Do Not Remove From Folder

RESEARCH PROJECT

TECHNICAL COMPLETION REPORT

Submitted to the

Director,

Water Resources Research Institute

Oklahoma State University

August 19, 1968

Reduction of Water Application Losses Through Improved Distribution Channel Design

OWRRI Project Number A-004 Oklahoma

Agreement Number 14-01-0001-1404

by

James E. Garton
Principal Investigator
Professor of Agricultural Engineering
Oklahoma State University
Stillwater, Oklahoma 74074

and

John M. Sweeten, Jr.
Graduate Research Assistant
Oklahoma State University
Stillwater, Oklahoma 74074

The work upon which this report is based was supported by funds provided by the United States Department of the Interior, Office of Water Resources Research, as authorized under the Water Resources Research Act of 1964.

ABSTRACT

These studies were directed toward improving the uniformity of application of water with furrow irrigation. Increased efficiency of application can best be achieved by improving uniformity.

Values of Manning's n were obtained with gradually varied flow in a level ditch without siphon tubes and for a ditch with unprimed siphon tubes with different diameters, spacing, and submergence. Relationships of Manning's n to the variables were determined.

The tubes were primed and values of Manning's n were determined for various diameters, spacing, and submergence with spatially varied flow.

The results of these studies were used to determine the uniformity of water application which could be expected with various ditch slopes and outlet arrangements.

A method was derived for calculating the water surface elevations at various points in a level distribution bay without resorting to a computer solution.

Various diameters of circular weirs and orifices were studied to determine the head-discharge relationship for weirs with a 45 degree slope operating as side discharge devices.

Rectangular weirs of various length were studied to determine the head-discharge relationship for weirs with a 45 degree slope.

KEYWORDS: Furrow Irrigation/ *Irrigation Canals/ *Irrigation Design/ Irrigation Engineering/ Hydraulic Engineering/ *Laboratory Tests/ Open Channels/ Mannings Equation

RESEARCH PROJECT TECHNICAL COMPLETION REPORT
OWRR PROJECT NUMBER A-004 OKLAHOMA

Project Title: Reduction of Water Application Losses Through Improved Distribution Channel Design

Agreement Number: 14-01-0001-1404

Principal Investigators:

James E. Garton, Ph.D., Agricultural Engineering

John M. Sweeten, M. S., Agricultural Engineering

Institution: Oklahoma State University, Stillwater, Oklahoma

Project Began: May, 1965 Scheduled Completion: June, 1968

Introduction

About one-half of the water used in the United States is utilized in irrigation. Researchers (5, 6) have reported that less than 40 percent of the water diverted from its supply source is placed in the plant root zone when the application depth is four to five inches.

Non-uniform water distribution is a major contribution to water losses in furrow irrigation, since water is usually applied until the smallest furrow stream has reached the downstream end of the field. The large furrow streams cause excessive deep percolation and runoff losses to occur. The accurate design of distribution channels with precise metering devices would greatly improve uniformity of irrigation. Most irrigation distribution channels are incorrectly designed because of the use of equations developed for uniform and gradually varied flow.

Since withdrawal of water into furrows causes a decrease in discharge along the length of the channels, irrigation distribution channels should be designed using the theory of decreasing spatially varied flow. However, the water surface profile in such channels could not be accurately computed for design purposes since energy losses in distribution channels had not been evaluated.

Besides distribution uniformity, labor requirements, working conditions, and the scarcity of labor continue to impede the growth and often determine the success of surface irrigation. The usual methods of diverting water from the distribution channel into furrows are siphon tubes (usually with nonuniform outlet elevation) and notches cut through the side of an earthen ditch. Both methods require the constant presence of one or more laborers and make uniform irrigation a practical impossibility.

The research performed under Project Number A-004 Oklahoma was aimed at helping to solve the problems of non-uniform water distribution and high labor requirements which plague surface irrigation. The major areas of work under the project were as follows:

1. Evaluating energy losses in distribution channels under conditions of decreasing spatially varied flow and developing design criteria and procedures so that distribution channels can be designed for greater irrigation uniformity.

2. Determining the discharge characteristics of precise furrow metering devices such as hooded inlet tubes, weirs, and orifices so that automatic cutback surface irrigation systems can be designed and constructed.

These research areas will be discussed separately in the following sections.

Hydraulic Roughness of Concrete-Lined Irrigation Channels

Siphon tube irrigation is a form of decreasing spatially varied flow. Uniform irrigation with siphon tubes requires that each tube have the same operating head. The channel roughness by influencing the profile of the water surface in the channel affects the head on each siphon tube. The longitudinal slope of the channel affects the siphon tube head by influencing the water surface profile and by limiting the outlet elevation of the tubes.

The hypothesis was stated that a level distribution channel with the siphon tube outlets placed at the same elevation would result in uniform water distribution. Gradually varied steady flow experiments were conducted to determine the hydraulic properties of a concrete-lined irrigation channel both with and without siphon tubes. Decreasing spatially varied steady flow experiments were conducted to evaluate the hydraulic roughness and irrigation uniformity of a horizontal channel with discharging siphon tubes whose outlets were placed at the same elevation.

Objectives

The objectives of this research project were:

1. To determine if the roughness coefficients obtained from gradually varied flow experiments could be used to predict water surface profiles for decreasing spatially varied flow.
2. To determine, if necessary, new roughness coefficients to satisfy the spatially varied flow conditions.
3. To modify, if necessary, the present theory of decreasing spatially varied flow to more readily predict the measured water surface profiles.
4. To evaluate siphon tube discharge uniformity in a horizontal channel with the tube outlets placed at the same elevation.
5. To determine if a level irrigation channel distributes the water more uniformly than a sloping channel.

Apparatus

An experimental irrigation distribution channel, shown in Figure 1, was constructed and instrumented at the Outdoor Hydraulic Laboratory at Stillwater, Oklahoma. The straight test section of the 320-foot long concrete-lined channel was designed for a 2-foot depth, 1-foot bottom width, and 1:1 side slopes. The bottom as placed had an adverse slope of 0.022 percent.

Eleven 10-inch diameter gauge wells, interconnected to the channel with ½-inch diameter plastic pipe were spaced at 30-foot intervals along the channel. The water surface elevations in the channel were measured with point gauges mounted inside the gauge wells.

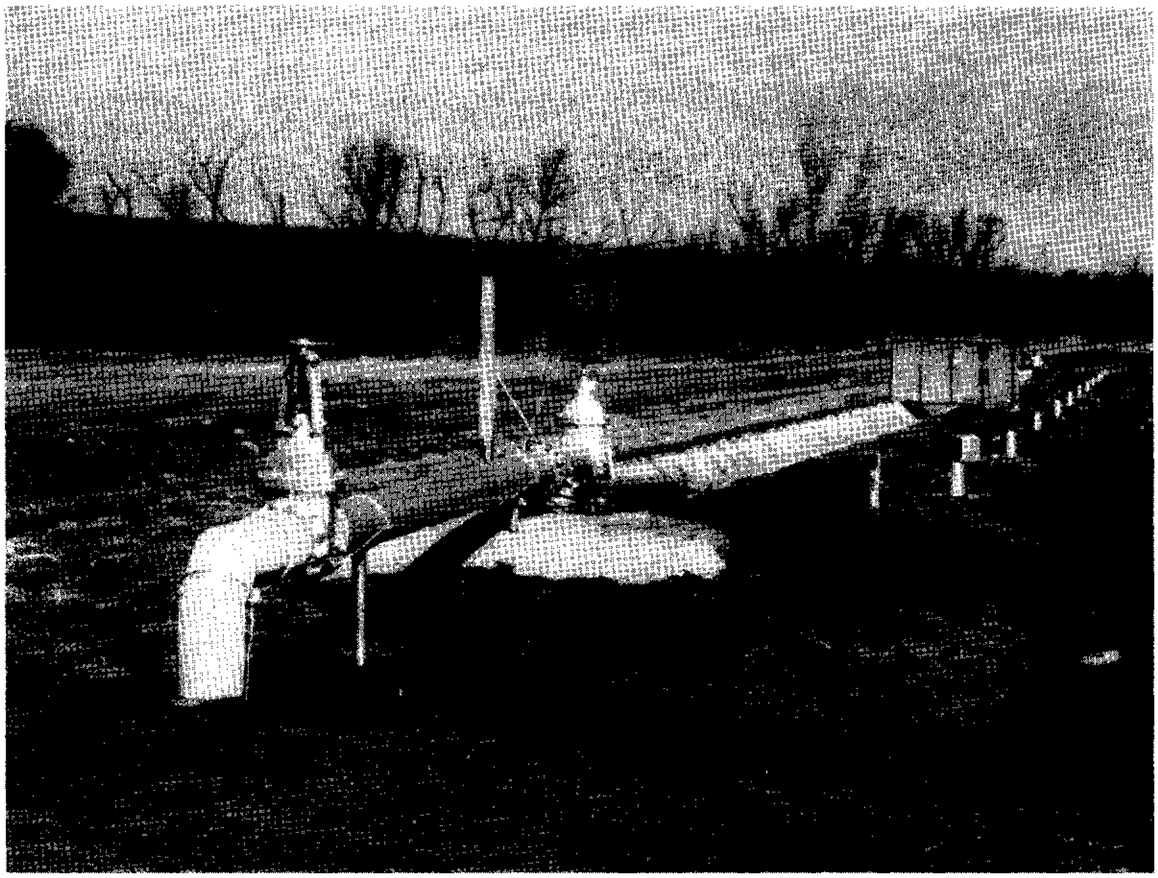

Figure 1. A General View of the Experimental Setup Showing the Water Supply Line, Orifice, Manometer, Gauge Wells, and Channel With the 2 in. Tubes in Place for a Roughness Test

The inflow, supplied through a 12-inch pipe, was measured with calibrated orifice plates and a 60-inch water-air manometer. Most of the inflow turbulence was dissipated by cross-sectional and surface stilling devices. A canvas check dam, adjusted with a small winch and cable arrangement, controlled the depth at the downstream end of the channel.

Double bend plastic siphon tubes were primed to create spatially varied flow conditions. The tube outlets as shown in Figure 2 were mounted on the top surfaces of structural steel angles, which were adjusted to the same elevation using specially built mounts. The siphon tubes were restrained to the channel wall at the desired spacing and vertical inlet location using fine steel wire fastened to anchor screws.

A portable apparatus was built for the purpose of taking cross-sectional velocity profiles at selected stations along the channel. Pictured in Figure 3, this apparatus consisted of an 8-inch aluminum H-beam, carriage, point guage, Ott propeller current meter, and a F-4 digital counter.

Accessory equipment included (1) light durable channel covers to reduce wind effects, (2) a FW-1 stage recorder for observing flow stability, and (3) a portable gasoline powered pump for priming the siphon tubes.

