	17.2	23.0	28.9	34.7	40.5	46.4
1.27	1.44	3.01	4.43	5.76	8.67	11.6	17.4	23.3	29.2	35.1	41.1	47.0
1.28	1.45	3.04	4.48	5.82	8.77	11.7	17.7	23.6	29.6	35.6	41.6	47.6
1.29	1.47	3.08	4.53	5.89	8.88	11.9	17.9	23.9	30.0	36.0	42.1	48.2
1.30	1.49	3.12	4.59	5.96	8.99	12.0	18.1	24.2	30.3	36.5	42.6	48.8
1.31	1.50	3.16	4.64	6.03	9.09	12.2	18.3	24.5	30.7	36.9	43.1	49.4
1.32	1.52	3.19	4.69	6.10	9.20	12.3	18.5	24.8	31.1	37.4	43.7	50.0
1.33	1.54	3.23	4.75	6.18	9.30	12.4	18.8	25.1	31.4	37.8	44.2	50.6
1.34	1.56	3.27	4.80	6.25	9.41	12.6	19.0	25.4	31.8	38.3	44.7	51.2
1.35	1.58	3.31	4.86	6.32	9.52	12.7	19.2	25.7	32.2	38.7	45.3	51.8
1.36	1.59	3.35	4.92	6.39	9.63	12.9	19.4	26.0	32.6	39.2	45.8	52.5
1.37	1.61	3.39	4.97	6.56	9.74	13.0	19.6	26.3	33.0	39.7	46.4	53.1
1.38	1.63	3.43	5.03	6.53	9.85	13.2	19.9	26.6	33.3	40.1	46.9	53.7
1.39	1.65	3.47	5.08	6.60	9.96	13.3	20.1	26.9	33.7	40.6	47.4	54.3
1.40	1.67	3.51	5.14	6.68	10.1	13.5	20.3	27.2	34.1	41.1	48.0	55.0
1.41	1.69	3.55	5.19	6.75	10.2	13.6	20.6	27.5	34.5	41.5	48.5	55.6
1.42	1.70	3.59	5.25	6.82	10.3	13.8	20.8	27.8	34.9	42.0	49.1	56.2
1.43	1.72	3.63	5.31	6.89	10.4	13.9	21.0	28.1	35.3	42.5	49.6	56.9
1.44	1.74	3.67	5.37	6.97	10.5	14.1	21.2	28.5	35.7	42.9	50.2	57.5
1.45	1.76	3.71	5.42	7.04	10.6	14.2	21.5	28.8	36.1	43.4	50.8	58.1
1.46	1.78	3.75	5.48	7.12	10.7	14.4	21.7	29.1	36.5	43.9	51.3	58.8
1.47	1.80	3.79	5.54	7.19	10.8	14.5	21.9	29.4	36.9	44.4	51.9	59.4
1.48	1.82	3.83	5.59	7.26	11.0	14.7	22.2	29.7	37.3	44.9	52.5	60.1
1.49	1.84	3.87	5.65	7.34	11.1	14.9	22.4	30.0	37.7	45.3	53.0	60.7
1.50	1.86	3.91	5.71	7.41	11.2	15.0	22.6	30.3	38.1	45.8	53.6	61.4
1.51	-	-	5.77	7.49	11.3	15.2	22.9	30.7	38.5	46.3	54.2	62.1
1.52	-	-	5.83	7.57	11.4	15.3	23.1	31.0	38.9	46.8	54.7	62.7
1.53	-	-	5.89	7.64	11.5	15.5	23.4	31.3	39.3	47.3	55.3	63.4
1.54	-	-	5.94	7.72	11.7	15.6	23.6	31.6	39.7	47.8	55.9	64.0
1.55	-	-	6.00	7.80	11.8	15.8	23.8	32.0	40.1	48.3	56.5	64.7
1.56	-	-	6.06	7.87	11.9	15.9	24.1	32.3	40.5	48.8	57.1	65.4
1.57	-	-	6.12	7.95	12.0	16.1	24.3	32.6	40.9	49.3	57.7	66.1
1.58	-	-	6.18	8.02	12.1	16.3	24.6	32.9	41.3	49.8	58.2	66.7
1.59	-	-	6.24	8.10	12.2	16.4	24.8	33.3	41.8	50.3	58.8	67.4

Table 1--Continued

Free-flow discharge table for Parshall measuring flumes--3 inches to 8 feet

Head, H_a (feet)	Discharge, Q, in second feet, for throat widths, W, of--											
	3 inches	6 inches	9 inches	1 foot	1.5 feet	2 feet	3 feet	4 feet	5 feet	6 feet	7 feet	8 feet
1.60	-	-	6.31	8.18	12.4	16.6	25.1	33.6	42.2	50.8	59.4	68.1
1.61	-	-	6.37	8.26	12.5	16.7	25.3	33.9	42.6	51.3	60.0	68.8
1.62	-	-	6.43	8.34	12.6	16.9	25.5	34.3	43.0	51.8	60.6	69.5
1.63	-	-	6.49	8.42	12.7	17.1	25.8	34.6	43.4	52.3	61.2	70.2
1.64	-	-	6.55	8.49	12.8	17.2	26.0	34.9	43.9	52.8	61.8	70.9
1.65	-	-	6.61	8.57	13.0	17.4	26.3	35.3	44.3	53.3	62.4	71.6
1.66	-	-	6.67	8.65	13.1	17.6	26.5	35.6	44.7	53.9	63.0	72.3
1.67	-	-	6.73	8.73	13.2	17.7	26.8	35.9	45.1	54.4	63.6	73.0
1.68	-	-	6.79	8.81	13.3	17.9	27.0	36.3	45.6	54.9	64.3	73.7
1.69	-	-	6.86	8.89	13.5	18.0	27.3	36.6	46.0	55.4	64.9	74.4
1.70	-	-	6.92	8.97	13.6	18.2	27.6	37.0	46.4	56.0	65.5	75.1
1.71	-	-	6.98	9.05	13.7	18.4	27.8	37.3	46.9	56.5	66.1	75.8
1.72	-	-	7.04	9.13	13.8	18.5	28.1	37.7	47.3	57.0	66.7	76.5
1.73	-	-	7.11	9.21	13.9	18.7	28.3	38.0	47.7	57.5	67.3	77.2
1.74	-	-	7.17	9.29	14.1	18.9	28.6	38.3	48.2	58.1	68.0	77.9
1.75	-	-	7.23	9.38	14.2	19.0	28.8	38.7	48.6	58.6	68.6	78.7
1.76	-	-	7.29	9.46	14.3	19.2	29.1	39.0	49.1	59.1	69.2	79.4
1.77	-	-	7.36	9.54	14.4	19.4	29.3	39.4	49.5	59.7	69.9	80.1
1.78	-	-	7.42	9.62	14.6	19.6	29.6	39.7	49.9	60.2	70.5	80.8
1.79	-	-	7.48	9.70	14.7	19.7	29.9	40.1	50.4	60.7	71.1	81.6
1.80	-	-	7.54	9.79	14.8	19.9	30.1	40.5	50.8	61.3	71.8	82.3
1.81	-	-	7.61	9.87	15.0	20.1	30.4	40.8	51.3	61.8	72.4	83.0
1.82	-	-	7.68	9.95	15.1	20.2	30.7	41.2	51.7	62.4	73.0	83.8
1.83	-	-	7.74	10.0	15.2	20.4	30.9	41.5	52.2	62.9	73.7	84.5
1.84	-	-	7.81	10.1	15.3	20.6	31.2	41.9	52.6	63.5	74.3	85.3
1.85	-	-	7.87	10.2	15.5	20.8	31.5	42.2	53.1	64.0	75.0	86.0
1.86	-	-	7.94	10.3	15.6	20.9	31.7	42.6	53.6	64.6	75.6	86.8
1.87	-	-	8.00	10.4	15.7	21.1	32.0	43.0	54.0	65.1	76.3	87.5
1.88	-	-	8.06	10.5	15.8	21.3	32.3	43.3	54.5	65.7	76.9	88.3
1.89	-	-	8.13	10.5	16.0	21.5	32.5	43.7	54.9	66.3	77.6	89.0
1.90	-	-	8.20	10.6	16.1	21.6	32.8	44.1	55.4	66.8	78.2	89.8
1.91	-	-	8.26	10.7	16.2	21.8	33.1	44.4	55.9	67.4	78.9	90.5
1.92	-	-	8.33	10.8	16.4	22.0	33.3	44.8	56.3	67.9	79.6	91.3
1.93	-	-	8.40	10.9	16.5	22.2	33.6	45.2	56.8	68.5	80.2	92.1
1.94	-	-	8.46	11.0	16.6	22.4	33.9	45.5	57.3	69.1	80.9	92.8
1.95	-	-	8.52	11.1	16.7	22.5	34.1	45.9	57.7	69.6	81.6	93.6
1.96	-	-	8.59	11.1	16.9	22.7	34.4	46.3	58.2	70.2	82.2	94.4
1.97	-	-	8.66	11.2	17.0	22.9	34.7	46.6	58.7	70.8	82.9	95.1
1.98	-	-	8.73	11.3	17.2	23.1	35.0	47.0	59.1	71.4	83.6	95.9
1.99	-	-	8.80	11.4	17.3	23.2	35.3	47.4	59.6	71.9	84.3	96.7
2.00	-	-	8.87	11.5	17.4	23.4	35.5	47.8	60.1	72.5	84.9	97.5
2.01	-	-	-	11.6	17.6	23.6	35.8	48.1	60.6	73.1	85.6	98.3
2.02	-	-	-	11.7	17.7	23.8	36.1	48.5	61.0	73.7	86.3	99.1
2.03	-	-	-	11.8	17.8	24.0	36.4	48.9	61.5	74.2	87.0	99.8
2.04	-	-	-	11.8	18.0	24.2	36.7	49.3	62.0	74.8	87.7	100.6
2.05	-	-	-	11.9	18.1	24.3	36.9	49.7	62.5	75.4	88.4	101.4
2.06	-	-	-	12.0	18.2	24.5	37.2	50.1	63.0	76.0	89.1	102.2
2.07	-	-	-	12.1	18.4	24.7	37.5	50.4	63.5	76.6	89.8	103.0
2.08	-	-	-	12.2	18.5	24.9	37.8	50.8	63.9	77.2	90.4	103.8
2.09	-	-	-	12.3	18.7	25.1	38.1	51.2	64.4	77.8	91.1	104.6

Table 1--Continued

Free-flow discharge table for Parshall measuring flumes--3 inches to 8 feet

Head H_a (feet)	Discharge, Q, in second feet, for throat widths, W, of--											
	3 inches	6 inches	9 inches	1 foot	1.5 feet	2 feet	3 feet	4 feet	5 feet	6 feet	7 feet	8 feet
2.10	-	-	-	12.4	18.8	25.3	38.4	51.6	64.9	78.4	91.8	105.4
2.11	-	-	-	12.5	18.9	25.5	38.6	52.0	65.4	79.0	92.5	106.2
2.12	-	-	-	12.6	19.0	25.6	38.9	52.4	65.9	79.6	93.3	107.0
2.13	-	-	-	12.6	19.2	25.8	39.2	52.8	66.4	80.2	94.0	107.9
2.14	-	-	-	12.7	19.3	26.0	39.5	53.2	66.9	80.8	94.7	108.7
2.15	-	-	-	12.8	19.5	26.2	39.8	53.5	67.4	81.4	95.4	109.5
2.16	-	-	-	12.9	19.6	26.4	40.1	53.9	67.9	82.0	96.1	110.3
2.17	-	-	-	13.0	19.7	26.6	40.4	54.3	68.4	82.6	96.8	111.1
2.18	-	-	-	13.1	19.9	26.8	40.7	54.7	68.9	83.2	97.5	111.9
2.19	-	-	-	13.2	20.0	27.0	41.0	55.1	69.4	83.8	98.2	112.8
2.20	-	-	-	13.3	20.2	27.2	41.3	55.5	69.9	84.4	98.9	113.6
2.21	-	-	-	13.4	20.3	27.3	41.5	55.9	70.4	85.0	99.7	114.4
2.22	-	-	-	13.5	20.5	27.5	41.8	56.3	70.9	85.6	100.4	115.3
2.23	-	-	-	13.6	20.6	27.7	42.1	56.7	71.4	86.3	101.1	116.1
2.24	-	-	-	13.7	20.7	27.9	42.4	57.1	71.9	86.9	101.8	116.9
2.25	-	-	-	13.7	20.9	28.1	42.7	57.5	72.4	87.5	102.6	117.8
2.26	-	-	-	13.8	21.0	28.3	43.0	57.9	72.9	88.1	103.3	118.6
2.27	-	-	-	13.9	21.2	28.5	43.3	58.3	73.5	88.7	104.0	119.5
2.28	-	-	-	14.0	21.3	28.7	43.6	58.7	74.0	89.4	104.8	120.3
2.29	-	-	-	14.1	21.4	28.9	43.9	59.2	74.5	90.0	105.5	121.2
2.30	-	-	-	14.2	21.6	29.1	44.2	59.6	75.0	90.6	106.2	122.0
2.31	-	-	-	14.3	21.7	29.3	44.5	60.0	75.5	91.2	107.0	122.9
2.32	-	-	-	14.4	21.9	29.5	44.8	60.4	76.1	91.9	107.7	123.7
2.33	-	-	-	14.5	22.0	29.7	45.1	60.8	76.6	92.5	108.5	124.6
2.34	-	-	-	14.6	22.2	29.9	45.4	61.2	77.1	93.1	109.2	125.4
2.35	-	-	-	14.7	22.4	30.1	45.7	61.6	77.6	93.8	110.0	126.3
2.36	-	-	-	14.8	22.5	30.3	46.0	62.0	78.1	94.4	110.7	127.2
2.37	-	-	-	14.9	22.6	30.5	46.4	62.4	78.7	95.1	111.5	128.0
2.38	-	-	-	15.0	22.8	30.7	46.7	62.9	79.2	95.7	112.2	128.9
2.39	-	-	-	15.1	22.9	30.9	47.0	63.3	79.7	96.3	113.0	129.8
2.40	-	-	-	15.2	23.0	31.1	47.3	63.7	80.3	97.0	113.7	130.7
2.41	-	-	-	15.3	23.2	31.3	47.6	64.1	80.8	97.6	114.5	131.5
2.42	-	-	-	15.4	23.3	31.5	47.9	64.5	81.3	98.3	115.3	132.4
2.43	-	-	-	15.5	23.5	31.7	48.2	65.0	81.8	98.9	116.0	133.3
2.44	-	-	-	15.6	23.7	31.9	48.5	65.4	82.4	99.6	116.8	134.2
2.45	-	-	-	15.6	23.8	32.1	48.8	65.8	82.9	100.2	117.6	135.1
2.46	-	-	-	15.7	23.9	32.3	49.1	66.2	83.5	100.9	118.3	135.9
2.47	-	-	-	15.9	24.1	32.5	49.5	66.7	84.0	101.5	119.1	136.8
2.48	-	-	-	15.9	24.2	32.7	49.8	67.1	84.5	102.2	119.9	137.7
2.49	-	-	-	16.0	24.4	32.9	50.1	67.5	85.1	102.8	120.6	138.6
2.50	-	-	-	16.1	24.6	33.1	50.4	67.9	85.6	103.5	121.4	139.5

Depth X is 1.69 feet (D minus H_b) which is the elevation of the crest above the channel bottom. For this setting the flow of 20 cfs will occur at 70 percent submergence and there will be no reduction in discharge caused by downstream backwater effects.

The structure itself, however, is an obstruction in the channel and will produce a backwater effect that will extend upstream from the flume and will raise the headwater elevation. This difference in elevation of the flow upstream from the structure with and without the flume in place is the head loss caused by the flume.

The head loss value can be determined from Figure 7.

Starting at the base of the diagram follow the 70 percent submergence line vertically to intersect the slanting 20 cfs discharge line; project a horizontal line through this point to intersect the slanting throat width line for W = 4 feet; project this point vertically downward to the loss of head scale and determine that the head loss L is 0.40 foot.

The water surface elevation of the approaching flow will then be,

$$D + L = 2.50 + 0.40 = 2.90$$

H_a is therefore located 2.90 feet above the channel bottom.

An examination of the site dimensions should now be made to determine whether the freeboard that would exist for these conditions (if the 4-foot flume were to be installed as indicated above) is satisfactory.

For the 3-foot flume, following the same analysis,

$$H_b = 0.97 \text{ foot}$$

$$X = 1.53 \text{ feet}$$

$$L = 0.54 \text{ foot}$$

and the upstream depth of water, H_a, would be located 3.04 feet above the bottom.

