A FIELD STUDY OF AN AUTOMATED IRRIGATION FLUME

By

JOHN HENRY EDWARDS

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Arkansas State University

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Thesis Approved:

James E. Garton
Thesis Adviser

A. D. Barefoot

D. D. Durham
Dean of the Graduate College

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CHAPTER I

INTRODUCTION

The Problem

For more economical and efficient distribution of irrigation water, the operator must have better control of the water as it flows onto the land. When uncontrolled streams of water are diverted into crop rows, waste, inefficiency, and uneven distribution are almost certain to result (3). With the rising costs of today, the irrigator needs to more profitably utilize his two major resources--water and labor (4). The use of water for irrigation has been relatively inefficient in the past because of poor design, or the improper operation of well designed systems (24).

Water for surface irrigation systems is usually made available at the high point of the field and flows to the low end by gravity (3). Earthen channels, concrete channels, or concrete or plastic pipelines with low pressure gated pipe are employed to distribute the water. Earthen channels have the disadvantages of losing water by seepage and requiring constant maintenance work. They are also a source of weed infestation. Although concrete lined channels retard seepage and reduce maintenance, they are permanent and are not always adaptable to changing cropping systems. They also hinder machine mobility. Gated pipe, while being portable and eliminating evaporation, has the problems of unequal furrow discharge, cost, and limited capacity.

Furrow irrigation usually has a high labor requirement and results in non-uniform water application (13). This non-uniform water application is not only true in the same furrow but also between adjacent furrows. Since water normally recedes faster than it advances, the upstream end of the field has a longer intake-opportunity time. This may cause deep percolation losses at the upstream end (23). Pope (21) measured furrow flows for some rows nearly twice those of others in the same irrigation set. Excessive tail-water runoff resulted from the rows with the larger flows.

Garton (11) designed an automated cut-back system to combat the problems associated with non-uniform furrow flow. His design consisted of a trapezoidal-shaped concrete channel with hooded-inlet tubes, mounted in the side. It was constructed as a series of horizontal bays, staircased down the slope of the field. A near horizontal water surface occurred in each bay. Therefore, very uniform furrow flows resulted.

A similar system to the one above, using circular orifices as furrow outlet devices, was proposed and built by Barefoot and Garton (2). One disadvantage of such a system is that the design depends on a constant inflow (Q). The constant, Q, may be difficult to maintain, especially where deep wells supply the water. The system is also permanent.

Uhl (32) designed an automated semi-portable sheet metal flume that functioned as a cut-back system. The differences in Garton's and Uhl's designs were that the latter was rectangular in shape, used side orifices for furrow discharge, and was portable. Since Uhl's design was portable, it could be set up after planting and moved before harvesting.

This design could also be adaptable to a change in Q and could be relocated to cope with changes in cropping patterns from year to year.

The research reported in this thesis dealt with a field study of an automated, semi-portable sheet metal flume. This study was a continuation of the laboratory study conducted by Uhl (32).

Objectives

The objectives of this study were:

1. To obtain information on rate of advance of various sizes of furrow streams for a farm at Guymon, Oklahoma.

2. To design a system based on the information obtained in (1) together with measurements of water supply available.

3. To determine operating characteristics of the system as designed and installed.

4. To evaluate reliability, stability of system, labor requirements, and farmer acceptance.

CHAPTER II

REVIEW OF LITERATURE

Automated Irrigation

With the continual migration of the farm laborers to the cities, much work has been done toward developing automated irrigation systems. The basic reasons for automating irrigation systems are to save labor and water, which in crop production have become more and more limited (9).

There are two classifications of mechanical irrigation structures, devices, and systems. They are semi-automatic and automatic, depending upon their method of operation. Semi-automatic systems and equipment require manual attention during each irrigation. Automatic structures normally operate without attention from the operator other than for periodic inspections. Automated structures sense the need for irrigation, introduce water to the farm distribution channels, and complete the irrigation without operator intervention (17).

Several automated surface irrigation systems have been devised to reduce labor requirements. However, attention has not been directed toward improving distribution uniformities with some of these systems (4, 9, 11, 13, 17, 32). One exception is a semi-automated cut-back irrigation system conceived by Garton (11, 13).

A cut-back system is one which uses a large initial flow to water the length of the furrow. Before tailwater flow occurs, the flow is reduced or cut-back to the intake rate of the furrow.

Figure 1 presents a drawing of the cut-back system. This system consists of a concrete-lined ditch, constructed as a series of level bays following the slope of the land in steps. Level furrow outlet tubes are set at the same elevation in a given bay. The difference in elevation between bays is equal to the difference in head required at initial and cut-back furrow flows.

The principles of design and operation are as follows: as water is turned into the channel, the water rises in the first bay until it is discharging at initial flow from each tube (11, 12, 13). The number of tubes in the first bay is determined by the total flow to be handled and the flow per tube. When the furrows irrigated by this bay are watered through their length, the check dam located at the end of the first bay automatically releases.

Water now enters the second bay and rises until the tubes are discharging at initial flow. The tubes in the first bay are now at cut-back flow. The head on the cut-back tubes is equal to the initial head minus the amount of drop between bays. The number of tubes needed in bay 2 depends on the supply flow minus the discharge of bay 1 at cut-back flow. After the furrows irrigated by the second bay are watered through, the check dam at the end of this bay automatically releases. Initial flow begins in the third bay. Bay 2 is now flowing at the cut-back flow, and the tubes in bay 1 will not flow, since the water surface will be below the level of the invert. The number of tubes in bay 3 and subsequent bays hinges on the supply flow minus the amount discharged by the preceding bay at the cut-back flow.

According to Garton (12) the following general points must be con-

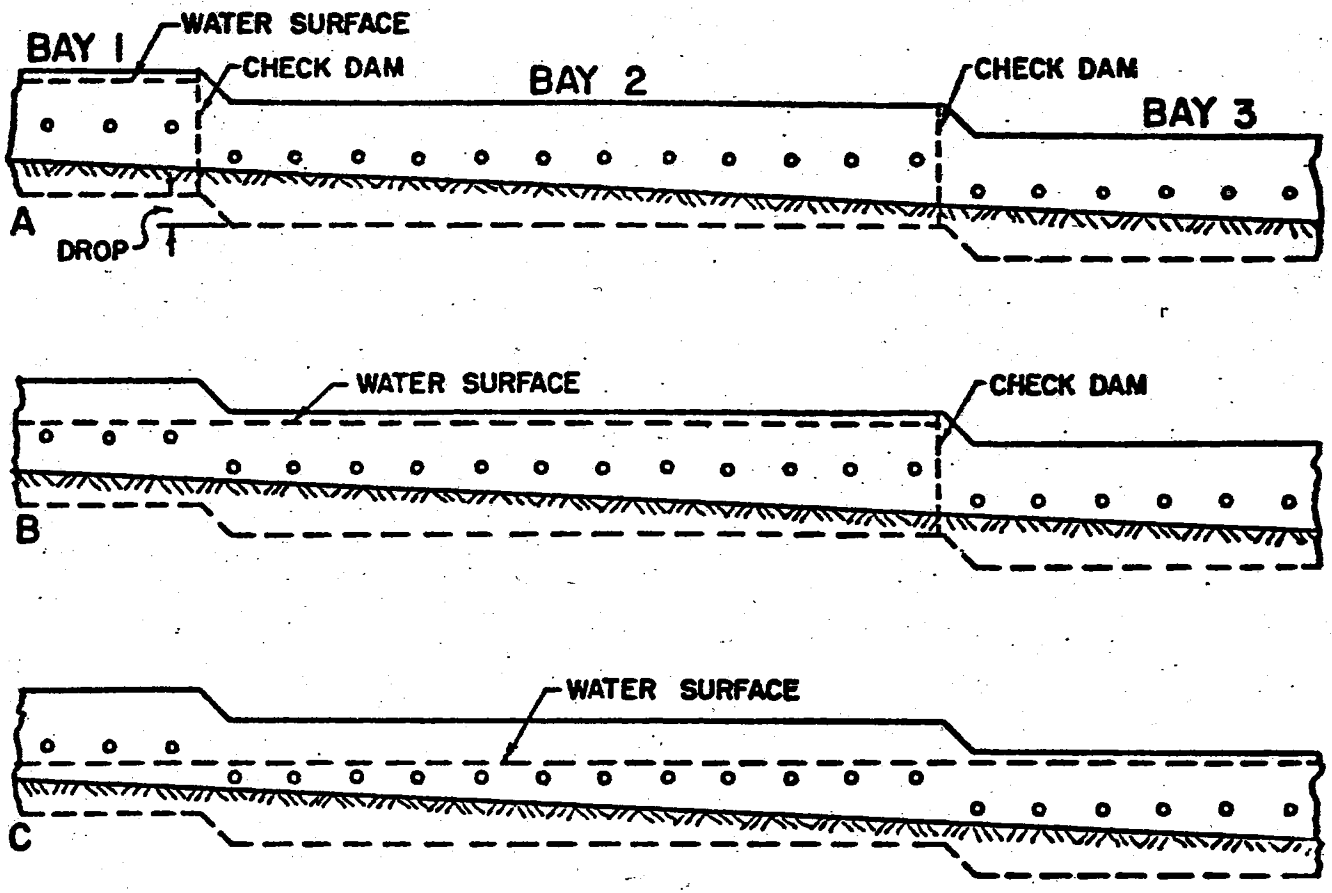

Figure 1. Elevation Drawing of a Cut-Back Furrow Irrigation System Using one Cut-Back

sidered in the design of such a system:

1. Water supply;
2. Length of channel;
3. Slope of the land surface;
4. Furrow stream size;
5. Selection of tube size and head.

Open Channel Flow

Open channel flow is encountered in most surface irrigation systems. It is described as water which flows with a free surface, subject to atmospheric pressure (5).

The concept of total energy to describe open channel flow is shown in Figure 2. Referenced to a datum line, it is the sum of the elevation represented by Z; the piezometric height, Y; the velocity head, $\frac{V^2}{2g}$; and the loss of energy, h_f.

Bernoulli's equation applies the theory of the conservation of energy, where the total energy at station 1 is equal to the total energy at station 2 plus friction losses. The equation is written as:

$$Z_1 + Y_1 + \alpha\frac{V_1^2}{2g} = Z_2 + Y_2 + \alpha\frac{V_2^2}{2g} + h_f \qquad (2\text{-}1)$$

α, the kinetic-energy correction factor, can be taken as unity without serious error (19).

King and Brater (19) and Chow (5) classified types of open channel flow according to the change in flow depth with respect to time and space. Their classifications are as follows:

Time as the standard: Steady flow occurs when the velocity is

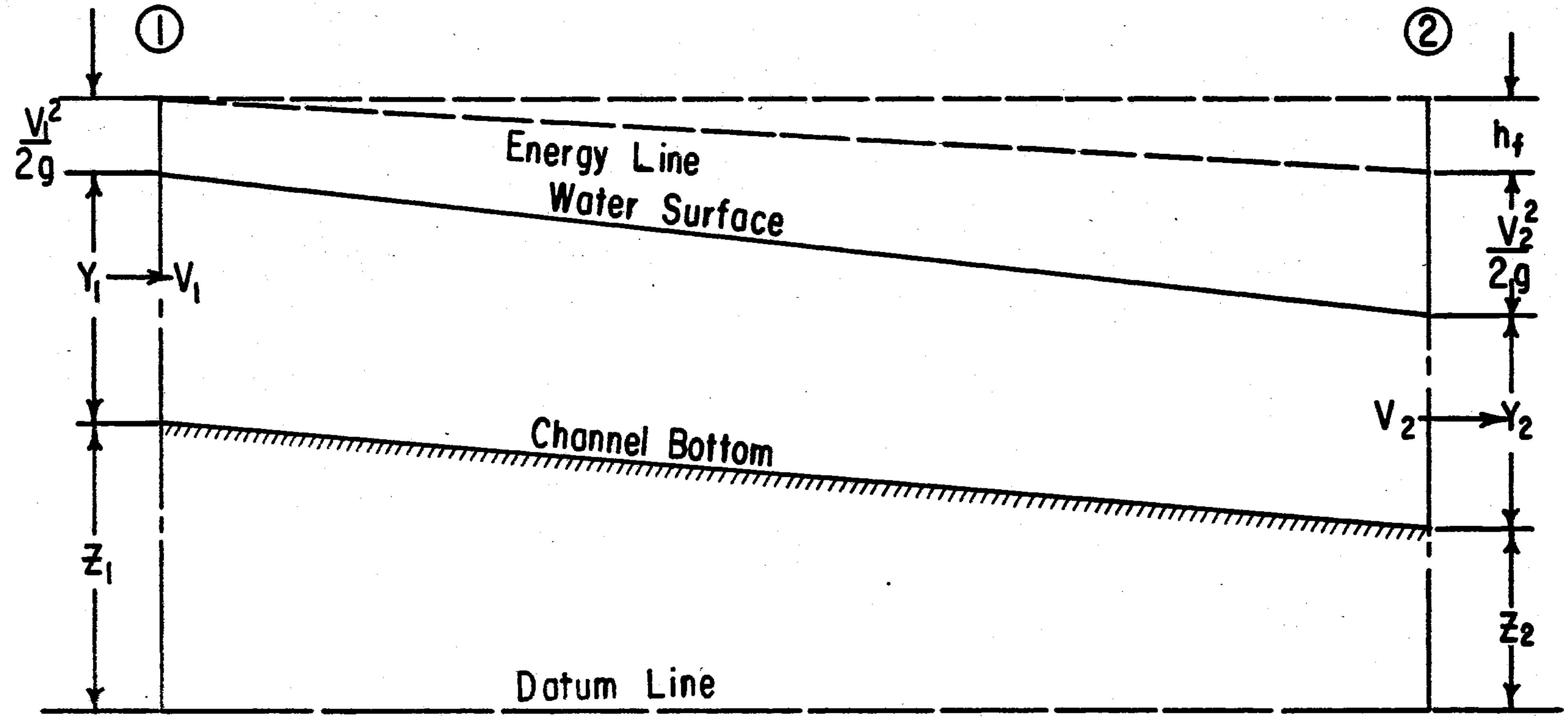

Figure 2. Graphic Description of Total Energy in Open Channel Flow

constant with respect to time. The flow is unsteady if the velocity varies with time.

Space as the criterion: If the depth of flow (velocity) is the same at every point of the channel, it is uniform. The flow is non-uniform (varied) if the depth changes with distance.

Combinations may occur between steady, unsteady, uniform and non-uniform flows.

Hydraulic Characteristics of Furrow Flow

To properly design new irrigation systems or efficiently operate present ones, necessitates determining the rate of advance and intake rate of the soil.

The fluid flow of surface irrigation is a case of unsteady, non-uniform, open channel flow over a porous bed (16, 31). It is practically impossible to obtain an exact mathematical solution that includes all the pertinent quantities. However, it is possible to obtain approximate solutions. The following paragraphs summarize some of the procedures being used.

Rate of Advance

The most widely used equation for approximating the rate of advance of the wetting front in irrigated furrows takes the form

$$x = aT^b \tag{2-2}$$

in which

x = distance water has advanced (feet)

T = total time since water was introduced in the furrow (minutes) a, b = constants

The accuracy of the above equation is dependent on several variables which affect the rate of advance. These factors include size of stream flowing into furrow, intake rate, slope of land surface, surface roughness, and shape of flow channel. In applying equation (2-2), these factors are assumed to be constant throughout the length of the run.

Intake Rate

Shull (25, 26) used measurements of inflow to a furrow, rate of advance, and volume of water in channel storage to ascertain furrow intake rate. This procedure is called the inflow-advance-storage method for determining infiltration. After comparing plots of infiltrometer data and several infiltration equations, he computed an equation of the form

$$I = C + A'\sqrt{T'} + BT' \qquad (2\text{-}3)$$

where

I = intake rate

T' = total time in minutes which water has been at the given location

C, A', B = constants.

This equation is similar to the one proposed by Israelsen and Hansen (18) for the accumulated depth of water applied.

Solving equation (2-2), integrating it and two other equations, Shull (25, 26) obtained an equation for the total infiltration in the entire wetted length of the furrow. The equation is as follows:

$$I_t = x(C + A'K\sqrt{T} + \frac{BT}{b+1}) \qquad (2\text{-}4)$$

in which

I_t = total infiltration in the entire wetted length of a furrow

$K = 1 - \frac{b}{2(b+1)} - \frac{b}{8(b+2)} - \frac{b}{16(b+3)}$

Smerdon and Hohn (39) using the inflow-advance-storage method and Criddle, et al (6), using the inflow-outflow method, showed that intake rate can be expressed by the simple empirical exponential equation

$$I = kT^c \qquad (2\text{-}5)$$

where

k and c = constants.

Several investigators (10, 18) state that the above equation is widely applied as the expression for infiltration for a short irrigation period, generally within twenty-four hours. However, the following equation may yield better results for a long irrigation period:

$$I = kT^c + C \qquad (2\text{-}6)$$

Since c is negative, I will decrease with an increase in T. Therefore, the intake rate, I, will approach a constant value, C, as time increases.

