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THE HYDRAULICS OF SPATIALLY VARIED FLOW IN AN IRRIGATION DISTRIBUTION CHANNEL WITH FURROW OUTLETS

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ABSTRACT

THE HYDRAULICS OF SPATIALLY VARIED FLOW IN AN IRRIGATION DISTRIBUTION CHANNEL WITH FURROW OUTLETS

This project consisted of three phases,

I. The hydraulics of a concrete lined trapezoidal channel with rectangular side weir outlets

II. The hydraulics of a semi-portable sheet metal flume with circular outlets, and

III. Field tests of a semi-portable sheet metal flume with circular outlets.

For phase I, discharge relationships were determined for different sizes of rectangular weirs with 45° slope. The channel hydraulic properties were defined and a method derived for directly computing water surface profiles. The variations in individual furrow discharge resulting from various flow, depths, weir spacing and size were determined.

For phase II, a galvanized sheet metal flume was designed for structural stability and hydraulic efficiency. This flume was tested in the outdoor Hydraulic Laboratory to determine the Manning's roughness coefficients for spatially varied flow. Tests were run on circular orifices with full flow and with weir flow. Uniformity of discharge was determined for a two bay system.

For phase III, the sheet metal flume design was revised, 710 feet of channel was built, and two years of field tests were conducted to determine its performance under field conditions. The stability tests of the system indicate a need for improved supports.

Keywords: spatially varied flow/ hydraulics/ automation of irrigation/ Mannings Equation / uniformity of irrigation

This research project produced two M.S. theses, a Ph.D. dissertation, and three technical papers. One of the papers has been published, one presented at a national meeting of ASAE and submitted for publication, and a manuscript has been prepared for the third.

The objectives as outlined in the original proposal have been achieved. The level of knowledge of the hydraulics of spatially varied flow has been advanced. Information is now available for the relationship between flow rate and head for sloping rectangular weirs. The discharge relationship for small circular weirs has also been established. The extent of variation in furrow discharge and its causes have been determined.

The objectives for the three different phases of the project are listed with the report for that particular phase.

Theses titles:

John M. Sweeten, Jr., Hydraulics of a Side Weir Irrigation System. Unpublished Ph.D. dissertation, Agri. Engr. Dept., Oklahoma State University, May, 1969.

Uhl, Vincent W., Jr. A Semi-Portable Sheet Metal Flume for Automated Irrigation. Unpublished M.S. thesis, Agri. Engr. Dept., Oklahoma State University, May, 1970.

Edwards, John H. A Field Study of an Automated Irrigation Flume. Unpublished M.S. thesis, Agri. Engr. Dept., Oklahoma State University, May, 1972.

Paper titles:

Sweeten John M., Jr. and James E. Garton. The Hydraulics of an Automated Furrow Irrigation System with Rectangular Side Weir Outlets. Transactions of the ASAE. 13(6) pp. 746-751, 1970.

Uhl, Vincent W., Jr. and James E. Garton. A Semi-Portable Sheet Metal Flume for Automated Irrigation. (In Press) Transactions of the ASAE.

Edwards, John, H. and James E. Garton. A Field Study of an Automated Irrigation Flume. (manuscript prepared)

Report on Phase I of the Project

THE HYDRAULICS OF AN AUTOMATED FURROW IRRIGATION SYSTEM WITH RECTANGULAR SIDE WEIR OUTLETS*

by

John M. Sweeten and James E. Garton

INTRODUCTION

Conventional surface irrigation systems are characterized by high labor requirements and excessive water wastes. Non-uniform water application generates a substantial portion of the irrigation water wastes by causing increased deep percolation and runoff.

Automatic surface irrigation systems have been devised (3) (9) (11) (13) to reduce labor requirements through automatic regulation and diversion of the supply flow. However, attention has not been directed toward improved distribution uniformities with some of these systems. One exception was an automatic cut-back irrigation system conceived by Garton (7) (9) in which level hooded inlet tubes were installed along a concrete channel constructed as a series of level distribution bays. The tubes were placed at the same elevation in each bay to provide uniform discharge. A similar system using circular orifices as the furrow outlet devices was proposed by Barefoot and Garton (1)

Peter (17) discussed an irrigation system comprised of a concrete channel containing multiple side outlets with all the inflow discharged through the outlets. The outlet dimensions were individually adjusted in an attempt to provide uniform water application from the channel. Peter (12) later performed hydraulic experiments using a model of a terminal side outlet.

Fitzgerald and Lauder (5) studied the energy losses in an earthen irrigation distribution ditch which supplied eight 40 ft

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wide border strips using long concrete side weirs. Reducing the channel roughness by vegetation clearance enhanced the distribution uniformity.

Side Weir Irrigation System

A side weir irrigation system consisting of weirs installed in the side of a temporary or a permanent channel could conceivably offer certain advantages over most existing surface irrigation systems. These advantages include (1) self-priming discharge, (2) simplicity of construction, (3) precise metering of discharge into furrows or border strips, (4) adaptability into completely automated surface irrigation systems, and (5) low operating heads and hence shallower, less costly channels. When designed as a series of level distribution bays, side weir irrigation systems would be adaptable to flatter land slopes than would automatic cutback irrigation systems using either level tubes or orifices.

Objectives

Hydraulic experiments were designed and conducted (14) to obtain information which could be used in the design of automatic surface irrigation systems utilizing side weirs. The objectives of this research project were:

1. To determine the discharge characteristics of rectangular side weirs installed in a trapezoidal channel with 1:1 side slopes.

2. To determine the hydraulic roughness of a concrete-lined irrigation channel with gradually varied flow and with decreasing spatially varied flow through rectangular side weirs.

3. To develop procedures for predicting water surface profiles in an irrigation channel with side weirs installed at the same crest elevation, and with a drop in crest elevation between the upstream and downstream distribution bays.

4. To determine the effects of weir crest length, weir spacing, head, and drop in weir crest elevation between bays on the distribution uniformity of a side weir irrigation system.

Theory

Energy Principle for Spatially Varied Flow

Side weir discharge is a form of decreasing spatially varied flow (SVF) in which the diversion of water over the weir crest does not affect the total energy in the channel (4) (22). The water surface profile in a channel reach containing a side weir can be calculated using the Bernoulli energy equation, the continuity principle, a side weir discharge equation, and a uniform flow formula for expressing the energy losses (4) (6) (17) (22).

For decreasing spatially varied flow from short side weirs spaced along an irrigation bay of small bottom slope, assuming an energy coefficient α of unity, the Bernoulli equation can be written for downstream and upstream stations 1 and 2, respectively, as:

$$z_2 + y_2 + \frac{Q_2{}^2}{2gA_2{}^2} = z_1 + y_1 + \frac{Q_1{}^2}{2gA_1{}^2} + S_f\ \Delta x \qquad (1)$$

where

z = channel bottom elevation, ft

y = flow depth in a vertical plane, ft

Q = channel discharge, ft^3/sec

g = gravitational acceleration, ft/sec^2

A = cross sectional area, ft^2

S_f = energy gradient, ft/ft

Δx = longitudinal distance between stations 1 and 2, ft

For sufficiently short channel reaches, the energy gradient can be computed from the Manning formula written as (6) (17) (22):

$$S_f = \frac{n^2\ (Q_1 + Q_2)^2}{2.208\ R_a{}^{4/3}\ (A_1 + A_2)^2} \qquad (2)$$

in which

$R_a = (R_1 + R_2)/2$ = average hydraulic radius, ft

n = Manning's roughness coefficient, $ft^{1/6}$

When Q_1 is known or equal to zero at the downstream end of the irrigation bay, the upstream channel discharge will be (6) (22):

$$Q_2 = Q_1 + Q_w \tag{3}$$

where

$Q_w = C\, L_w^{\beta_1}\, h^{\beta_2}$ = side weir discharge, ft^3/sec

C = side weir discharge coefficient

L_w = weir crest length, ft

β_1, β_2 = empirical exponents

and the average head h above the weir crest is given by (5):

$$h = \tfrac{1}{2}\left[(y_1 + z_1) + (y_2 + z_2)\right] - P \tag{4}$$

in which P is the vertical height of the weir crest above the channel bottom.

The water surface profile along a channel with side weirs can be calculated by applying equations (1), (2), (3), and (4) using successive approximations for y_2 (and hence h, Q_w, Q_2, R_a, A_2, and S_f) until equation (1) is balanced. Continuation of this procedure across consecutive weir outlets, progressing in an upstream manner, will yield the flow profile for the entire distribution bay. The value of Manning's n, the side weir discharge relationship, and the geometric elements of the channel cross section must be known.

Roughness Coefficients

Values of Manning's n can be obtained from decreasing SVF experiments by first computing the water surface profile from equations (1) through (4) using an assumed value of n, and then incrementing n until the calculated profile matches the observed profile at the end points. The value of n finally obtained has been designated $\overline{n}$ (14) (15).

An effective roughness coefficient n_e has been defined (14) (15) from the change in total energy head between the ends of the irrigation bay of length L. When the channel inflow Q is completely discharged through the side weirs, the channel velocity is zero at the downstream end of the irrigation bay, so that solving equation (1) for the effective energy slope S_{fe} yields:

$$S_{fe} = \frac{1}{L}\left(\frac{V_i^2}{2g} + y_i + z_i - y_o - z_o\right) \tag{5}$$

where the subscripts i and o refer to the upstream and downstream ends of the irrigation bay. From Manning's equation, the effective roughness coefficient n_e is then (14) (15):

$$n_e = 1.486 \frac{R_i^{2/3}}{V_i} S_{fe}^{1/2} \tag{6}$$

The following relationship between the roughness coefficients $\bar{n}$ and n_e was derived (19) (20):

$$\bar{n} = \sqrt{3}\, n_e \tag{7}$$

This equation is valid for a single distribution bay.

Direct Solution to SVF Profiles

In a previous study, Sweeten (19) (20) derived a direct solution to the water surface profiles in a level irrigation bay from the energy principle using the following assumptions, which also apply to equation (7):

1. A horizontal, prismatic channel with a constant depth.
2. A linear decrease in channel velocity to V = 0 at the downstream end of the distribution bay.

The change in water surface elevation ΔZ_x between the upstream cross section at x = 0 and a downstream location at x = X was expressed as (19) (20):

$$\Delta Z_x = \Delta h_v - h_f \tag{8}$$

The velocity head recovery Δh_v is:

$$\Delta h_v = \frac{V_i^2}{2g} - \frac{V_x^2}{2g} \tag{9}$$

and the friction losses h_f between the two stations are:

$$h_f = \frac{\bar{n}^2}{2.208\, R^{4/3}} \int_0^X V_x^2 \, dx \tag{10}$$

in which R is the mean (constant) hydraulic radius. The velocity V_x can be expressed as:

$$V_x = V_i \left(1 - \frac{x}{L}\right) \tag{11}$$

Substituting equations (9) and (10) into equation (8) and evaluating the terms containg V_x using equation (11) finally produced (19) (20):

$$\Delta Z_x = \frac{V_i^2}{2g}\left(\frac{2X}{L} - \frac{X^2}{L^2}\right) - \frac{\bar{n}^2 V_i^2}{2.208\ R^{4/3}}\left(X - \frac{X^2}{L} + \frac{X^3}{3L^2}\right) \quad (12)$$

Equation (12) provides a direct solution to the water surface profiles along a horizontal irrigation bay and avoids the use of the Bernoulli equation which must be solved by iteration for this application.

The geometric ratios in parentheses in equation (12) can be replaced by the constants C_1 and C_2 for given values of X/L so that equation (12) reduces to:

$$\Delta Z_x = \frac{V_i^2}{2g}(C_1) - \left(\frac{1}{3}\right)\frac{\bar{n}^2 V_i^2 L}{(2.208)\ R^{4/3}}(C_2) \quad (13)$$

Values of C_1 and C_2 are plotted as functions of X/L in Figures 1 and 2. The constant C_1 is the proportion of the upstream velocity head recovered from x = 0 to x = X, while C_2 is the relative amount of friction loss sustained in the distant X. The fraction 1/3 in the second term of equation (13) occurs because the energy loss h_f predicted with equations (10) and (11) for spatially varied flow has one-third the magnitude of h_f for uniform or gradually varied flow, assuming the same roughness, depth, and upstream channel velocity.

Water surface profiles in a system involving two distribution bays, as shown in Figure 3, can be calculated using equation (12). For a uniform side weir discharge Q_{w1} in the upstream bay, the virtual channel length L_1' necessary to completely discharge the inflow Q is:

$$L_1' = L_S \frac{Q}{Q_{w1}} \quad (14)$$

where L_S is the side weir spacing. By substituting $L = L_1'$ in equation (12), the water surface profile for the upstream bay can be calculated to the intersection of the two bays at $x = L_1$. Based on this water surface elevation, the profile for the downstream bay $L_1 \leq x \leq (L_1 + L_2)$ can be calculated from equation (12) with $L = L_2$, $X = x - L_1$, and with the velocity V_m at the intersection of the bays substituted for V_i. If a change in area occurs between the bays, the water surface elevation downstream from the change should reflect the alteration in velocity head.

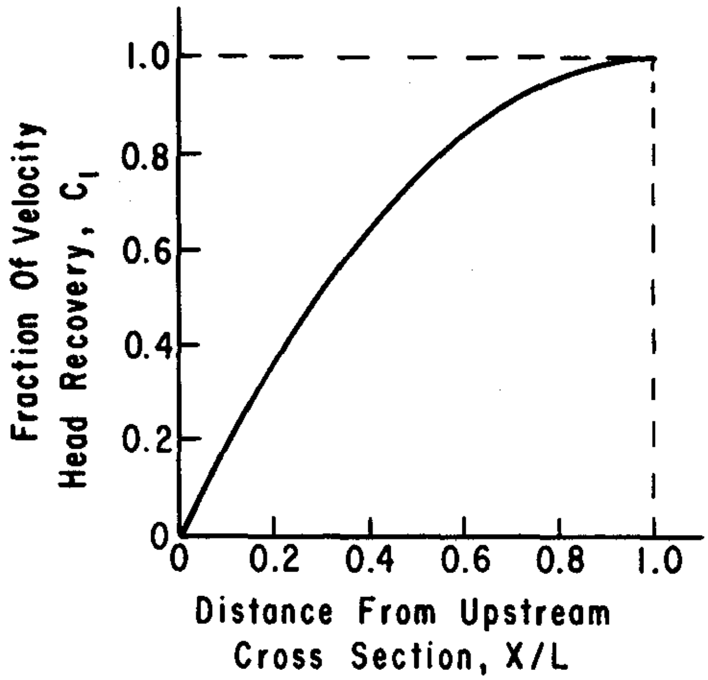

Figure 1. Proportion of the Initial Velocity Head Recovered Between x = 0 and x = X.

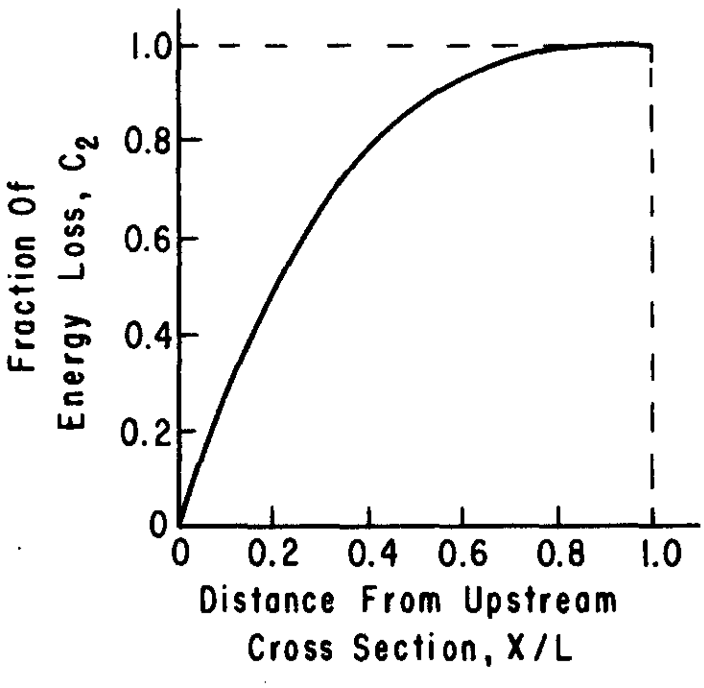

Figure 2. Proportion of Energy Losses Sustained Along an Irrigation Bay Between x = 0 and x = X.

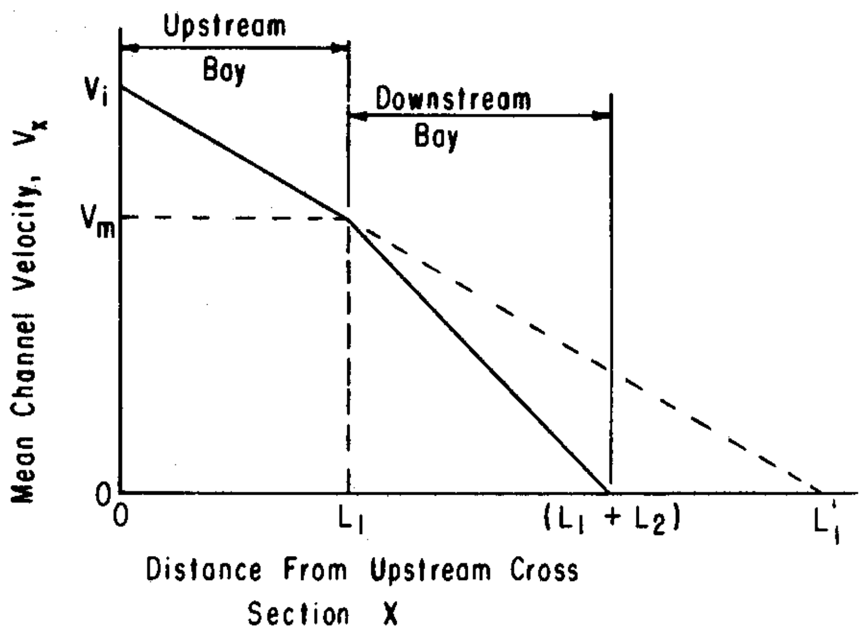

Figure 3. Assumed Decrease in Channel Velocity in a Two Bay Irrigation System.

Side Weir Irrigation Experiments

The decreasing spatially varied steady flow experiments were conducted in a horizontal concrete-lined irrigation channel to determine the hydraulic roughness of the channel with discharging side weir outlets and to evaluate the variations in side weir discharge that occur along an irrigation bay. The variables for these experiments were the weir crest length (2, 4, 6, 8, and 12 in.), weir spacing (40-320 in.), drop in crest elevation between irrigation bays (0.00, .025, .050, and .100 ft) and channel inflow rate (0.80 - 4.10 cfs). Conditions of steady flow were established for each test, with the entire channel inflow discharged through the side weir outlets. Some gradually varied steady flow experiments were also performed to determine the hydraulic roughness of the distribution bay with sheet metal plates covering the weir outlets.

