Bureau of Reclamation
Denver Federal Center

IRRIGATION OPERATORS' WORKSHOP--1962

WATER MEASUREMENT PROCEDURES

PART II

Orifices, Venturi Meters, Metergates, and Constant Head Orifice Meters

by

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FOREWORD

Part I of this series was presented during the Workshop sessions last year and was concerned primarily with measuring devices such as weirs, flumes, and other open channel measuring equipment. To prevent repetition, a new group of measuring devices has been selected for discussion in Part II. Many of the principles discussed in Part I are applicable, in some degree, to the devices discussed here, and copies of Part I have been distributed to you to provide a more complete analysis of the more common measuring devices used in the field.

This lecture period will be devoted to analyzing the performance of screw lift vertical metergates and constant-head orifice metergates. However, to understand the workings and peculiarities of these devices, it is desirable to first understand the principles of an orifice, the basic component of the more sophisticated meters. The submerged orifice meter and the Venturi meter are also part of the evolution cycle and these are also analyzed. The metergate and constant-head orifice turnout may appear quite complicated unless the basic factors affecting performance and accuracy are understood; these are discussed in terms of field conditions and limitations.

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.

The second lecture period on water measurement will be an application of the principles discussed and analyzed in the first period.

A transparent hydraulic model of a constant-head orifice turnout has been specially constructed to demonstrate the hydraulic principles involved in obtaining accurate flow measurements. Students will work in squads, obtaining data and working out sample problems, to prove the accuracy of the theories and recommendations made during the lecture. A better understanding of the many factors affecting flow measurement accuracy should result.

INTRODUCTION

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 and plans will be formulated for extending the use of the water.

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. 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.

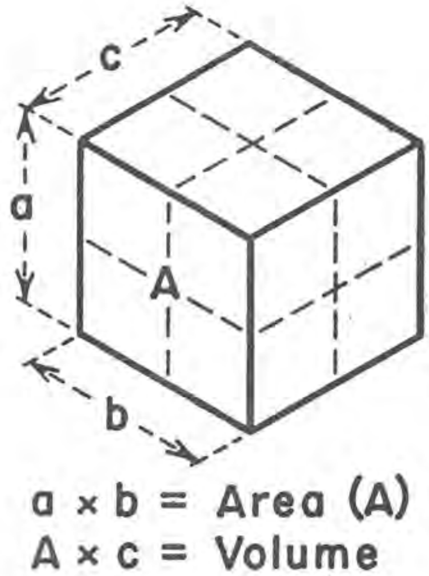
This actual occurrence illustrates another point which is important in water measurement. Practically all measuring devices, when in rundown condition or when improperly installed, deliver more water than they indicate they are delivering. It is almost impossible for a device to deliver less water than 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.

ORIFICE METERS

Derivation of Orifice Discharge Equation

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



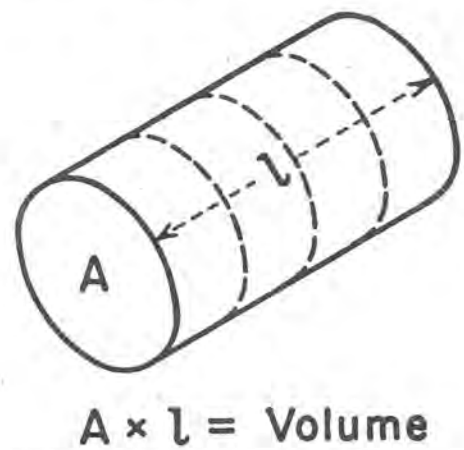
a times b gives the area A

2 x 2 = 4 sq ft

A times c gives the volume

4 x 2 = 8 cu ft

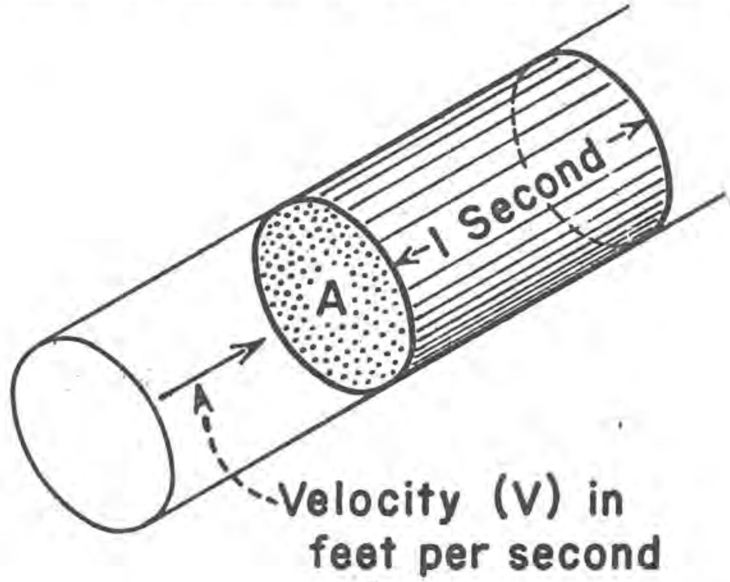
The volume of a cylinder (or pipe) may be similarly found



Area, A, times l gives the volume

2 x 4 = 8 cu ft

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



Instead of using 1, as before, substitute the velocity v so that

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

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

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.

