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REUSE SYSTEM DESIGN FOR BORDER IRRIGATION

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REUSE SYSTEM DESIGN
FOR BORDER IRRIGATION

by

Muluneh Yitayew

A Dissertation Submitted to the Faculty of the

DEPARTMENT OF CIVIL ENGINEERING AND ENGINEERING MECHANICS

In Partial Fulfillment of the Requirements
For the Degree of

DOCTOR OF PHILOSOPHY
WITH A MAJOR IN CIVIL ENGINEERING

In the Graduate College

THE UNIVERSITY OF ARIZONA

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THE UNIVERSITY OF ARIZONA
GRADUATE COLLEGE

As members of the Final Examination Committee, we certify that we have read the dissertation prepared by Muluneh Yitayew

entitled Reuse System Design for Border Irrigation

and recommend that it be accepted as fulfilling the dissertation requirement for the Degree of Doctor of Philosophy.

W G Matlock — Date 14 APR 82

E. M. Laursen — Date 4/14/82

Gary L. Ohng — Date 4/14/82

Simon Ince — Date 4/19/82

D D Fangmeier — Date 4/19/82

Final approval and acceptance of this dissertation is contingent upon the candidate's submission of the final copy of the dissertation to the Graduate College.

I hereby certify that I have read this dissertation prepared under my direction and recommend that it be accepted as fulfilling the dissertation requirement.

D D Fangmeier
Dissertation Director

Date 4/19/82

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SIGNED:

TO

MY FATHER AND MY BROTHERS

ACKNOWLEDGEMENTS

This work would not have been possible without the support of the author by the Soils, Water and Engineering Department at the University of Arizona and USDA Water Conservation Laboratory in Phoenix. Dr. Delmar D. Fangmeier's efforts in arranging financial support and his technical advice, encouragement and above all his patience throughout my study at the University are sincerely appreciated. No researcher could wish for more cooperation than was received from Dr. Fangmeier.

The author would like to thank Dr. Simon Ince, his academic advisor; and Dr. Emmett Laursen, Dr. Gerald Matlock and Dr. Gary Ammy, who served as members of the graduate committee. The critical suggestions and constructive discussions provided by Dr. Emmett Laursen are greatly appreciated.

Special thanks are due to Albert J. Clemmens of the U.S. Water Laboratory in Phoenix for his generous contribution of his time throughout this work, specially in making the latest version of the Zero-Inertia mathematical model available and his valuable suggestions in some portion of the study.

My sincere appreciation goes to Mimi Kibret who not only overcame my frustrations and depressions, but also gave me more support in the early part of my study than anyone else.

I would like to express my profound gratitude to Sharon for her time, effort and patience in the preparation of this manuscript.

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ABSTRACT

Advances in mathematical modelling and the availability of high speed computers with considerable memory size is making it possible to study the hydraulics of border irrigation in a greater depth than ever before. A zero inertia mathematical model was found to be reliable and inexpensive among the models available in border irrigation hydraulics and was used for this study to simulate free outflow flowing border irrigation. Special emphasis was given to the runoff produced from such a system.

This study dealt particularly with, the identification of pertinent open channel variables affecting runoff in border irrigation, presentation of predictive graphical and mathematical solution to quantify runoff, and with utilization of these solutions in developing reuse system design criteria.

Inflow rate, surface resistance, border slope, soil infiltration characteristics, application time (time of cutoff and length of run of the border) were among other variables studied. As one might expect, runoff was found to increase with slope, flow rate, application time and decrease with increase in infiltration rate, length of run and bed and vegetation drag.

Considering the number of variables affecting runoff characteris tics from a given irrigation, it was obvious to see a thorough examina tion of each variable in dimensional terms was practically impossible. Also, presentation of the results would have required too many graphs. Dimensional analysis was used to solve this problem and in developing dimensionless runoff curves.

The ability to quantify runoff made it possible to develop reuse system design formula for proper sizing of reuse systems under several operational requirements.

Shape function for the ultimate infiltrated depth profile was used to get times of runoff and also calculate various efficiencies which are useful for evaluating the system.

The study shows, through the use of reuse system, the poential application efficiency can be changed from present values of 60 percent to 90 percent in Arizona. It also can be used to demonstrate the saving in energy that can be realized through such system.

