THE HYDRAULIC PROPERTIES OF ORIFICES AND CIRCULAR WEIRS WITH A 45 DEGREE SLOPE

By

ARMOND DUDLEY BAREFOOT

Bachelor of Science

Oklahoma State University

Stillwater, Oklahoma

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Thesis Approved:

Thesis Adviser

Charles E Rice

F. R. Crow

N Durham
Dean of the Graduate College

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CHAPTER I

INTRODUCTION

Labor requirements, working conditions, and the unavailability of labor continue to impede the growth, and often the success, of surface irrigation. Low water application efficiencies (normally in the 50 percent range) and the difficulty of obtaining uniform distributions are other major problems which may determine the degree of success of a surface irrigation system.

Sprinkler irrigation has made giant strides in recent years by designing to reduce labor requirements. The design of many sprinkler systems uses the unavailability of labor as a primary restraint.

The logical solution to surface irrigation problems would be to design systems capable of reducing labor and improving uniformity. Garton (7) described a procedure to accomplish these objectives. The system he proposed uses a series of level bays with short tubes with hood inlet entrances to discharge the water into the furrows. It utilizes an initial flow and a cut-back flow. The initial flow should be small enough to prevent excessive erosion but large enough to prevent excessive deep percolation. The cut-back flow should equal the intake rate of the soil plus evaporation losses.

Labor requirements are reduced drastically by this method as only the lowering of a check dam is necessary to move from one bay to the next. Labor can be virtually eliminated by use of automated check dams which may be actuated by a time clock or soil moisture sensing element. The level bay concept insures uniform discharge from each outlet tube which is not possible when irrigating from a sloping channel with siphon tubes. A more uniform distribution down the row is usually accomplished by cut-back irrigation. Water application efficiencies should be increased by reducing deep percolation losses and/or tailwater losses.

Approximately one-third of the construction cost of this system is related to the installation of the short tubes. The tubes must be placed at the same elevation in each bay with the hood inlet oriented and projecting into the channel one diameter or greater. The tubes must be installed level in a foundation capable of holding them in this position. Instead of tubes, sheet metal orifices or weir plates attached to the channel bank should accomplish the same objectives with easier installation and less cost if the hydraulic properties could be defined.

Cut-back flows usually vary from about one-third to two-thirds of the initial flow. If the initial flow is three times the cut-back flow, this will require a variation in head of approximately nine to one for orifice flows. This amount of head may not be available so it may be necessary, for some conditions, to use circular weir flows or flows in the transition zone between weir flow and orifice flow.

Limitations of the Study

The study was limited to only 45 degree slopes with the plates installed on the side of the experimental channel. Only one plate thickness was used. Seven diameters of 1.0 through 8.0 inches were tested for heads of zero through 0.4 foot.

Objectives

1. To determine the head versus discharge relationships for sloping plate orifices.
2. To determine the head versus discharge relationships for sloping circular weirs.
3. To determine the head versus discharge relationships for the transition zone between weir and orifice flow.
4. To determine the range of applicability of each of the above relationships.

Definition of Symbols

Symbol	Quantity	Dimensions
A	Area of orifice	ft^2
C	Coefficient, discharge	dimensionless
C_c	Coefficient, contraction	dimensionless
C_d	Coefficient, discharge	dimensionless
C_v	Coefficient, velocity	dimensionless
D	Diameter of orifice	ft
E	The complete elliptic integral of the second kind	dimensionless
g	Gravitational acceleration	ft/sec^2
h	Head above centerline of orifice	ft
H	Head above invert	ft
k	Modulus of the elliptic integrals = $\sqrt{H/D}$	dimensionless

Symbol	Quantity	Dimensions
K	The complete elliptic integral of the first kind	dimensionless
K	Coefficient, discharge	nonhomogeneous
L	Coefficient, exponent	dimensionless
L	Length of rectangular weir	ft
M	Coefficient, discharge	nonhomogeneous
N	Coefficient, exponent	dimensionless
Q	Discharge	cfs
Q_t	Discharge, theoretical	cfs
y	Distance from invert to dy	ft

CHAPTER II

REVIEW OF LITERATURE

This chapter contains basic hydraulic theory and previous research pertinent to the study. The principal topics reviewed were discharge of a circular orifice, discharge of a circular weir, side weirs, sloping orifices and weirs, and the effect of vortices.

Circular Orifices

The orifice flow formula $Q = CA\sqrt{2gh}$ is generally used to determine discharge for all orifices. The theoretical discharge is $Q = A\sqrt{2gh}$. The true discharge from an orifice is always less than the theoretical discharge because of the effects of friction, viscosity and the contraction of the area as flow passes through the orifice. The coefficient of discharge, C, is the product of C_v, coefficient of velocity, and C_c, coefficient of contraction. The effects of friction and viscosity are very small for sharp-edged orifices, the average value of C_v being about 0.98. The mean value of C_c for sharp-edged orifices is approximately 0.62; therefore, the principal decrease in discharge from the theoretical is caused by the contraction of the jet.

King (9) stated,

> Values of C_c and C_v are difficult to obtain experimentally, and these coefficients are of theoretical rather than practical value. Numerical values of C are obtained by measuring the discharge from an orifice of known dimensions and determining the ratio of this discharge to the theoretical discharge.

Coefficients of discharge of sharp-edged circular orifices for various diameters and heads are included in most handbooks of hydraulics.

Circular Weirs

Cone (4) stated in 1916,

> Apparently no experiments have ever been made with circular or semi-circular notches placed in a vertical position with heads less than the height of the opening.

He proceeded to conduct experiments through two circular weirs approximately 0.5 and 1.0 foot in diameter and two semi-circular weirs approximately 1.5 and 2.0 feet in diameter. The results were given graphically for each diameter. No data were given and no attempt was made to develop a flow equation.

Greve (8) conducted extensive circular weir experiments. He used a concrete channel 6 feet deep, 8 feet wide at the entrance end and 5 feet wide at the discharge end, and 38 feet long. He conducted tests for 14 plate diameters ranging from 0.250 to 2.495 feet. He used an empirical formula of the form $Q = M(H)^N$ to describe his results. A value of M and N was determined for each diameter from the above equation. He used an average value of N to calculate M for each diameter. The relation of M to diameter was determined in the equation $M = K(D)^L$. Combining these two equations gave a general formula

$$Q = K(D)^L(H)^N.$$

Greve stated in the results of his paper that,

> A change occurs in the logarithmic relation between head and discharge when the depth in the plane of the notch equals the radius and again when the depth is equal to the maximum hydraulic radius or 81% of the diameter. It may be well to emphasize that a change in the slope of the graph

depends directly upon the depth at the notch and that weir flow may persist even though the observed head be greater than the diameter.

Data were too few to permit the drawing of conclusions concerning the correlation between head and discharge for depths greater than the radius. The empirical formula for the actual rate of discharge is:

$$Q = M(H)^{1.87} \tag{19}$$

. . . . Using 1.870 for the average value of N in the preceding equation will lead to a probable error of ± 0.0036 for the entire group of notches and of ± 0.0130 for any one notch. The tabular values of M were computed using N = 1.870.

The relation of M to diameter is:

$$M = 179(D)^{0.637} \tag{20}$$

Therefore

$$Q = 179(D)^{0.637}(H)^{1.87} \tag{21}$$

The maximum difference between Q by experiment and Q as computed from Equation 21, based upon the former quantity, ranged from +0.79 to -1.79%.

The discharge in the above equations was given in pounds per second. When converted to cfs, Equation 21 became

$$Q = 2.87(D)^{0.637}(H)^{1.87}.$$

Of the 14 plate diameters tested by Greve, only three were in the range used in this study. Analysis of his 0.250, 0.500 and 0.666 foot diameters gave the following equations:

$$Q = 2.858D^{0.563}H^{1.884} \text{ for } H < 0.5D \tag{1}$$

$$Q = 2.511D^{0.824}H^{1.660} \text{ for } H > 0.5D \tag{2}$$

The standard deviations of Equations 1 and 2 were 0.001 and 0.006 cfs, respectively.

Stevens (13) used an elliptic formula to determine the theoretical flow through vertical circular weirs. The formula was:

$$Q_t = 4/15 \sqrt{2g}\, D^{5/2} \left[2(1-k^2 + k^4)\, E - (2-k^2)\ (1-k^2) K\right]$$

where

$k = \sqrt{H/D}$

K = the complete elliptic integral of the first kind

E = the complete elliptic integral of the second kind

The above formula was developed by transformation of the basic hydraulic equation

$$Q_t = 2\sqrt{2g} \int_0^H \sqrt{(D-y)\ y\ (H-y)}\ dy$$

where

y = the distance from the invert to the increment dy

He used experimental data by Cone, Greve, Dodge (5), and Thijsse (15) to obtain an average coefficient of discharge of 0.59. The elliptic formula was modified and used to determine some tables of discharge for various heads and diameters. A table of values of $[2(1-k^2 - k^4)\ E - (2-k^2)\ (1-k^2)\ K]$ for values of k^2 from 0.001 to 1.000 was also included.

Concerning the experimental data used, Stevens stated,

> The Greve experiments are the most consistent of all and are believed to be entirely trustworthy.
>
> It is hoped that some laboratories in this country will make more experiments on the flow through circular sharp-crested weirs and thereby obtain more discharge coefficients.

Side Weirs

Collinge (3) reviewed side weir experiments prior to 1956. He also conducted some side weir experiments in a rectangular channel constructed such that the channel width, weir length and weir height above the channel could be varied. He concluded that for a free nappe the apparent value of the exponent of head did not change from the general sharp-crested rectangular weir equation for discharge.

The general equation is:

$$Q = C_d \sqrt{2g}\, Lh^{1.5}$$

The percent reduction in C_d, coefficient of discharge, for a free nappe and tranquil (subcritical) flow was approximately equal to 0.3 times the mean velocity of flow along the weir in feet per second.

No previous research was found for circular side weirs; however, the above research indicated only a very small decrease in discharge for a rectangular side weir as compared to a rectangular weir.

Sloping Orifices and Weirs

Bilton (2) concluded, "The discharge of a circular orifice under any given head is the same, whether the jet be horizontal, vertical, or at any intermediate angle.". The orifice flow formula $Q = CA\sqrt{2gh}$ could be used to determine the discharge for sloping orifices; however, the value of C would vary at low heads and small diameters.

Etcheverry (6) stated,

> The crest of the weir should be level from end to end, and the weir board placed vertically. The importance of having the weir board placed exactly vertical is not great, and is shown by the following results of experiments

made by Bazin (1) on thin-edged weirs inclined at various angles.