Procedures

Referencing - Accurate referencing of the channel bottom and the mounted point gauges was essential for obtaining the desired degree of precision in the data analysis. At each of the eleven stations brass plugs had been embedded in the

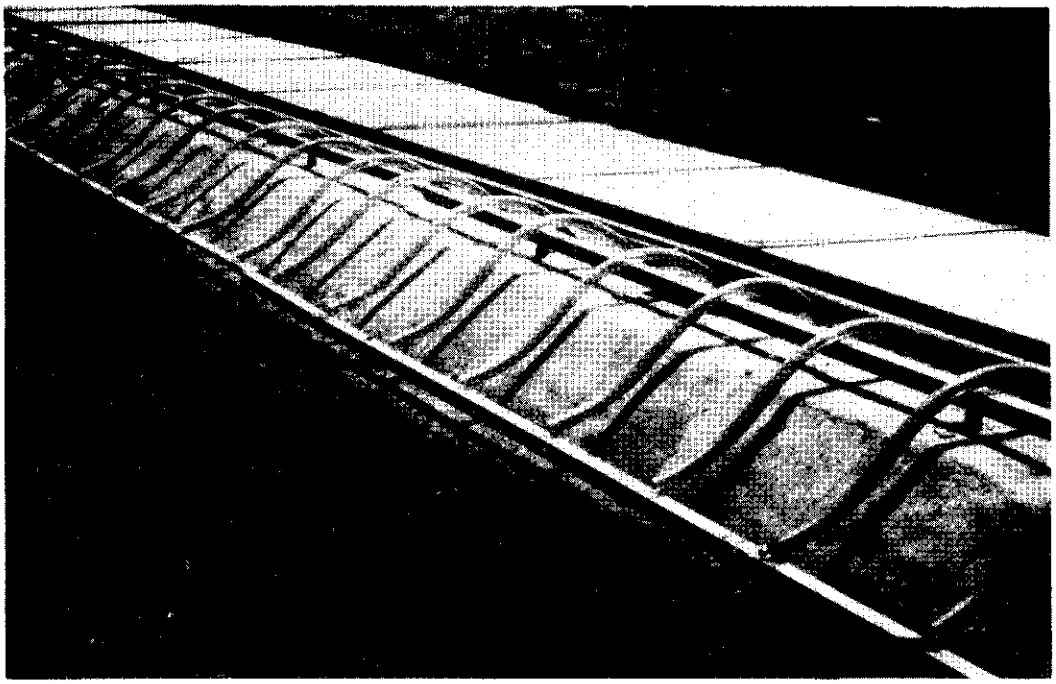

Figure 2. Siphon Tube Outlets Placed at the Reference Elevation, with Wind Panels Installed in the Background.

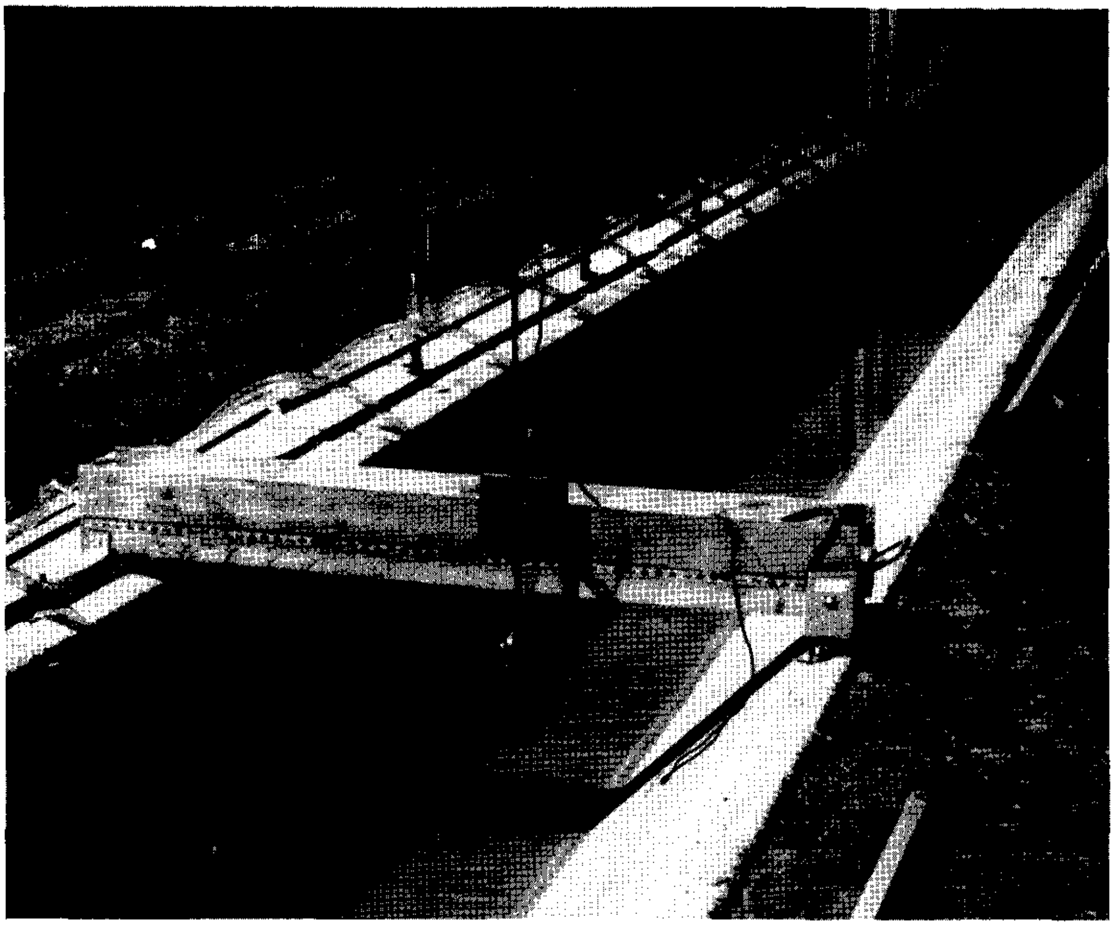

Figure 3. Point Velocity Measuring Apparatus. The Component Parts Are: 8 in. Aluminum H Beam, Carriage, 3 ft. Point Gauge, OTT Laboratory Current Meter, and OTT F-4 Revolution Counter

concrete channel bottom. The brass plug elevations were determined by surveying with an engineer's level and point gauge. Gauge zero elevations were determined for the mounted point gauges by surveying and by taking point guage readings on a level water surface. Fitted equations for the geometric elements of area, wetted perimeter, and top width were obtained from cross-sectional boundary traverses at each station.

Experimental Procedure - Various conditions of inflow, depth, and siphon tube placement were employed for both the gradually varied and spatially varied flow tests. Roughness conditions were established using (1) siphon tube spacings of 20, 40, 60, and 80 inches, (2) tube diameters of 0.75, 1.00, 1.25, 1.50, 2.00, and 3.00 inches, (3) tube inlet locations of 0.0, 0.5, and 1.0 feet above the channel bottom. An inlet location of 1.0 feet was employed for all spatially varied flow tests.

Essentially the same procedures of measurement were employed for the spatially varied flow (SVF) experiments and the gradually varied flow (GVF) tests. For the SVF tests, inflow was initiated, the check dam was raised, and the siphon tubes were primed with the portable pump. Adjustment of the 12-inch gate valve produced the desired manometer reading. Equilibrium between inflow and outflow was indicated by a straight line trace on the FW-1 recorder cylinder. Five point gauge readings were taken at each gauge well. For the GVF experiments, priming of the siphon was unnecessary. Many of these tests were conducted to duplicate the conditions of inflow, depth, and tube placement used in spatially varied flow test.

Velocity profiles were taken for most of the experiments by measuring point velocities at 0.2 foot horizontal and vertical intervals in a cross-section. The number of meter revolutions in a 15-second interval were recorded and converted to flow velocity at each point.

Theory for Spatially Varied Flow

The accurate computation of water surface profiles for decreasing spatially varied flow requires the knowledge of the correct magnitude of a suitable roughness coefficient. The relationship between water surface profiles and hydraulic roughness can be established using the Manning equation to compute the energy slope in the general energy equation.

Adjusted $\bar{n}$ - For spatially varied flow the roughness coefficient $\bar{n}$ can be defined as that value of Manning's n which, when used in conjunction with the Manning formula and the energy equation, will produce the actual water surface profile for spatially varied flow. For a channel of small bottom slope, the energy equation can be written as (2):

$$\alpha_2 \frac{V_2^2}{2g} + Y_2 + Z_2 = \alpha_1 \frac{V_1^2}{2g} + Y_1 + Z_1 + S_f \, \Delta X \qquad (1)$$

where

α = energy coefficient

V = mean velocity, ft/sec

g = acceleration of gravity, ft/sec^2

y = flow depth, ft

Z = bottom elevation, ft

S_f = energy gradient

ΔX = incremental channel length, ft

and the subscripts 1 and 2 refer to downstream and upstream cross-sections, respectively. Substituting the Manning formula, the continuity equation, and the approximation $\alpha_1 \approx \alpha_2 \approx 1.00$ into equation (1) yields:

$$\frac{Q_2^{\,2}}{2gA_2^{\,2}} + Y_2 + Z_2 = \frac{Q_1^{\,2}}{2gA_2^{\,2}} + Y_1 + Y_2 + \frac{Q_2^{\,2}N^2 \;\Delta X}{2.208\; R_a^{\,4/3}\; A_a^{\,2}} \qquad (2)$$

where

Q_2 = discharge, cfs

$A_a = \frac{1}{2}\,(A_1 + A_2)$ = average area, ft^2

$R_a = \frac{1}{2}\,(R_1 + R_2)$ = average hydraulic radius, ft

Equation (2) can be used to compute SVF profiles with an estimated value of Manning's n and a known downstream depth. The value of n which yields a profile matching the observed profile is the correct roughness coefficient for the given discharge, roughness condition, and depth. This correct magnitude of the roughness coefficient was designated $\bar{n}$ and is applicable only to decreasing spatially varied flow. It contains the effect of nonuniform velocity distribution.

Effective n - Another roughness coefficient for spatially varied flow can be defined to predict the water surface elevation at the downstream end of an irrigation bay. This coefficient, termed effective n or n_e , can be computed using the entering discharge, the entering depth, and the change in total energy head between the ends of the bay.