Apparatus

The 320 ft long prototype concrete-lined irrigation channel shown in Figure 4 had a trapezoidal cross section with a nominal depth of 2.0 ft, a bottom width of 1.0 ft, and side slopes of 1:1. The 120 ft test section had an adverse bottom slope of 0.00011 ft/ft.

The channel inflow rate was measured with orifice plates inserted in the 12 inch diameter pipe shown in Figure 5. The differential head was measured with an air-water manometer. Eleven gauge wells, equipped with point gauges readable to ±0.001 ft for measuring the channel water surface profiles, were placed 30 ft apart at stations 0+00 through 3+00.

The test section of the concrete channel was located between stations 1+20 and 2+40 and contained 36 side outlets at centerline spacings of 40 inches. The side weirs were installed over the outlets at modular spacings of 40, 80, 160, and 320 inches.

A stage recorder mounted on the gauge well at station 0+00 allowed visual determinations of inflow-outflow equilibrium and of water surface instability caused by wind effects. The influence of wind was minimized during experimentation by the installation of wind panels over the channel.

Side Weirs

The side weir crest elevations were vertically adjusted to attain either one or two bays of discharging side weirs as desired. The weirs in a given bay were set at the same crest elevation within a reading tolerance of ±0.001 ft as determined by surveying using a self-leveling level and a point gauge. For

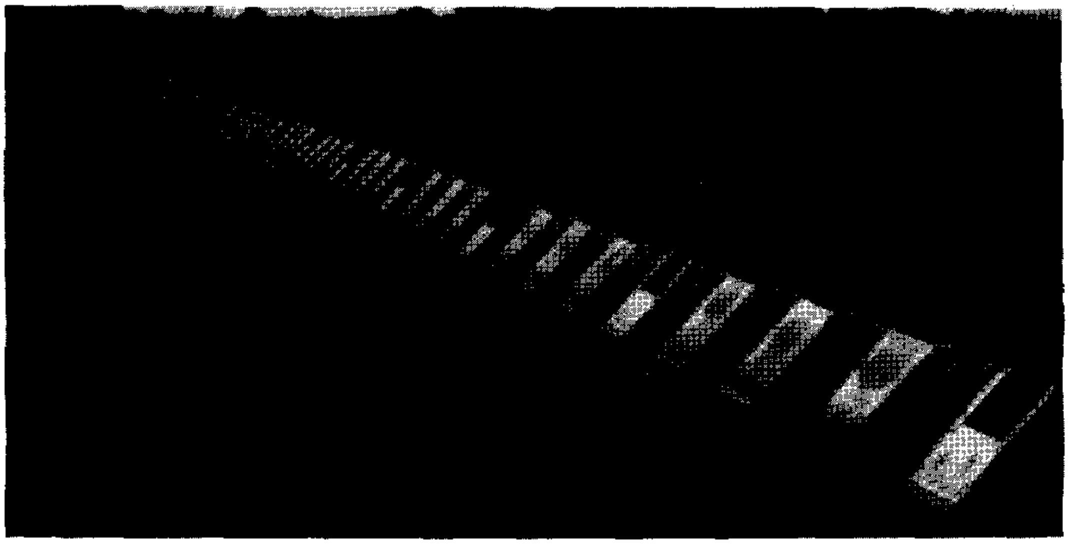

Figure 4. Concrete-Lined Irrigation Channel of Trapezoidal Cross Section With 12 inch Side Weirs Placed at 160 inch Spacing. Water Surface Elevations Were Measured in the Gauge Wells.

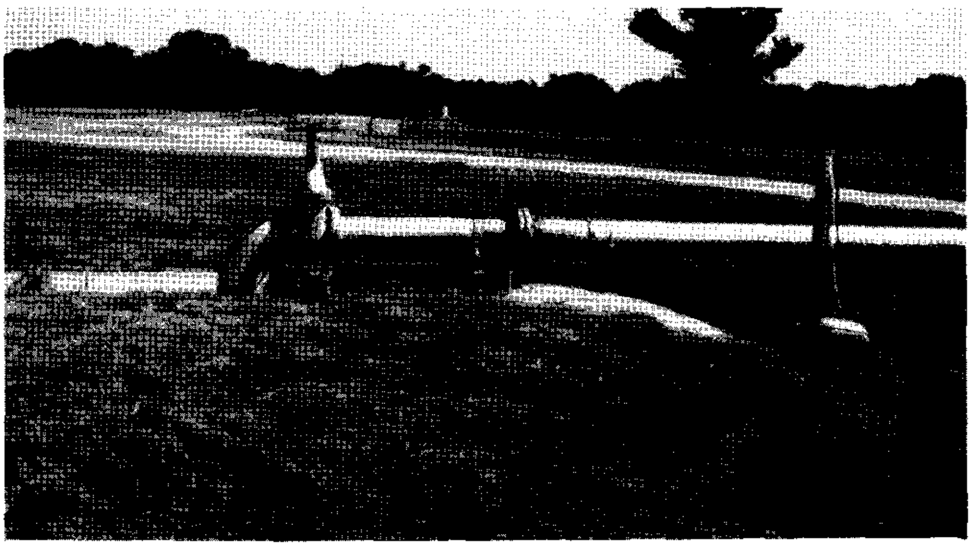

Figure 5. The Channel Inflow System With the 12 inch Supply Pipe, Control Valve, Orifice Flanges, and Manometer.

the 40 inch spacing experiments involving two distribution bays, the drop in weir crest elevation occurred at station 1+80. The approximate crest heights ranged from 1.28 to 1.38 ft.

Three calibrated 0.75 ft H flumes were spaced at locations near the upstream, middle, and downstream ends of the distribution bay to measure the diverted discharges from the side weirs whose centerlines lay at stations 1+31.7, 1+78.3, 1+85.0, and 2+38.3 ft. The sheet metal ductwork at the middle H flume was specially designed to permit measurement of discharge from the weir at station 1+85.0 as well as simultaneous measurement of the weir discharges at stations 1+78.3 and 1+85.0 in the upstream and downstream bays, respectively. Typical H flume installations are shown in Figures 6 and 7.

Testing Procedure

Twenty-seven gradually varied flow (GVF) experiments were conducted with rectangular plates covering the side outlets. The bottom edges of the 18 gauge sheet metal plates lay about 7 3/4 inches vertically above the channel bottom. The channel discharges ranged from 1.60 to 4.35 cfs while the average depths varied from 0.823 to 1.868 ft.

A total of 155 spatially varied flow (SVF) experiments were performed. The side weirs were installed at the proper spacing and crest elevation. The point gauge zero elevations for the H flume and channel gauge wells, along with the H flume lip elevations, were determined. Inflow to the channel was initiated through the control valve, and the desired manometer reading was carefully approximated. All the side weirs were inspected for the undesired conditions of submerged or clinging nappes. When the water surface in the channel had stabilized, five point gauge readings were taken in the channel gauge wells and in the H flumes. For the cutback irrigation experiments, two separate head measurements were required at the middle H flume. Finally, the channel inflow was determined from the drop in piezometric pressure across the orifice plates, as indicated on the water-air manometer and from a water temperature reading.

Results of the Gradually Varied Flow Experiments

Values of Manning's n were determined from the gradually varied flow experiments. The Bernoulli equation, equation (1), was solved for the energy gradient S_f between the ends of the 120 ft test section in which $Q_1 = Q_2 = Q$. Manning's n was then calculated from:

$$n = 1.486\ AR^{2/3}\ S_f^{1/2}/Q \qquad (15)$$

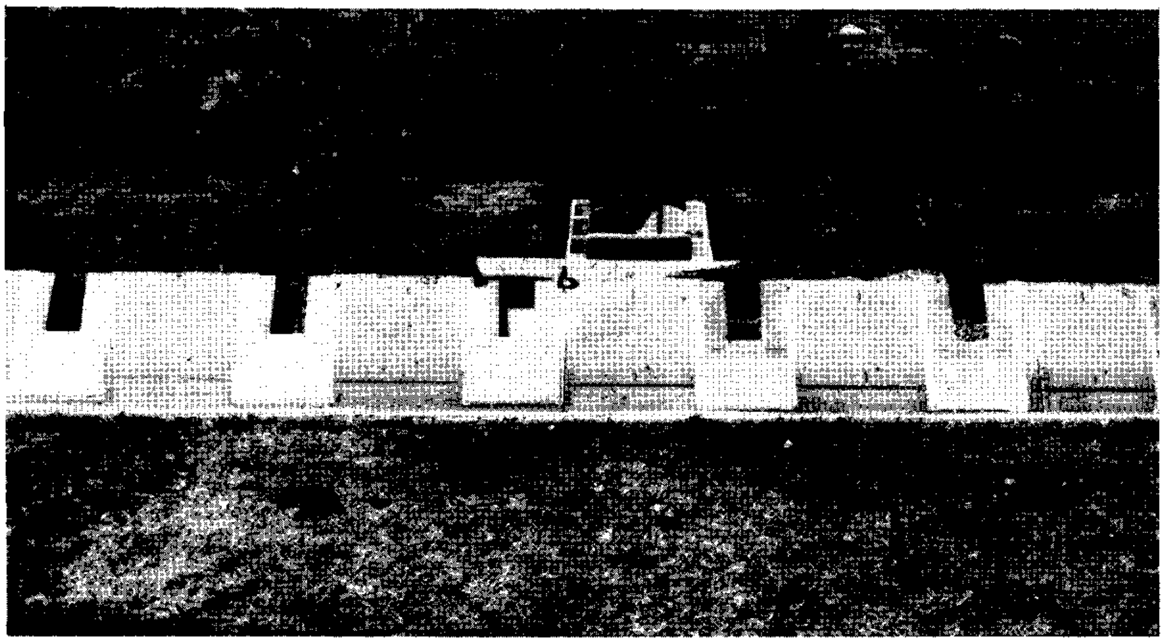

Figure 6. Side Weirs With 6 inch Crest Lengths Placed at 40 inch Spacings, with an H Flume in the Background.

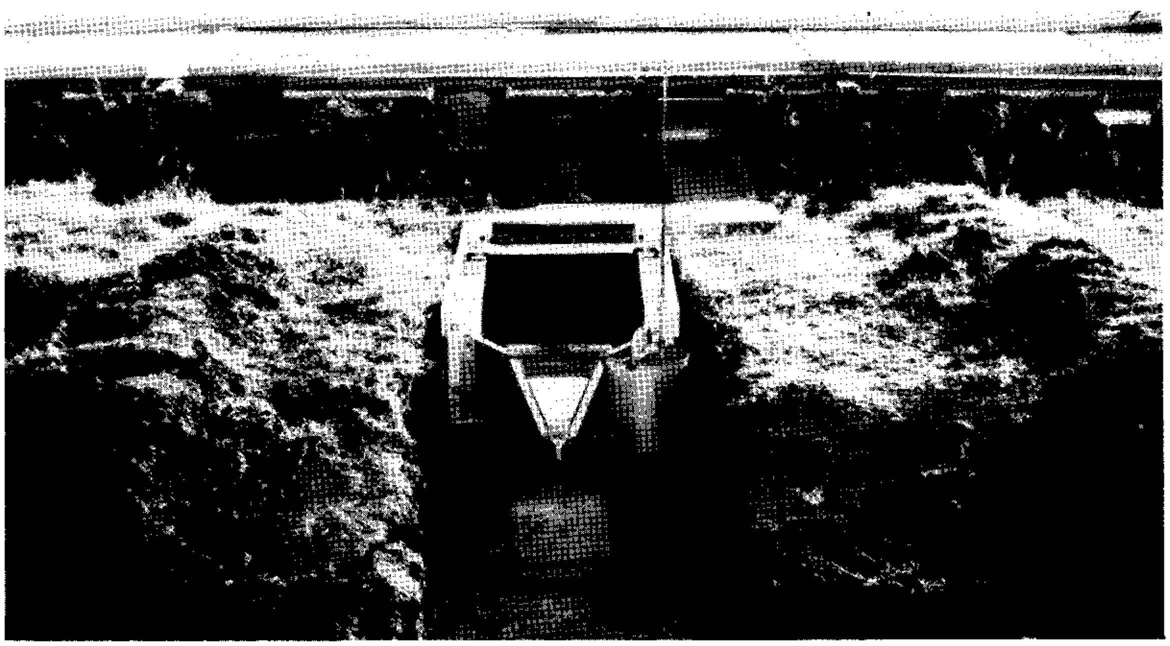

Figure 7. Typical H Flume Installation With Discharge From a 12 inch Side Weir Entering at Upper Right Side.

where the average value of the product $AR^{2/3}$, determined at the five gauging stations within the test reach, was utilized. The 27 values of n ranged from 0.0116 to 0.0227. The mean value of $(n)_{ave}$ = 0.0132 lay within the 95 per cent confidence interval of:

$$0.0123 \leq (n)_{ave} \leq 0.0141$$

with an estimate of the standard deviation of s = 0.0023.

Results of the Spatially Varied Flow Experiments

Channel Inflow vs. Outflow

For each SVF experiment, the total outflow from the channel distribution section was computed by two methods and was compared with the measured channel inflow Q. The total channel outflows indicated by the H flume measurements of side weir discharge agreed satisfactorily with the measured Q values. The maximum deviation between the estimated outflows and the measured inflows was 4.6 per cent, while the mean square deviation was only 1.77 per cent.

The channel outflow was also estimated from the average of the calculated side weir discharges in each distribution bay. The heads in the four side weirs whose discharges were measured were obtained by subtracting the crest elevations from the observed water surface elevations. Values of Q_w were then computed from side weir discharge relationships obtained from indoor laboratory calibration experiments (18), which were conducted in a 16 ft long plywood flume with the same cross sectional dimensions as the concrete channel. Except for two of the SVF experiments, the predicted outflows exceeded the measured inflows by as much as 23.0 per cent. The mean square deviation for all experiments was 10.0 per cent. Consequently, the laboratory calibration equations for the side weirs were believed inadequate for predicting Q_w from a concrete irrigation bay.

Discharge from Multiple Side Weirs

Improved prediction equations for multiple side weir discharge from a concrete channel were developed using the H flume measurements of Q_w. Cubic equations which related the observed water surface elevations to distances along the channel were computed. Discharge relationships for multiple side weirs were then developed by relating the measured values of Q_w to the head, computed from the cubic profile regression equations, and the crest length. From all SVF experiments, 529 values of Q_w, h, and the channel Reynolds number upstream from the weirs were determined. The heads ranged from 0.035 to 0.462 ft while the Reynolds numbers, defined as $N_R = VR/\nu$ where ν is the fluid kinematic viscosity, varied from about zero to 81,000. The following discharge equation was obtained:

$$Q_w = 4.08\, L_w^{0.907}\, h^{1.571} \tag{16}$$

The standard deviation from regression was s = 0.0055 cfs, or 4.55 per cent, with a correlation coefficient of r = 0.9993. The discharge coefficient in equation (16) could not be successfully correlated with the weir spacing and the channel Reynolds number.

Head versus discharge relationships were computed for the individual weir sizes using the equation $Q_w = ah^b$. The coefficients a and exponents b are listed in Table I along with the correlation coefficients and the mean square deviations. A characteristic plot of log Q_w versus log h is shown in Figure 8 for 4 inch side weirs.

TABLE I

HEAD VS. DISCHARGE RELATIONSHIPS FOR MULTIPLE SIDE WEIRS DISCHARGING FROM A CONCRETE CHANNEL

Average Crest Length Inches	No. of Observations	Intercept a	Slope b	Correlation Coefficient	Std. Dev. Per Cent
2.010	80	0.812	1.556	0.999	2.50
3.999	123	1.591	1.625	0.999	4.23
6.006	130	2.206	1.576	0.999	3.40
8.000	139	2.826	1.569	0.999	3.45
12.010	57	3.576	1.486	0.999	2.47

<u>Roughness Coefficients for Spatially Varied Flow</u>

Two roughness coefficients were calculated for decreasing spatially varied flow. The roughness coefficient $\bar{n}$ was the value of Manning's n that predicted the observed water surface profiles from an iterative solution of the Bernoulli equation. Equation (1) was solved between successive stations 3.333 ft apart, beginning at the downstream end of the irrigation bay,

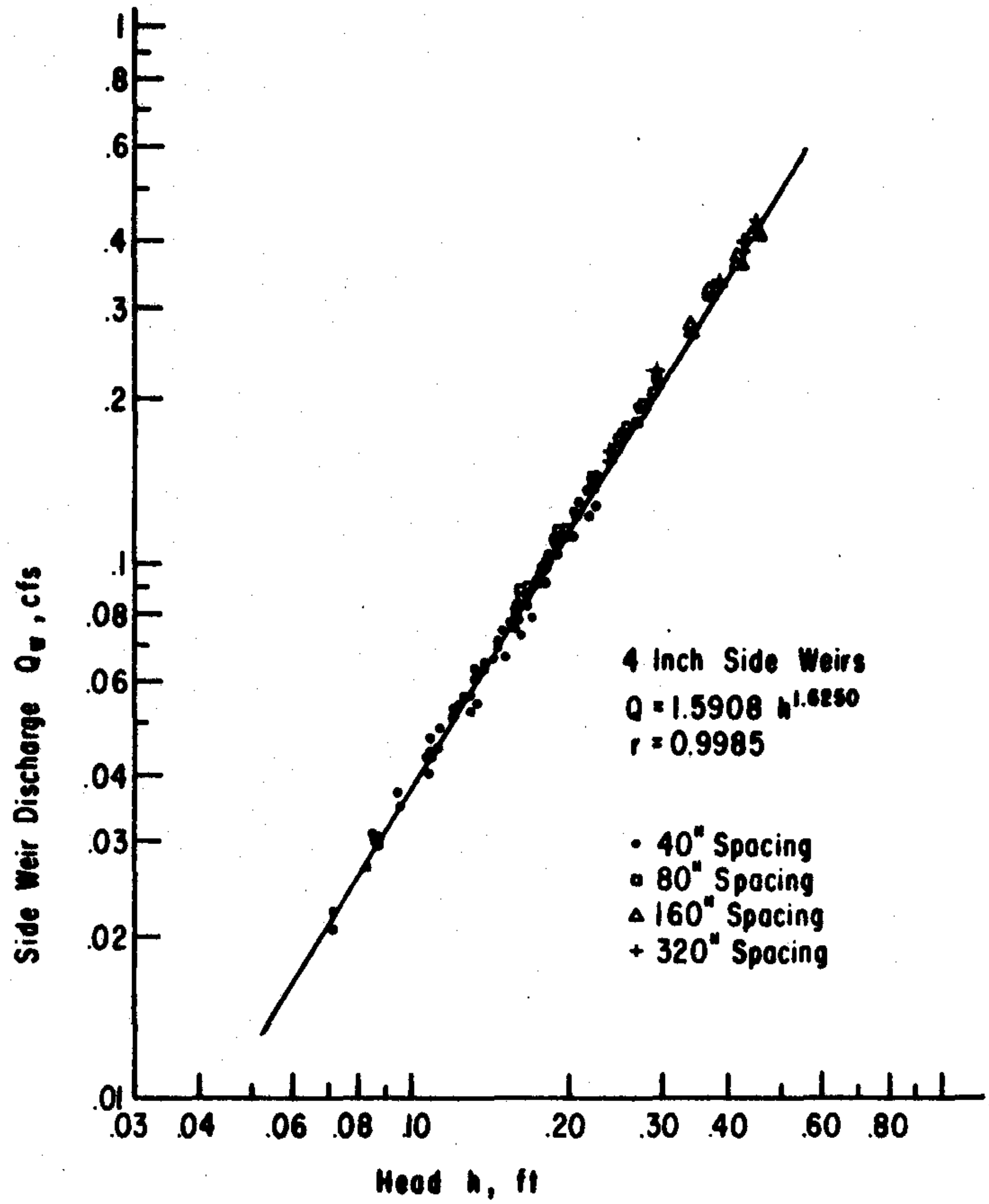

Figure 8. Head vs. Discharge Relationship for 4 inch Side Weirs in a Concrete Irrigation Channel.

by incrementing y_2 until both sides of this equation agreed within ±0.000005 ft. The iterative solution was continued upstream to station 1+20 where the calculated water surface elevation was compared with the elevation computed from the cubic profile regression equation. After incrementing n, the flow profile was recomputed until a value of $n = \bar{n}$ caused the calculated and least squares profile to agree within ±0.00005 ft at station 1+20.

