Bureau of Reclamation
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IRRIGATION OPERATORS' WORKSHOP--1963

WATER MEASUREMENT PROCEDURES

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CONTENTS

CONTENTS--Continued

FOREWORD

This discourse contains the essential parts of two previous workshop sessions presented as Part I and Part II in 1961 and 1962, respectively, plus some new material never before presented. Some of the material has been rewritten to provide a better basic understanding of water measuring procedures and practices, and an attempt has been made to simplify the presentation to an ultimate degree.

Standard and nonstandard devices are defined and their implications in regard to water measurement are discussed.

Water measuring devices are classified under two categories: (1) the velocity type and (2) the head type.

Under "Some Basic Hydraulics," the concepts of continuity 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 general aspects of water measurement accuracy are discussed.

Flumes and weirs, the most commonly used devices are called upon to furnish examples of good and poor practices. This is done because the majority of irrigation operators are more familiar with these devices and not to condemn or praise these structures as measuring devices. Furthermore, much of what an operator can see and which applies to weirs can also be applied to other devices which are difficult to observe or see in a photograph but which are considered during this session of the workshop.

Then, more sophisticated devices and techniques such as the submerged orifice meter, Venturi meter, metergate, and constant-head orifice turnout measuring device are investigated.

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

Progress in new water measuring techniques and equipment is reported, and an evaluation report on a new open channel deflection meter is mentioned.

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. 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 be formulated for extending the use of the water.

One way to increase the 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. A recent 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 cubic foot 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. It is almost 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 wasted. Proper division is necessary to establish the economic advisability of providing canal linings to reduce losses or improve measuring devices or techniques to provide equitable distribution and fair charges.

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, installed correctly, and maintained sufficiently to 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 such as current meter ratings.

Calibration tests are 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 use standard discharge tables.

In maintaining a standard structure it is only necessary to visually check a few items to be sure that the measuring device has not departed from the standard performance. In maintaining a nonstandard device it is difficult to determine whether accuracy is being maintained except by recalibration.

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 computations 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:

A. Float and stopwatch
B. Current meters
C. Clausen-Pierce weir gage (or stick)
D. Propeller meters
E. Flow boxes
F. Vane deflection meters

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:

A. Rectangular weirs
B. V-notch or multiple notched weirs
C. Cipoletti weir
D. Parshall flume
E. Meter gates
F. Orifice or venturi meters

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.

SOME BASIC HYDRAULICS

DERIVATION OF DISCHARGE EQUATION

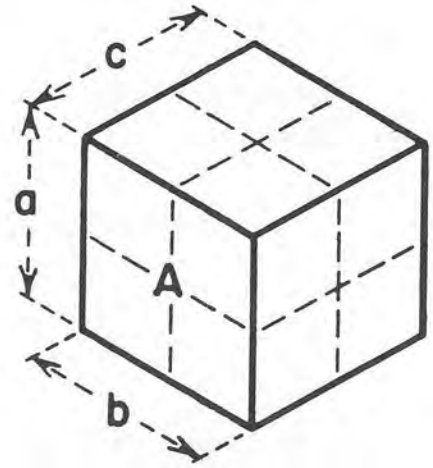
The volume of a cube (Figure 1) may be found by multiplying a times b times c

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



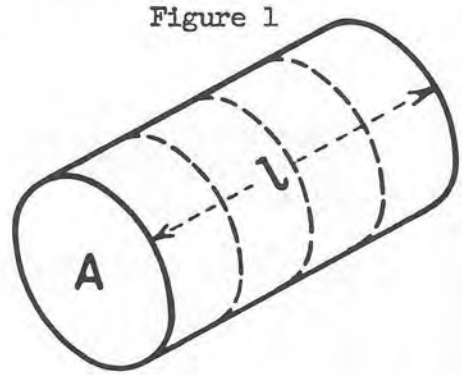
a × b = Area (A)
A × c = Volume

Figure 1

The volume of a cylinder (or pipe) may be similarly found (Figure 2). 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



A × l = Volume

Figure 2

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

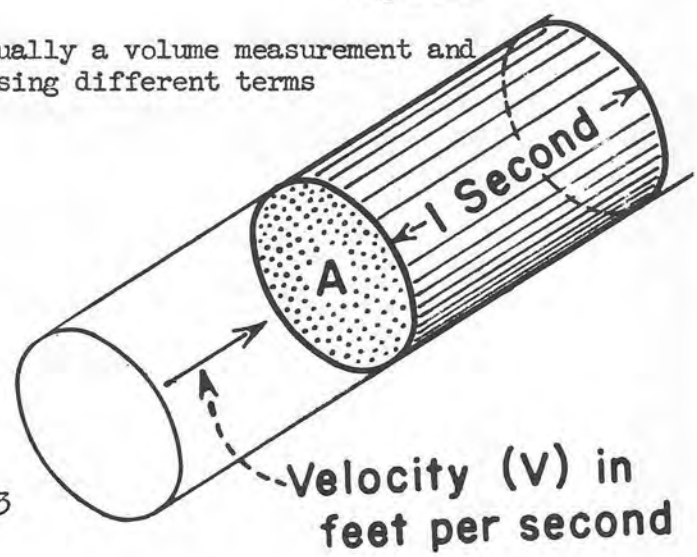


Figure 3

Instead of using 1, as before, substitute the velocity v (Figure 3) and,

Discharge = Area x Velocity

or

$$Q = AV$$

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

Q = square feet times feet per second

Q = cubic feet per second

Therefore, discharge in cubic feet per second is equal to Area of flow cross section in square feet times the velocity in feet per second.

The first basic equation is

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

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, 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. Therefore, the acceleration due to gravity is 32.2 feet per second per second.

Since the acceleration due to gravity, g, is constant, the velocity values in Table 1 increase 32.2 for each succeeding second.

An equation to express these facts is

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

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.

Equation 2 may also be rewritten

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

Square both sides

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

The latter expression is often used to express velocity head, i.e., the head necessary to produce any particular velocity.

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).

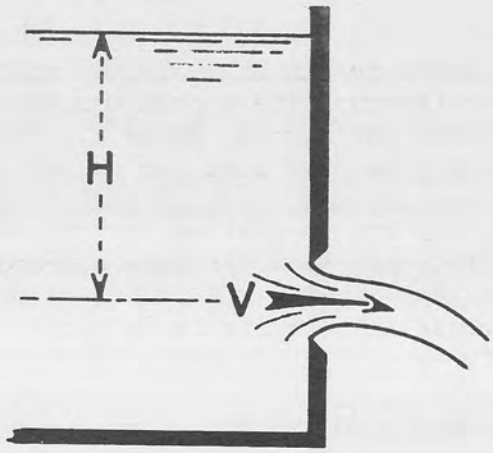


Figure 4

To find the velocity of flow at 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

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

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

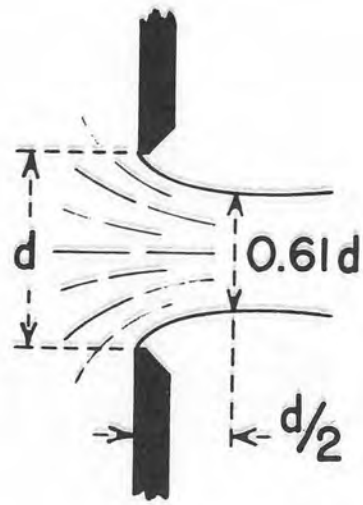


Figure 5

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 six-tenths of the area of the orifice. The maximum velocity also occurs at this point, and so Equation 4 must contain a coefficient "C" (in most cases 0.61) to determine the quantity of water being discharged.