Figure 3 denotes the relationship between the total depth and the wetted distance down the furrow with time as the parameter (7, 15). The rate of flow down the furrow at the head end equals the inflow rate, Q, where d_o is the depth of water. The depth of water that has infiltrated at the upstream end of the furrow, o, after successive time intervals, Δt, is represented by d_1 through d_5. Points 1 through 5

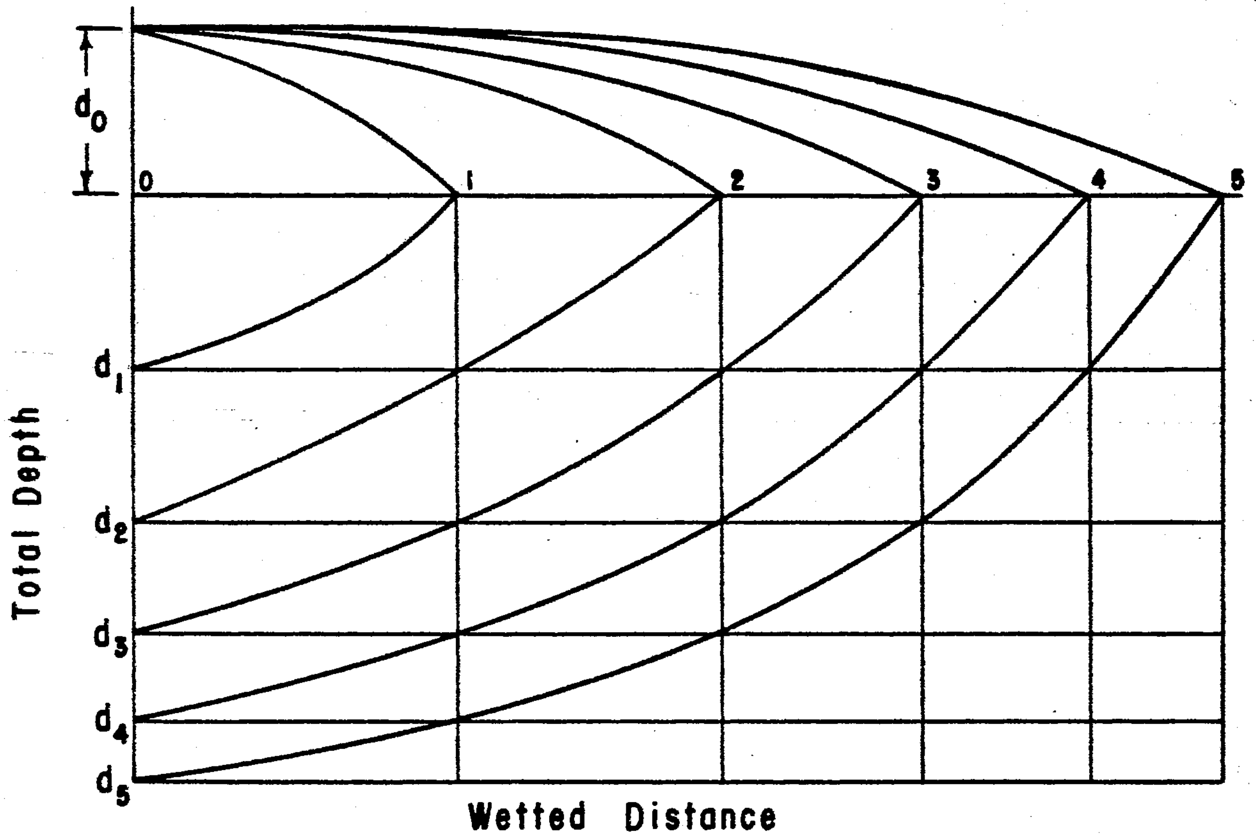

Figure 3. Total Depth as Related to Wetted Distance with Time as the Parameter

delineate the points of water advance after equal time intervals. The total inches applied to the wetted portion of the field expressed as surface inches of average depth is denoted by the symbol d in this thesis.

Manning's Equation

Robert Manning first presented his equation in 1889. It was later simplified to its present form:

$$V = \frac{1.49}{n} R^{\frac{2}{3}} S_f^{\frac{1}{2}} \qquad (2\text{-}7)$$

where

V = mean velocity (feet per second)

n = roughness coefficient (Manning's n)

R = hydraulic radius (feet)

S_f = slope of the energy line.

Because of its simplicity and practical applications, the Manning formula has become the most widely used equation for uniform open channel flow. The greatest difficulty in applying the Manning formula lies in the determination of the coefficient of roughness (n).

Uhl (33) conducted a series of gradually varied flow tests to determine the hydraulic roughness of the channel. The tests were run with three depths of flow for three selected flow rates. A few random tests were also run. He combined the energy equation with Manning's equation to solve for roughness. A log-log plot was made of Manning's n versus VR, where VR was the average velocity times the hydraulic radius. From the graph it was determined that $n = 0.0090(VR)^{-0.101}$. Uhl also found the average value of n for the tests to be 0.0096.

Spatially Varied Flow

Spatially varied flow is non-uniform open channel flow, where water runs in or out along the course of flow. There are two types of spatially varied flow:

1. Increasing spatially varied flow;
2. Decreasing spatially varied flow.

Irrigation systems are a form of decreasing spatially varied flow, where water is taken out or discharged along the reach. In this type of flow, diverted water is assumed to not affect the energy head. Therefore, the energy equation is used in solving this type of problem. The total energy at a channel section is

$$H = Z + Y + \frac{\alpha Q^2}{2gA^2}$$

Differentiating with respect to X, the above equation becomes

$$\frac{dy}{dx} = \frac{S_o - S_f - \frac{\alpha Q q}{2gA^2}}{1 - \frac{\alpha Q^2}{gA^2 D}} \tag{2-8}$$

This equation is the dynamic equation for decreasing spatially varied flow (5). Replacing the differentials, dx and dy, with finite increments Δx and Δy, the water surface profiles can be calculated.

In recent years, several researchers have studied decreasing spatially varied flow in irrigation distribution channels. This research was conducted to improve furrow discharge uniformity.

Mink (20) used a trapezoidal concrete channel with siphon tubes. He assumed that the resistance values (n) for gradually varied flow conditions were applicable at the same depth (y) and flow (Q) in the spatially varied flow tests. Using the calculated n, Mink found that the computed water surface profiles underestimated the observed profiles. Thus, the values of n obtained for gradually varied flow conditions were lower than the values of n for spatially varied flow.

Mink (20) calculated an adjusted value of Manning's n which he called $\bar{n}$. The value of $\bar{n}$ was obtained by incrementing n in Manning's equation

$$S_f = \frac{v^2 n^2}{2.208\ R^{\frac{4}{3}}}$$

and solving the Bernoulli energy equation, written for spatially varied flow as:

$$\frac{V_2^2}{2g} + Y_2 + Z_2 = \frac{V_1^2}{2g} + Y_1 + Z_1 + S_f \Delta X \qquad (2\text{-}9)$$

This was done until the calculated profile and observed profile agreed to within ±0.0001 foot at the upstream end of the primed bay. Mink concluded that $\bar{n}$ would best predict the water surface profiles for spatially varied flow.

Mink (20) also calculated an effective n, which he called n_e. The value of n_e was computed from the Manning equation. The energy slope was computed by

$$S_f = \frac{1}{L}\left(\frac{V_i^2}{2g} + (Y_i + Z_i) - (Y_o + Z_o)\right) \qquad (2\text{-}10)$$

where

V_i = average entering velocity

Y_i = upstream depth

Z_i = upstream bottom elevation

Y_o = downstream depth

Z_o = downstream bottom elevation.

Sweeten (29) derived the following theoretical relationship between the roughness coefficients $\bar{n}$ and n_e.

$$\bar{n} = \sqrt{3}\, n_e \tag{2-11}$$

Using the same channel as Mink, Sweeten (29) substantiated Mink's findings concerning $\bar{n}$. He made a significant contribution in deriving an equation which provided a direct solution for water surface profiles. According to his research, only velocity head gain and friction head loss contributed to the difference in potential energy in a horizontal bay. The difference in potential energy or change in water surface elevation (ΔWS) between an upstream station and a downstream point at X distance was expressed as:

$$\Delta WS = \Delta h_v - h_f \tag{2-12}$$

where change in velocity head was

$$\Delta h_v = \frac{V_i^2}{2g} - \frac{V_x^2}{2g} \tag{2-13}$$

and

$$\frac{V_x^2}{2g} = \frac{V_i^2}{2g}\left(\frac{L - X}{L}\right) \tag{2-14}$$

L is the length of the irrigation bay. Combining Equations (2-13) and (2-14) produced

$$\Delta h_v = \frac{V_i^2}{2g}\left(\frac{2X}{L} - \frac{X^2}{L^2}\right) \tag{2-15}$$

The friction losses between the stations from 0 to X_1 was calculated by

$$h_f = \frac{\bar{n}^2 V_i^2}{2.208 R^{\frac{4}{3}}} (X_1 - \frac{X_1^2}{L} + \frac{X_1^3}{3L^2}) \tag{2-16}$$

By substituting Equations (2-15) and (2-16) into Equation (2-12), the following equation was obtained:

$$\Delta WS = \frac{V_i^2}{2g} (\frac{2X}{L} - \frac{X^2}{L^2}) - \frac{\bar{n}^2 V_i^2}{2.208 R^{\frac{4}{3}}} (X - \frac{X^2}{L} + \frac{X^3}{3L^2}) \tag{2-17}$$

Equation (2-17) was used to directly solve water surface profiles or to solve for $\bar{n}$ when profiles were observed in a horizontal bay.

Sweeten and Garton (30) developed a method for using Equation (2-17) to solve water surface profiles in a two bay system. The solution for the upstream bay depended on the virtual length (L_1') of the bay. L_1' was the length of the bay necessary to completely discharge the inflow.

$$L_1' = L_s \frac{Q_I}{q_c} \tag{2-18}$$

where

L_s = orifice or weir spacing

Q_I = inflow

q_c = average cut-back discharge of one orifice or weir

The cut-back bay profile was solved by substituting L_1' in place of L and solving Equation (2-17) until X = L. Figure 4 shows the concept of virtual length, L_1', and the decrease in velocity in a two bay system. The sudden change in velocity at the intersection of the two bays was attributed to the drop causing the cross-sectional area of flow to

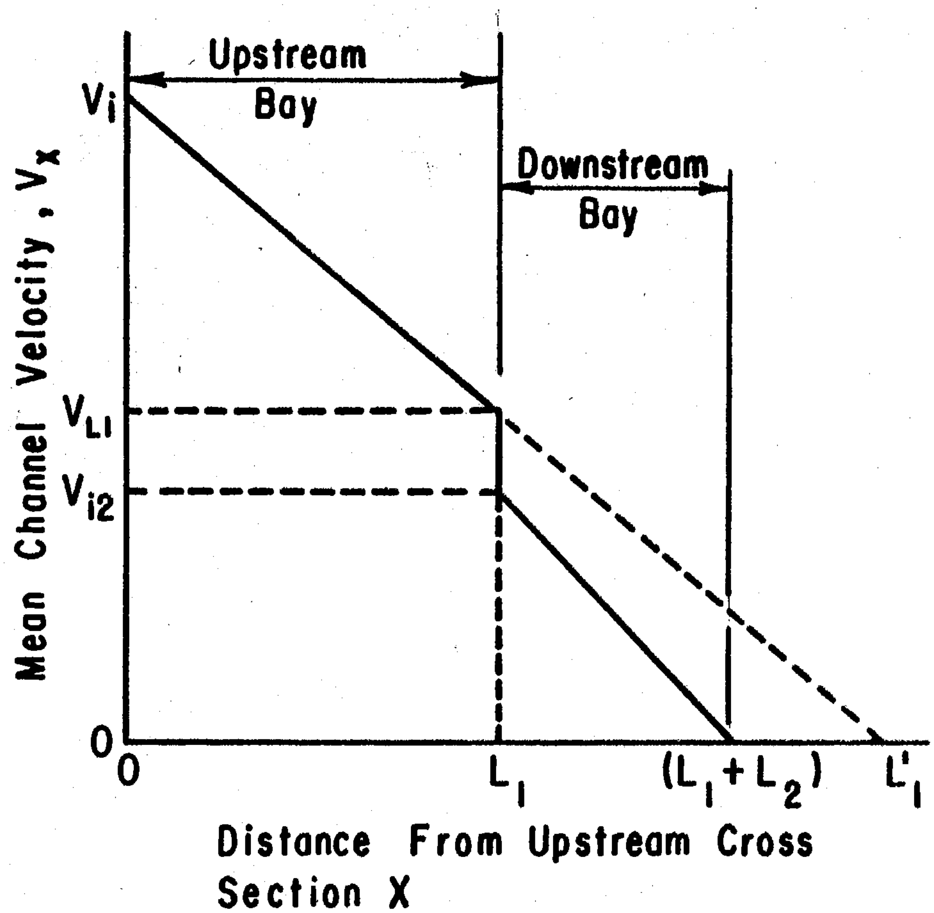

Figure 4. Assumed Decrease in Channel Velocity in a Two Bay Irrigation System with Change in Area Between Bays

be larger in the downstream bay.

Calculation of the water surface profile in the downstream or initial bay proceeded as if it were a single bay system with $L = L_2$ and Q_I being the inflow into the bay. The initial water surface elevation (d_{L2}) in the downstream bay was determined by

$$d_{L2} = d_{L1} + \text{drop} - k\left(\frac{V_{L1}^2 - V_{i2}^2}{2g}\right) \qquad (2\text{-}19)$$

where

d_{L1} = water surface elevation at the end of the upstream bay, determined from Equation (2-17) at $X = L_1$

drop = change in elevation between bays

$k\left(\frac{V_{L1}^2 - V_{i2}^2}{2g}\right)$ = head loss in the drop

Figure 5 demonstrates the change in water surface profile and the sudden change in elevation in a two bay system separated by a drop. The drop is represented by $(d_{L3} - d_{L1})$, while $(d_{L3} - d_{L2})$ depicts $k\left(\frac{V_{L1}^2 - V_{i2}^2}{2g}\right)$ which equals the head loss in the drop.

Uhl's (32) research was concerned with the hydraulics of an automated, semi-portable sheet metal flume with side orifice outlets. He found in initial flow or single bay tests mean values of n_e (0.0073) and $\bar{n}$ (0.0126). The value of $\bar{n}$ for cut-back bays was 0.0107. Uhl further determined the relationship between $\bar{n}$ and VR for both single and two bay tests as being

$$\bar{n} = 0.00952\ (VR)^{-0.2628} \qquad (2\text{-}20)$$

with a correlation coefficient $(r) = 0.885$. The standard deviation was $(s) = 0.022$.

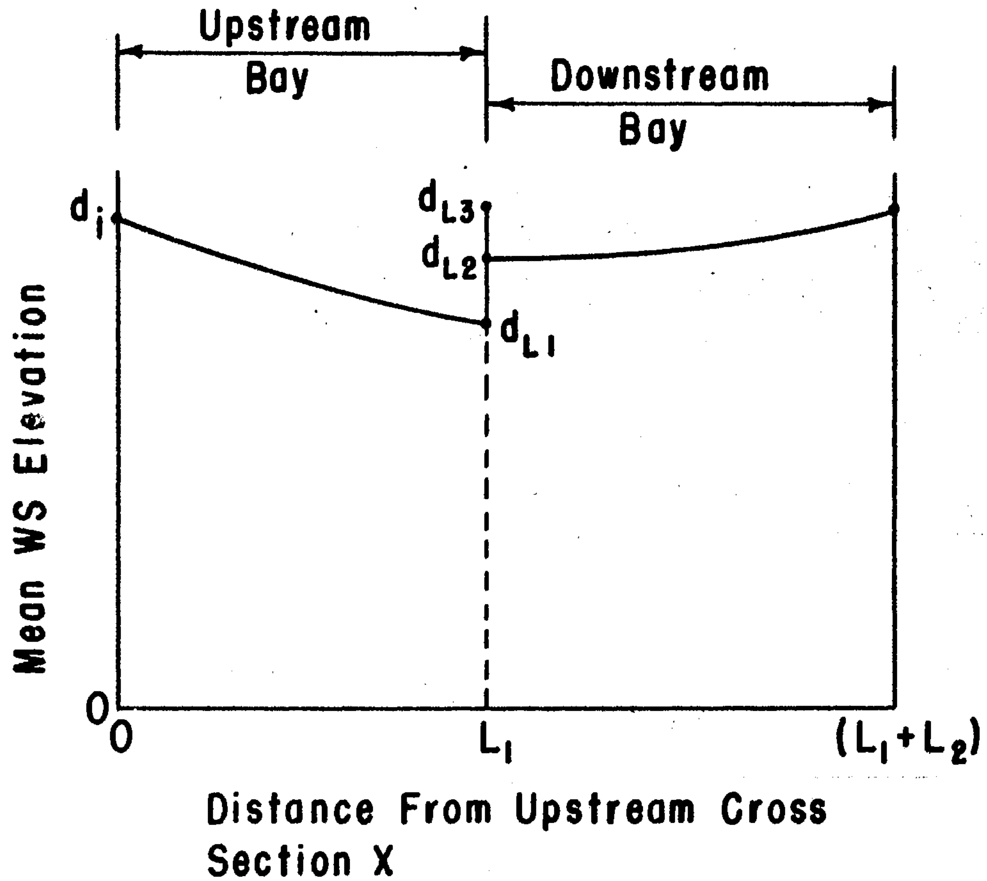

Figure 5. Water Surface Profile in a Two Bay Irrigation System with Change in Area Between Bays

Discharge from Circular Weirs and Orifices

Greve (14) conducted extensive experiments on vertical circular weirs. He ran tests on fourteen 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. Greve calculated a value for M and N for each diameter (D). The relationship between M and D was determined in the equation $M = k_1(D)^{c_1}$. A general formula of the form $q = k_1(D)^{c_1}(H)^N$ was devised by combining the above equations.