The effective roughness coefficient n_e was computed using equations (5) and (6) which involve the change in the water surface elevation between the ends of the irrigation bay, the entering velocity, and the upstream hydraulic radius. The upstream and downstream water surface elevations $Z_i = (y_i + z_i)$ and $Z_o = (y_o + z_o)$, respectively, were evaluated for use in equation (5) from the cubic profile regression equation. The calculation of n_e from equation (6) is analogous to the calculation of Manning's n for gradually varied flow.

For the 94 experiments performed in a single distribution bay, the experimental relationship between $\bar{n}$ and n_e as shown in Figure 9 was:

$$\bar{n} = 1.616\, n_e \qquad (17)$$

for which r = 0.993 and s = 0.0005.

Attempts to relate the roughness parameter $(n_e/R_i^{1/6})$ to the upstream Reynolds number, the weir spacing, and the crest length resulted in low correlation. An analysis of the expected measurement error in n_e was performed. Tolerance bands were constructed around the mean of n_e on a plot of n_e versus N_R. Only four of the 94 values of n_e lay outside the tolerance bands which corresponded to an error of ±0.001 ft in measuring the difference in water surface elevation between the upstream and downstream ends of the irrigation bay. Apparently, variations in n_e were primarily caused by the inability to accurately measure the comparatively small changes in water surface elevation which occurred under SVF conditions rather than by the effects of the experimental parameters.

A 95 per cent confidence interval was established about the mean value of 0.0096 for n_e. This confidence interval was:

$$0.0092 \leq (n_e)_{ave} \leq 0.0102$$

The estimate of the standard deviation was s = 0.0025. Similarly, a 95 per cent confidence interval on the mean value of $\bar{n}$ of 0.0156 was:

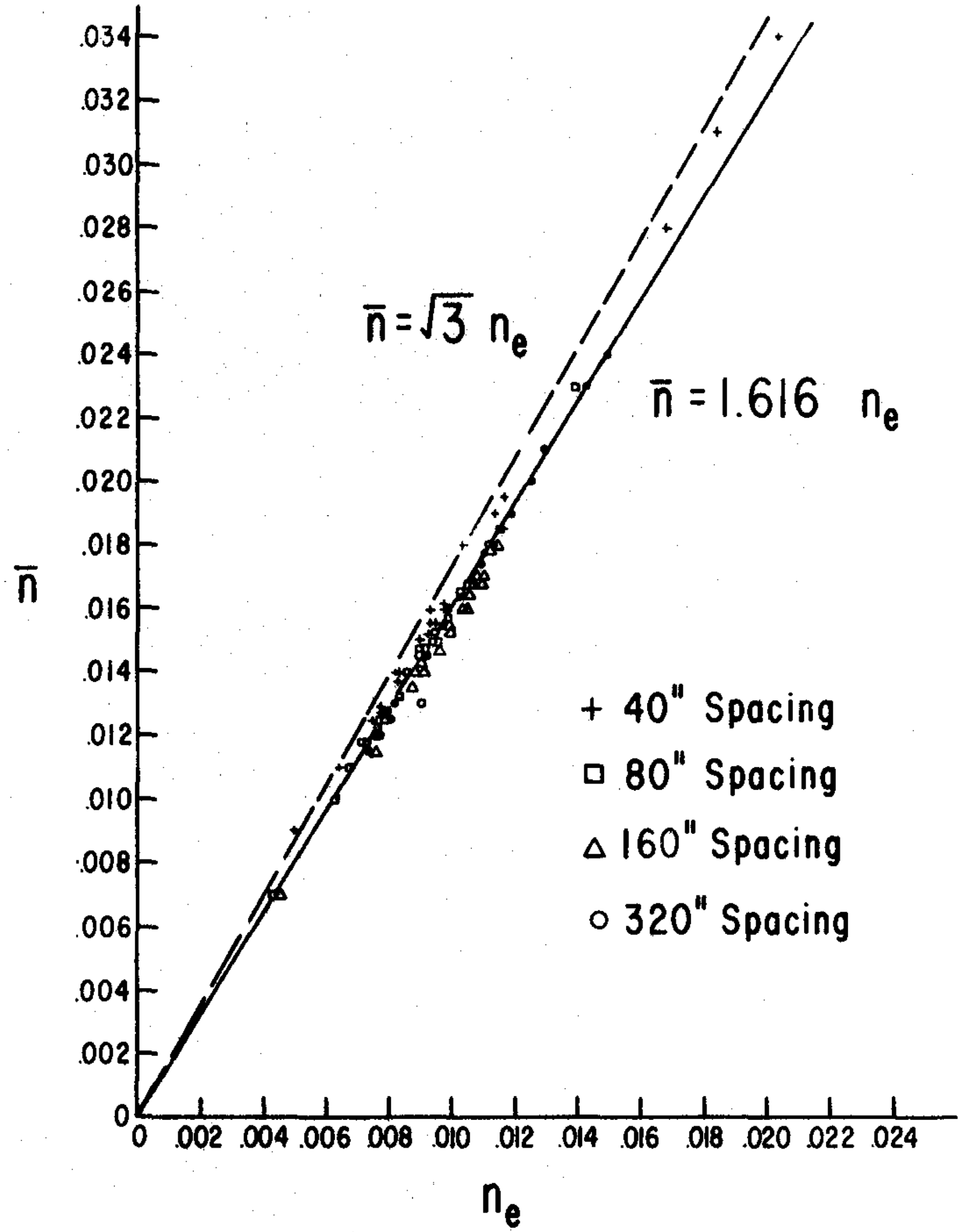

Figure 9. Relationship Between the Experimental Values of $\bar{n}$ and n_e.

$$0.0147 \leqq (\overline{n})_{ave} \leqq 0.0164$$

for which s = 0.0041.

Observed Water Surface Profiles

The observed water surface profiles for most SVF experiments exhibited a rising shape. The largest rises in water surface elevation were associated with the largest channel inflows and the narrowest spacings. The maximum recorded profile rise was 0.009 ft while the largest decline was 0.001 ft.

Figure 10 illustrates the effect of a drop in side weir crest elevation at station 1+80 on the observed water surface profiles for constant values of inflow, spacing, and crest length. For the profiles shown in Figure 10, the ratio of the total side weir discharge from the upstream bay to the initial Q was 1/3 and 1/5, respectively, for drops in weir crest elevation of 0.050 and 0.099 ft. The friction losses in the upstream bay increased and the velocity head recovery diminished with increasing drops in weir crest elevation. In the downstream bay, however, steeper profile rises occurred when the drop in crest elevation was increased, since higher rates of conversion of kinetic energy to potential energy and lower values of friction loss were present with increasing drop.

The effect of channel inflow rate on the water surface profiles in one bay is exemplified in Figure 11. The rises in water surface elevation along the test reach were consistently magnified with increasing channel inflow rate.

Figure 12 shows the effect of weir crest length at a given spacing and channel discharge. The water surface slope increased slightly with increasing crest length, probably the result of smaller depths and hence more kinetic energy present at station 1+20 with the larger side weirs.

The influence of side weir spacing on the observed water surface profiles is depicted in Figure 13. For given values of Q and L_W, the overall change in water surface elevation ΔZ_L decreased from 0.009 to 0.004 ft as the spacing was widened from 40 to 320 inches. Higher velocity head recoveries resulted from the smaller flow depths at the narrowest spacings. Furthermore, at the widest spacings, larger channel flow rates were conveyed greater distances along the channel, causing increased friction losses, than for closer spacings.

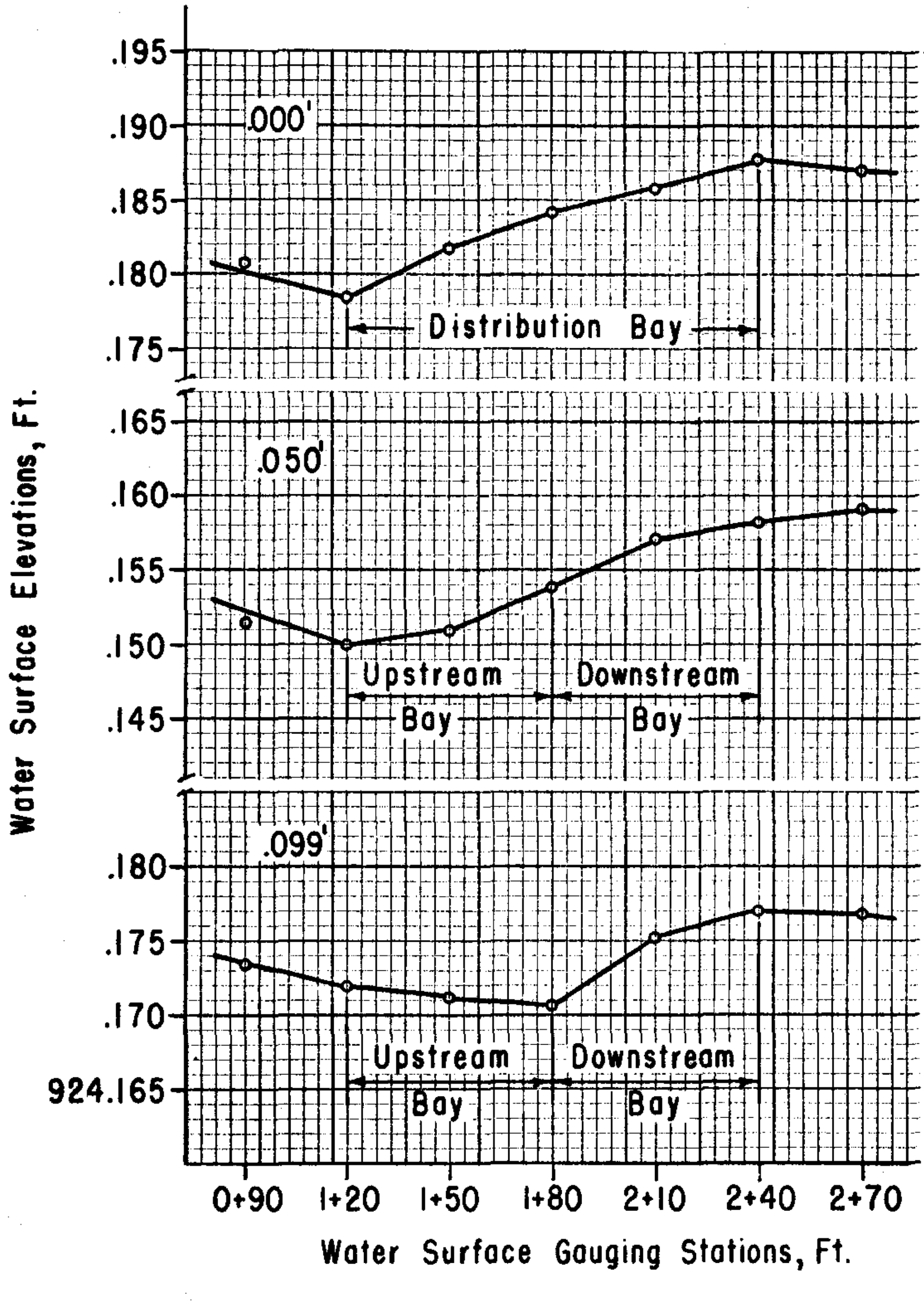

Figure 10. The Effect of Drop Between Distribution Bays on Observed Water Surface Profiles for 8 inch Side Weirs at 40 inch Spacing (Q ≈ 3.6 cfs).

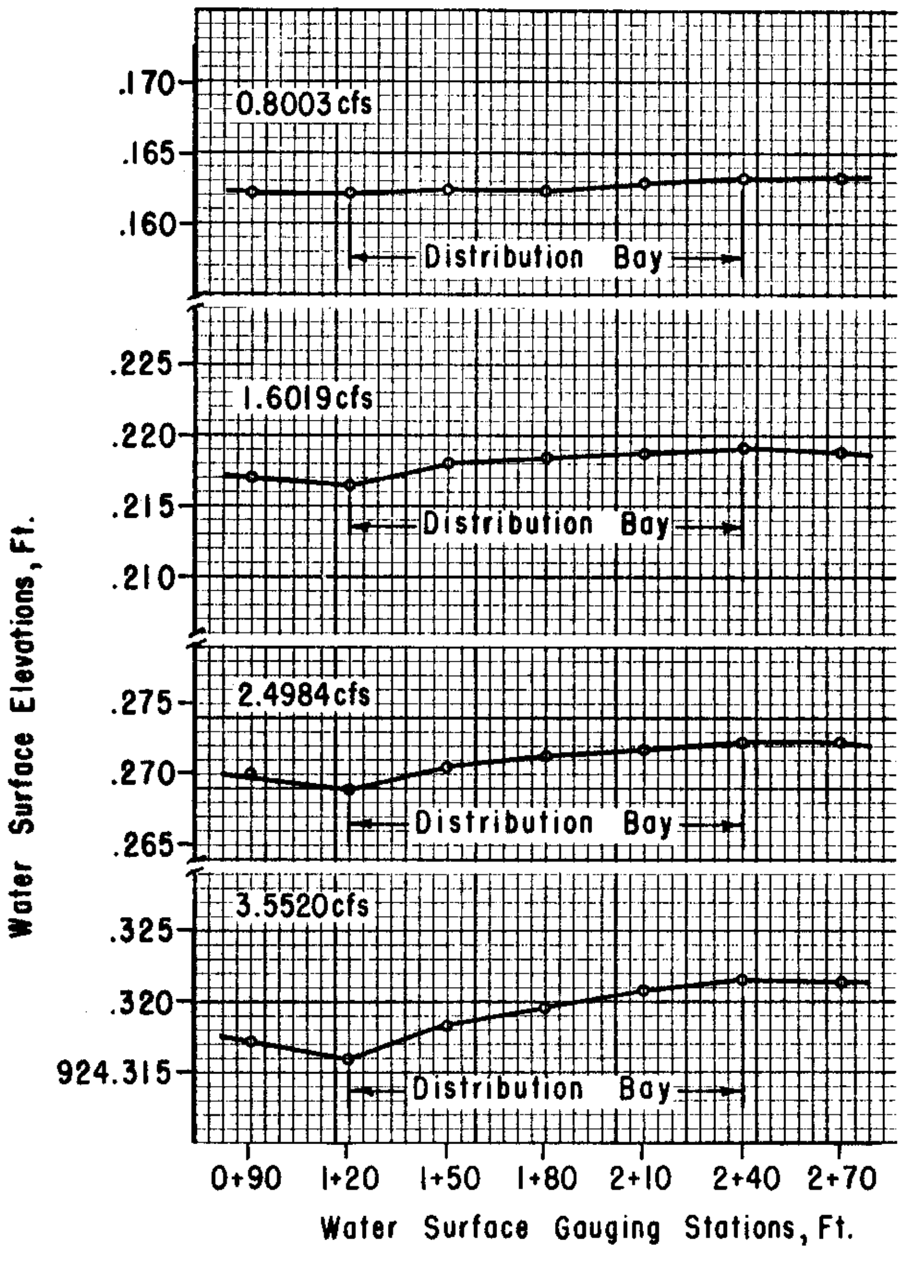

Figure 11. The Effect of Channel Inflow Rate on Observed Water Surface Profiles in a Single Distribution Bay (2 inch Weirs at 40 inch Spacing).

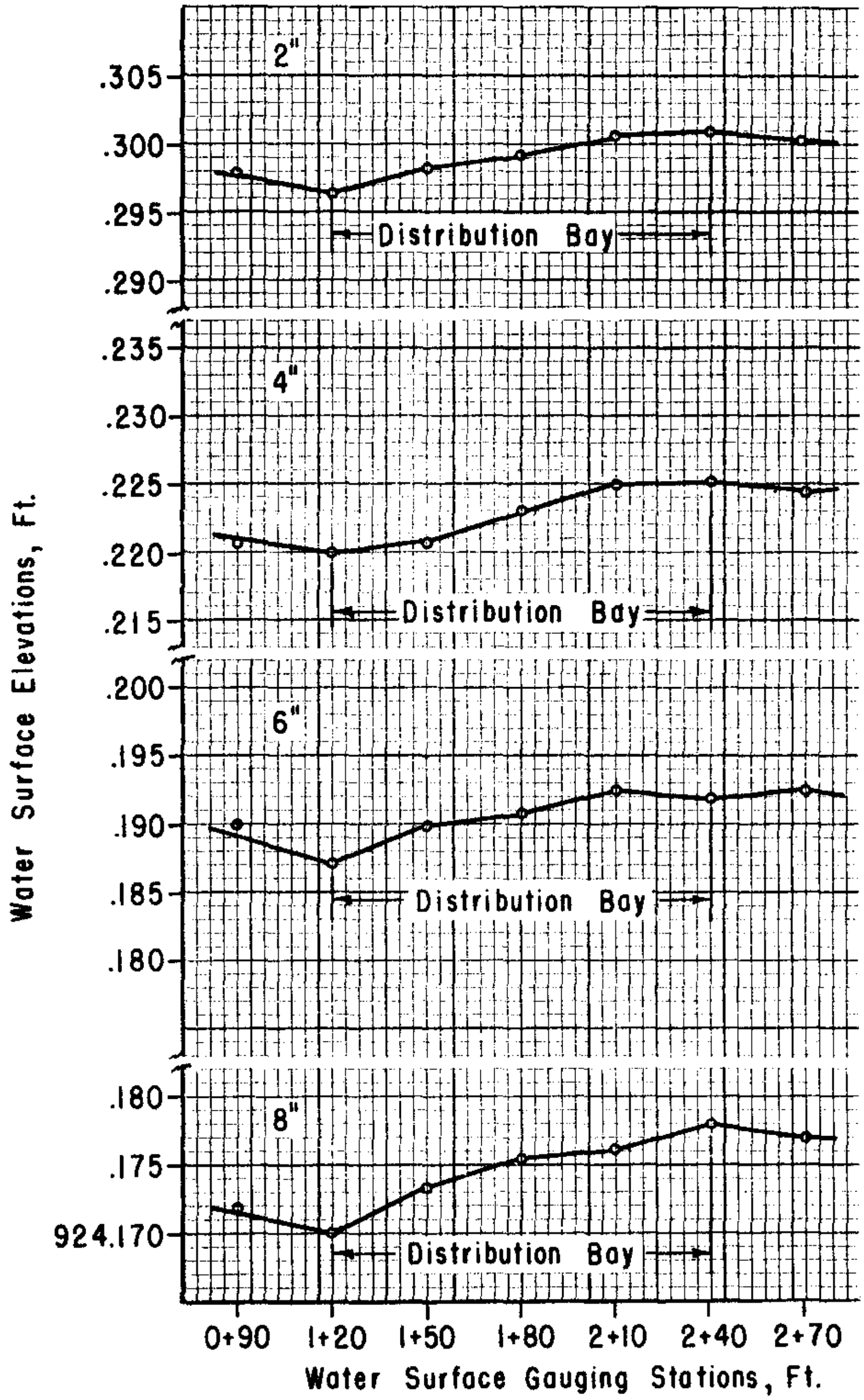

Figure 12. The Effect of Weir Crest Length on Observed Water Surface Profiles in a Single Distribution Bay (40 inch Spacing, Q ≈ 3.0 cfs).