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

For the case discussed, the equation would become

$$Q = 0.61\ A\sqrt{2g}\ \sqrt{H}$$

$$= 0.61\ A\ 8.02\sqrt{H}$$

$$= 4.89\ A\sqrt{H}$$

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).

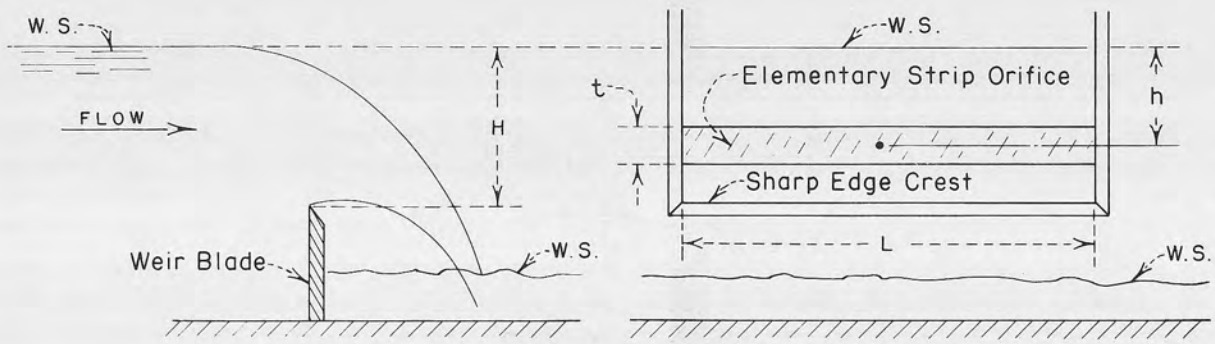


Figure 6

This is done by dividing the flow into small elementary strips and considering each 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 (V) in 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.

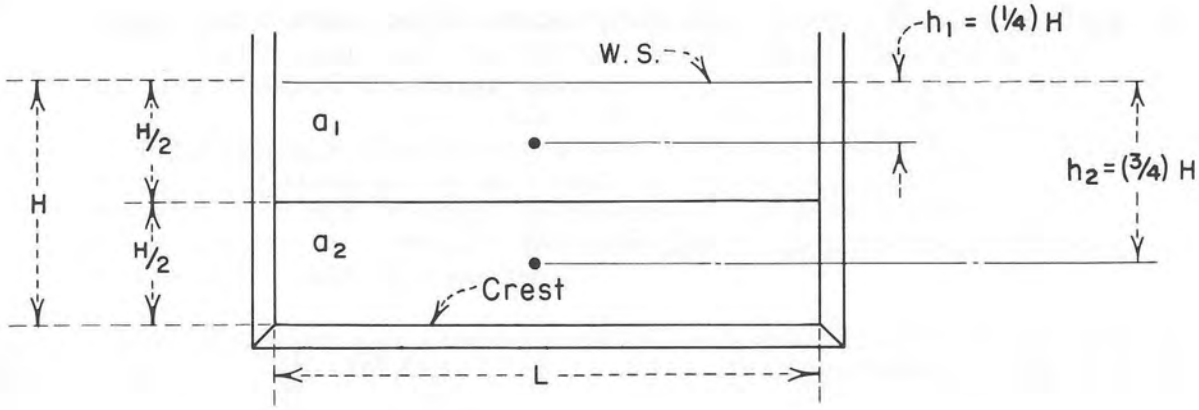


Figure 7

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

$$Q = a_1V_1 + a_2V_2$$

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

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

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

This relationship is the most basic weir formula and can be modified to account for weir blade shape and velocity of approach. C usually varies from about 3.3 for a broad-crest weir to about 3.8 or more for a sharp-crested weir.

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. Many of the facts and principles established for weirs and flumes

also apply to other water measuring devices. Many of these principles will be recalled 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, it should first be determined whether the device is a head of 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 approach area. In general, the approach 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 clear length 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.

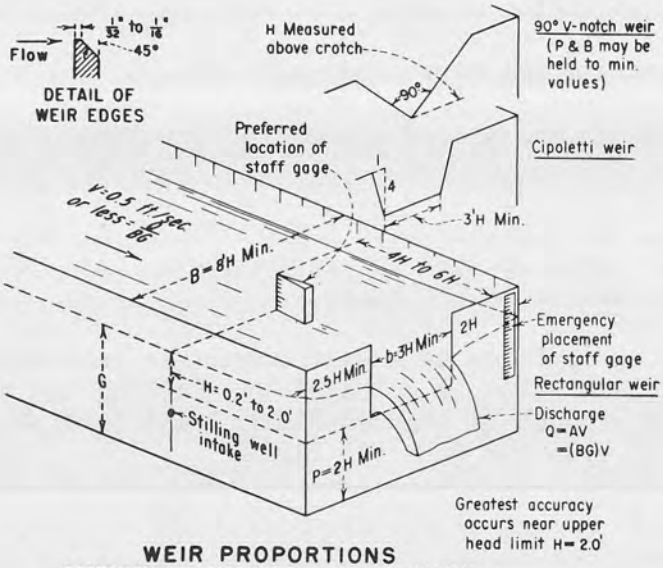
Figure 8 shows an example of excessive turbulence that can be produced by a variety of causes. The high-velocity, turbulent stream is approaching the weir at a considerable angle. Head measurement is difficult, and the weir does not discharge a "standard" quantity. Standard weir installations for rectangular, 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 3-dimensional eddies

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 weir does not discharge a "standard" quantity. PX-D-30664 Figure 8



WEIR PROPORTIONS
RECTANGULAR, CIPOLETTI AND 90° V-NOTCH

Figure 9

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.

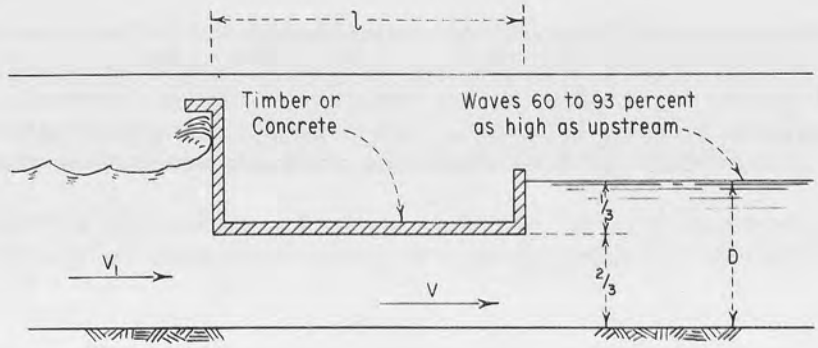
Excessive turbulence will adversely affect the accuracy of any measuring device but is particularly objectionable when using current meters or propeller meters of any kind. Turbulence can be objectionable even without "white water" caused by air entrainment. Turbulence is usually caused by a stilling 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 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.

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

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

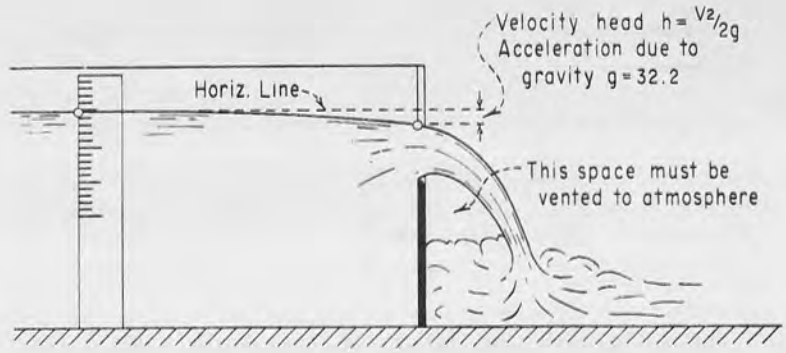
It can be observed that as flow approaches a weir the water surface becomes lower on a gradually increasing curve, Figure 13.