Figure 7

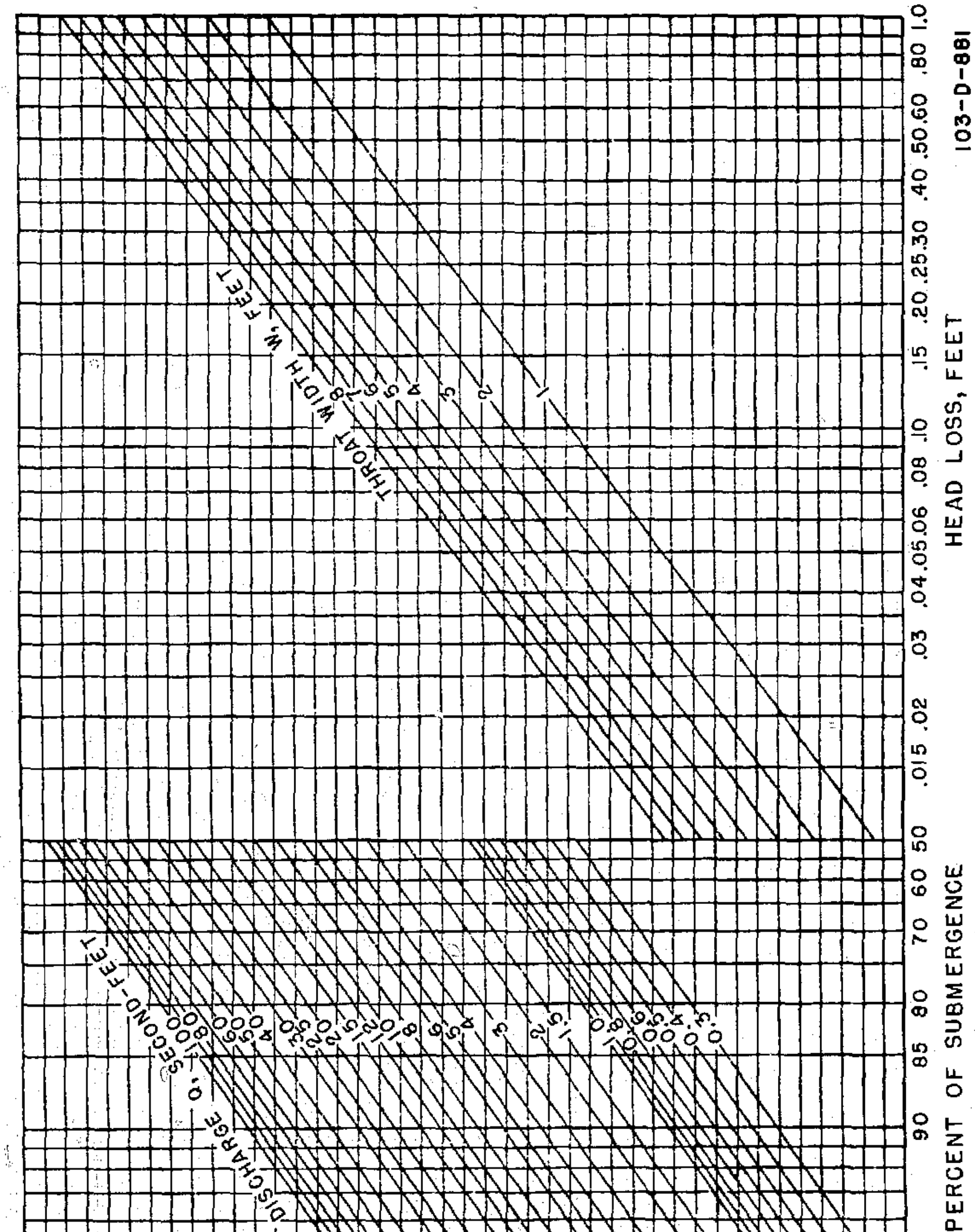


Diagram for determining the loss of head (increase in upstream depth) through Parshall measuring flumes, 1- to 8-foot sizes

For the 2-foot flume,

$$H_b = 1.27 \text{ feet}$$

$$X = 1.23 \text{ feet}$$

$$L = 0.70 \text{ foot}$$

and the depth of water upstream, H_a, would be located 3.20 feet above the bottom.

Thus the upstream depths for the 4-, 3-, and 2-foot flumes would be 2.90, 3.04, and 3.20 feet, respectively. Assuming the channel banks to be sufficiently high and any other special considerations to be favorable, the 2-foot flume would be chosen because of its lower first cost. However, when the width of the channel is considered in terms of constructing wingwalls to provide a nonoverflow section on each side of the flume, the 3- or 4-foot-width flume might be chosen to simplify construction or reduce the cost of the installation. As a general rule, the most economical flume size (W), is from one-third to one-half the width of the channel.

It should be noted in the above analysis that the increase in upstream flow depth is considerably less than the distance the crest is set above the bottom of the channel, i.e., L is less than X, Figure 6. For the 4-foot flume the increase in water level is only 24 percent of the height of the crest above the channel bottom, 35 percent for the 3-foot flume, and 57 percent for the 2-foot flume.

It is usually desirable to set the flume somewhat higher than absolutely necessary, rather than lower. This procedure will help to compensate for unknown or changing factors downstream which usually tend to increase the expected submergence. Deposits of sediment may increase the resistance to flow as may weeds, tree roots, moss, or other biological growths. A general lowering of the channel by scour or degradation, an unusual occurrence, is the only factor that would tend to decrease submergence.

Setting the flume higher than necessary will increase the velocity of the flow leaving the structure and may make it necessary to provide protection from local scour at the downstream end. However, these are the factors that must be considered in selecting a flume size and in setting the crest elevation at the most desirable elevation.

If the Parshall flume is never to be operated above the 70 percent submergence limit, there would be no need to construct the portion of the flume downstream from Station 1, Figure 6. When only this portion of the flume is constructed, the flume is sometimes referred to as the "Montana" flume. When installing this abbreviated version, the crest should be set above the channel bottom (the same distance as worked out in the previous example) just as though the downstream portion of the flume were in place. This will insure that the flow profile over the crest section is not modified by having too high a channel bottom downstream from the crest. As a suggested procedure, the channel bottom should be stabilized to a shape which is always below the level of the lower dotted line in Figure 6, to insure unchanged or "standard" operation of the flume.

Submerged flow

If the analysis of the free flow data for the 4-foot flume showed that it was necessary or desirable to lower the upstream water surface elevation as much as possible, the effect of operating the flume at 95 percent submergence (or any other value between 70 and 95) at the maximum discharge should be investigated. Operation at the higher submergence would allow the entire structure to be lower in the channel and would provide more channel bank freeboard upstream. Proceed as follows:

Using the same data given in the free flow analysis, the maximum discharge of 20 cfs is to be passed with a depth of 2.5 feet, but with 95 percent submergence. As in the free flow analysis from Table 1

$$H_a = 1.15 \text{ feet}$$

For 95 percent submergence

$$\frac{H_b}{H_a} = 0.95 = 95 \text{ percent}$$

$$\frac{H_b}{1.15} = 0.95$$

$$H_b = 1.09 \text{ feet}$$

In Figure 6 (D minus H_b) = X

$$(2.5 - 1.09) = X = 1.41 \text{ feet.}$$

Therefore, for 95 percent submergence the crest of the 4-foot flume should be set 1.41 feet above the bottom of the channel. (For 70 percent submergence the corresponding dimensions is 1.59 feet.)

To find the increase in headwater elevation (loss of head) resulting from 95 percent submergence, use Figure 7. Intersect the 95 percent submergence vertical line with the slanting 20-cfs line; project a horizontal line from this point to intersect the slanting line W = 4 feet; project a vertical line downward to "Head Loss" scale and find L = 0.007 foot.

The headwater elevation will be

$$L + D = 0.07 + 2.5 = 2.57 \text{ feet above channel bottom}$$

Since the headwater elevation for 70 percent submergence was found to be 2.90 feet, the difference in upstream water levels for 70 and 95 percent submergence is (2.90 - 2.57) = 0.33 foot. Lesser degrees of submergence may be investigated, if necessary, in a similar manner. After all possibilities concerning the size of flume and the proper degree of submergence have been investigated, the most economical size of flume can be chosen and the optimum elevation of the crest for best performance may be selected.

DISCHARGE DETERMINATIONS--3-INCH TO 8-FOOT SIZES

The method of determining the discharge of flumes installed in the field depends on whether the flume is submerged, or discharging freely.

FREE FLOW AND SUBMERGED FLOW

When the flow is free, or when the submergence is 50 percent or less for the very small flumes, 70 percent or less for the small flumes, and 80 percent or less for the large flumes, the discharge may be read directly from Table 1, using the upstream head, H_a, and the throat width, W, of the flume. When the percent of submergence is greater than the above limiting values the flow is considered to be "submerged." In other words, the downstream water surface is enough above the elevation of the flume crest to reduce the discharge that would otherwise occur for free flow. Both heads, H_a and H_b, must then be used to determine the discharge, i.e., a correction is made to the values shown in Table 1. Corrections for various sizes of flumes are discussed separately.

SUBMERGENCE CORRECTION FOR 1-FOOT FLUMES

Figure 8 shows the corrections in cubic feet per second to obtain submerged flow values from the free flow values given in Table 1. For example, in a 1-foot flume with $H_a = 1.00$ foot; the discharge from Table 1 is 4.00 cfs. If H_b is measured to be 0.8, the submergence $\frac{H_b}{H_a}$ is equal to 80 percent. In Figure 8, for $H_a = 1.00$ and a submergence of 80 percent, the correction is 0.35 cfs. Submergence would therefore reduce the discharge (4.00 - 0.35) to 3.65 cfs.

Figure 8 may be used to make corrections for a wide range of submergence in a 1-foot flume, using the method illustrated. In another example the discharge through a 1-foot flume with $H_a = 1.60$ feet, $H_b = 1.41$ feet is 6.38 cfs. As an exercise, check this.

For flumes larger than 1 foot, apply the corrections as indicated below.

SUBMERGENCE CORRECTION FOR LARGER THAN 1-FOOT FLUMES

When the submergence correction is for a 1.5- to 8-foot-wide flume the values obtained from Figure 8 should be multiplied by the factors shown in the table in Figure 8 before the product (the correction) is subtracted from the free discharge value.

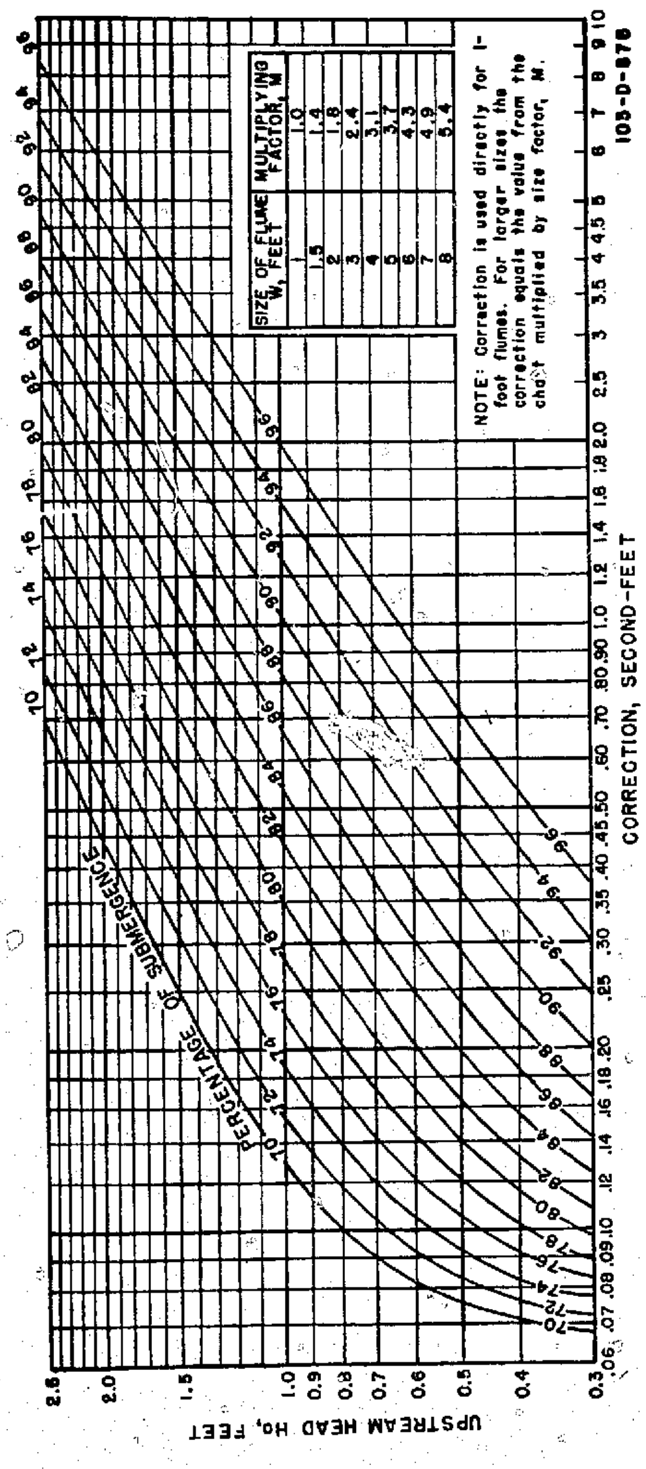
For example, submerged flow occurs in a 3-foot flume where $H_a = 2.10$ feet, and $H_b = 2.00$ feet. The submergence ratio $\frac{2.00}{2.10}$ is 0.95 or 95 percent submergence. The free flow discharge, Table 7, for a 3-foot flume and $H_a = 2.10$, is found to be 38.4 cfs. In Figure 8, for $H_a = 2.10$ and 95 percent submergence (which is an invisible interpolated line halfway between the 94 and 96 percent lines) the correction is 5.75 cfs. This is for a 1-foot flume, however, and the correction must be multiplied by the value 2.4 given in the table for a 3-foot flume (2.4 x 5.75) to get the total correction 13.8 cfs. The correction discharge is therefore 38.4 minus 13.8 or 24.6 cfs.

As another example, the submerged flow through a 6-foot flume when $H_a = 1.79$ feet and $H_b = 1.65$ feet is 46.7 cfs. Check this.

SUBMERGENCE CORRECTION FOR SMALLER THAN 1-FOOT FLUMES

When 3-, 6-, and 9-inch flumes are submerged the correction should be made directly, using Figures 9, 10, and 11, respectively. Example: What is the discharge through a 3-inch flume when $H_a = 1.32$ feet and $H_b = 1.20$ feet? The submergence ratio $\frac{1.20}{1.32}$ is 0.91 or 91 percent. In Figure 9, estimate 91 percent along the left-hand vertical scale (halfway between 90 and 92) and follow the invisible 91 percent line horizontally

Figure 8



Correction diagram for determining rate of submerged flow through a 1-foot Parshall flume to intersect the curved line for $H_a = 1.32$ (one-fifth the distance between the 1.3 and 1.4 lines), then move vertically downward from this point to the scale at the base of the diagram and find the submerged rate of flow to be 0.88 cfs.

In another example, the submerged flow rate through a 9-inch flume when the H_a head is 1.40 feet and H_b is 1.20 feet, is 4.19 cfs. Check this.

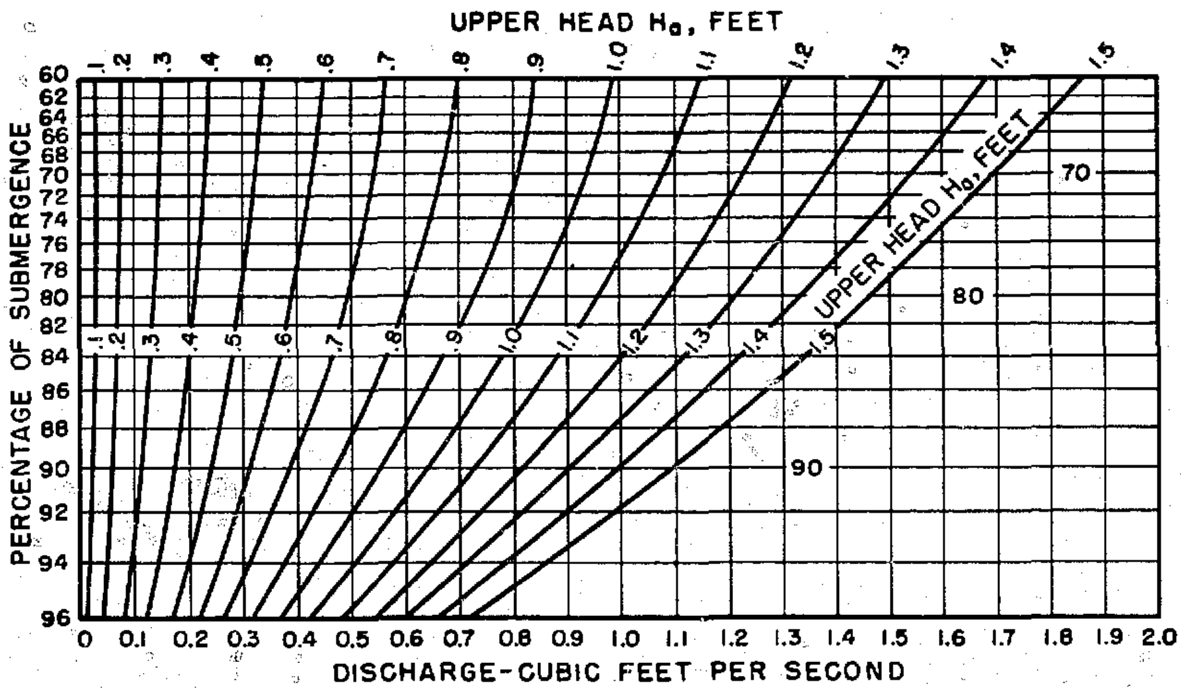


Figure 9. Rate of submerged flow, in cubic feet per second, through a 3-inch Parshall flume

DESCRIPTION OF FLUMES OF LARGE SIZE--10 FEET TO 50 FEET

The large size flumes include those having throat widths W of from 10 to 50 feet. Figure 1 shows the minimum and maximum capacities, the lengths and dimensions of various parts of the flumes, and other pertinent information regarding flumes of 10-, 12-, 15-, 20-, 25-, 30-, 40-, and 50-foot throat widths. In general, these large flumes are similar in appearance and operation to the intermediate sizes but there are significant differences in certain dimensions. Also, a more thorough analysis is required to determine the proper vertical setting of the flume.