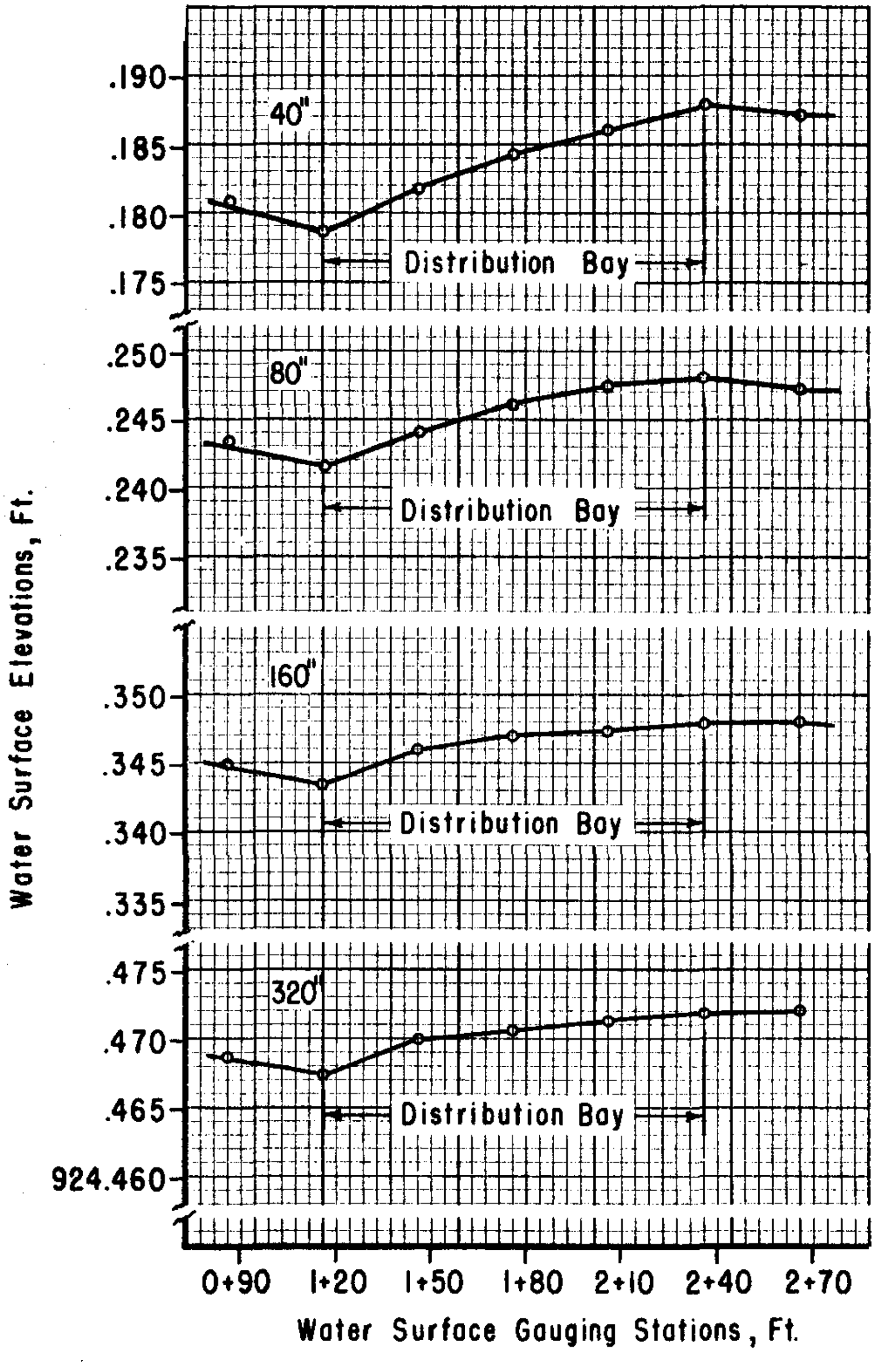

Figure 13. The Effect of Side Weir Spacing on Observed Water Surface Profiles (8 inch Weirs, Q ≃ 3.6 cfs).

Calculated SVF Profiles

Water surface profiles in a single distribution bay were calculated from the Bernoulli equation, equation (1), and the direct solution, equation (12), using the experimentally determined values of $\bar{n}$. The Bernoulli equation, solved at intervals of 3.333 ft beginning at station 2+40, predicted falling profiles between the side weirs for spacings of 80, 160, and 320 inches because of gradually varied flow conditions in these short reaches. As shown in Figure 14, a sharp rise was predicted across the side weir outlets.

The flow profiles calculated from the direct solution, equation (12), deviated from the observed profiles by less than 0.001 ft at the downstream end. The closest agreement between observed and calculated profiles occurred with 40 and 80 inch spacings.

Water surface profiles in two interconnected distribution bays were computed from equation (12). The velocity function V_x, depicted in Figure 3, was defined by numerical values of V_i, V_m, and Q_w obtained from the profiles computed with the Bernoulli equation. The virtual length of the upstream bay L_1' was enumerated using equation (14).

Three profiles calculated from equation (12) for experiments with drops in weir crest elevation of 0.00, 0.050, and 0.099 ft are presented in Figure 15. The mean value of $(\bar{n})_{ave}$ = 0.0156 was utilized in each case. Equation (12) predicted the water surface profiles in one bay and in two interconnected bays with comparable precision.

Variations in Side Weir Discharge

The side weir distribution uniformity was determined for each SVF experiment from the H flume measurements of side weir discharge. For a given bay, the percent variation in discharge D_p was computed from the expression:

$$D_p = \frac{Q_d - Q_u}{Q_d} \times 100 \qquad (18)$$

where Q_d and Q_u are the downstream and upstream side weir discharges. Negative values of D_p were obtained for falling water surface profiles while positive values were indicated for rising profiles.

The largest variations, up to 20.0 per cent, were observed in the upstream bay of the cutback furrow irrigation experiments. For these tests, appreciable changes in water surface elevation along the distribution bay were experienced at low weir heads. Drops in crest elevation of 0.050 ft generally produced larger

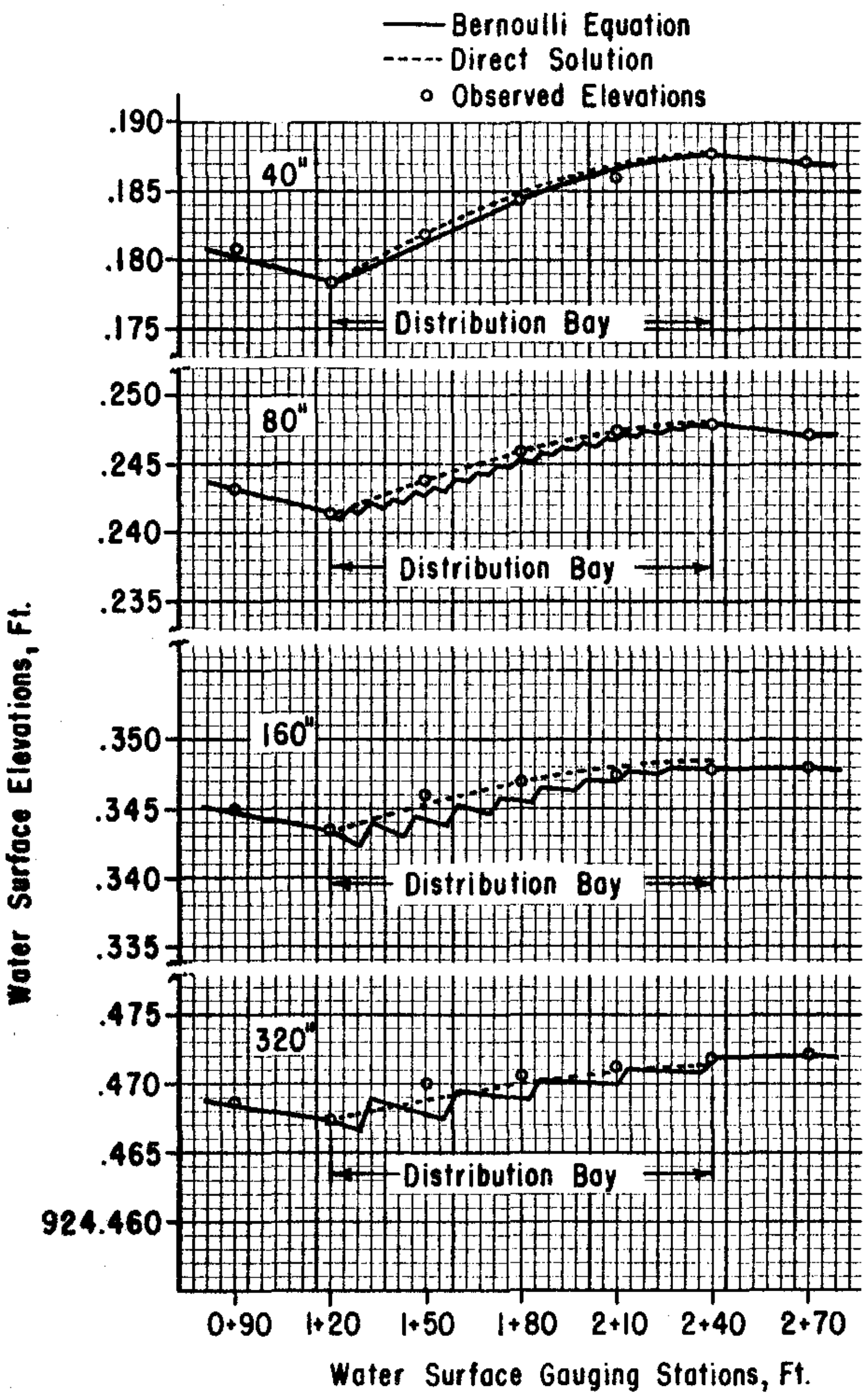

Figure 14. Comparison of SVF Profiles Calculated With Equation (12) and With the Bernoulli Equation for Various Side Weir Spacings ($Q \approx 3.6$ cfs, $L_w = 8$ in.).

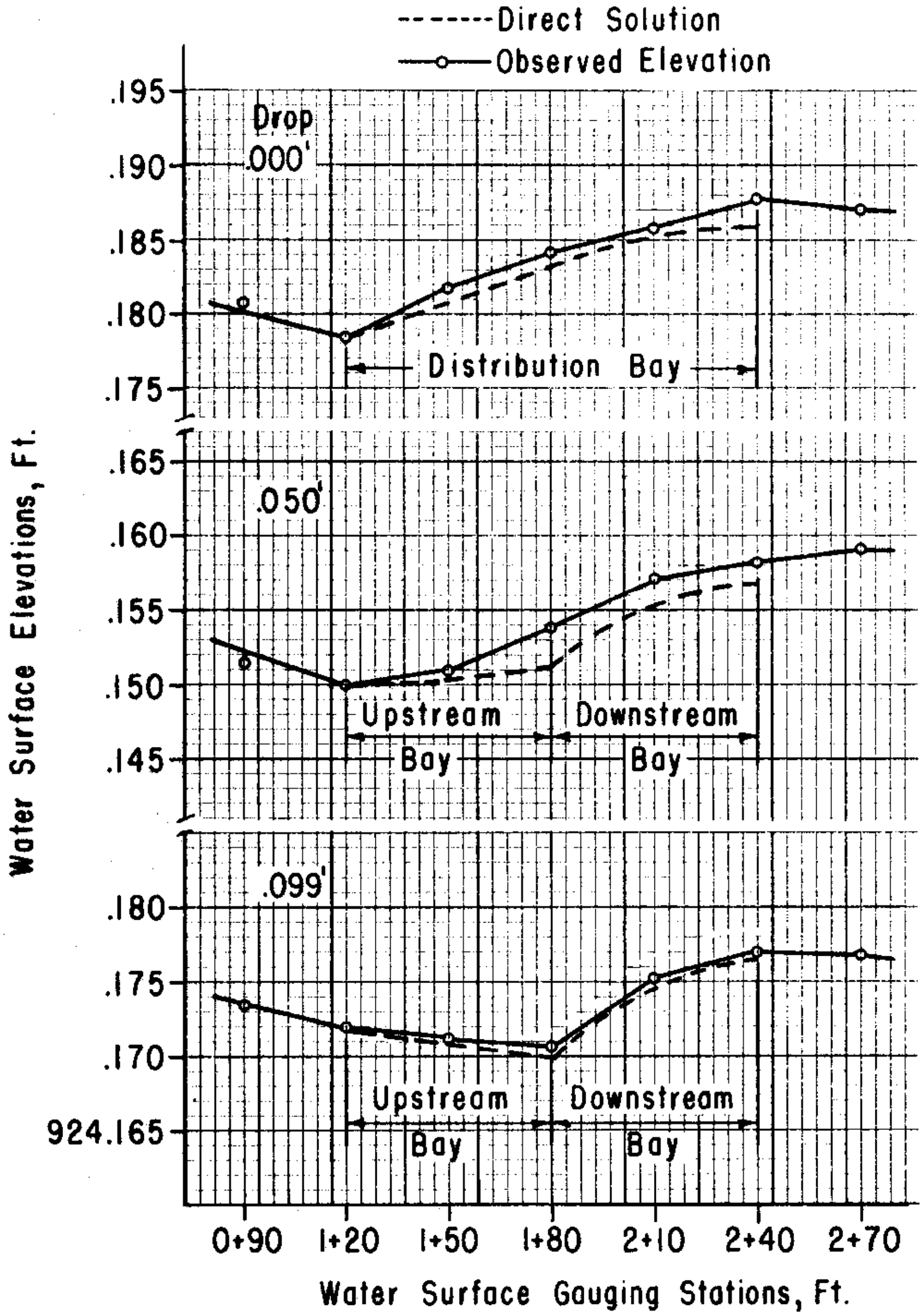

Figure 15. SVF Profiles Calculated from Equation (12) for One Distribution Bay and for Two Interconnected Bays ($Q \approx 3.6$ cfs, L_w = 8 in., L_s = 40 in.).

variations than were observed for drops of either 0.025 or 0.100 ft. Some experiments with 8 inch weirs with a drop between bays of 0.099 ft produced negative variations, averaging -6.7 per cent, in the upstream bay and positive values of D_p, ranging from 4.7 to 9.7 per cent in the downstream bay. For a single distribution bay, the highest deviations, up to 15.6 per cent were observed in conjunction with the narrowest weir spacings and the maximum inflows.

For constant values of crest length and spacing, the variations in side weir discharge generally increased with channel inflow rate. The variations D_p at the maximum flow rates (3.5 - 4.1 cfs) were from 1.1 to 4.5 times greater than the variations at Q = 1.6 cfs.

The shortest crest lengths produced slightly smaller variations than did the larger weirs, apparently the result of lower heads required by the greater crest lengths for the same weir discharges. For example, the mean values of D_p for 40 inch weir spacings in a single distribution bay were 5.4 and 7.3 per cent for crest lengths of 2 and 8 inches, respectively.

For a given channel inflow and crest length, the weir discharge variations increased considerably with decreased spacing. For the 6 inch weirs, the ratio of the mean variation at 40 inch spacing was 7.2 times the average value of D_p at the 320 inch spacing. For weir spacings of 80, 160, and 320 inches, the maximum measured variations in side weir discharge were 9.4, 7.2, and 4.2 percent, respectively.

Summary

1. Discharge from rectangular side weirs placed in an irrigation distribution channel can be accurately predicted using the discharge relationships developed from the prototype side weir irrigation experiments.

2. From the experiments in a single distribution bay, the mean values of n_e and $\bar{n}$ were 0.0096 and 0.0156, respectively, while the ratio of $\bar{n}$ to n_e was 1.62 as compared to the theoretical ratio of 1.732. The average value of Manning's n for gradually varied flow was 0.0132.

3. Water surface profiles for decreasing spatially varied flow in a horizontal distribution bay and in two adjacent distribution bays can be accurately calculated from a direct solution, equation (12), provided that the stated assumptions for this equation are approximated.

4. Side weir discharge variations ranging from -7.9 to +20.0 per cent occurred in the upstream bay of a cutback irrigation system.

5. Side weir discharge variations for equal crest elevations were always less than 10 per cent for weir spacings of 80 to 320 inches (6.67 - 26.67 ft).

6. The variations in side weir discharge along an irrigation distribution bay were reduced by diminished channel inflow rates, shortened weir crest lengths, and increased side weir spacings.

Report on Phase II of the Project

A SEMI-PORTABLE SHEET METAL FLUME FOR[1] AUTOMATED IRRIGATION

by

Vincent W. Uhl, Jr. and James E. Garton

INTRODUCTION

Traditional surface irrigation is inhibited by high labor costs and low water application efficiency (normally in the 50% range). Non-uniform water applications cause both insufficient applications and tailwater waste.

In a response to the need for more efficient and labor saving irrigation systems, several researchers have done work on various aspects of automated irrigation.

Garton (8)* developed an automated cut-back furrow irrigation system which dealt with the problem of non-uniform flow in a distribution bay. Rather than have a sloping channel, the system consisted of a series of horizontal bays staircased down the slope of a field. In a horizontal bay a near horizontal water surface occurred, and for siphon tubes at the same elevation, very uniform furrow flows resulted. A cut-back system uses a large furrow flow to initially wet the length of the furrow. After the initial wetting and before tailwater flow occurs, the flow is reduced to balance the intake rate of the soil for a given length of furrow.

One problem with such a system is that the design of the system is dependent on a constant inflow Q, which can be difficult to maintain, especially where deep wells are used to supply the water. The system is also permanent and hence interferes with machine mobility especially during planting and harvesting operations. The system is also difficult to adapt to changed cropping patterns.