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. P245-D-30666

Figure 11

Underpass-type wave suppressor significantly reduces turbulence and waves in Parshall flume, making accurate discharge determination a routine matter. Compare with Figure 4. P245-D-30663

Figure 12



SECTION ON LONGITUDINAL CENTERLINE

Figure 13

At the weir blade, the water surface is considerably lower than say, 5 or 10 feet upstream. The difference in elevation between two 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 relation between head (h) and velocity (V) is expressed as

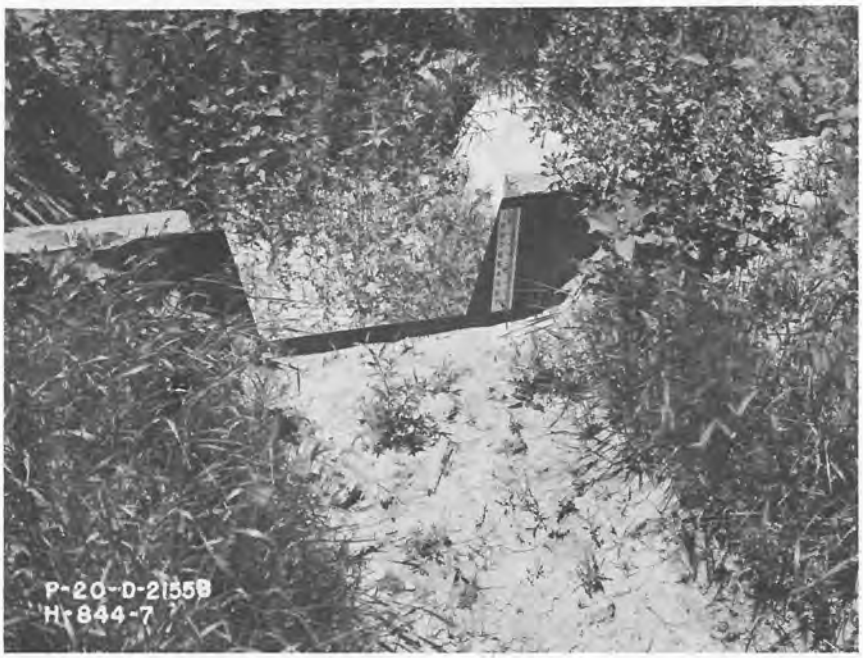
$$h = \frac{V^2}{2g}$$

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 above 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 discharging would be about 35 percent, assuming that the indicated or reported discharge was 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. Along with the head gage being located close to the weir blade, discharges over the weir would be larger than indicated in "standard" tables.
P-20-D-21558

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. 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 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 occur sufficiently to submerge or tend to submerge a device designed for free flow. Occasionally, a Parshall flume is set too low and backwater submerges 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). 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. 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 12.

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 for a moment with the hand or a shovel, to allow a full supply of air to enter beneath the nappe. After removing the obstruction, 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 immediately

after being fully ventilated it may be assumed that the weir has sufficient ventilation.

EQUIPMENT CHARACTERISTICS REDUCING ACCURACY OF MEASUREMENT

Measuring devices themselves may be at fault in producing measurment 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 have water leaks. 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, 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 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 meaurable 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 percent 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.
P-20-D-21557

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.

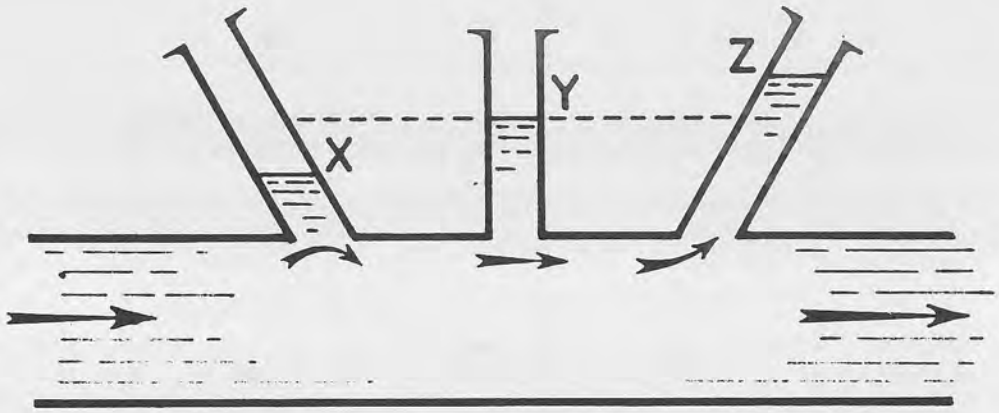


Figure 16

Note: Piezometer openings 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 bank-side structures should be investigated in each case to be sure that a true head reading can be obtained. Placing a rule or a Claussen-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, meter gates, or any other device dependent on a head measurement for discharge determination.

Improper gage location or 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 are usually small because of the relative ease of making accurate length measurements. (Operators should measure lengths 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

Weeds protruding through the opening and sediment in the approach pool will result in inaccurate discharge determinations.
PX-D-30665 Figure 17

Cippoletti 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 expected under these conditions.
POAX-D-18350 Figure 18

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.

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 from a reference point on another structure.

In short, it is desirable that each operator understand the measurement he is trying to make, and then to critically examine each operation to be sure that 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 at the time 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 which 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 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 "Free Flow" tables is thoroughly understood. This is explained in detail in Bulletin No. 426-A, March 1953, "Parshall Flumes of Large Size," by R. L. Parshall, U.S. Department of Agriculture, Fort Collins, Colorado.

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 meter gates, 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 which can best cope with the unusual condition to be encountered.

ELABORATIONS ON THE BASIC WATER MEASURING DEVICES AND TECHNIQUES

SUBMERGED ORIFICE

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

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, there may be some backwater to prevent free-flow conditions as shown in Figure 19.

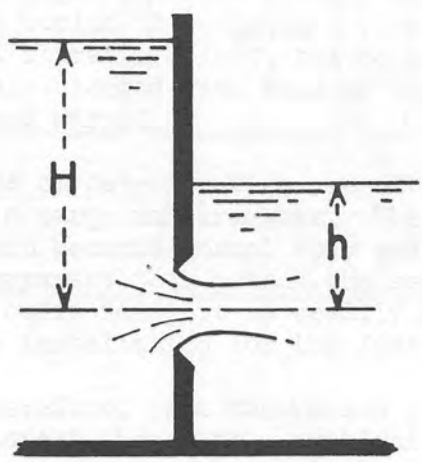


Figure 19

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.

$$Q = CA\sqrt{2g(H-h)} \qquad \ldots \quad (6)$$

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.