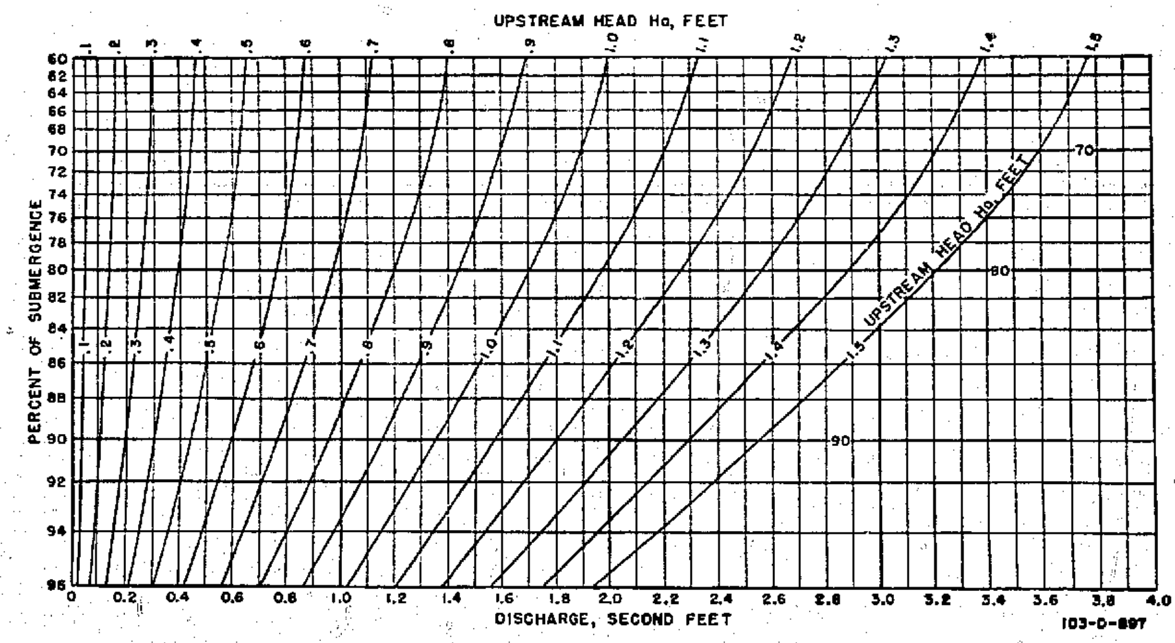


Figure 10. Rate of submerged flow through a 6-inch Parshall flume 103-D-897

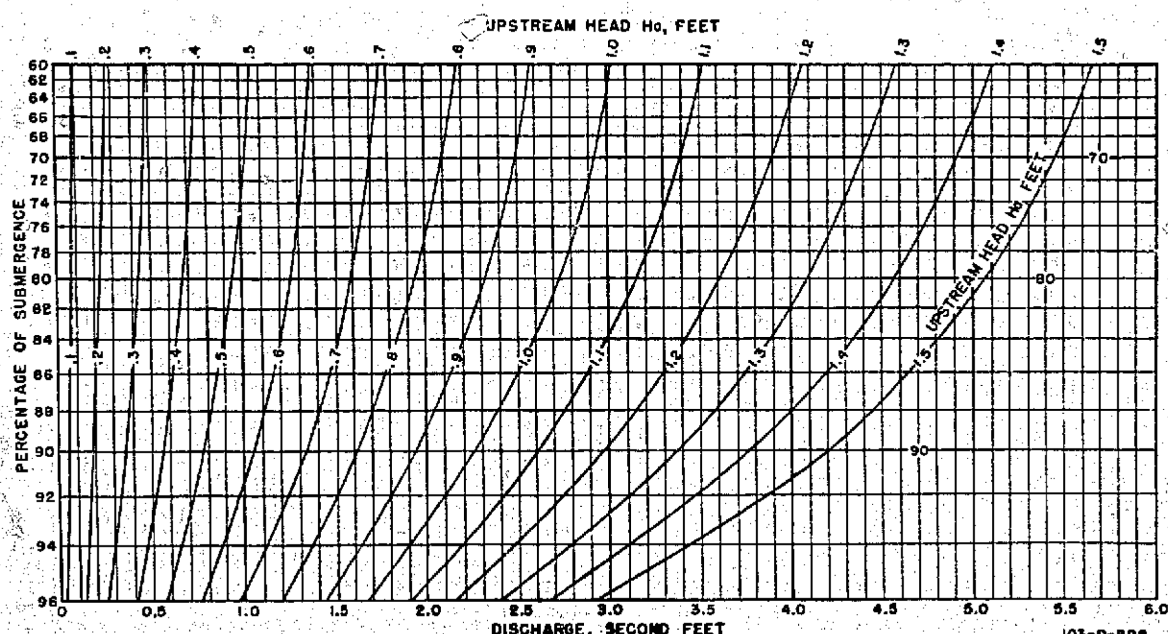


Figure 11. Rate of submerged flow through a 9-inch Parshall flume 103-D-898

In the small flumes, Figure 1, the length of the wall, A, of the converging section, in feet, is

$$\frac{W}{2} + 4 = A$$

where W is the throat width.

Also, the dimension used to locate the upstream head measuring station, H_a, is 2/3 A. In the large flumes the length of the converging section is considerably longer, in proportion, than in the small flumes. This was found during flume development to be necessary to obtain an adequately smooth flow pattern in the upstream part of the structure. The gage point, H_a, is maintained at the 2/3 A point, but A is a longer length in the large flumes.

Also, in the large flumes the downstream or H_b gage point is always located 12 inches upstream from the low point in the floor and 9 inches above the crest. In the small sizes, the H_b gage point is located much closer to the low point in the floor and more nearly at the crest elevation. In all flumes, the gage zeros should be established so that both the H_a and H_b gages give the depth of flow above the crest and not the depth of water above the H_a and H_b pressure openings.

SIZE SELECTION AND SETTING OF LARGE FLUMES

The rules and procedures for setting large flumes are, in general, the same as those for setting the intermediate sizes. However, because of the greater cost of large flumes and the finality of the placement insofar as adjusting the structure after it is built is concerned, greater care and more extensive analyses are usually made before the size and vertical placement are finalized. A brief mention of these extra precautions will be made and a sample problem illustrating the usual procedure is given.

For the large flumes it is very important that the crest be set at the most favorable elevation with reference to the gradeline of the channel. The freeboard of canal banks must be considered as well as the possibility of reducing the full capacity of the channel. Also, interference with the operation of established turnouts should be avoided if possible.

Consideration should be given to the fact that it is usually more desirable to measure accurately the smaller discharges which usually occur when water is scarce and valuable, than to measure accurately the maximum discharges which may occur infrequently when water is plentiful. If compromises are necessary it is usually preferable to sacrifice accuracy when maximum discharges are being measured.

The desirability of setting the flume sufficiently high to eliminate the need for the downstream H_b gage readings (insofar as is possible) cannot be overemphasized, Figure 12. On the other hand, excessive vertical fall of the water may result from this action and the higher exit velocities may make it necessary to provide scour protection (perhaps riprap) at the flume exit, Figure 2. Proper riprap sizes and placement practices are discussed in Item 6 in Acknowledgements.

In analyzing the setting of a large flume, it is necessary to have certain data on hand rather than to rely on estimates of the stream characteristics and capacity. For example, if it is desired to select a flume size and establish its vertical placement in a channel 50 feet wide having a capacity of 950 cfs, the following information should be available. The maximum discharge to be measured has been established as 500 cfs. (Quantities above 500 cfs may contain floodwaters which are not accountable or saleable.) Maximum submergence, based on accuracy requirements and ease of operation, has been set at 80 percent. From a rating curve obtained from actual field measurements the discharges and gage heights (flow depths) are:

Gage height feet	Discharge cfs	Gage height feet	Discharge cfs
0.5	18	3.5	398
1.0	45	4.0	500
1.5	86	4.5	607
2.0	145	5.0	718
2.5	218	5.5	892
3.0	303	6.0	950

Based on the general rule that the flume width, W, should be between one-third and one-half of the stream width, and on an approximate matching* of the gage height given above for 500 cfs with the H_a values for 500 cfs in the flume discharge tables, Tables II - IX, a 20-foot flume is tentatively selected.

In Table V, H_a is 3.24 feet for a flow of 500 cfs. The gage height for this discharge (see table above) is 4.0 feet. At 80 percent submergence H_b will be equal to 80 percent of 3.24, or 2.59 feet. The elevation of the crest above the channel, X, in Figure 6, is 4.00 - 2.59 = 1.41 feet. For this setting the maximum discharge 950 cfs will be submerged. To determine the actual value, a cut-and-try process is necessary; assume 90 percent submergence. The gage height for 950 cfs (in table above) is

*H_a for a first trial should be chosen to be somewhat less than the gage height to allow for freeboard and head loss through the structure.

6.0 feet; the H_b gage reading will then be 6.0 - 1.41 or about 4.6 feet. For 90 percent submergence the corresponding H_a value will be $\frac{4.6}{0.90}$ = 5.11 feet. The free flow value for this H_a value, Table V, is 1,037 cfs. From the correction diagram, Figure 13, the reduction in discharge for 90 percent submergence in a 20-foot flume, for H_a = 5.11, is about 145 cfs and is determined as follows: In Figure 13, project a horizontal line through the vertical scale, at the point where H_a = 5.11, to intersect the sloping 90 percent submergence line. The value on the horizontal scale vertically below this point is 72.5 cfs. Because the corrections are

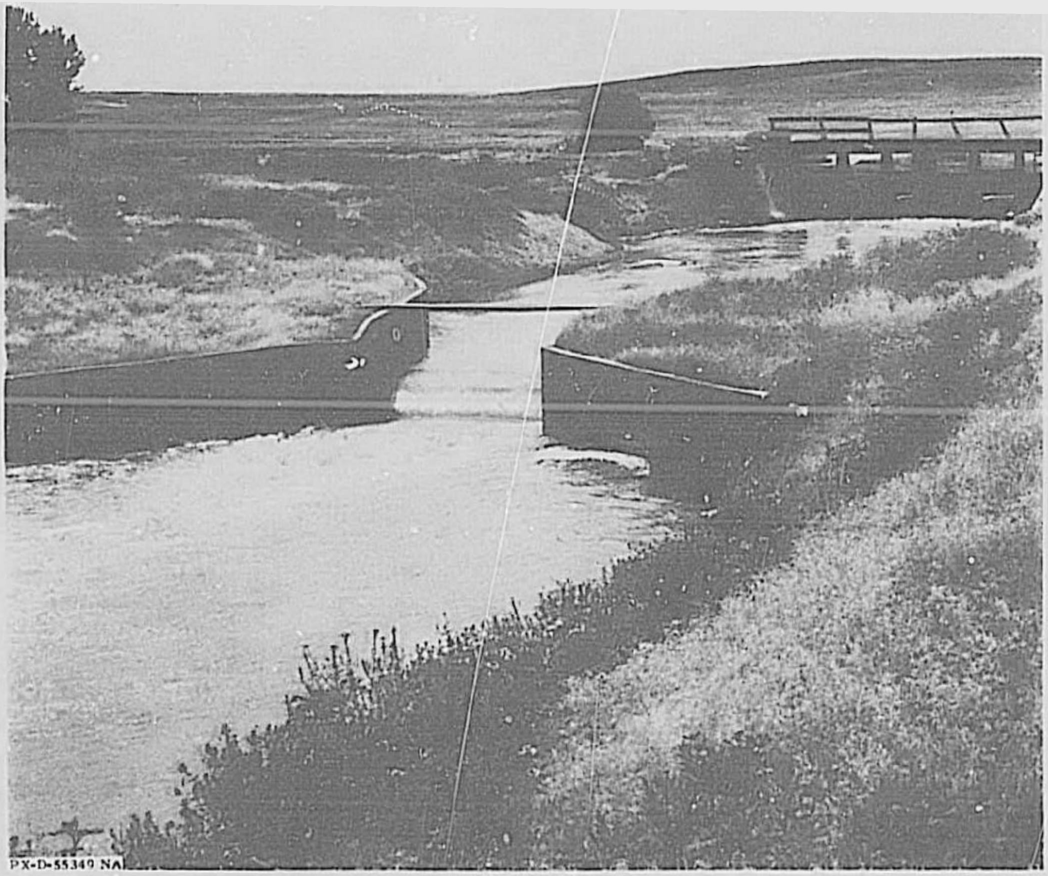


Figure 12. Twelve-foot Parshall flume discharging 82 cfs under free flow conditions.

TABLE II
FREE-FLOW DISCHARGE 10-FOOT PARSHALL MEASURING FLUME

FORMULA $Q = 39.38\,H_A^{1.6}$

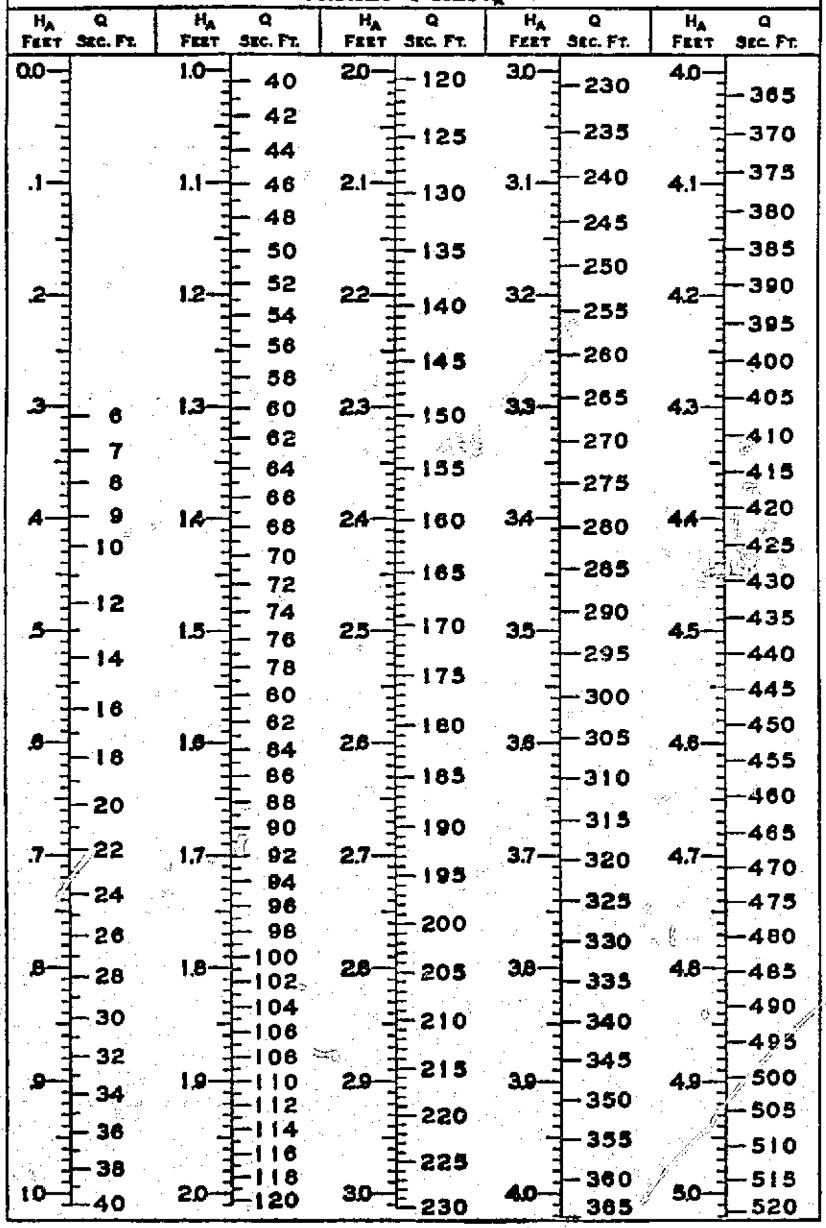


TABLE III

FREE-FLOW DISCHARGE 12-FOOT PARSHALL MEASURING FLUME

FORMULA $Q = 46.75\ H_A^{1.6}$

H_A Feet	Q Sec. Ft.
0.0	
.1	
.2	
.3	
	8
	10
.4	
	12
	14
.5	16
	18
.6	20
	22
	24
.7	26
	28
	30
.8	32
	34
	36
	38
.9	40
	42
	44
	46
1.0	48

H_A Feet	Q Sec. Ft.
1.0	
	48
	50
	52
1.1	54
	56
	58
	60
1.2	62
	64
	66
	68
	70
1.3	72
	74
	76
	78
1.4	80
	82
	84
	86
	88
1.5	90
	92
	94
	96
	98
1.6	100
	102
	104
	106
	108
1.7	110
	112
	114
	116
	118
1.8	120
	122
	124
	126
	128
1.9	130
	132
	134
	136
	138
	140
2.0	142

H_A Feet	Q Sec. Ft.
2.0	142
	144
	146
	148
	150
2.1	155
	160
2.2	165
	170
	175
2.3	180
	185
2.4	190
	195
	200
2.5	205
	210
2.6	215
	220
	225
2.7	230
	235
	240
2.8	245
	250
	255
2.9	260
	265
3.0	270

H_A Feet	Q Sec. Ft.
3.0	270
	275
	280
3.1	285
	290
	295
3.2	300
	305
	310
3.3	315
	320
	325
3.4	330
	335
	340
3.5	345
	350
	355
	360
3.6	365
	370
	375
3.7	380
	385
	390
3.8	395
	400
	405
	410
3.9	415
	420
	425
4.0	430

H_A Feet	Q Sec. Ft.
4.0	430
	435
	440
4.1	445
	450
	455
	460
4.2	465
	470
	475
	480
4.3	485
	490
	495
4.4	500
	505
	510
	515
4.5	520
	525
	530
	535
4.6	540
	545
	550
4.7	555
	560
	565
	570
4.8	575
	580
	585
	590
4.9	595
	600
	605
	610
5.0	615