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[1]The research reported in this paper was financed in part by the United States Department of the Interior as authorized under Public Law 88-379. This research was conducted as a project of the Oklahoma Water Resources Research Institute (Okla. - A-015), Dr. Marvin T. Edmison, Director. The financial support of the Institute is appreciated.

*Number in parentheses refers to appended references.

A semi-portable irrigation channel designed with the functions of a cut-back system would have certain advantages. The channel could be set up after the planting season prior to the first irrigation and removed before harvesting. Such a system would also be adaptable to a change in inflow Q and could be relocated to cope with changes in cropping patterns.

Objectives

1. To hydraulically design a portable irrigation channel to function as a cut-back system and to function automatically.

2. To structurally design and construct the channel.

3. To determine an average roughness coefficient for gradually varied flow.

4. To determine an average roughness coefficient for decreasing spatially varied flow.

5. To determine the discharge characteristics of the orifices in the channel.

6. To determine discharge uniformity.

THEORY

Flow from an irrigation distribution bay is a form of decreasing spatially varied flow. Mink (14) and Sweeten (18, 19) in previous research have investigated decreasing spatially varied flow in a horizontal concrete distribution channel, where the flow is taken off by siphon tubes and weir plates. Mink found that Manning's n for gradually varied flow did not accurately predict water surface profiles for decreasing spatially varied flow.

Mink (14) defined a roughness coefficient called $\bar{n}$ which is the mean value of Manning's n, which when used in conjunction with the Manning formula and the energy equation will produce the actual water surface profiles for decreasing spatially varied flow.

Sweeten (19) derived an equation that solves water surface profiles for decreasing spatially varied flow. For decreasing spatially varied flow in a horizontal irrigation distribution bay, two energy components, velocity head recovery and resistance loss, influence the rise or fall of flow profiles.

Thus the energy equation can be written:

$$\Delta ws = \Delta H_v - H_f \tag{1}$$

where:

Δws = change in water surface elevation (positive for rising profiles).

H_v = velocity head recovery.

H_f = energy loss due to hydraulic resistance.

Developing the components H_f and H_v separately, equation (2) was arrived at:

$$\Delta ws = \frac{V_i^2}{2g}\left(\frac{2X}{L} - \frac{X^2}{L^2}\right) - \frac{\bar{n}^2 V_i^2}{2.208\ R_i^{4/3}}\left(X - \frac{X^2}{L} + \frac{X^3}{3L^2}\right) \qquad (2)$$

where:

X = distance from the upstream end of the distribution bay.

L = length of the distribution bay.

V_i = upstream velocity.

R_i = upstream hydraulic radius.

Given a change in water surface elevation along a bay, equation (2) can also be used to solve for $\bar{n}$.

For a two bay situation, equation (2) can be used to solve for the water surface profile. For a uniform weir or orifice discharge Q_{w_i} in the upstream bay, the virtual channel length L_i necessary to completely discharge the inflow Q in the first bay is:

$$L_i = L_s\ Q/Q_{w_i} \qquad (3)$$

Where L_s is the orifice spacing, L_i is substituted for L in equation (2) and that equation is used to solve for the water surface profile in the first bay. The profile for the downstream bay can be calculated from equation (2) with $L = L_2$, and the velocity at the upstream end of bay 2 substituted for V_i. Since a change in area occurs between the two bays, the water surface elevation downstream from the change should reflect the alteration in velocity head.

Mink (14) defined another roughness coefficient for decreasing spatially varied flow which is useful in designing irrigation systems. This coefficient, termed effective n or n_e can be computed using the entering discharge, the entering depth, and the change in total energy between the ends of the bay. Equation (4), which is the energy equation combined with Manning's equation can be used to solve for n_e:

$$n_e = \frac{1.486\ A_i R_i^{2/3}}{Q_i} \left[\frac{1}{L} \left(\frac{Q_i^2}{2gA_i^2} + ws_i - ws_o\right)\right]^{1/2} \qquad (4)$$

Where i and o refer to the upstream and downstream sections, Q_i is the inflow in cfs, R_i is the upstream hydraulic radius, A_i is upstream area. Ws = y + z = water surface elevation.

Hydraulic Design

An automated cut-back irrigation system was designed utilizing a sheet metal flume as a conveyance channel. Factors affecting the design are: The dimensions of the channel section, the height of orifice above the bottom of the channel, diameter of orifice, orifice spacing, land slope, friction factors n and $\bar{n}$, and range of initial and cut-back flows.

The first step in the design was to determine the optimum dimensions of the channel section. Next, the optimum height of orifice was determined. Lastly, a design table was constructed for the channel in which the range of acceptable inflows Q, slopes, orifice diameter, and initial and cut-back flows are presented.

A section with bottom width = 1.5 feet and depth = 1.25 feet was found to be optimum with stability of water surface profiles being the main design criteria.

The main consideration in determining optimum height of orifice is that in a multiple bay operation, e.g., when bay 3 is discharging the initial flow, and bay 2 is at cut-back flow there must be no flow from bay 1. That is, the orifice elevation must be greater than the water surface elevation in all of bay 1. For a given field problem the allowable difference in elevation of the water surface between the upper and lower ends of bay 1 is:

$$h_f = Z_1 + Z_2 - h_1 - W \qquad (5)$$

where:

h_1 is the initial head in bay 3.

Z_1 is the drop between bay 1 and bay 2.

Z_2 is the drop between bay 2 and bay 3.

W is the distance the bottom of the orifice must be set above the normal water surface at the upper end of the bay to prevent discharge due to wave action by the wind. W is equal to 0.01 feet for design purposes.

Now the actual difference h_f in the cut off bay is simply the friction loss for gradually varied flow and can be compared to the allowable h_f to justify the design or not.

The final design for height of orifice is a compromise between friction loss h_f and h_1 available for the initial furrow flow. A height of orifice = 0.75 feet was chosen allowing 0.40 feet for initial furrow flow, and 0.1 feet for freeboard.

Design tables were computed for varying inflows Q, initial and cut-back flows, land slopes, and furrow spacings. The design tables and accompanying nomograph enable a planner to select the system which best corresponds to a specific field problem. The design table ensures that friction loss h_f in the cut-off bay will be allowable and gives the combinations of initial and cut-back flows that can be used to ensure allowable h_f. Figure 1 is a nomograph solution to the hydraulic design table. The nomograph can be used to solve for the size of orifice for various operating conditions or if diameter size is fixed then the initial furrow flow can be decided.

Figure 2 is a plot of cut-back furrow flow, q , versus initial furrow flow, q , for different orifice diameters. The plot gives the combinations of initial and cut-back flows that can be used in design with the inflow Q = 2.0 cfs. Once the cut-back furrow flow is known, then figure 2 can be used to find the design initial flow.

Structural Design

The objective of the design was to design a channel section which was light enough to be easily handled, portable, leak free, easily assembled, and strong enough to withstand permanent bending and buckling. Also, it was necessary to support the channel above the ground and to be able to make minor adjustments in the field.

Several types of design were investigated considering mainly the problems of facilitative assemblage in the field, support of the channel section in the field, and leakage. The basic design consists of a sheet metal section 10 feet long, 1.5 feet wide, and 1.25 feet deep supported by a structural steel angle framework.

Several combinations of material design were considered and investigated theoretically. Three sizes of structural steel angle framework and three sizes of sheet metal were selected. From these, six combinations of frameworks and sheet metal were loaded and tested. The final design selected was a 24 gage sheet metal section supported by a 3/4" x 3/4" x 1/8" framework with 1/2" x 1/2" x 1/8" side and bottom braces. The design was chosen because of its relative advantages of weight, strength, and portability.

The supporting system consisted of a 1/2" steel rod welded to a 3" x 3" x 1/8" base. The two rod-base assemblies were fastened to a redwood base 26" x 8" x 1". Figure 3 is an overall view of the channel and shows the 10 foot sections assembled together, and a test being conducted.

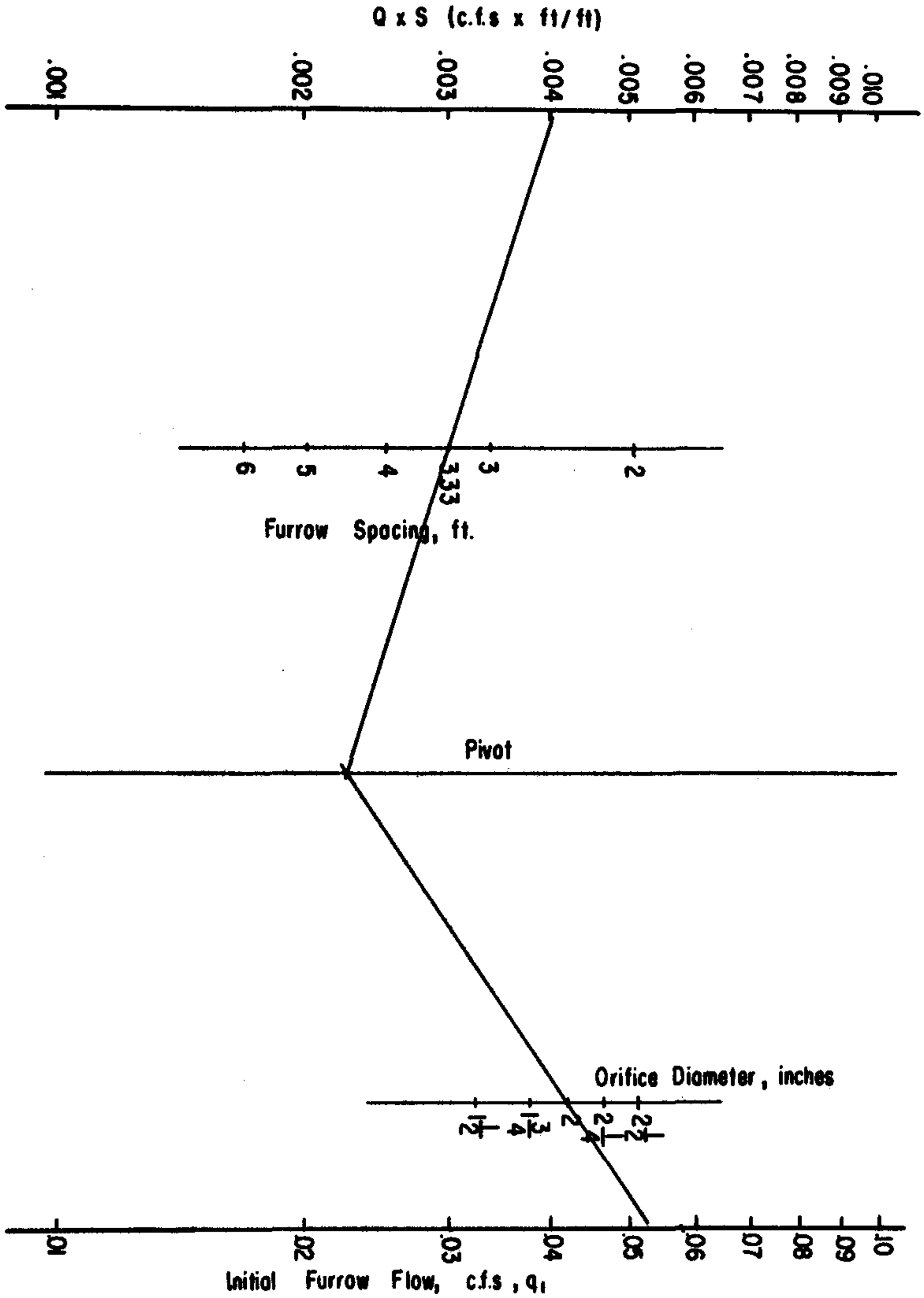

Figure 1 Hydraulic Design Nomograph

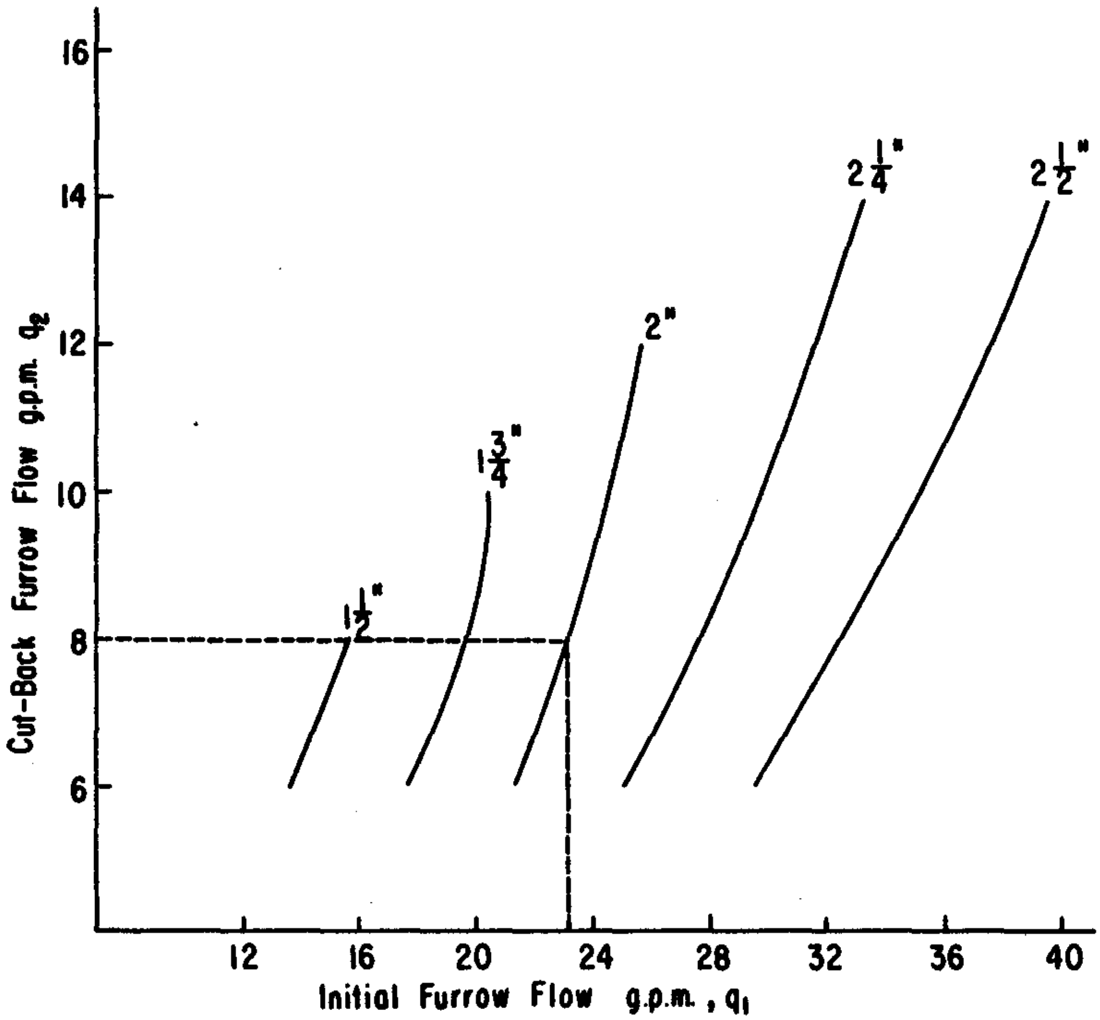

Figure 2 Plot of Acceptable Cut-Back Furrow Flows, q_1, Versus Initial Furrow Flows, q_2, for Different Size Orifices When Inflow Q = 2.0 c.f.s.

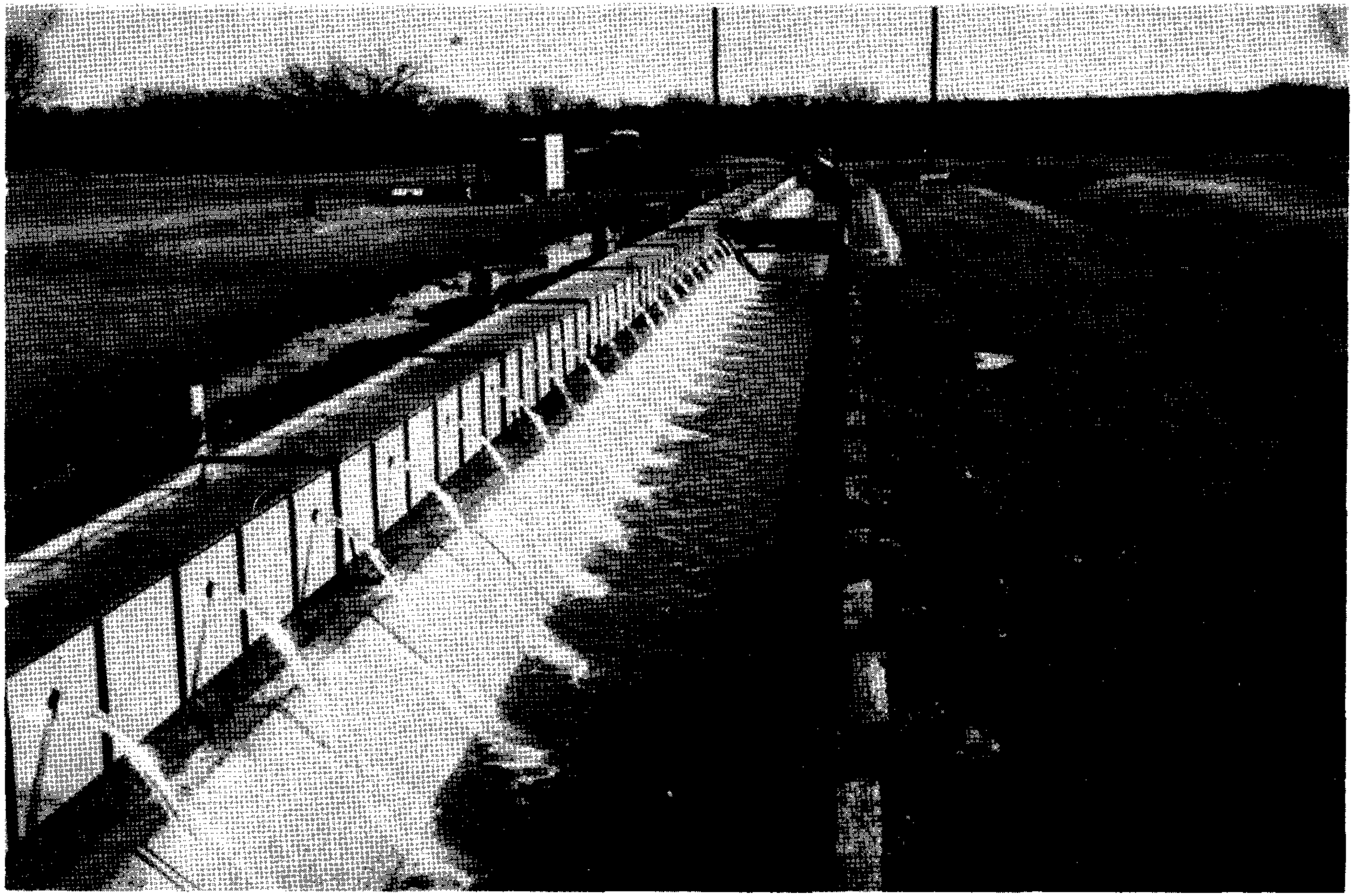

Figure 3 Overall View of the Channel

Apparatus

Facilities at the U.S.D.A. outdoor hydraulic laboratory in Stillwater were utilized for testing. Two hundred and thirty feet of channel were assembled with a test reach two hundred feet in length.