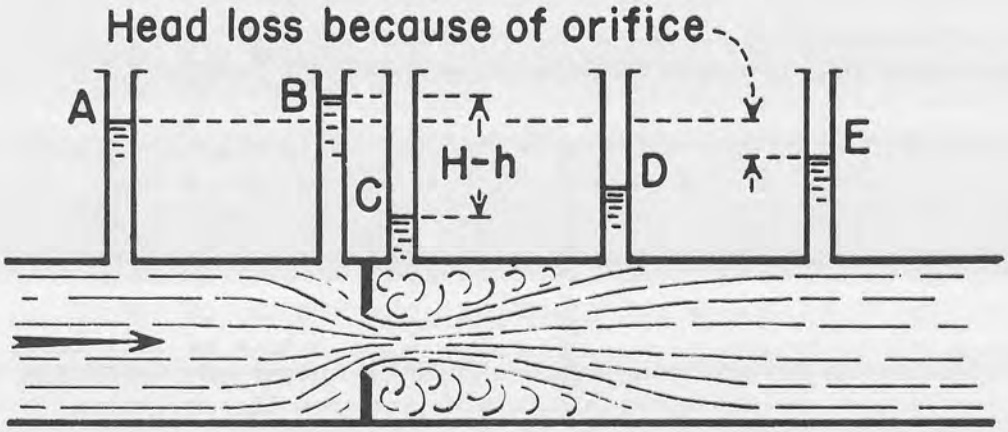


Figure 20

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, in this paper.

If piezometers were placed in a pipe as shown, 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 be used up 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 do not create sufficient head differential to provide accurate measurements and the head tends to fluctuate severely, making it difficult to get a good head determination.

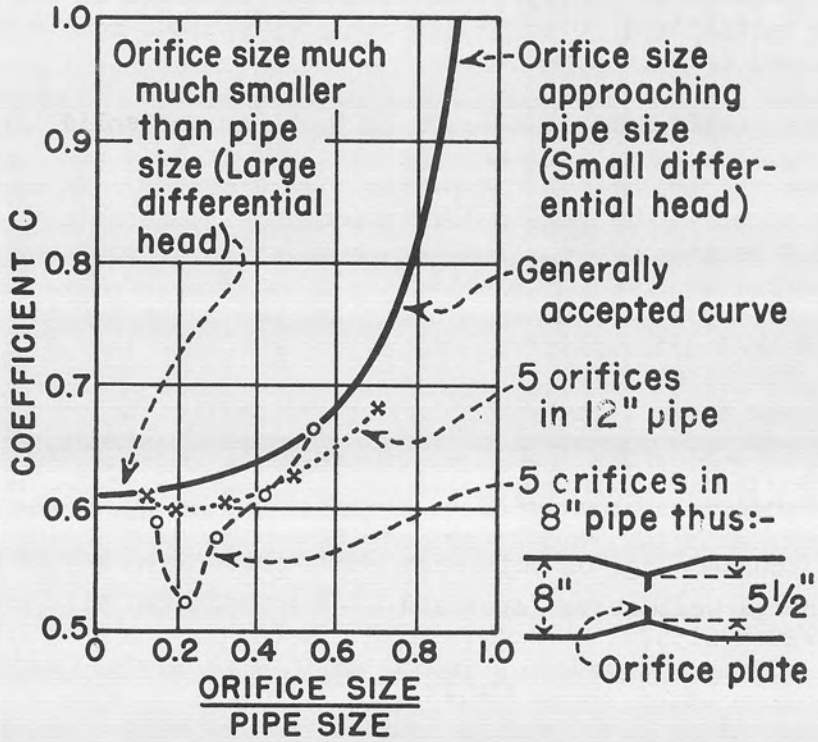
Thus, orifice installations should provide sufficient head (and/or differential) to eliminate head reading errors. In fact, it has been shown that the head on a freely discharging orifice should be at least twice the diameter of the orifice. For lower heads, the coefficient falls off rapidly and may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

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



ORIFICE COEFFICIENTS

FIGURE 21

abscissa in the sketch. Here, again, is a departure from the generally accepted coefficient curve and if a coefficient had been assumed from the published data, 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, or other basic means. This may not be possible, and it may then be desirable to use some other device. 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. 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 changing heads on the orifice coefficient. The submerged orifice equation (6) may be used along with a coefficient of 0.61.

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

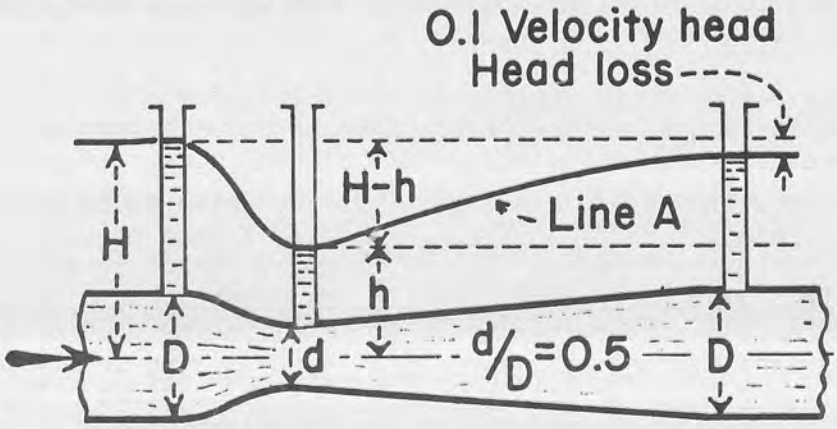
$$Q = 0.61\ (1 + 0.15r)\ A\sqrt{2gH} \qquad \ldots \quad (7)$$

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.

VENTURI METERS

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 construction and a gradual expansion to the pipe diameter as shown in Figure 22.



VENTURI METER

Figure 22

A typical Venturi meter is shown in the sketch for a constriction of one-half the pipe diameter ($d/D = 0.5$). 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 shows the elevation of the water surface which would be indicated if countless 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 (8-inch pipe).

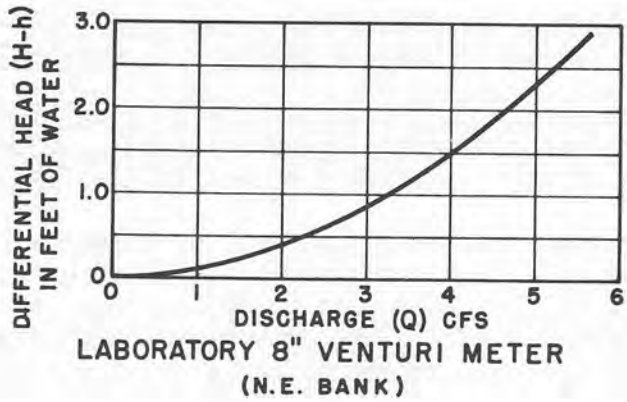


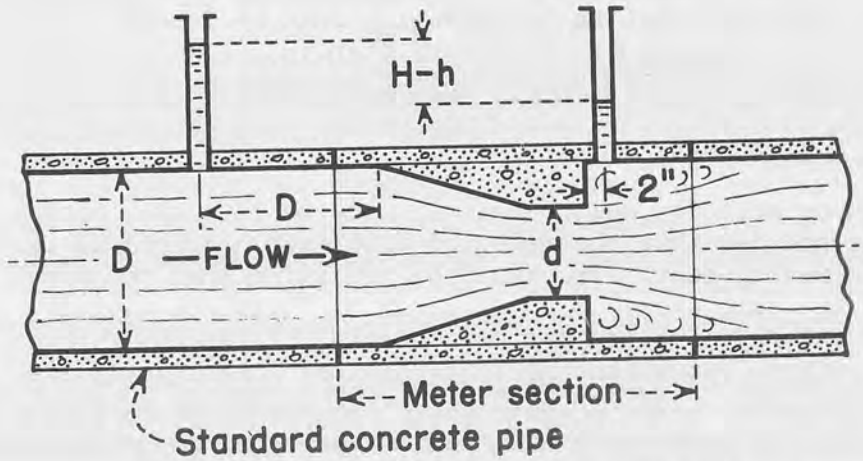
Figure 23

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. 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.