TABLE IV

FREE-FLOW DISCHARGE 15-FOOT PARSHALL MEASURING FLUME

FORMULA $Q=57.81\,H_A^{1.6}$

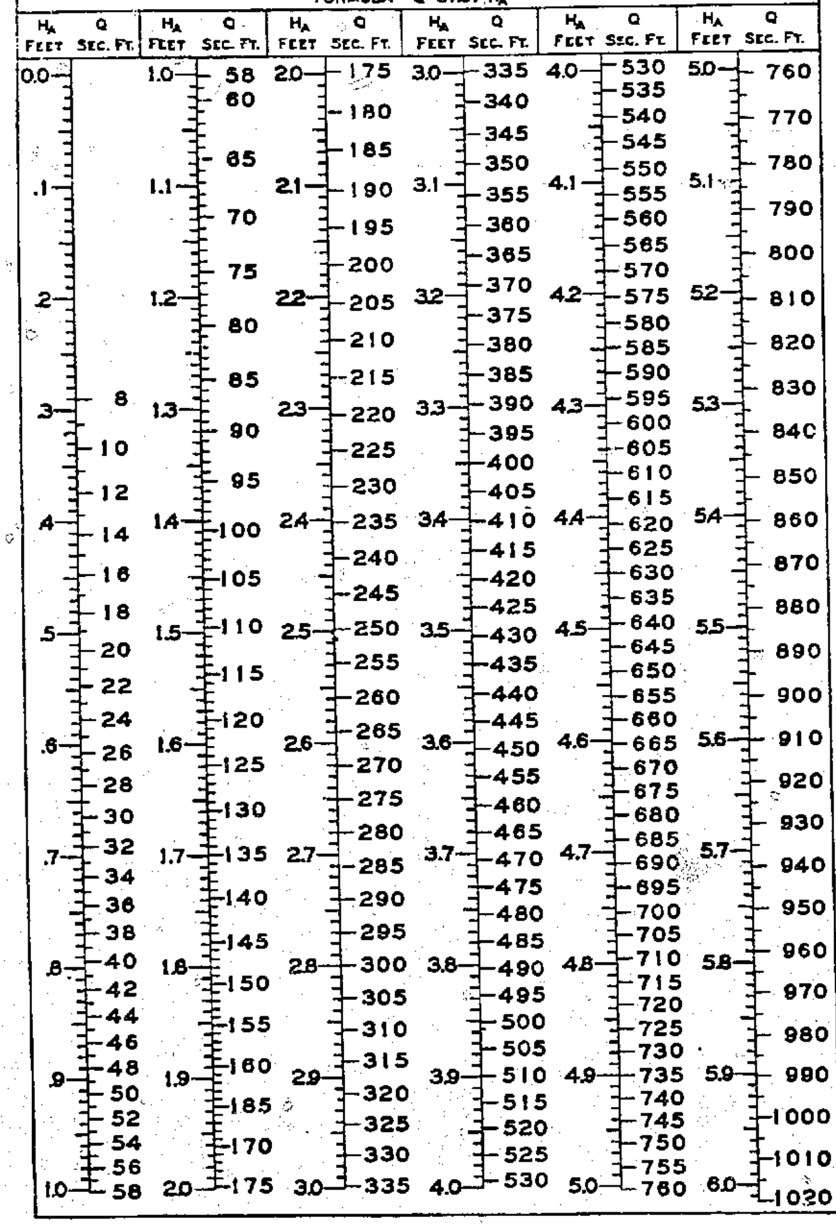


TABLE V
FREE-FLOW DISCHARGE 20-FOOT PARSHALL MEASURING FLUME
FORMULA $Q = 76.25\ H_A^{1.6}$

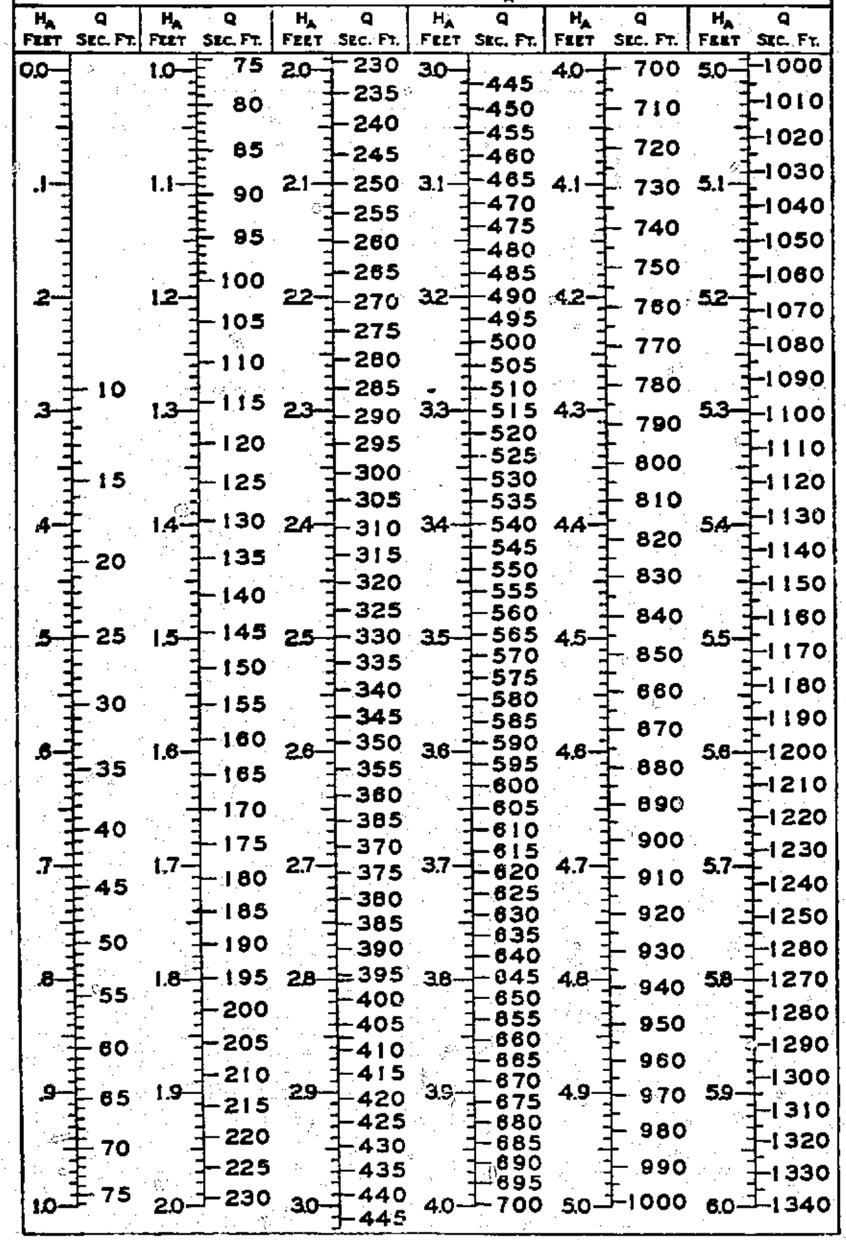


TABLE VI
FREE-FLOW DISCHARGE 25-FOOT PARSHALL MEASURING FLUME

FORMULA $Q = 94.69\, H_A^{1.6}$

H_A Feet	Q Sec. Ft.
0.0	
.1	
.2	
.3	15
	20
.4	
	25
.5	30
	35
	40
.6	
	45
	50
.7	55
	60
.8	65
	70
	75
.9	80
	85
	90
1.0	95

H_A Feet	Q Sec. Ft.
1.0	95
	100
	105
1.1	110
	115
	120
1.2	125
	130
	135
	140
1.3	145
	150
	155
1.4	160
	165
	170
	175
1.5	180
	185
	190
	195
1.6	200
	205
	210
	215
1.7	220
	225
	230
	235
1.8	240
	245
	250
	255
	260
1.9	265
	270
	275
	280
	285
2.0	290

H_A Feet	Q Sec. Ft.
2.0	290
	300
2.1	310
	320
2.2	330
	340
	350
2.3	360
	370
2.4	380
	390
	400
2.5	410
	420
	430
2.6	440
	450
2.7	460
	470
	480
2.8	490
	500
	510
2.9	520
	530
	540
3.0	550

H_A Feet	Q Sec. Ft.
3.0	550
	560
	570
3.1	580
	590
	600
3.2	610
	620
	630
3.3	640
	650
	660
3.4	670
	680
	690
3.5	700
	710
	720
	730
3.6	740
	750
	760
3.7	770
	780
	790
3.8	800
	810
	820
	830
3.9	840
	850
	860
4.0	870

H_A Feet	Q Sec. Ft.
4.0	870
	880
	890
	900
4.1	910
	920
	930
4.2	940
	950
	960
	970
4.3	980
	990
	1000
4.4	1010
	1020
	1030
	1040
4.5	1050
	1060
	1070
	1080
4.6	1090
	1100
	1110
	1120
4.7	1130
	1140
	1150
	1160
4.8	1170
	1180
	1190
4.9	1200
	1210
	1220
	1230
	1240
5.0	1250

H_A Feet	Q Sec. Ft.
5.0	1250
	1260
	1270
	1280
5.1	1290
	1300
	1310
	1320
5.2	1330
	1340
	1350
	1360
5.3	1370
	1380
	1400
	1400
5.4	1410
	1420
	1430
	1440
5.5	1450
	1460
	1470
	1480
5.6	1490
	1600
	1510
	1620
	1530
5.7	1640
	1650
	1560
	1570
5.8	1580
	1590
	1600
	1610
5.9	620
	1630
	1640
	1650
	1660
6.0	670

TABLE VII
FREE-FLOW DISCHARGE 30-FOOT PARSHALL MEASURING FLUME

FORMULA $Q = 113.13\,H_A^{1.6}$

H_A Feet (scale labels)	Q Sec. Ft. (scale labels)
0.0, .1, .2, .3, .4, .5, .6, .7, .8, .9, 1.0	15, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65, 70, 75, 80, 85, 90, 95, 100, 105, 110, 115

H_A Feet	Q Sec. Ft.
1.0, 1.1, 1.2, 1.3, 1.4, 1.5, 1.6, 1.7, 1.8, 1.9, 2.0	115, 120, 125, 130, 135, 140, 145, 150, 155, 160, 165, 170, 175, 180, 185, 190, 195, 200, 205, 210, 215, 220, 225, 230, 235, 240, 245, 250, 255, 260, 265, 270, 275, 280, 285, 290, 295, 300, 305, 310, 315, 320, 325, 330, 335, 340, 345

H_A Feet	Q Sec. Ft.
2.0, 2.1, 2.2, 2.3, 2.4, 2.5, 2.6, 2.7, 2.8, 2.9, 3.0	345, 350, 360, 370, 380, 390, 400, 410, 420, 430, 440, 450, 460, 470, 480, 490, 500, 510, 520, 530, 540, 550, 560, 570, 580, 590, 600, 610, 620, 630, 640, 650, 660

H_A Feet	Q Sec. Ft.
3.0, 3.1, 3.2, 3.3, 3.4, 3.5, 3.6, 3.7, 3.8, 3.9, 4.0	660, 670, 680, 690, 700, 710, 720, 730, 740, 750, 760, 770, 780, 790, 800, 810, 820, 830, 840, 850, 860, 870, 880, 890, 900, 910, 920, 930, 940, 950, 960, 970, 980, 990, 1000, 1010, 1020, 1030, 1040

H_A Feet	Q Sec. Ft.
4.0, 4.1, 4.2, 4.3, 4.4, 4.5, 4.6, 4.7, 4.8, 4.9, 5.0	1040, 1050, 1060, 1070, 1080, 1090, 1100, 1110, 1120, 1130, 1140, 1150, 1160, 1170, 1180, 1190, 1200, 1210, 1220, 1230, 1240, 1250, 1260, 1270, 1280, 1290, 1300, 1310, 1320, 1330, 1340, 1350, 1360, 1370, 1380, 1390, 1400, 1410, 1420, 1430, 1440, 1450, 1460, 1470, 1480, 1490

H_A Feet	Q Sec. Ft.
5.0, 5.1, 5.2, 5.3, 5.4, 5.5, 5.6, 5.7, 5.8, 5.9, 6.0	1490, 1500, 1510, 1520, 1530, 1540, 1550, 1560, 1570, 1580, 1590, 1600, 1610, 1620, 1630, 1640, 1650, 1660, 1670, 1680, 1690, 1700, 1710, 1720, 1730, 1740, 1750, 1760, 1770, 1780, 1790, 1800, 1810, 1820, 1830, 1840, 1850, 1860, 1870, 1880, 1890, 1900, 1910, 1920, 1930, 1940, 1950, 1960, 1970, 1980, 1990

TABLE VIII

FREE-FLOW DISCHARGE 40-FOOT PARSHALL MEASURING FLUME

FORMULA $Q = 150.00\ H_A^{1.6}$

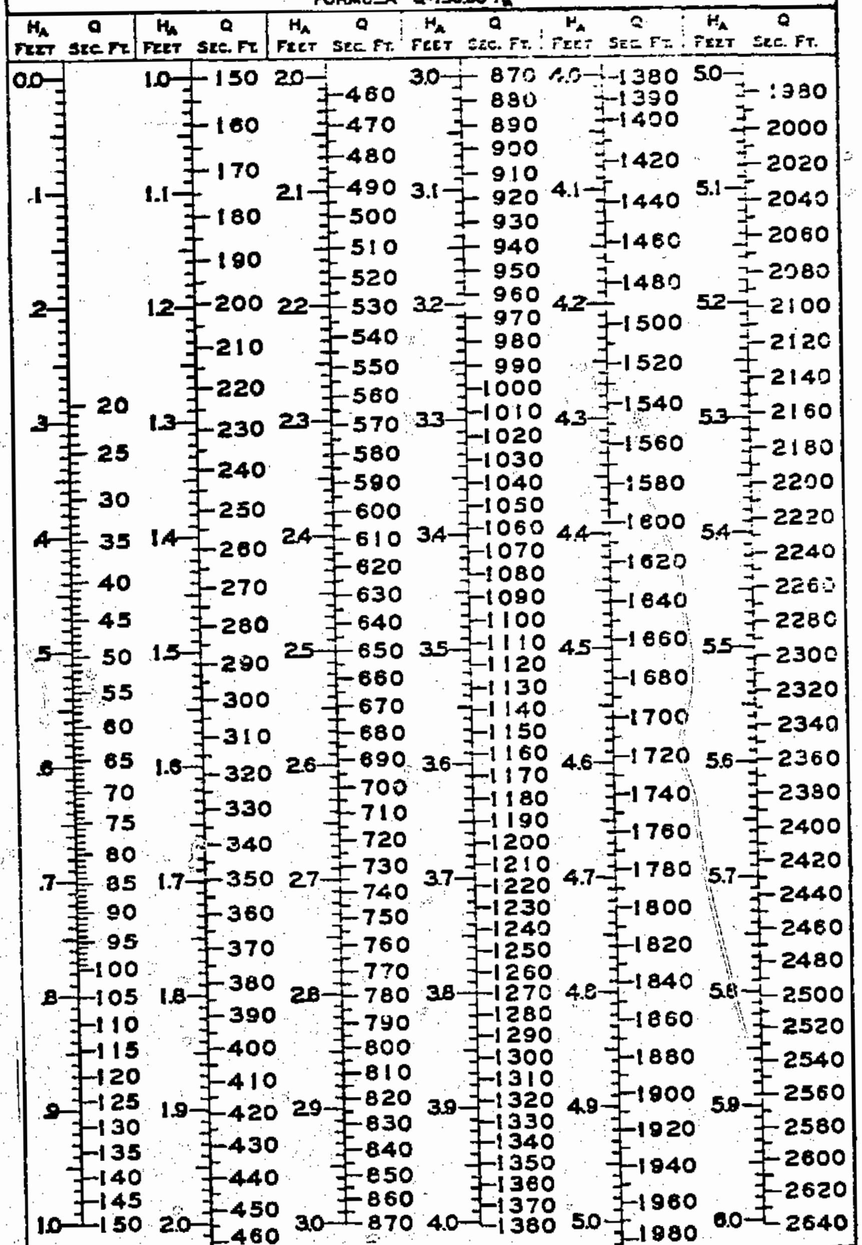


TABLE IX
FREE-FLOW DISCHARGE 50-FOOT PARSHALL MEASURING FLUME

FORMULA $Q = 186.88\, H_A^{1.6}$

H_A Feet	Q Sec. Ft.
0.0	
.1	
.2	
.3	25
	30
	35
	40
.4	45
	50
	55
	60
.5	65
	70
	75
	80
.6	85
	90
	95
	100
.7	105
	110
	115
	120
	125
.8	130
	135
	140
	145
	150
	155
.9	160
	165
	170
	175
	180
1.0	185
	190