The effective energy gradient S_{fe} can be computed from the expression:

$$S_{fe} = \frac{1}{L} \left(\frac{V_i^2}{2g} + WS_i - WS_o\right) \tag{3}$$

where

L = length of the irrigation bay, ft

WS = Y + Z = water surface elevation, ft

Y = mean flow depth, ft

Z = bottom elevation, ft

and the subscripts i and o refer to the upstream and downstream ends of the irrigation bay. Effective n can then be calculated by substituting equation (3) into the Manning equation to yield:

$$n_e = 1.486 \frac{R_i^{2/3}}{V_k} \left[\frac{1}{L} \left(\frac{V_i^2}{2g} + WS_i - WS_o\right)\right]^{1/2} \tag{4}$$

in which

R_i = hydraulic radius of inflow section, ft

V_i = inflow velocity, ft

Direct Solution to Water Surface Profiles

For decreasing spatially varied flow in a horizontal irrigation channel, only two energy components--velocity head recovery and friction loss--influence the rise or fall of water surface profiles. The energy equation can be written as:

$$\Delta WS = \Delta H_v - H_f \tag{5}$$

where

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

ΔH_v = velocity head recovery, ft

H_f = energy loss due to hydraulic resistance, ft

In a level primatic channel in which the depth and the unit outflow are nearly constant, the potential energy gain due to diminishing velocity head will be:

$$\Delta H_v = \frac{V_i^2}{2g} - \frac{V_i^2}{2g} \left(\frac{L - X}{L}\right)^2 \tag{6}$$

where

X = distance from the upstream end of the irrigation bay, ft

The first term on the right in equation (6) is the initial velocity head while the second term is the velocity head at any location X. When evaluated at some $X = X_1$, equation (6) becomes:

$$\Delta H_v = \frac{V_i^2}{2g} \left(\frac{2X_1}{L} - \frac{X_1^2}{L^2}\right) \tag{7}$$

The velocity head recovery described by equation (7) will be offset by energy losses. The energy gradient S_{fi} at the inflow section of the irrigation bay is given by:

$$S_{fi} = \frac{n^2 \, V_i^2}{2.208 \, R_i^{4/3}}$$

At any location X, if the depth and the discharge per unit length of the channel are nearly constant, the energy gradient becomes:

$$S_{fx} = \frac{n^2 \; V_i^{\,2}}{2.208 \; R_x^{\,4/3}} \; \left(\frac{L - X}{L}\right)^2$$

The mean energy gradient $\overline{S}_f$ over the distance $X = X_1$ is obtained from the following integration:

$$\overline{S}_f = \frac{1}{X_1} \int_o^{X_1} S_{fx} \, dx$$

Substituting for S_{fx} using $\overline{n}$ and the mean hydraulic radius R produces:

$$\overline{S}_f = \frac{\overline{n}^2 \; V_i^{\,2}}{2.208 \; R^{4/3}} \; \frac{1}{X_1} \int_o^{X_1} \left(\frac{L - X}{L}\right)^2 \; dx$$

Integration of this relationship yields the equation for energy loss in the distribution bay:

$$H_f = \overline{S}_f \; X_1 = \frac{\overline{n}^{\,2} \; V_i^{\,2}}{2.208 \; R^{4/3}} \left(X_1 - \frac{X_1^{\,2}}{L} + \frac{X_1^{\,3}}{3L^2}\right) \qquad (8)$$

Substituting equations (7) and (8) into equation (5) produces a direct solution to SVF water surface profiles in a horizontal channel:

$$\Delta WS_x = \frac{V_i^{\,2}}{2g} \left(\frac{2X_1}{L} - \frac{X_1^{\,2}}{L^2}\right) - \frac{\overline{n}^{\,2} \; V_i^{\,2}}{2.208 \; R^{4/3}} \; \left(X_1 - \frac{X_1^{\,2}}{L} + \frac{X_1^{\,3}}{3L^2}\right) \quad (9)$$

The quantity ΔWS_x in equation (9) is the change in water surface elevation between the upstream end of the distribution bay and the point $X = X_1$.

When X = L, equation (9) reduces to:

$$\Delta WS_L = \frac{V_i^{\,2}}{2g} - \frac{\overline{n}^{\,2} \; V_i^{\,2} \; L}{6.624 \; R^{4/3}} \qquad (10)$$

from which ΔWS_L can be predicted for a known roughness channel length, and inflow conditions. The same value of ΔWS_L can be obtained from equations (3) and (4) using n_e as the roughness coefficient.

<u>Relationship Between $\bar{n}$ and n_e</u>

The energy loss H_f between the ends of an irrigation bay can be estimated using both the mean and effective energy slopes. Consequently, at X = L,

$$H_f = \bar{S}_f \, L = S_{fe} \, L$$

Substituting equations (3), (4), and (8) evaluated at X = L into the relationships results in:

$$H_f = \frac{\bar{n}^2 \, V_i^{\,2}}{2.208 \, R^{4/3}} \; (\frac{L}{3}) = \frac{n_e^{\,2} \, V_i^{\,2} \, L}{2.208 \, R_i^{\,4/3}}$$

For decreasing spatially varied flow in a horizontal channel of constant cross-sections, $R \approx R_i$, so that the cancellation of like terms produces

$$\frac{\bar{n}^2}{3} = n_e^{\,2}$$

or

$$\bar{n} = \sqrt{3} \, n_e \qquad (11)$$

Presentation and Data Analysis

A total of 199 gradually varied flow experiments and 64 spatially varied flow tests were conducted. Of the former category, 69 were performed without siphon tubes while the remaining 130 involved siphon tubes of varying diameter, spacings, and vertical inlet locations. The gradually varied flow experiments were aimed at determining the hydraulic roughness and velocity distribution coefficients for the channel. The results of the gradually varied flow tests were also used as the basis for comparing roughness coefficients from the more complex spatially varied flow experiments.

For the spatially varied flow experiments, the topics of major interest were roughness coefficients ($\bar{n}$ and n_e), water surface profiles and siphon tube discharge uniformity.

Velocity Distribution Coefficients

Velocity measurements were used to determine the energy and momentum coefficients α and β using a modification of the O'Brien and Johnson method (7). The energy coefficient α was obtained by numerically integrating a curve of V_i^3 vs. A_i, where V_i is a given velocity and A_i is the area of the channel cross-section between the successive isovels V_i and $V_i + 1$. The area under the V_i^3 vs. A_i curve was multiplied by the factor A_t^2/Q^3 to yield:

$$\alpha = \left[\sum_{i=1}^{n} \left(\frac{V_i^3 + V_i + 1^3}{3} \right) A_i \right] \frac{A_t^2}{Q^3} \qquad (12)$$

where

$$A_t = \sum_{i=1}^{n} A_i = \text{total cross sectional area, ft}^2$$

Q = total discharge, cfs

Similarly, β was obtained from V_i^2 vs. A_i curves by the following equation:

$$\beta = \sum_{i=1}^{n} \left(\frac{V_i^2 + V_{i+1}^2}{2}\right) A_i \quad \frac{A_t}{Q^2} \tag{13}$$

A measure of the methods accuracy was given by discharge ratio D_r, which theoretically approaches unity:

$$D_r = \sum_{i=1}^{n} \left(\frac{V_i + V_{i+1}}{2}\right) \Delta A_i \quad \frac{1}{Q} \tag{14}$$

Several prediction equations for α were obtained from the experimental data. For gradually varied flow without siphon tubes, the following equation was obtained:

$$\alpha = \frac{R_n}{-634 + 0.95\ R_n} \tag{15}$$

where the Reynolds Number R_n was computed as $\frac{Vy}{\nu}$. Equation (15) was developed for the range $3000 < R_n < 10^7$.

For gradually varied flow with siphon tubes, a simple linear equation which satisfactorily described the data was:

$$\alpha = 1.1782 + 0.04270\ y - 0.03455Q - 0.11983\ T_L + 0.05947d \tag{16}$$

where

y = depth of flow, ft.

T_L = vertical location of tube inlet, ft

d = tube diameter, ft

A dimensionless equation of about equal reliability was

$$\alpha = 1.07468 + 0.17469\ \frac{Su}{y} + 0.61324\ \frac{d}{y} - 1.619 \times 10^{-6}\ R_n \tag{17}$$

Table I

NON-UNIFORMITY COEFFICIENTS AT STATION 0 + 60 FOR THE EXPERIMENTAL CHANNEL WITH SIPHON TUBES (STEADY GRADUALLY VARIED FLOW)

Tube Diameter 1.5 Inches

Average Depth (ft.)	Q (cfs)	Tube Location From Bottom (ft)	Discharge Ratio	Beta	Alpha	Submergence
1.514	1.007	1.0	1.002	1.097	1.180	0.514
1.115	1.007	1.0	0.996	1.030	1.085	0.115
0.695	1.004	1.0	1.003	1.056	1.125	0.000
1.707	3.027	1.0	1.000	1.060	1.141	0.707
1.354	3.010	1.0	1.004	1.047	1.109	0.354
1.067	3.011	1.0	1.004	1.042	1.094	0.067
1.736	4.633	1.0	1.004	1.005	1.127	0.736
1.435	4.659	1.0	1.004	1.043	1.099	0.435
1.266	4.639	1.0	1.001	1.026	1.062	0.266
1.748	4.738	0.5	0.995	1.036	1.107	1.248
1.569	4.776	0.5	1.000	1.050	1.131	1.069
1.339	4.795	0.5	1.005	1.063	1.153	0.839
1.714	2.983	0.5	1.000	1.167	1.279	1.214
1.367	2.975	0.5	1.004	1.078	1.179	0.867
1.308	1.002	0.5	1.004	1.102	1.234	0.808
0.990	0.996	0.5	1.004	1.066	1.153	0.490
0.702	1.003	0.5	1.004	1.063	1.141	0.202
1.768	4.784	0.0	1.000	1.054	1.144	1.768
1.507	4.750	0.0	0.999	1.071	1.191	1.587
0.839	1.002	0.0	1.004	1.163	1.375	0.839
1.129	0.999	0.0	1.003	1.161	1.406	1.129
0.745	0.996	0.0	1.003	1.154	1.344	0.745
1.721	2.983	0.0	1.001	1.157	1.326	1.721
1.323	2.991	0.0	1.005	1.103	1.247	1.323
[illegible]	2.972	0.0	1.005	1.101	1.253	1.173

The roughness coefficient n for a concrete channel with siphon tubes can be obtained from the following prediction equation:

$$n = 00510 + R^{1/6}\,(0.00319 + 0.00821\,\frac{d}{R})\,\frac{Su}{R} + \frac{0.44175\,R_n}{218500 + 85.61\,R_n} \qquad (20)$$

in which Su is the tube submergence $(y - T_L)$ in feet. This equation is adequate for the ranges:

$$T_L < y\ ;\ 0.00 < d \leq 0.25\ ;\ 10^4 < R_n$$

Representative experimental n values are listed in Table II.

__Adjusted $\bar{n}$ and n_e for Spatially Varied Flow__

Values of $\bar{n}$ were determined by adjusting n in equation (2) until that expression predicted the actual water surface profile. The following initial conditions were established at the downstream end of the irrigation bay:

1. $Q_1 = 0$
2. y_1 computed from a least squares curve through the observed water surface elevations.
3. n assumed from gradually varied flow

A profile was obtained for each value of n assumed by solving equation (2) between each pair of siphon tubes. By incrementing y_2, the value $\bar{n}$ produced agreement within $\pm$ 0.0001 ft. between observed and calculated profiles.

Values of n_e were computed from equation (4) with WS_1 and WS_o obtained from the least squares profile curve.