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 ± 1.7 percent. Some of the early work on this subject is contained in a Master of Science Thesis, 1942, "Hydraulic Characteristics of Simplified Venturi Meters," by Mr. R. A. Elder, Oregon State University, Corvallis, Oregon.

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_1). 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) belongs to a large class of water measuring devices known generally as Venturi flumes. 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.

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. Parshall flumes have been previously described in this report.

All principles of measurement accuracy that apply to Parshall flumes apply to all the other varieties of Venturi flumes. However, the floor drop in the Parshall flume was not considered in the discussion. In discussing Venturi flumes this effect is considered.

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 or placement. But, like all measuring devices they should be either calibrated before use or be constructed exactly like an existing flume (standard device) which has been calibrated.

From studies made on Venturi flumes it has been found that for any given flume, each valve 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 USBR 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 for this flume was found to be

$$Q = 3.5\ H^{2.18}$$

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. This study presents the calibration test results on seven flumes with side slope (θ) ranging from 30 to 60°, throat bottom width varying from 0 to 4 inches, and contraction angle (φ) varying between 8 and 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 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 back filling.

For best results the flat-bottomed flume should be set flush with respect to the bottom of the incoming canal. If possible, the cross section 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

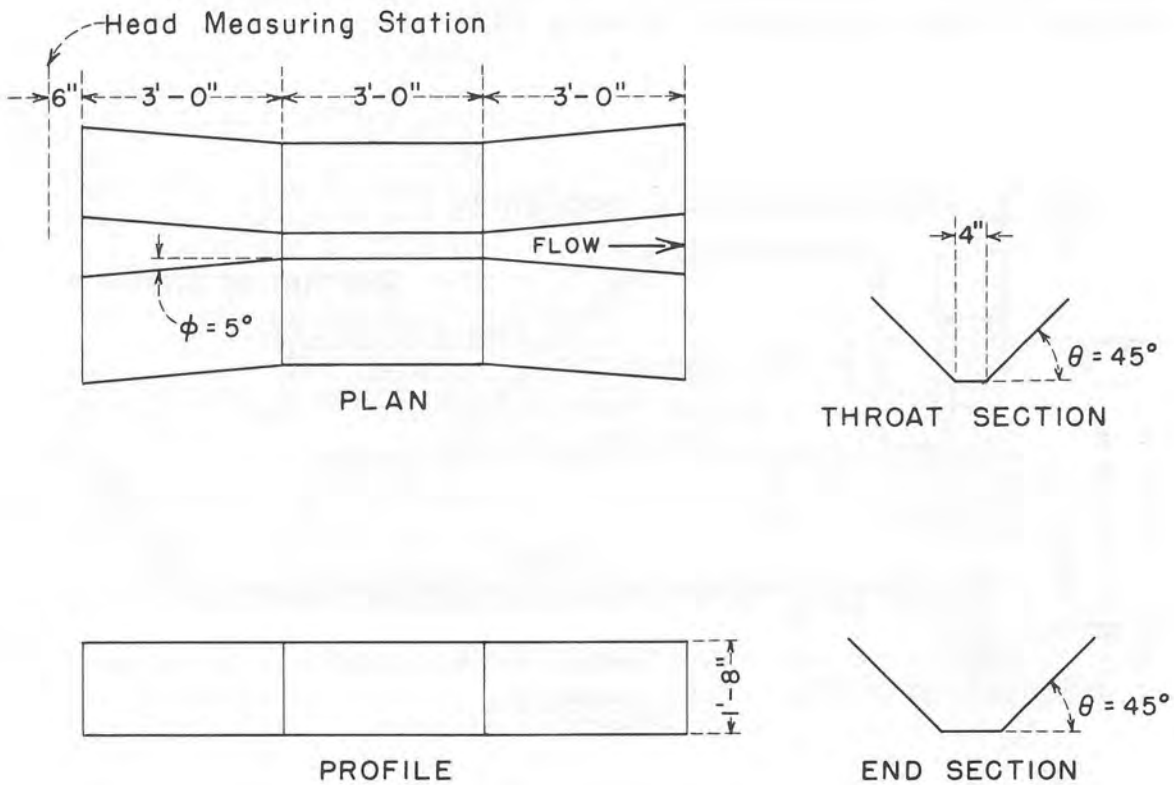


Figure 25

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.

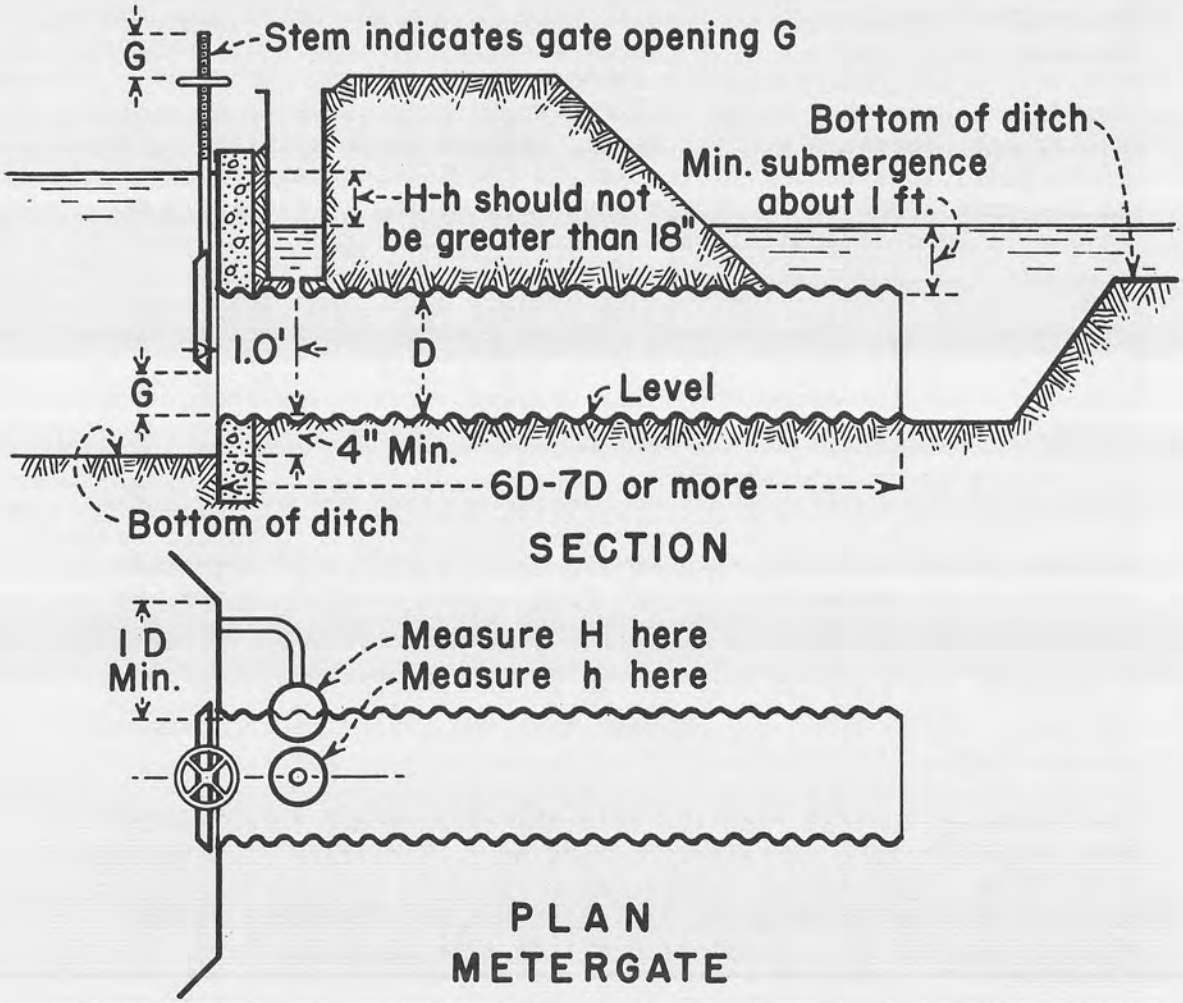


Figure 26

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 18 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 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 (wing walls) 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 location of 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 and several inches (at least) from any change in headwall alinement in plan (see sketch), 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, excactly 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.

