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ERRORS IN MEASUREMENT OF IRRIGATION WATER

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Synopsis

Devices and structures for measuring irrigation water are subjected to changes in water levels upstream and downstream from the point of measurement. The generally accepted approach to meet this problem is standardization and calibration of the measuring equipment. The use of tables, graphs, or charts developed (from the calibrations) for determining discharge in the field is based on the criteria that the field structure is a replica of the device from which the data were derived and that the flow conditions are identical. Deviations from these standards will result in errors. The magnitude of errors resulting from changes in certain dimensions, incorrect settings, changes in flow patterns, and other deviations is evaluated for some of the commonly used measuring devices. Measurements obtained from equipment capable of operating with a high degree of accuracy may be subjected to errors unless care is exercised in fabrication, installation, operation, and maintenance.

Introduction

The device or structure used in measuring flow in irrigation systems may not yield results as accurate as might be expected. In all probability, the order of accuracy of the flow measurements derived from the various devices will show variations. This deviation will be evident between the different types of structures and methods used. When units of the same type are compared, the deviation may be apparent.

Some of the reasons for the variations in accuracy are:

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1. The principle of operation (volumetric or velocity);
2. The degree of exactness to which the flow coefficients have been established;
3. The workmanship and the care that are exercised in fabrication, construction, and installation;
4. The adaptability of the particular device to the existing conditions;
5. The proper setting with respect to approach and exit flow conditions;
6. Proper maintenance;
7. The manner of obtaining the final result (that is, whether the rate of flow is determined directly from the device or method, or whether head is measured and the end result obtained by computation, tables, charts, or curves);
8. The means for obtaining the needed information (for example, the use of a staff gage in the flow, a gage in a stilling well, or a continuous recorder for obtaining heads);
9. The care in obtaining the data; and
10. Other factors, such as extraneous material in the water, the range of flows to be covered, and similar items.

Absolute accuracy will not be obtained in all instances. The reduction of errors to a minimum may be possible, however, if all factors are considered.

In general, there are two classes of errors: (*a*) Avoidable errors, which result from carelessness and can be eliminated by thorough supervision and attention to detail, and (*b*) unavoidable errors of degree. Although the latter cannot be eliminated completely, they may be alleviated and satisfactory overall results can be obtained by using extreme care and by a knowledge of their nature and magnitude.

In the United States many devices and structures have been developed for measuring flow in irrigation systems. Nearly all these developments are designed to operate in conjunction with separate equipment to control the flows. A few serve the dual purpose of control and measurement. Except in rare cases, the control is operated manually.

The design of most open-channel irrigation systems is such that fluctuations of the water levels both upstream and downstream of the measuring device are tolerated. The measuring devices have been developed to accommodate this design procedure. In effect, any change in the upstream water level and, generally, in the downstream water level is reflected as a change in discharge through the measuring device.

The accepted approach to the solution of the problem of fluctuating water levels is the standardization and calibration of the measuring equipment. The result is a device that, when built and installed in accordance with established standards, will pass a range of known discharges for a range of upstream and downstream water levels. The exact instantaneous discharge can be determined by observing the upstream and downstream heads, by use of correctly referenced gages, and by entering charts, graphs, or tables that have been developed by prior calibration of the device under carefully controlled conditions.

Such a procedure assumes that the field installation is a suitable replica of the installation that was calibrated, usually in a hydraulic laboratory. Further assumptions are that conditions of flow, especially in regard to velocity-distribution patterns, are similar and that the heads and other necessary measurements can be determined with comparable accuracy.

Because discharge tables, charts, or graphs are the results of calibration, they are based on empirical relationships and not on a rational analysis in all instances. Therefore, they are not necessarily susceptible to accurate extrapolation beyond the range of observations from which they were developed.

In order to obtain accurate flow measurements in an irrigation system, it is necessary to know the standards developed and the conditions of calibration. Emphasis will be given herein to some of the possible measurement errors that result from disregarding the adherence to close tolerances in the developed standards, from failure to make accurate observations, and from other deficiencies. The cited examples are of devices used in open-channel irrigation systems. Similar arguments can be applied to equipment and structures used in closed-conduit systems.

The devices are not necessarily those most susceptible to errors, nor are all the cited cases those that may cause measurement errors. Weirs are used widely in the United States for measuring irrigation and drainage water. Therefore, more attention has been given to possible errors in this type of structure. Sharp-crested weirs will be examined because broad-crested weirs are not generally used in irrigation measurements.

The usual practice in irrigation systems is to read the head gages on measuring devices at daily intervals as it is not practicable to obtain numerous readings every day at each point of measurement. Because an irrigation system rarely reaches steady flow conditions the foregoing practice may result in major errors. However, until a continuous recorder is developed that will be economically feasible for installation this practice will be followed.

TABLE 1.—Discharge Error for Weirs

Weir length, l, in feet	Percentage error $(Q' - Q/Q) \times 100$
1.0	1.0
2.0	0.5
3.0	0.33
4.0	0.25

Sources of Error

Faulty Fabrication or Construction.—It is possible for errors to be introduced in flow measurements as a result of the faulty construction of measuring devices. Incorrect dimensioning of the structure is evaluated readily and can be used as an example of the error.

Table 1 shows the discharge error for rectangular weirs or Cipolletti weirs for an incorrect weir-crest length measurement of only 0.01 ft as compared with the calibrated standard from which the flow formula was derived. Because discharge is related directly to length in the flow equation, an error in length of 0.05 ft would cause the discharge to be in error by five times the values shown in Table 1 for any observed head. Eqs. 1 through 4 are used to determine the percentage error as shown in Table 1:

$$Q = C\,L\,H^{3/2} \tag{1}$$

$$Q' = C\,(L + \Delta L)\,H^{3/2} \tag{2}$$

$$\frac{Q'}{Q} = \frac{L + \Delta L}{L} \tag{3}$$

and

$$Q' = \left(\frac{L + \Delta L}{L}\right) Q \qquad (4)$$

Table 2 shows the discharge error caused by a 0.01-ft incorrect measurement of the throat width for Parshall flumes having standard widths of from 1 ft to 4 ft. A constant head of 0.2 ft has been assumed. The error introduced by faulty measurements of 0.02 ft and 0.03 ft is also shown in Table 2. This error is essentially constant for different values of measured head. The flow equation (Eq. 5) used in developing Table 2 was derived empirically from calibrations[2]:

$$Q = 4\, W\, H_a^{1.522\, W^{0.026}} \qquad (5)$$

Similarly, an error in the measurement of width or breadth of a rectangular submerged orifice will cause a considerable error in discharge. Because the

TABLE 2.—DISCHARGE ERROR FOR PARSHALL FLUME

W, in feet (1)	H_a, in feet (2)	Q, in cubic feet per second (3)	Q', in cubic feet per second (4)	Percentage error $(Q' - Q/Q) \times 100$ (5)
Error in width measurement of 0.01 ft				
1.0	0.2	0.348	0.351	0.86
2.0	0.2	0.656	0.659	0.45
3.0	0.2	0.972	0.975	0.30
4.0	0.2	1.264	1.267	0.23
Error in width measurement of 0.02 ft				
1.0	0.2	0.348	0.355	2.0
Error in width measurement of 0.03 ft				
1.0	0.2	0.348	0.359	3.1

discharge is related directly to the area of the orifice, the magnitude of error is similar to that for the weir, and an error in either the length or breadth measurement will be constant for various heads on the orifice or gate.