The test reach constituted two 100 feet horizontal bays with a drop between the bays. The drop was located at station 1+20. At the end of each bay a time operated check gate was located. When wind became a serious problem, light wooden covers were placed on top of the channel.

The water surface profile was measured at six stations in each bay, and the measurements were read at a common stilling well located at station 1+20. The measurement at the stilling well was done with a two foot point gage readable to 0.001 feet. Three H-Flumes placed at different orifices along the channel were used to measure weir and orifice flow for calibration.

The inflow, supplied through a 12-inch pipe was measured with calibrated orifice plates and a 60-inch water-air manometer.

Experimental

Three distinct types of tests were run: gradually varied flow, single bay spatially varied flow, and two bay spatially varied flow tests. Basically the same testing procedure was used for gradually varied flow and spatially varied flow tests.

Inflow was initiated and adjusted to the desired flow rate by use of the gate valve and manometer. An average of about ten minutes was required for the flow to completely stabilize after which the taking of measurements began.

Inflow through the orifice plate was measured by two variables, the differential head read by the manometer and the water temperature. Observations of the differential head were recorded for each experiment and the average reading was used.

For gradually varied flow, and single bay decreasing spatially varied flow tests, the water surface elevations were measured at the first six stations. For the two bay decreasing spatially varied flow tests, the water surface elevations were measured at all twelve stations.

The three H-Flumes were read several times during a test run and an average value at each H-Flume was used.

Finally, at each of the stations where the water surface elevation was measured, the bottom elevation and the elevation of the orifice was also measured.

Analysis of Data

Gradually varied flow and decreasing spatially varied flow tests were conducted. The objective of the gradually varied flow tests was to determine hydraulic roughness coefficients for various depths and rates of flow. After the gradually varied flow tests were run, orifices were cut and decreasing spatially varied flow tests were started. Decreasing spatially varied flow tests were run in one bay and two bays. The objectives of the tests were to determine (1) hydraulic roughness coefficients $\bar{n}$ and n_e; (2) water surface profiles; (3) weir and orifice discharge uniformity. Weir and orifice discharge was calibrated by the H-Flumes and the orifice plates in the line during the tests.

Gradually Varied Flow Tests

Eleven gradually varied flow tests were conducted to determine the hydraulic roughness of the channel. A series of tests were run with three depths of flow for three selected flow rates. A few random tests were also run. The energy equation combined with Manning's equations was used to solve for roughness n. n versus VR was plotted and $n = 0.0090(VR)^{-0.101}$. An Average value of n for the tests was 0.0096. (6)

Decreasing Spatially Varied Flow Tests

Several one bay and two bay decreasing spatially varied flow tests were conducted. Values of $\bar{n}$ were determined by solving for $\bar{n}$ in equation (2) which can be solved for $\bar{n}$ since V_i, Δws, R_i, and L are all known.

The effective roughness coefficient n_e was calculated using equation (4). The hydraulic roughness coefficients $\bar{n}$ and n_e for both one and two bay tests were calculated.

For the single bay tests an average value of $\bar{n} = 0.0126$ was found. For the two bay tests a different condition was found in the cut-back bay. Entering flows were quite high, but depths of flow were quite low, e.g., .865 - .910 feet since it was the cut-back bay. High inflows and low depths caused high velocities and consequently high values of VR (up to .777) compared to a maximum value of VR = .44 for the single bay tests. These values resulted in low values of $\bar{n}$ and an average value of $\bar{n} = 0.01065$ was found necessary for design purposes. Values of $\bar{n}$ computed for the initial flow bay compared with similar tests run in a single bay. The average value of the effective roughness coefficient $n_e = 0.00730$. Figure 4 is a plot of $\bar{n}$ versus VR for the decreasing spatially varied flow tests. Values of $\bar{n}$ for both the single and two bay tests are plotted in figure 4. A linear regression run in log-log space gave the following equation:

$$\bar{n} = 0.00952(VR)^{-0.262} \qquad (7)$$

with a correlation coefficient r = 0.885.

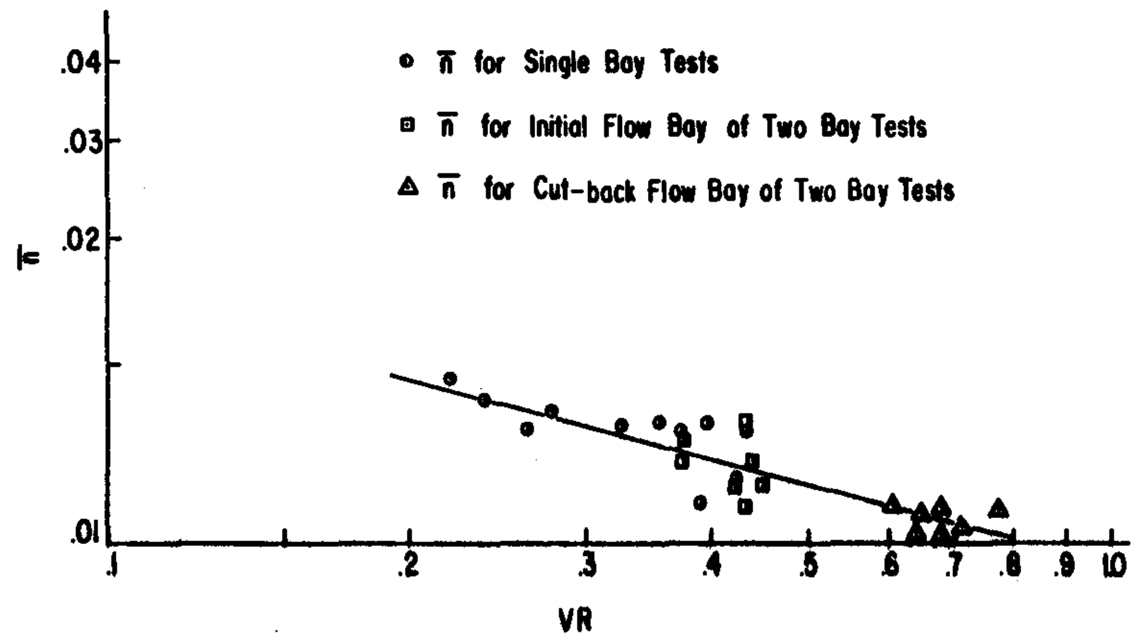

Figure 4. $\bar{n}$ Versus VR for Decreasing Spatially Varied Flow Tests

A ratio of $\bar{n}/n_{gvf}$ was computed for single bay tests and compared to Sweeten's (6,7) values for similar single bay tests using siphon tubes and weir plate. $\bar{n}/n_{gvf} = 0.0126/0.0096 = 1.31$ was arrived at for the single bay tests for the sheet metal flume as compared to Sweeten's values of $\bar{n}/n_{gvf} = 1.25$ and 1.28 for the weir plate and siphon tube tests, respectively.

Weir and Orifice Calibration

While one bay and two bay decreasing spatially varied flow tests were being conducted, orifices in both bays were being calibrated by use of three H-Flumes. Figure 5 is a plot of head versus flows for both weir and orifice flow. For weir flow the best fit equation for the 2.01 inch diameter orifice was:

$$q = 387.97h^{1.843} \tag{8}$$

With a correlation coefficient r = 0.981 and standard deviation s = 0.02132.

For orifice flow the best fit equation was:

$$q = 55.62h^{0.741} \tag{9}$$

With a correlation coefficient r = 0.984 and standard deviation s = 0.00152, where q is the orifice or weir flow in gallons per minute and h is the head of water in feet above the bottom of the orifice. The results correlate closely with former work by Greve (10) and Barefoot (1).

Water Surface Profiles

Water surface profiles were measured for both single bay and two bay tests. Figure 6 is a plot of observed and calculated profiles for single bay tests. Water surface profiles were affected by two energy components which influence profile shapes in a horizontal distribution channel, friction losses and velocity head recovery. Rising profiles were observed in all the single bay tests. This is attributed to velocity head recovery exceeding friction loss which results from kinetic energy being converted to potential energy. As inflow Q increased from 0.538 cfs to 1.579 cfs, the change in water surface elevation between the ends of the bay increased from 0.0015 feet to 0.008 feet. The profiles were calculated using equation (2) with $\bar{n} = 0.0126$.

Observed and calculated water surface profiles for two bay tests are plotted in figure 7. Two series of two bay tests were run. For one series both bays were 100 feet in length with a drop of 0.185 feet between bays. For the second set of tests, the downstream bay was 80 feet long with a drop of 0.220 feet between bays.

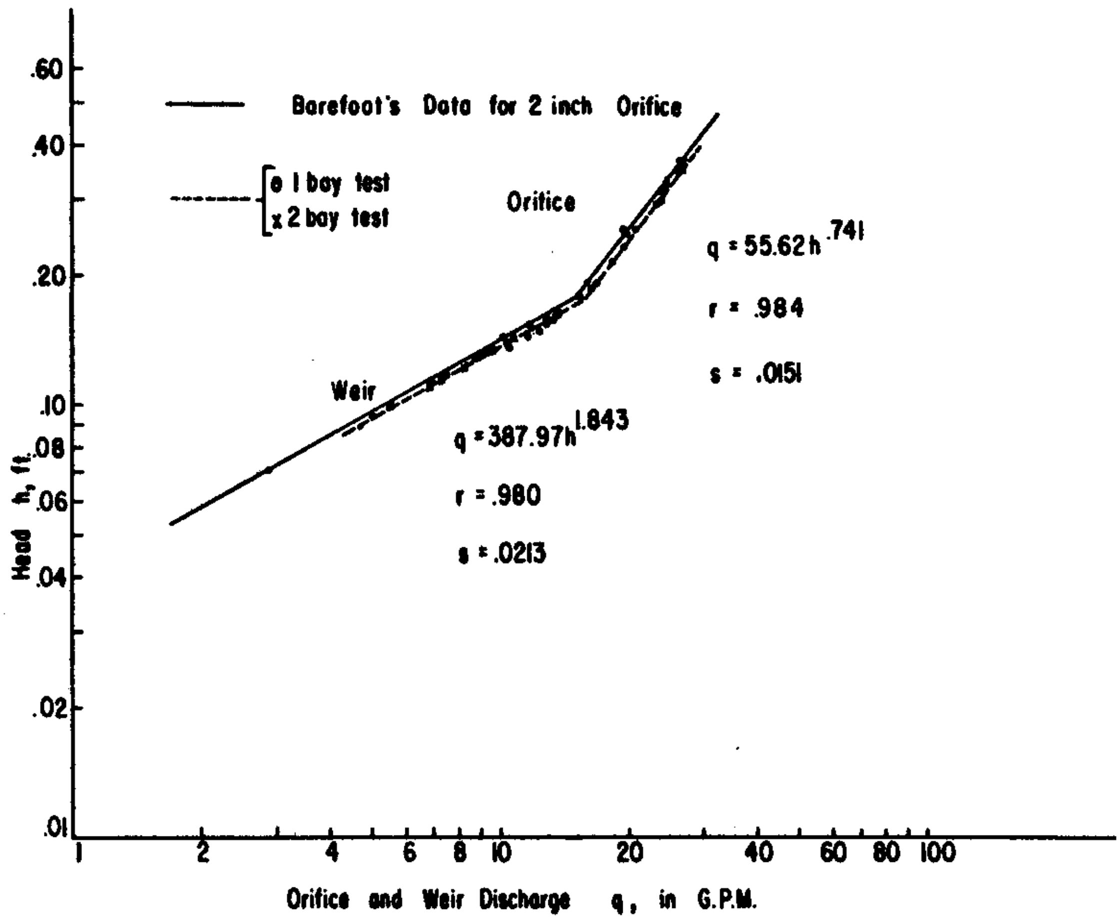

Figure 15. Head, h, Versus Weir and Orifice Flow q, Compared to Barefoot's Data

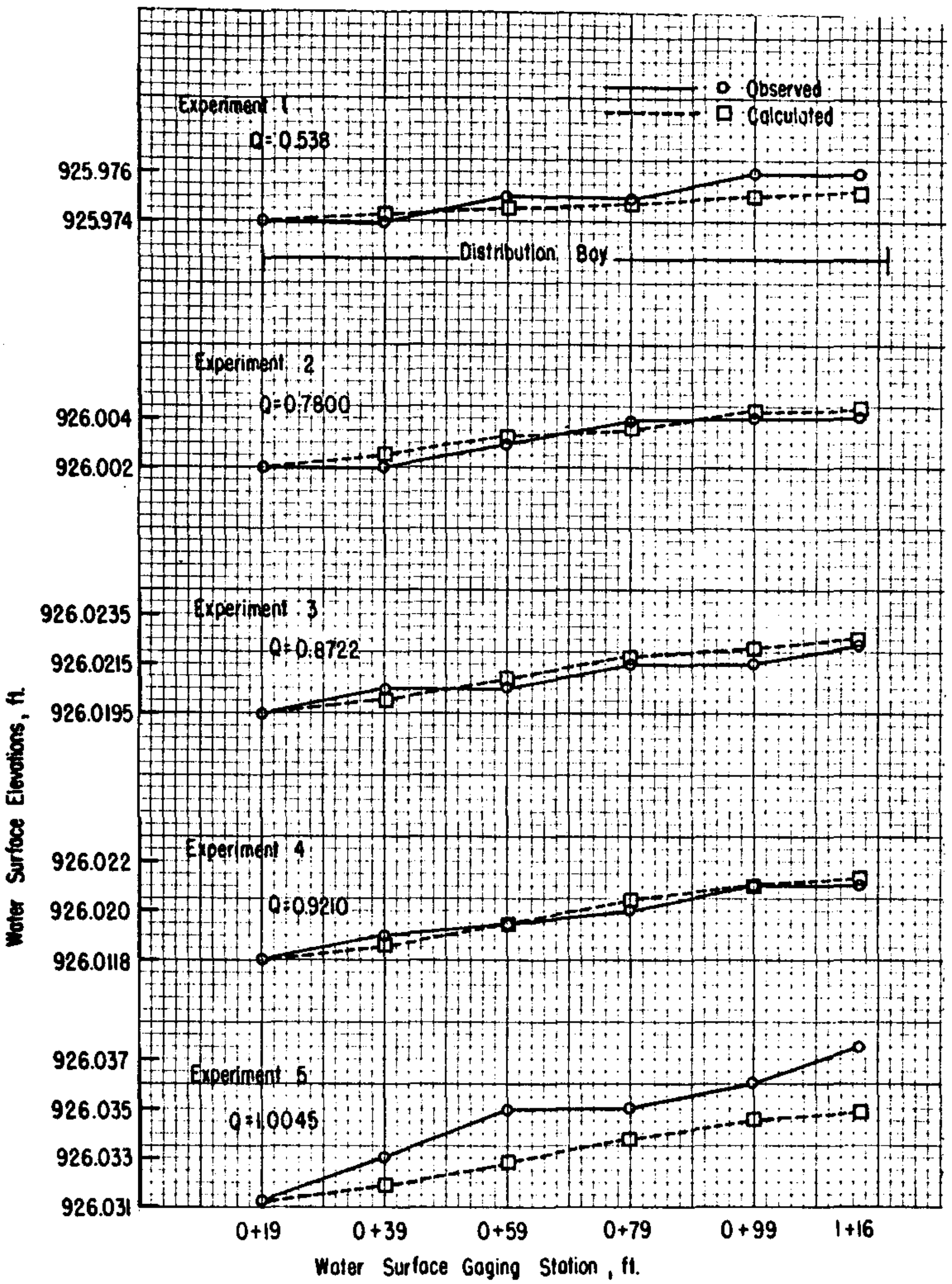

Figure 6. Observed and Calculated Water Surface Profiles for Decreasing Spatially Varied Flow in One Bay

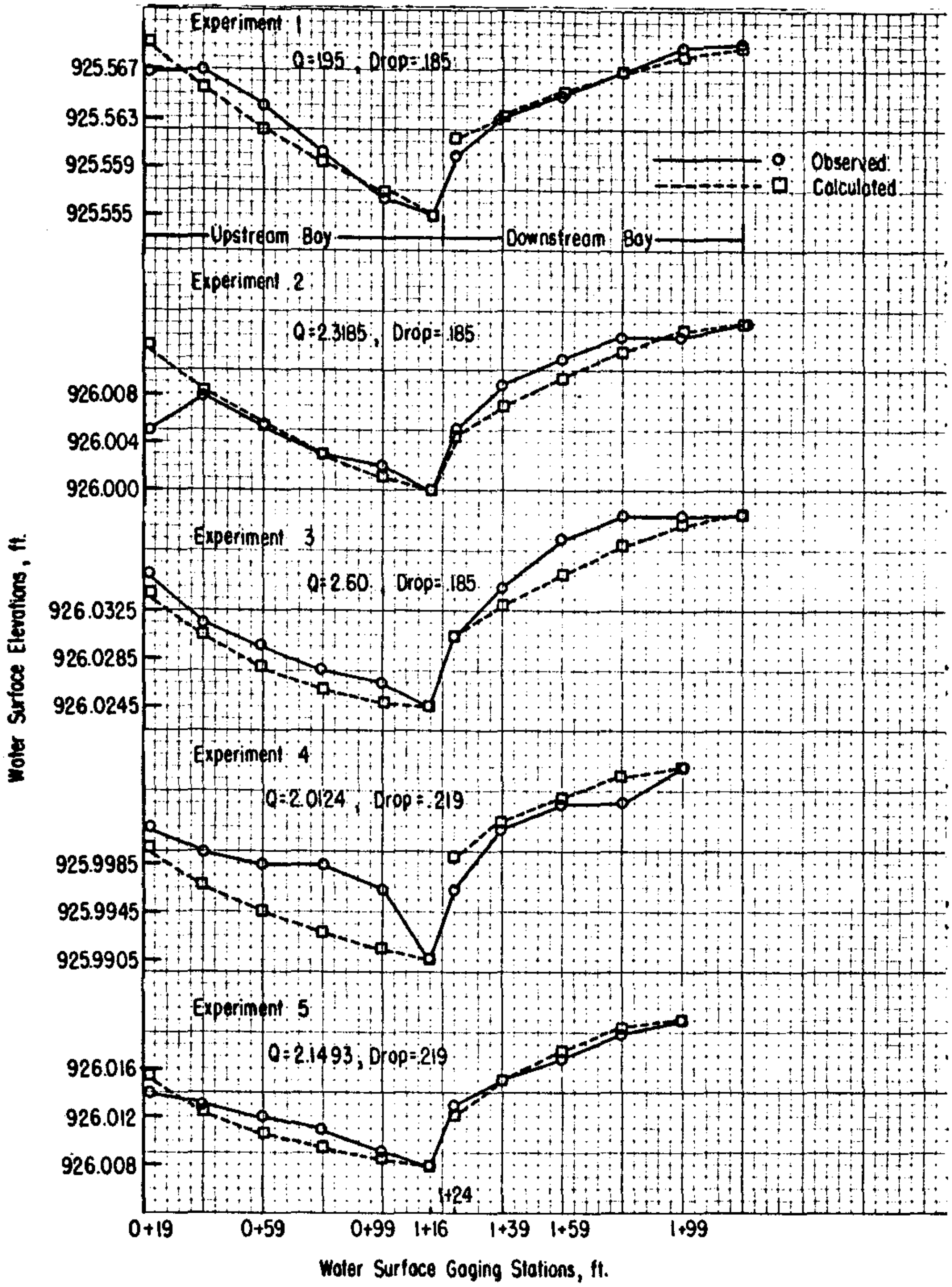

Figure 7 Observed and Calculated Water Surface Profiles for Decreasing Spatially Varied Flow in Two Bays

In all of the two bay tests, falling profiles were observed in the upstream bay. This was caused by friction losses exceeding velocity head recovery. Larger declines in the water surface profiles in the upstream bay were observed at lower inflow rates because friction losses predominated. Rising profiles were observed in the downstream bay and steeper profiles occurred as inflow increased as were observed in single bay tests. An overall rise in water surface elevation for the two bays occurred in all the tests. The drop between the bays produced a change in velocity head h_v which produced a rise in water surface elevation somewhat less than the change in h_v. This difference was due to turbulence and friction.