Step by step procedures for the design of reuse system using graphical and mathematical solutions are presented with a sample problem worked out. It is expected that the result of this study can be used by designers as well as operators of border irrigation systems with out any difficulty with the aid of a simple pocket calculator.

Other uses of the study include getting optimal design for the system itself by evaluating various possible designs and classroom instruction on the application of dimensional analysis to open channel hydraulics problems and design of reuse systems.

CHAPTER I

INTRODUCTION

The quantity of water needed for survival of human beings is increasing at a rate where some parts of society are facing critical water shortages. The shortage is due to increased consumption by households, industries and agriculture, lack of more advanced technology to produce more usable water, unpredictable natural phenomena like droughts, pollution of available water sources and improper management. Several studies are being made with respect to pollution control, and prediction of droughts with promising results. With respect to production of more usable water, approaches like cloud seeding, using icebergs, evaporation suppression and some other water harvesting programs are being conducted but have a long way to go to be feasible for use and to gain acceptance. As some of these studies take time and water becomes scarce and a more valuable commodity, the efficient use of the available water by conservation practices offers great promise in every aspect of life.

Of all the users, agriculture consumes the largest portion of the world's available water resource. In the United States alone agriculture was responsible for 83 percent of the total water consumed in 1975 (U.S. Water Resources Council, 1978). In the same year, of all the water made available, 82 billion gallons per day of fresh groundwater were withdrawn for use, of which 56 billion gallons per day pr 68 percent for were used by irrigation. This represents 35 percent of all the water used for irrigation. From the statistics presented above, it is very clear indeed that agriculture is a primary candidate for any water conservation measure to avoid further critical shortages.

In agriculture irrigation is the major consumer of the available water. The purposes for using irrigation as listed by Israelsen and Hansen (1963), include:

(1) supply of moisture essential for plant growth,

(2) provide crop insurance against short duration droughts,

(3) cool the soil and atmosphere, thereby making a more favorable environment for plant growth,

(4) wash out or dilute salts in the soils,

(5) reduce the hazards of soil piping and

(6) soften tillage pans.

Depending on the purpose and resources available, irrigation may be accomplished by sprinkler, trickle, subsurface or surface methods. Of all these, surface irrigation is the most widely practiced system in most parts of the world, and this study concentrates on one type of surface method. In surface methods, water is spread over a field by gravity, unlike most others where additional driving forces other than gravity are required.

Surface irrigation is broadly classified as controlled and uncontrolled flooding. In uncontrolled flooding water is spread over the land following its natural slope and topography. This sytem, though not efficient, is low in first cost and labor, which explains its wide use in the third world countries, mainly in southeast Asia where labor is cheap. Controlled flooding uses basins, furrows, borders and corrugations to spread the water over the field. This type of flooding is mostly used for reasons of better control and efficiency of water use. Specifically, border irrigation will be consideredin this study.

Border irrigation is a method where water is introduced at the upper end of a sloping plane rectangular field with low dikes along both edges of the field. Water flows in a broad shallow sheet by gravity between the two parallel dikes. the lower end of the border can be diked to contain runoff or free flowing.

In a typical graded free out-flow border system, part of the water introduced to the border at the upper end goes into soil storage, part evaporates, some water percolates past the root zone and part goes past the lower field boundary as runoff. While recognizing the importance of each of these components, this study concentrates on the runoff component of border irrigation. Estimates attribute from 5 to 30 percent of the amount of water delivered to a farm to runoff.

Part of the runoff from the field is consumed by weeds and some evaporates and is lost to the atmosphere. The remaining portion of the water, unless controlled, may pond in roads, ditches and other low areas. There are several problems associated with uncontrolled runoff including:

1. Water that would have been beneficially used by the crop is made unavailable as it runs off from the cropping area.
2. Substantial amounts of soil, nutrients, pesticides, and herbicides are carried away with the water.

3. Ponding of runoff in areas other than the crop area can act as a seed nursery for weeds and breeding area for mosquitos.
4. Pesticides and chemicals used, which are carried by the runoff, can find their way to the drinking water and streams and may be harmful both to crops and human uses.
5. Water lost during runoff by evaporation as well as phreatophytes may be a serious problem to groundwater recharge in areas where groundwater levels are declining and pumping costs are increasing.
6. Water ponding at the lower end of the field or adjoining fields for a long period of time may reduce yields.
7. Runoff from cropping area may damage farm roads and adjacent property.
8. Spreading of weeds and crop disease is possible from one area to another unless control measures are taken.