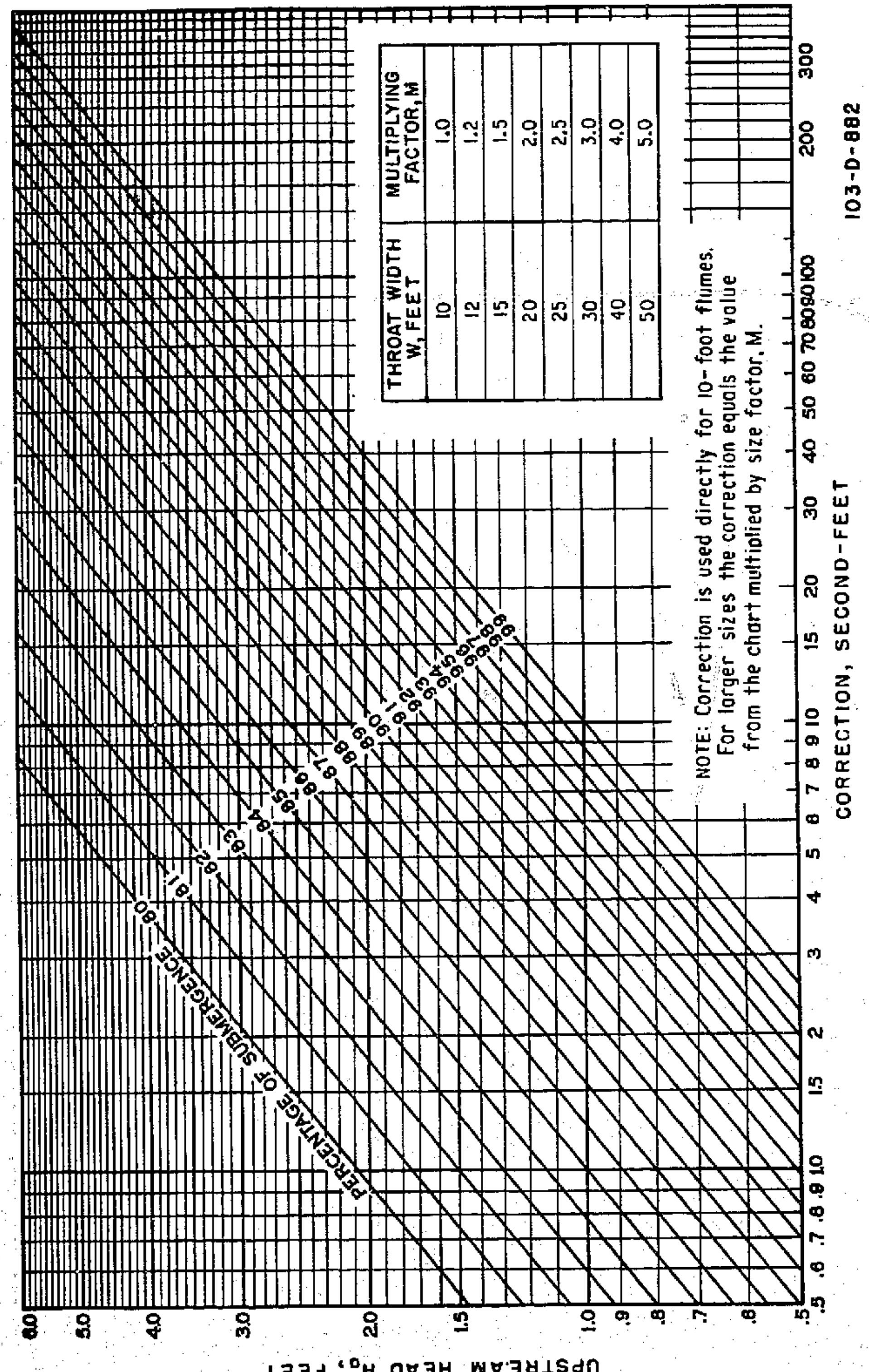
H_A Feet	Q Sec. Ft.
1.0	190
	195
	200
	210
1.1	220
	230
	240
1.2	250
	260
	270
1.3	280
	290
	300
	310
1.4	320
	330
	340
	350
1.5	360
	370
	380
	390
1.6	400
	410
	420
	430
1.7	440
	450
	460
	470
1.8	480
	490
	500
	510
1.9	520
	530
	540
	550
	560
2.0	570

H_A Feet	Q Sec. Ft.
2.0	570
	580
	590
	600
2.1	610
	620
	630
	640
	650
2.2	660
	670
	680
	690
	700
2.3	710
	720
	730
	740
	750
2.4	765
	780
	795
2.5	810
	825
	840
	855
2.6	870
	885
	900
2.7	915
	930
	945
	960
2.8	975
	990
	1005
2.9	1020
	1035
	1050
	1065
3.0	1080
	1095

H_A Feet	Q Sec. Ft.
3.0	1095
	1110
	1125
3.1	1140
	1155
	1170
	1185
3.2	1200
	1215
	1230
	1245
3.3	1260
	1275
	1290
	1305
3.4	1320
	1335
	1350
	1365
3.5	1380
	1395
	1410
	1425
	1440
3.6	1455
	1470
	1485
	1500
3.7	1515
	1530
	1545
	1560
3.8	1575
	1590
	1605
	1620
	1635
3.9	1650
	1665
	1680
	1695
4.0	1710
	1725

H_A Feet	Q Sec. Ft.
4.0	1725
	1740
	1760
4.1	1780
	1800
	1820
	1840
4.2	1860
	1880
	1900
4.3	1920
	1940
	1960
	1980
4.4	2000
	2020
	2040
	2060
4.5	2080
	2100
	2120
4.6	2140
	2160
	2180
	2200
4.7	2220
	2240
	2260
	2280
4.8	2300
	2320
	2340
	2360
4.9	2380
	2400
	2420
	2440
5.0	2460

H_A Feet	Q Sec. Ft.
5.0	2460
	2480
	2500
	2520
5.1	2540
	2560
	2580
	2600
5.2	2620
	2640
	2660
	2680
5.3	2700
	2720
	2740
	2760
5.4	2780
	2800
	2820
	2840
5.5	2860
	2880
	2900
	2920
5.6	2940
	2960
	2980
	3000
5.7	3020
	3040
	3060
	3080
	3100
5.8	3120
	3140
	3160
	3180
5.9	3200
	3220
	3240
	3260
6.0	3280
	3300



Correction diagram for submerged flow for large flumes 10 to 50 feet in width.

shown in terms of 10-foot lengths of crest, the correction must be multiplied by 2 for the 20-foot flume. The resulting correction is therefore 145 cfs, and the corrected discharge is 1,037 - 145 = 892 cfs. Because this corrected discharge value is less than 950 cfs the assumed submergence is too great and another trial calculation is necessary. For 88 percent submergence, H_a = 5.23 and the discharge is 972 cfs, which is greater than 950 cfs. For 89 percent the discharge is 934 cfs. The submergence for 950 cfs will therefore be between 88 and 89 percent.

The increase in depth upstream from the structure (loss of head) should next be investigated using Figure 14. For a 20-foot flume set 1.4 feet above the bottom of the channel, discharging 950 cfs with a submergence of 90 percent, the head loss shown in Figure 14 is about 1 foot. This value is obtained as follows: Intersect the vertical 90 percent submergence line with the curved discharge line for 950 cfs (halfway between 900 and 1,000). Project a horizontal line through this point to intersect the diagonal line for W = 20 feet. This point has a value 0.9 foot on the "loss of head" scale below.

Referring to Figure 6, X = 1.4; H_a = 5.2; L = 0.9; the total depth is 7.5 feet. This is 1.5 feet greater than the gage height for 950 cfs. If this extra depth would cause overtopping, flooding, or other problems upstream, a larger flume should be investigated.

Using similar procedures for a 25-foot flume, X is found to be 1.7 feet, submergence will be 91 percent for 950 cfs, and L will be 0.8 foot. The upstream depth will therefore be (1.7 + 4.7 + 0.8) = 7.2 feet; about a third of a foot lower if the 25-foot flume is built. Check this analysis and investigate other larger flume sizes as an exercise.

The decision as to which flume size to use will depend therefore on whether the extra depth which occurs upstream for the 20-foot flume can be tolerated and whether the greater cost of the 25-foot or larger flume would be excessive. An analysis of depths versus discharges over the entire range of gage heights provided in the original data might need to be made, using the methods just described, before coming to a final decision. Any of several sizes of flume would satisfactorily measure the range of flows.

STILLING WELLS

To obtain accurate flow measurements in any flume, it is necessary to accurately measure the effective heads, H_a and H_b, but this is particularly true in the larger sizes. If staff gages are used on the inside face of the walls, only approximate head determinations can usually be made because of waves and fluctuations at the upstream gage and turbulent conditions within the throat section at the downstream gage. The stilling wells (and flushing system) specified in the standard drawings of

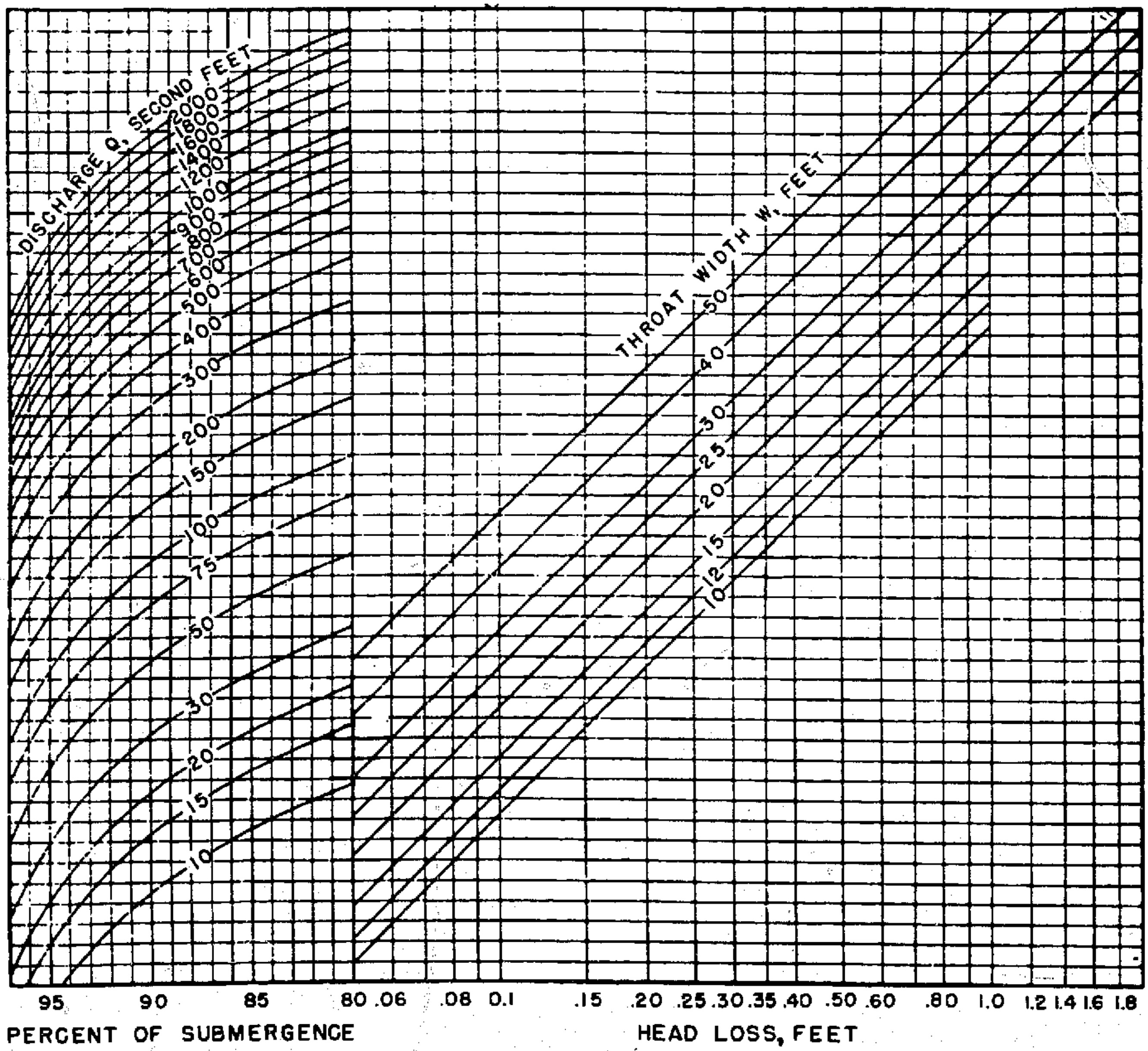


Diagram for total loss of head through large flumes 10 to 50 feet in width. Note: Loss of head is the same as the increase in depth upstream from flume after the flume is installed.

large Parshall flumes should always be used. Before making a measurement the wells should be flushed with clear water to be sure they are free of sediment or other foreign material, including ice in the pipelines, which could be the cause of erroneous head determinations. The head recording equipment should be checked and serviced regularly. Cross checks should be made between the staff gages, hook gages, plumb bobs, recorder values, and any other discharge indicators to expose possible system errors.

DISCHARGE EQUATION AND TABULAR VALUES FOR FREE FLOWS

The equation which expresses the true relationship between head and discharge for the small size flumes (W = 1 to 8 feet) is

$$\text{Discharge } Q = 4WH_a^{1.522W^{0.026}}$$

If this equation is used to compute discharges through the large flumes (W = 10 to 50 feet) the computed value is always larger than the actual discharge. Therefore, the more accurate equation developed for use with flumes ranging in size from 8 to 50 feet should be used. The equation is also simpler to use.

$$\text{Discharge } Q = (3.6875W + 2.5)\, H_a^{1.6}$$

The difference in computed discharges obtained by using the two above formulas for an 8-foot flume is less than 1 percent; the difference becomes greater as the flume size increases.

Because of the difficulties in regularly using these equations, discharge tables have been prepared for use with the large flumes. Tables II through IX give the free discharge in cfs for a complete range of H_a values for flumes of throat widths 8 to 50 feet. It is possible by estimation to read these tables with an error of less than 1 percent.

DETERMINATION OF SUBMERGED FLOW DISCHARGES

The method of determining the discharge rate for submerged flows in large flumes is similar to that described for the smaller flumes. The ratio H_b to H_a expressed in percent and the H_a value are used in the graph of Figure 13 to obtain the correction to be subtracted from the free flow discharge determined from H_a and Tables II to IX. The correction values, indicated in cfs along the base of the diagram in Figure 13 give the number of cfs to be subtracted for each 10 feet of crest width, W. As an aid in determining the multiplying factor, the table in Figure 13 should be used.

For example, a 30-foot flume has a multiplying factor of 3.0. Corrections obtained from Figure 13 should be multiplied by 3.0 to get the total correction for a 30-foot flume.

To determine the discharge for submerged flow through a 20-foot flume when H_a is 3.25 feet and H_b is 3.06 feet, first determine the submergence ratio.

$$\frac{H_b}{H_a} = \frac{3.06}{3.25} = 0.941 = 94.1 \text{ percent}$$

Enter at the left side of the diagram of Figure 13, and at H_a = 3.25, project a horizontal line to intersect the 94 percent line, then continue on to 1/10 of the distance between the 94 and the 95 percent lines. Vertically below this point on the horizontal scale is the correction value 56 cfs. For a 20-foot flume the multiplying factor is 2.0; the total correction is therefore 2 x 56 = 112 cfs. The free discharge value from Table V for H_a = 3.25 is about 503 cfs. The submerged flow, therefore, is (503 - 112) or 391 cfs. Additional examples of the use of Figure 13 and Table V are given in the section on "Size Selection and Setting of Large Flumes."

VERY SMALL FLUMES--1-, 2-, 3-INCH SIZES

The original development work on Parshall flumes did not include the 2- and 1-inch throat width sizes. Over the years, however, a need for these smaller sizes became apparent, and calibration and standardization were completed and published by the Colorado Agricultural Experiment Station in 1957. The work included some redesign and reevaluation of the 3-inch flume because of difficulties reported in the use of the H_b gage. The capacity of the "very small"* flumes ranges from about 0.01 cfs for H_a = 0.10 foot in the 1-inch flume, to the 1.134 cfs for H_a = 1.0 foot in the 3-inch flume. The capacity of each size of flume overlaps that of the next size by about half the discharge range. To visualize the magnitude of the very small flows, it may be helpful to know that 0.01 cfs is (approximately) equal to 5 gallons per minute.

Turbulence and the relatively deep and narrow throat section make reading the H_b gage extremely difficult in the 1- and 2-inch flumes and difficult on the 3-inch flume. Consequently, the H_c gage, located downstream near the end of the diverging portion of the flume, Figure 15, was added to the very small flumes. This gage may be read in place of the H_b gage. H_c readings are then converted to H_b readings, using a graph,

*The term "very small" is used in this publication to differentiate between the two groups of flumes originally referred to as "small flumes."

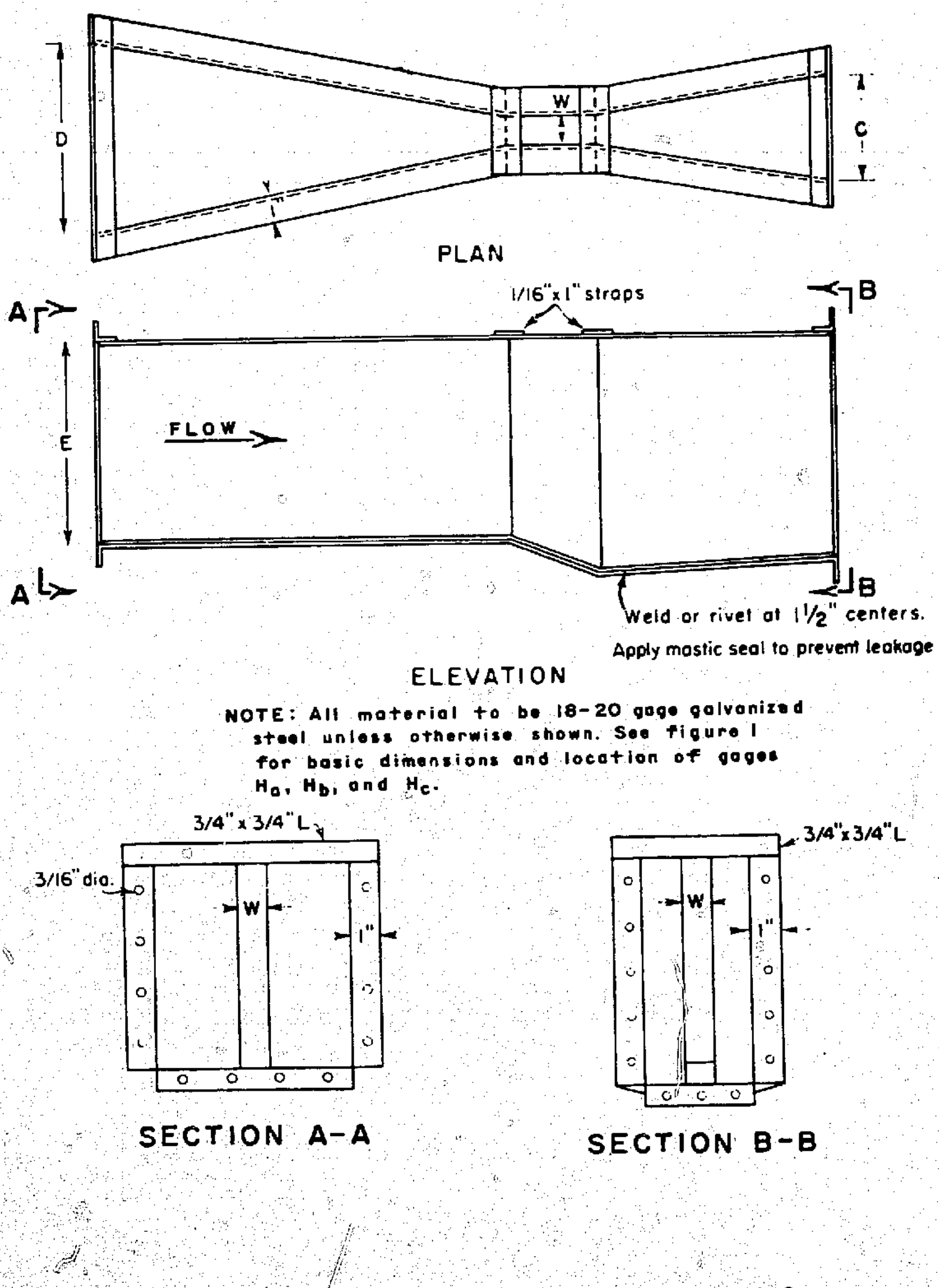


Figure 15. Construction of 1-, 2-, and 3-inch standard Parshall flumes

as explained later. The converted H_b values are used in the discharge determinations.