The theoretical discharge for an orifice is $q = A\sqrt{2gh}$. The true discharge through an orifice is always less than the theoretical discharge and is estimated by $q = C_d A\sqrt{2gh}$. The coefficient of discharge (C_d) is the product of the coefficient of velocity (C_v) and the coefficient of contraction (C_c).

Barefoot (1) studied the hydraulic properties of sloping weirs and orifices. He chose seven diameters ranging from one to eight inches for the experiment. From his research, he concluded that the vertical height of the orifice from the channel bottom did not affect the orifice discharge. The flow rate passing the orifice during decreasing spatially varied flow also did not affect the orifice discharge. Barefoot determined the maximum deviation of calculated values from observed values of discharge as being twenty-eight per cent.

While working under Barefoot, a student researched discharge characteristics of vertical circular orifices which were unpublished. Barefoot's results for vertical orifices and weirs compared favorably with Greve's findings. Thus, Barefoot's results were used for the preliminary design of the research project presented in this thesis.

CHAPTER III

EQUIPMENT

Sheet Metal Flume

Hydraulic Design

Several factors affected the hydraulic design. Included were dimensions of the channel section, height of the orifice above the bottom of the channel, diameter of the orifice, and spacing of the orifice. Land slope, range of initial and cut-back flows, and the friction factors n and $\bar{n}$ had some bearing on this design.

Uhl (32) adopted stability of water surface profiles as the main design criteria. He found the rectangular section to be the most feasible, having a bottom width of 1.5 feet and a depth of 1.25 feet.

Of primary consideration was orifice height. The bay immediately upstream from the bay flowing at cut-back could not be discharging any furrow flow. Therefore, the orifice elevation must be greater than the water surface elevation in the whole bay. Uhl chose an orifice height of 0.74 foot, allowing 0.40 foot for initial and cut-back flows, and 0.10 foot for free board.

Uhl (32) computed a table for the hydraulic design, varying inflows (Q), initial and cut-back flows (q), land slopes, and furrow spacing. This enabled a planner to select the system design which corresponded best to a specific field problem.

The range of variables in the design table was as follows:

1. Initial and cut-back furrow flows -- 6.0 g.p.m. to 40.0 g.p.m.

2. Incoming flow -- 1.0 to 2.0 c.f.s.

3. Land slope -- 0.05% to 1.0% slope

4. Furrow spacing -- 3.33 feet

5. Orifice diameter -- 1.5 inches to 2.5 inches

Structural Design

Uhl (32) established several objectives for the structural design. For the section to be portable, it was necessary that it be easily handled and easily assembled. The structural design in addition to being leak free also had to be strong enough to withstand permanent bending and buckling.

The final structural, angle iron framework chosen by Uhl is shown in Figure 6. This design could withstand full loading. The weight was only eight pounds per linear foot.

Since it was necessary to support the channel above the ground, a system was designed by Uhl (32) for this purpose. Stands were devised to enable minor adjustments of the channel elevation in the field, Figure 7.

Three changes were made in Uhl's structural design. Side cross-braces were installed to strengthen the system, Figure 8. This added approximately one pound per linear foot to the total weight of the section. The end top braces were removed from each section, enabling the channels to be turned upside down and be placed inside each other, Figure 9. Thus, twice as many channels could be hauled at one time. A smaller size orifice helped obtain the initial and cut-back flows de-

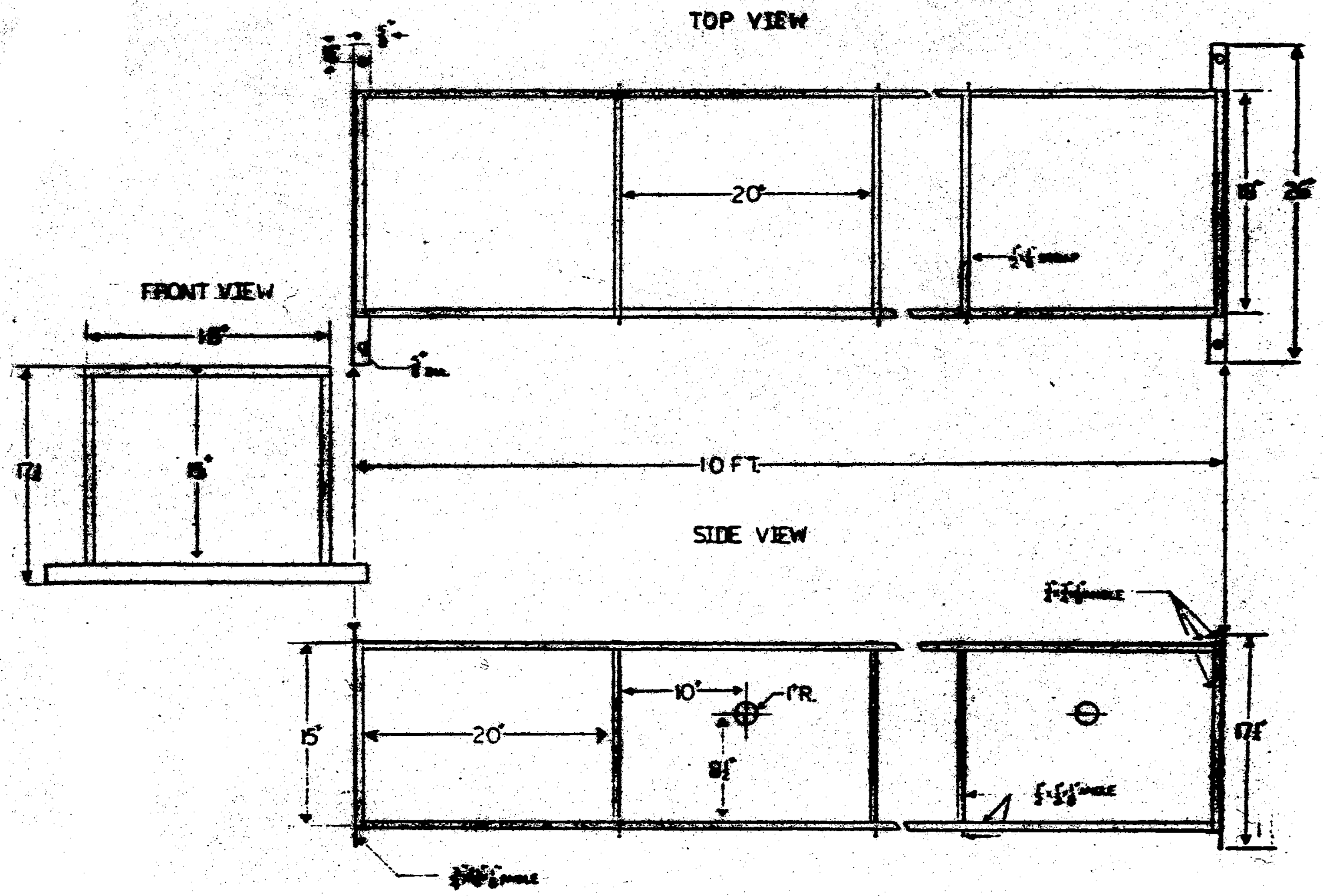

Figure 6. Graphic Illustration of the Structural Steel Angle Framework

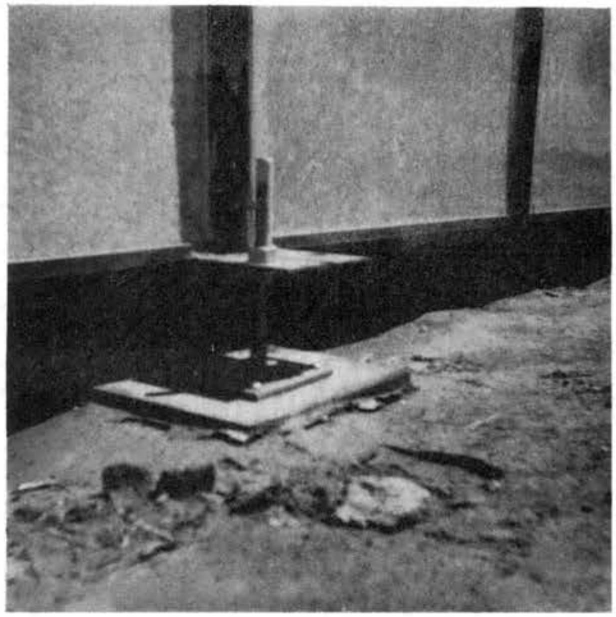

Figure 7. View of Supporting Stand

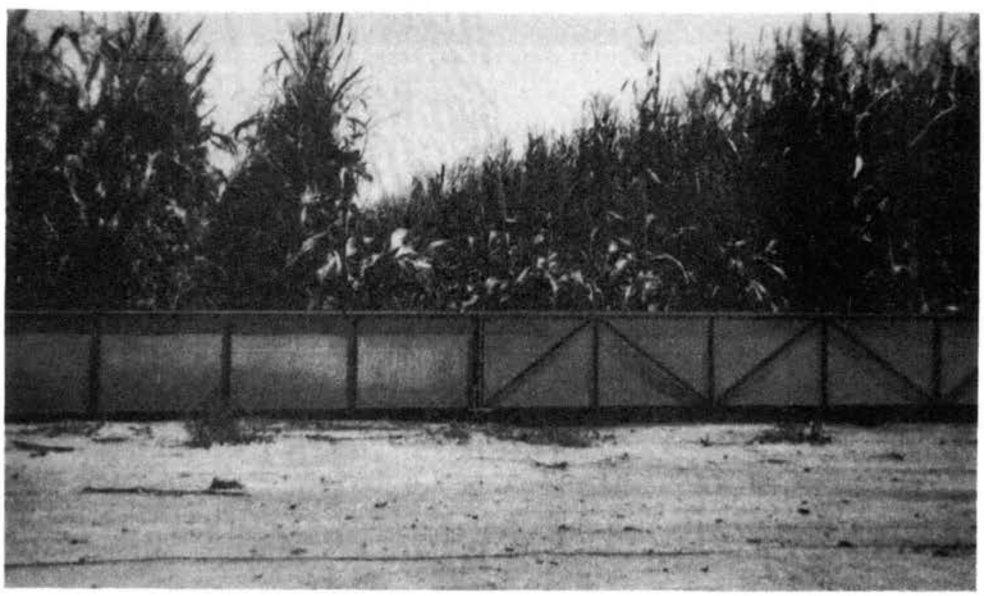

Figure 8. Side Cross Braces Added to the New Channel Sections

sired, Figure 10. This was accomplished by pop-riveting a plate with a smaller size orifice over the larger orifice.

Automatic check dams, designed by Pope (22), were employed for field testing. These check dams were operated by a solenoid rack-puller, a twelve-volt battery, and a time clock.

Measuring Equipment

HS Flume

Three variations of HS flumes were constructed and calibrated in the Oklahoma State University Agricultural Engineering Laboratory. The flumes were built according to the specifications of the United States Department of Agriculture for larger sized flumes (8). These flumes had an approximate head capacity of 0.30 foot, 0.35 foot, and 0.40 foot with maximum discharge capacities estimated of 17 g.p.m., 26 g.p.m., and 36 g.p.m., respectively.

Laboratory calibration of the flumes was obtained by pumping water from a 1350 gallon capacity sump. Pumped through a piping system into the HS flume, the water was discharged into a five-gallon bucket. The pumping system consisted of a $\frac{1}{2}$-horsepower, motor-driven, Bell and Cossett centrifugal pump. It was connected by a two-inch pipe line.

Inflow into the system was measured by the weight-time method. Measuring equipment consisted of a stop watch, a five-gallon bucket, and a set of Toledo platform scales.

The flumes were leveled by a circular leveling bubble, mounted on the forebay of each. The forebay also was equipped with a baffle to dissipate turbulence.

Through the regulation of the gate valve, inflows from 0.30 g.p.m.

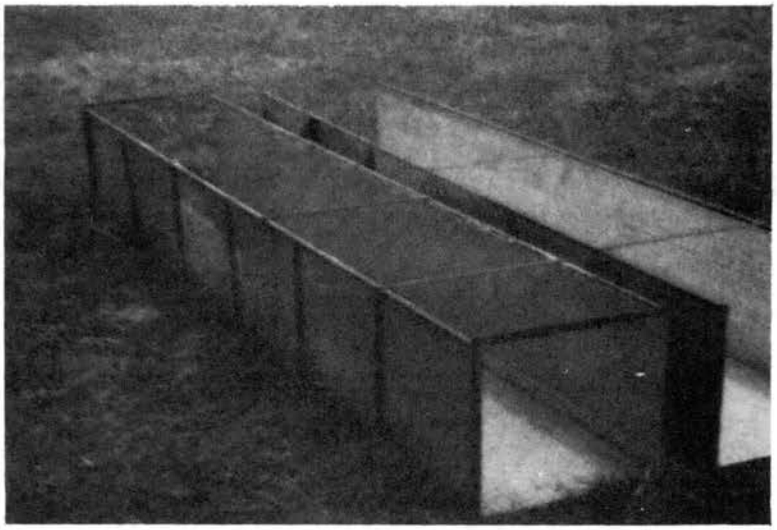

Figure 9. Top End Braces Removed from Channel Sections

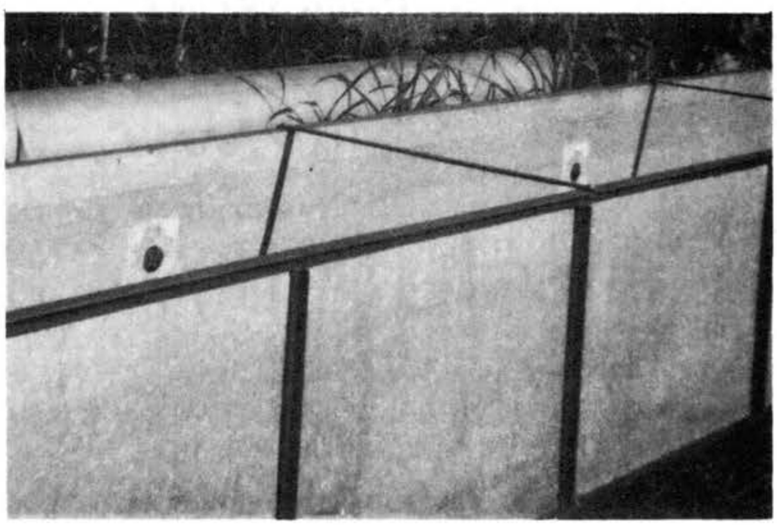

Figure 10. Smaller Orifices Pop-Rivetted Over the Two-Inch Orifice

to 28.70 g.p.m. were initiated. Stabilization of the water surface occurred before any readings were recorded. The discharge rates were calculated by weight and time measurements. Figure 11 shows the HS flumes being calibrated in the laboratory.

The results of the HS flumes' calibrations are presented in Appendix A. Each set of data fits a linear relationship of $\log_{10}$ q versus $\log_{10}$ h, where q is the HS flume discharge and h is the head.

To obtain direct field readings, a staff gage incremented in g.p.m. was prepared from the calibration data of each flume. Staff gages were mounted with rubber cement and varnish in the throat section of each flume.

The flumes were constructed in such a way that they could be hand-held. This allowed quick, accurate (±10%) field measurement of small flow rates.

Point Gage

A point gage was useful in measuring head for weir and orifice flow. It was also employed to determine surface elevations in the sheet metal channel.

A movable bridge, fitting square to each side of the channel, was built. It was equipped with a movable bracket that could be clamped tight at any location. The point gage was mounted on the bracket. The bridge, bracket, and mounted point gage are shown measuring water surface elevations during orifice flow in Figure 12. This apparatus was also used to determine orifice elevations in the channel.