For years, these problems were ignored, and farmers were able to live with them. Today the continuous increase in the cost of energy and the need for more water have made water management measures that control the runoff water on the farm a cost effective practice to be considered in the design of irrigation systems. In Arizona the new groundwater law sets a "water duty" for each farm based on efficient water use although the state does not dictate a specific method to attain the required level of efficiency. Moreover, Public Law 92-500 treats irrigation as a point source of pollutants and an irrigation project has to consider the constraints imposed by this law for increased environmental quality.

There are several ways for controlling irrigation runoff and getting higher efficiency. These approaches include the use of properly designed sprinkler and trickle irrigation systems. No surface runoff results when water application rates are equal to or less than the infiltration rate. These systems can be used to reduce runoff but, as pointed out by Batty, et. al. (1975), use more energy per unit area irrigated than gravity systems. They also require substantially higher initial capital investment in addition to limitation on their use due to soil and topographic conditions (e.g., low intake rates).

The cutback method of water application for furrow irrigation has been suggested by some researchers (Criddle, 1956) as a means of reducing tailwater. It's effect on uniformity of application is not clearly known though lower uniformity is expected. Lack of such information has made it unpopular with farmers to date.

Use of advanced irrigation scheduling with proper irrigation water measuring devices can be used to reduce runoff. This is done by having frequent irrigations to induce uniform infiltration at both ends of the field and applying only the amount needed. This method requires more skilled labor and some automation which will be more effective for pressurized systems than for gravity systems.

In addition to the problems stated for the various approaches mentioned above, established preference for graded border irrigation systems supported by low intake soils, flat lands and good quality water seem to encourage the use of free outflowing borders.

One of the technologies that promises to increase efficiency and uniformity in addition to saving energy in free outflowing borders is a of reuse system, possibly at a small additional cost. Such a system can improve application efficiency from a present average value of about 60 percent to 80 percent with improved uniformity. It has a great potential in saving energy, (Yitayew, et. al., 1981). This approach is the main subject of this study.

Objectives

Motivation to study this approach comes from the desire to successfully design border irrigation systems that use available water in the most efficient way possible. Three objectives were developed: first, to identify pertinent open channel hydraulic variables that affect runoff characteristics in free outflowing borders; second, to develop predictive graphical solutions for calculating runoff from border irrigation and third, to utilize these solutions in establishing design criteria for reuse systems with major emphasis given to practical application in design and operation of sump and pump systems.

In meeting these objectives, open channel hydraulics phenomena of shallow, non-uniform unsteady flow over a porous media has had used as the basis. A zero-inertia mathematical model developed by Strelkoff and Katopodes (1977) has been used to simulate these phenomena.

CHAPTER II

THEORY AND PROCEDURE

The first requirement in design of irrigation runoff control structures including the reuse system and outlets, and an evaluation of the irrigation system is accurate determination of the runoff quantity and distribution with time. This is part of the general problem of hydraulics of border irrigation which is discussed below.

Border irrigation is defined as a controlled surface irrigation whereby water is introduced at the upper end of a sloping rectangular strip with low dikes along both edges of the field. The dikes define the lateral boundaries of the flow. In sloping borders the strip of land slopes downward along its length and has essentially no slope normal to the direction of flow. Hydraulically, it is considered shallow since the depth of flow to width ratio is small enough to neglect the shear along the boundaries; gradually-varied since the vertical accelerations are small compared to the total acceleration; non-uniform since the depth of flow in the downward direction decreases with increasing infiltration area; and unsteady since infiltration rate decreases with time approaching a constant value for large times.