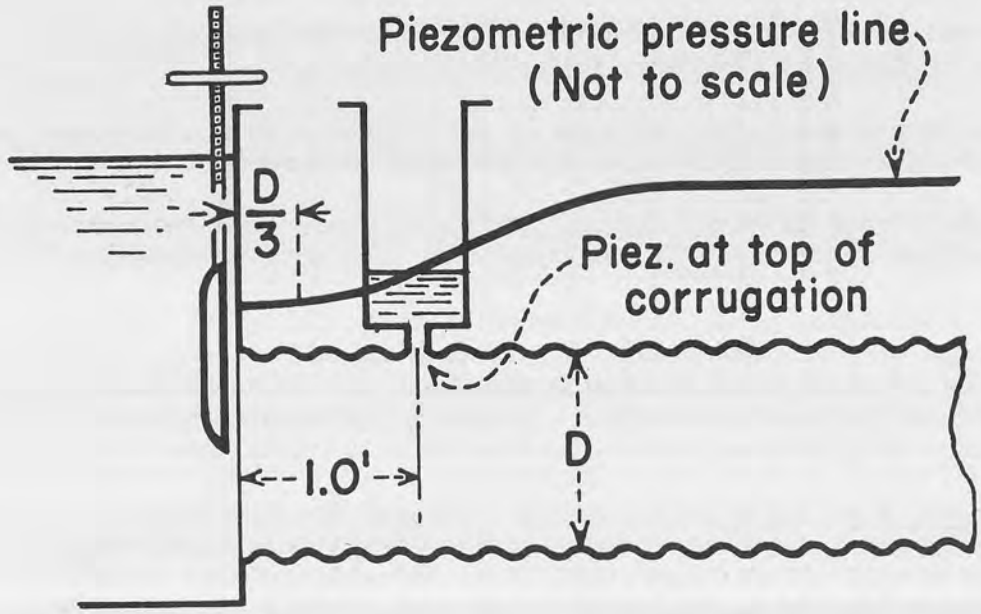


Figure 27

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.

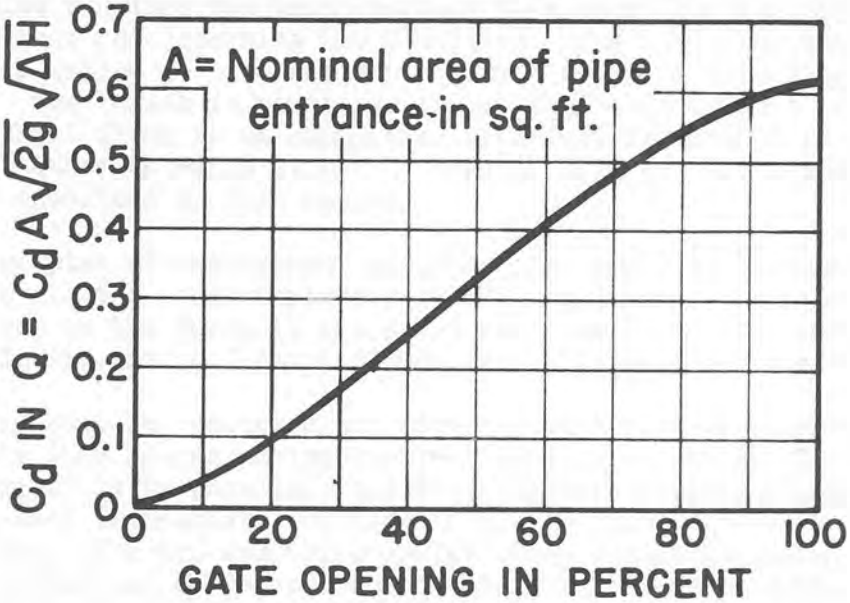


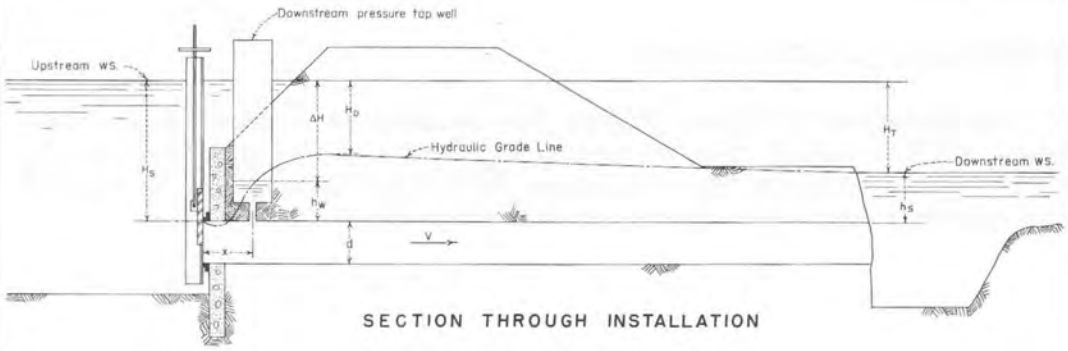
Figure 28

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 ± 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_t = 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.) (Figures 13 and 16).
   For 75 percent gate opening coefficient of discharge, $C_d \approx 0.5$, and maximum $\Delta H \approx 1.85\ H_D$ (Figures 17, 18 and 19) $\Delta H \approx 1.85\ (1.0) \approx 1.85$ 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.
   $H_f = \frac{0.025\ (50)\ (4.53)^2}{0.5\ (64)} = 0.21$ 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.
   From figures 17, 18 and 19, $\frac{\Delta H}{H_D}$ (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 − ΔH − 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 $x = \frac{2d}{3}$ (12 inches from entrance on 18-inch gate) is about 0.75 (Figure 13).
   $\frac{\Delta H}{H_d}$ for $x = \frac{2d}{3} \approx 1.1$ (Figure 17)
   $\Delta H = 1.1\ (0.79) \approx 0.87$
   From $Q = CA\sqrt{2g\ \Delta H}$
   $\approx 0.75\ (1.767)\ (8.02)\ 0.93$
   ≈ 9.9 cfs.