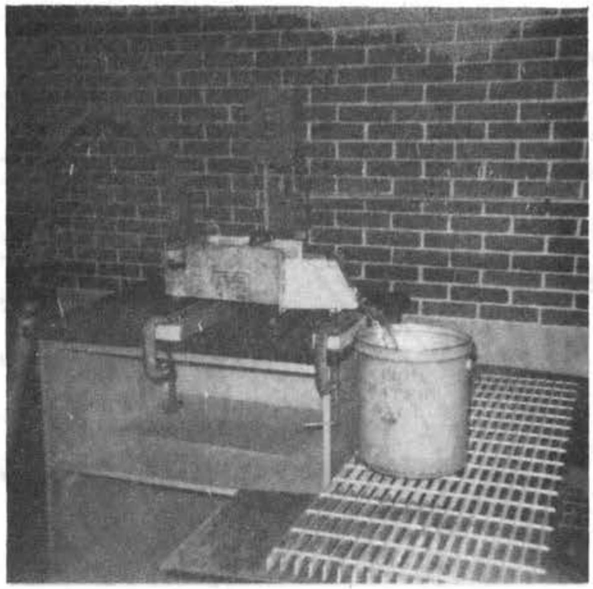

Figure 11. Laboratory Calibration of HS Flume

Figure 12. Point Gage and Assembly

Inflow Meter

An eight-inch propeller-type Badger meter measured inflow into the channel. Used to connect the gated pipe with the channel, the meter was inserted and clamped inside a vinyl-coated neoprene sleeve. The meter was calibrated in the laboratory to insure accuracy and to enable corrections for flows above 200 g.p.m. A $7\frac{1}{2}$-horsepower, motor-driven, Berkley centrifugal pump was connected to a six-inch pipe line.

A six-inch Sparling meter installed in the line at the OSU laboratory served as the primary measuring device for calibration of the field meter. This instrument had been calibrated with a sharp edge orifice and a U-tube manometer. Calibration was accomplished at the Outdoor Hydraulic Laboratory near Stillwater, Oklahoma.

Stop watches and volume totalizer dials of each meter determined the rate of discharge. The readings were taken simultaneously for the meters at the various discharges. Flow rates recorded from the inflow meter were then plotted on arithmetic paper against corresponding corrected readings from the laboratory meter. This data produced a straight line from which the actual inflow was predicted.

Anemometer

A five-digit dial, three-cup anemometer determined wind velocity. The accuracy of this instrument was checked with a three-cup anemometer located on an experimental pond at Oklahoma State University. The five-digit distance totalizer and a watch were used to reveal the average wind velocity over a period of time. From the collected data, the anemometer was found to be accurate within ±1.0%.

CHAPTER IV

METHOD AND PROCEDURE

Design Procedure

The design procedure described in this chapter is essentially the same as the one proposed by Garton (11, 12). Some changes were made in the method of determining the amount of drop between bays.

A hypothetical situation was devised to illustrate the techniques involved. The following conditions were assumed:

Length of channel	620.0	ft.
Slope of field in channel direction	0.0014	ft./ft.
Furrow slope	0.002	ft./ft.
Furrow spacing	3.33	ft.
Orifice size	2.0	in.
Supply flow	900.0	g.p.m.
Initial furrow flow	23.1	g.p.m.
Cut-back furrow flow	8.0	g.p.m.

Initial Design

The purpose of the initial design was to ascertain the average drop between bays. In this design the cut-back discharge was held constant at 8.0 g.p.m. The initial discharge varied according to the amount of supply flow in excess of cut-back discharge.

Each channel section was ten feet long, containing three orifices. Therefore, the number of orifices per bay had to be a multiple of three.

The initial design was determined through the following procedure. In bay 1 thirty-nine orifices were needed to discharge the supply flow of 900 g.p.m. Initial furrow flow of each orifice reached 23.1 g.p.m. The thirty-nine orifices required 312 g.p.m. at the cut-back discharge of eight gallons per minute. By subtracting 312 g.p.m. from 900 g.p.m., 588 g.p.m. were carried by the orifices in the second bay at initial flow. Twenty-seven orifices at an initial flow of 21.8 g.p.m. were needed. Following this process the initial design was determined. Table I shows the results of the calculations for each bay.

TABLE I

INITIAL DESIGN OF THE SYSTEM FOR THE EXAMPLE CONDITIONS

Bay No.	Q_I gpm	Q_C gpm	No. of Orifices	Orifices Cut-Back	q_i gpm	q_c gpm	Distance ft.
1	900		39		23.1		130
		312		39		8.0	
2	588		27		21.8		220
		216		27		8.0	
3	684		30		22.8		320
		240		30		8.0	
4	660		30		22.0		420
		240		30		8.0	
5	660		30		22.0		520
		240		30		8.0	
6	660		30		22.0		620

To determine the drop between bays, a graph of head versus discharge was plotted for the two-inch orifice. This graph contained plots for both weir and orifice flow. It was applied to determine initial and cut-back heads for the respective furrow flow rates in the initial design. The differences of the two heads in each bay was recorded as the drop between bays.

The average drop between bays was equal to the total drop divided by the number of drops. The final bay did not have the cut-back discharge designed in it. Table II shows the flow rates, their corresponding heads, and the final average drop.

TABLE II

FINAL DROP BETWEEN BAYS FOR THE EXAMPLE DESIGN

Bay No.	q_i gpm	h_i ft.	q_c gpm	h_c ft.	Difference ft.
1	23.1	0.280	8.0	0.124	0.136
2	21.8	0.260	8.0	0.124	0.152
3	22.8	0.276	8.0	0.124	0.139
4	22.0	0.263	8.0	0.124	0.139
5	22.0	0.263	8.0	0.124	0.139
6	22.0	0.263			
				Total Drop =	0.705

$$\text{Average drop} = \frac{0.705}{6 - 1} = 0.141 \text{ ft.}$$

Final Design

The drop between bays was set at a constant value and was used to calculate the initial orifice discharge. The sum obtained by adding the drop to the cut-back head equaled the initial head.
The number of orifices and the initial discharge per orifice were the same in bay 1 for the initial and final designs. The remainder of the final design was calculated by trial and error, using the initial design as a guide.

In the final design, the cut-back discharge was chosen arbitrarily. This discharge specified the initial discharge of the downstream bay. The cut-back and initial flows were computed by multiplying the cut-back and initial discharge per orifice by the number of orifices at cut-back (39--bay 1) and initial discharge (27--bay 2) respectively. The summation of cut-back flow and initial flow in the immediate downstream bay was approximately 900 g.p.m. If this sum was not accomplished, a new cut-back discharge was chosen. This trial and error process was repeated until the sums equaled approximately 900 g.p.m. The rest of the final design shown in Table III was completed in this manner.

Computed in almost the same manner as Table II, Table IV enabled a check in initial head and average drop. In Table IV the difference in initial and cut-back heads was doubled. When twice the difference exceeded the initial head, no furrow flow in the bay upstream from the bay at cut-back could occur.

TABLE III

FINAL DESIGN OF SYSTEM FOR THE EXAMPLE CONDITIONS

Bay No.	Q_I gpm	Q_C gpm	No. of Orifices	Orifices Cut-Back	q_i gpm	q_c gpm	Distance ft.
1	900		39		23.1		130
		308		39		7.9	
2	596		27		22.1		220
		230		27		8.5	
3	672		30		22.4		320
		231		30		7.9	
4	669		30		22.1		420
		231		30		7.9	
5	669		30		22.1		520
		231		30		7.9	
6	669		30		22.1		620

TABLE IV

CHECK OF FINAL DESIGN

Bay No.	q_i gpm	h_i ft.	q_c gpm	h_c ft.	Difference ft.	Twice Difference
1	23.1	0.280				
			7.9	0.124	0.141	0.282
2	22.1	0.265				
			8.5	0.128	0.141	0.282
3	22.4	0.269				
			7.9	0.124	0.141	0.282
4	22.1	0.265				
			7.9	0.124	0.141	0.282
5	22.1	0.265				
			7.9	0.124	0.141	0.282
6	22.1	0.265				

Location

A farm in the panhandle of Oklahoma, owned and operated by Mr. S. Perkins, was selected for the field tests. The field was located one-half mile east of Guymon, Oklahoma, Texas county. The soil, classified as Pullman-Richfield, exhibited deep, dark, clayey characteristics (28).

Preliminary Tests

In the summer of 1970, preliminary field tests were conducted in two segments.

Part One

Results of advance rate of flow tests aided in determination of the desired initial and cut-back discharges for the irrigation system. At this time the slope of the field in the direction of the channel was determined. Furthermore, measurement of available supply flow from the well was recorded. The results of these tests and measurements made possible the determination of the preliminary design.

Part Two

Upon completion of the design and the installation of the system at Guymon, various tests enabled a study of the operating characteristics of the system. These tests included rate of advance and measurement of variation in furrow discharge.

Figure 13 is a view of the field set-up of the cut-back system. Note the gates that were in place and the initial and cut-back discharges which were occurring in the two downstream bays.

Figure 14 is a close-up of the initial and cut-back discharges. Observe the difference in the horizontal distance traveled by the two flow rates. The initial discharge was at orifice flow while the cut-back was at weir flow.

Rate of advance tests were accomplished by timing flow rates through the field. A stop watch was used to time the flow rates. The selected furrows were staked at one hundred foot stations.

A five-gallon bucket and a stop watch were used to measure various initial and cut-back discharges. It was desirable to study the variation of furrow discharge along the reach of a bay. The selection of orifices for the test was by a randomized method.

Final Field Tests

From the data taken during the preliminary field tests, it was determined that some minor changes were needed in the design. The system was redesigned and set up as specified in the final design.

The final field tests were taken in the summer of 1971. Criddle's (6) procedures for collecting data on advance rate of flow were followed. The sequence of steps is listed below:

1. Stakes were set at one hundred foot stations down the furrow.

2. Outflow-measuring points were selected down the test furrows.

3. The time water started to flow into each test furrow was recorded.

4. The time water reached each station was recorded, Figure 15.

5. Streams were measured periodically along the furrows to determine intake rate, Figure 16.

The initial and cut-back furrow flow rates were calculated using

Figure 13. Field Set-Up of Irrigation System

Figure 14. Close-Up of Initial and Cut-back Discharges

Figure 15. Wetting Front Approaching a Station

Figure 16. HS Flume Used to Measure the Quantity of Water Flowing Past a Station

the point gage assembly described in Chapter III. When the wind became a factor, the five-gallon bucket and stop watch, employed in the preliminary field tests, were used. The point gage assembly was also used to measure water surface profiles.

An engineer's level and a Philadelphia rod were used to level the bays before each irrigation. These instruments also determined the top elevations of the channel to ±0.001 foot. This was done at the locations where water surface profiles were measured for reference elevations. Utilizing the reference elevations and the point gage readings, the water surface elevation was measured to ±0.001 foot.

A five-digit dial, three-cup anemometer was located midway between upstream and downstream ends of the channel. It was placed two feet to the west of the channel. This was done to eliminate error due to channel boundary effects. The data collected from the anemometer enabled the study of the effects of the wind on the water surface profile. The field set-up is shown in Figure 17.

A constant record of inflow into the channel was kept. This was accomplished by inserting an eight-inch Badger meter between the gated pipe and the upstream end of the channel, as shown in Figure 18. Four hose clamps made the sleeve connections water tight.

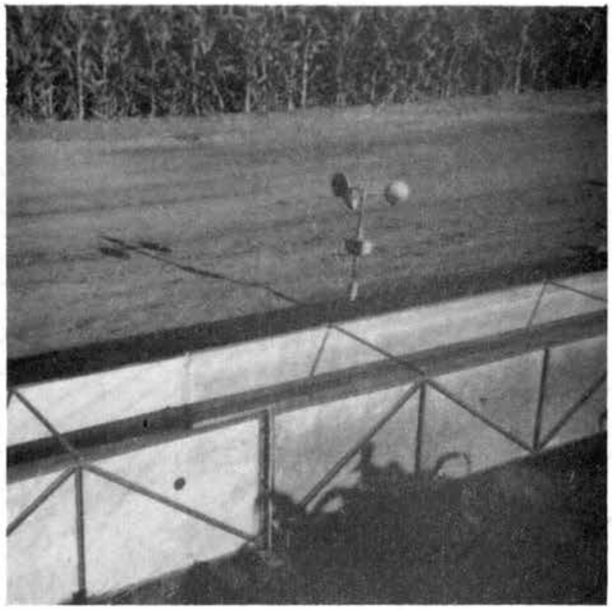

Figure 17. Five-Digit Dial Three-Cup Anemometer

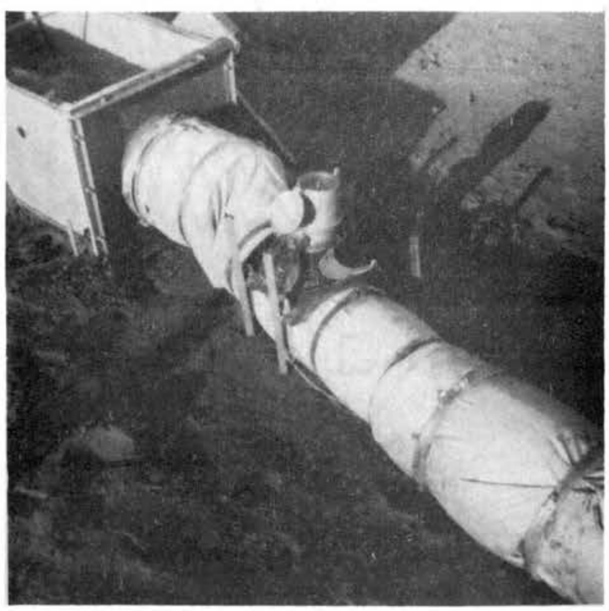

Figure 18. Eight-Inch Badger Meter Used to Measure Inflow into the Flume

CHAPTER V

PRESENTATION AND ANALYSIS OF DATA

Introduction

Preliminary field tests were conducted in two parts during the summer of 1970. The objective of part one was to design a cut-back irrigation system. Part two tested the design through a field study. The irrigation system, installed after the first irrigation, was used for two irrigations. During this period, the orifices were calibrated.

In the summer of 1971, final field tests were conducted after the irrigation system was redesigned and installed. The channel was set up before the first irrigation and used for all the waterings. Before each irrigation, channel sections were leveled.

Preliminary Field Tests

Rate of Advance (Part One)

To design a cut-back irrigation system properly, one must understand the hydraulic characteristics of the field. This information, when combined with past irrigation performances of the field, establishes the required initial and cut-back discharges.

Several test furrows were chosen. It was assumed that these furrows were a true representation of the field. Flow rates, varying from 30.5 g.p.m. to 4.8 g.p.m., were released into test furrows. The advance of these furrow discharges was timed at 100-foot increments.

A digital computer was utilized to run a regression analysis on the rate of flow advance data. Contrary to popular belief, the best fit equation was not of the form of equation (2-2). The researcher assumed that this discrepancy was caused by several conditions present in the field. The most important factors were the varying amounts of debris, the type of soil, and the variation of soil moisture along the furrow.

The expression that best fit the data was a quadratic equation of the form

$$x = a + bT + cT^2 \qquad (5\text{-}1)$$

where

x = distance water has advanced (feet)

T = total time since water was introduced in the furrow (minutes)

a, b, c = constants

The resulting equations of the regression analysis are shown below. The furrow discharges (q), application time ranges (minutes), correlation coefficients (r), and standard deviations (s) are also listed for each expression.

q gpm	Equation--x vs T	Range min.	r	s ft.
30.5	$x = -12.0 + 21.99T - 0.0893T^2$	4 - 80	0.999	16.6
19.0	$x = 39.0 + 14.62T - 0.0368T^2$	4 - 96	0.999	12.2
16.7	$x = 49.0 + 12.21T - 0.0290T^2$	5 - 105	1.000	9.9
10.0	$x = 72.0 + 6.07T - 0.0108T^2$	7 - 300	0.908	21.1
4.8	$x = 57.0 + 3.17T - 0.0058T^2$	9 - 300	0.998	11.9

The results obtained using the above equations are presented in Figure 19. The 30.5 g.p.m., 19.0 g.p.m., and 16.7 g.p.m. discharges watered through the field in a relatively short period of time. The 10.0 g.p.m. flow advanced approximately three-fourths of the furrow length, while the 4.8 g.p.m. flow progressed only one-third of the furrow length. The curves plotted in Figure 19 were used in the determination of the desired initial and cut-back furrow flow rates.

Depth Applied--Rate of Advance (Part One)

An equation for depth applied (d) in inches was determined. The following known relationships were used:

Row spacing = 3.33 ft.

Acre-inch = 3,630 $ft.^3$

cfs = 3,600 $ft.^3/hr.$

Acre = 43,560 $ft.^2$

Time = 60 min./hr.

The equation was found to be

$$d = 0.48 \frac{(gpm)(T)}{x} \tag{5-2}$$

Using equation (5-2) and the rate of advance data, the depth applied for each station was determined. The following relationships were calculated by the digital computer from the depth-time data. The furrow discharges (q), applicable ranges (minutes), correlation coefficients (r), and standard deviations (s) are also shown.