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

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

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

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

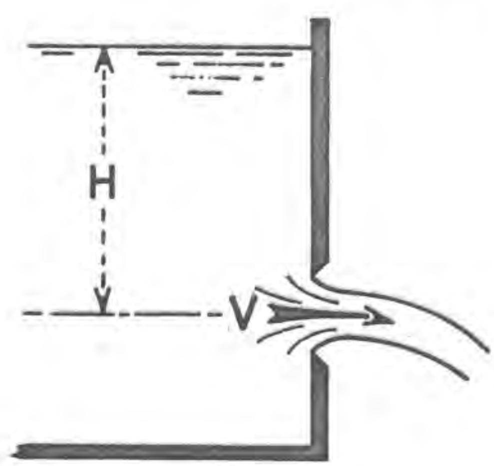
Square both sides

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

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

These equations, $Q = AV$ and $V = \sqrt{2gH}$, are two of the most useful equations in hydraulics, and all irrigation operators should be familiar with them.

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).



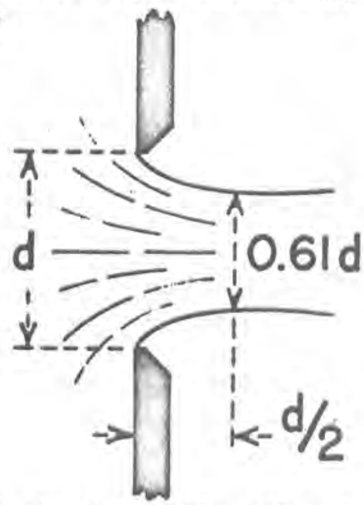
To find the velocity of flow at the orifice, use Equation 2, $V = \sqrt{2gH}$. Then using the size or area of the hole, the quantity of water being discharged, Q, can be determined using Equation 1, $Q = AV$. This can be accomplished in one step, however, if the two equations are combined.

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

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

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 the sketch.



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.

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

For the case discussed, the equation would become

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

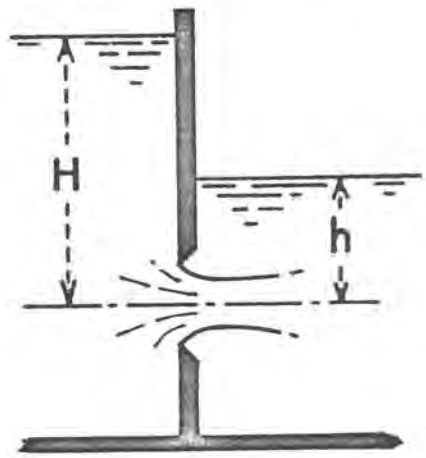
$$= 0.61\ A\ 8.02 \sqrt{H}$$

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

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 the orifice is discharging into a ditch, there may be some backwater to prevent free-flow conditions.



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.

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

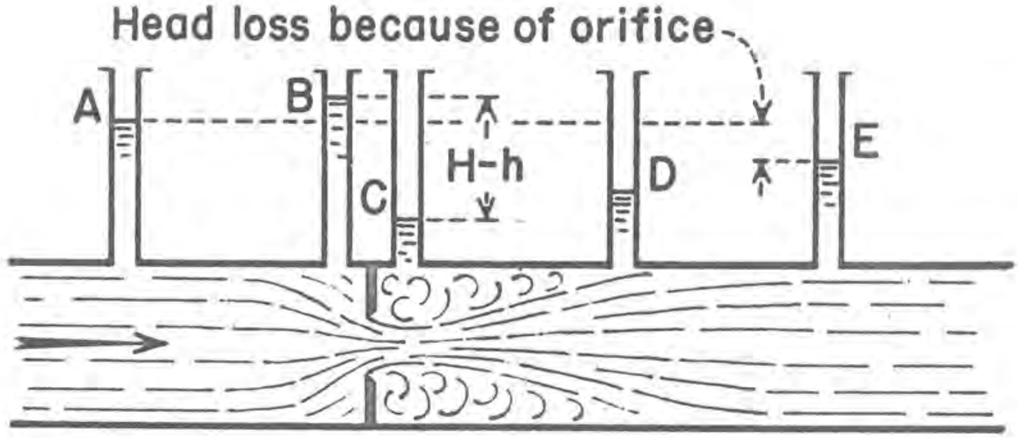
In the preceding problem, if h was 1 foot, the discharge would be

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

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

Orifice Characteristics

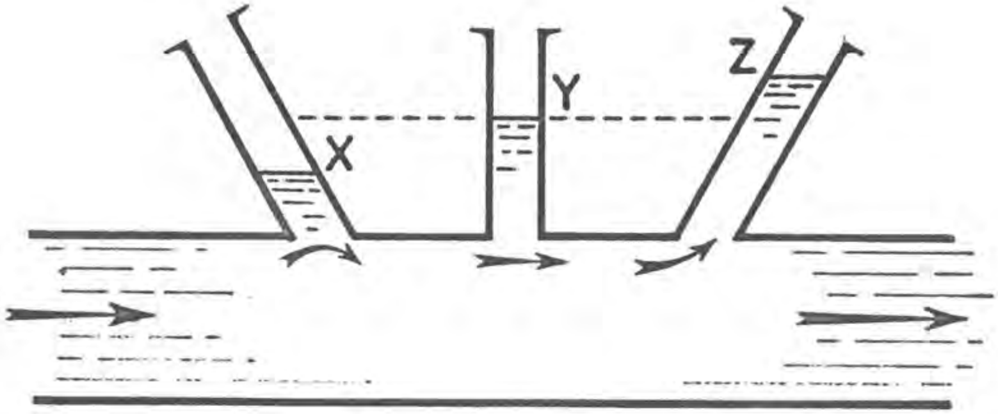
If the orifice is placed in a pipeline, there is almost certain to be a backwater effect, and it will be necessary to measure both the upstream and downstream heads. Since there is no free water surface in a full pipe, piezometers or pressure taps must be used to obtain the necessary data.