Error in Discharge Measurements Due to Transverse Slope of Weir Crest.—When installing Cipolletti weirs and rectangular weirs in the field, it is necessary to set the crest exactly horizontal. If it is known that the crest is not level, it is common practice to consider the effective head to be the average head on the weir. The error caused by this practice is shown in Fig. 1. An inclination of approximately 6° will cause an error of on the order of 1%. An angle of this magnitude should be detected by eye and corrective measures taken.

[2] "Improving the Distribution of Water to Farmers by Use of the Parshall Measuring Flume," by Ralph L. Parshall, *Bulletin No. 488*, Colorado Agri. Experiment Station, Fort Collins, Colo., May, 1945.

Eq. 6 was used to determine the percentage discharge error shown in Fig. 1:

$$\text{Percentage of error} = \frac{100\, S^2\, L^2}{32\, H^2} \qquad (6)$$

in which S is the slope of the weir.

A more precise method would be to compute the discharge, using the head at the low and high ends and averaging the discharge derived from these two computations.[3] Thus, the error is reduced.

If it is not known that the weir is inclined and the gage zero is referenced to either the high end or the low end, the resulting error is considerably greater. Measurement of the head in this manner applies the error in reading to the entire crest length, and there is no compensation for a part of the crest being

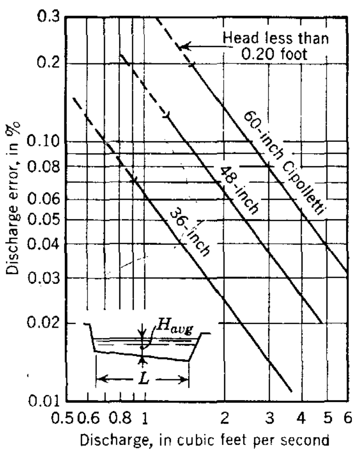

FIG. 1.—ERROR IN DISCHARGE DUE TO TRANSVERSE SLOPE OF CIPOLLETTI WEIR CREST WITH $S = 0.01$

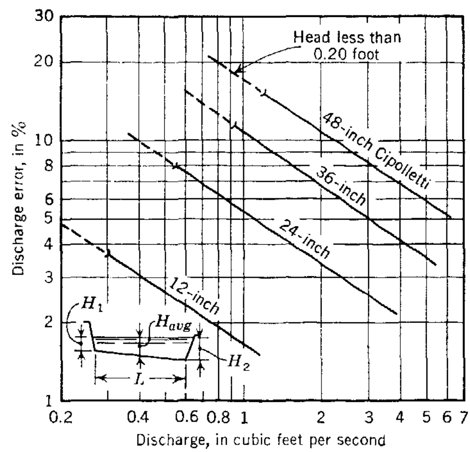

FIG. 2.—ERROR IN DISCHARGE FOR HEAD MEASURED AT END OF CIPOLLETTI WEIR WITH $S = 0.01$

above and a part being below the reference level. Fig. 2 shows the magnitude of the discharge error resulting from measuring the head at either end of 12-in., 24-in., 36-in., and 48-in. Cipolletti weirs having a transverse slope of 0.01 instead of using the average head over the weir.

Error in Discharge Due to Incorrect Head Reading.—In measuring irrigation water perhaps the most common error is to misread the head. This may result from (1) incorrect gage location, (2) dirty head gage, (3) failure to use a stilling well, (4) considerable fluctuation of the water surface, and (5) carelessness in not obtaining a correct average reading at the time the gage is observed.

Fig. 3(*a*) shows the error in discharge resulting from a 0.01-ft incorrect head reading on 12-in.-to-48-in. Cipolletti weirs and on a 90° V-notch weir. This figure demonstrates that an error of approximately $7\frac{1}{2}\%$ in discharge results when the lower heads are measured. For greater heads the error is less.

[3] "Error in Discharge Measurements Due to Transverse Slope of Weir Crest," by Warren E. Wilson, *Civil Engineering*, Vol. 9, No. 7, 1939, p. 429.

It can be noted as well that for the longer weirs this slight error in head reading results in significant errors in discharge measurements.

As in the case of weirs, the head at the throat of a Parshall flume is easily misread in the field. Parshall flumes with throat widths of from 6 in. to 36 in. are shown in Fig. 3(*b*). This figure shows the error in discharge measurements resulting from a 0.01-ft misreading of the gage. The error is approximately the same as for an equal misreading of the head when weirs are considered.

Fig. 4 shows the measurement error resulting from a 0.01-ft misreading of the head on 8-in., 12-in., and 18-in. metergates and on an 18-in. screw-lift gate. The metergate has a circular leaf, and the screw-lift gate has a rectangular leaf. Fig. 4 also shows the error caused by a 0.01-ft incorrect head reading for the constant-head orifice turnout.[4] The percentage error in discharge resulting from misreading the head on an orifice is less, in general, than the same misreading on a weir.

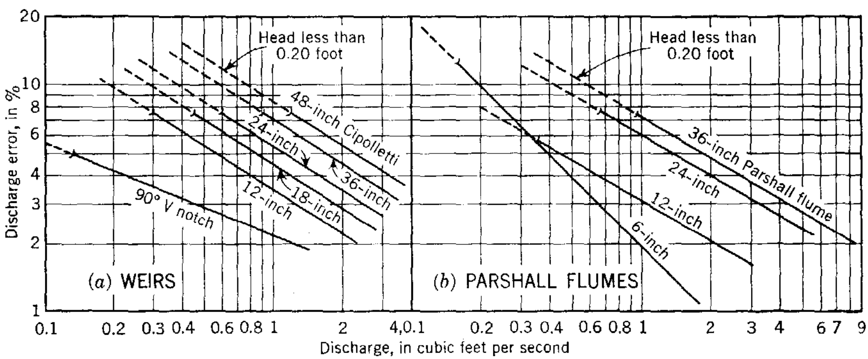

FIG. 3.—DISCHARGE ERROR FOR 0.01-FT INCORRECT HEAD READING

Error in Discharge Measurement Caused by Incorrect Zero Setting.—The error for incorrect zero setting of the head gage is of the same magnitude as for an equal misreading of the head. The improper positioning of the gage used to read the head is probably the most common error. In the field it is difficult to reference the exact zero of the gage to the crest of a weir, to a submerged orifice, or to a turnout gate. Extreme care should be exercised in setting the gages because incorrect settings cause errors at all flows (Figs. 3(*a*), 3(*b*), and 4).

Errors Resulting from Improper Gage Location.—Proper gage location to obtain head readings on measuring devices is important. In most instances flow relationships have been determined empirically, with a particular type of gaging device placed in a specific location. Hence, the over-all calibration includes a secondary calibration effect of the gaging system used to obtain head. Because of changes in the flow pattern of the stream as it passes through the measuring section, minor deviations from the standard used in gage design and location may appreciably affect the quality of the measurements.

[4] "Water Measurement Manual," Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo., 1st Ed., May, 1953, p. 77.