SCREW LIFT VERTICAL METERGATE

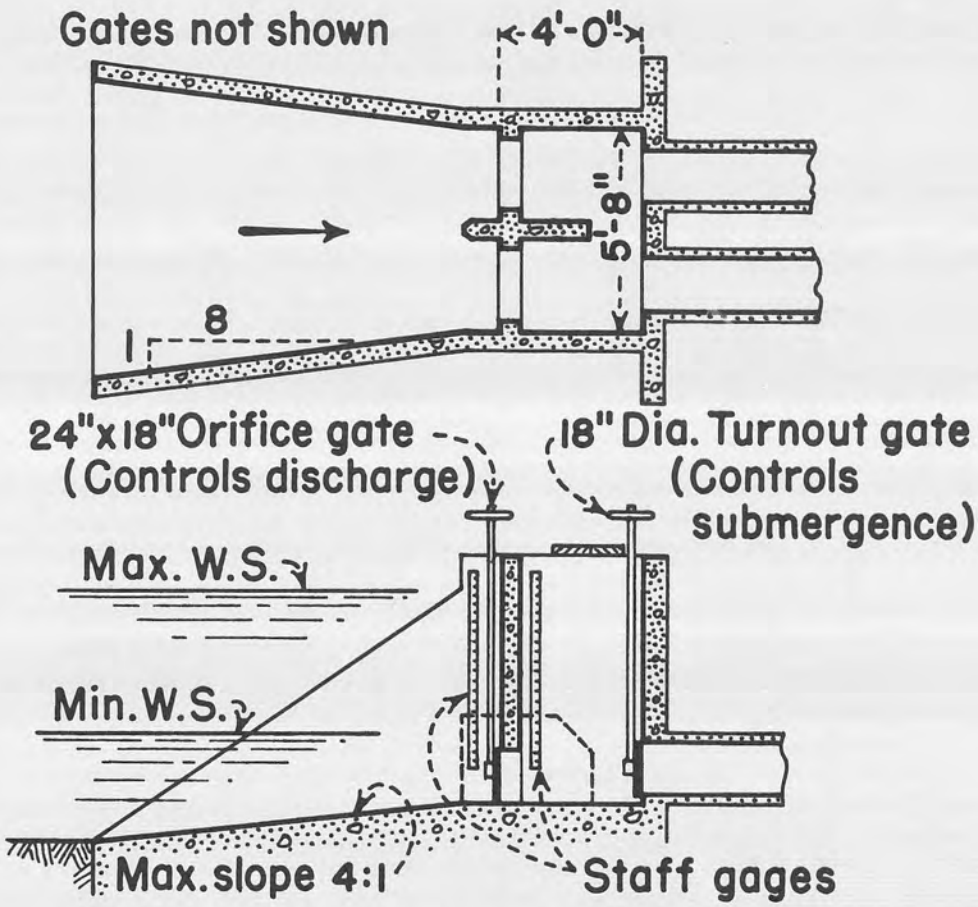
INSTALLATION CRITERIA AND EXAMPLE

573

Figure 29

CONSTANT-HEAD ORIFICE TURNOUT

The constant-head orifice (Figure 30) turnout 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. The orifice gate opening is set and 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 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 (0.2H)
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.

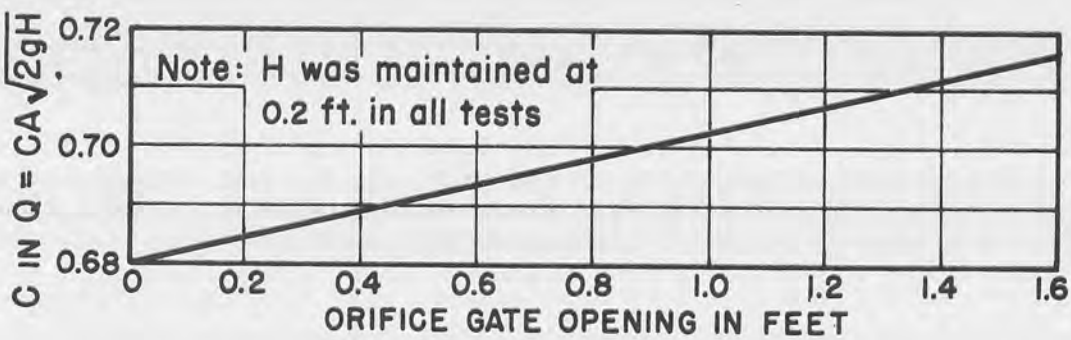


Figure 31

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 below 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 of 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
$\sqrt{0.20} = 0.4472$			
	. . . . .	0.0218	
$\sqrt{0.22} = 0.4690$			

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 ±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 into a dark hole at a steep angle 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.

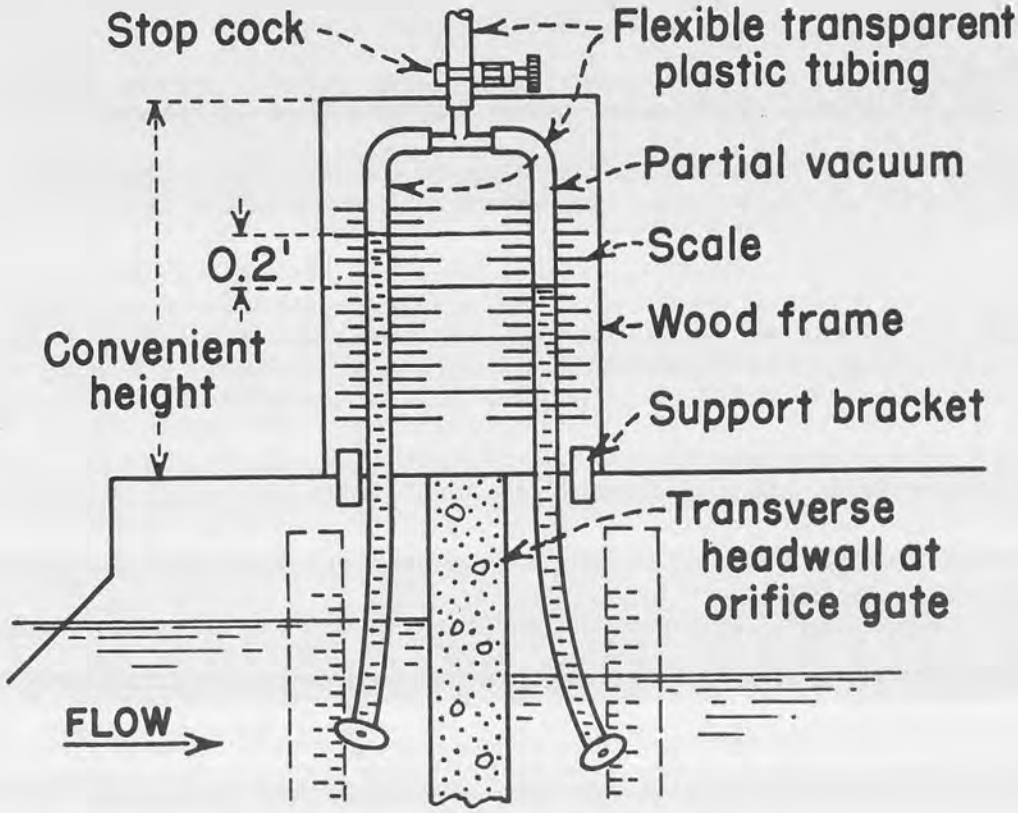


Figure 32

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 disks with small holes (± 1/8 inch) drilled in them may not be necessary but will help to stabilize the water columns if surges are a problem. The disks 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 disks 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 disk 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.