The roughness coefficients from several SVF experiments are presented in Table III. Values of Manning's n from the

Table II

VALUES OF n, n_α, f AND f_α FOR REACH 0 + 00 TO 1 + 80
WITH TUBES

Average Depth	Q	Reynolds Number	n	n_α	α	f	f_α
			Tubes Twelve Inches From Bottom Tube Size 2.0 Inches				
0.658*	1.002	23694.95	0.0105	0.0105	1.071	0.01785	0.01775
1.681	2.981	35305.74	0.0144	0.0144	1.158	0.02605	0.02605
[illegible]	3.019	44195.18	0.0126	0.0126	1.087	0.02158	0.02151
1.028	2.998	52121.28	0.0108	0.0108	1.071	0.01683	0.01668
[illegible]	4.499	50790.63	0.0144	0.0144	1.118	0.02573	0.02570
1.424	4.524	61260.07	0.0133	0.0133	1.081	0.02330	0.02319
1.106	4.521	70124.86	0.0118	0.0119	1.071	0.01939	0.01919
			Tube Size 1.0 Inch				
0.679*	1.000	23560.13	0.0110	0.0111	.072	0.01951	0.01942
1.083	1.000	17021.56	0.0108	0.0108	1.152	0.01650	0.01652
[illegible]	3.010	54029.41	0.0108	0.0108	1.071	0.01676	0.01662
[illegible]	2.995	45537.04	0.0118	0.0118	1.086	0.01876	0.01869
[illegible]	3.002	36727.08	0.0128	0.0128	1.157	0.02048	0.02049
[illegible]	4.526	75198.84	0.0109	0.0109	1.071	0.01650	0.01631
[illegible]	4.514	67034.75	0.0120	0.0120	1.078	0.01914	0.01903
1.691	4.492	56473.05	0.0132	0.0132	1.105	0.02186	0.02182
			Tube Size 1.5 Inches				
0.664	1.004	26372.94	0.0109	0.0109	1.071	0.01903	0.01893
1.678	3.027	39486.90	0.0117	0.0117	1.154	0.01736	0.01736
1.321	3.010	47549.65	0.0119	0.0119	1.090	0.01909	0.01904
[illegible]	3.011	55419.22	0.0114	0.0115	1.072	0.01887	0.01872
[illegible]	4.633	57871.62	0.0.33	0.0133	1.104	0.02232	0.02227
[illegible]	4.659	68268.41	0.0125	0.0125	1.078	0.02068	0.02057
[illegible]	4.639	75427.42	0.0117	0.0117	1.071	0.01881	0.01862

*Tubes were not submerged

Table III

CHANGE IN WATER SURFACE ELEVATIONS AND ROUGHNESS COEFFICIENTS FOR VARIOUS SIPHON TUBE SPACINGS, DIAMETERS, CHANNEL DISCHARGES AND DEPTHS

Exp. No.	Tube Spac. in.	Tube Size in.	Q cfs	Avg. Depth ft.	Upstream Surface Elevation	Downstream Surface Elevation	Adjusted $\overline{n}$	n_e	n_{gvf}
1	20.0	2.0	4.287	1.811	924.4819	924.4873	0.01900	0.01157	0.01700
2	20.0	2.0	4.279	1.813	924.4840	924.4893	0.01900	0.01174	0.01700
3	20.0	2.0	3.310	1.616	924.2880	924.2924	0.01920	0.01093	0.01670
4	20.0	2.0	3.300	1.616	924.2888	924.2921	0.02095	0.01228	0.01670
7	20.0	1.5	3.850	1.823	924.4958	924.4981	0.01900	0.01150	0.01600
8	20.0	1.5	2.498	1.550	924.2230	924.2250	0.01750	0.01004	0.01600
17	40.0	1.5	4.226	1.902	924.5771	924.5760	0.01847	0.01037	0.01272
18	40.0	1.5	3.612	1.752	924.4276	924.4260	0.01819	0.01020	0.01269
19	40.0	1.5	2.884	1.609	924.2841	924.2833	0.01692	0.00943	0.01267
20	40.0	1.5	2.137	1.475	924.1496	924.1496	0.01519	0.00854	0.01269
25	60.0	2.0	4.225	1.791	924.4654	924.4662	0.01615	0.00928	0.01440
26	60.0	2.0	3.305	1.620	924.2938	924.2956	0.01425	0.00804	0.01400
29	60.0	1.5	2.508	1.815	924.4891	924.4903	0.01370	0.00779	0.01470
32	80.0	2.0	3.404	1.847	924.5211	924.5222	0.01547	0.00874	0.01410
33	80.0	2.0	2.482	1.619	924.2930	924.2940	0.01435	0.00805	0.01260

gradually varied flow experiment having the same roughness condition (denoted n_{gvf}) are shown for comparison.

The experimental values of n_e and $\bar{n}$ were correlated by regression through the origin (8). the values of $\bar{n}$ and n_e listed in Table III yielded:

$$\bar{n} = 1.728\ n_e$$

as compared with the theoretical relationship

$$\bar{n} = 1.732\ n_e$$

Nineteen values of $\bar{n}$ and seventeen values of n_{gvf} were analyzed as a group experiment (8). The calculated value of student's t was significant at the $\alpha = 0.05$ level indicating a 95 percent chance exists that the population of $\bar{n}$ and n have different means.

Multivariable equations were computed to relate n_e and $\bar{n}$ to siphon tube spacing, diameter, and submergence. The highest correlation between observed and calculated roughness coefficients was obtained with equations of the form:

$$Y = C_1 + C_2 X_1 + C_3 X_1^2 + C_4 X_1^3 + C_5 X_2 + C_6 X_2^2 + C_7 X_2^3 + C_8 X_3 + C_9 X_3^2 + C_{10} X_3^3 + C_{11} X_1 X_2 + C_{12} X_1 X_3 + C_{13} X_2 X_3 \qquad (21\ \&\ 22)$$

where

$Y = \bar{n}$ or n_e

X_1 = siphon tube spacing, ft

X_2 = siphon tube diameter, ft

X_3 = siphon tube submergence, ft

and C_1, C_2, . . . C_{13} are experimental coefficients listed in Table IV.

Equations (21) and (22) have the following ranges of applicability:

$1.667 \le \text{Spacing} \le 6.667$

$0.125 \le \text{Diameter} \le 0.167$

$0.592 \le \text{Submergence} \le 0.903$

A linear equation for n_e was found to be:

$$n_e = 0.00712 - 0.00072\ X_1 + 0.0188\ X_2 + 0.00343\ X_3 \quad (23)$$

Water Surface Profiles for Spatially Varied Flow

The flow profile for spatially varied flow were found to have three general shapes: rising, descending, or level. The maximum observed rise for any profile was 0.005 ft. while the maximum decline was 0.002 ft.

Water surface profiles were calculated from equation (2) using the iterative procedure outlined previously and using $\bar{n}$ and n_{gvf} as the roughness coefficients. Typical profiles together with the observed water surface elevations are plotted in Figure 4 for the channel reach involving spatially varied flow. The profiles calculated with n_{gvf} generally underpredicted the actual flow profiles.

Water surface profiles were also calculated from equation (9). Typical flow profiles from equation (9) are shown in Figure 5 in which the velocity head recovery and the offsetting friction losses are portrayed.

Table IV

EXPERIMENTAL COEFFICIENTS, STANDARD DEVIATIONS AND CORRELATION COEFFICIENTS OF MULTIVARIABLE EQUATIONS FOR COMPUTING $\bar{n}$ AND n_e

	$\bar{n}$	n_e
	$r = 0.983$ $s = 0.0004$	$r = 0.981$ $s = 0.0003$
C_1	0.03329	0.02801
C_2	0.01102	0.00474
C_3	-0.00414	-0.00187
C_4	0.00033	0.00015
C_5	0.01650	0.08794
C_6	-1.98242	-2.17188
C_7	11.18750	9.50000
C_8	-0.09103	-0.08858
C_9	0.19920	0.16323
C_{10}	-0.10686	-0.08398
C_{11}	0.01137	0.00622
C_{12}	0.00203	0.00045
C_{13}	-0.20483	-0.08076

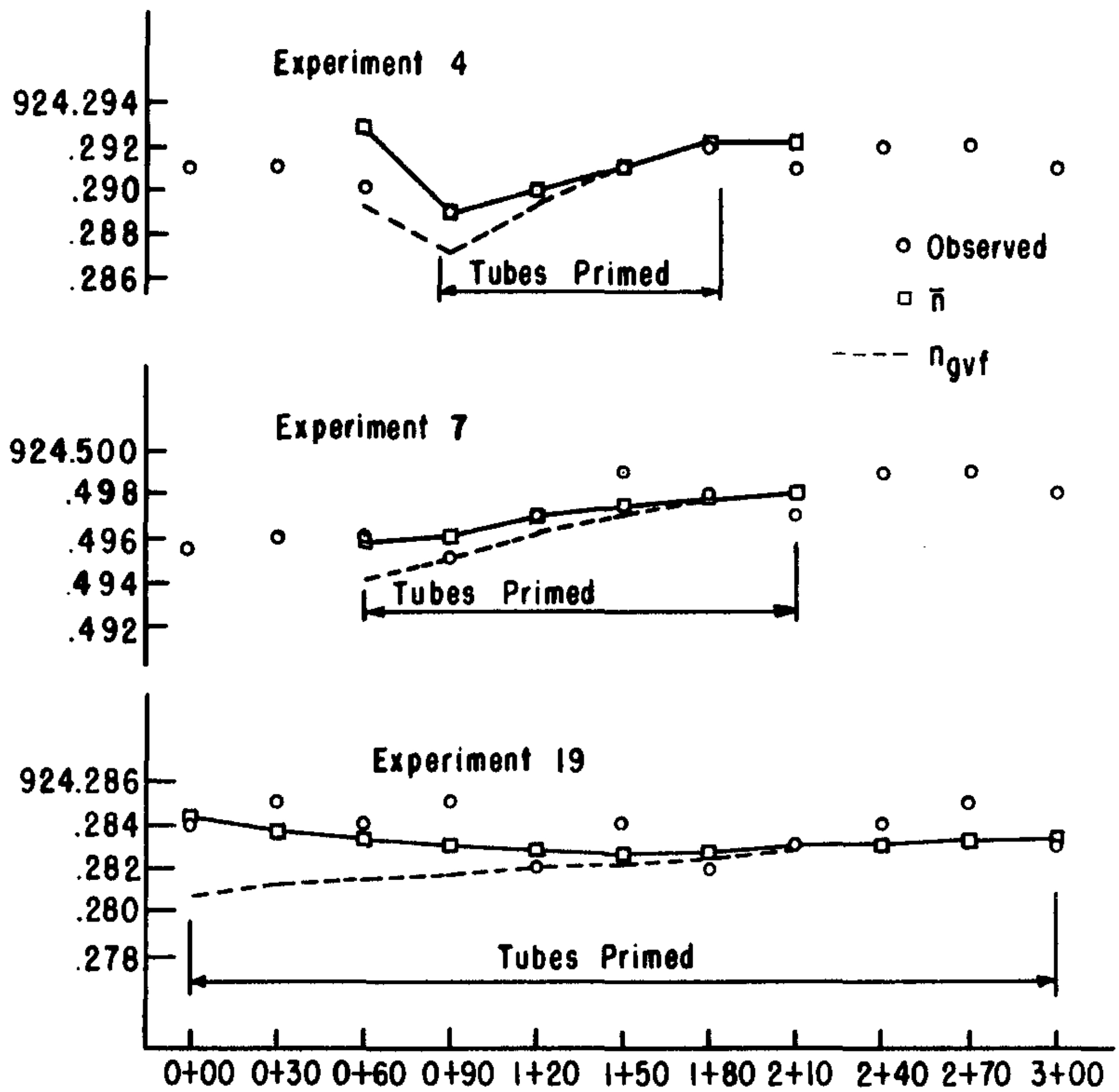

Figure 4. Observed Water Surface Elevations and SVF Profiles Calculated from Equation (2).