Piezometers are no more than small-diameter standpipes in which water rises sufficiently to balance the pressure inside the pipe.

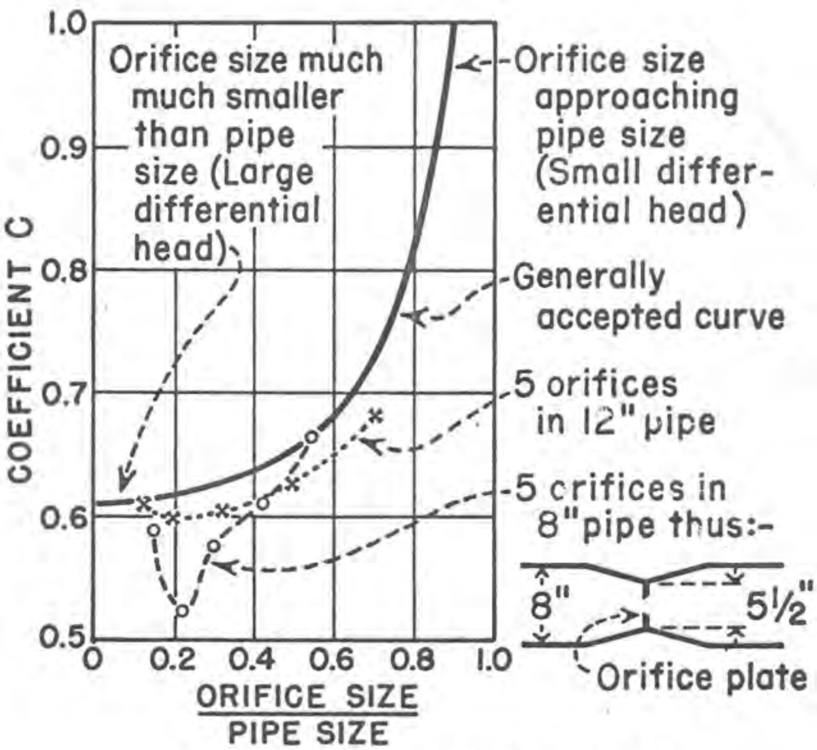
If piezometers were placed in a pipe as shown, the differential pressure H - h (between B and C) would be the head producing flow to be used in the orifice discharge equation. The head at A would be lower than at B because some of the total head would be used up in producing the velocity in the pipe. 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.

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, three piezometers installed as below 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.



Note: Piezometer openings are shown larger than they should be constructed in practice. Always use the smallest diameter opening consistent with the results desired.

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 below shows the variation in C to be expected for various combinations of pipe size and orifice diameter.



ORIFICE COEFFICIENTS

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 3 times the percent that the radius of rounding is of the diameter of the orifice.

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

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

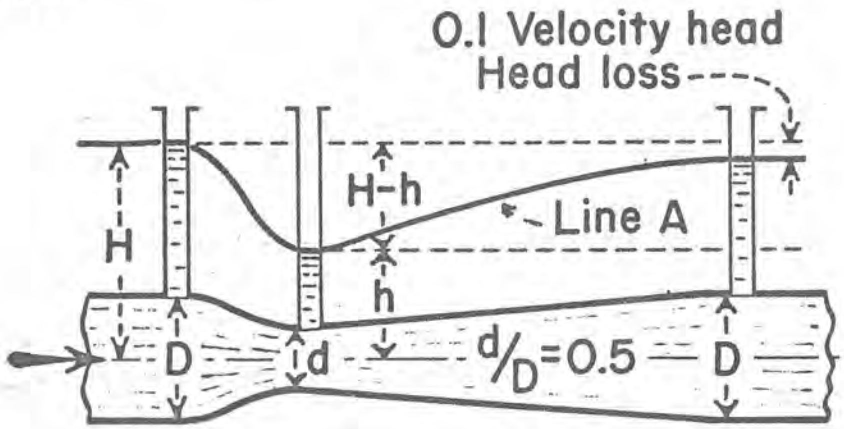
If the orifice is suppressed (hindered by floor, walls, or other) from a normal approach flow pattern, use the equation

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

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.

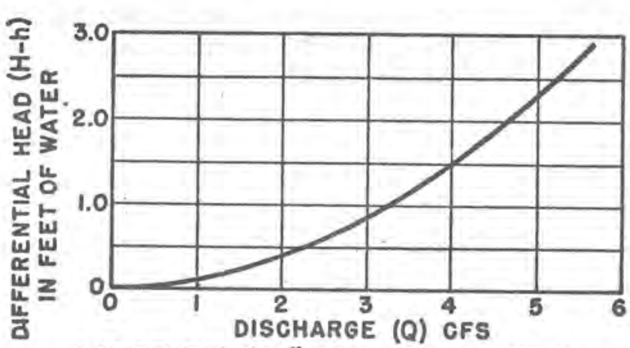
The Venturi meter is basically a streamlined orifice meter and was devised to reduce the head loss produced by the orifice meter. The meter consists primarily of a constriction in a pipe with a curved approach to the constriction and a gradual expansion to the pipe diameter.