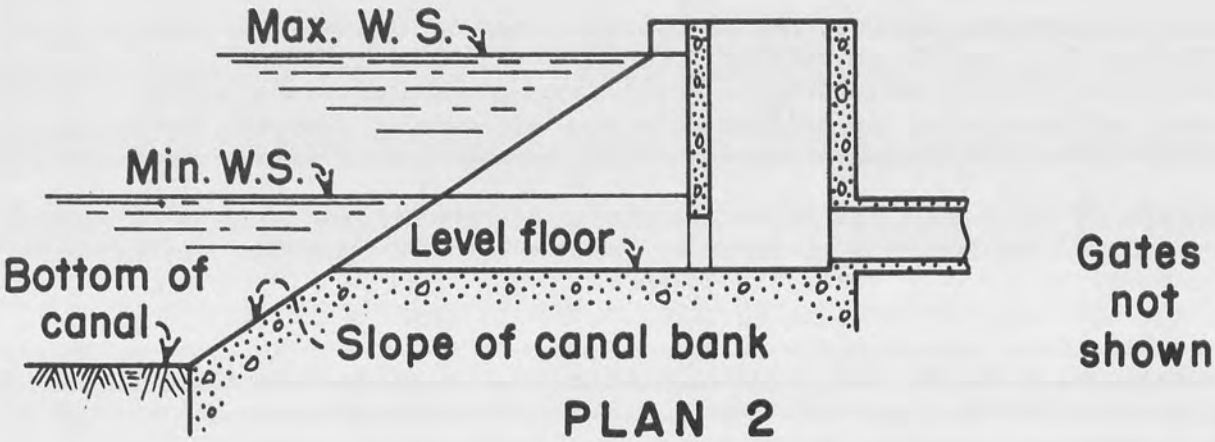
Other errors in discharge measurement might be caused by other factors such as discussed for metergates and orifices.

Effect of Entrance Structure Geometry

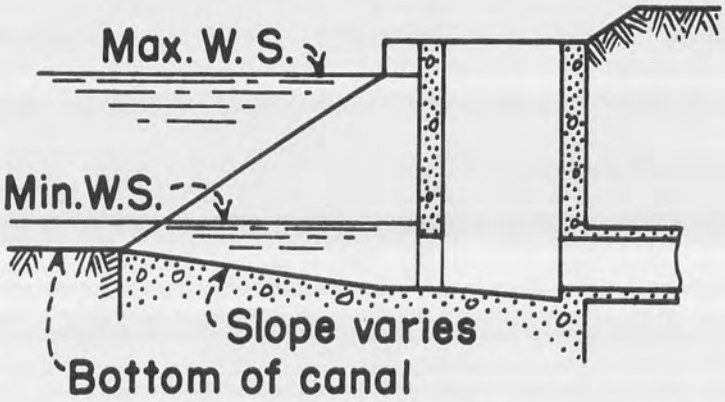
The preceding sketch of the constant-head orifice turnout indicates 8:1 flaring walls in plan on the entrance or approach structure and a 4:1 sloping floor. The floor slopes downward away from the orifice gate, Plan 1, Figure 30. Other common installations are shown in Figure 33, each has 8:1 flaring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

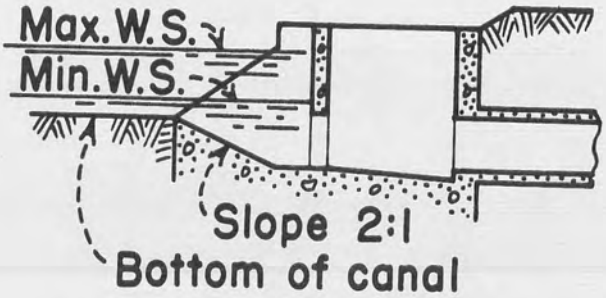
It has been noticed that some structures in the field are now of the Plan 4 type, even though they were Plan 2 type when installed. Sediment and debris have collected near the entrance to the turnout and maintenance crews have cleaned for only a short distance upstream, producing the sharp downward slope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



PLAN 2



PLAN 3



PLAN 4

Figure 33

CURRENT METER GAGING STATIONS

The current meter technique uses the velocity principle to obtain a value of discharge. Therefore, enough readings must be taken to insure accurate values for both the area of the flow section and the average velocity.

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 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. In general, the site should produce flow conditions that meet the requirements discussed under the section, "Flow Characteristics Reducing Accuracy of Measurement."

A staff gage and/or water stage recorder should be installed and carefully referenced to some meaningful datum. In operating the station, a standard procedure should be established and followed for each measurement. Procedures are given in detail in the Bureau's Manual for Measurement of Irrigation Water.

Current meters should be in good mechanical condition and should have a recent calibration or calibration check. The use of two meters during a measurement can show up meter deficiencies if they exist. Careful handling of meters by experienced personnel will help to insure good ratings. 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 the 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.

NEW MEASURING DEVICES AND TECHNIQUES

It was reported in Part I that a portable vane flowmeter was on the market and that, according to the manufacturer's claims, the meter appeared to be accurate and useful. Since the last reporting, the meter has been evaluated from comprehensive tests made under simulated field conditions in the Hydraulic Laboratory. The claims of the manufacturer were found to be quite truthful and 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. 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. Instantaneous discharges only may be read; no totalizing device is available. Installation is simple and the cost is reasonable, especially if several or more ditches can be served with one meter.

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.

Further testing has been done on the radioisotope method of discharge measurement. In this method a radioactive tracer* is introduced into the water, flowing in an open canal or a pipe. At a point downstream, after thorough mixing has taken place, the flow is sampled and the radioactivity of the sample is "counted" using an instrument similar to a Geiger counter. Knowing the quantity of tracer introduced and the "count" or dilution of the tracer, it is possible to calculate the discharge. It is not necessary to know velocity, cross-sectional area, depth or water surface level.

Theoretically, and mathematically, the method is sound; it remains to work out practical details, particularly the problem of mixing the tracer thoroughly with the flow. Recent tests in a canal showed reasonable results but indicated that further refinements in techniques and procedures are required before the method becomes practical for general use. Tests made in a pipeline in the Hydraulic Laboratory showed a discharge of 1.03 cfs using the dilution method, 1.01 cfs using the total count method, and 0.99 cfs on the laboratory calibrated Venturi meter, which is known to be accurate to about one-half of 1 percent.

It is expected that experiments will continue in this field at an accelerated rate and that the true value of the method can be evaluated in the near future.

Another method of measuring discharge is to be investigated as part of the radioisotope program. A new electronic instrument is available which can detect the presence of very small quantities of certain fluorescent dyes in water. This instrument could be used in place of the "counter" and the dye would be used in place of the radioactive tracer. Harmless dye would be injected into the flowing water and a sample taken downstream. The dilution of the dye measured by the electronic instrument would indicate the quantity of water flowing. Present work on this method includes a study to determine whether the dyes would be absorbed or modified after contact with earth, weeds, sediment, and concrete.

Experiments are being made to determine the feasibility of using sound waves to measure discharges. A source of sound on one side

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*The quantity of tracer used is too small to be dangerous to downstream water users.

of a flowing stream is beamed at a receiver on the other side. The degree of interruption of the sound waves by the water velocity would indicate the quantity of water flowing.

CONCLUSIONS

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 rundown and in need of maintenance or that his measurement procedures are not compatible with what he is trying to measure. 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 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 tube problems, and information on head losses and other flow phenomena.

The Bureau reports contain practical information pertaining to discharge measurements through meter gates 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 meter gates 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 text books 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 25, Colorado.

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.

1. Water Measurement Manual, United States Department of the Interior, Bureau of Reclamation, First Edition, Denver, Colorado, May 1953

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. Parshall Flumes of Large Size, R. L. Parshall, Bulletin 426-A, March 1953, U.S. Department of Agriculture, Colorado State University, Fort Collins, Colorado

9. Improving the Distribution of Water to Farmers by Use of the Parshall Measuring Flume, R. L. Parshall, Bulletin 488, 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. E. 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-422

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

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