In the case of a weir, there is a downward curve of the water surface as the flow passes through the notch. This curved surface, or drawdown, extends a considerable distance upstream; the exact distance is dependent on local conditions. The head of the weir must be measured beyond the effect of the drawdown. In the development of the basic weir formulas, the head was observed at distances upstream from the weir notch varying from approximately four times to nine times the maximum head over the weir. Therefore, many authorities have accepted a minimum distance of four times the maximum head

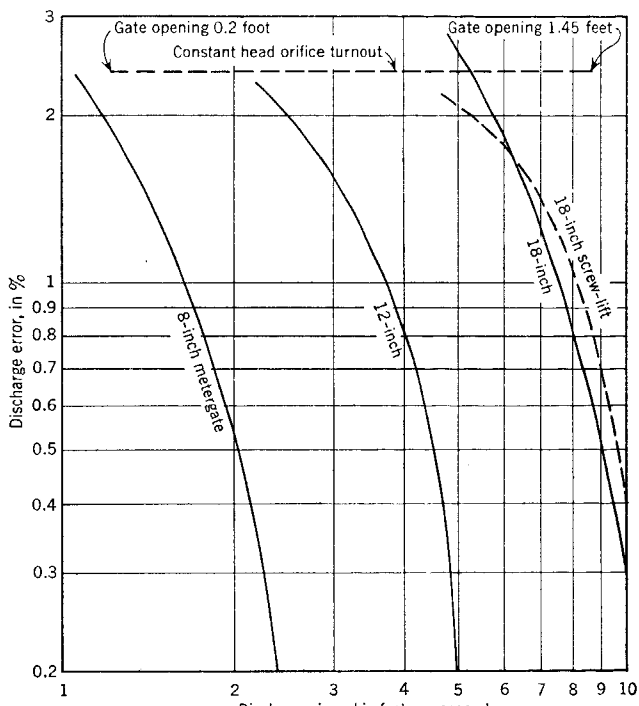

FIG. 4.—DISCHARGE ERROR FOR 0.01-FT INCORRECT HEAD READING ON GATES (METERGATES AND SCREW-LIFT GATE FULLY OPEN)

to be measured. However, Horace W. King[5] states that the distance should be at least two and one-half times the maximum head. Experiments have shown that there is some drawdown effect to a distance upstream of approximately six times the head on the weir.[6,7] However, the influence at this distance is minor. Within the practical limits of the gages used at weir installations in irrigation systems, it appears that a distance upstream of four times

[5] "Handbook of Hydraulics," by Horace W. King, McGraw-Hill Book Co., Inc., New York, N. Y., 3d Ed., 1939, p. 91.

[6] "Description of Some Experiments on the Flow of Water Made During the Construction of Works for Conveying the Water of Sudbury River to Boston," by Alphonse Fteley and Frederic P. Stearns, *Transactions*, ASCE, Vol. XII, 1883, p. 1.

[7] "Verification of the Bazin Weir Formula by Hydrochemical Gaugings," by Floyd A. Nagler, *ibid.*, Vol. 83, 1919, p. 105.

the maximum head is quite adequate, provided that other criteria are complied with.

Results of brief studies conducted in the hydraulic laboratory of the Bureau of Reclamation (United States Department of the Interior) show that it is extremely difficult to detect differences of head on enameled staff gages located at two, four, and ten times the head on the weir. These studies showed that positioning the enameled staff gage on the weir bulkhead, a practice sometimes followed in irrigation measurements, may result in errors. Some positions on the bulkhead, with respect to the weir notch, gave a higher reading for certain flows than a correctly positioned gage. At other flows the reading was less. As the gage on the bulkhead was moved away from the weir notch, more consistent results were evident. For the flow conditions tested it was found that when the gage was placed on the weir bulkhead at a minimum distance of twice the maximum recommended head for the weir, the difference in the heads read on this gage and one correctly placed upstream was within the limits of visual observation. Fig. 5 shows an enameled gage for observing head fastened close to the weir notch.

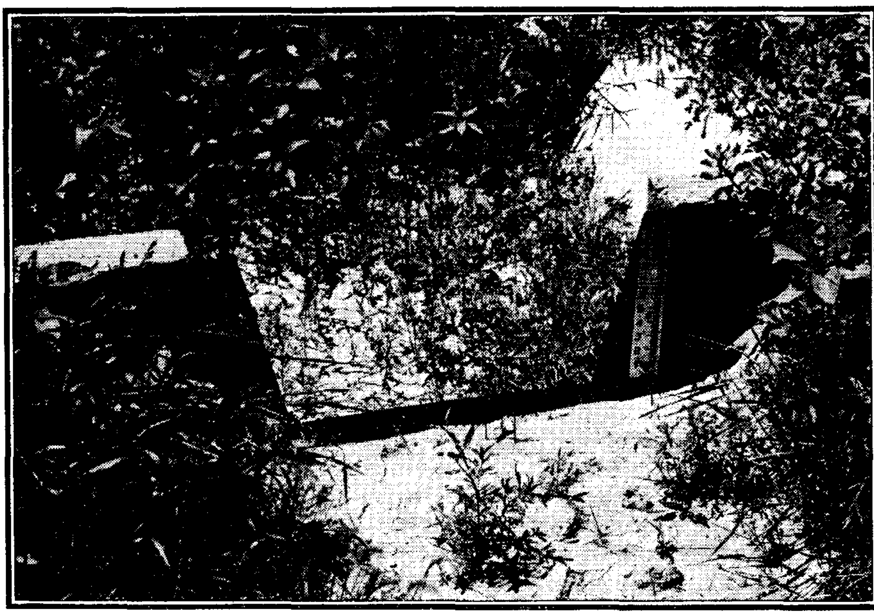

Fig. 5.—Enameled Gage Fastened Too Close to Weir Notch

When the velocity of approach is high and the irrigation channel has a high loss coefficient, there is a danger of placing the gage so far upstream from the weir crest that an error will prevail. Therefore, a correction must be made for the loss of head due to channel friction between the point of measurement and the weir.

Measurement errors can occur easily if the gages used in a Parshall flume are not placed in the manner and location developed in the standards. The ratings for this flume include a calibration of the gage positions. The gages are in drawdown areas. Under these conditions, movement of the gage upstream or downstream from the standardized location will change the head reading, and an error in discharge will result. For similar reasons if a stilling well is used the type and location of the entrance to the wells should be as specified. Substantial errors in field measurements have been traced to changes in location or design of the stilling-well entrances.

The foregoing statements apply to the location of the two gages used in the constant-head orifice turnout. The discharge tables developed from the calibration of this device are accurate only if the gages are placed as shown in the standard drawings.

Discharge Errors Due to Neglecting Velocity of Approach.—In practice the cross-sectional area of the approach channel can be made sufficiently large in

TABLE 3.—DISCHARGE ERROR (IN PERCENTAGE) RESULTING FROM FAILURE TO CORRECT FOR VELOCITY OF APPROACH

Velocity of approach, in feet per second	OBSERVED HEAD OVER WEIR, H, IN FEET				
	0.2	0.4	0.6	0.8	1.0
	DISCHARGE ERROR				
0.5	2.7	1.3	0.9	0.6	0.6
1.0	9.8	5.1	3.4	2.7	2.2
1.5	20.8	10.9	7.5	5.7	4.7
2.0	33.5	18.1	12.6	9.7	7.9
2.5	48.0	26.6	18.7	14.5	11.9
3.0	63.7	36.1	25.6	19.9	16.5

comparison with the weir notch to render the effect of the approach velocity negligible. However, if the approach velocity is not maintained at or less than 0.5 ft per sec, it must be taken into account and a correction must be applied. Thus, when discharge is obtained from measured head, an error will result if the equations, charts, or tables that are normally used are not corrected.