CONSTRUCTION OF FLUMES

Experience has shown that the very small flumes are best constructed of galvanized sheet metal. Folded 18-20 gage steel reinforced with 3/4-inch steel angles, using welds or rivets, is a recommended standard of construction. One-inch folded flanges, punched for bolts, at each end and at the top and bottom of the flume for rigidity are also recommended, Figure 15. Because the flumes are small, their accuracy is dependent, to a large extent, on the accuracy with which the throat and adjacent features are constructed. Particular care should be taken to avoid twists or warps in the floor which prevent setting both ends of the flume level. A suggested tolerance on the throat width is plus or minus 1/64 inch; other dimensions plus or minus 1/32 inch. The throat sidewalls must be parallel and at right angles to the crest. The gages must be accurately located if the standard calibration curves are to be used for discharge determinations. Figure 1 shows the dimensions for the three sizes of flumes referred to in this publication as "very small" flumes.

DETERMINATION OF DISCHARGE

FREE FLOW

Tables 1, 2, and 3 give the free flow discharge in cfs for the very small flumes over the complete range of useful heads, H_a. Tables 4, 5, and 6 give the discharge in gallons per minute. If a flume throat width dimension is found to be inaccurate, the discharge rate may be corrected on the basis that each unit width of crest discharges the same quantity of water as every other unit. For example, a so-called 1-inch flume with a throat width of 1-1/16 inches would discharge 1.06 times the tabular discharge indicated for the particular H_a value, because the throat is (about) 6 percent oversize. The average width of the throat, determined by several measurements over the depth occupied by the flowing water, should be used to make the correction.

SUBMERGED FLOW

Submergence begins to reduce the discharge through the very small flumes when the submergence ratio exceeds 0.50, or 50 percent. To determine discharges for submerged flows, the heads H_a and H_b are used with Figures 16, 17, and 18, to obtain the discharge rate directly. H_a and H_b gages should be zeroed to give the depths above the elevation of the flume crest.

Figures 16, 17, and 18 should be used in the following manner. In a 3-inch flume, H_a = 0.20 foot and H_b = 0.17 foot; determine the discharge.

Table 1.--Free flow discharge through 1-inch Parshall measuring flume in second feet. Computed from formula $Q = .338\ H_a^{1.55}$

Upper head H_a	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
				Cubic Feet per Second						
Feet										
0.00						0.0032	0.0043	0.0055	0.0068	0.0081
0.10	0.0095		0.0126	0.0142	0.0160	.0179	.0196	.0216	.0237	.0257
0.20	.028	.030	.032	.035	.037	.039	.042	.045	.047	.050
0.30	.052	.055	.058	.061	.064	.066	.069	.072	.075	.078
0.40	.082	.085	.088	.091	.095	.098	.101	.105	.108	.112
0.50	.115	.119	.123	.126	.130	.134	.138	.141	.145	.149
0.60	.153	.157	.161	.165	.169	.173	.177	.182	.186	.190

Table 2.--Free flow discharge through 2-inch Parshall measuring flume in second feet. Computed from formula $Q = .676\ H_a^{1.55}$

Upper head H_a	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
				Cubic Feet per Second						
Feet										
0.00						0.0065	0.0087	0.0109	0.0135	0.0162
0.10	0.0191		0.0251	0.0284	0.0321	.0358	.0392	.0433	.0473	.0513
0.20	.055	.060	.065	.070	.074	.079	.094	.089	.094	.099
0.30	.105	.110	.116	.121	.127	.132	.139	.145	.151	.157
0.40	.163	.170	.176	.182	.189	.196	.203	.210	.217	.224
0.50	.230	.238	.245	.253	.260	.268	.275	.283	.290	.298
0.60	.306	.314	.322	.330	.338	.347	.355	.363	.372	.381
0.70	.389	.397	.406	.415	.424	.433	.442	.451	.459	.469

Table 3.--Free flow discharge through 3-inch Parshall measuring flume in second feet. Computed from formula $Q = .992\ H_a^{1.55}$

Upper head H_a	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
				Cubic Feet per Second						
Feet										
0.10	0.028	0.033	0.037	0.042	0.047	0.053	0.058	0.064	0.070	0.076
0.20	.082	.089	.095	.102	.109	.117	.124	.131	.138	.146
0.30	.154	.162	.170	.179	.187	.196	.205	.213	.222	.231
0.40	.241	.250	.260	.269	.279	.289	.299	.309	.319	.329
0.50	.339	.350	.361	.371	.382	.393	.404	.415	.427	.438
0.60	.450	.462	.474	.485	.497	.509	.522	.534	.546	.558
0.70	.571	.584	.597	.610	.623	.636	.649	.662	.675	.689
0.80	.702	.716	.730	.744	.757	.771	.786	.800	.814	.828
0.90	.843	.858	.872	.887	.902	.916	.931	.946	.961	.977
1.00	.992	1.007	1.023	1.038	1.054	1.070	1.086	1.102	1.118	1.134

Table 4.--Free Flow discharge through 1-inch Parshall measuring flume in gallons per minute. Computed from cfs x 448.8

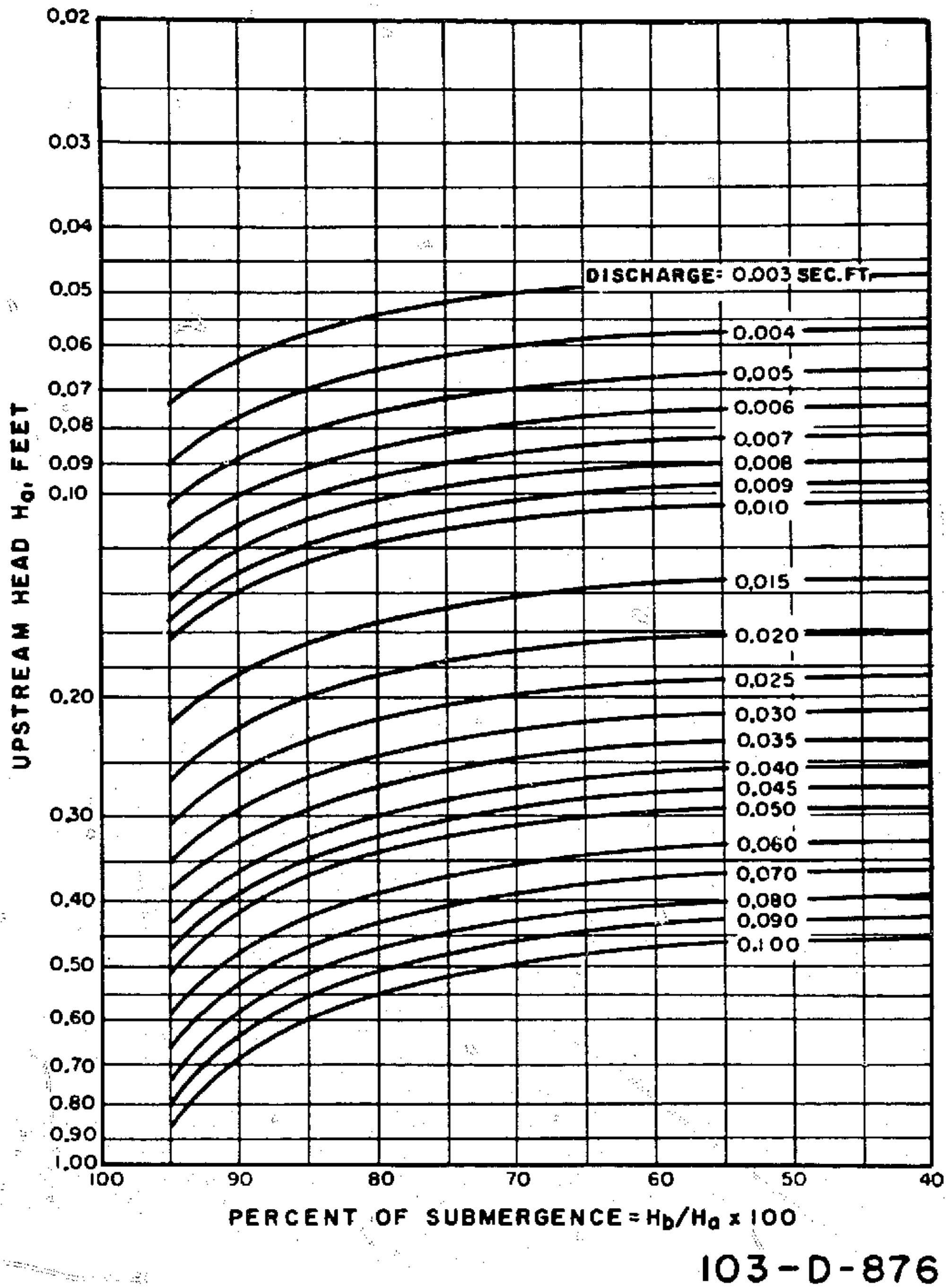
Upper head H_a	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
					Gallons per Minute					
Feet										
0.00						1.4	1.9	2.5	3.1	3.6
0.10	4.3		5.7	6.4	7.2	8.0	8.8	9.7	10.6	11.5
0.20	12.6	13.5	14.4	15.7	16.6	17.5	18.8	20.2	21.1	22.4
0.30	23.3	24.7	26.0	27.4	28.7	29.6	31.0	32.3	33.7	35.0
0.40	36.8	38.1	39.5	40.8	42.6	44.0	45.3	47.1	48.5	50.3
0.50	51.6	53.4	55.2	56.5	58.3	60.1	61.9	63.3	65.1	66.9
0.60	68.7	70.5	72.3	74.1	75.8	77.6	79.4	81.7	83.5	85.3

Table 5.--Free Flow discharge through 2-inch Parshall measuring flume in gallons per minute. Computed from cfs x 448.8

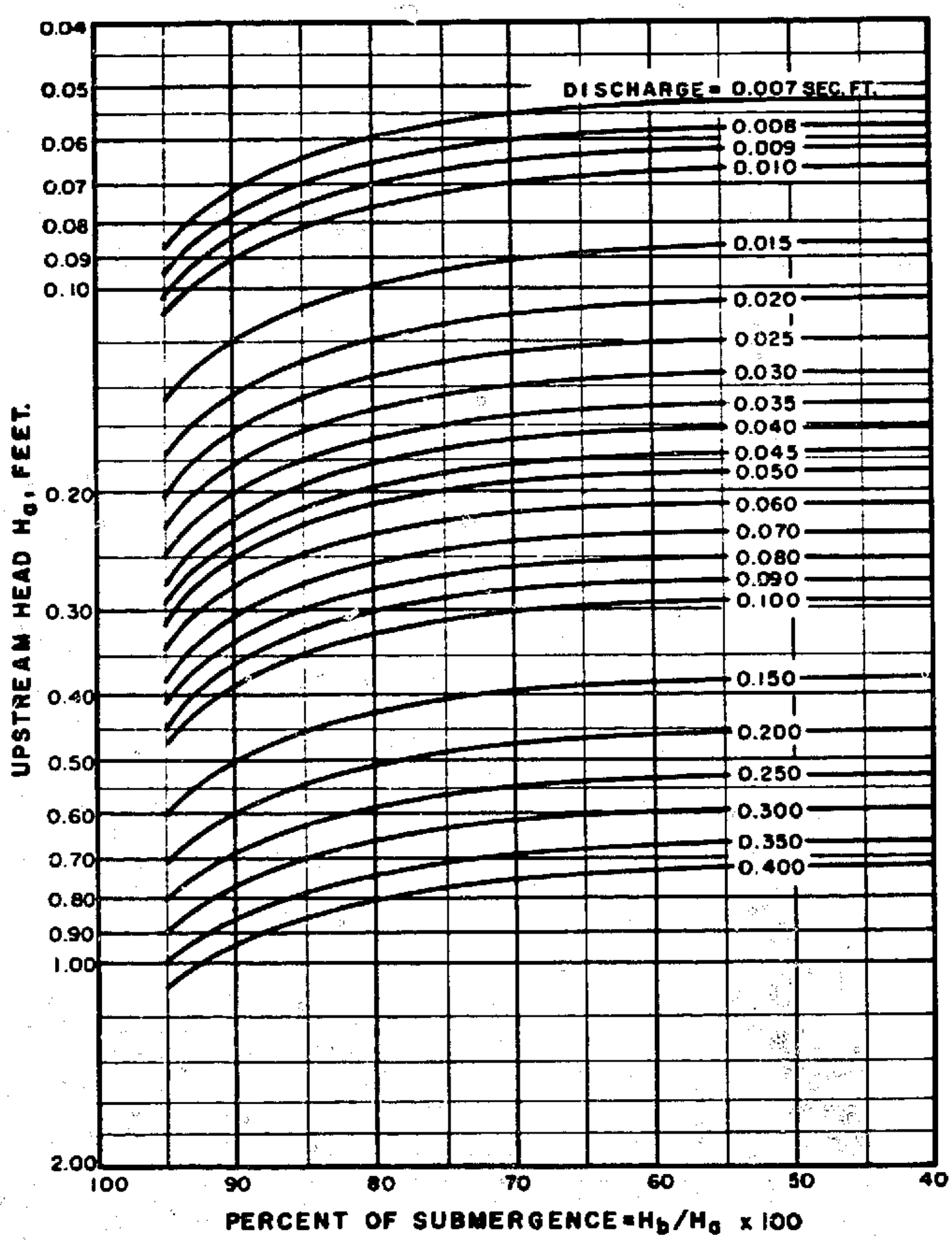
Upper head H_a	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
					Gallons per Minute					
Feet										
0.00						2.9	3.9	4.9	6.1	7.3
0.10	8.6		11.3	12.7	14.4	16.1	17.6	19.4	21.2	23.0
0.20	24.7	26.9	29.2	31.4	33.2	35.5	37.7	39.9	41.2	44.4
0.30	47.1	49.4	52.1	54.3	57.0	59.2	62.4	65.1	67.8	70.5
0.40	73.2	76.3	79.0	81.7	84.8	88.0	91.1	94.2	97.4	100.5
0.50	103.2	106.8	110.0	113.5	116.7	120.3	123.4	127.0	130.1	133.7
0.60	137.3	140.9	144.5	148.1	151.7	155.7	159.3	162.9	167.0	171.0
0.70	174.6	178.2	182.2	186.3	190.3	194.3	198.4	202.4	206.0	210.5

Table 6.--Free flow discharge through 3-inch Parshall measuring flume in gallons per minute. Computed from cfs x 448.8

Upper head H_a	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
					Gallons per Minute					
Feet										
0.10	12.6	14.8	16.6	18.8	21.1	23.8	26.0	28.7	31.4	34.1
0.20	36.8	39.9	42.6	45.8	48.9	52.5	55.7	58.8	61.9	65.5
0.30	69.1	72.7	76.3	80.3	83.9	88.0	92.0	95.6	99.6	103.7
0.40	108.2	112.2	116.7	120.7	125.2	129.7	134.2	138.7	143.2	147.7
0.50	152.1	157.1	162.0	166.5	171.4	176.4	181.3	186.3	191.6	196.6
0.60	202.0	207.3	212.7	217.7	223.1	228.4	234.3	239.7	245.0	250.4
0.70	256.3	262.1	267.9	273.8	279.6	285.4	292.3	297.1	303.9	309.2
0.80	315.1	321.3	327.6	333.9	339.7	346.0	352.8	359.0	365.3	371.6
0.90	378.3	385.1	391.4	398.1	404.8	411.1	417.8	424.6	431.3	438.5
1.00	445.2	451.9	459.1	465.9	473.0	480.2	487.4	494.6	501.8	508.9