VENTURI METER

A typical Venturi meter is shown in the sketch for a constriction of one-half the pipe diameter ($\frac{d}{D}$ = 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 below shows a typical rating curve for a commercial 8-inch Venturi meter (8-inch pipe).



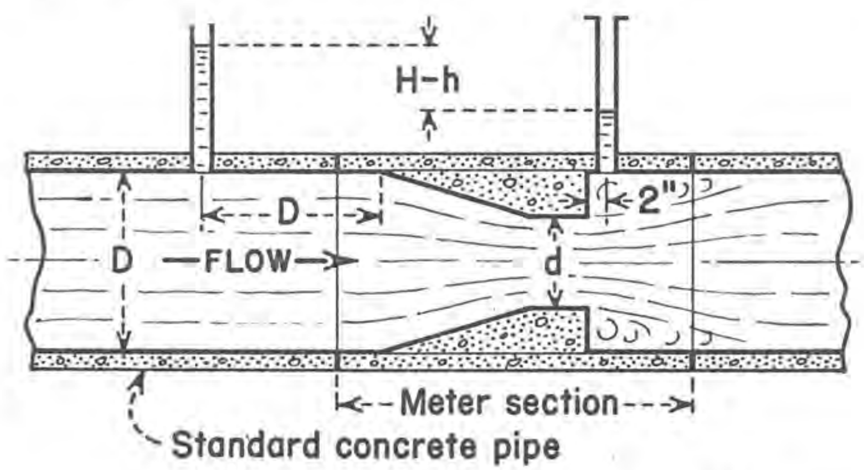
LABORATORY 8" VENTURI METER
(N.E. BANK)

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 are always calibrated because it is impossible to calculate discharges accurately.

Venturi meters are usually machined castings and are quite expensive, although cast concrete has been used in some cases. Some success has been achieved in making satisfactory 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 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

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 Information Branch, Bureau of Reclamation, Denver Federal Center, Denver, Colorado.