In irrigation practice the velocity of approach to a weir is usually increased to more than that for which it was originally designed by (*a*) a general restriction of the cross-sectional area of the weir pool as a result of vegetal-growth deposits, or (*b*) sediment or other accumulations at the bottom of the weir pool. Either item (*a*) or item (*b*) will change the standards to which the weir installation should conform.

A general reduction of the cross-sectional area of the weir pool will cause an increase in approach velocity that is related directly to the degree of restriction. The percentage error for a range of approach velocities and heads over weirs, except for the V-notch type, is given in Table 3. The error is such that the discharge is actually greater than that obtained from the discharge tables by the percentages shown in Table 3.

For accurate results the crest of the weir should be a distance that is not less than twice the depth of the water over the crest above the bottom of the approach channel. When practicable a greater weir-crest height is preferred. A weir installed in an irrigation channel in accordance with this standard may retain its accuracy for a short period only, because of the reduction of weir-pool depth by sediment deposits. The tables that are regularly used will no longer apply, but the error may be reduced or possibly eliminated by use of Rehbock's formula[8] for computing discharge from the head observations on a rectangular sharp-crested weir:

$$Q = K\,L\,H^{3/2} = \frac{2}{3}\sqrt{2\,g}\,(L\,H^{3/2})\left(0.605 + \frac{1}{320\,H - 3} + 0.08\,\frac{H}{P}\right) \quad (7a)$$

in which

$$K = 3.235 + \frac{1}{60\,H - 0.56} + 0.428\,\frac{H}{P} \quad (7b)$$

In Eq. 7*b*, *H* is the observed head over the weir, in feet, and *P* denotes the depth from the weir crest to the channel bottom.

TABLE 4.—ERROR IN DISCHARGE FOR CHANGES IN HEIGHT OF WEIR

Weir height, P (1)	H/P (2)	Coefficient, K (3)	Percentage error $(K - K_\infty/K_\infty) \times 100$ (4)
Head = 0.2 ft			
0.5	0.4	3.49	5.1
1.0	0.2	3.41	2.7
2.0	0.1	3.37	1.5
3.0	0.07	3.35	0.9
∞	0	3.32	0
Head = 0.5 ft			
0.5	1.0	3.70	13.1
1.0	0.5	3.48	6.4
2.0	0.25	3.38	3.4
3.0	0.17	3.34	2.1
∞	0	3.27	0

Table 4 shows the percentage error in discharge that will occur if regular weir tables are used instead of correcting for the reduced height of weir by use of the Rehbock formula. The table is divided into two parts. The first section is computed for a constant head of 0.2 ft over a weir, and the second section is computed for a constant head of 0.5 ft. The value of the ratio of *H* to *P* is varied, and the error is shown. This percentage error is introduced in the field by the improper maintenance and cleaning of the weir pool. As the pool fills, both the ratio of *H* to *P* and the error increase.

In the field there have been instances in which weirs have been placed in channels having relatively high gradients. Under these conditions, it is diffi-

[8] "Handbook of Hydraulics," by Horace W. King, McGraw-Hill Book Co., Inc., New York, N. Y., 3d Ed., 1939, p. 87.

cult to maintain a properly proportioned weir pool and to obtain smooth flow through the weir notch. The increased velocity of approach and turbulence will cause errors in measurement. Channel curvature and consequent, poor velocity distribution over the weir crest will also cause excessive errors that cannot be evaluated easily. Laboratory experiments[9] have shown that the extreme difference in discharge over a weir for a constant head, but with varied upstream velocity distribution, was 26%. A weir with poor approach conditions is shown in Fig. 6.

Discharge Error Due to Turbulence and Surges.—Turbulence and surges occur in approach channels to weirs and other measuring devices. These dis-

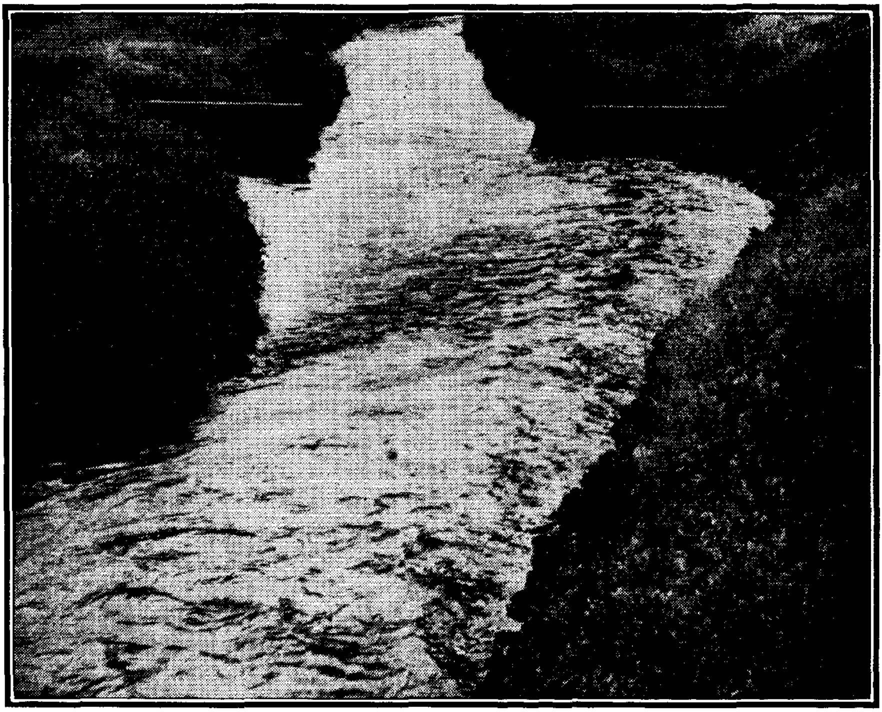

FIG. 6.—WEIR WITH POOR APPROACH CONDITIONS

turbances are usually evidenced by erratic measurement results. Their cause is ordinarily a high approach velocity, but gates, valves, and sudden changes in section may yield the same results. Disturbances on the surface rarely follow a true sine-wave pattern. Therefore, an average reading of the head may cause appreciable error. Corrections are not applied readily to the computations as the pattern is complex. Corrective measures to quiet the flow provide the best solution, but such measures may not be easy to apply.

Weir Blade Sloping Upstream or Downstream.—It is necessary that the plane of the upstream face of the weir be vertical to obtain accurate measurements.

[9] "Precise Weir Measurements," by Ernest W. Schoder and Kenneth B. Turner, *Transactions*, ASCE, Vol. 93, 1929, p. 999.

Experiments with sloping weirs show that the coefficient changes if the weir blade is tilted in an upstream direction or a downstream direction. The change is slight, and the weir face may be out of plumb a few degrees before the accuracy of the measurement is affected seriously.[10]

Roughness of Upstream Face of Weir and Bulkhead.—To obtain consistent and accurate flow measurements, the upstream face of the bulkhead and weir blade must be smooth. Offsets, protruding bolt heads, and surface roughness must be avoided on installation. Maintenance is necessary to retain a smooth surface. Sufficient work has not been performed to provide an exact evaluation of the errors resulting from the many possible roughnesses. It was found from one series of experiments[9] that

> "The percentage increase in discharge due to changing the roughness of the up-stream face of the weir bulkhead from that of a polished brass plate to that of a coarse file for a distance of 12 in. below the crest is shown to range from about 2% for 0.50-ft head to about 1% for 1.35-ft head."