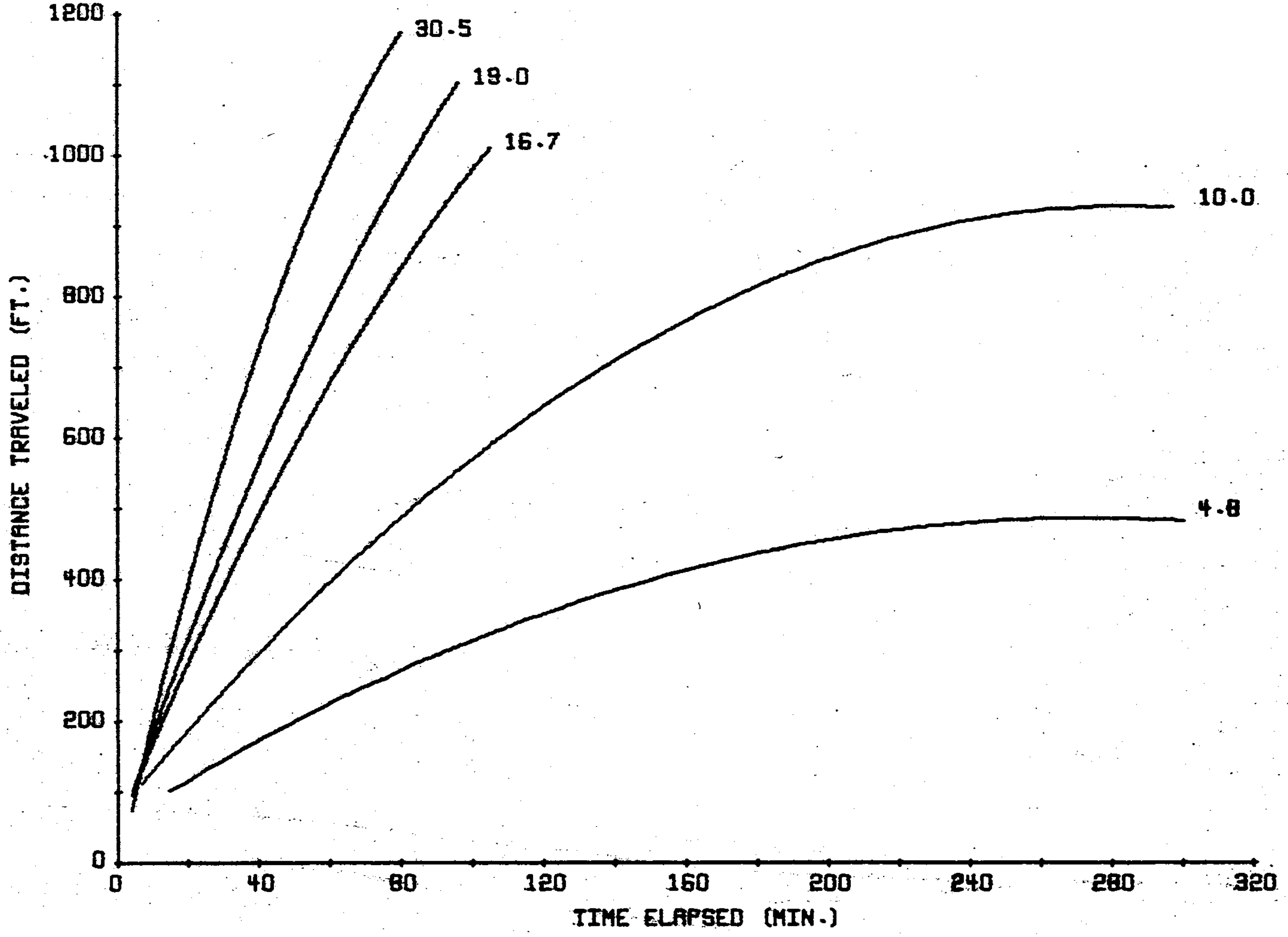

Figure 19. Distance Versus Time for Various Furrow Discharges Represented in g.p.m.

q gpm	Equation--d vs T	Range min.	r	s in.
30.5	$d = 0.69 + 0.0028T + 0.000013T^2$	4 - 80	0.954	0.04
19.0	$d = 0.43 + 0.0066T - 0.000031T^2$	4 - 96	0.979	0.03
16.7	$d = 0.34 + 0.0093T - 0.000047T^2$	5 - 105	0.958	0.05
10.0	$d = 0.41 + 0.0046T - 0.000003T^2$	7 - 300	0.994	0.04
4.8	$d = 0.33 + 0.0041T - 0.000002T^2$	9 - 300	0.999	0.02

The above equations were used to determine the relationship between depth (d) and time (T) for the different discharges, Figure 20. The 19.0 g.p.m., 16.7 g.p.m. and 10.0 g.p.m. curves were essentially identical. Thus, the depth versus time relationships were independent of discharges between 10.0 g.p.m. and 19.0 g.p.m. Since it was virtually impossible to obtain exact furrow discharges during each irrigation, the depth-time functions were used for comparison purposes.

Preliminary Design

As discussed in Chapter II, several factors must be considered in the designing of a cut-back irrigation system. These factors are furrow stream size, water supply available, slope of the land surface in the channel direction, length of the channel, and orifice size.

Mr. S. Perkins, the cooperating farmer, applied an average of 9.6 g.p.m. to 10.3 g.p.m. of water per furrow for twenty-four hours. From Figure 19, this appeared reasonable. Therefore, the amount was chosen for the design average. An initial flow of 12.6 g.p.m. and a cut-back discharge of 7.8 g.p.m. were selected for the preliminary design.

The supply flow available from the well equaled 670 g.p.m. A propeller-type Sparling meter was used to measure the well discharge.

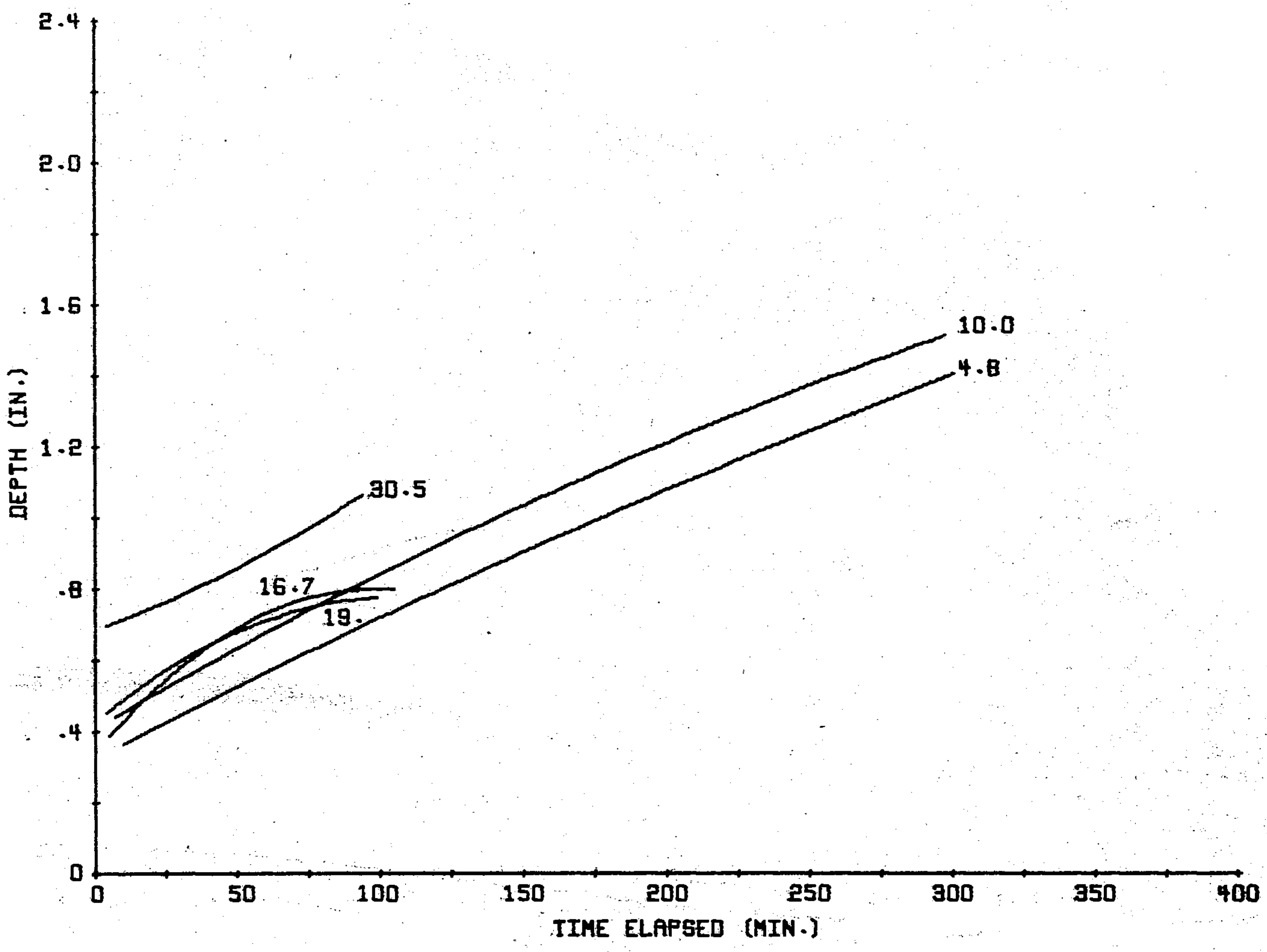

Figure 20. Depth Versus Time for Various Furrow Discharges Represented in g.p.m.

The slope of the field in the direction of the channel was determined to be 0.218 per cent. A Philadelphia rod and an engineer's level were used to calculate the fall in elevation of the land surface.

The length of the system was restricted by the number of ten-foot channel sections available. Seventy-two sections were built.

Barefoot's equations were used to plot graphs containing both weir and orifice flows of several different sized orifices. A $1\frac{5}{16}$ inch orifice was selected for the system, since is appeared most feasible for the desired flows.

The preliminary design was determined by the procedure discussed in Chapter IV. The results of the calculations are shown in Appendix B.

The design consisted of six bays with a total length of 710 feet. The bays were separated by 0.24-foot drops. The initial furrow flow rates varied from 12.9 g.p.m. to 12.5 g.p.m., while the cut-back furrow discharges varied from 7.6 g.p.m. to 6.8 g.p.m. between bays.

The maximum non-erosive furrow stream for an average soil is approximately

$$\text{gpm} = \frac{10}{\%\ \text{slope}} \qquad (5\text{-}3)$$

The slope of the furrow was 0.402 per cent. For this field the maximum non-erosive furrow discharge was

$$\frac{10}{0.402} = 24.9 \text{ gpm}$$

Therefore, the design furrow streams were expected to be non-erosive.

Depth Applied--Rate of Advance (Part Two)

When a tractor pulling a planter is driven through a field, two rows receive a higher compactive effort. This higher compactive effort is due to the tractor tires.

In the summer of 1970, Mr. S. Perkins hooked weighted drums to the back of the planter. These drums were dragged through the field to compensate for the compacting effect of the tractor tires. By doing this, the farmer hoped to obtain a uniform application rate between rows.

Two furrows compacted by a tractor tire and drum were selected in each bay, while two furrows compacted by a drum only were also selected. Rate of advance data were collected from these furrows. Substituting these data into equation (5-2) the depth applied was calculated for each station.

Following the procedures previously discussed, equations were found for the depth-time data. The data used for these calculations were taken for bay 1 during the second irrigation.

Tractor tire and drum compaction:

$$d = 0.84 + 0.0020T$$

with a range of 9 minutes to 385 minutes, a correlation coefficient $r = 0.957$, and a standard deviation $s = 0.06$.

Drum compaction:

$$d = 0.77 + 0.0025T$$

with a range of 9 minutes to 360 minutes, a correlation coefficient $r = 0.985$, and a standard deviation $s = 0.05$.

Figure 21 is a plot of the above equations. The drum and tractor tire curve had a smaller slope than the drum curve. Therefore, less water per unit time was applied. Since both rows received water at the same rate, the water in the drum and tractor tire furrow advanced at a faster rate. This difference in rate of advance was expected since the drum and tractor tire furrow received a higher energy input for compaction.

Variations in Initial and Cut-Back Discharges

The main problem with the type of cut-back irrigation system adopted in this study was the variation of orifice elevations along the length of the channel. When this orifice variation in elevation was combined with rising or falling water surface profiles, variations in both weir and orifice discharges occurred.

Both weir and orifice discharge uniformities were determined for each bay during every irrigation. Several orifices along the reach of the bay were chosen for the tests. The time-volume method was used to calculate the discharges. For a given bay the mean ($\bar{X}$) and standard deviation (s) were also computed. A summary of the above tests is located in Appendix B.

In each bay larger variations were observed for weir flow as opposed to orifice flow. This was anticipated since for weir flow, q varies with $h^{1.873}$ and for orifice flow with $h^{0.709}$. The largest variation in furrow discharge was observed as 3.8 g.p.m., while the largest standard deviation was calculated to be 1.17 g.p.m.

For each irrigation the observed initial and cut-back furrow discharges were below the corresponding design discharges. From the cal-

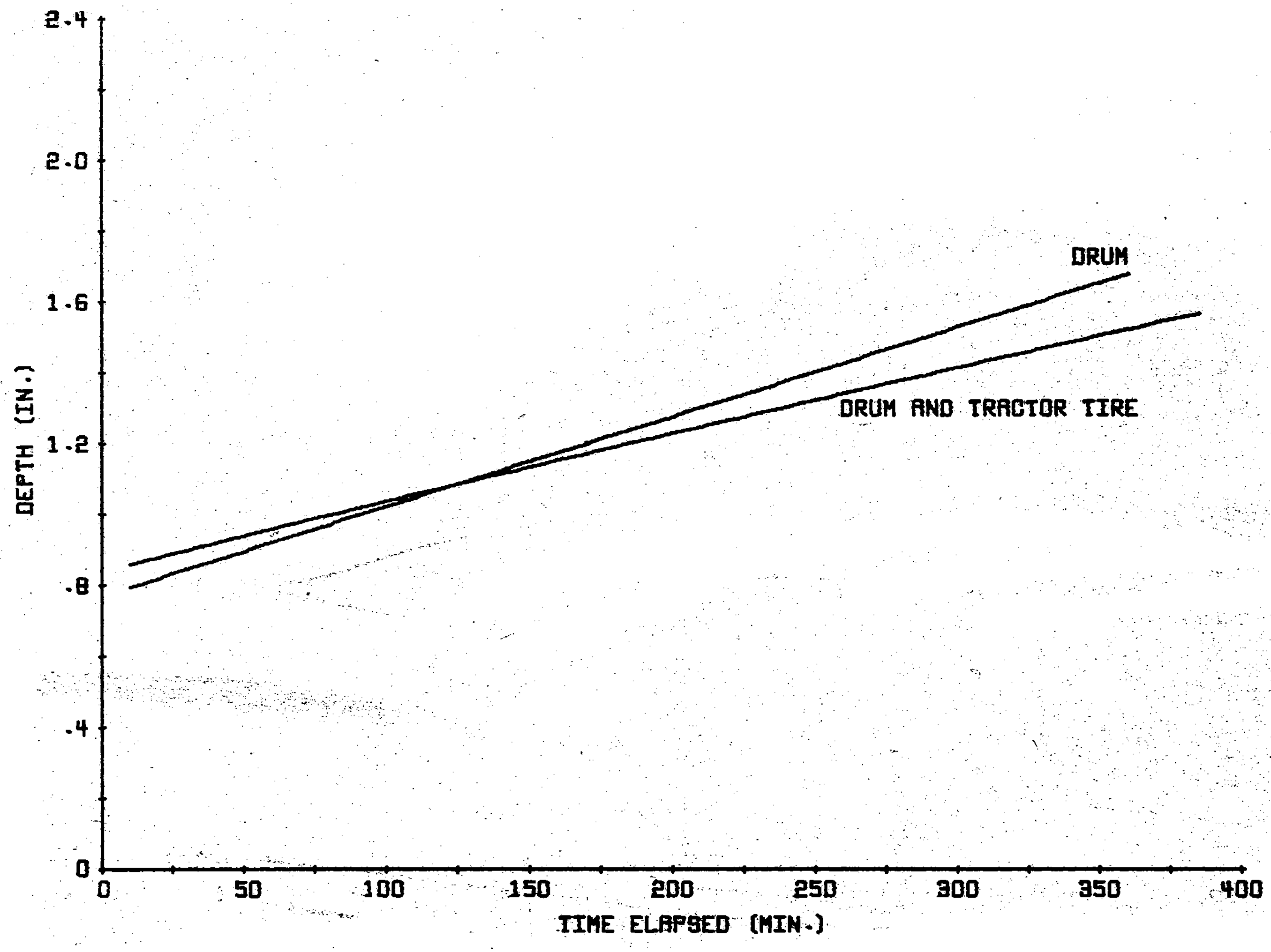

Figure 21. Depth Versus Time for Bay 1 During the Second Irrigation, Summer, 1970

culated values, the greatest differences were indicated in the cut-back discharge values. These discrepancies were the results of an over prediction of the supply flow delivered to the system.