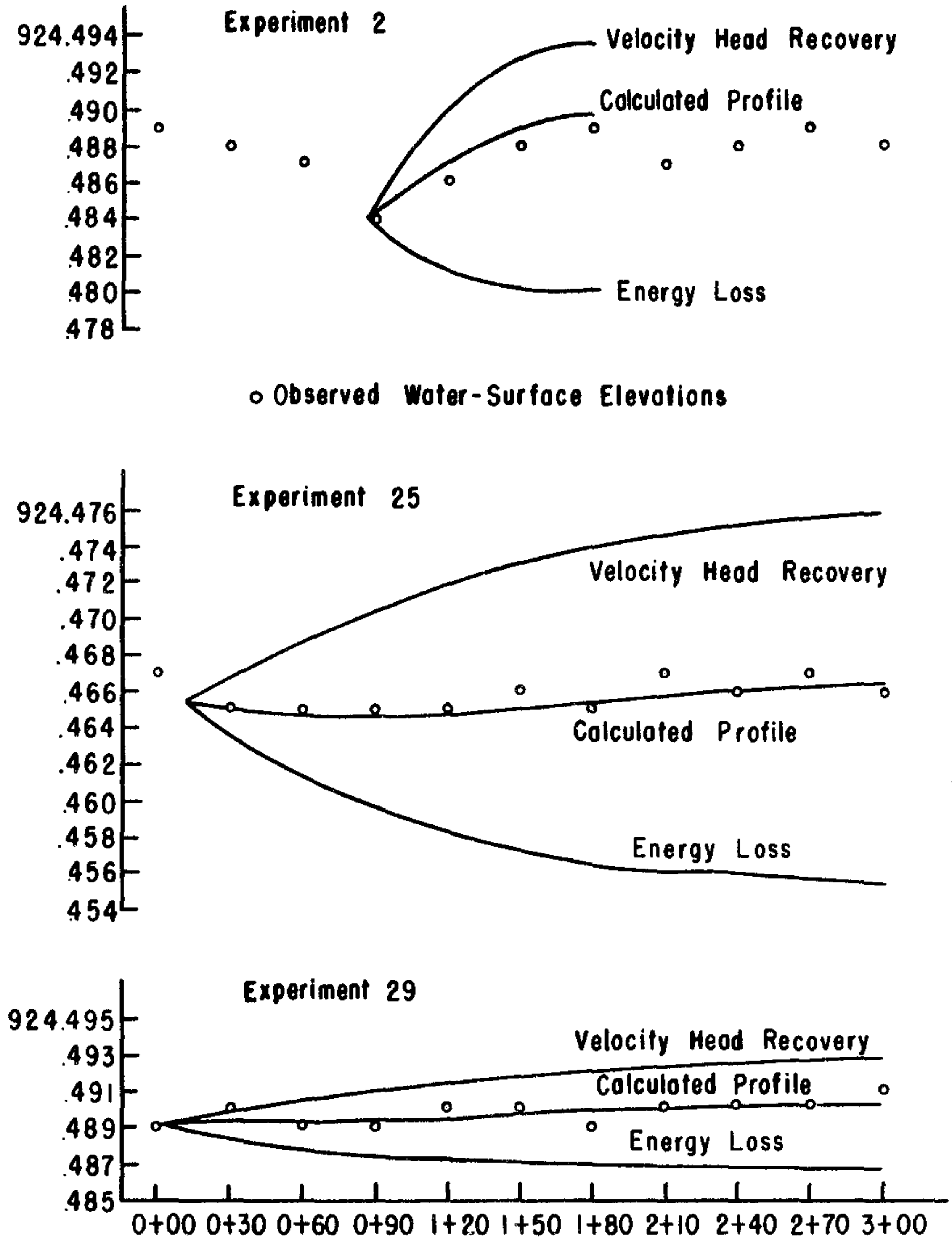

Figure 5. Comparison of Observed Water Surface Elevations to Velocity Head Recoveries, Friction Losses, and Resultant Flow Profiles.

Equations (2) and (9) produced flow profiles that differed a maximum of $\pm$ 0.0005 ft. The similarity of these profiles demonstrates the validity of equation (9) for the conditions under which it was derived.

Nomographs were constructed to predict the change in water surface elevation between the ends of an irrigation bay. These nomographs shown in Figures 6 and 7 in effect solve equation (10) by yielding values of the initial velocity head and the energy loss in the distribution bay when the inflow, upstream depth, length of bay, and either n_e or $\bar{n}$ are shown.

Uniformity of Siphon Tube Discharge

The discharge of every siphon tube within each experiment was computed using the flow profile calculated with $\bar{n}$ and the calibration equation for the particular tube size. The siphon tube heads were obtained by subtracting the standard tube outlet elevation from the calculated surface elevation. A general discharge equation for plastic siphon tubes was found to be

$$Q = 0.0245\ D^{2.111}\ h_o^{\ 0.612} \tag{24}$$

where

d = nominal tube diameter inch

h_o = head, ft.

A more reliable expression requiring a different coefficient C and exponent N for each size was of the form:

$$Q = C\ h_o^{\ n} \tag{25}$$

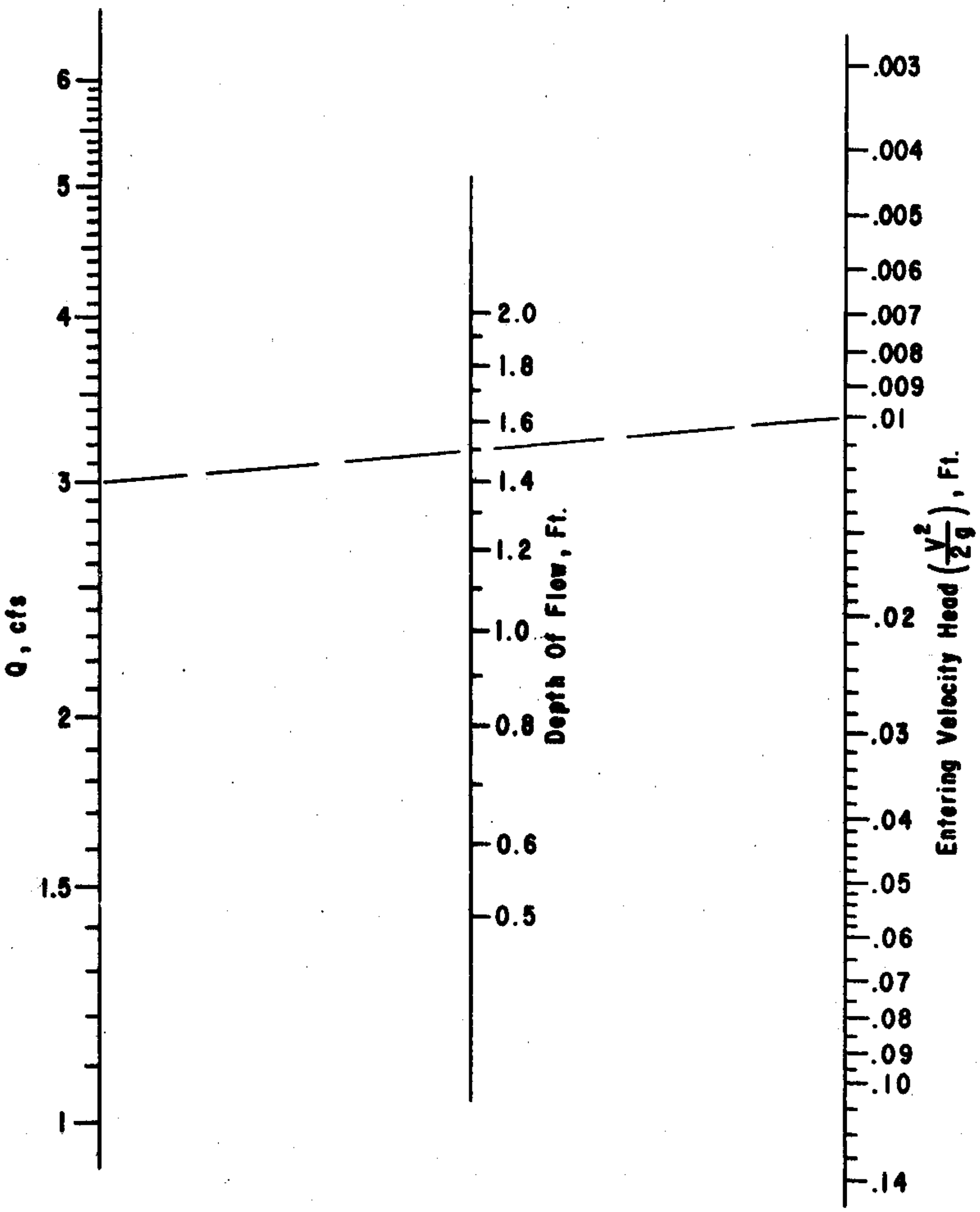

Figure 6. Nomograph for Finding Velocity Head Recovery

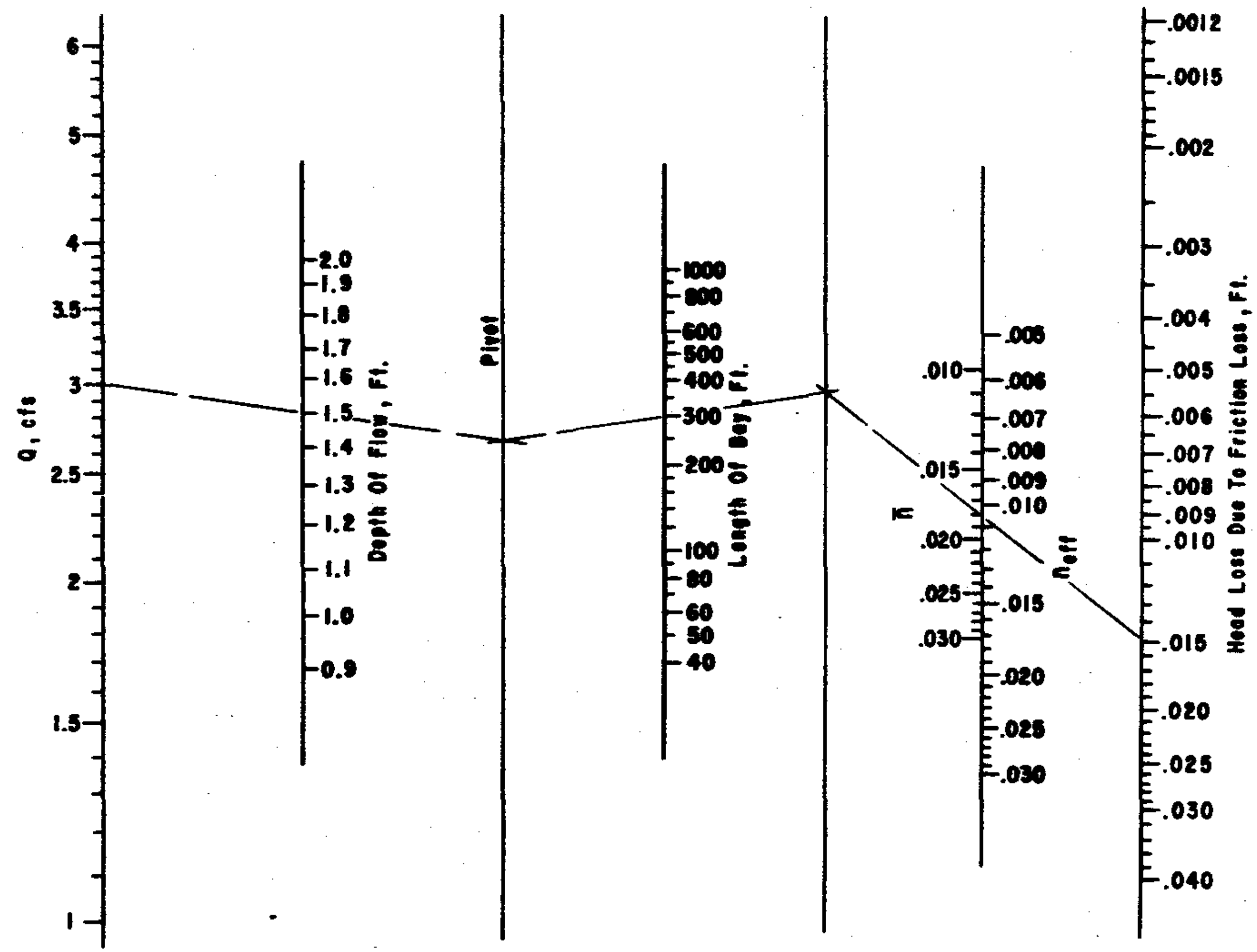

Figure 7. Nomograph for Finding Friction Head Loss

where the values of C and N are given in Table V.