The experiments of Floyd A. Nagler,[11] M. ASCE, showed that when the upstream face of the weir was roughened to the crest with coarse sand (retained on No. 8 standard sieve and passing No. 4 standard sieve), the increase in discharge ranged from 6.5% for a 0.2-ft head to 4.7% for a 0.5-ft head. The larger projections caused by the addition, in Nagler's experiment, of nuts and pieces of metal on the bulkhead below the crest caused a similar increase in discharge.

Rounding of Sharp Edge at Crest of Weir.—When weirs are constructed of wood the original sharp edge of the crest soon becomes rounded. Rust and corrosion also produce a rounding effect on metal weir blades. Rounding the edge causes an increase in the flow rate for a given head when compared with a sharp-crested weir. The results of experiments to evaluate the effect of the rounding of the crest show that the percentage increase in discharge due to the rounding decreases as the head increases. For a head of 0.5 ft increases of 2%, 3%, $5\frac{1}{2}$%, 11%, and $13\frac{1}{2}$% can be expected for roundings having radii of 1/24 in., $\frac{1}{8}$ in., $\frac{1}{4}$ in., $\frac{1}{2}$ in., and $\frac{3}{4}$ in., respectively. Data are lacking for the higher heads with larger radius roundings. However, with radii smaller than those described, the increases become consistently less as the head increases. For example, the 2% increase in discharge, given for the 1/24-in. rounding at a 0.5-ft head, becomes 0.7% at a 1.0-ft head and approximately 0.5% at a 1.35-ft head.

Submergence of Weirs.—When irrigation water is measured, weirs are usually not installed wherever submergence is anticipated. However, changes in the regimen of the channel downstream may cause a weir to operate under submerged conditions. Because submerged flow is relatively unstable, the results of the studies made of submerged weirs are not in agreement. It can be concluded that measurements made by use of a submerged weir should be considered as approximate.[4,5] If practicable, the cause of submergence should be removed from the downstream channel.

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[10] *Boulder Canyon Project Final Reports, Pt. VI, Hydraulic Investigations, Bulletin 3, Studies of Crest for Overflow Dams*, Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo., 1948.

[11] Discussion by Floyd A. Nagler of "Precise Weir Measurements," by Ernest W. Schoder and Kenneth B. Turner, *Transactions*, ASCE, Vol. 93, 1929, p. 1115.

Aeration of the Downstream Nappe of a Weir.—A general condition for accurate and consistent measurements by contracted weirs is that air circulates freely on all sides of the flow issuing from the weir notch. This condition ordinarily is not difficult to obtain. The weir bulkhead of irrigation structures in many instances is constructed of concrete. The use of metal weir blades that do not project sufficiently from the concrete, or an improper bevel of the concrete downstream from the blades, can easily restrict the desired air circulation. The effect of this restriction of air is to increase the flow rate for a given head. The increase in discharge can be appreciable and depends on the degree of air restriction.

The problem is more pronounced when suppressed weirs are used. For standard suppressed rectangular weirs used in irrigation, the side walls are generally carried straight through the structure. Thus, auxiliary means must be provided to supply air to the underside of the nappe. Unless adequate air is provided to this area to replace that removed by the jet, a partial vacuum will form. The result is a lowering of the nappe and an increase in discharge over that obtained with adequate aeration. A condition of instability may also exist, in which case erratic measurements would be obtained. Joe W. Johnson,[12] M. ASCE, found that the discharge would be increased approximately $3\frac{1}{2}\%$ at a 0.5-ft head and approximately 2% at a 1.0-ft head when the pressure under the nappe was reduced to only 0.8 in. of water below atmospheric. When the pressure was reduced further to 1.2 in. of water below atmospheric, the increase in discharge was approximately 5% and $2\frac{3}{4}\%$ for heads of 0.5 ft and 1.0 ft, respectively. The vent size adequate to relieve this negative pressure will depend on conditions at the weir. Both Johnson[12] and Joseph W. Howe,[13] M. ASCE, have developed solutions for computing the size of vents. The important consideration is to design the vents of adequate proportions to relieve the low pressure in so far as possible.

Other Factors Affecting Discharge-Measurement Accuracy.—Other factors may cause errors in discharge measurements made with weirs, and many apply equally well to other types of structures and devices.

Obstructions in the measuring section cause errors that are proportionate to the magnitude of such obstructions. In irrigation systems, floating detritus, weeds, moss, and other vegetation may obstruct the water passage. Frequent and close inspection accompanied by remedial measures will relieve this condition.

Changes in the viscosity and surface tension of the fluid alter the flow coefficient. However, the effect of these factors is negligible in irrigation systems in which the flow media is water, and wide variations of temperature are not encountered. Furthermore, provided that the restrictions on high and low heads over the weir are complied with, such changes are of little importance.

At very low heads flow over a weir may become quite unstable, and errors and inconsistencies in the measurements will result. Heads of less than 0.2 ft will not produce reliable results when the usual discharge tables or formulas

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[12] "The Aeration of Sharp Crested Weirs," by Joe W. Johnson, *Civil Engineering*, Vol. 5, No. 3, 1935, p. 177.

[13] "Aeration Demand of a Weir Calculated," by Joseph W. Howe, Go Chean Shieh, and Arturo Obadia, *Civil Engineering*, May, 1955, p. 59.

are used, because of viscous drag and the tendency of the nappe to adhere to the weir crest.

The results of many experiments on weirs show that the formulas developed for rectangular weirs do not apply when the head exceeds approximately one-third the length. There are indications that the discharge formula for the Cipolletti weir, in lengths of more than 1 ft, is slightly in error at heads that are less than one-third the length.[14] If errors are to be reduced to a minimum perhaps the rule should be that the head should not exceed one-fourth the length.

As stated previously, the flow formulas for weirs have been developed empirically and are not necessarily susceptible to extrapolation. Many of the data have been derived for heads as great as 2.0 ft. Although some data are available for higher heads, it is generally agreed that a 2.0-ft head should not be exceeded for a weir of any length if good results are desired.

It was noted previously that the percentage of error in discharge resulting from a given error in measuring the head will decrease as the head increases. Therefore, the minimum error and the greatest accuracy can be expected if the discharge occurs under the maximum head commensurate with the aforementioned limitations.

Careful visual inspections at regular intervals will remove many of the cited sources of error. These inspections should also disclose other sources, such as leaks around the measuring structure, through weir bulkheads, or from drains in the structure.

Conclusions

The charts, tables, and studies presented herein are not intended to indicate all the possible errors in the devices and structures used in measuring irrigation water. However, from the examples cited the following conclusions can be made:

To obtain accurate irrigation-water measurements it is necessary to study carefully the selection of a proper device to fit the conditions at the site. Even with careful planning and selection of an excellent primary measuring device, it is probable that errors may be introduced into the measurements unless care is exercised in fabrication, installation, operation, and proper maintenance of the devices or structures. The magnitude of these errors can be appreciable, and the value of a well-planned measuring program can be reduced considerably by not anticipating and removing the cause of the errors.

The possible errors are both negative and positive and may tend to cancel each other. However, especially in the case of weirs, more careful scrutiny shows that probably negative errors predominate. Indications are that usually more water is being delivered than is apparent from the measurements.