Variations in Orifice and Weir Discharge

One problem with such a channel was that orifice elevations varied in a random fashion along the length of the channel. The variation was slight, for example, usually no greater than 0.01 feet. But orifice elevation variation when combined with rising or falling water surface profiles produced variations in weir and orifice discharge along a bay.

Weir and orifice discharge uniformity was determined for each experiment in the following manner: During an experiment, in either bay, head, h, for six orifices was measured. Using the calibration curve, Figure 5, and the six value of h, the values of orifice or weir flow were computed. This represented a random sample of flows for one bay.

For a given bay, the percent variation V_D was computed from the equation

$$V_D = \frac{q_{max} - q_{min}}{q_{max}} \quad X \quad 100 \tag{10}$$

where q_{max} is the maximum orifice or weir flow in a bay and q_{min} is the minimum flow in a bay.

Larger variations were observed for weir flow as opposed to orifice flow and the largest variation occurred during the two bay tests in the upstream bay.

Generally as inflow increased, there was less variation in weir or orifice flow for both one bay and two bay tests.

Variations from 2.1 percent to 16.5 percent occurred in the single bay tests. For the two bays tests, the variation in the cut-back bay ranged from 11.2 percent to 26.10 percent. Variations in the initial flow bay ranged from 1.2 percent to 6.3 percent.

SUMMARY

A sheet metal flume for automated cut-back irrigation was designed hydraulically and structurally and constructed using the best design of those tested.

The optimum hydraulic section was designed. A hydraulic design table and nomograph solution were computed. These design aids enable a planner to accurately select the best system for a given set of conditions.

Preceding the structural analysis, several types of designs were investigated considering the problems of ease of assemblage in the field, support of the channel in the field, and leakage. The basic design consists of a sheet metal section supported by a structural steel angle framework. Taking the basic framework selected, several combinations of material design were considered and investigated theoretically. From these investigations three structural steel angle frameworks were built and three sizes of sheet metal sections chosen. Six combinations were tested and a final design was selected.

A 230 foot section was assembled in the field for testing. Values of Manning's n were determined from gradually varied flow tests for various flows and depths of flow. Decreasing spatially varied flow tests were run in one and two bays. Values of roughness coefficients $\bar{n}$ and n_e were determined, orifice and weir calibration were determined, and discharge and uniformity was calculated.

The average values of n, $\bar{n}$, and n_e were computed for design purposes in initial flow bays or single bays were $n = 0.0096$, $\bar{n} = 0.0126$, and $n_e = 0.00730$. The value of $\bar{n}$ for cut-back bays was $\bar{n} = 0.01065$

The equations for weir and orifice flow were determined; for weir flow, $q = 387.97h^{1.843}$ and for orifice flow, $q = 55.62h^{0.741}$, with h (ft) measured from the bottom of the orifice and q is gpm.

Uniformity of discharge was determined for the single and two bay tests. Variations from 2.1 percent to 16.5 percent occurred in single bay tests. The median of the values was 8.29 percent. For the two bay tests, the variation in the cut-back bay ranged from 11.2 percent to 26.1 percent with a median value of 21.5 percent. Variation in the initial flow bay ranged from 1.2 percent to 6.3 percent with a median value of 3.16 percent.

Water surface profiles were plotted from the observations made in the field. Water surface profiles were calculated for each test, using the average values of $\bar{n}$, and plotted for both single and two bay tests.

CONCLUSIONS

1. The hydraulic cross-section selected was a reasonable compromise.

2. Structurally the channel functioned well. The supporting system was a functional design. The structural strength versus weight combinations seemed to satisfy both the functions of strength and portability.

3. Orifice and weir calibration curves obtained compared favorablly with Barefoot's data for a 2.00 inch orifice.

4. Calculated and observed water surface profiles agreed closely. Thus, $\bar{n} = 0.0126$ for the initial flow bay and $\bar{n} = 0.01065$ for the cut-back flow bay were satisfactory for design purposes. This also confirmed the validity of Sweeten's (7) derivation of the equation for computing water surface profiles in two bays with a drop between them.

5. Measured values of velocity head conversion between bays were less than the theoretical values because of turbulence losses.

6. The ratio of $\bar{n}/n_{gvf} = 1.31$ for single bay tests compared with Sweeten's (6,7) values of $\bar{n}/n_{gvf}$ for single bay tests run under similar conditions using siphon tubes and side weirs in a concrete trapezoidal irrigation channel. Sweeten obtained values of $\bar{n}/n_{gvf} = 1.28$ for siphon tube experiments, and 1.25 for side weir experiments.

Report on Phase III of the Project

A FIELD STUDY OF AN AUTOMATED IRRIGATION FLUME[1]

by

John H. Edwards and James E. Garton

INTRODUCTION

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 (1).

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. One exception is a semi-automated cut-back furrow irrigation system developed by Garton (7). This cut-back furrow irrigation system dealt with the problem of non-uniform flow in a distribution bay. Instead of having a sloping channel, the system was constructed as a series of horizontal bays, staircased down the slope of the field. A near horizontal water surface occurred in each bay. For outlet tubes at the same elevation, very uniform furrow flows resulted.

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.

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.

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Uhl (23) designed an automated semi-portable sheet metal flume that functioned as a cut-back system. The only differences in Garton's and Uhl's designs were that the latter used side orifices for furrow discharge and was portable. Since Uhl's design was portable, it could be adaptable to a change in Q and could be relocated to cope with changes in cropping patterns from year to year. To determine the feasibility of Uhl's design, a field study was conducted in the Oklahoma Panhandle.

Objectives

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.

THEORY

Hydraulic Characteristics of Furrow Flow

The fluid flow of surface irrigation is a case of unsteady, non-uniform, open channel flow over a porous bed (12, 21). It is practically impossible to obtain an exact mathematical solution which includes all the pertinent quantities. However, it is possible to obtain approximate solutions.

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

$$x = aT^b \tag{1}$$

where

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 (1), these factors are assumed to be constant throughout the length of the run.

Using the following known relationships, an equation for total inches applied to the wetted portion of the field can be determined.

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 is

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

where

d = depth applied (inches)

gpm = furrow flow rate (gpm)

Using equation (2) and the rate of advance data, the depth applied for each station was determined.

Spatially Varied Flow

Irrigation systems are a form of decreasing spatially varied flow, where water is taken out or discharged along the reach. Mink (14) and Sweeten (19) in previous research investigated decreasing spatially varied flow in a horizontal concrete distribution channel.

Mink (14) found that the values of Manning's n for gradually varied flow were lower than the values of n for spatially varied flow. He defined a roughness coefficient called $\bar{n}$ which is an adjusted value of Manning's n. When used with the Manning formula and the energy equation, $\bar{n}$ will produce the actual water surface profiles for decreasing spatially varied flow.

Sweeten (19) derived an equation which provided a direct solution for water surface profiles. According to his research, only velocity head gain and friction head loss influence the rise or fall of flow profiles.

Thus the energy equation can be written:

$$\Delta WS = \Delta H_v - H_f \tag{3}$$

where

ΔWS = change in water surface elevation (positive for rising profiles)

ΔH_v = velocity head recovery

H_f = energy loss due to hydraulic resistance

By developing the components of H_f and H_v and substituting them into Equation (3), Equation (4) was determined to be:

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

where

X = distance from the upstream end of the distribution bay

L = length of the distribution bay

V_i = upstream velocity

R = upstream hydraulic radius.

Equation (4) can be used to solve water surface profiles in a two bay system. The solution for the upstream bay depends on the virtual length (L_1'), the length of bay necessary to completely discharge the inflow, is:

$$L_1' = L_s\left(\frac{Q}{q_c}\right) \tag{5}$$

where

L_s = orifice or weir spacing

Q = inflow

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

SHEET METAL FLUME

Hydraulic Design

Uhl (23) adopted stability of water surface profiles as the main hydraulic design criteria. He found the rectangular section to be most feasible, having a bottom width of 1.50 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. Uhl (23) chose an orifice height of 0.75 feet, allowing 0.40 feet for initial and cut-back flows, and 0.10 feet for freeboard.

Structural Design

According to Uhl (23) the objective of the structural design was to develop a channel section which was light, easily handled, portable, leak free, easily assembled, and strong enough to withstand permanent bending and buckling. Furthermore, it was necessary to support the channel above the ground to enable minor adjustments to be made in the field. The final design selected by Uhl was a 24-gage sheet metal section, supported by a 3/4" x 3/4" x 1/8" framework with 1/2" x 1/2" x 1/8" side and bottom braces. The design was chosen because of its advantages of weight, strength, and portability. The supporting system consisted of a 1/2" steel rod, welded to a 3" x 3" x 1/8" base. The two rod-base assemblies were fastened to a redwood base 26" x 8" x 1".

EXPERIMENT

The tests were divided into two parts: preliminary tests, conducted in two segments, and final field tests.

Part one of the preliminary tests consisted of rate of advance tests. The field slope in the direction of the flume and supply flow from the well were also determined at this time.

Upon completion of the design and the installation of the system at Guymon, the second part of the preliminary tests were conducted. These tests included rate of advance and measurement of variation in furrow discharge.

After completion of the preliminary tests, the system was redesigned and set up for final field tests. These tests involved rate of advance, variation in furrow discharges, and measurement of water surface profiles.

ANALYSIS OF DATA

Preliminary field tests were conducted in 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 flume installed after the first irrigation, was used for two irrigations.

In the summer of 1971, final field tests were conducted, following the redesigning and installation of the irrigation flume. The channel was set up before the first irrigation and used for all of the waterings.

Rate of Advance (Part One)

Several test furrows were chosen. It was assumed that these furrows were a true representation of the field. Flow rates, varying from 30.50 g.p.m. to 4.80 g.p.m., were released into the 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 (1). 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 soil type, and the variation of soil moisture along the furrow. The expression that best fit the data was a quadratic equation of the form

$$x = a + bT + cT^2 \tag{6}$$

where

x = distance water has advanced (feet)

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

a, b, c = constants

Figure 1 is a plot of the best fit equations to the rate of advance data collected during part one of the preliminary tests. The 30.50 g.p.m., 19.00 g.p.m., and 16.70 g.p.m. discharges watered through the field in a relatively short period of time. The 10.00 g.p.m. flow advanced approximately three-fourths of the furrow length, while the 4.80 g.p.m. flow progressed only one-third of the furrow length. The curves in Figure 1 were helpful in determination of the desired initial furrow flow rates for the system's design.

Depth Applied

Using Equation (2) and the rate of advance data collected during part one of the preliminary tests, the relationship between depth (d) and time (T) for the different discharges was determined. Figure 2 is a plot of the equations fitted to the depth-time data. The 19.00 g.p.m., 16.70 g.p.m., and 10.00 g.p.m. curves were essentially identical. Thus, the depth

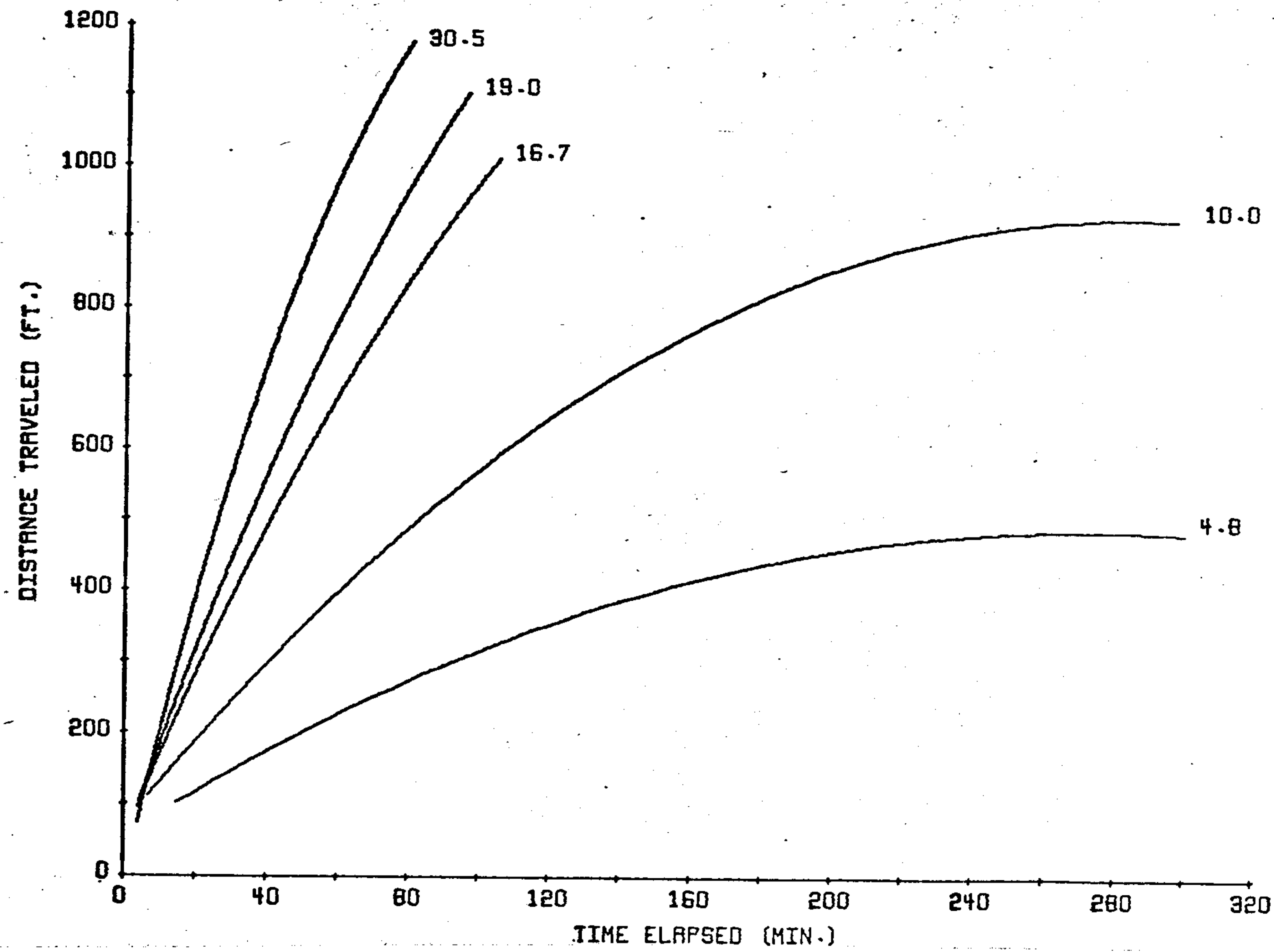

Figure 1. Distance Versus Time for Various Furrow Discharges Represented in g.p.m.

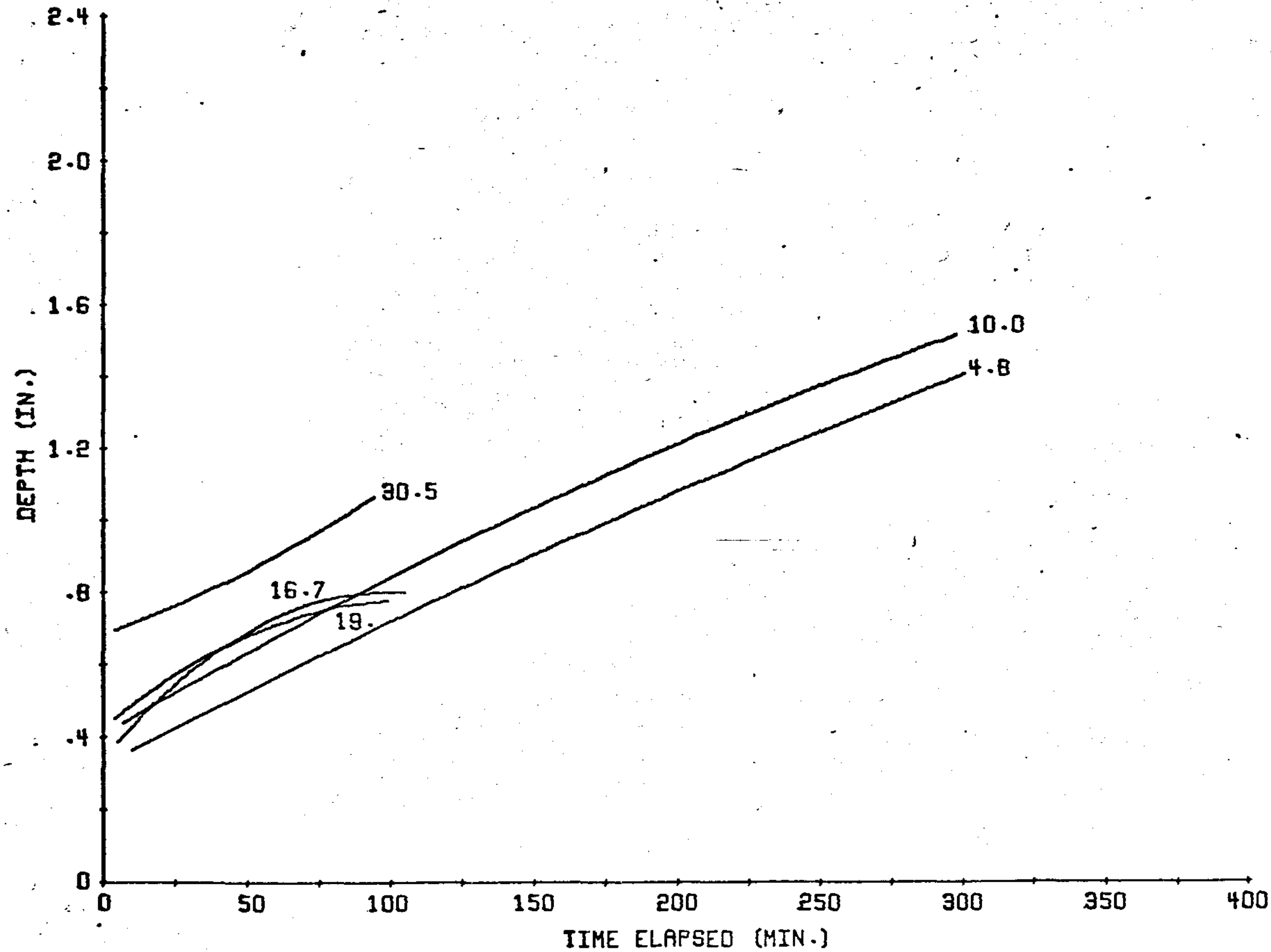

Figure 2. Depth Versus Time for Various Furrow Discharges Represented in g.p.m.

versus time relationships were independent of discharges between 10.00 g.p.m. and 19.00 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.