Relative Discharge of Inclined Weirs and
Vertical Weirs by Bazin's Experiments

		Bazin's Modulus
Vertical Weir		1.00
Upstream Inclination Of The Weir	(1 horizontal to 1 vertical	0.93
	(2 horizontal to 3 vertical	0.94
	(1 horizontal to 3 vertical	0.96
Downstream Inclination Of The Weir	(1 horizontal to 3 vertical	1.04
	(2 horizontal to 3 vertical	1.07
	(1 horizontal to 1 vertical	1.10
	(2 horizontal to 1 vertical	1.12
	(4 horizontal to 1 vertical	1.09

Source: B. A. Etcheverry and S. T. Harding, Irrigation Practice and Engineering, Vol. III, McGraw-Hill, New York, 1916.

Bazin's Modulus was a ratio of the discharge for various slope conditions divided by the discharge of a rectangular vertical weir with the same dimensions and head.

Cone (4) obtained similar discharge increases by sloping a 2-foot rectangular weir downstream.

Effect of Vortices

Posey (12) tested a 4-inch, sharp-crested circular orifice centered in the bottom of a 6-foot diameter tank with a constant head of 1.63 feet. He found that as the strength of the vortex above the orifice increased the orifice discharge decreased. The orifice discharge reduction was related to a factor called tan Θ. Tan Θ was defined as the average tangential component of velocity divided by the average radial component of velocity. Tan Θ was constant for any radial distance from the center of the orifice. The vortex was controlled by making the inflow tangential, radial or combinations of the two. Posey concluded that, if there were no tangential component, no vortex would form. However, if a vortex did form, it had a negligible effect on the discharge coefficient.

Stevens (14) obtained similar results; however, he related the strength of the vortex to a circulation factor. He controlled the tangential component by a series of vanes. The tangential component enhanced stability of the vortex. Vortices with uniform approach conditions had small tangential components and were small and unstable with no definite pattern as to direction of rotation.

CHAPTER III

EXPERIMENTAL EQUIPMENT

The Channel

An experimental plywood channel was constructed in the Agricultural Engineering Laboratory at Oklahoma State University. The trapezoidal shaped channel was 16 feet long, 2 feet deep with a 1 foot bottom width and 1:1 side slopes. This is the standard shape of the concrete lined slip-form irrigation channels being built in this area. A general view of the experimental channel is shown in Figure 1.

Five-eighths-inch plywood sheets were used for the channel, bracing and base. The joints were sealed, sanded, and the channel painted. The base was 4 feet wide with 2- by 10-inch wooden runners on 20-inch centers perpendicular to the channel. The channel was leveled by shimming the runners. The entrance end of the channel was closed and the discharge end was left open. A butyl rubber check gate was attached to the channel near the discharge end. A hand crank and gear attached near the end of a 1-inch shaft was installed across the channel about 3 feet from the discharge end. The shaft was fastened to the floor at each end by a metal framework. A small cable connected the top center of the gate and the center of the shaft. The desired elevation of the gate was obtained by rotating the shaft. A chain attached to the channel bracing and gear held the gate at the desired elevation. The discharge end of the channel is shown in Figure 2.

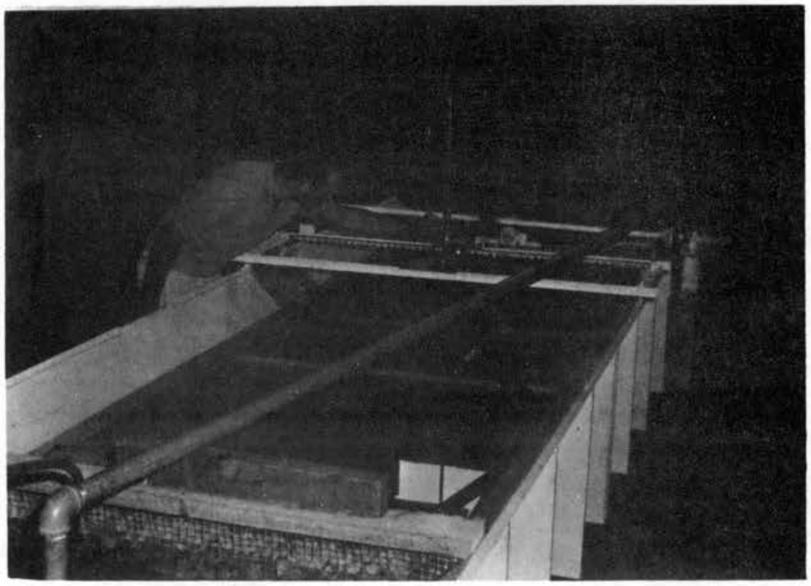

Figure 1. The Head Measurements in the Experimental Channel Were Made with a Point Gage on a Traversing Carriage.

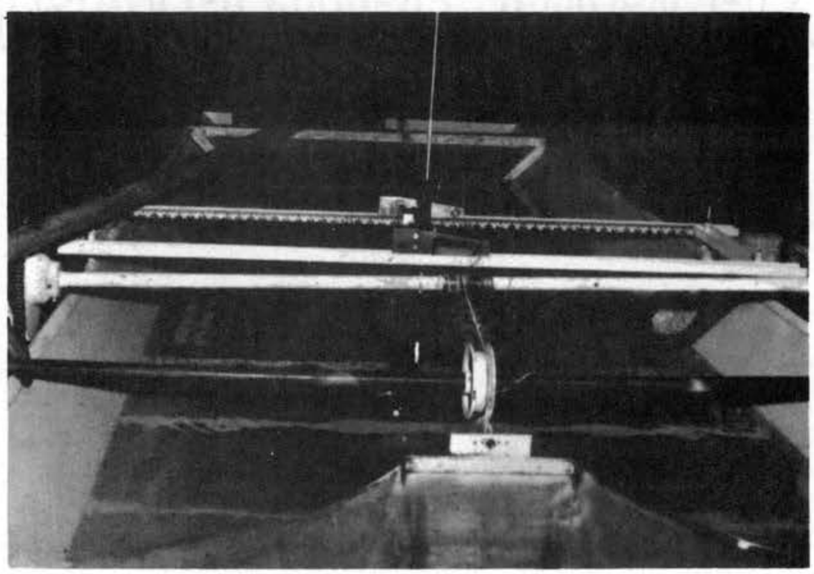

Figure 2. Downstream View of the Experimental Channel Showing the Adjustable Check Dam.

System Cycle

Inflow into the circulatory system was accomplished by 2-inch and 6-inch supply pipes. The inflow was discharged vertically about 3 inches from the bottom of the channel. A rock baffle 4 inches thick was placed 30 inches from the entrance end. Sheet metal stripping about 5 inches wide was attached to the downstream side of the baffle. The spacing of the strips were 2 inches horizontally and 6 inches vertically. This should produce flow lines approximately parallel to the channel.

The channel was elevated about 10 inches above the floor and discharged into a sump with dimensions of 9 feet by 5 feet by 5 feet.

Head Measurements

A 1- by 1- by 1/8-inch structural steel angle was attached to the top edges of the channel to support a traversing carriage and point gage. The carriage was mounted on four adjustable shanks welded to a 3-inch strip of 3/4- by 3/4- by 1/8-inch angle to allow mobility and leveling along the channel. The point gage and carriage was used to measure the channel profile, water surface elevation and the invert elevation.

Orifice and Weir Plates

The orifice and weir plates were made from 18 gage galvanized sheet metal. Seven diameters ranging from approximately 1 to 8 inches were selected for the experiment. The holes were cut in the seven plates by use of a rotary expansion bit. The invert of each hole was located the same distance from the bottom of each plate. A notch was cut in the side of the channel 13 inches wide and 21 inches long. The plates were 16 inches by 22 inches and were attached to the interface of the channel by recessed wood screws. A caulking compound was used between the plate and channel to prevent leakage. Each plate was used for both circular weir and orifice flows. Figure 3 shows circular weir flow entering the 8-inch plate. Figure 4 shows discharge from the 8-inch plate.

Flow Measurements

Large inflow into the system was measured with the Sparling meter shown in Figure 5. The Sparling meter was calibrated with a sharp-edge orifice and U-tube manometer at the Outdoor Hydraulic Laboratory near Stillwater, Oklahoma. The Sparling meter was installed in the 6-inch inflow pipe. A nutating disk water meter calibrated by weight and time measurements was installed in the 2-inch pipeline to measure small flows.

Discharge measurements were made by use of a stop watch, bucket and calibrated scales at low flows. The 0.75-foot H-flume, shown in Figure 6, was used at high flows. The flume was calibrated in place and discharged into the sump.

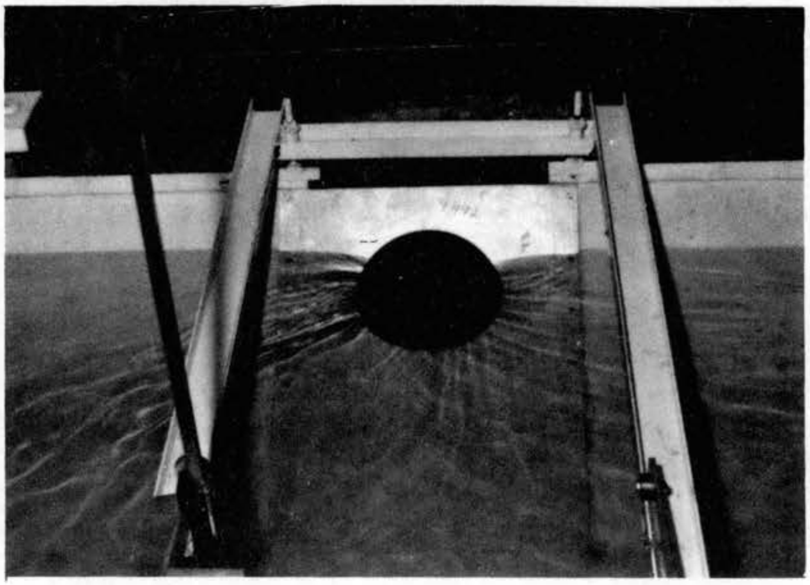

Figure 3. Circular Weir Flow Entering the 8-Inch Diameter Plate.