Weir and Orifice Calibration

Three orifices were selected and calibrated in the Agricultural Engineering Laboratory at OSU. The inflow was measured with a two-inch, nutating disk, water meter. A point gage measured the head.

A micrometer was used to determine the actual orifice diameter. Several measurements were taken, and the average diameter was found to be 1.328 inches.

Figure 22 is a plot of head versus discharge for both weir and orifice flows. The graph also contains a plot of Barefoot's equations for a 1.328 inch orifice. Figure 22 was used as the weir and orifice calibration curve.

The best fit equation for weir flow was found to be

$$q = 317.35h^{1.873}$$

with a correlation coefficient $r = 0.996$ and a standard deviation $s = 0.071$. The best fit equation for orifice flow was found as

$$q = 26.15h^{0.709}$$

with a correlation coefficient $r = 0.997$ and a standard deviation $s = 0.207$,

where

q = weir and orifice flow (gpm)

h = head of water above the bottom of the orifice (feet)

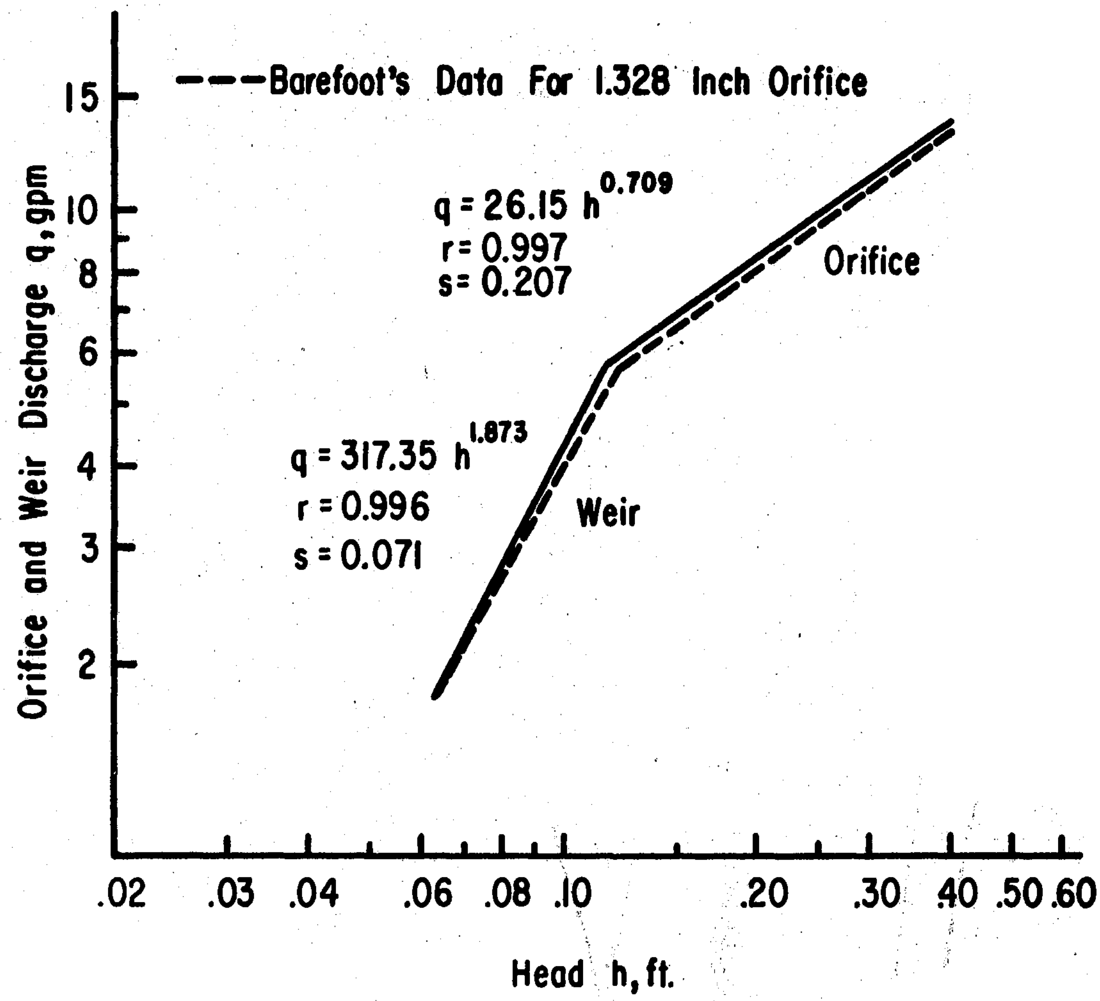

Figure 22. Weir and Orifice Flow (q), Versus Head (h) Compared to Barefoot's Data

Final Field Tests

Final Design

The preliminary field tests determined that the design inflow of 670 g.p.m. was not achieved. This necessitated some minor changes in the design.

The system was redesigned for an inflow of 575 g.p.m., retaining the drop of 0.24 foot between bays. The desired initial and cut-back discharges were changed to 12.0 g.p.m. and 3.7 g.p.m. respectively. The results of the final design are shown in Appendix C.

The final design consisted of five bays with a total length of 630 feet. The initial flow rate varied from 12.0 g.p.m. to 12.1 g.p.m. The cut-back furrow discharges varied from 3.7 g.p.m. to 4.1 g.p.m. between bays.

Depth Applied

During the growing season of 1971, the farmer did not compensate for the compacting effect of the tractor tires. The only compaction obtained was that done by the tractor tires.

Two furrows compacted by a tractor tire and two furrows with no tire compaction were selected in each bay. Rate of flow advance data were collected from these furrows as weather and time permitted. These data were substituted into equation (5-2) to determine the depth of water applied at each station.

A digital computer was used to fit equations to the depth-time data. The following equations were fitted to collected data for bay 1 during the second irrigation.

Tractor tire compaction:

$$d = 0.31 + 0.0041T - 0.000008T^2$$

with a range of 6 minutes to 250 minutes, a correlation coefficient r = 0.995, and a standard deviation s = 0.02.

No tire compaction:

$$d = 0.41 + 0.0084T - 0.000012T^2$$

with a range of 9 minutes to 350 minutes, a correlation coefficient r = 1.000, and a standard deviation s = 0.02.

Figure 23 is a plot of the above equations. The tractor tire curve had a much smaller slope than the no tire curve. At an elapsed time of 200 minutes, there was a 100 per cent difference in the two curves. In most cases the tractor tire rows were watered through the field before water in the other rows had reached the half-way point. The large variation in application uniformity resulted in a large percentage of runoff occurring from the tractor tire rows.

The graphs in Figures 21 and 23 were combined for comparison purposes, Figure 24. The tractor tire curve of 1971 had a smaller slope than the drum and tractor tire curve of 1970. Thus, the furrow compacted by the tractor tire only obtained a higher density.

When the average soil is moist or wet a higher degree of compaction can be achieved with less energy input. The 1971 crop was planted while the field was still wet from June rains. This relationship between soil moisture and compaction probably accounted for the greater compaction of the tractor tire middle.

From Figure 24, it was apparent that the application uniformity

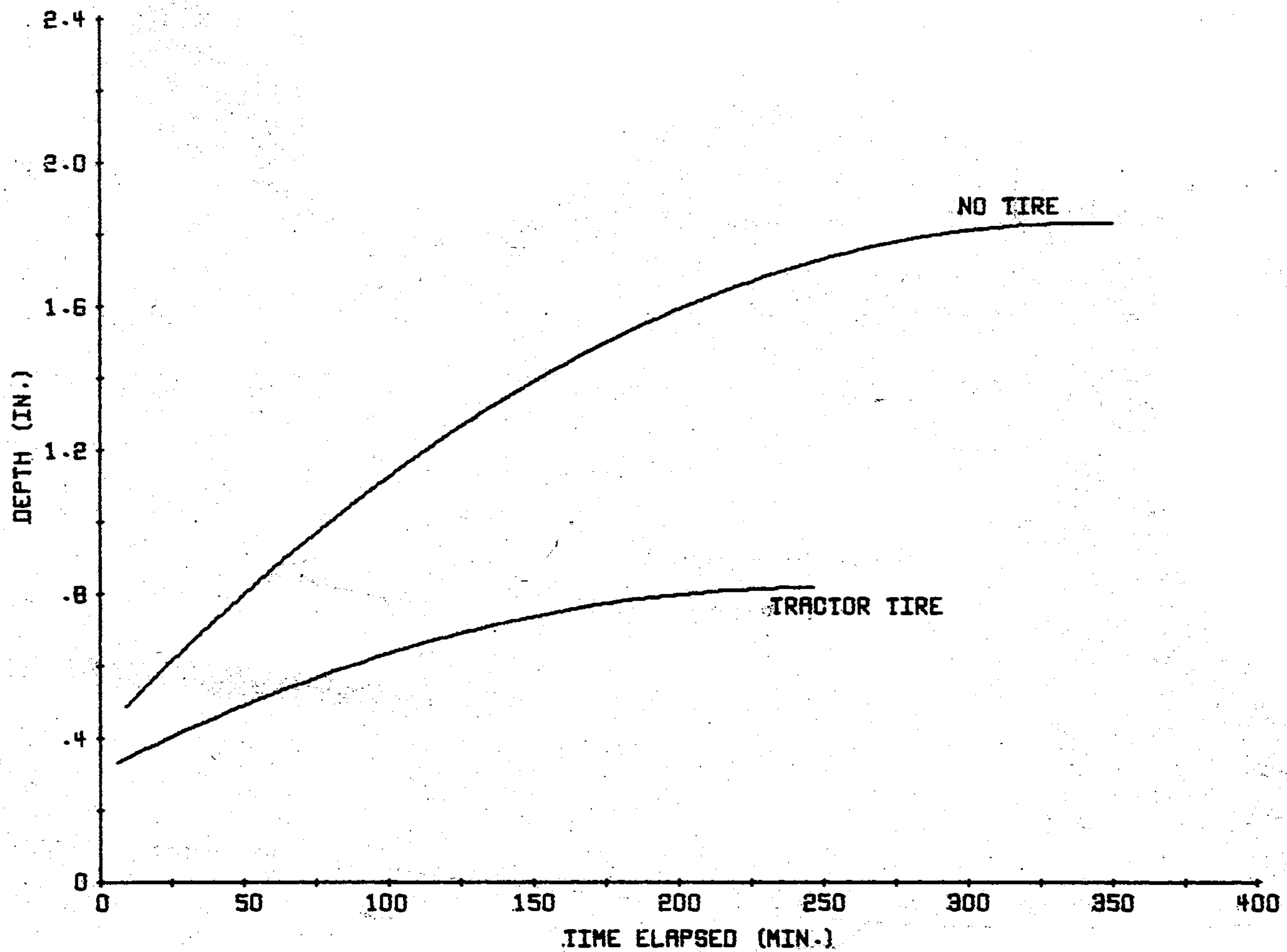

Figure 23. Depth Versus Time for Bay 1 During the Second Irrigation, Summer, 1971

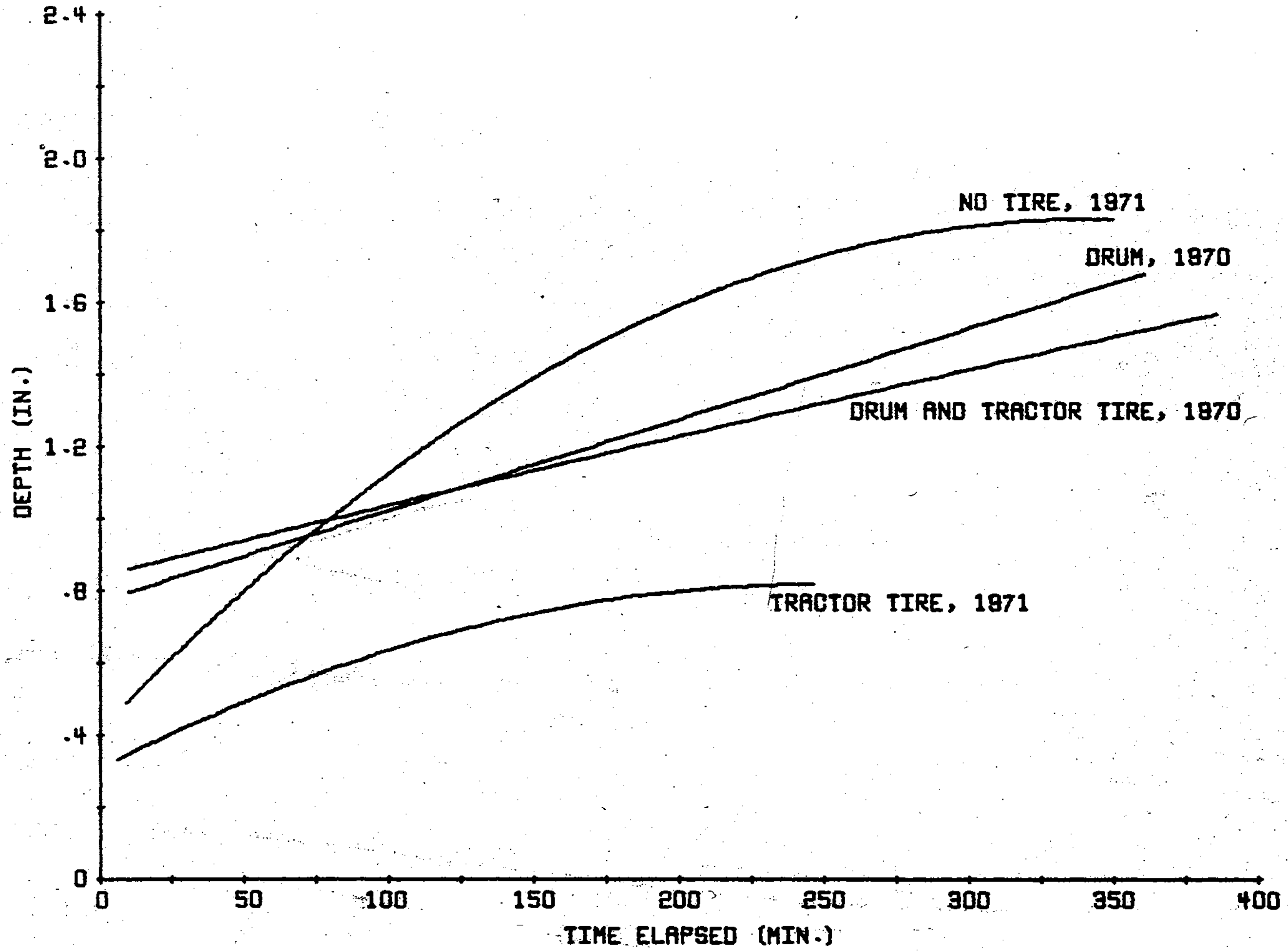

Figure 24. Depth Versus Time for Bay 1, Second Irrigation During the Summers of 1970 and 1971

was achieved by dragging drums behind the planter, which compensated for the tractor tire compaction.

The following equations were fitted to the depth-time data for each irrigation of 1971. Included with each equation is the irrigation number (IN), compaction effort (CE), range of applicability (minutes), correlation coefficient (r), and standard deviation (s). The two types of compactive efforts used were tractor tire (T) and no tire (NT).

IN	CE	Equation	Range	r	s
1	T	$d = 0.33 + 0.0033T - 0.000003T^2$	5-275	0.953	0.06
1	NT	$d = 0.34 + 0.0067T - 0.000005T^2$	5-305	0.925	0.19
2	T	$d = 0.28 + 0.0033T - 0.000003T^2$	4-250	0.961	0.04
2	NT	$d = 0.36 + 0.0078T - 0.000011T^2$	5-350	0.978	0.10
3	T	$d = 0.49 + 0.0039T - 0.000007T^2$	8-235	0.934	0.06
3	NT	$d = 0.51 + 0.0078T - 0.000013T^2$	9-240	0.992	0.05
4	T	$d = 0.60 - 0.0016T + 0.000019T^2$	9-155	0.972	0.02
4	NT	$d = 0.59 + 0.0012T + 0.000015T^2$	9-170	0.996	0.03

The above equations were plotted in Figure 25 to determine if any changes in the depth-time relationship occurred as the season progressed. There were no consistent changes between irrigations.

Several uncontrollable factors affected the depth versus time functions. Two of these varied between irrigations, temperature and surface moisture. The researcher felt that these factors accounted for the inconsistency shown in Figure 25.