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. Weighted drums may be hooked to the back of the planter and dragged through the field to compensate for the compactive effect of the tractor tires. By doing this, the farmer hopes to obtain a uniform application rate between rows.

Figure 3 is a plot of depth-time data collected from two furrows. One furrow was compacted by a tractor tire and drum, while the other was compacted by a drum only. 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 furrow advanced at a faster rate.

Figure 4 is a plot of depth-time data collected from two furrows with no drum compaction. One furrow was compacted by a tractor tire, while the other furrow received no tire compaction. 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% 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. This large variation in application uniformity resulted in a large percentage of run-off occurring from the tractor tire rows.

Comparing Figures 3 and 4, it was apparent that the application uniformity between furrows was significantly higher when drums were dragged behind the planter.

Equations were fitted to the depth-time data for each irrigation of 1971. These equations were plotted in Figure 5 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 5.

Variations in Initial and Cut-Back Discharges

Larger variations were observed for cut-back flows as opposed to orifice flows during both the preliminary and final field tests.

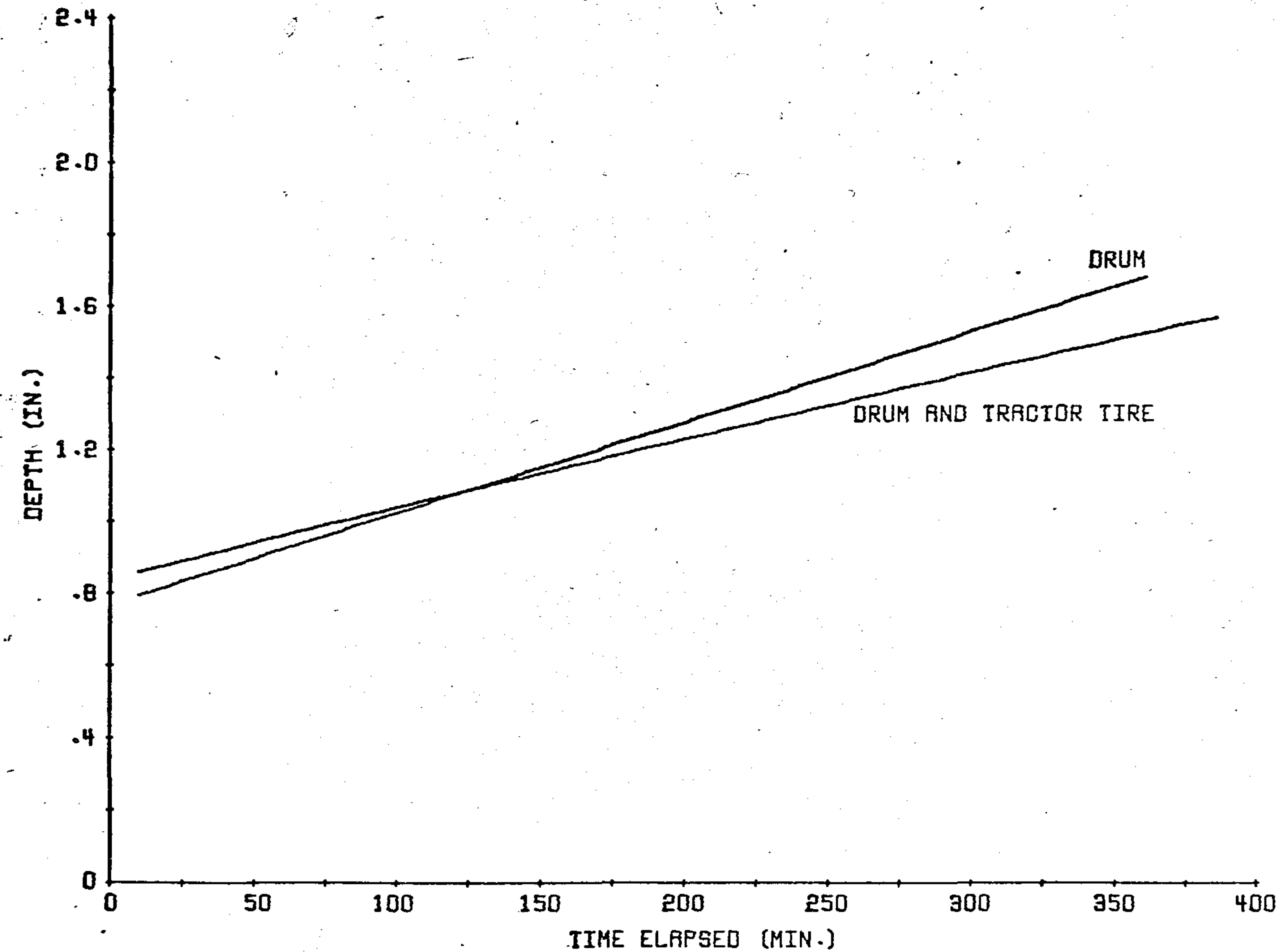

Figure 3. Depth Versus Time for Bay 1 During the Second Irrigation, Summer, 1970

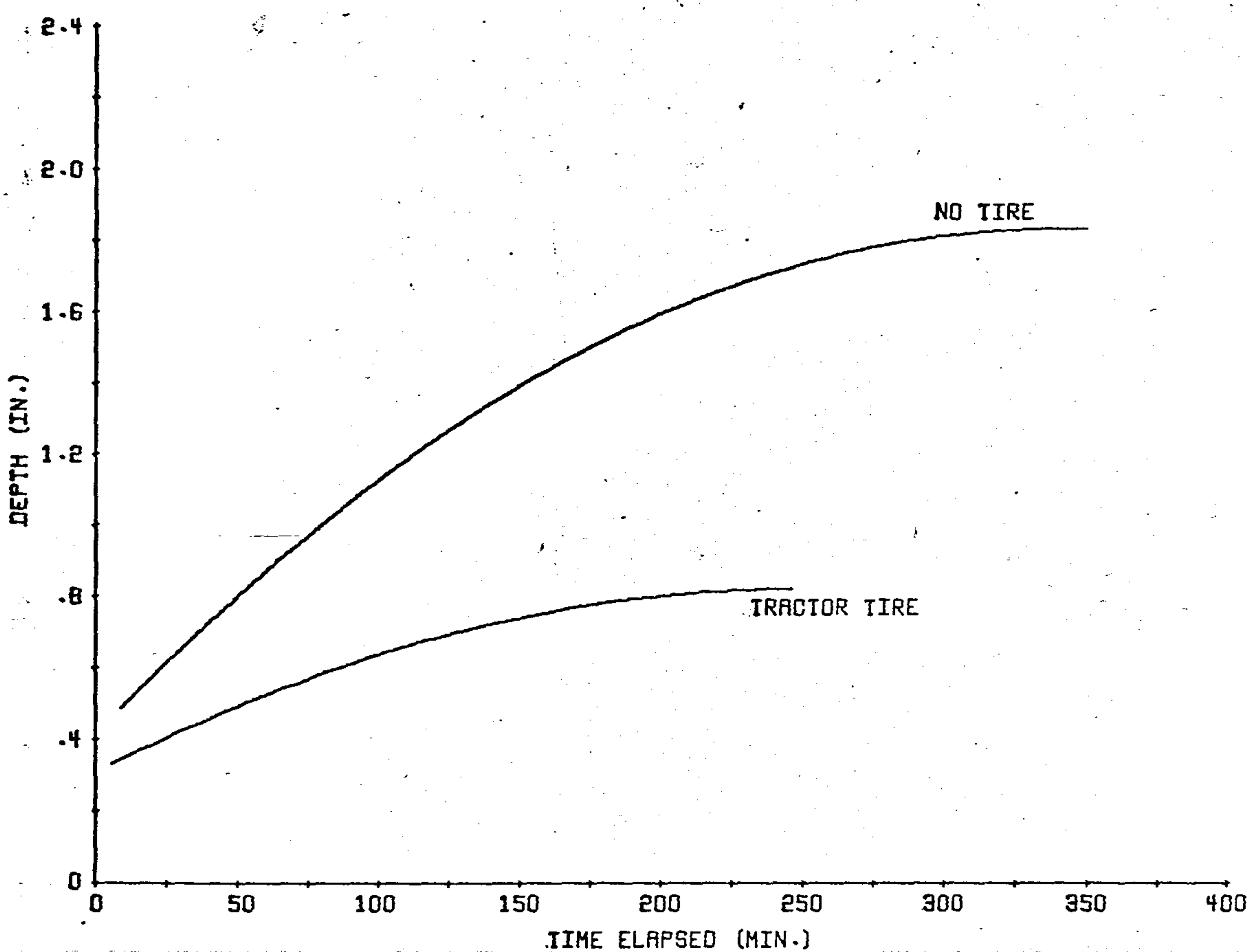

Figure 4. Depth Versus Time for Bay 1 During the Second Irrigation, Summer, 1971

The greatest variation was observed as 4.20 g.p.m. The largest standard deviation was calculated to be 1.17 g.p.m. Table I illustrates this variation in discharges for the first irrigation of 1971.

For the two irrigations of 1970 and the first three irrigations of 1971, 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 of 1971 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 discharges were slightly above the design's cut-back discharge.

Water Surface Profiles

Figure 6 is a plot of observed and calculated profiles for the first irrigation of 1971. 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 losses, resulting from kinetic energy being converted to potential energy. Falling profiles occurred in bays three and four while at initial flow. It was assumed that this was caused by excessive weathering of channel sections in the two bays. 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 discharges. Equations (4) and (5) were used to calculate the water profiles plotted in Figure 6.

TABLE I

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

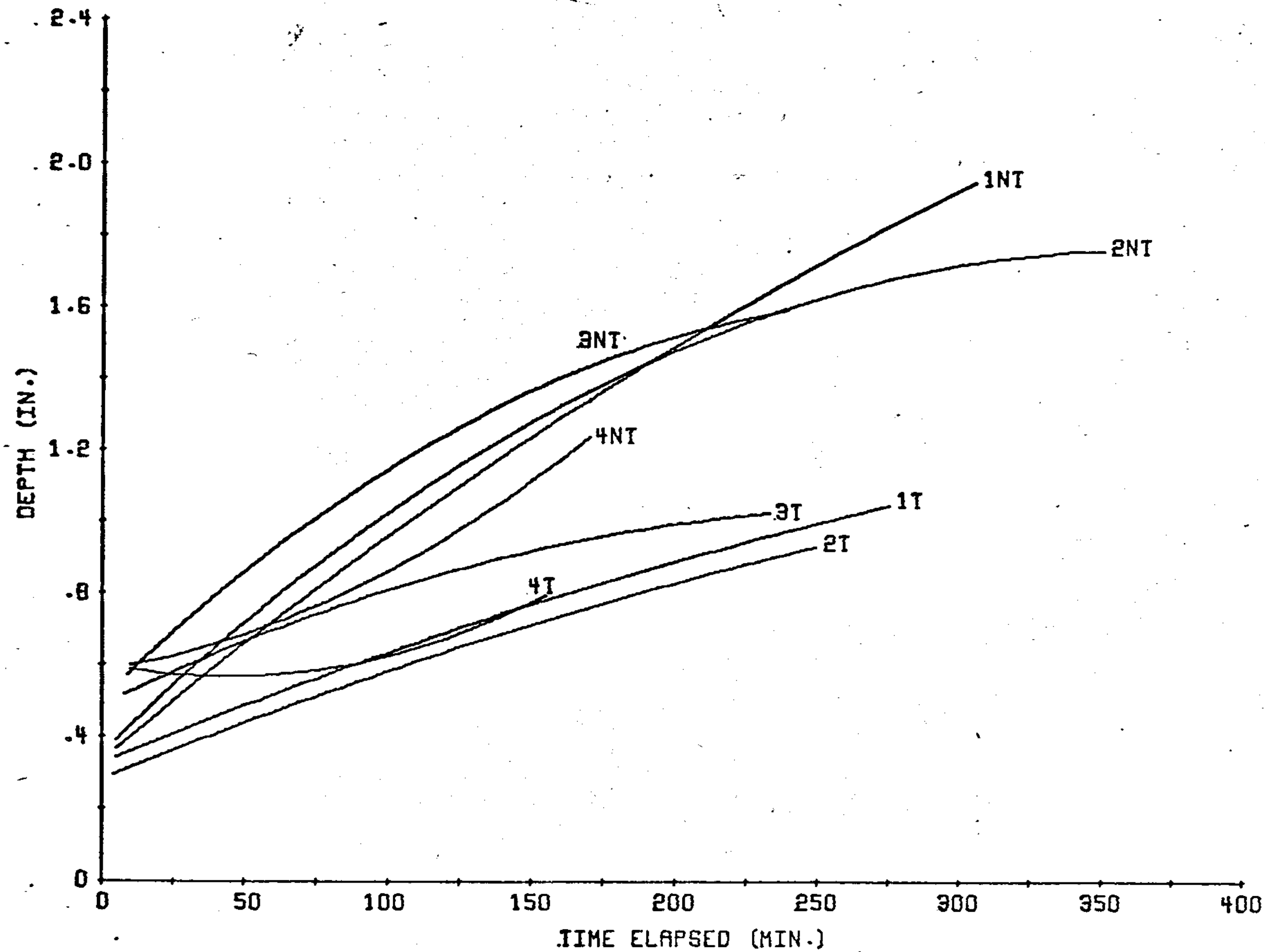

Figure 5. Depth Versus Time for the 1971 Irrigations. The Numbers Indicate First, Second, Third, and Fourth Irrigations. The Symbol (T) Represents Tractor Tire Furrow While NT Represents No Tire Furrow

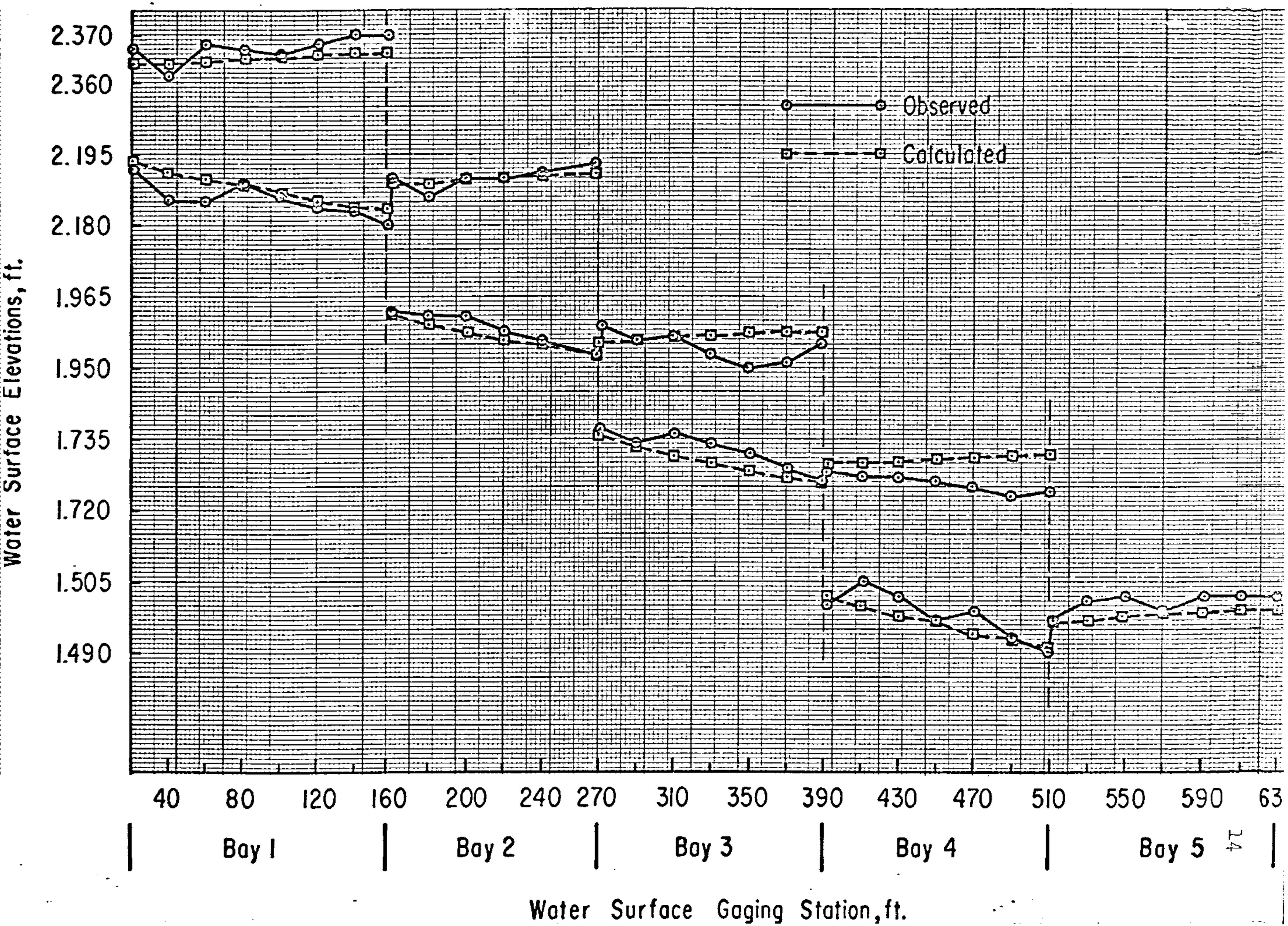

Figure 6. Observed and Calculated Water Surface Profiles for the First Irrigation

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 10.00 g.p.m. and 19.00 g.p.m.

During the preliminary and final field tests, depth-time relationships were used to study furrow characteristics of each irrigation. Application uniformity between furrows was increased by dragging weighted drums behind the planter.

Initial and cut-back furrow discharge uniformities were determined during the preliminary and final field tests. In every case, larger variations were observed for cut-back flows as opposed to orifice flows. The greatest variation was observed as 4.20 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.

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.

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