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[14] "The Discharge of Three Commercial Cippoletti Weirs," by Robert B. Van Horn, *Bulletin No. 85*, Eng. Experiment Station, Univ. of Washington, Seattle, Wash., November, 1935.

DISCUSSION

ÖDÖN STAROSOLSZKY[15].—The accuracy of flow measurement is related closely to the metering equipment used. Economic aspects indicate the necessity for investigations into the degree of accuracy and into possible errors, especially if water rates are based on similar observations.

The deficiencies that Mr. Thomas cites and that are encountered in practice are not accidental and tend, in general, to result in regular errors. However, it should be noted that accidental errors also occur that cannot be eliminated and must be taken into account. Therefore, it is practicable to establish limits for the resultant of accidental errors and regular (systematic) errors—for example, to specify the permissible tolerance. The measuring device cannot be accepted unless the deviation of the value observed during calibration from the theoretical value accepted as correct remains within the specified tolerance—that is, the accuracy limit expressed in percentage.

Individual errors encountered in practice can be traced back to errors in factors in the characteristic hydraulic relationship. The probable discharge error can be computed from the error of individual factors in the discharge formula by use of the error theorem of Karl Friedrich Gauss.[16] With the probable error, E, defined as the relative value,

$$E_Q = \sqrt{E_m^2 + E_b^2 + (a\,E_H)^2} \qquad (8)$$

in which E_Q, E_m, E_b, and E_H denote the probable errors of discharge, flow coefficient, characteristic length (the length of the crest in the case of weirs), and stage observation, respectively. The characteristic constant, a, of the measuring device (3/2 for rectangular weirs) is the exponent of the head, H, in the discharge relationship. Summing up individual errors is thereby possible.

The relative probable error being inversely proportionate to the head, the poorest accuracies are obtained by the measuring device at low heads. The minimum head that can still be used for measuring should therefore be specified as a function of the permissible probable error (for example, of tolerance) for each measuring device. The lowest head suitable for measurement, considering accidental errors only, depends on the graduation of the gage, the tolerance of the discharge measurement and of the flow coefficient, and the constant, a. The minimum heads required for discharge measurement within accuracy limits (tolerance) of 3.5% and 10%, respectively, have been compiled in Table 5 for gage graduations of 8/10 in., 4/10 in., and 2/10 in., and for a 2% error in the flow coefficient. As revealed by Table 5, no reliable results can be obtained for low heads. However, it is equally correct that test results are valid up to a certain head limit only. These factors should be considered during design.

High heads involve extensive head losses. Consequently, for the conditions prevailing in the Hungarian plains area, in which gradients are as low as from

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[15] Research and Hydr. Engr., Research Inst. for Water Resources, Budapest, Hungary.

[16] "Theoria motus corporum coelestium (Theorie der Bewegung der Himmelskörper—Methode der Kleinsten Quadrate)," by Karl Friedrich Gauss, Göttingen, 1809.

0.05 per 1,000 to 0.15 per 1,000, it was necessary to propose measuring devices involving low head losses at suitably high heads.[17]

Errors resulting from submergence are due partly to the inaccuracy introduced by the method of computation itself and partly to the unreliable determination of the limit of submergence. The influence of tailwater does not begin even in the case of sharp-crested weirs when the tailwater reaches the level of the crest, as shooting flow can still develop, but begins at a well-defined limit value of the tailwater-headwater ratio—for example, at the limit of submergence. However, the latter value depends on hydraulic conditions within

TABLE 5.—RELATIONSHIP BETWEEN ERROR IN MEASUREMENT AND MINIMUM HEAD FOR $F_m = 2\%$

Measuring device		Value of E_Q: 3			5			10		
		Gage Graduations, in Inches								
		8/10	4/10	2/10	8/10	4/10	2/10	8/10	4/10	2/10
Rectangular weir, Venturi flume ($a = 3/2$)	E_H	1.48			3.05			6.5		
	H_{min}	2.2	1.1	0.55	1.08	0.54	0.27	0.50	0.25	0.12
Thomson notch weir ($a = 5/2$)	E_H	0.90			1.88			3.9		
	H_{min}	3.6	1.8	0.9	1.74	0.87	0.43	0.84	0.42	0.21
Linear weir and flume ($a = 1$)	E_H	2.23			4.58			9.8		
	H_{min}	1.46	0.83	0.42	0.72	0.36	0.18	0.34	0.17	0.08
Venturi tube, submerged flow ($a = 0.7$)	E_H	3.17			6.50			13.9		
	H_{min}	1.04	0.52	0.26	0.50	0.25	0.12	0.24	0.12	0.06

the approach channel and on the degree of contraction as well as on the measuring device itself. A mathematical analysis of this type of error is thus extremely difficult.

As indicated by Mr. Thomas, head observations at sufficiently close intervals or continuous records (obtained by a recording gage) are essential to any reliable measurement of conveyed quantities because even the most accurate device will fail to yield reliable results if the observed discharge is other than representative. The head varies during the period between successive observations. The error involved in the determination of flow quantities is depend-

[17] "Vízgazdálkodási Tudományos Kutató Intézet: Az öntözővíz mérése (Irrigation Water Measurement)," by Z. Károlyi and Ö. Starosolszky, *Proceedings of the Research Inst. for Water Resources*, No. 1, Budapest, 1957.

ent, therefore, on the daily stage readings. An additional error is introduced into the determination of flow quantities by computing for the period between observations, using a discharge that is not representative of the flow passing during the period. The error in flow quantities is shown in Fig. 7 for a meas-

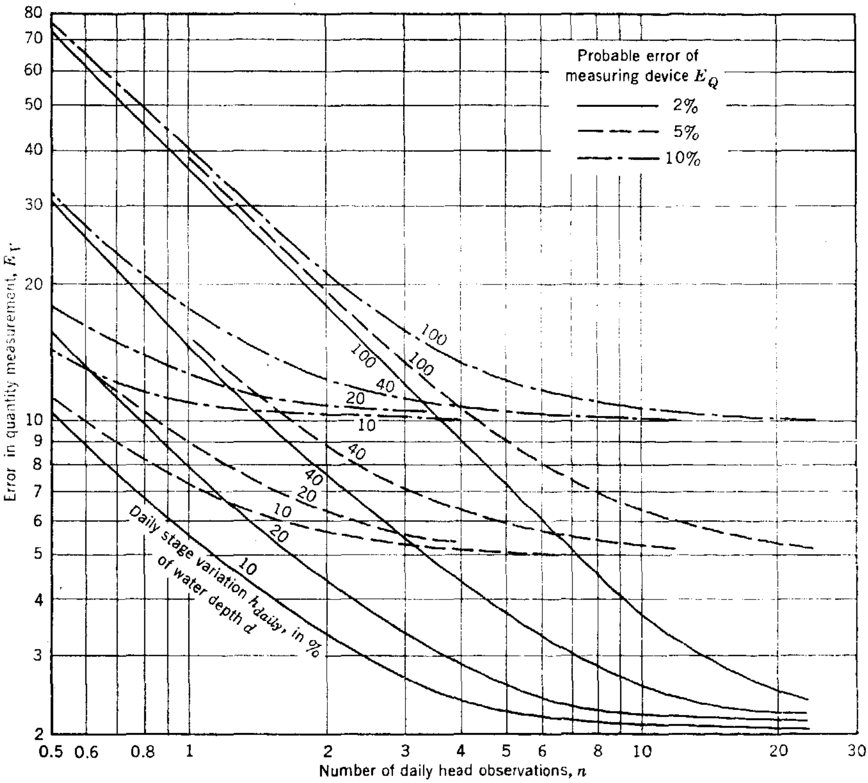

FIG. 7.—ERROR IN QUANTITY MEASUREMENT OF SUBMERGED FLOW

uring device operating on the staged difference principle (for example, a Venturi tube). The error in quantity measurement is determined as follows:

$$E_v = \sqrt{E_Q^2 + \left(\frac{1}{2}\,\frac{h_{\text{daily}}}{dn}\right)^2} \qquad (9)$$