Rate of submerged flow through a 1-inch Parshall flume



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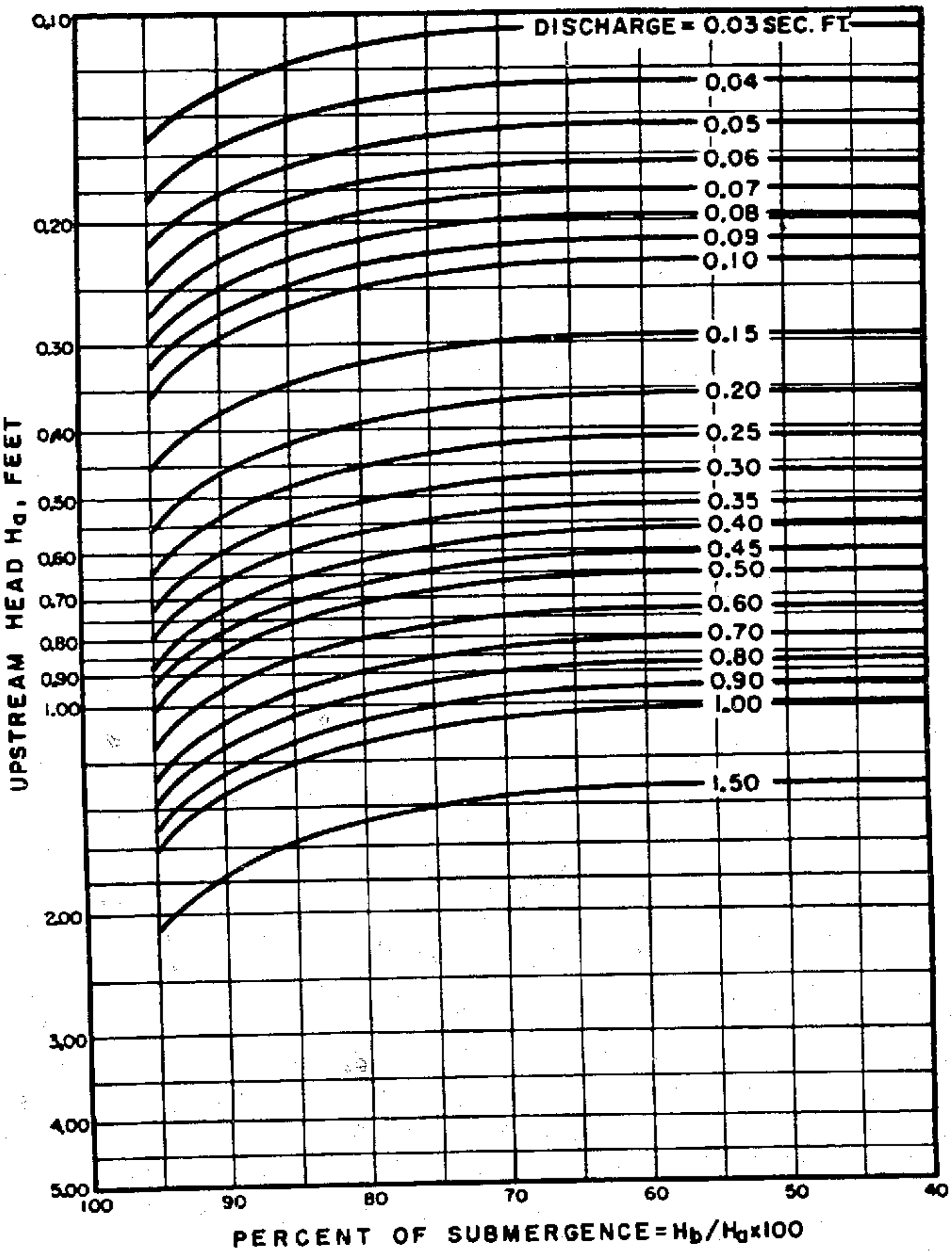
Rate of submerged flow through a 2-inch Parshall flume



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Rate of submerged flow through a 3-inch Parshall flume

Submergence $\frac{H_b}{H_a} = \frac{0.17}{0.20} = 0.85 = 85$ percent. Enter Figure 18 with the upper head value, $H_a = 0.20$; intersect the horizontal line through this point with the vertical line for $\frac{H_b}{H_a} = 85$ percent. Note that this intersection point lies about 7/10 of the distance from the curved discharge line for 0.06 cfs, toward the 0.07-cfs line. The interpolated discharge value is, therefore, 0.067 cfs. It should be noted that the free discharge value for $H_a = 0.20$ foot, in Table 3, is 0.082 cfs.

These figures indicate that submergence of 85 percent reduced the apparent discharge from 0.082 cfs to 0.067 cfs, a reduction of some 18 percent.

If the H_b gage is difficult to read accurately, the H_c gage readings may be used to improve the accuracy of the discharge measurement. The H_c gage location is given in Figure 1. The relationship between H_b and H_c gage values, determined during the calibration tests is given in Figure 19. A straight edge placed on the curve in the graph shows that the value of H_c, on the vertical scale, is slightly greater than the H_a value shown on the horizontal scale. There is some recovery of velocity head at the H_c gage because of the expanding flow area (a reduction in velocity always produces a greater depth of flow).

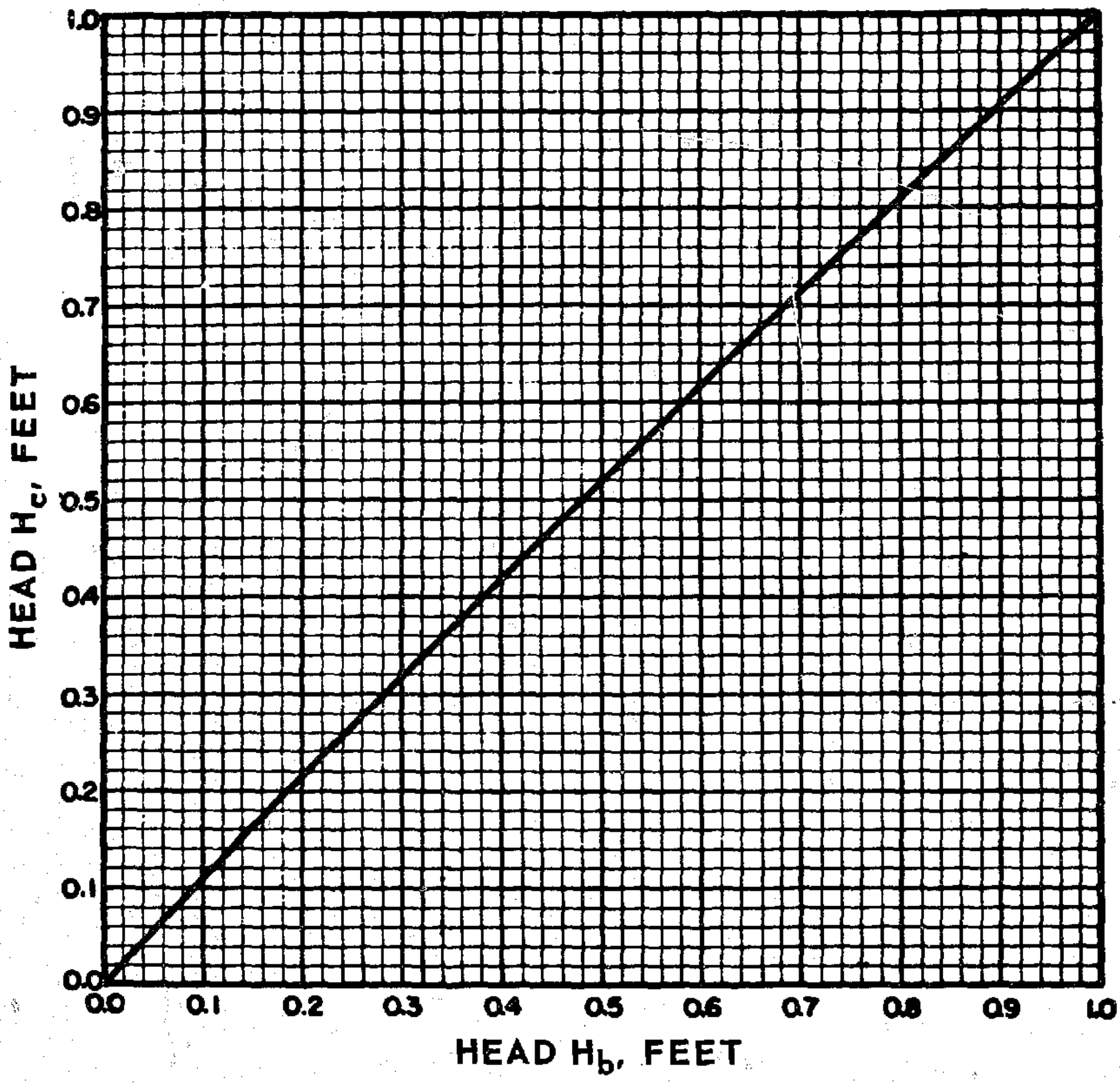
Proper use of Figure 19 is illustrated in the following problem. Determine the discharge if $H_a = 0.36$ and $H_c = 0.32$ in a 3-inch flume. Enter Figure 19 with $H_c = 0.32$ foot; the corresponding H_b is 0.30 foot. $\frac{H_b}{H_a} = 0.83 = 83$ percent. From Figure 18, the line $H_a = 0.36$ intersected with the 83 percent submergence line, gives a discharge of 0.17 cfs (interpolated).

SIZE-SELECTION SETTING AND USAGE

The size of the very small flume to be selected depends on the quantity of water to be measured and on the elevation the crest is to be set. Whenever possible the flume should be set to operate under free flow conditions. If this is not possible, the setting should be made to create minimum submergence.

An example will indicate the method of selecting a flume size and making the proper setting. A discharge of about 0.14 cfs is to be measured in a channel 0.52 foot deep, flowing 0.30 foot deep.

In Table 1, for a discharge of 0.15 cfs through the 1-inch flume, $H_a = 0.6$ foot; this is too deep, the banks would be overtopped. In



103-D-879

Relationship of H_c and H_b gages for 1-, 2-, and 3-inch Parshall flumes for submergences greater than 60 percent

Table 2, $H_a = 0.38$ and in Table 3, $H_a = 0.30$; therefore, either a 2- or 3-inch flume could be used. The submerged flow condition should be investigated to determine which size would be preferable.

For the very small flumes, a reduction in discharge is evident as soon as the submergence exceeds about 50 percent, or $\frac{H_b}{H_a} = 0.5$. In this example, since $H_b = H_a \times 0.5$, H_b should not exceed $0.38 \times 0.5 = 0.19$ in the 2-inch flume and $0.30 \times 0.5 = 0.15$ in the 3-inch flume. In Figure 6, the relationship between flume dimensions is

$$X = D - H_b$$

(X is the distance the crest is placed above the channel bottom.)

For the 2-inch flume, $X = 0.30 - 0.19 = 0.11$ foot.
For the 3-inch flume, $X = 0.30 - 0.15 = 0.15$ foot.

The upstream depth for the 2-inch flume will be

$$H_a + X = 0.38 + 0.11 = 0.49 \text{ foot}$$

For the 3-inch flume $H_a + X = 0.30 + 0.15 = 0.45$ foot. The small difference in upstream depths would probably help in ruling in favor of the 2-inch flume because of lower cost; also, the channel banks will not be overtopped.

In installing the very small flumes, the approach floor should be used as an index for leveling rather than, say, using the tops of the walls as a support for a carpenter's level. Extraneous parts of the flume may have no known relationship to the plane of the crest. Procedures should be such that the crest is the leveled portion of the flume. Careful leveling is necessary in both longitudinal and transverse directions if standard discharge tables are to be used with the flume. In setting flumes in the field, two main precautions should be taken. The first, the flume should be set on a solid foundation, perhaps on footings or pilings, to prevent settlement or heaving. Second, a collar bolted to the upstream and/or downstream flanges extending from the bottom and sides of the flume, well out into the channel banks and down into the earth could help to prevent flow from bypassing the structure and eroding the foundation. Careful zeroing and reading of the staff gages are also "musts" if accuracy is to be obtained. An error of 0.01 foot (about one-eighth inch) in setting the flume, the gage zero, or with a 0.01 foot error in reading the staff gage could result in an 8 percent discharge determination error in the 2-inch flume (mid-range). In the 1-inch flume, similar setting and reading errors could result in a 12 percent discharge determination error. The importance of using extra precautions in setting and reading gages in these small flumes is evident.

In reading a staff gage it should be noted whether the gage tends to attract the water (crawl up the gage) to give a high reading, or tends to repel the water to give a low reading. Visual compensation may be possible to improve accuracy. Figure 20 illustrates the meniscus shapes (exaggerated for demonstration) that often occur at a staff gage

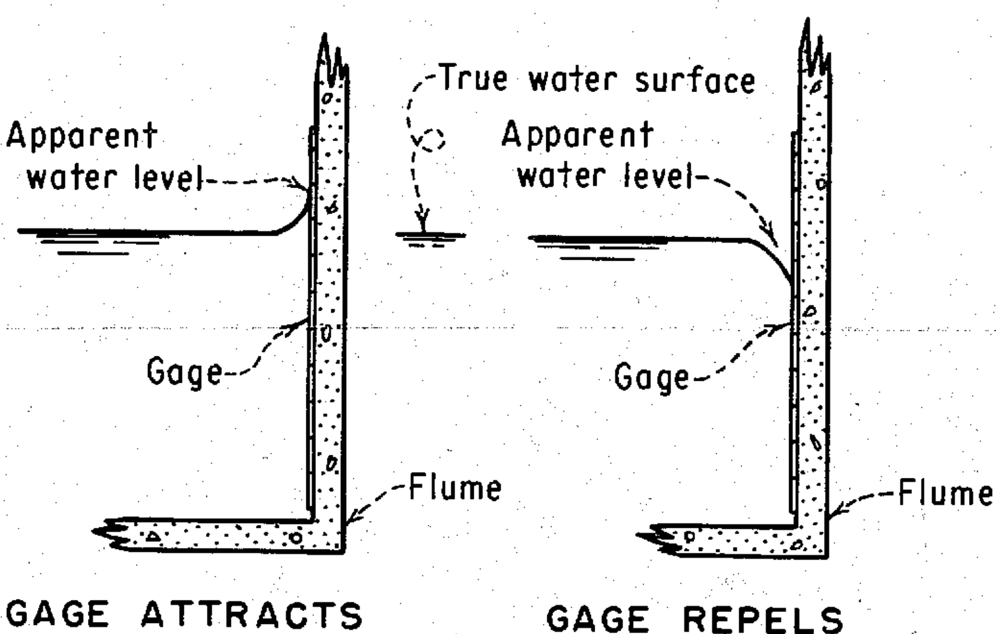


Figure 20

Sections through flume walls. Meniscus effects at staff gages.

ACKNOWLEDGEMENTS

The writing in this discourse on Parshall flumes is original, but the source of some of the tabular and other material is from the following:

1. R. L. Parshall--Bulletin 426-A, March 1953 (Reprint of Bulletin 386), Parshall Flumes of Large Size, Colorado Agricultural Experiment Station, Colorado State University, Fort Collins, Colorado

2. R. L. Parshall--Soil Conservation Circular No. 843, Measuring Water in Irrigation Channels with Parshall Flumes and Small Weirs, U.S. Department of Agriculture

3. A. R. Robinson--Parshall Measuring Flumes of Small Sizes, Technical Bulletin No. 61, 1957, Agricultural Experiment Station, Colorado State University, Fort Collins, Colorado

4. A. R. Robinson--Simplified Flow Corrections for Parshall Flumes Under Submerged Conditions, Civil Engineering, ASCE, September 1965

5. F. Caplan--Nomograph for Free Flow Discharge Through a Parshall Flume - Water and Sewage Works, May 1963

6. A. J. Peterka--Engineering Monograph No. 25, Bureau of Reclamation, "Hydraulic Design of Stilling Basins and Energy Dissipators"

CONCLUSIONS

The accurate measurement of water is both an art and a science and requires the full understanding of the irrigation operator. To achieve success, the operator must understand the workings of his water-measuring equipment and must apply his knowledge in a practical way to the degree necessary to achieve the desired accuracy of measurement. He must be alert to ever-changing flow conditions and make the necessary alterations in equipment or procedures. He must exercise proper judgment in all matters and this is best accomplished when as many facts as possible regarding the behavior of water are known to him.

It is difficult, if not impossible, to establish definite rules which apply generally to water-measurement procedures and equipment. Similarly, one measurement device cannot be recommended over any other device until all variables at the particular installation site are considered and properly weighted. It is therefore necessary for each operator to learn as much as possible about the device he is using and to evaluate the effect of each variable (at the particular site) on the measurement he is making.

Each operator must learn to look objectively at his equipment and procedures. He must be able to "see" that his equipment is run down and in need of maintenance or that his measurement procedures are not compatible with what he is trying to measure. He should become familiar with various types of measuring equipment, learn the advantages and disadvantages of each, and decide whether the existing equipment is the best for the job at hand. He should try to find fault with his equipment and every step he uses to make a discharge measurement, and try to improve wherever possible. This means that he must understand the basic measurement he is trying to make and then modify, if necessary, his methods of getting it. He should try to understand why he is doing the things he does and develop confidence in his knowledge. He should read available literature as much as possible to get background information on water measurement. He will thereby not only obtain more meaningful information, but will also have the satisfaction of knowing his job is well done.

REFERENCES

The following publications are suggested as an aid in acquiring background in water measurement practices. The items have been selected to provide practical help or background information, or both, and should be of value to both new and experienced personnel. Copies may be obtained from a public library or from the sources listed. The textbook chapters referred to are not difficult to read and will supply background material in gage reading problems and examples, orifice theory, gate and meter problems, and information on head losses and other flow phenomena.

The Bureau reports contain practical information pertaining to discharge measurements through metergates and constant-head turnout orifices which can be applied to field problems directly. In fact, a complete analysis of the flow through screw lift vertical metergates is contained in these reports and may cover specific problems encountered in your area. The report on the constant-head orifice turnout contains calibration data and operating instructions.

The textbooks are available in most public libraries and the reports should be on file in your Bureau Regional Office or Project Office, or can be obtained from the Technical and Foreign Services Branch, Bureau of Reclamation, Denver Federal Center, Denver, Colorado 80225.

Items 1, 2, and 8 are valuable as handbooks and would be of permanent value as reference books. The principles discussed will be found helpful in understanding and operating almost every type of measuring device. Item 8 contains a wealth of information on stream gaging techniques developed by USGS.