METERGATES

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

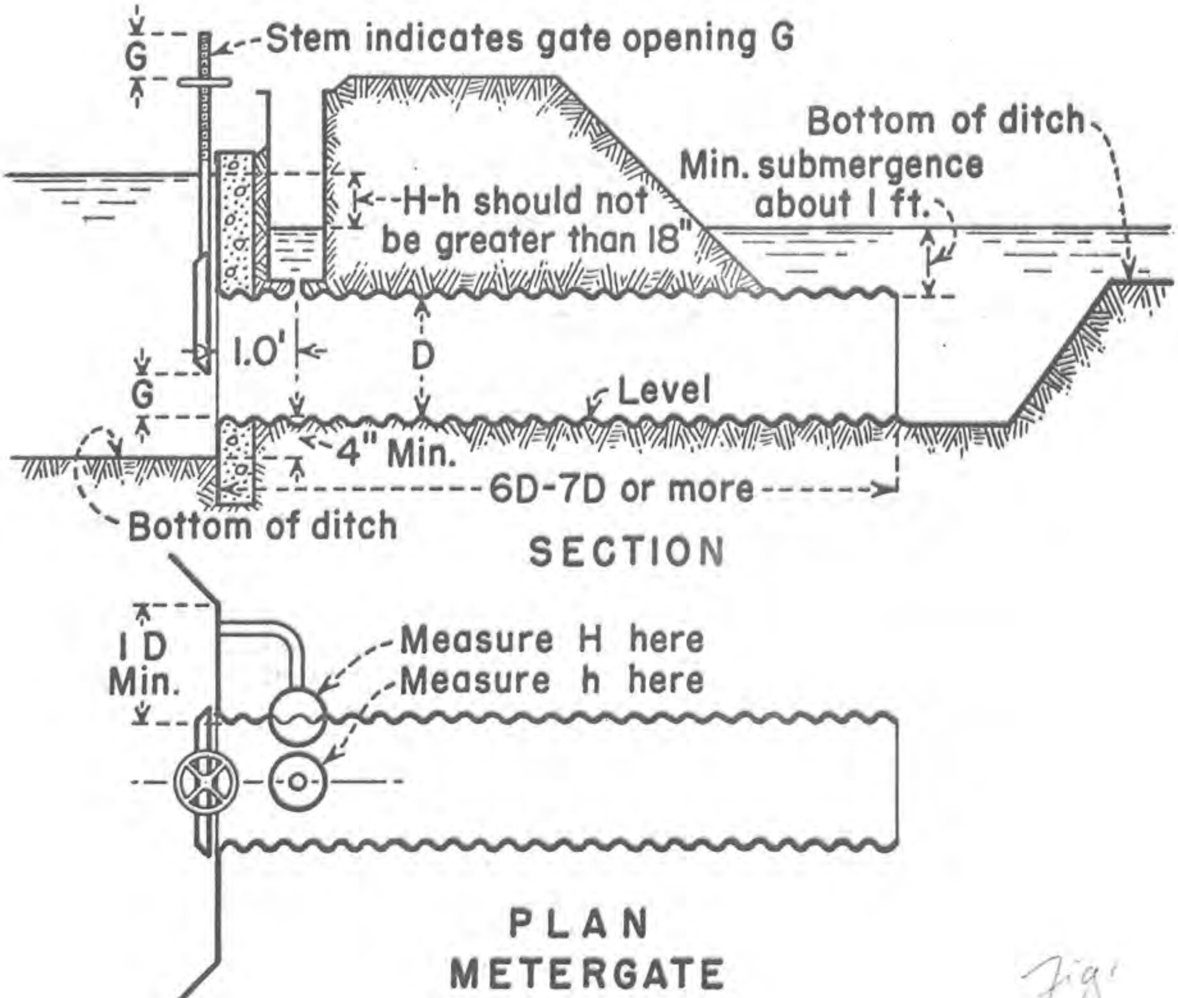


Fig.

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 1 pipe diameter (preferably 2) 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 1 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 6 or 7 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 feet 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).

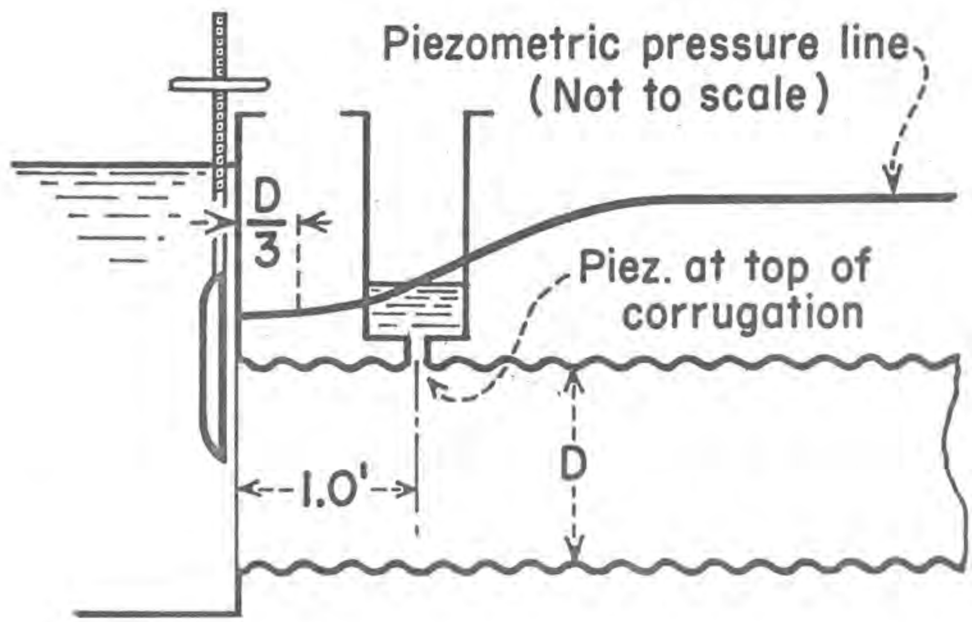
Other methods of achieving a larger differential might include reducing the backwater level, if excessive, or reducing the pipe length, if it is considerably longer than 6 or 7 diameters, to reduce the backwater effect; change location of downstream stilling-well and recalibrate the meter (discussed 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, exactly 1 foot downstream from the downstream face of the gate. The intake pipe must be flush with the inside surface of the pipe (grind off any projections beyond corrugated or smooth surface) and absolutely vertical (see effect of tilted piezometers in Orifice Meter section).

As shown in the sketch below, 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.



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 is valid for all sizes of metergates under certain standard conditions. These include:

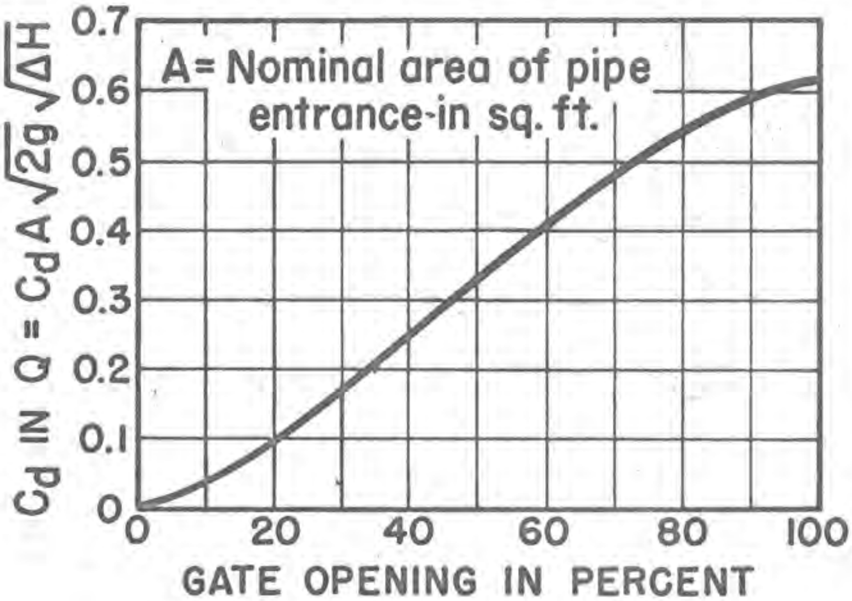
1. Approach channel floor sloping upward, 2:1, toward gate with downstream end of floor 0.17D below pipe entrance invert.