The depth of flow in the section under consideration is d, and the daily variation in stage is h_{daily}. Thus, the relative daily variation is

$$100\,\frac{h_{\text{daily}}}{d} \qquad (10)$$

Furthermore, the number of daily readings is n. The value of the increment error has been assumed to equal the variation in stage between observations. Curves have been drawn for errors of 25% and 10% in the discharge measurement, E_Q. In general, readings taken once a day are unsatisfactory unless the stage fluctuation is small. Efforts to maintain a fairly constant water level in the system are, therefore, of great significance. The necessary daily gage readings can also be seen in Fig. 7.

In view of the foregoing a tolerance of from $\pm 10\%$ to 20% should be considered. The discrepancy due to accidental errors may attain the same magnitude.

STEPONAS KOLUPAILA[18].—Although the references in the paper contain material compiled and published in the United States, much research on similar topics has been performed by European engineers. Studies by B. Gentilini,[19] R. Hailer,[20] W. Dietrich,[21] O. Dillmann,[22] and others[23,24] demonstrate that a dangerous error is caused by different velocity distribution before the weir.

The author has applied the formula proposed by Theodor Rehbock in one of its older forms (1912). During a seventeen-year period Rehbock[25,26] collected 280 new measurements and improved his formula so that it corresponded better to dimensional requirements. This later formula was published in 1929 and should replace Eq. 7*a* as follows:

$$Q = \frac{2}{3}\sqrt{2\,g}\,L\,H^{\frac{3}{2}}\left(0.604 + 0.0813\,\frac{H}{P} + \frac{0.000895}{P}\right)\left(1 + \frac{0.00361}{H}\right) \quad . \quad (11)$$

ARMANDO BALLOFFET,[27] M. ASCE.—The fact that every measurement necessarily involves an error leads some field operators to consider any error as unavoidable. Consequently, all too often no possible correction is attempted for the results of field determinations.

The measurement error begins with an improper selection of the measuring device for the particular operating conditions. The practice of utilizing sharp-crested weirs for field measurements is perhaps a source of the unavoidable errors mentioned in the paper. Under laboratory conditions Cipolletti weirs, rectangular weirs, and V-notch weirs are accurate to precisions of 1.0%. When these weirs are installed in an irrigation ditch or canal, however, the calibration conditions are irretrievably lost unless those laboratory conditions are reproduced carefully. Such a precaution usually is impossible because of either the cost of installation or maintenance.

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[18] Prof. of Civ. Eng., Univ. of Notre Dame, Notre Dame, Ind.

[19] "Stramazzi in parete sottile liberi e rigurgitati," by B. Gentilini, *L'Energia Elettrica*, Vol. 13, Nos. 3, 5, and 10, 1936.

[20] "Fehlerquellen bei der Überfallmessung," by R. Hailer, *Mitteilungen des Hydraulischen Instituts der Technischen Hochschule München*, Munich, Vol. 3, 1929, pp. 1–21.

[21] "Wassermessungen mit Ueberfall in der Zentrale Handeck der Kraftwerke Oberhasli," by W. Dietrich, *Schweizerische Bauzeitung*, Vol. 99, Nos. 1 and 2, 1932, pp. 1–4 and pp. 20–22.

[22] "Untersuchungen an Überfällen," by O. Dillmann, *Mitteilungen des Hydraulischen Instituts der Technischen Hochschule München*, Munich, Vol. 7, 1933, pp. 26–52.

[23] "Der Einfluss der Geschwindigkeitsverteilung auf den Überfallbeiwert von Messwehren," by H. Gerber, *Wasserkraft und Wasserwirtschaft*, Vol. 32, Nos. 10–11, 1937, pp. 119–123.

[24] "Neue Untersuchungen an Überfällen mit Seiteneinschnürung," by E. Zschiedrich, *Mitteilungen aus dem Flussbaulaboratorium der Technischen Hochschule Dresden*, Leipzig, 1939.

[25] "Wassermessung mit scharfkantigen Überfallwehren," by Th. Rehbock, *Zeitschrift des Vereins Deutscher Ingenieure*, Vol. 73, No. 24, 1929, pp. 817–824.

[26] "Die Stetigkeit des Abflusses bei scharfkantigen Wehren," by Th. Rehbock, *Der Bauingenieur*, Vol. 11, No. 48, 1930, pp. 821–826.

[27] Prin. Design Engr., Tippetts-Abbett-McCarthy-Stratton, New York, N. Y.; Lecturer in Civ. Eng., Columbia Univ., New York, N. Y.

The calibration of measuring devices should not generally be extrapolated beyond the range of observations. Figs. 1, 2, 3, and 4 show that the most important errors are a result of not appraising the correct approach velocity either because the bottom upstream of a weir has silted or because the gage was located incorrectly.

The disadvantages of the devices cited in the paper leads to the belief that the best field results can be expected from a device having the following characteristics:

1. No tendency to cause sediment deposition in the upstream channel;
2. Ability to produce the flow conditions required for the measurement by itself (that is, independence of the upstream section);
3. Sufficient accuracy under adverse conditions; and
4. Safe extrapolation of calibration within the range of conditions encountered in practice.

In most instances weirs and orifices fail to meet all these conditions. They may cause a rise in the level of the bottom of the canal, and care must be taken to maintain the specified upstream depth. In addition, the approach velocity depends on the section of the canal upstream from the weir, which may change as a result of silt deposition, weeds, or sliding of banks. The accuracy of weirs is affected directly by the off-laboratory conditions of irrigation operation. As a result, unless rather expensive precautions are taken when the weirs are installed or operated, their apparent simplicity is deceptive.

However, Venturi flumes are installed in such a manner that they do not cause an appreciable rise in the bottom elevation, thus avoiding most of the inconvenience derived from silt deposition. The flumes may be prefabricated easily to contain all the required approach conditions in a single unit. Because the flow meter acts as a control section, the discharge determines the upstream depth in the same way as for a weir under laboratory conditions.

In the United States the most commonly used critical flow meter is the Parshall flume. It is an excellent measuring device when it is used strictly within the range of discharges, heads, and dimension ratios originally used when calibrated. Usually calibrations cannot be extrapolated because the critical section is in a convergent canal. The discharge coefficients must change substantially unless the angle of convergence and the remaining geometrical parameters are kept equal or proportional.