The percent variation in siphon tube discharge within an experiment was computed with respect to the siphon tube furthest upstream. The largest discharge variations for rising and falling profiles were 0.89 and 0.33 percent, respectively.

Effect of Channel Slope as Irrigation Uniformity

The values of $\bar{n}$ obtained from the spatially varied flow experiments with 40 inch spacing were used to calculate water surface profiles for a 300-foot prismatic channel with bottom slopes of 0.000 to 0.0025 ft/ft. The observed downstream depth was used as y_1 in equation (2) to initiate the profile computations. Declining profiles were evidenced for all cases. The smallest changes in water surface elevation were obtained from the experiments having the highest discharge due to the large velocity head recovery.

When a constant tube outlet elevation was assumed, the maximum drop in water surface elevation ΔWS_L was 0.040 ft, while the largest variation in tube discharge was 5.0 percent for 1.5 inch tubes. These values are depicted in Figures 8 and 9. Thus, a level channel produces only slightly better uniformity than a sloping channel provided all siphon tube outlets can be placed at the same elevation.

The same procedure was applied to the case where the tube outlets were assumed to lie in a place parallel to the sloping channel bottoms. The largest magnitude ΔWS_L was only 0.013 ft.

TABLE V.

TUBE DIAMETERS, COEFFICIENTS, AND EXPONENTS FOR CALCULATING DISCHARGE FROM PLASTIC SIPHON TUBES

Nominal Tube Diameter Inches	Average Inside Diameter Inches	Coefficient* K	Coefficient** C	Exponent** N
0.75	0.747	4.95	0.01292	0.60697
1.00	1.028	4.43	0.02511	0.58794
1.25	1.245	5.32	0.03986	0.60844
1.50	1.478	5.43	0.05806	0.62449
2.00	1.972	5.30	0.10639	0.65513
3.00	2.898	5.06	0.24450	0.85109

*After Keflemariam (11)

**After Mink (16)

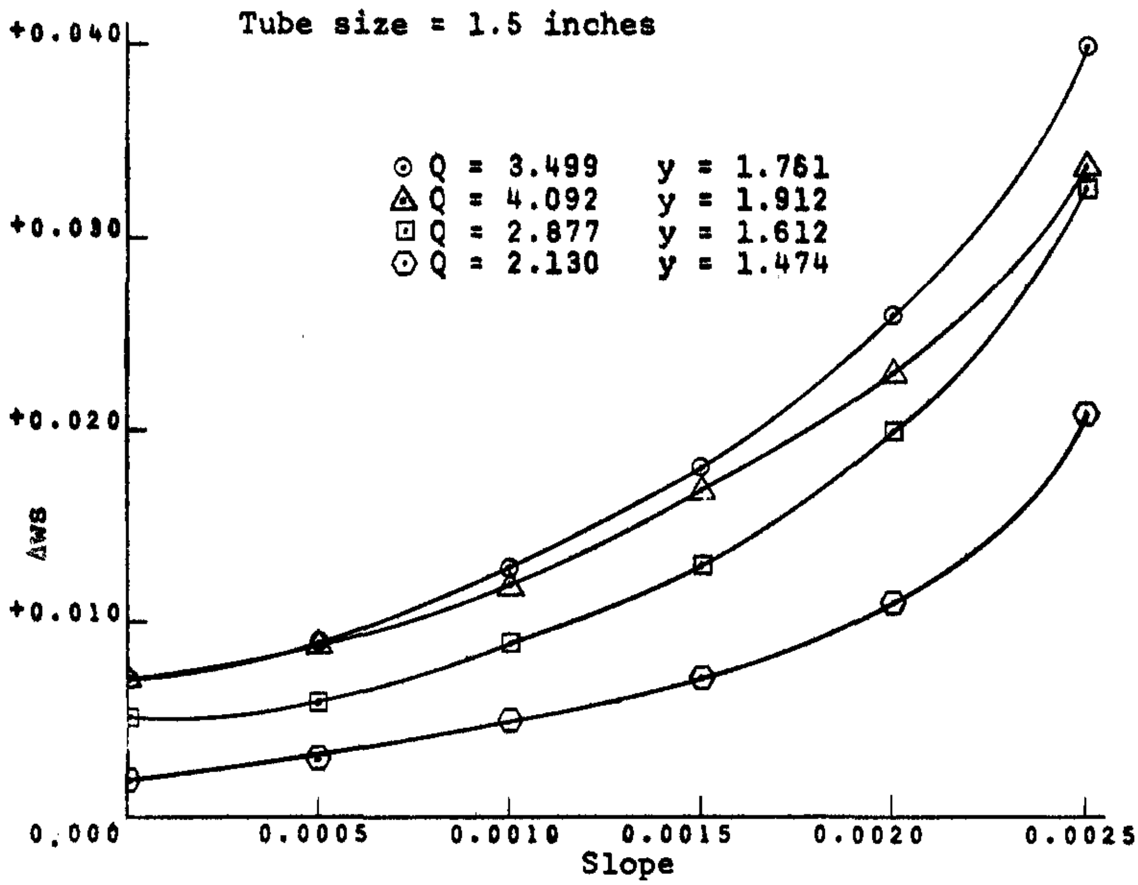

Figure 8. Change in Water-Surface Elevation Between the Upstream and Downstream End of the Prismatic Channel as a Function of Slope for the 1.5-Inch Tubes with Various Depths and Entering Q's

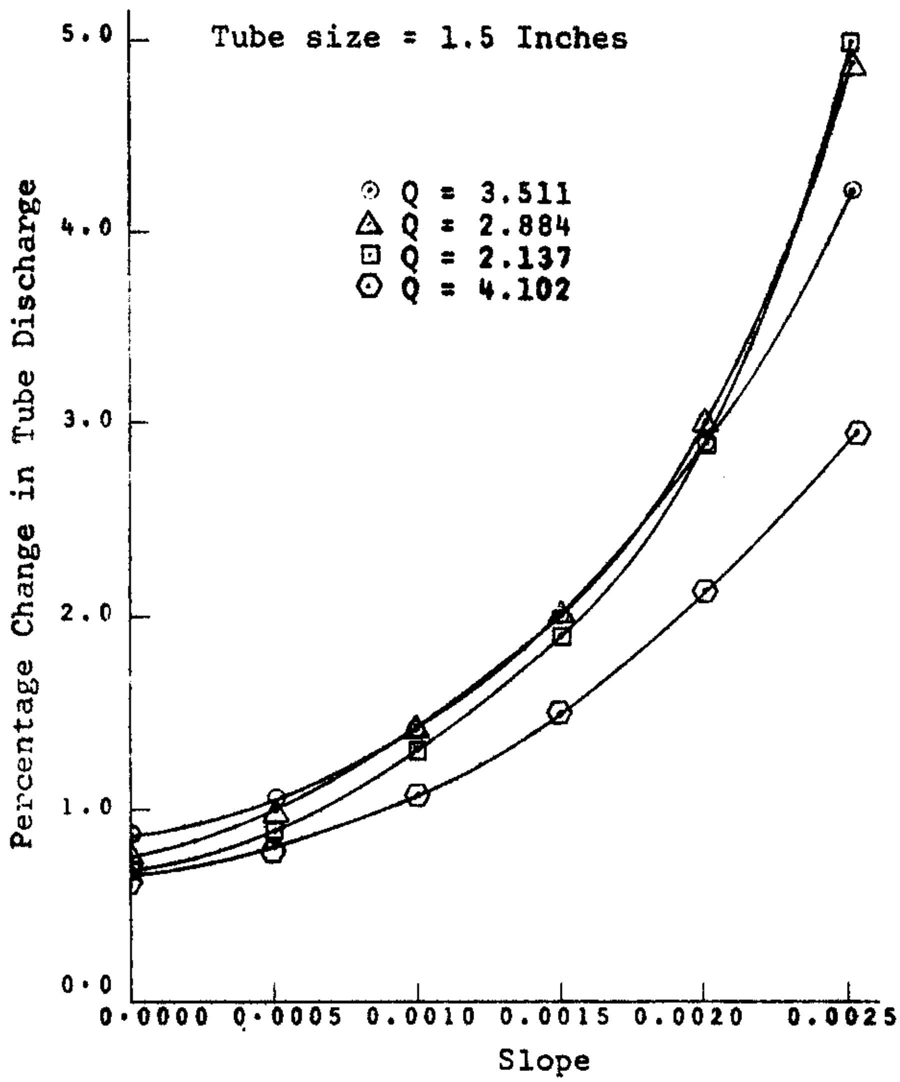

Figure 9. Percentage Change in the 1.5-Inch Tube Discharge for Tubes At a Constant Elevation in a Prismatic Channel with Various Slopes

However, discharge variations of up to 100.0 percent were calculated due to the large differences in tube elevation. The range of variations was from 0.2 percent for small slopes and large Q's to 100.0 percent at the largest slope and small Q values.

Effect of Roughness on Uniformity

The effect of using inaccurate values of $\bar{n}$ in predicting siphon tube discharge uniformities was calculated for the experiments presented in Table III. The value of $\bar{n}$ was varied in 5 percent increments from 75 percent to 125 percent of $\bar{n}$. The water surface elevations at six points in the channel were calculated for each assumed roughness value using equation (9). Typical profiles for .75 $\bar{n}$ to 1.25 $\bar{n}$ are depicted in Figure 10. Calculated values of ΔWS_L ranged from 0.0073 ft to -0.0082 ft. The deviations in tube discharge varied from 1.10 percent to -1.06 percent.

Hence, the effect of channel roughness on siphon tube discharge variations is much smaller than the effects of bottom slope and differences in tube outlet elevation.

Unrestrained Siphon Tube Experiments

A small number of experiments were performed in which the inlet ends of the siphon tubes were free to move with the channel current. Gradually varied and spatially varied flow experiments were conducted with discharges of 2.50 and 3.25 cfs, a tube diameter of 1.5 inches, a spacing of 40 inches, and an inlet height of 1.0 foot above the channel bottom.