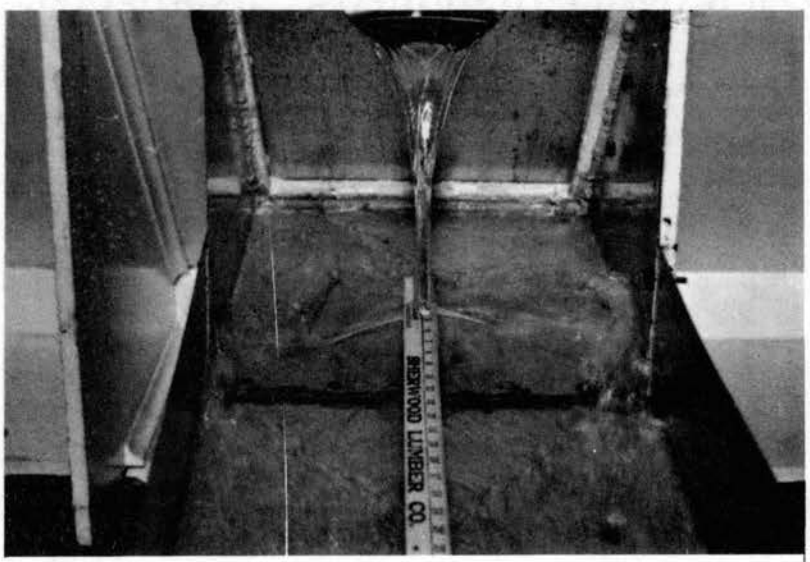

Figure 4. Circular Weir Discharge From the 8-Inch Diameter Plate.

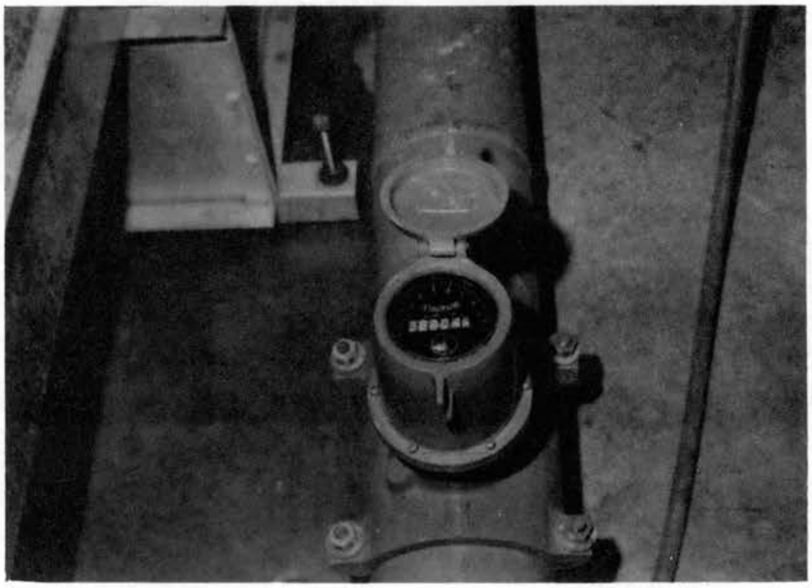

Figure 5. A Sparling Meter Was Used for Inflow Measurements.

Figure 6. A 0.75 H-Flume Was Used for Discharge Measurements.

CHAPTER IV

PRELIMINARY INVESTIGATION AND PROCEDURE

A preliminary investigation to determine the effect on discharge of height of the plate above the channel bottom was thought necessary. The effect of decreasing spatially varied flow was also investigated.

Plate Height Above Channel Bottom

The influence of height above the channel bottom on discharge was investigated using 2-inch diameter plates. Vertical heights from the bottom of the channel to the invert of the orifice plate of approximately 11, 16 and 19 inches were used. No significant differences in discharge were detected at the three plate locations. The data collected are shown graphically in Figure 7 and are included in Appendix A. Figure 7 also shows the best fit curve for the 2-inch plate calculated later in the study. The solid line represents weir flow and the dashed line orifice flow.

Decreasing Spatially Varied Flow

Decreasing spatially varied flow occurs along an irrigation distribution channel. Four flow rates past the plate were selected to simulate this condition. Flow rates of approximately 0.0, 0.5, 1.0, 1.5 and 2.0 cfs past a 2-inch diameter plate were used. Several wooden

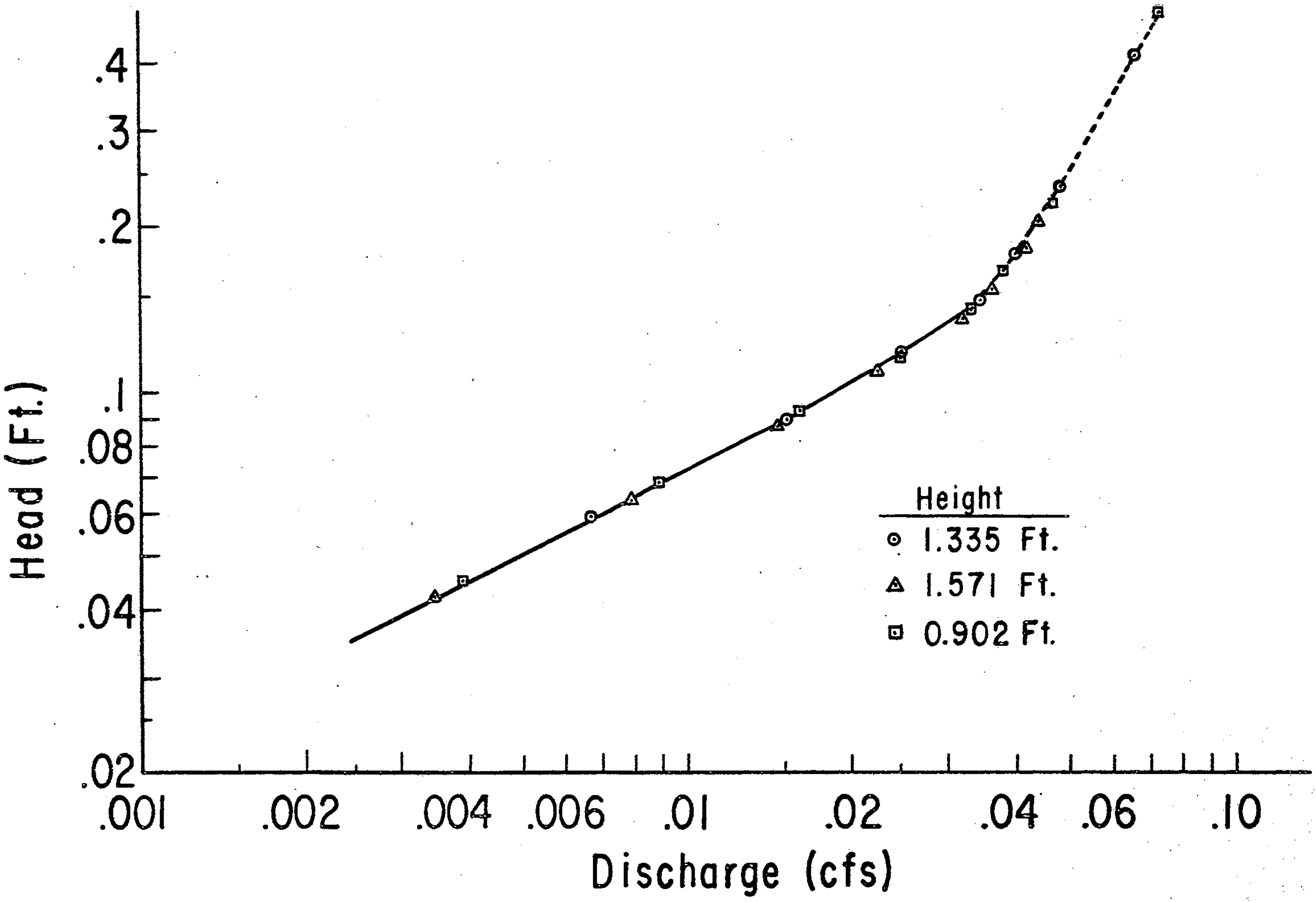

Figure 7. Head Versus Discharge for Three Plate Heights.

floats were installed upstream from the plate to decrease wave action for the high flows.

Data were collected for the five inflow rates at eight headwater elevations. The head and discharge data are included in Appendix A and are shown graphically in Figure 8 with the best fit curve described above. The upper weir flow data point and all orifice flow data points were not plotted as their differences were so small.

Effect of Flow Past the Plate for Circular Weir Flows

The circular weir flow data indicated a decrease in discharge as flow by the plate was increased. This trend was more pronounced at low flows. The results shown in Figure 8 suggest a further study of this trend would be desirable.

The largest decrease in discharge occurred at an inflow equal to the maximum flow of 869 gpm. An additional experiment was conducted using a 6-inch diameter plate for this inflow rate. The head and discharge data appear in Appendix A and a graphic presentation of this data is shown in Figure 9. The results of this experiment indicated the decrease in discharge due to flow past the plate is a factor of head above the invert. Flows by the plate did not affect the discharge at higher circular weir flows. Only random variations in discharge were detected for head above the invert values greater than about 0.1 foot.

Effect of Vortices for Orifice Flows

Vortices formed at all orifice flows with flow past the plate. The formation and strength of a vortex is dependent on the tangential component of velocity. Greater flows past the orifice increased the