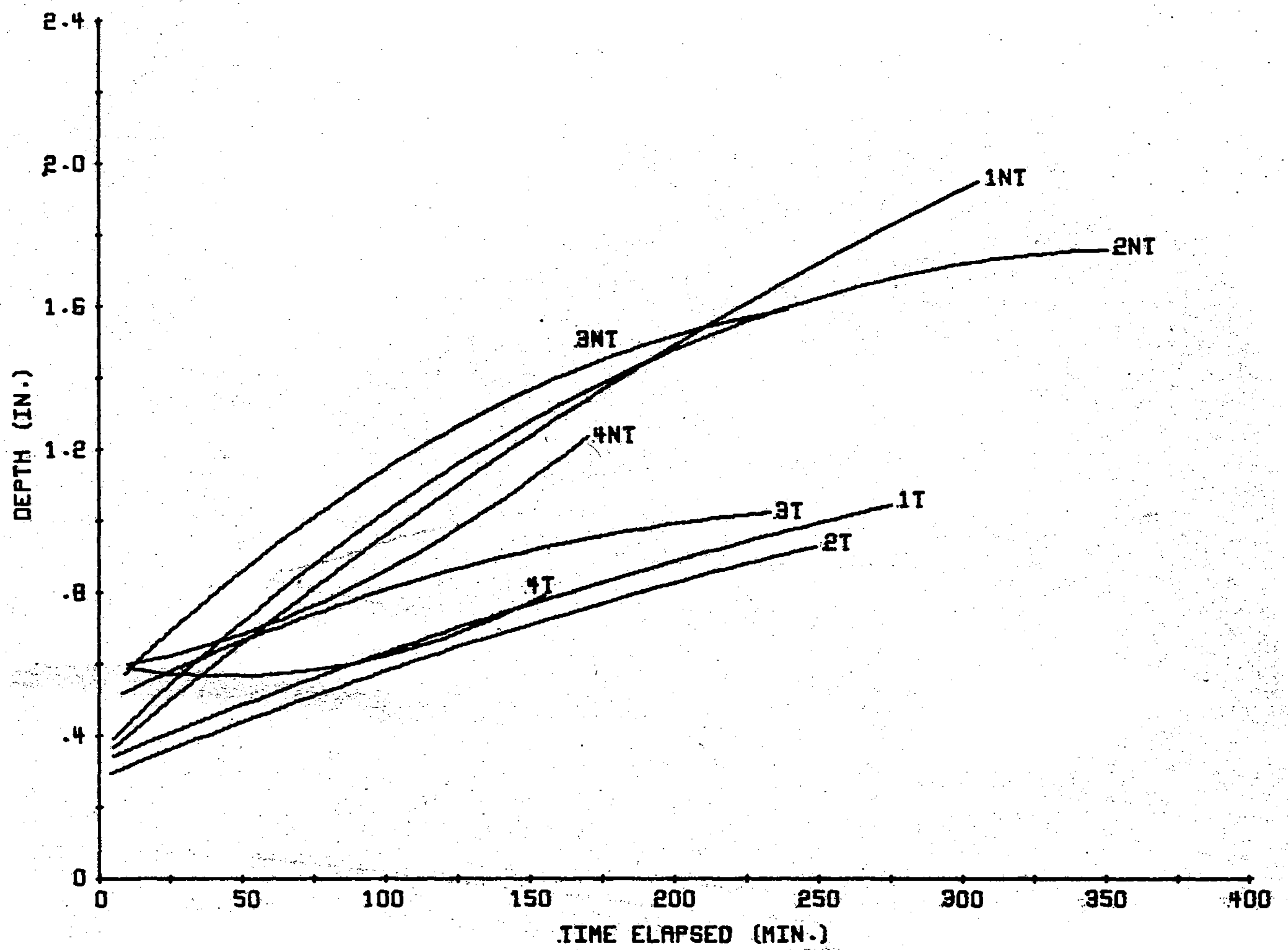

Figure 25. Depth Versus Time for the 1971 Irrigations. The Numbers Indicate First, Second, Third, and Fourth Irrigations. The Symbol (T) Represents Tractor Tire Furrow While NT Represents No Tire Furrow

Water Surface Profiles

Water surface profiles were measured during each irrigation. These were two-bay tests for spatially varied flow. Figures 26, 27, 28, and 29 are plots of observed profiles for the first, second, third and fourth irrigations, respectively. The calculated profiles are also plotted in each figure.

Rising profiles were observed in most of the downstream bays which were at initial flow. This rise was attributed to velocity head recovery exceeding friction loss which resulted from kinetic energy being converted to potential energy.

During each irrigation, falling profiles occurred in bays 3 and 4 while at initial flow. It was assumed that this was caused by excessive weathering of channel sections in these two bays. The bottom roughness of the sections identified was increased by the weathering, causing the friction loss to increase.

In all the irrigations, falling profiles were observed in the upstream bay which was at cut-back discharge. This drop in surface elevation was due to friction losses exceeding velocity head recovery. The increase in friction loss was caused by the increased velocity associated with cut-back discharge.

Equation (2-17) was used to calculate the water surface profiles. The method proposed by Sweeten and Garton (30) in similar research was followed. An apparent or virtual length (L_1') was used in solving for the upstream profiles. L_1' was the length of bay necessary to completely discharge the inflow (Q) from the upstream bay.

An average value of $\bar{n} = 0.0126$ was used to calculate water surface elevations in initial flow bays. In the cut-back flow bays a value of

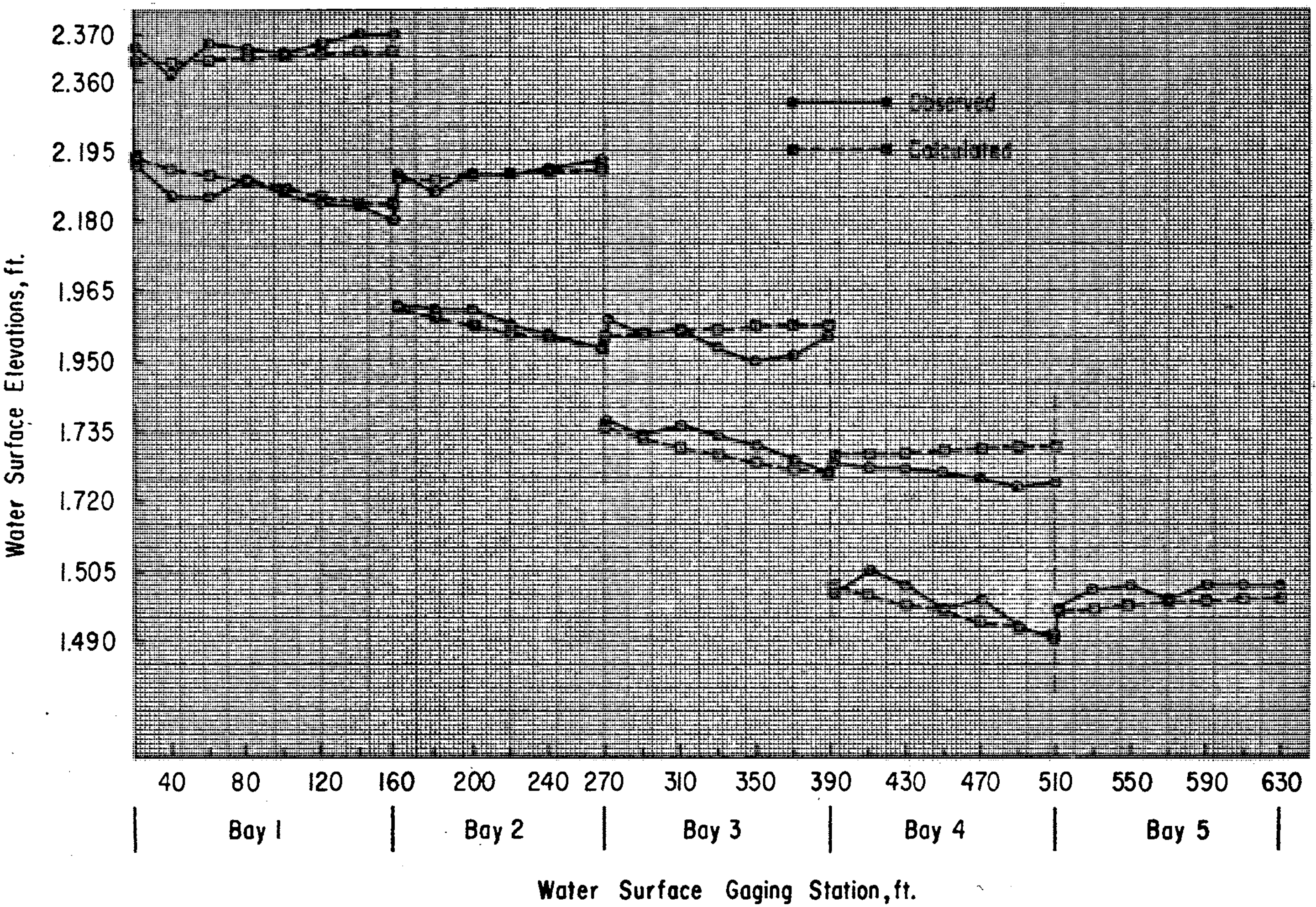

Figure 26. Observed and Calculated Water Surface Profiles for the First Irrigation