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.



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 $\pm$ 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.

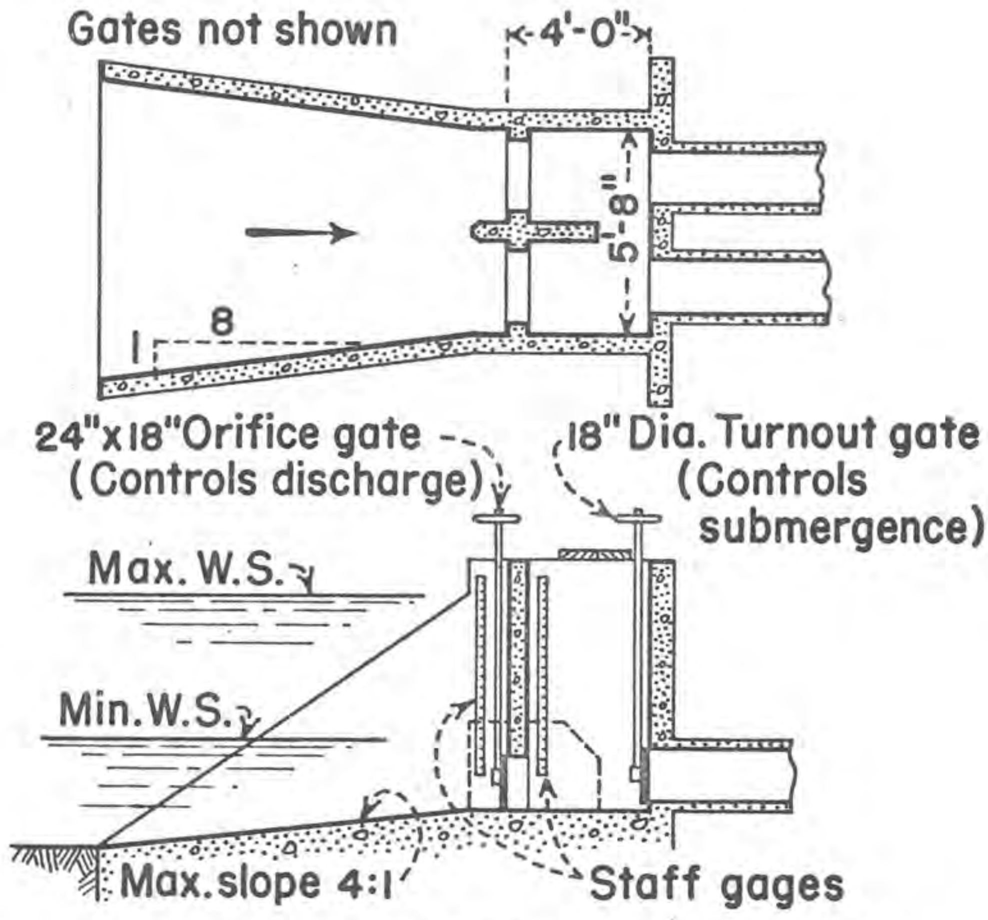
Metergate Installation

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 on the next page. 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.

CONSTANT-HEAD ORIFICE TURNOUT

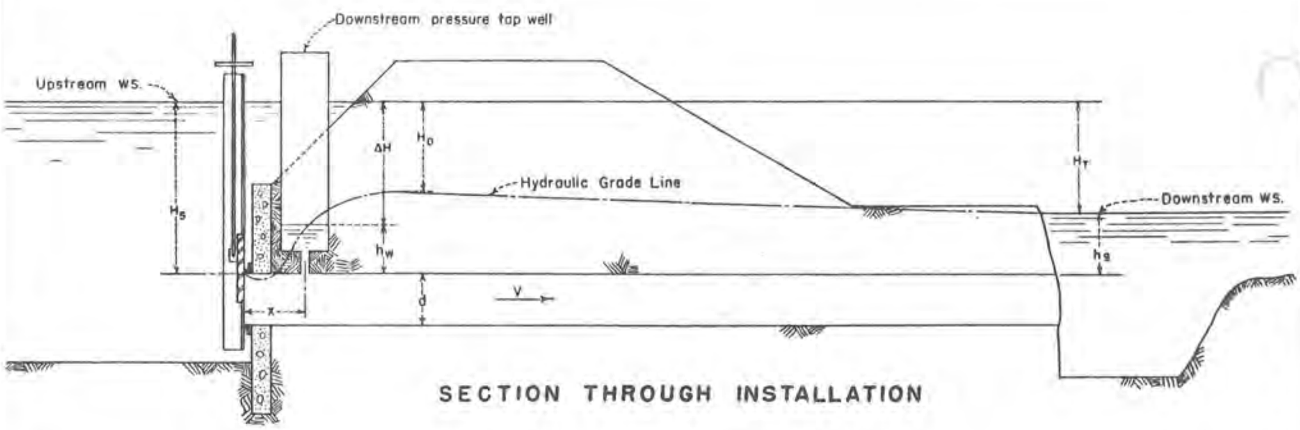
The constant-head orifice turnout is essentially a submerged orifice-meter type of measuring device. The upstream or orifice gate controls the discharge while the downstream or turnout gate controls the submergence on the upstream gate. See sketch.