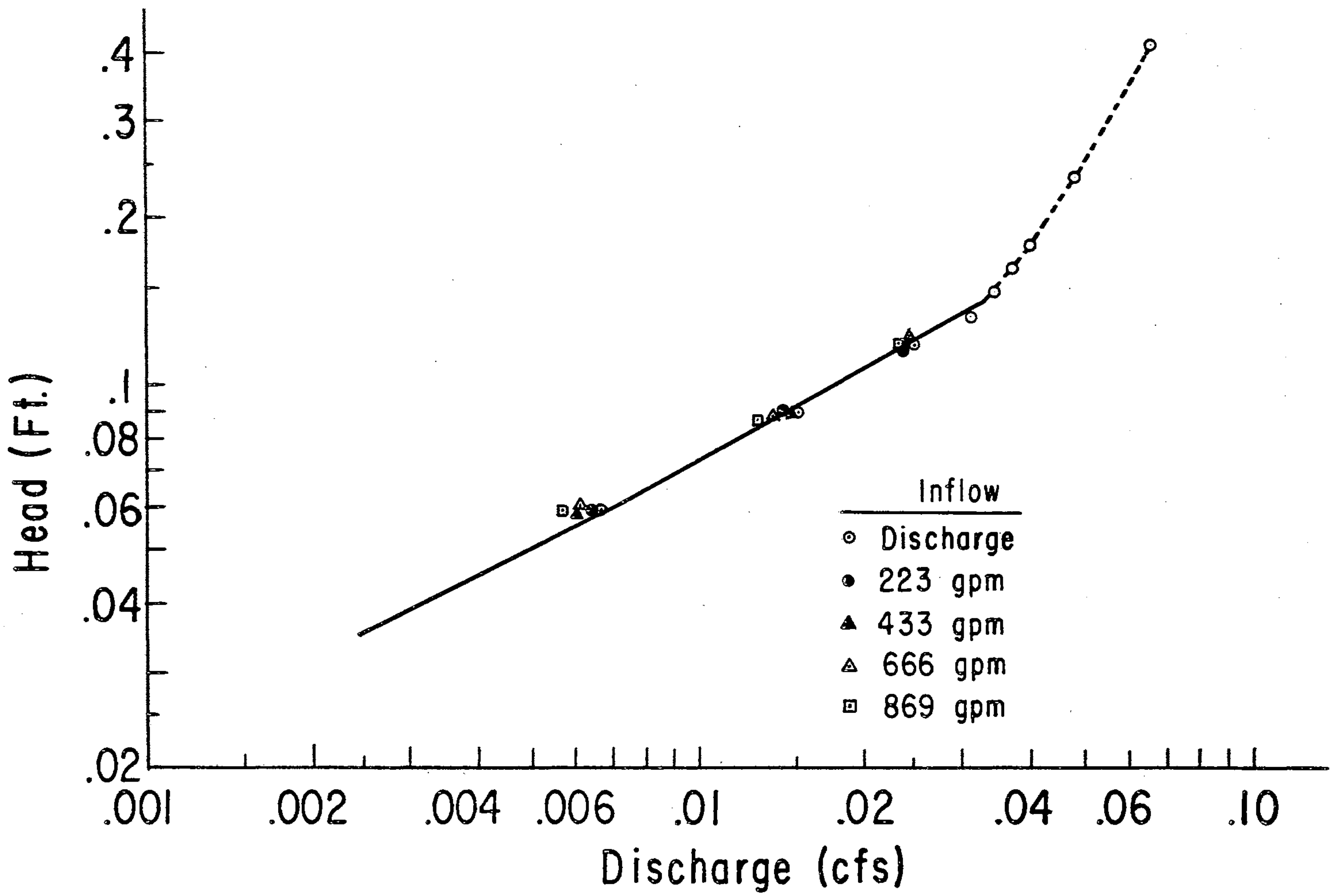

Figure 8. Head Versus Discharge for Four Flows Past the 2-Inch Diameter Plate is Compared with Calculated Curve.

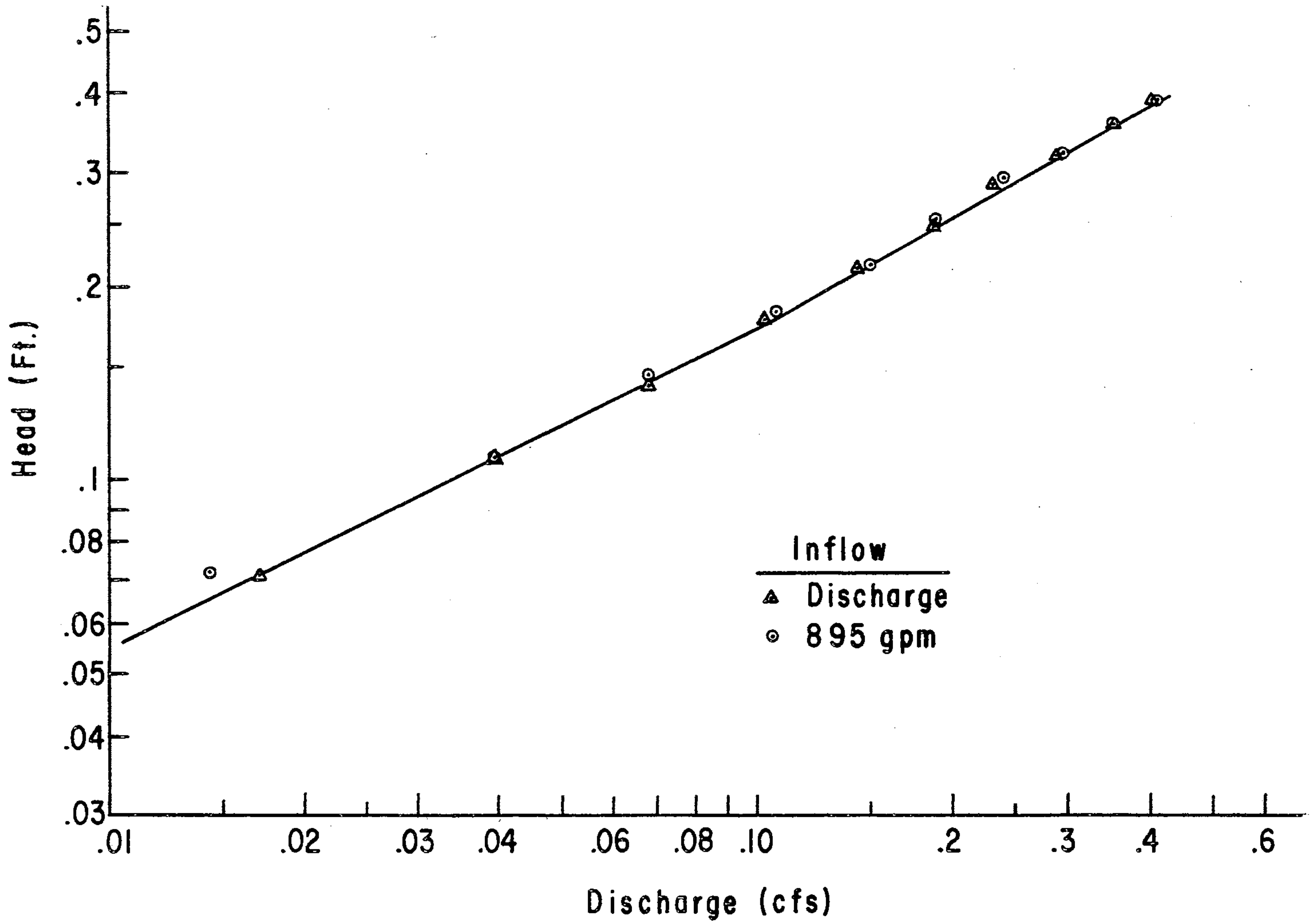

Figure 9. Comparison of Two Inflow Rates for a 6-Inch Diameter Plate.

tangential component of velocity. The vortices increased in size and stability as the flows past the orifice were increased. Figures 10, 11, 12, and 13 demonstrate this phenomenon.

The vortices were eliminated by use of a plywood float above the orifice. Head and discharge measurements were taken for this condition and compared with the measurements taken when vortices were present. The maximum decrease in discharge because of vortex action was 3.4 percent.

Procedure

Head above the invert and discharge measurements were collected for orifice and circular weir plate diameters of 0.0833, 0.1249, 0.1660, 0.2507, 0.3350, 0.4993 and 0.6660 feet. The invert of each plate was installed approximately 16 inches above the bottom of the channel since no difference in plate location was detected. Flow entering the channel was the same as the discharge through the plate.

From 10 to 16 measurements were collected for each diameter. The readings were more closely spaced in the transition zone between weir and orifice flow. The transition zone included H/D values from 0.707 to 1.060. This transition zone would represent approximately the same range as H/D values from 1.0 to 1.5 for a vertical plate. When a circular weir is installed at a 45 degree slope, the water profile flowing through the weir has the approximate shape of a partial ellipse, except that the contractions do not occur in the same vertical plane. The horizontal or major axis is 1.414 times the vertical or minor axis.

The procedure for taking head and discharge measurements was as follows:

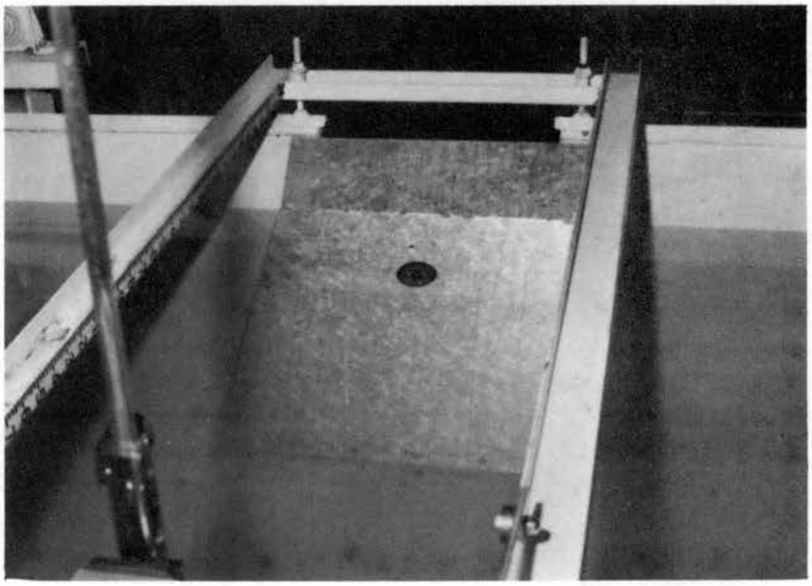

Figure 10. A 2-Inch Diameter Orifice Plate with No Flow Past the Plate.

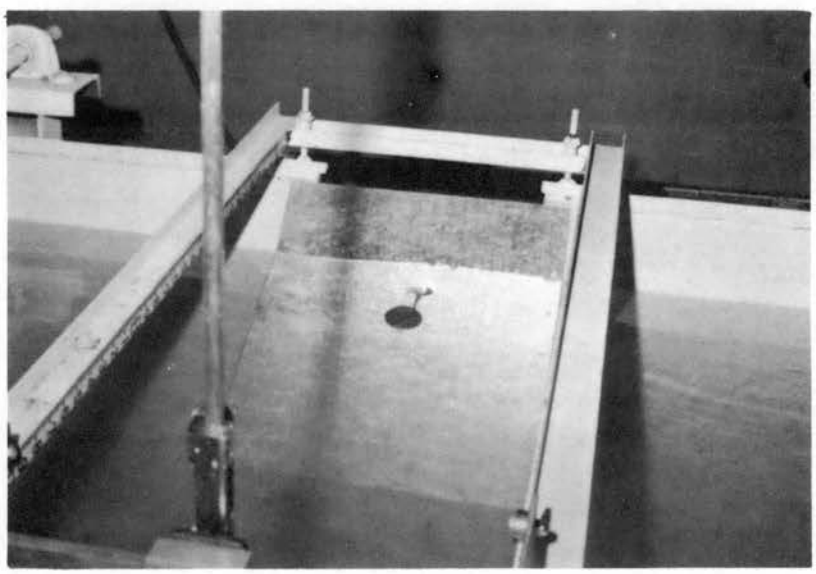

Figure 11. A 2-Inch Diameter Orifice Plate with 0.5 cfs Flow Past the Plate.