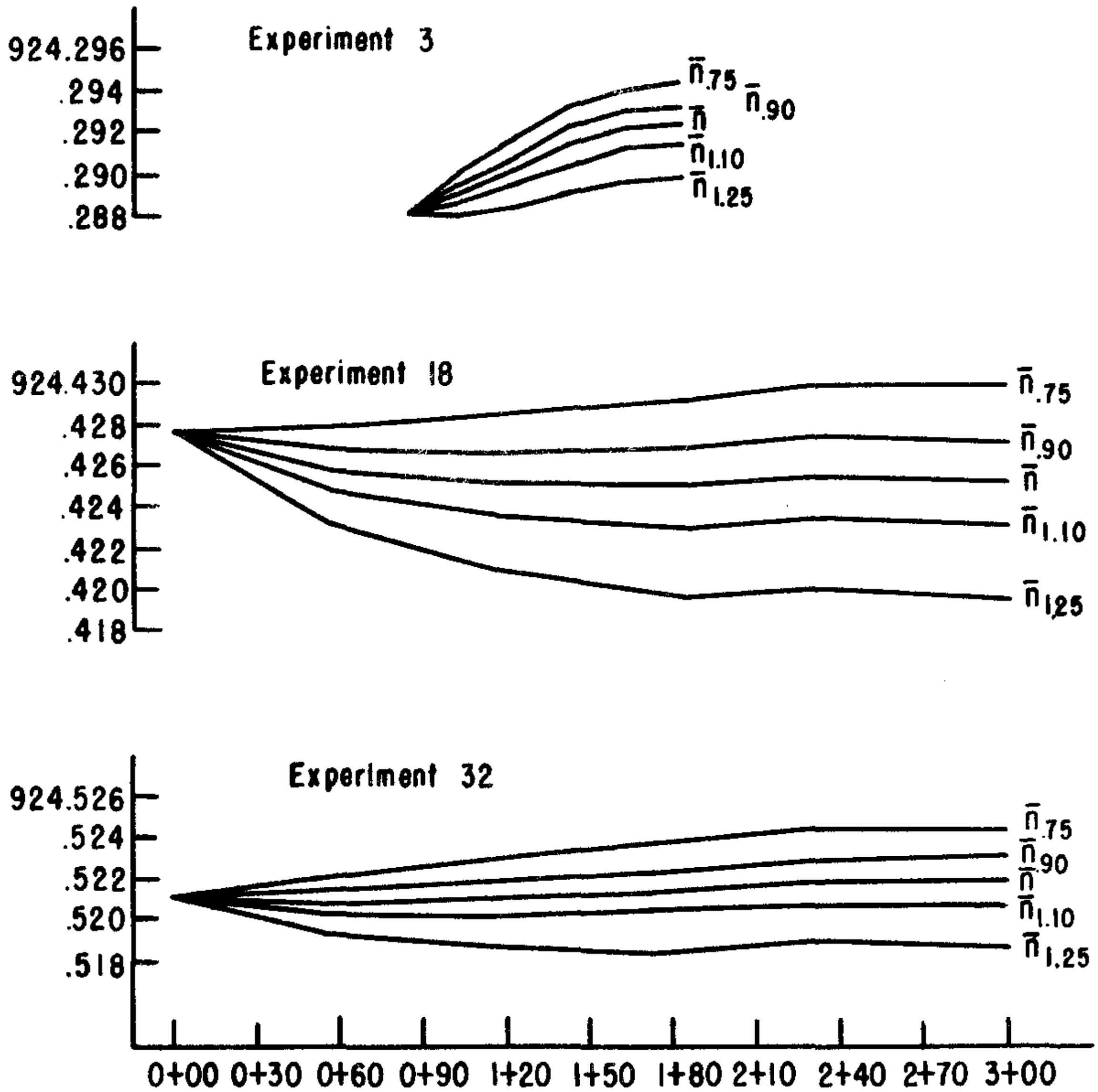

Figure 10. Water Surface Profiles Calculated from Equation (8) for 0, ± 10, and ± 25 Percent Variation in the Roughness Coefficient $\bar{n}$.

The values of Manning's n calculated for the GVF experiments were compared with predicted values from equation (20). The calculated values of n for restrained tubes and the observed values for unrestrained tube experiments differed by less than 8 percent.

Values of $\overline{n}$ and n_e were calculated for the spatially varied flow experiments involving unrestrained siphon tubes. These values compared favorably with the predicted values from linear multivariable models for $\overline{n}$ and n_e, such as equation (23).

Conclusions

The following conclusions are based on the analysis and interpretation of the experimental data for the hydraulic roughness tests:

1. For gradually varied flow, the energy coefficient α is a function of siphon tube diameter, submergence, depth, and Reynold's Number.
2. Manning's n for gradually varied flow is dependent upon Reynold's Number, hydraulic radius, siphon tube diameter, and tube submergence.
3. Water surface profiles for spatially varied flow can be predicted more accurately using $\overline{n}$ than with Manning's n from gradually varied flow.
4. For spatially varied flow, hydraulic roughness can be calculated as a function of siphon tube spacing, tube diameter, and tube submergence.

5. The theoretical relationships between $\bar{n}$ and n_e equation (11) is valid for the conditions under which it was derived.
6. The effective roughness coefficient n_e, calculated from Manning's formula and based on the hydraulic characteristics of spatially varied flow is useful for predicting the change in told energy head between the ends of a horizontal irrigation bay.
7. The water surface elevation at any cross-section in a horizontal prismatic irrigation bay can be directly calculated from equation (9).
8. Uniform irrigation can be attained from a sloping channel with the siphon tube outlets at the same elevation, provided land slope and furrow elevations allow the latter condition to be met.
9. Nonuniform distribution of irrigation water would result from a sloping channel with the siphon tube outlets a fixed height above the channel bottom.
10. Large changes in the roughness of an irrigation distribution channel would produce slight variation in the uniformity of siphon tube discharge.
11. A distribution channel consisting of a series of level bays with constant siphon tube outlet elevation in a given bay would produce the greatest irrigation uniformity of any system tested, yielding variations in siphon tube discharge of less than one percent.

Hydraulic Characteristics of Furrow Outlet Devices

The water application efficiency of furrow irrigation can be increased when a cutback system is employed. Cutback irrigation entacts the application of (1) an initial furrow discharge capable of traversing the entire furrow length at a non-erosive velocity and without deep percolation losses, and (2) a smaller cut-back furrow discharge equal to the sum of the soil intake rate and the evaporation rate. The initial flow is generally one and one-half to three times the cut-back furrow flow.

The principal disadvantage of a cut- back system is the increased amount of labor required to alter individual furrow discharges during an irrigation.

An automatic cut-back system can be designed using short hooded inlet tubes. Other self-regulating devices such as orifices and weirs also offer promise as furrow outlet devices.

Hooded Inlet Tubes - Automation of cut-back furrow irrigation requires the use of a device which will prime at low heads, and which has a predictable discharge over the practical range of discharges. Previous studies (1) demonstrated that short level hooded inlet tubes have the desired characteristics.

Hooded inlet tubes are formed by cutting the inlet end at a 45 degree angle with the longitudinal axis. They are placed horizontally through the channel wall with the longest edge directly above the shortest edge.

The effects of wall thickness, length, and tube roughness on the discharge of level hooded inlet tubes had not been defined. An investigation was undertaken to study the remaining aspects of hooded inlet tube discharges.

Objectives

1. To determine the effects of length, wall thickness, and tube composition on full pipe flow through short level hooded inlet tubes.
2. To develop a relationship to accurately predict the discharge through short hooded inlet tubes for various heads, tube lengths, diameter, and wall thicknesses.

Equipment and Procedure

The experimental apparatus consisted of these elements: (1) sump, (2) electric pump, (3) water meter, (4) forebay with rock baffle, (5) hooded inlet tube, and (6) point gage. Water was pumped from the sump through a calibrated nutating disk water meter into the bottom of a barrel which served as the forebay. The kinetic energy of the entering flow was dissipated by passage through the baffle. The outlet tube was mounted and accurately leveled with a compression coupling through the side of the barrel. The head water elevation was measured with a point gage to $\pm$ 0.001 ft.

Aluminum tubing of 1.5 inch diameter cut at a 45 degree angle with its longitudinal axis was selected for use in the experiments. Tube lengths of 1.0, 2.0, and 3.0 ft. were tested. The tubes were accurately machined for a distance of two diameters downstream from the inlet invert. The desired wall thicknesses ranges from 0.0049 - 0.0209 ft. Approximately fifteen values of head (measured from the water surface elevation to the invert or lowest point inside the tube inlet) ranging from 0.10 to 1.00 ft. were set for each tube length and wall thickness. The discharge for each test was determined from the water meter readings.

Results

A general dimensionless relationship of the following form was desired:

$$\frac{Q^2}{gD^5} = f\ (T/D,\ L/D,\ H/D) \tag{26}$$

where

Q = tube flow rate, cfs

g = accelerating gravity, ft/sec^2

D = inside thickness, ft

T = wall thickness, ft

L = tube length, ft

H = head on inlet invert, ft

Component relationships were obtained between each independent pi term on the right side of equation (26) and the dependent quantity (Q^2/gD^5). When combined, these component equations yielded the final prediction equation for flow through short, level hooded inlet tubes:

$$\frac{Q^2}{gD^5} = \frac{\left[0.989 + 0.112\,\frac{T}{D}\right]\left[\frac{H}{D} - (0.604 + 0.00787\,\frac{L}{D}\right]}{1.423 + 0.0166\ L/D} \tag{27}$$

Equation (27) shows that Q decreases as L/D increases and vice versa. Also, an increase in H/D results in an increased valuing Q as would be expected. For the range of wall thicknesses tested, the term 0.989 + 0.112 T/D remains relatively constant and causes approximately 0.5 percent variation in Q.

The accuracy of the prediction equation was checked using observed values of Q and Q values calculated from equation (27). The maximum difference between observed and calculated Q values was 2.6 percent with 80% of the values showing less than 1.0 percent variation.

The applicability of equation (27) to different materials was also investigated using 1½ inch plastic tubing, electrical conduit, and galvanized pipe. For lengths of two feet, the variations between observed and calculated Q values were all less than 1.0 percent. However, a three foot length of galvanized pipe produced lower discharges than were predicted from equation (27) due to the higher roughness of the galvanized pipe as compared to aluminum tubing. The greatest deviations occurred at the highest flows.

Conclusions

The analysis of data for the hooded inlet tube experiments produced the following conclusions:

1. Equation (27) accurately predicted the flow through the short hooded inlet tubes studied.
2. Tube wall thickness has less than 0.5 percent influence on tube discharge for the range of thickness studied.

3. For a given value of H/D, discharge through a short tube decreases significance as the tube length increases and vice versa.
4. Equation (27) accurately predicted flow through two foot length of galvanized plastic and electrical conduit tubing.

Rectangular Side Weirs

The installation of weirs as furrow outlet devices would allow the use of a shallower, less costly irrigation channel, especially when a cut-back system is considered. With rectangular weir flow, the discharge varies approximately as the head to the 1.5 power, whereas the discharge with short tubes varies as the square root of the head, if the orifice analogy is used. Side discharging weirs are adaptable to flatter land slopes parallel to the channel. Thus, a smaller drop between bays could be utilized to attain the same ratio of initial to cut-back flows. Also, less debris problems should occur with a weir system.

The design of a cut-back irrigation system with side discharging weirs would require that the relationship governing discharge be determinable. Therefore, laboratory experiments were designed with the following objectives:

1. To determine the head versus discharge relationship for a rectangular weir placed in the side of a trapezoidal channel.
2. To determine the influence of flow past the weir crest and of crest height on the discharge of rectangular

side weirs.