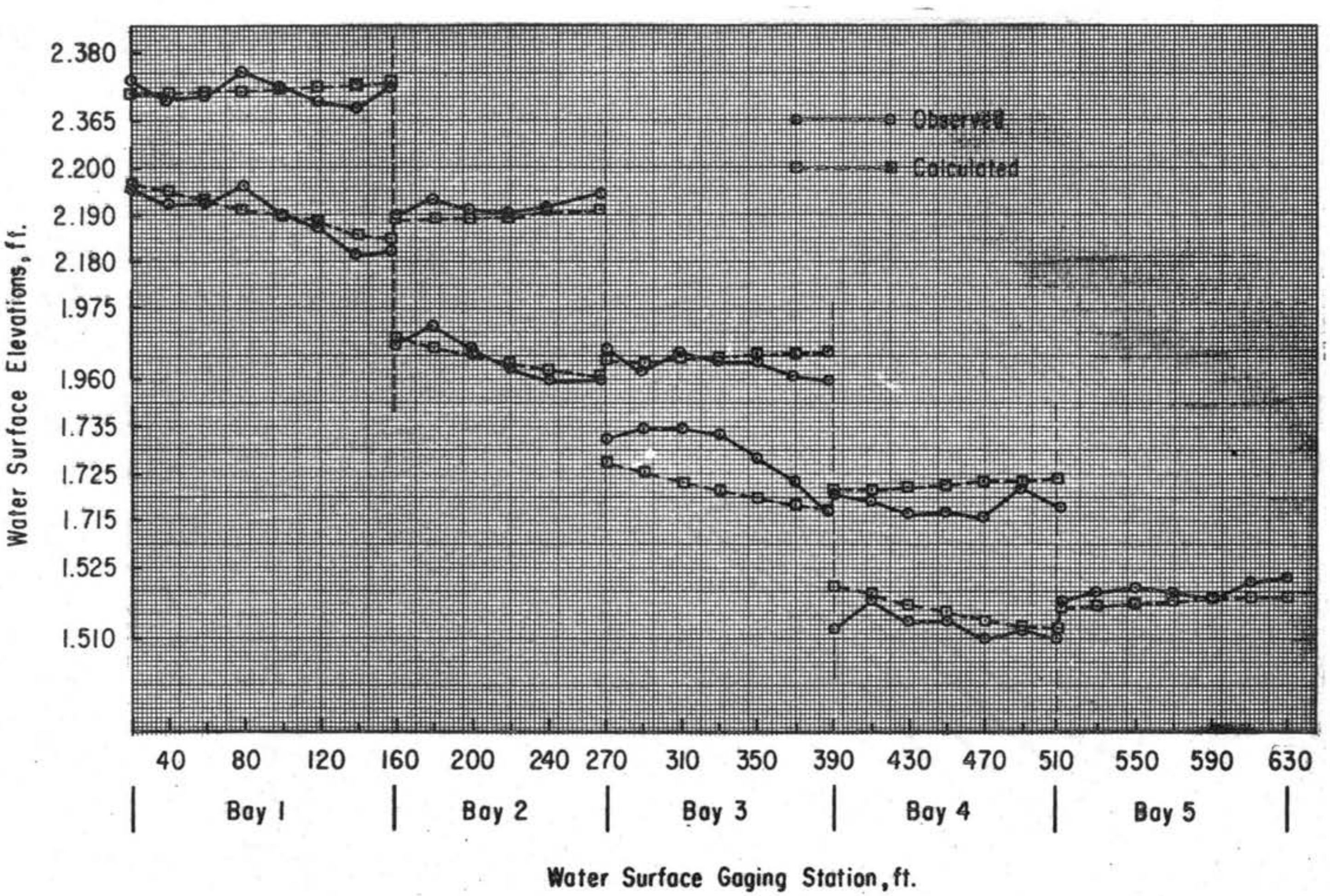

Figure 27. Observed and Calculated Water Surface Profiles for Second Irrigation

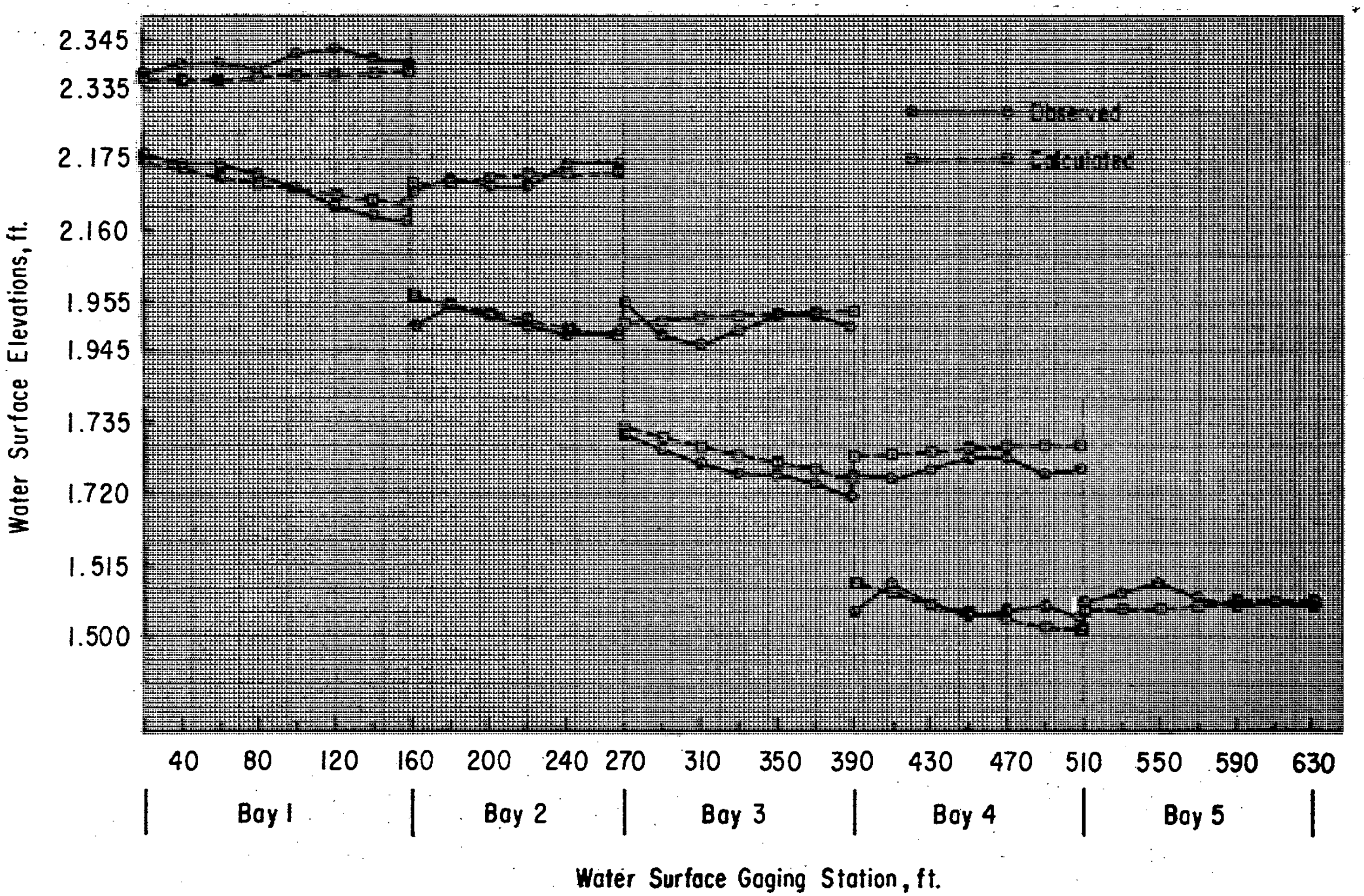

Figure 28. Observed and Calculated Water Surface Profiles for Third Irrigation

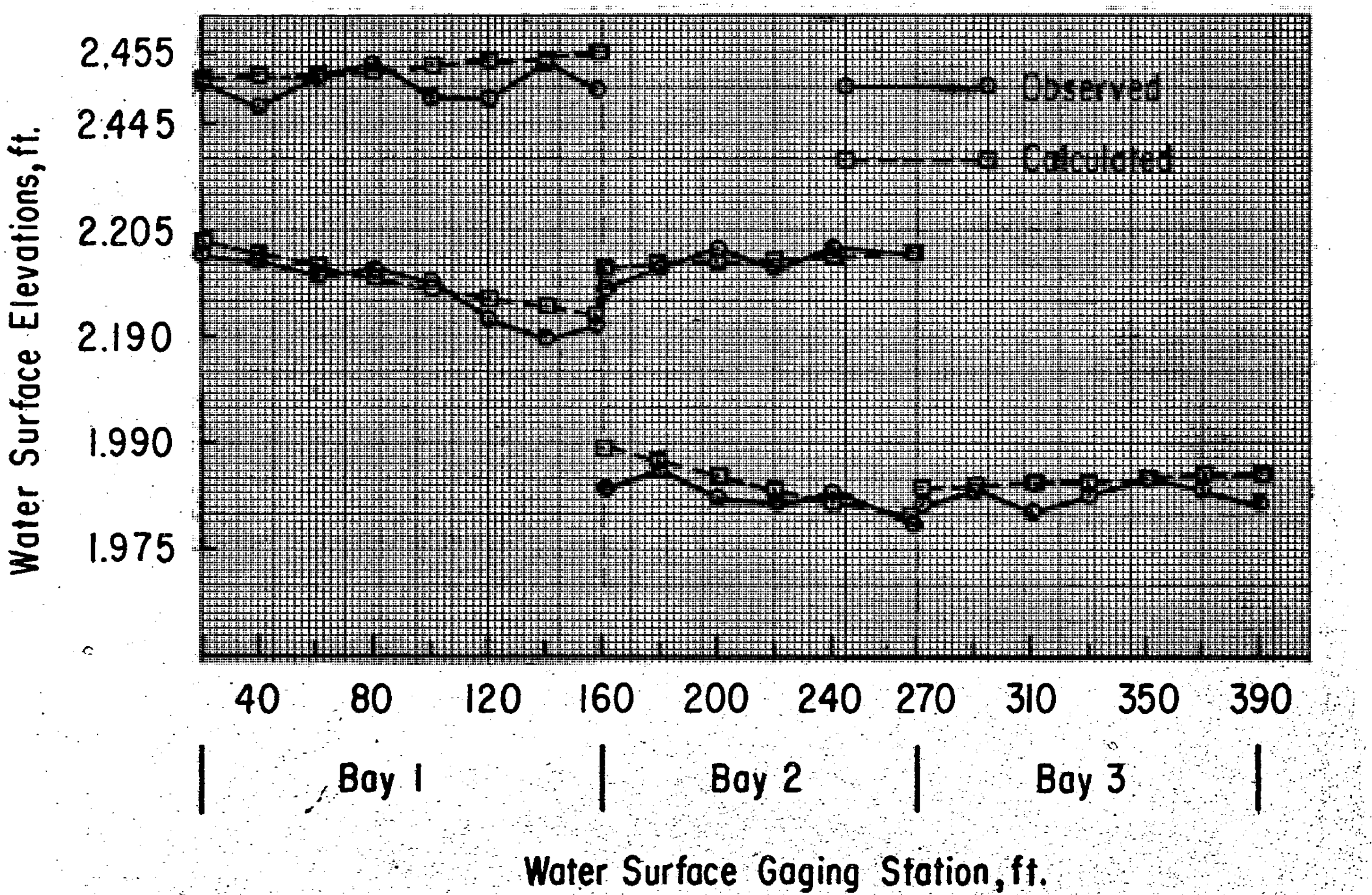

Figure 29. Observed and Calculated Water Surface Profiles for Fourth Irrigation

$\bar{n} = 0.01065$ was used.

Variations in Initial and Cut-Back Discharges

Both weir and orifice discharge uniformities were determined during each irrigation. The second and third orifices of each ten-foot channel section were chosen for the tests. The point gage apparatus described in Chapter III was used to measure the head of water above the orifice bottom. Table XIII of Appendix C is a summary of the above tests. The mean ($\bar{X}$) and standard deviation (s) for each bay are also shown in this table.

In each bay, larger variations were observed for weir flow as opposed to orifice flow. The largest variation was observed as 4.2 g.p.m., while the largest standard deviation was 1.10 g.p.m.

Before the irrigation flume was installed, a blade was used to level a pathway for the stands. Trucks were driven up and down the path for compaction. Even with this preparation, the channel sections settled during each irrigation. This settlement could account for part of the discharge variation.

For the first three irrigations the observed initial and cut-back furrow discharges were below the corresponding design discharges. The cut-back discharges differed the greatest from the calculated values.

The fourth irrigation was the only irrigation where the actual supply flow exceeded the predicted supply flow. During this irrigation the average initial furrow flow was slightly below the design's initial discharge. On the other hand, the average cut-back furrow discharge was slightly above the designated cut-back flow.

The summer of 1971 was unusual in the panhandle, because the wind was never a dominate force. The lack of wind prohibited research concerning wind effect on water surface profiles.

CHAPTER VI

SUMMARY AND CONCLUSIONS

Summary

Rate of advance tests were conducted in a field in the Oklahoma Panhandle. Several different furrow flow rates were chosen for the advance tests. The results of the above tests were used to determine the hydraulic characteristics of the field's furrows.

The depth of water applied was calculated from the rate of advance data. From the plots made, it was determined that the depth versus time relationships were independent of furrow discharges between 10.0 g.p.m. and 19.0 g.p.m.

When the irrigation system was designed and installed in the field, an initial flow of 12.6 g.p.m. and a cut-back discharge of 7.8 g.p.m. were selected for the preliminary field tests.

Based on the information obtained and personal observation during preliminary field tests, the system was redesigned for final field tests. The desired initial and cut-back discharges were changed to 12.0 g.p.m. and 3.7 g.p.m., respectively.

During the preliminary and final field tests, initial cut-back furrow discharge uniformities were determined. In every case, larger variations were observed for cut-back flows as opposed to orifice flows. The greatest variation was observed as 4.2 g.p.m. The largest standard deviation was calculated to be 1.17 g.p.m.

The over prediction of supply flow available accounted for the differences in design and actual furrow discharges.

Depth versus time relationships were used to study furrow characteristics during irrigations. Application uniformity between furrows was increased by dragging weighted drums behind the planter. No consistent changes in furrow behavior occurred between irrigations.

Water surface profiles were plotted from the observations made during the final field tests. The profiles were calculated and plotted for each test.

Conclusions

1. This study suggests that rate of advance data provides a good basis for designing an irrigation system.

2. Design procedures used in this study work reasonably well. The main difficulty encountered was the variation in water supply flow from the well.

3. In this study, variations in the water supply flow have the greatest effect on the cut-back discharges.

4. Friction losses may increase from year to year due to weathering. The increase in roughness could cause an increase in discharge variations in the cut-back bay and a decrease in the initial flow bay. Part of the friction effect on discharge variations could be remedied by setting the elevation of channel sections parallel to the hydraulic grade line of each bay.

5. The present structural design and support system are not feasible for field operation. Too much time and labor are required to assemble and level the channel sections. Settlement occurs during

irrigations, necessitating the leveling of the system prior to each irrigation. This settlement could account for part of the discharge variation. A system of support posts is recommended.

6. The following comments are offered by the cooperating farmer, Mr. S. Perkins. The system could cover more ground with less water and obtain a more even furrow discharge rate. Excluding installation, this increase in efficiency could be done with less effort. The system is more reliable than setting gated pipe by hand. The idea is excellent and has promise, especially as the price of water, labor, and land rise.

Suggestions for Future Research

1. A study to find a more adequate method of assembling and supporting the system in the field should be conducted.

2. The effects of wind on water surface profiles would be a profitable study.

3. A more complete study of furrow characteristics during consecutive irrigations should be investigated.

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APPENDIX A

HS FLUME CALIBRATION DATA

TABLE V

HS FLUME CALIBRATION DATA

Size ft.	Head ft.	Discharge gpm	Size ft.	Head ft.	Discharge gpm
0.30	0.10	1.37	0.35	0.09	1.27
0.30	0.12	2.10	0.35	0.11	1.93
0.30	0.13	2.51	0.35	0.14	3.37
0.30	0.14	3.00	0.35	0.15	3.93
0.30	0.16	4.03	0.35	0.17	5.20
0.30	0.17	4.64	0.35	0.18	5.90
0.30	0.19	6.00	0.35	0.20	7.49
0.30	0.21	7.45	0.35	0.22	9.20
0.30	0.22	8.40	0.35	0.24	11.45
0.30	0.24	10.40	0.35	0.26	13.50
0.30	0.26	11.40	0.35	0.29	17.40
0.30	0.28	14.79	0.35	0.32	21.50
0.40	0.10	1.47			
0.40	0.12	2.65			
0.40	0.14	3.74			
0.40	0.16	4.90			
0.40	0.18	6.35			
0.40	0.23	11.00			
0.40	0.25	13.16			
0.40	0.28	16.80			
0.40	0.30	19.48			
0.40	0.33	23.95			
0.40	0.35	27.20			

APPENDIX B

PRELIMINARY FIELD TESTS

TABLE VI

INITIAL PRELIMINARY DESIGN

Bay No.	Q_I gpm	Q_C gpm	No. of Orifices	Orifices Cut-back	q_i gpm	q_c gpm	Distance ft.
1	670		54		12.4		180
		421		54		7.8	
2	249		18		13.8		240
		141		18		7.8	
3	529		42		12.6		380
		328		42		7.8	
4	342		27		12.7		470
		211		27		7.8	
5	459		36		12.8		590
		281		36		7.8	
6	389		30		13.0		690

TABLE VII

AVERAGE DROP BETWEEN BAYS FOR PRELIMINARY DESIGN

Bay No.	q_i gpm	h_i ft.	q_c gpm	h_c ft.	Difference ft.
1	12.4	0.400			
			7.8	0.193	0.292
2	13.8	0.485			
			7.8	0.193	0.217
3	12.6	0.410			
			7.8	0.193	0.222
4	12.7	0.415			
			7.8	0.193	0.227
5	12.8	0.420			
			7.8	0.193	0.237
6	13.0	0.430			
				Total drop =	1.195

Average drop $= \dfrac{1.195}{6 - 1} = 0.24$ ft.

TABLE VIII

FINAL PRELIMINARY DESIGN

Bay No.	Q_I gpm	Q_C gpm	No. of Orifices	Orifices Cut-Back	q_i gpm	q_c gpm	Distance ft.
1	670	405	54	54	12.4	7.5	180
2	269	143	21	21	12.8	6.8	250
3	525	319	42	42	12.5	7.6	390
4	348	212	27	27	12.9	7.5	480
5	460	252	36	36	12.8	7.0	600
6	416		33		12.6		710

TABLE IX

CHECK OF FINAL PRELIMINARY DESIGN

Bay No.	q_i gpm	h_i ft.	q_c gpm	h_c ft.	Difference ft.	Twice Difference
1	12.4	0.400	7.5	0.180		
					0.240	0.480
2	12.8	0.420	6.8	0.164		
					0.240	0.480
3	12.5	0.404	7.6	0.183		
					0.240	0.480
4	12.9	0.423	7.5	0.180		
					0.240	0.480
5	12.8	0.420	7.0	0.170		
					0.240	0.480
6	12.6	0.410				

TABLE X

SUMMARY OF INITIAL AND CUT-BACK DISCHARGES DURING PRELIMINARY FIELD TESTS

Second Irrigation

Bay No.	Initial $\bar{X}$ gpm	High q_i gpm	Low q_i gpm	Initial s gpm	Cut-back $\bar{X}$ gpm	High q_c gpm	Low q_c gpm	Cut-back s gpm
1	10.4	11.3	9.5	0.43				
					6.1	7.5	4.1	1.15
2	12.0	12.7	11.4	0.35				
					4.3	6.6	2.8	1.17
3	11.6	12.0	11.1	0.28				
					5.9	6.3	5.0	0.42
4	12.3	12.8	12.0	0.28				
					4.7	5.7	3.9	0.57
5	12.7	13.0	12.4	0.20				
					5.3	6.3	4.2	0.63
6	12.1	12.6	11.7	0.24				

Third Irrigation

Bay No.	Initial $\bar{X}$ gpm	High q_i gpm	Low q_i gpm	Initial s gpm	Cut-back $\bar{X}$ gpm	High q_c gpm	Low q_c gpm	Cut-back s gpm
1	11.0	11.2	10.4	0.20				
					6.1	6.6	5.4	0.33
2	12.5	12.6	12.3	0.21				
					3.5	5.0	1.8	0.95
3	12.0	12.6	11.2	0.49				
					5.7	6.8	4.2	0.82
4	12.5	12.7	12.2	0.18				
					5.1	5.6	4.6	0.34
5	12.2	12.4	11.9	0.14				
					4.5	5.0	3.8	0.39
6	11.9	12.3	11.4	0.31				

APPENDIX C

FINAL FIELD TESTS

TABLE XI

FINAL DESIGN

Bay No.	Q_I gpm	Q_C gpm	No. of Orifices	Orifices Cut-back	q_i gpm	q_c gpm	Distance ft.
1	575	178	48	48	12.0	3.7	160
2	396	135	33	33	12.0	4.1	270
3	436	133	36	36	12.1	3.7	390
4	432	133	36	36	12.0	3.7	510
5	432		36		12.0		630

TABLE XII

CHECK OF FINAL DESIGN

Bay No.	q_i gpm	h_i ft.	q_c gpm	h_c ft.	Difference ft.	Twice Difference
1	12.0	0.333	3.7	0.093	0.240	0.480
2	12.0	0.333	4.1	0.098	0.240	0.480
3	12.1	0.338	3.7	0.093	0.240	0.480
4	12.0	0.333	3.7	0.093	0.240	0.480
5	12.0	0.333				

TABLE XIII

SUMMARY OF INITIAL AND CUT-BACK DISCHARGES DURING FINAL FIELD TESTS

First Irrigation

Bay No.	Initial $\bar{X}$ gpm	High q_i gpm	Low q_i gpm	Initial s gpm	Cut-back $\bar{X}$ gpm	High q_c gpm	Low q_c gpm	Cut-back s gpm
1	10.4	10.7	9.9	0.40	2.7	3.8	1.7	0.65
2	11.7	12.3	11.3	0.29	3.2	5.6	1.4	1.10
3	11.2	11.7	10.9	0.17	2.7	4.7	1.5	0.76
4	11.2	11.5	11.0	0.14	2.8	3.3	2.0	0.38
5	11.1	12.8	10.9	0.51				

Average supply flow = 512 gpm

Second Irrigation

Bay No.	Initial $\bar{X}$ gpm	High q_i gpm	Low q_i gpm	Initial s gpm	Cut-back $\bar{X}$ gpm	High q_c gpm	Low q_c gpm	Cut-back s gpm
1	10.4	10.7	10.1	0.16	3.1	4.2	2.1	0.48
2	11.5	12.0	11.2	0.23	3.2	5.2	1.9	0.95
3	11.4	11.7	11.1	0.17	2.6	3.5	1.5	0.54
4	11.3	11.5	11.0	0.14	3.2	3.7	2.7	0.39
5	11.7	12.0	11.3	0.16				

Average supply flow = 519 gpm

TABLE XIII (CONTINUED)

Third Irrigation

Bay No.	Initial $\bar{X}$ gpm	High q_i gpm	Low q_i gpm	Initial s gpm	Cut-back $\bar{X}$ gpm	High q_c gpm	Low q_c gpm	Cut-back s gpm
1	9.6	9.9	9.2	0.19	2.7	3.9	1.6	0.59
2	11.3	11.7	10.9	0.22	2.6	3.9	1.7	0.62
3	10.9	11.4	10.7	0.18	2.2	4.0	1.4	0.58
4	11.5	11.7	11.2	0.14	2.3	2.6	1.8	0.22
5	11.4	11.8	11.1	0.17				

Average supply flow = 495 gpm

Fourth Irrigation

Bay No.	Initial $\bar{X}$ gpm	High q_i gpm	Low q_i gpm	Initial s gpm	Cut-back $\bar{X}$ gpm	High q_c gpm	Low q_c gpm	Cut-back s gpm
1	11.9	12.3	11.5	0.22	4.7	5.9	3.4	0.66
2	11.6	12.0	11.4	0.15	5.4	6.3	4.6	0.55
3	11.6	12.2	11.1	0.21	3.6	5.7	2.6	0.69
4	11.7	12.0	11.3	0.25	---	---	---	----
5	----	----	----	----				

Average supply flow = 597 gpm

VITA 2

John Henry Edwards

Candidate for the Degree of

Master of Science

Thesis: A FIELD STUDY OF AN AUTOMATIED IRRIGATION FLUME

Major Field: Agricultural Engineering

Biographical:

Personal Data: Born at Memphis, Tennessee, March 11, 1948, the son of Winifred G. and Jerome O. Edwards.

Education: Graduated from Sewanee Military Academy, Sewanee, Tennessee, in 1966; received a Bachelor of Science degree in Agriculture from Arkansas State University in 1970; completed requirements for the Master of Science degree in May, 1972.

Professional Experience: Graduate Research Assistant, School of Agricultural Engineering, Oklahoma State University, 1970-1971.

Professional and Honorary Organizations: Student Member, American Society of Agricultural Engineers; Engineer in Training, State of Oklahoma; Member, Phi Eta Sigma, Delta Tau Alpha, and Scabbard and Blade.