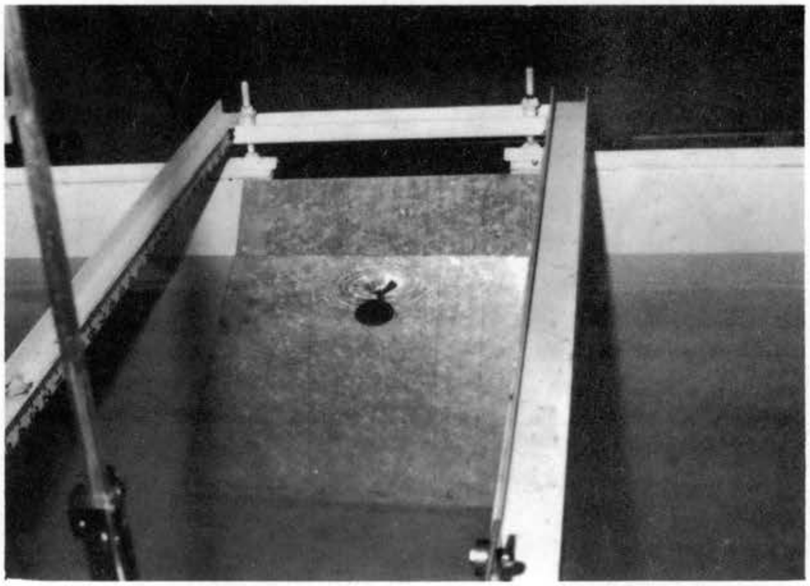

Figure 12. A 2-Inch Diameter Orifice Plate with 1.0 cfs Flow Past the Plate.

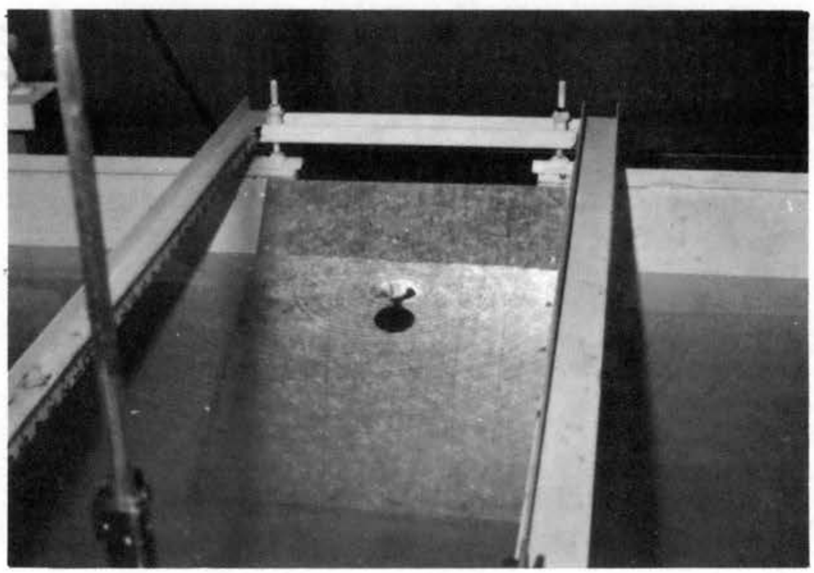

Figure 13. A 2-Inch Diameter Orifice Plate with 2.0 cfs Flow Past the Plate.

1. The plate to be tested was installed and checked for leakage.
2. The water temperature was measured with a mercury-filled, glass thermometer.
3. The elevation of the invert was read with the point gage on the traversing carriage.
4. A correction for the deflection of the carriage tracks was made. This gave a point gage reading at the center line of the channel equal to the elevation of the invert.
5. The desired heads and the corresponding point gage readings were calculated.
6. Inflow was admitted and adjusted to obtain a constant headwater elevation near the calculated one.
7. After the headwater elevation was constant for at least one minute, the discharge was measured.
8. The channel water surface point gage was read again; and, if the elevation had changed more than 0.001 foot, the discharge measurement was discarded and the run repeated.

Steps 6, 7 and 8 were repeated until measurements for all desired heads were made and recorded. Water temperature and invert elevation measurements were repeated prior to removal of the plate.

The recorded data were used to calculate the head above the invert.

The 7040 digital computer was used to find the best fit equations of the form described by Greve (8) using multivariable functional relationships with logarithmic transforms discussed by Natrella (11).

CHAPTER V

RESULTS AND DISCUSSION

The data collected were recorded, and values of head above the invert (H) and discharge (Q) were calculated and printed by a 7040 digital computer. The diameter, head and discharge data appear in Appendix A.

Equations equating discharge as a function of diameter and head were calculated. A graphical presentation of the calculated curves and the data points is shown in Figure 14.

Analysis of Data

The head versus discharge data were plotted on log-log paper. Data points for heads greater than 0.035 foot and less than 0.35D (water surface elevation at the centerline of the orifice plate) formed straight parallel lines. The graphs also indicated an additional slope change in the transition zone between weir and orifice flow. This indicated three equations would be needed to describe the hydraulic properties for headwater elevations above 0.035 foot.

Effect of Surface Tension

Head values below 0.035 foot were not used in the analysis of data as they were affected by surface tension. Discharge began at head values of 0.015 to 0.020 foot. The extremely low flows at heads below 0.035

foot were erratic and less than expected. These flows are not thought to be of value in the design of an irrigation channel.

Transition Zone Between Circular Weir and Orifice Flow

The transition zone for vertically positioned circular weirs and orifices is between head values of D and 1.5D. The values become 0.71D and 1.06D when the plate is installed at a 45 degree slope.

The data for headwater elevations from 0.35D to 0.71D were analyzed to find the best fit equation equating discharge as a function of diameter and head above the invert. An equation of the data above 1.06D was determined by the same method except the head was measured from the center of the orifice plate. The two equations were extended to include values in the transition zone for each of the seven diameters. The equation which best fit the data was extended until the equations intersected. A straight line equation for the seven intersection points was found to be $H/D = (0.89 - 0.23D)$. This equation describes the intersection points, but should not be extended to other diameters or other plate slopes.

Analysis Using Greve's General Formula

The 7040 digital computer was used to find the best fit equations of the form, $Q = K(D)^L(H)^N$, described by Greve (8). Multivariable functional relationships with logarithmic transforms discussed by Natrella (11) were used to calculate the equations.

The following equations equating discharge as a function of diameter and head above the invert were calculated for the headwater elevation ranges listed:

Equation	Range	
$Q = 4.542\ D^{0.549}\ H^{1.953}$	0.035 ft < H < 0.35D	(3)
$Q = 3.710\ D^{0.662}\ H^{1.797}$	0.35 < H/D < (0.89 - 0.23D)	(4)
$Q = 3.450\ D^{1.947}\ (H - 0.35D)^{0.463}$	H/D > (0.89 - 0.23D)	(5)

The results obtained using the above equations together with the observed data points are presented in Figure 14.

The standard deviations of Equations 3, 4 and 5 were 0.002, 0.005, and 0.004 cfs, respectively. The maximum deviation of calculated from observed values of discharge was 5.28 percent, which is not thought excessive for design purposes. The range of flows tested were from 0.002 cfs for a head of 0.035 foot and a plate diameter of 0.0833 foot to 0.546 cfs for a head of 0.40 foot and a plate diameter of 0.666 foot. The equations derived in this study may not be applicable for heads greater than 0.40 foot or diameters greater than 0.666 foot or less than 0.0833 foot.

Analysis by Dimensionless Parameters

Some previous hydraulic researchers have used the relationship between two dimensionless groupings (H/D and Q^2/gD^5) to equate Q as a function of H and D. Values of H/D and Q^2/gD^5 were calculated by the digital computer from observed head, diameter and discharge data. These values were plotted on log-log paper with results similar to the head versus discharge graph.

The dimensionless equation used was

$$Q^2/gD^5 = C_1\ (H/D)^{C_2} \tag{6}$$

where

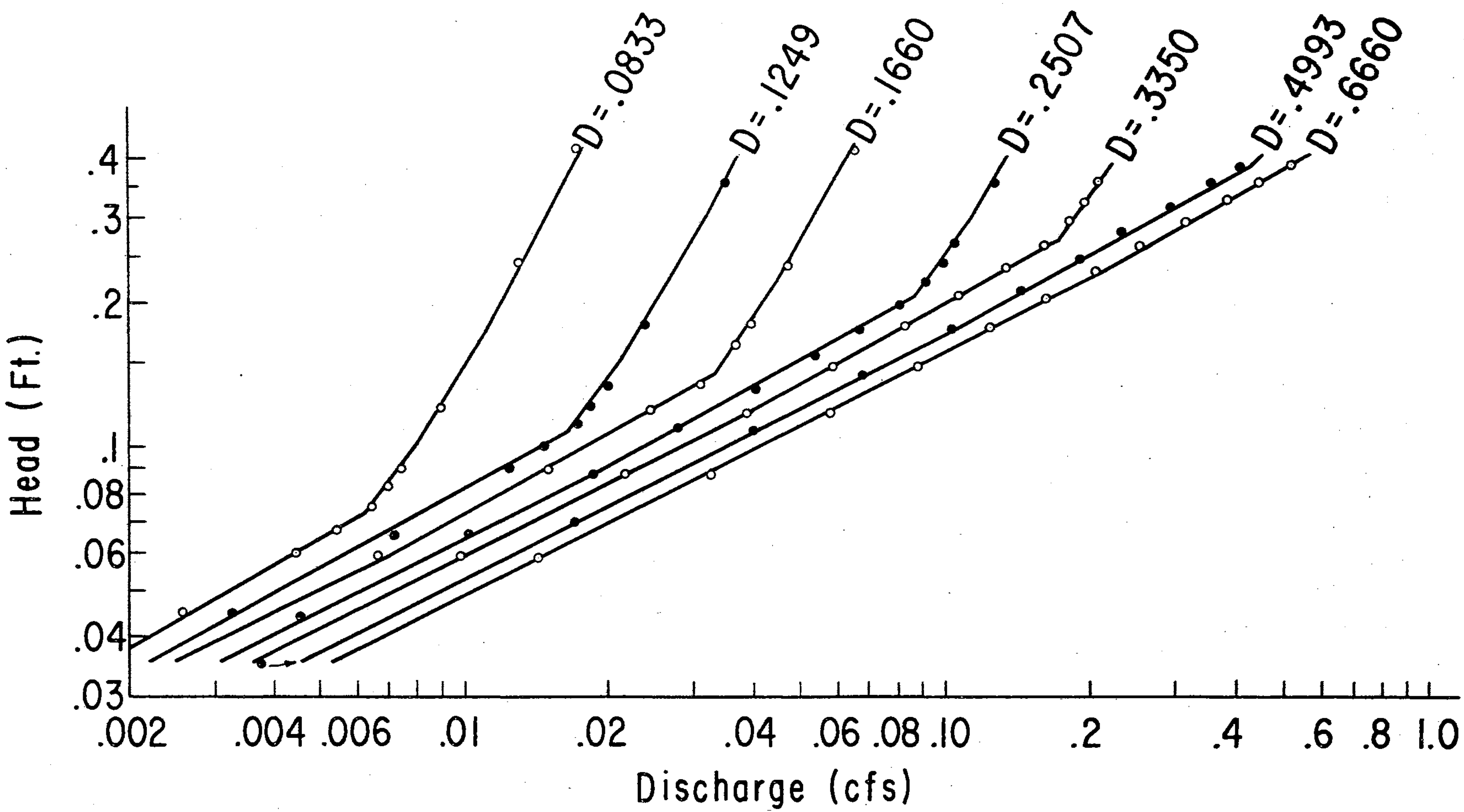

Figure 14. Calculated Curves with Observed Data Points.