1. Water Measurement Manual, United States Department of the Interior, Bureau of Reclamation, Second Edition, Denver, Colorado, 1966

2. Handbook of Hydraulics, H. W. King, Third Edition, McGraw-Hill Book Company, Incorporated, New York City, New York

3. The Discharge of Three Commercial Cipoletti Weirs, R. B. Van Horn, Engineering Experiment Station Series Bulletin No. 85, University of Washington, Seattle, Washington, November 1935

4. Precise Weir Measurements, E. W. Schoder and K. B. Turner

5. Accuracy of the V-Notch Weir Method of Measurement, Transactions ASME, Volume 48, 1926, page 939

6. Weir Experiments, Coefficients, and Formulas, R. E. Horton, Water Supply Paper 200, Geological Survey, U.S. Department of the Interior, Washington, D.C.

7. Stream-gaging Procedure, D. M. Corbett, United States Department of the Interior, Geological Survey Water Supply Paper 888, 1945, U.S. Government Printing Office, Washington, D.C.

8. Stream Flow, Nathan C. Grover and Arthur D. Harrington, Reprint by Dover Publications, New York

9. Improving the Distribution of Water to Farmers by Use of the Parshall Measuring Flume, R. L. Parshall, Bulletin 438, U.S. Department of Agriculture, Colorado State University, Fort Collins, Colorado

10. Measurement of Debris-laden Stream Flow with Critical Depth Flumes, H. G. Wilm, V. S. Cotton, and H. S. Storey, Transactions ASCE, Volume 103, 1938, page 1237

11. World Practices in Water Measurement at Turnouts, C. W. Thomas, Proceedings ASCE, Journal of the Irrigation and Drainage Division, Volume 86, June 1960, Part 1

12. Common Errors in the Measurement of Irrigation Water, C. W. Thomas, Proceedings ASCE, Journal of the Irrigation and Drainage Division, Volume 83, September 1957

13. Hydraulics, Fifth Edition, King, Wisler and Woodburn, John Wiley and Sons, New York, Chapters II and VI

14. Hydraulics, R. L. Daugherty, McGraw-Hill Book Company, Chapters II, VI, and VII

15. Hydraulics for Engineers, Robert W. Angus, Sir Issaac Pitman and Sons, Canada, Chapters II and V

16. Hydraulics, George E. Russell, Henry Holt and Company, New York, Chapters V and VII

17. Flow Characteristics and Limitations of Armco Metergates, J. B. Summers, Hydraulic Laboratory Report No. Hyd-314

18. Flow Characteristics in a Pipeline Downstream from a Square-cornered Entrance, H. A. Babcock and W. P. Simmons, Jr., Hydraulic Laboratory Report No. Hyd-442

19. Flow Characteristics and Limitations of Screw Lift Vertical Metergates, J. W. Ball, Hydraulic Laboratory Report No. Hyd-471

20. Calibration of the Constant-head Orifice Turnout, B. R. Blackwell, Hydraulic Laboratory Report No. Hyd-216

21. Flow Characteristics of 8-, 10-, 12-, and 18-inch Concrete Fresno Irrigation Flowmeters, J. B. Summers, Hydraulic Laboratory Report No. Hyd-340

22. Study of the Effects of Turnout Design on Registration Accuracy of Propeller Meters Placed in Downstream Ends of Turnout Pipes, Hydraulic Laboratory Report No. Hyd-478

23. Investigation of the Effect of Turnout Geometry on the Registration Accuracy of a Propeller-type Open Flowmeter, Hydraulics Branch Report No. Hyd-545

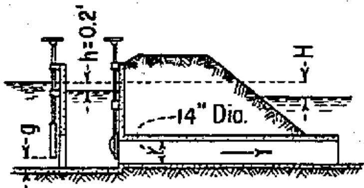
24. Builders-Providence, Division of BIF Industries, Catalog Engineering Information

25. Principles and Practice of Flowmeter Engineering, L. K. Spink, 8th Edition, The Foxboro Company, Foxboro, Massachusetts

26. Discharge of V-Notch Weirs at Low Heads, Fred W. Blaisdell, Civil Engineering, Volume 9, No. 8, August 1939, page 495

27. Vertical Adjustable Weir, J. C. Schuster, Hydraulics Branch Report No. Hyd-553

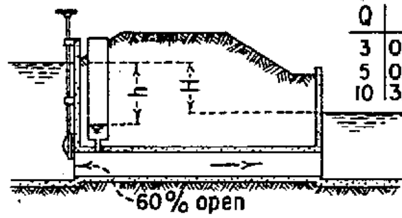
28. Canal Discharge Measurements with Radioisotopes, J. C. Schuster, ASCE Journal of Hydraulics Division HY2, March 1965



Q	H	g
3	0.32	30
5	0.48	50
10	0.93	99

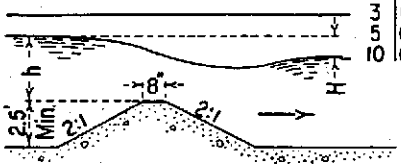
g = Gate open, %

18-BY 24-INCH CONSTANT-HEAD ORIFICE TURNOUT (DOUBLE BARREL)



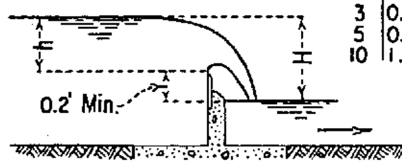
Q	H	h
3	0.30	0.31
5	0.81	0.82
10	3.39	3.46

15-INCH METER GATE



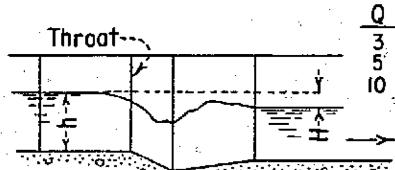
Q	H	h
3	0.30	0.38
5	0.42	0.53
10	0.66	0.82

TRAPEZOIDAL WEIR (CREST LENGTH 4.0 FEET)



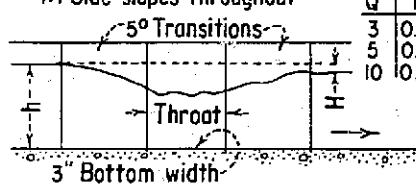
Q	H	h
3	0.58	0.38
5	0.73	0.53
10	1.05	0.85

CONTRACTED RECTANGULAR WEIR (CREST LENGTH 4.0 FEET)



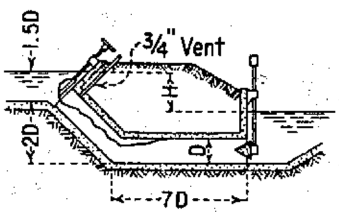
Q	H	h
3	0.20	0.53
5	0.25	0.74
10	0.36	1.15

2-FOOT PARSHALL FLUME



Q	H	h
3	0.11	0.94
5	0.14	1.17
10	0.19	1.60

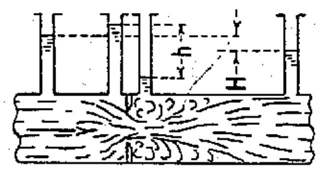
TRAPEZOIDAL VENTURI FLUME



Q	H	D
3	1.64	12
5	1.77	14
10	2.19	18

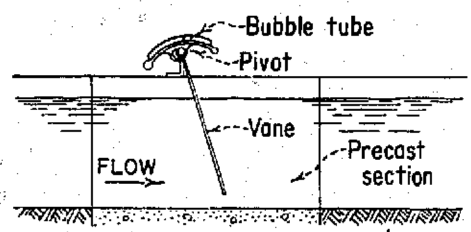
D = Pipe dia. in inches

OPEN FLOW METER*



Q	H	h
3	6.8	9.4
5	18.7	26.0
10	74.2	103.0

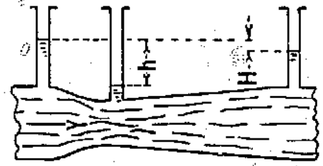
6-INCH ORIFICE IN 12-INCH PIPE



DEFLECTION METER*

FRESNO FLOWMETER

Q	H	h
3	0.38	1.00
5	1.10	2.75
10	4.50	12.50



Q	H	h
3	0.02	1.90
5	0.06	5.28
10	0.25	21.14

12-INCH VENTURI METER

NOTES

Q = Discharge in second-feet.
H = Head required in feet.
h = Measuring head in feet.
* = Substantially no head required to operate.

COMPARISON OF OPERATING AND MEASURING HEADS FOR VARIOUS WATER MEASURING SYSTEMS

103-D-991

Comparison of Operating and Measuring Heads for Various Water Measuring Systems

CONVERSION FACTORS--BRITISH TO METRIC UNITS OF MEASUREMENT

The following conversion factors adopted by the Bureau of Reclamation are those published by the American Society for Testing and Materials (ASTM Metric Practice Guide, January 1964) except that additional factors (*) commonly used in the Bureau have been added. Further discussion of definitions of quantities and units is given on pages 10-11 of the ASTM Metric Practice Guide.

The metric units and conversion factors adopted by the ASTM are based on the "International System of Units" (designated SI for Systeme International d'Unites), fixed by the International Committee for Weights and Measures; this system is also known as the Giorgi or MKSA (meter-kilogram (mass)-second-ampere) system. This system has been adopted by the International Organization for Standardization in ISO Recommendation R-31.

The metric technical unit of force is the kilogram-force; this is the force which, when applied to a body having a mass of 1 kg, gives it an acceleration of 9.80665 m/sec/sec, the standard acceleration of free fall toward the earth's center for sea level at 45 deg latitude. The metric unit of force in SI units is the newton (N), which is defined as that force which, when applied to a body having a mass of 1 kg, gives it an acceleration of 1 m/sec/sec. These units must be distinguished from the (inconstant) local weight of a body having a mass of 1 kg; that is, the weight of a body is that force with which a body is attracted to the earth and is equal to the mass of a body multiplied by the acceleration due to gravity. However, because it is general practice to use "pound" rather than the technically correct term "pound-force," the term "kilogram" (or derived mass unit) has been used in this guide instead of "kilogram-force" in expressing the conversion factors for forces. The newton unit of force will find increasing use, and is essential in SI units.

Table I

QUANTITIES AND UNITS OF SPACE

Multiply	By	To obtain
	LENGTH	
Mil	25.4 (exactly)	Micron
Inches	25.4 (exactly)	Millimeters
	2.54 (exactly)*	Centimeters
Feet	30.48 (exactly)	Centimeters
	0.3048 (exactly)*	Meters
	0.0003048 (exactly)*	Kilometers
Yards	0.9144 (exactly)	Meters
Miles (statute)	1,609.344 (exactly)*	Meters
	1.609344 (exactly)	Kilometers
	AREA	
Square inches	6.4516 (exactly)	Square centimeters
Square feet	929.03*	Square centimeters
	0.092903	Square meters
Square yards	0.836127	Square meters
Acres	0.40469*	Hectares
	4,046.9*	Square meters
	0.0040469*	Square kilometers
Square miles	2.58999	Square kilometers
	VOLUME	
Cubic inches	16.3871	Cubic centimeters
Cubic feet	0.0283168	Cubic meters
Cubic yards	0.764555	Cubic meters
	CAPACITY	
Fluid ounces (U.S.)	29.5737	Cubic centimeters
	29.5729	Milliliters
Liquid pints (U.S.)	0.473179	Cubic decimeters
	0.473166	Liters
Quarts (U.S.)	946.358*	Cubic centimeters
	0.946331*	Liters
Gallons (U.S.)	3,785.43*	Cubic centimeters
	3.78543	Cubic decimeters
	3.78533	Liters
	0.00378543*	Cubic meters
Gallons (U.K.)	4.54609	Cubic decimeters
	4.54596	Liters
Cubic feet	28.3160	Liters
Cubic yards	764.55*	Liters
Acre-feet	1,233.5*	Cubic meters
	1,233,500*	Liters

Table II

QUANTITIES AND UNITS OF MECHANICS

Multiply	By	To obtain
	SS	
Grains (1/7,000 lb)	[illegible] (exactly)	Milligrams
Troy ounces (480 grains)	31.1035	Grams
Ounces (avdp)	28.3495	Grams
Pounds (avdp)	0.45359237 (exactly)	Kilograms
Short tons (2,000 lb)	907.185	Kilograms
	0.907185	Metric tons
Long tons (2,240 lb)	1,016.05	Kilograms
	CE/AREA	
Pounds per square inch	0.070307	Kilograms per square centimeter
	0.689476	Newtons per square centimeter
Pounds per square foot	4.88243	Kilograms per square meter
	47.8803	Newtons per square meter
	MAS LUME (DENSITY)	
Ounces per cubic inc	1.72999	Grams per cubic centimeter
Pounds per cubic foc	16.0185	Kilograms per cubic meter
	0.0160185	Grams per cubic centimeter
Tons (long) per cubi yard	1.32894	Grams per cubic centimeter
	MASS/CAPACITY	
Ounces per gallon (U.S.)	7.4893	Grams per liter
Ounces per gallon (U.K.)	6.2362	Grams per liter
Pounds per gallon (U.S.)	119.829	Grams per liter
Pounds per gallon (U.K.)	99.779	Grams per liter
	BENDING MOMENT OR TORQUE	
Inch-pounds	0.011521	Meter-kilograms
	1.12985×10^6	Centimeter-dynes
Foot-pounds	0.138255	Meter-kilograms
	1.35582×10^7	Centimeter-dynes
Foot-pounds per inch	5.4431	Centimeter-kilograms per centimeter
Ounce-inches	72.008	Gram-centimeters
	VELOCITY	
et per second	30.48 (exactly)	Centimeters per second
	0.3048 (exactly)*	Meters per second
et per year	0.965873×10^{-6}*	Centimeters per second
les per hour	1.609344 (exactly)	Kilometers per hour
	0.44704 (exactly)	Meters per second
	ACCELERATION*	
econd^2	0.3048*	ers per second^2
	FLOW	
Cubic feet per second (second-feet)	0.028317*	Cubic meters per second
Cubic feet per minute	0.4719	Liters per second
Gallons (U.S.) per minute	0.06309	Liters per second
	FORCE*	
Pounds	0.453592*	Kilograms
	4.4482*	Newtons
	4.4482×10^{-5}*	5
	WORK AND ENERGY*	
British thermal unit (Btu)	0.252*	Kilogram calories
	1,055.06	Joules
Btu per pound	2.326 (exactly)	Joules per gram
Foot-pounds	1.35582*	Joules
	POWER	
Horsepower	745.700	Watts
Btu per hour	0.293071	Watts
Foot-pounds per second	1.35582	Watts
	HEAT TRANSFER	
Btu in./hr ft^2 deg F (k, thermal conductivity)	1.442	Milliwatts/cm deg C
	0.1240	Kg cal/hr m deg C
Btu ft/hr ft^2 deg F	1.4880*	Kg cal m/hr m^2 deg C
Btu/hr ft^2 deg F (C, thermal conductance)	0.568	Milliwatts/cm^2 deg C
	4.882	Kg cal/hr m^2 deg C
Deg F hr ft^2/Btu (R, thermal resistance)	1.761	Deg C cm^2/milliwatt
Btu/lb deg F (c, heat capacity)	4.1868	J/g deg C
Btu/lb deg F	1.000*	Cal/gram deg C
Ft^2/hr (thermal diffusivity)	0.2581	Cm^2/sec
	0.09290*	M^2/hr
	WATER VAPOR TRANSMISSION	
Grains/hr ft^2 (water vapor transmission)	16.7	Grams/24 hr m^2
Perms (permeance)	0.659	Metric perms
Perm-inches (permeability)	1.67	Metric perm-centimeters

Table III

OTHER QUANTITIES AND UNITS

	By	To obtain
Cubic feet per square foot per day (seepage)	304.8*	Liters per square meter per day
Pound-seconds per square foot (viscosity)	4.8824*	Kilogram second per square meter
Square feet per second (viscosity)	0.092903*	Square meters per second
Fahrenheit degrees (change)*	5/9 exactly	Celsius or Kelvin degrees (change)*
Volts per mil	0.03937	Kilovolts per millimeter
Lumens per square foot (foot-candles)	10.764	Lumens per square meter
Ohm-circular mils per foot	0.001662	Ohm-square millimeters per meter
Millicuries per cubic foot	35.3147*	Millicuries per cubic meter
Milliamps per square foot	10.7639*	Milliamps per square meter
Gallons per square yard	4.527219*	Liters per square meter
Pounds per inch	0.17858*	Kilograms per centimeter

ABSTRACT

"Water Measurement Procedures" was written primarily for use in the Irrigation Operators' Workshop classes as a teaching aid in presenting the fundamentals of water measurement to field personnel engaged in irrigation work. Technical material has been simplified to provide a clear understanding of water measurement devices and procedures. Basic hydraulics presented includes the discharge equation, velocity head concept, orifice and weir relationships, and the effect of submergence. Factors affecting the accuracy of measurement such as worn equipment, infrequent head measurement, use of wrong measuring device, and others are analyzed. Commonly used devices and methods are discussed including orifices, weirs, Venturi meters, Parshall and Venturi flumes, meter gates, constant head turnouts and propeller meters; new devices and methods include vane deflection meters, acoustic and magnetic meters, and the dilution and radioisotope methods of measurement. Hints for troubleshooting poorly operating devices, suggestions to operators on how to do a good job, and a selected reading list for operators are given. This edition supersedes the previous 1965 and 1966 editions numbered Hyd 552 and 565. Has 34 references.

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Peterka, A J
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DESCRIPTORS-- *hydraulics/ *hydraulic structures/ *discharge measurement/ *irrigation/ water delivery/ *water measurement/ open channel flow/ errors/ closed conduit flow/ discharge coefficients/ weirs/ orifices/ water meters/ current meters/ water metering/ irrigation O&M/ turbulent flow/ Venturi meters/ instruction/ Venturi flumes/ Parshall flumes/ submerged orifices/ radioactive isotopes/ velocity/ field investigations/ flow meters/ laser/ bibliographies/ submergence/ instrumentation
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