Equipment and Procedure

A 16-foot long plywood flume of trapezoidal cross-section (2-ft depth, 1-ft bottom width, and 1:1 side slopes) was constructed to simulate a typical concrete-lined irrigation channel. Water was circulated by a 6-inch pump from a sump through the flume weir, and H-flume measuring device and was returned to the sump. Head readings were obtained by subtracting the crest elevation from the water surface elevation in the flume. Inflow to the flume was measured with a Sparling Meter while an H-flume was used to measure weir discharge.

Crest lengths of 2, 4, 6, 8, 10, and 12 inches were chosen for the experiments. Crest heights of 12, 16, and 18 inches above the channel bottom were used for the 12-inch crest lengths. Also for the 12-inch weir, two approximate discharges past the weir crest - 0.0 and 1.5 cfs were controlled using an adjustable tail gate and the inflow valve.

Results

For constant values of crest lengths, head, and crest height, the weir discharge decreased 4 percent or less when the discharge past the weir crest increased from 0.0 to 1.5 cfs. For constant values of downstream discharge, crest height, and head, increases of 5% or less in weir discharge were evidenced when the crest height was raised from 1.0 to 1.5 feet. The discharge variations due to changes in crest height and downstream discharge were greatest for the least heads of 0.020

feet and were negligible for the higher weir discharges at 0.20 to 0.40 feet of head.

For a constant crest height of 1.333 feet and a downstream velocity of zero, sixty experiments were conducted in which the crest length was varied from 2.0 to 12.0 inches while the head was varied from 0.020 feet to 0.400 feet. The prediction equation for these tests was found to be:

$$Qw = 3.854729\ L^{0.897029}\ H^{1.503014} \tag{28}$$

where

Q = weir discharge, cfs

L = crest length, feet

H = head, feet

The prediction equation for side weir discharge when the effects of downstream channel velocity and variable crest heights were included was:

$$Q = 3.815606\ L^{0.888923}\ H^{1.502821} \tag{29}$$

Equations (28) and (29) apply to the range $0.024 \leq H \leq .407$ ft. The average deviations of the measured weir flows from the discharges given by equation (28) and (29) were 2.4 percent and 2.3 percent respectively.

Circular Weirs and Orifices

Circular orifices offer promise as furrow outlet devices primarily because they are much easier to fabricate and install than either hooded inlet tubes or rectangular side weirs. However, as compared to weirs, they require higher heads to attain the same discharge. For example, a ratio of initial to cut-back flows of 3:1 would necessitate an orifice head ratio of approximately 9:1 which would be impractical for gently sloping land. The limitations on head can perhaps be overcome through the use of large orifice diameters with weir or transitional flow.

Experiments were conducted to determine the discharge characteristics of circular orifices installed in the side of an irrigation channel so that cut-back irrigation systems with orifice outlets as the furrow metering devices could be designed. These experiments were performed with the following objectives:

1. To determine the head-discharge relationships for sloping plate orifices with both weir and orifice flow conditions.
2. To determine the head-discharge relationships for the transition zone between the weir and orifice flow condition.
3. To determine the ranges of applicability of the head discharge relations developed.

Equipment and Procedure

The plywood flume described previously was used for the orifice experiments. This system is shown in Figure 11. Values of head on the orifices were defined as the height of the head water elevation in the channel minus the invert elevation (lowest point of the orifice crest). Orifice discharges were measured with the 0.75 ft. H-flume shown in Figure 12.

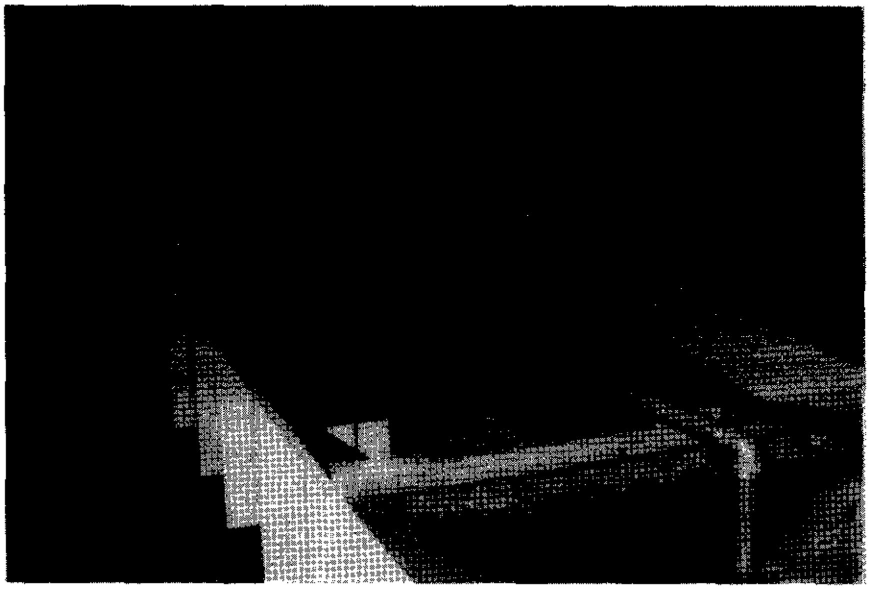

Figure 11. The head on the side discharge orifice plates installed in the experimental channel was measured with a point gauge on a traversing carriage.

Figure 12. A 0.75 ft. H-flume, calibrated in place, was used to measure the discharge.

The variables chosen for study were invert height, orifice diameter, head, and velocity past the orifice plate. Seven orifice diameters ranging from 1.000 to 8.000 inches, and three orifice invert locations - 11, 16, and 19 inches above the channel bottom were selected. Four flow rates past the orifice plates (0.0, 1.0, 1.5, and 2.0 cfs) were utilized to study the effect of decreasing spatially varied flow as occurs in an irrigation distribution channel on the orifice and weir discharges. From 10 to 16 values of head were established in the range $0.035 \leq H \leq 0.400$ ft.

Results

Variation of the invert height above the channel bottom caused no significant difference in the discharges within the weir and orifice ranges. Also, no significant difference in discharge was found for flow rates of 0.0 to 2.0 cfs past the orifice plates. For all orifice discharges with flow past the weir crest, vortices formed causing a maximum decrease in discharge of 3.4 percent as compared to the same head readings when the vortices were mechanically suppressed.

Three equations were found necessary to describe the head-discharge relationships which exist for weir flow, orifice flow, and the interposing transition zone. These equations along with their ranges of applicability are as follows:

Flow Type	Range	Equation	
Weir	$0.035 < H < 0.35D$	$Q=4.542\ D^{0.549}\ H^{1.953}$	(30)
Transitional	$0.35 < H/D < (0.89 - .23D)$	$Q=3.710\ D^{0.662}\ H^{1.797}$	(31)
Orifice	$(0.89 - 0.23D) < H/D$	$Q=3.450\ D^{1.947}(H-0.35D)^{0.463}$	(32)

in which H is the head in feet, D is the orifice diameter in feet, and Q is the discharge in cfs. The experimental values of D and H varied a maximum of 5.3 percent from the observed values.

An analysis was also conducted using the general dimensionless expression: $Q^2/gD^5 = C_1 \left(\frac{H}{D}\right)^{C_2}$ (33)

Values of C_1 and C_2 were obtained by regression for each of the three ranges given along with equations (30), (31), and (32). For headwater elevations below the orifice centerline ($H \leq 35D$) the equation best describing the data was:

$$Q = 4.529\ D^{0.548}\ H^{1.951} \qquad (34)$$

Above the orifice centerline, the equations developed by dimensionless parameters proved inferior as compared to equations (31) and (32).

Values of Q computed from equations (30) and (31) were compared with discharges from vertical orifices of similar diameter as reported by Greve (4). For the same heads and diameters calculated discharges from the orifices and weirs of 45 degrees slope were about 1.414 times greater than for the vertical orifices and weirs. Thus, when the slope angle is taken into account, the discharge coefficients for the sloping and vertical weirs and orifices were approximately equal.

The knowledge acquired from the circular weir and orifice investigation was applied to the design of an automatic cutback furrow irrigation system which was installed at the Irrigation Research Station at Altus, Oklahoma in March, 1968.

This system included four level distribution bays (total length of 400 feet) and 2-inch diameter sheet metal orifice plates serving as the furrow outlet devices.

Conclusions

The following conclusions were drawn from the interpretation of experimental results:

1. Sheet metal orifice and circular weir plates are feasible to use as a furrow metering device for concrete-lined irrigation channels.
2. The hydraulic properties of orifice and circular weir plates installed at 45 degree slope and acting as side weirs can be described by equations (30), (31), and (32) for the discharge range - 0.002 cfs. to 0.546 cfs.
3. The design for a middle range of field slopes parallel to the irrigation distribution channel can be accomplished with the proper plate diameter selection.

Project Related Publications and Theses

1. Barefoot, Armond D. "The Hydraulic Properties of Orifices and Circular Weirs with a 45 Degree Slope." Unpublished Master's thesis, Oklahoma State University, Stillwater, Oklahoma, 1968.
2. Barefoot, Armond D. and James E. Garton. "The Hydraulic Properties of Sheet Metal Orifices and Circular Weirs When Used as Furrow Metering Devices on Concrete Channels." Paper presented to the Southwest Section Meeting of the American Society of Agricultural Engineers, Baton Rouge, Louisiana, April 3-5, 1968.
3. Hamilton, Wilbur L. "The Effect of Length and Wall Thickness or Flow Through Short Tubes." National Student Journal, American Society of Agricultural Engineers, 1968.
4. Keflemariam, Joseph. "Flow Through Plastic Siphon Tubes." Unpublished Technical Report, Agricultural Engineering Department, Oklahoma State University, 1966.
5. Mink, Albert L. "The Hydraulics of an Irrigation Distribution Channel." Unpublished Ph. D. dissertation, Oklahoma State University, 1967.
6. Mink, Albert L., James E. Garton, and John M. Sweeten. "Hydraulics of an Irrigation Distribution Channel." Paper presented to the Southwest Section Meeting of the ASAE, Stillwater, Oklahoma, April 26-28, 1967.
7. Sweeten, John M., James E. Garton, and Albert L. Mink. "The hydraulic Roughness of an Irrigation Channel with Decreasing Spatially Varied Flow." Paper presented to the

National Winter Meeting of the ASAE, Detroit, Michigan, December 12-15, 1967.

8. Sweeten, John M. "The Hydraulic Roughness of an Irrigation Channel with Decreasing Spatially Flow." Unpublished M.S. thesis, Oklahoma State University, 1967.

Other References Cited

1. Albertson, Kenneth G. "Influence of Length/Diameter Ratio on Flow Through Level Hooded Inlet Tubes." Unpublished M. S. thesis, Oklahoma State University, Stillwater, Oklahoma, 1963.

2. Chow, Ven Te. Open Channel Hydraulics, New York: McGraw-Hill Book Company, Inc., 1959.

3. Garton, James E. "Designing an Automatic Cut-Back Furrow Irrigation System, Oklahoma Agricultural Experiment Station, Bulletin B-651, October, 1966.

4. Greve, F. W. "Flow of Water Through Circular, Parobolic, and Triangular Vertical Notch-Weirs, Engineering Experiment Station Research Series No. 40, Purdue University, March, 1957.

5. Hansen, V. E. "New Concepts in Irrigation Efficiency." Presented to the Annual Meeting of the American Society of Agricultural Engineers, East Lansing, Michigan, June 25, 1957.

6. Israelson, O. W. and H. F. Blaney. Water Application Efficiencies in Irrigation, United States Department of Agriculture, February, 1946.