C_1 = coefficient, dimensionless

C_2 = coefficient, exponent

Solving for Q gives the equation

$$Q = \sqrt{gC_1}\; D^{(5/2 - C_2/2)}\; H^{(C_2/2)} \qquad (7)$$

Analysis by this method results in the sum of the exponents of D and H being 5/2 for any value of C_2.

Values of C_1 and C_2 in Equation 6 were determined by using the multivariable functional relationship method for the ranged listed for Equations 3, 4 and 5. Equation 7 was used to equate discharge as a function of diameter and head. A value of 32.15 ft/sec^2 was used for g. The following equations were calculated for the headwater elevation ranges listed:

Equation	Range	
$Q = 4.529\ D^{0.548}\ H^{1.952}$	$0.035\ \text{ft} < H < 0.35D$	(8)
$Q = 3.943\ D^{0.683}\ H^{1.817}$	$0.35 < H/D < (0.89 - 0.23D)$	(9)
$Q = 4.031\ D^{1.998}\ (H - 0.35D)^{0.502}$	$H/D > (0.89 - 0.23D)$	(10)

The standard deviations of Equations 8, 9 and 10 were 0.002, 0.011, and 0.014 cfs, respectively. The maximum deviation of calculated from observed values of discharge was 11.2 percent. Equations 4 and 5 had smaller standard deviations than Equations 9 and 10. This indicates the equations calculated using the form of Greve's (8) general formula fit the observed data better than the equations calculated by the dimensionless parameters for headwater elevations above the centerline of the plate.

Comparison of Circular Weir Flow Equations with Those Calculated from Greve's Data

As stated in Chapter II Greve (8) conducted vertical circular weir tests with three diameters in the range used in this study. The three comparable diameters were 0.250, 0.500 and 0.666 foot. All of his data for the 0.250 foot diameter were collected above the centerline of the weir plate. The available data were analyzed by the multivariable functional relationship method. Equations 1 and 2 were evaluated and curves for head above the invert versus discharge for 0.250, 0.500 and 0.666 foot diameters were plotted. Values obtained from Equations 3 and 4 in this study were used to plot curves for head above the invert versus discharge for the same diameters on the same graph.

Discharge for a given head and diameter for the two studies are compared in Figure 15. The curve in Figure 15 has a slope of 1 vertical to 0.707 horizontal which is equal to the reciprocal of the sine of the slope of the plate. The relationship between the plotted data and the line indicates that slope of the weir plate may not change the coefficient of discharge to the extent that one might expect. An experiment with variable weir slopes and diameters should be conducted to evaluate the influence of these factors on the coefficient of discharge. If the data exactly fitted the line, this would indicate no change in coefficient of discharge between the studies.

Comparison of Calculated Orifice Coefficients of Discharge with a Graph from King

The Reynolds number is a dimensionless parameter which is often used for correlating coefficients. King (10) derived such a graph for

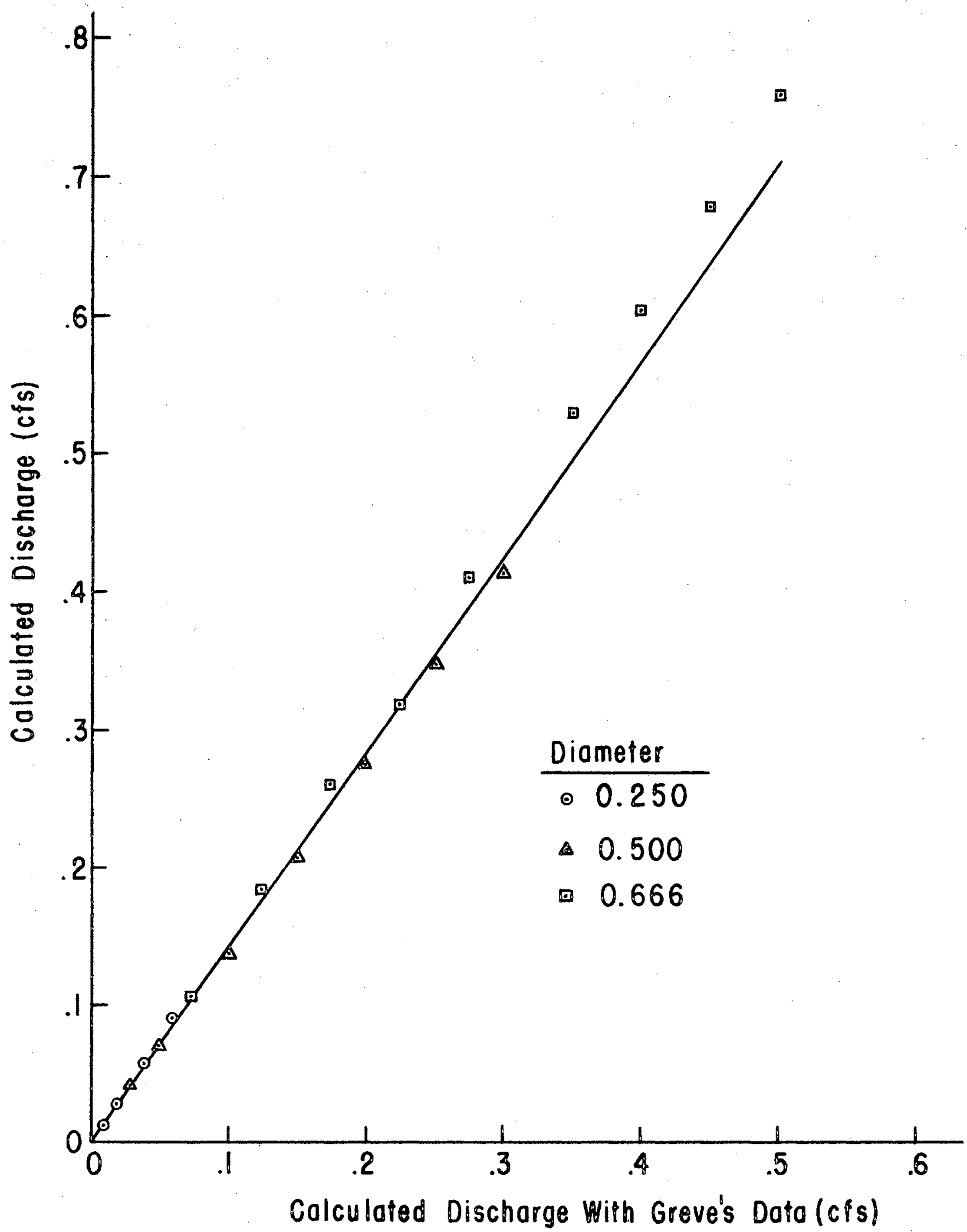

Figure 15. Calculated Discharge Versus Discharge Calculated from Greve's Data with a Curve with 1.414 Slope.

orifice coefficients from previous research data. A segment of this graph is reproduced in Figure 16.

Values of the coefficient of discharge were calculated for all observed orifice flow data points from the formula $Q = CA\sqrt{2gh}$. The Reynolds number was calculated for each of the data points. The calculated values of coefficient of discharge and Reynolds number are plotted in Figure 16.

The agreement of the values of discharge coefficient from this study and King's graph indicate that, as expected, the slope of the orifice has no influence on discharge in the orifice flow range.

Application of Results

Effect of Field Slope Parallel to the Channel on Design

The slope of the field parallel to the irrigation channel is important in the design of an automatic cut-back irrigation system. An example design is presented to illustrate the slopes which may be accommodated by plate diameter selection. Examples are presented in Table I for six plate diameters. The table consists of plate diameter, head for initial and cut-back flows, the drop between bays, and channel slope. An initial flow of 0.056 cfs, a cut-back flow of 0.018 cfs, a water supply of 2.0 cfs, and a furrow spacing of 3.33 feet was used to demonstrate the variations in channel slope.

Thirty-six orifice plates are needed to discharge the 2.0 cfs supply at an initial flow of 0.056 cfs in the first bay. The channel length of the first bay is 120 feet at the furrow spacing of 3.33 feet. As approximately one-fourth of the flow will be used for cut-back flow, the average bay length is 90 feet.