PLAN I

CONSTANT HEAD ORIFICE TURNOUT

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.



SECTION THROUGH INSTALLATION

DETERMINATION OF METERGATE INSTALLATION

GIVEN

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

FIND

1. SIZE OF METERGATE (One of two methods may be used)

   a. Where downstream scour may be a problem. Select exit velocity that will not cause objectionable scour, say 4 feet per sec.

   From $A = \frac{Q}{V} = \frac{8}{4} = 2.00$, $d = 19\frac{1}{8}$ inches.

   Requires 20-inch metergate.

   b. Where scour downstream is not a problem. Assume metergate to be operated at openings up to 75 percent. (The influences of entrance design, upstream submergence and downstream pressure tap location are minor for these openings.) (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.

   $H_f = \frac{0.025\,(26.3)\,20.5}{64.4} = 0.21$ feet.

   $H_D = 1.0 - 0.21 = 0.79$ feet.

   In order to have a measurable water surface in the downstream well for all gate openings and downstream tap positons the installation should be designed for maximum ΔH.

   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 $- \Delta H - h_w = 100.0 - 1.46 - 0.50 = 98.04$, say El. 98.0.

   Depth requirement for measurable water surface in downstream well is governing factor and gate should be set with crown of entrance not higher than El. 98.0.

3. MAXIMUM CAPACITY OF METERGATE (Full open)

   C_d for full gate opening with downstream pressure tap at $x = 2d/3$ (12 inches from entrance on 18-inch gate) is about 0.75 (Figure 13).

   $\frac{\Delta H}{H_d}$ for $x = 2d/3 \approx 1.1$ (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

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

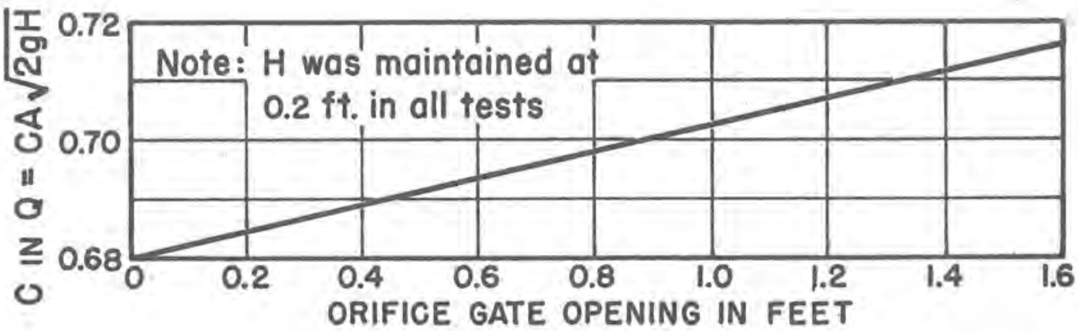
The discharge through a constant-head orifice turnout may be computed from the orifice equation

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

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 below.



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 foot 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

		Difference	
$\sqrt{0.18} = 0.4243$			
	. . . .	0.0229	0.0224 average
$\sqrt{0.20} = 0.4472$			
	. . . .	0.0218	
$\sqrt{0.22} = 0.4690$			

and

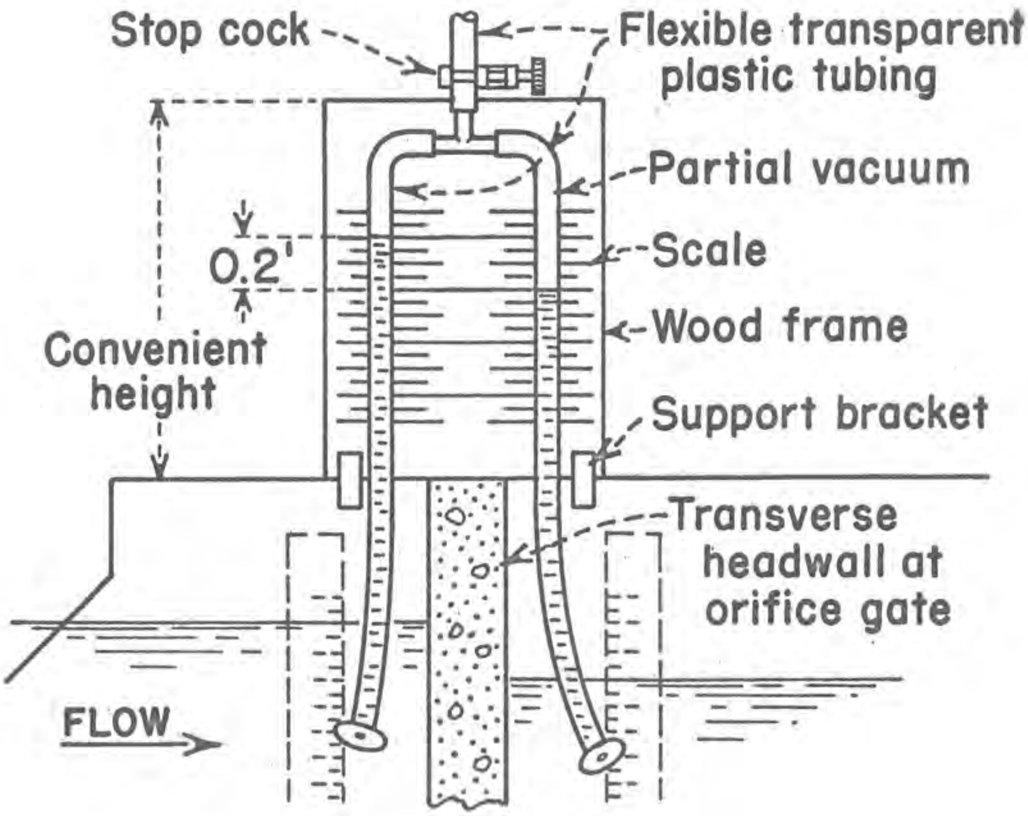
$$\frac{0.0224}{0.4472} \times 100 = \pm 5 \text{ percent}$$