A review of several types of critical flow meters developed in several European countries, India, and Argentina[28] showed that, in general, care was taken to define precisely the control or critical section in a long reach of rectangular and horizontal canal. The experiments of Giulio De Marchi and Francesco Contessini,[29] as well as the calibration by the writer of several models,[28] demonstrated that if this simple precaution is taken, even with the use of fairly abrupt transition curves at the entrance of the measuring section, meter equation fulfils the theoretical discharge relationship with an error of approxi-

[28] "Critical Flow Meters (Venturi Flumes)," by Armando Balloffet, *Proceedings Paper No. 743*, ASCE, July, 1955.

[29] "Dispositivo per la misura della portata dei canali con minima perdita di quota," by Giulio De Marchi and Francesco Contessini, *L'Energia Elettrica*, January, 1936–March, 1937.

mately 5% or less. This degree of precision is satisfactory in many field installations. The discharge coefficient to correct the theoretical equation was determined with an error of about 1%. In the writer's experiments, several models showed a remarkable constancy of this empirical discharge coefficient with respect to discharge.

The theoretical equation is based on the critical flow on the control section of the meter. If this section is well defined, the Froude number upstream from the section is related to the discharge coefficient by a numerical constant. If the Froude analogy is accomplished when the calibration results are extrapolated to a different meter, the discharge coefficients should be equal. The results of field calibration of one such meter with a discharge that was thirty-nine times the maximum laboratory discharge and a ratio of upstream depth to approach width that was 60% greater showed no difference in discharge coefficients beyond the range of experimental errors (approximately 1.5%).[28]

The independence of the discharge coefficient from the approach depth-to-width ratio has also been reported by H. R. Vallentine for sharp-edged contractions.[30] This property applies mostly to models with plane bottom between the gage section and the critical section.

Due to the relatively negligible difference between the theoretical equations and the actual equations when the critical section is well defined, Corrado Ruggiero and Piero Giudici[31] designed meters to fulfil a prescribed error law for a wide range of discharges. This law caused the magnitude of the errors in discharge due to inaccurate gage readings to be an inverse function of the annual discharge frequency. The meters could give a weighted indication of the volumes measured throughout the year. Although the critical sections for those meters were not rectangular, laboratory tests showed a fairly satisfactory agreement between the predicted and the experimental rating curves.

Finally, because most of the accelerated and curvilinear flow is close to or within the narrowed section, the location of the measuring scale in this type of meter need not be as precise as for the Parshall flumes.

CHARLES W. THOMAS,[32] M. ASCE.—The necessity for obtaining continuous records or representative head observations if accuracy is to be maintained has been emphasized further by Mr. Starosolszky. Presumably, each curve in the three families of curves shown in Fig. 7 represents a different daily percentage variation in differential head at the Venturi meter. However, the curves do not indicate what assumptions were made in computing each one.

In the case of a weir, the discharge varies more rapidly than the head. The relationship given by many weir formulas shows that the discharge varies as the 3/2 power of the head. Therefore, the volume of discharge obtained by reading the gage at intervals and averaging the readings will yield a discharge that is somewhat less than that actually passing the weir.

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[30] "L'écoulement dans des canaux rectangulaires présentant une section rétrécie," by H. R. Vallentine, *La Houille Blanche*, January–February, 1958.

[31] "Sui misuratori di portata a grande campo di variabilita," by Corrado Ruggiero and Piero Giudici, *L'Energia Elettrica*, April, 1952.

[32] Hydr. Engr., Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.

Assuming an initial head of zero (H_0) and a uniform rise to a depth H_2 at the end of the time interval, T, Robert E. Horton[33] showed that the ratio of discharge by integration is

$$\frac{\text{Volume by average head}}{\text{Actual volume}} = \frac{(\frac{1}{2})^{\frac{3}{2}}}{\frac{2}{5}} = 0.884$$

or the true discharge equals 1.13 times the discharge obtained by using the average head for the two readings taken at the beginning and end of the period.

This percentage of error will be the same regardless of maximum head (H_2) and for a rising stage or falling stage if the initial head (H_0) is zero. The error resulting from computing the discharge using the mean head will decrease as the ratio H_1/H_2 increases if there is an initial depth of flow, H_1, over the weir. The percentage of error will decrease as the readings per time interval are increased and approach zero, as for continuous records.

Short period surface fluctuations approach wave conditions. Arnold H. Gibson[34] has shown that wave disturbance increases the discharge over a weir. When waves are present the discharge can be expressed as $(1 + K)$ times the discharge under the same observed head with undisturbed water in the approach channel. The values of K derived by Mr. Gibson are:

With waves of sinusoidal form, for a thin-crested rectangular weir,

$$K = 0.19 \left(\frac{a}{H}\right)^2 \ldots\ldots\ldots\ldots (12)$$

and for a 90° V-notch weir,

$$K = 0.94 \left(\frac{a}{H}\right)^2 \ldots\ldots\ldots\ldots (13)$$

In Eqs. 12 and 13, a is equal to one-half the wave length.

With waves of trochoidal form the effect is less than with sinusoidal waves. The value of K for rectangular weirs varies from $0.070\ (a/H)^2$ to $0.182(a/H)^2$ for ratios of wave length over wave height varying from 3.14 (sharp-crested wave) to 15.0, respectively. For the same range of length-height ratios the value of K varies from $0.50\ (a/H)^2$ to $0.92\ (a/H)^2$, respectively, when applied to a 90° V-notch weir.

The error is negative as in many of the instances cited in the paper. Thus, when waves are present more water is delivered through the weir notch than the gage readings of the mean water level indicate.

With respect to the comments by Mr. Kolupaila, the purpose of the paper was to emphasize some of the reasons why measurements made in connection with the conveyance and delivery of irrigation water may not be as accurate as assumed. Also it was intended to indicate the order of magnitude of the error resulting from a few common deviations from accepted practice in the

[33] "Weir Experiments, Coefficients and Formulas," by Robert E. Horton, *Water Supply and Irrigation Paper No. 200*, Geological Survey, U. S. Dept. of the Interior, Washington, D. C., 1907.

[34] "The Effect of Surface Waves on the Discharge Over Weirs," by Arnold H. Gibson, *Selected Engineering Papers No. 99*, Inst. C. E., London, 1930.

general use of standard devices in the field. Much material has been and is being written on weirs. These devices have been subject to more hydraulic laboratory investigations than any others. Space limitations prohibit a complete list of references for weirs and other field measurement devices.

Engineers in the United States appreciate fully the value of writings done in other countries. Many articles by European engineers on weirs and other subjects have been studied and translated in full or abstracted, and the contents are well known. For example, the paper by Hailer[20] has been translated, and the work of Schoder and Turner[9] can be considered an international symposium on sharp-crested weirs and a review of earlier European material.

The statement by Mr. Balloffet that "the measurement error begins with an improper selection of the measuring device" merits extensive consideration. The selection of the proper measurement device to best fit the physical conditions at the site would solve many operating problems. With respect to economics, the cost of a structure should be considered to include more than initial cost or first cost. To operate and maintain a weir installation in such condition that it will give reasonably accurate results is usually expensive.

A Venturi flume can measure a wide range of discharges accurately, and, for this reason, should be used more extensively. This type of device has been studied and is recognized on an international basis, as for example the interesting flow relationships in these devices established by Friedrich V. A. E. Engel.[35,36]

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[35] "Non-uniform Flow of Water," by Friedrich V. A. E. Engel, *The Engineer*, April 21 and 28, 1933; May 5, 1933.

[36] "The Venturi Flume," by Friedrich V. A. E. Engel, *ibid.*, August 3 and 10, 1934.