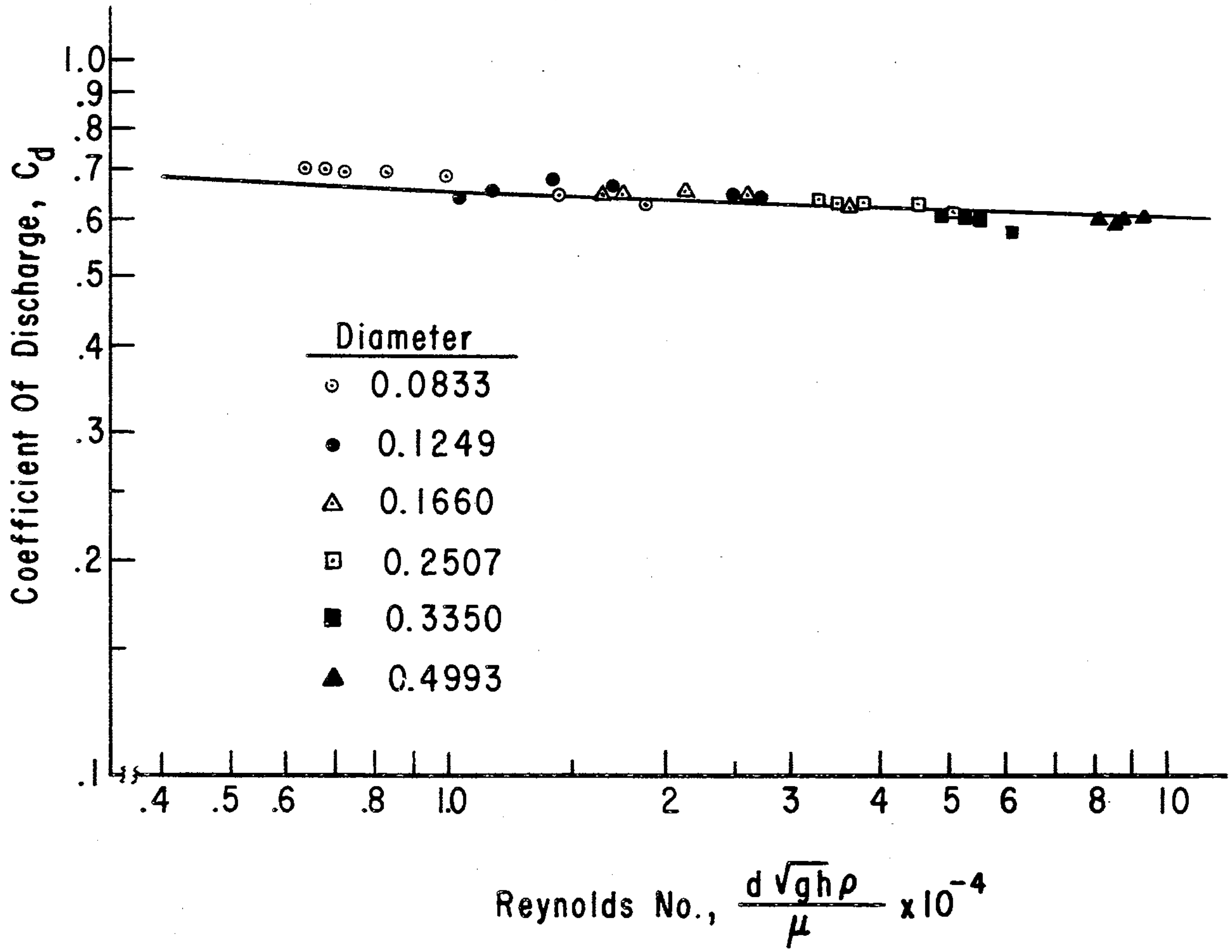

Figure 16. Comparison of Orifice Coefficients with Those Published by King (10).

TABLE I

DIAMETER, HEAD AND SLOPE RELATIONSHIPS FOR DESIGN OF A CUT-BACK SYSTEM

Plate Diameter Ft	Head for Initial Flow Ft	Head for Cut-Back Flow Ft	Drop Between Bays Ft	Channel Slope Percent
0.1249	0.903 (5)*	0.122 (5)	0.781	0.868
0.1660	0.318 (5)	0.100 (4)	0.218	0.243
0.2507	0.161 (4)	0.093 (4)	0.068	0.076
0.3350	0.145 (4)	0.080 (3)	0.065	0.072
0.4993	0.128 (3)	0.072 (3)	0.056	0.063
0.6660	0.122 (3)	0.066 (3)	0.056	0.063

(5)* Equation used for calculation of discharge.

A small plate diameter which uses the orifice flow equation for both initial and cut-back flows will require a steep channel to obtain a 3 to 1 cut-back rate. A plate diameter which uses the orifice flow equation for the initial flow and the circular weir flow equations for the cut-back flows will function with a moderate slope. Small drops between bays may be obtained by using the circular weir equations for both initial and cut-back flows. The head for the cut-back flow should not be much greater than the drop between bays. This is necessary to prevent excessive furrow discharge in the bay upstream from the bay flowing at the cut-back rate. A head of approximately 0.015 foot is necessary for flow to begin.

Weir plates could also be used on automated systems which do not use cut-back flow. These could have more application for border irrigation.

Four Bay System Constructed Using Sheet Metal Plates as Furrow Metering Devices

The design and construction of a four bay automatic cut-back irrigation system was recently completed at the Irrigation Research Station at Altus, Oklahoma. The system was designed using the procedure described by Garton (7). The hydraulic properties obtained in this study were used to determine the discharge, head and diameter relationships for the design.

The slip-form concrete lined irrigation channel was constructed at the desired bay elevations. Holes approximately four inches in diameter were cut in the uncured concrete at the desired furrow spacing and elevation. Later, the cured concrete was etched with acid to obtain better adhesion with the sheet metal plate. Figure 17 shows the treated concrete before the 8-inch by 8-inch plate was installed. Figure 18 shows a 2-inch diameter plate in place. A concrete nail was necessary to hold the plate in place until the adhesive became effective.

The drop between bays is shown in Figure 19 and a view of the entire system is shown in Figure 20.

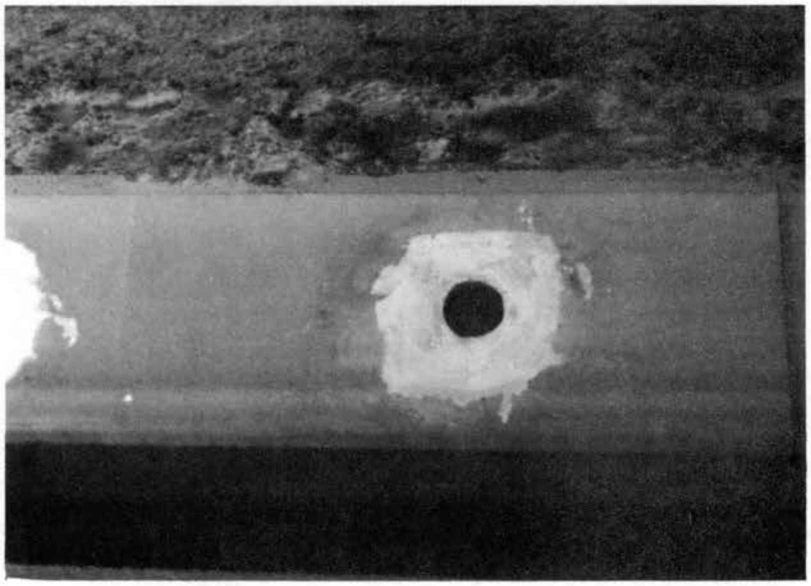

Figure 17. Treated Area of Concrete Lined Irrigation Channel Before Plate is Installed.

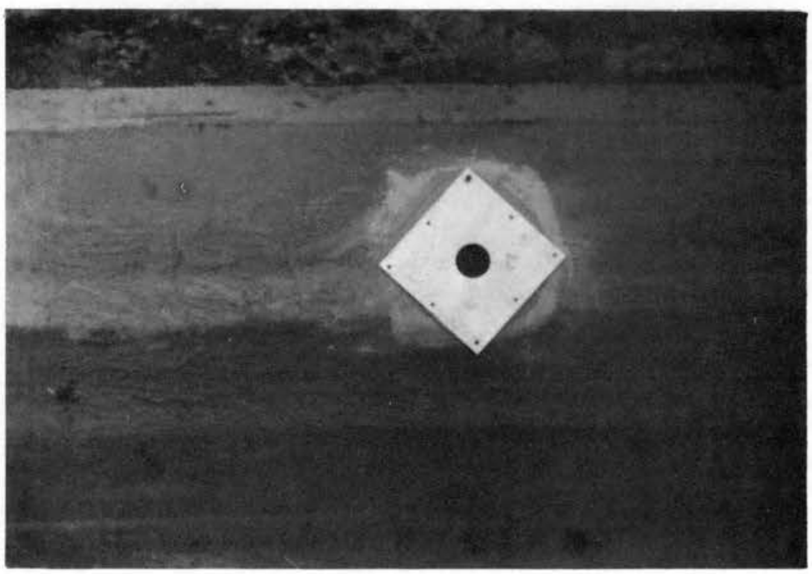

Figure 18. A Plate Installed with Adhesive and Concrete Nail.

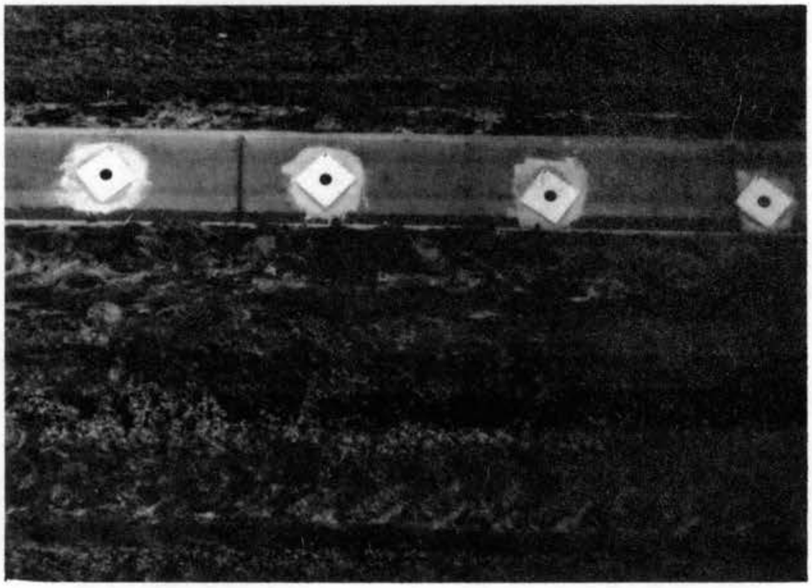

Figure 19. The Drop Between Bays is Demonstrated Between the Center Plates.

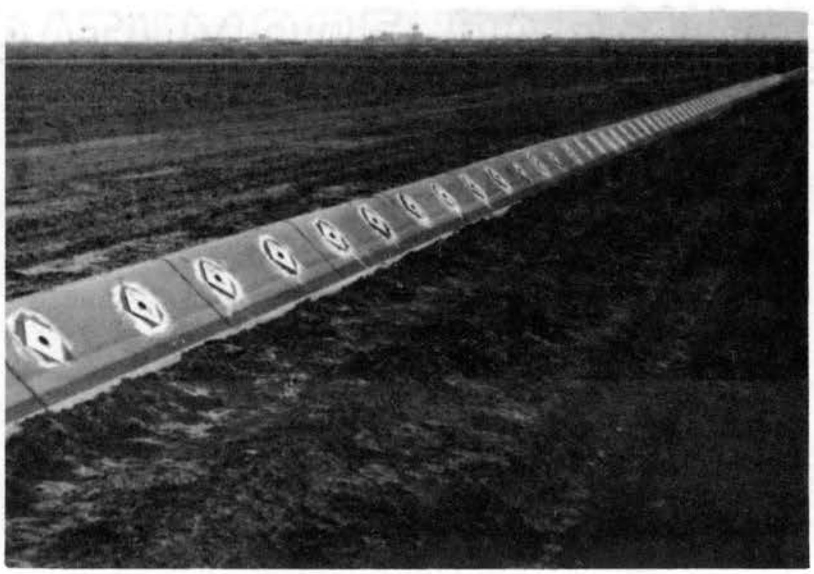

Figure 20. A Downstream View of the Four Bay System.