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 the sketch.



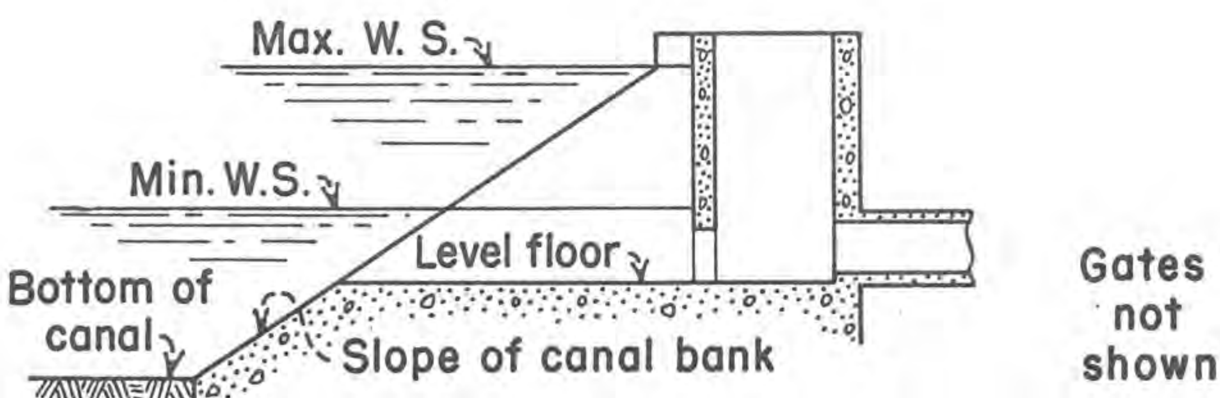
Flexible transparent plastic tubing is slipped over a metal tee (soldered copper tubing) so that both legs are interconnected at the top and lead to a stem fitted with a stopcock. Water is sucked up into the manometer to the desired height for ease of reading and the stopcock tightened to hold the water 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. The 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.

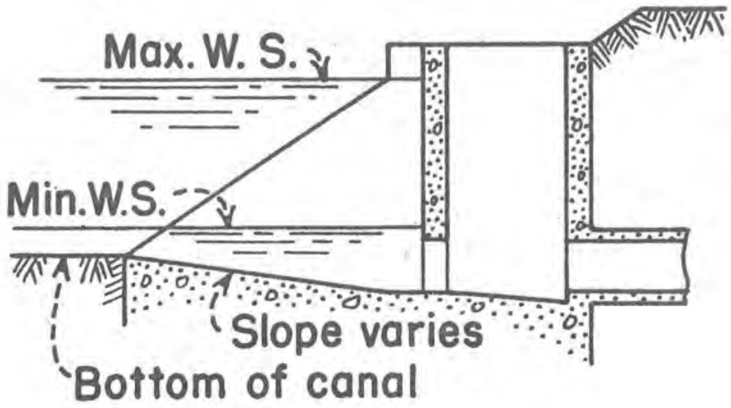
Other errors in discharge measurement might be caused by other factors such as discussed for metergates and orifices.

Effect of Entrance Structure Geometry

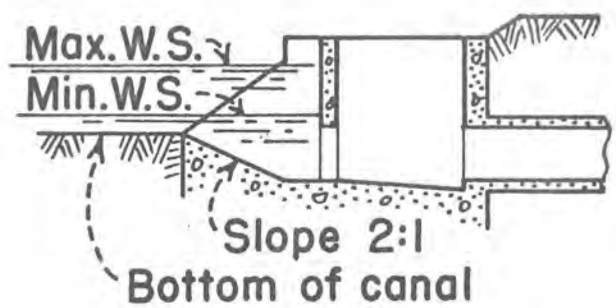
The preceding sketch of the constant-head orifice turnout indicates 8:1 flaring walls in plan on the entrance or approach structure and a 4:1 sloping floor. The floor slopes downward away from the orifice gate, Plan 1. Other common installations are shown below; each has 8:1 flaring walls in plan.



PLAN 2



PLAN 3



PLAN 4

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.

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 Information 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

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

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. Very recent 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 about 2 years.

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 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.

REFERENCES

The following publications are suggested as an aid in understanding the theory and practical aspects of orifices, gates, and associated structures. 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 Information Branch, Bureau of Reclamation, Denver Federal Center, Denver 25, Colorado.

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

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

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

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

5. Flow characteristics and Limitations of Armco Metergates, J. B. Summers, Hydraulic Laboratory Report No. HYD-314.

6. 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.

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

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

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