CHAPTER VI

SUMMARY AND CONCLUSIONS

Summary

The hydraulic properties of sheet metal orifice and circular weir plates installed at a 45 degree slope were studied. The following equations describe the head versus discharge relationships for seven plate diameters from 1 inch to 8 inches with heads (measured from the invert) ranging from 0.035 foot to 0.40 foot. The equations apply to the head ranges listed opposite the equation.

Equation	Range	
$Q = 4.542\ D^{0.549}\ H^{1.953}$	0.35 ft $< H < 0.35D$	(3)
$Q = 3.710\ D^{0.662}\ H^{1.797}$	$0.35 < H/D < (0.89 - 0.23D)$	(4)
$Q = 3.450\ D^{1.947}\ (H - 0.35D)^{0.463}$	$H/D > (0.89 - 0.23D)$	(5)

The maximum deviation of calculated from observed values of discharge was 5.28 percent. The standard deviations of Equations 3, 4 and 5 were 0.002, 0.005 and 0.004 cfs, respectively.

A graphical presentation of the calculated curves and the observed data is shown in Figure 14.

The equations were calculated from data taken with inflow equal to discharge through the plate. A preliminary investigation indicated a reduction in discharge with flow past the plate for circular weir flows below about 0.1 foot head. No reduction in discharge was observed for orifice flows with flow past the plate; however, vortices formed and caused a slight flow reduction.

Calculated values of discharge from Greve's (8) vertical circular weir test data for three equivalent diameters were compared with calculated values of discharge from this study. The comparison indicated a good correlation between the discharge determined in this study multiplied by the sine of the slope angle and Greve's discharge. This was not expected as the contractions do not occur in the same vertical plane.

A comparison of orifice coefficients of discharge calculated from the study with those published by King (10) was made. The comparison suggests the general orifice flow formula can be used for a sloping orifice plate if the proper coefficient of discharge is used.

Conclusions

The following conclusions are presented from the results of the study:

1. Sheet metal orifice and circular weir plates are feasible to use as a furrow metering device for concrete lined irrigation channels.
2. The hydraulic properties of orifice and circular weir plates installed at a 45 degree slope and acting as side weirs can be described by Equations 3, 4 and 5 for a range of discharge from 0.002 cfs to 0.546 cfs.

3. Easier installation and less cost are the primary advantages of the plates as compared to short, level, canopy-inlet tubes.
4. The design for a large range of field slopes parallel to the irrigation channel can be accomplished with a proper plate diameter selection.

Suggestions for Future Research

1. A study of the hydraulic properties of various shaped sheet metal orifice or weir plates and their application as a furrow metering device could be desirable for some designs.
2. The effects of decreasing spatially varied flow on the flow through side weir plates at low heads is needed.
3. A study of the design and feasibility of a sheet metal automatic cut-back irrigation channel might prove fruitful. The channel should be readily fabricated, portable and easily transported.
4. The effects of slope on discharge from a circular side weir should be investigated.
5. A procedure for installation of the sheet metal plates on a concrete lined channel is needed. Labor saving and economics would be the prime objective.
6. A study of the erosion caused by the jet from circular weir and orifice plates for several soil textures and discharges is needed.

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APPENDIX A

DIAMETER, HEAD AND DISCHARGE EXPERIMENTAL DATA

TABLE A-1

HEAD AND DISCHARGE MEASUREMENTS FOR THREE PLATE HEIGHTS

Vertical Distance From Invert To Ditch Bottom feet	Plate Diameter feet	Head Above Invert feet	Discharge cfs
0.902	0.1667	0.021	0.001
0.902	0.1667	0.045	0.004
0.902	0.1667	0.068	0.009
0.902	0.1667	0.092	0.016
0.902	0.1667	0.117	0.024
0.902	0.1667	0.141	0.032
0.902	0.1667	0.166	0.037
0.902	0.1667	0.219	0.046
0.902	0.1667	0.490	0.072
0.902	0.1667	0.845	0.097
1.335	0.1660	0.029	0.002
1.335	0.1660	0.059	0.007
1.335	0.1660	0.089	0.015
1.335	0.1660	0.118	0.024
1.335	0.1660	0.134	0.031
1.335	0.1660	0.147	0.034
1.335	0.1660	0.162	0.037
1.335	0.1660	0.179	0.040
1.335	0.1660	0.237	0.048
1.335	0.1660	0.410	0.065
1.571	0.1667	0.021	0.001
1.571	0.1667	0.042	0.003
1.571	0.1667	0.063	0.008
1.571	0.1667	0.087	0.015
1.571	0.1667	0.109	0.022
1.571	0.1667	0.136	0.032
1.571	0.1667	0.154	0.036
1.571	0.1667	0.182	0.041
1.571	0.1667	0.202	0.043

TABLE A-2

HEAD AND DISCHARGE MEASUREMENTS FOR A 2-INCH DIAMETER PLATE WITH FOUR INFLOW RATES

223 gpm Inflow		433 gpm Inflow	
Head ft	Discharge cfs	Head ft	Discharge cfs
0.027	0.001	0.029	0.001
0.058	0.006	0.058	0.006
0.087	0.014	0.088	0.015
0.116	0.023	0.118	0.024
0.147	0.034	0.148	0.033
0.180	0.040	0.180	0.040
0.237	0.048	0.235	0.048
0.408	0.065	0.410	0.065

666 gpm Inflow		869 gpm Inflow	
Head ft	Discharge cfs	Head ft	Discharge cfs
0.026	0.001	0.027	0.001
0.059	0.006	0.058	0.006
0.087	0.013	0.085	0.013
0.120	0.024	0.117	0.023
0.149	0.034	0.148	0.034
0.177	0.039	0.180	0.040
0.236	0.049	0.239	0.048
0.408	0.067	0.409	0.067

TABLE A-3

HEAD AND DISCHARGE MEASUREMENTS FOR A 6-INCH DIAMETER PLATE WITH 869 GPM INFLOW

Head ft	Discharge cfs
0.035	0.003
0.071	0.015
0.108	0.040
0.145	0.069
0.181	0.107
0.216	0.149
0.253	0.189
0.287	0.241
0.320	0.298
0.357	0.353
0.389	0.413
0.421	0.460

TABLE A-4

HEAD AND DISCHARGE MEASUREMENTS FOR SEVEN PLATE DIAMETERS

Diameter feet	Head feet	Discharge cfs	Diameter feet	Head feet	Discharge cfs
0.0833	0.030	0.001	0.1660	0.029	0.002
0.0833	0.045	0.003	0.1660	0.059	0.007
0.0833	0.060	0.004	0.1660	0.089	0.015
0.0833	0.067	0.006	0.1660	0.118	0.024
0.0833	0.075	0.006	0.1660	0.134	0.031
0.0833	0.082	0.007	0.1660	0.147	0.034
0.0833	0.090	0.007	0.1660	0.162	0.037
0.0833	0.120	0.009	0.1660	0.179	0.040
0.0833	0.240	0.013	0.1660	0.237	0.048
0.0833	0.415	0.017	0.1660	0.410	0.065
0.1249	0.020	0.001	0.2507	0.021	0.001
0.1249	0.043	0.003	0.2507	0.044	0.005
0.1249	0.065	0.007	0.2507	0.066	0.010
0.1249	0.088	0.012	0.2507	0.088	0.019
0.1249	0.099	0.015	0.2507	0.110	0.029
0.1249	0.111	0.017	0.2507	0.132	0.041
0.1249	0.121	0.018	0.2507	0.155	0.054
0.1249	0.133	0.020	0.2507	0.176	0.067
0.1249	0.176	0.024	0.2507	0.198	0.081
0.1249	0.352	0.035	0.2507	0.220	0.092
0.1249	0.409	0.038	0.2507	0.242	0.099
			0.2507	0.265	0.106
			0.2507	0.352	0.127
			0.2507	0.425	0.141

TABLE A-4 (Continued)

Diameter feet	Head feet	Discharge cfs	Diameter feet	Head feet	Discharge cfs
0.3350	0.029	0.002	0.6660	0.028	0.003
0.3350	0.059	0.010	0.6660	0.059	0.014
0.3350	0.088	0.022	0.6660	0.088	0.033
0.3350	0.118	0.039	0.6660	0.118	0.058
0.3350	0.147	0.059	0.6660	0.147	0.088
0.3350	0.177	0.083	0.6660	0.177	0.125
0.3350	0.206	0.107	0.6660	0.205	0.162
0.3350	0.236	0.134	0.6660	0.234	0.205
0.3350	0.265	0.160	0.6660	0.264	0.253
0.3350	0.295	0.181	0.6660	0.294	0.315
0.3350	0.324	0.195	0.6660	0.325	0.381
0.3350	0.354	0.206	0.6660	0.354	0.446
0.3350	0.433	0.225	0.6660	0.385	0.516
			0.6660	0.415	0.590
			0.6660	0.443	0.657
			0.6660	0.472	0.726
0.4993	0.035	0.004			
0.4993	0.071	0.017			
0.4993	0.108	0.040			
0.4993	0.141	0.068			
0.4993	0.177	0.104			
0.4993	0.212	0.144			
0.4993	0.248	0.190			
0.4993	0.283	0.233			
0.4993	0.317	0.293			
0.4993	0.353	0.353			
0.4993	0.387	0.409			
0.4993	0.424	0.464			
0.4993	0.460	0.501			
0.4993	0.495	0.535			
0.4993	0.532	0.564			

VITA

Armond Dudley Barefoot

Candidate for the Degree of

Master of Science

Thesis: THE HYDRAULIC PROPERTIES OF ORIFICES AND CIRCULAR WEIRS WITH A 45 DEGREE SLOPE

Major Field: Agricultural Engineering

Biographical:

Personal Data: Born at Hollis, Oklahoma, December 7, 1924, the son of Thomas C. and Bessie L. Barefoot.

Education: Graduated from Hollis High School, Hollis, Oklahoma, in 1943. Attended Kansas State Teachers College of Emporia, Kansas, in 1944; received the Bachelor of Science degree in Agricultural Engineering from Oklahoma State University in August, 1953. Completed the requirements for the Master of Science degree in May, 1968.

Professional Experience: Superintendent, Irrigation Research Station at Altus, Oklahoma, Oklahoma State University, 1953-64; Instructor, Agricultural Engineering Department, Oklahoma State University, 1964-68.

Professional Organizations: Associate member of the American Society of Agricultural Engineers; Registered Professional Engineer, State of Oklahoma.