The basic hydrodynamic equations describing the flow of water in border irrigation are the two partial differential equations of first order usually called the De Saint-Venant equations. These equations are basically continuity and momentum equations in a general form. The continuity equation in one dimension is given:

$$\frac{\partial Q}{\partial X} + \frac{\partial A}{\partial t} \pm I_x = 0 \tag{2-1}$$

and the momentum equation:

$$\frac{\alpha V}{g}\frac{\partial v}{\partial X} + \frac{\beta}{g}\frac{\partial v}{\partial t} + \frac{\partial y}{\partial x} = S_o - S_f + \beta\frac{V}{gA} I_x \tag{2-2}$$

where x and t are distance and time respectively, Q is the flow rate, A is the cross-sectional area of flow, y is the flow depth, g is the ratio of unit weight to mass density of the liquid, v is velocity of individual particles, V is the average velocity ($V = Q/A$), S_0 is the bottom bed slope, S_f is the friction slope, I_x is volumetric rate of infiltration and α and β are energy and momentum velocity distribution coefficients, respectively. Derivations of these equations were made by Henderson (1966), Strelkoff (1969), and Yevjevich and Barnes (1970). The basic derivation is given to show the qualitative values of the different variables and relate them to the computation of runoff in unsteady, shallow, gradually-varied flow of border irrigation. This helps also understand the basic assumptions involved so that the degree of agreement with the actual flow condition and the one computed can be judged.

Continuity Equation

The derivation following the Yevjevich and Barnes (1970) approach of continuity equation Eq. 2-1 is given as follows. In reference to Figure 1, at any given time t, the cross-sectional area of the flow at section X is A.

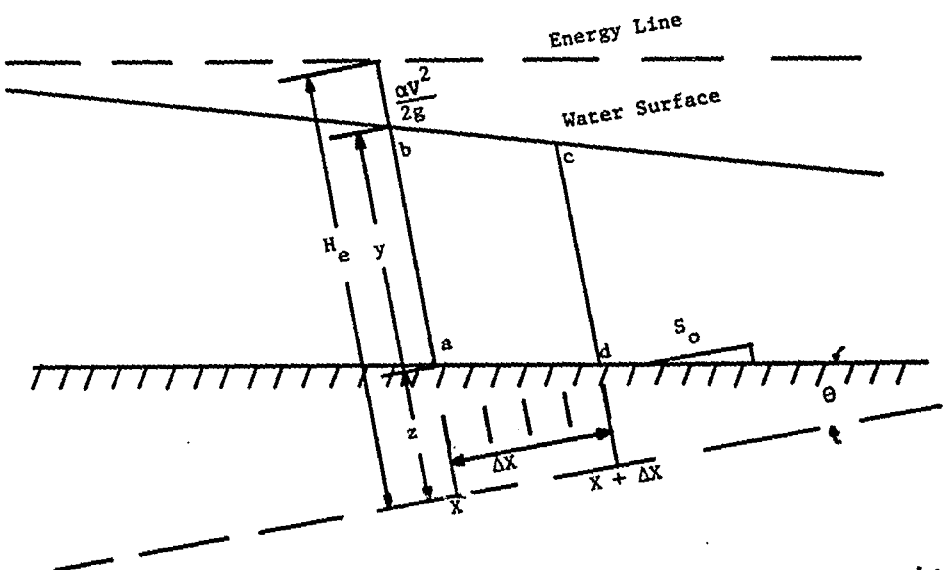

Figure 1. Definition diagram for derivation of continuity and momentum equation of unsteady free surface flow in open channel.

At the same time t and at section $X + \Delta X$, the area is $A + \frac{\partial A}{\partial X} \Delta X$ with an incremental length Δx. The mass of water between the two sections (slice abcd in Figure 1) is $\rho A \Delta X + 1/2 \rho \frac{\partial A}{\partial X} (\Delta X)^2$. Where ρ is the density of the liquid. Neglecting the higher order terms as ΔX goes to dX, the mass is $\rho A \Delta X$. Representing the lateral outflow by $-I_X$, the change of mass with time is:

$$\frac{d}{dt} (\rho A \Delta X) = \rho A \frac{d(\Delta X)}{dt} + \rho \Delta X \frac{dA}{dt} = \rho (-I_X) \Delta X \qquad (2\text{-}3)$$

The derivative $\frac{d(\Delta X)}{dt}$ where ΔX is only a function of time and as Δt goes to zero for a moving individual particle is given by using $\Delta X = v\Delta t$ as follows:

$$\frac{d(\Delta X)}{dt} = \lim_{\Delta t \to 0} \frac{(v + \frac{\partial v}{\partial X} \Delta X)\Delta t - v\Delta t}{\Delta t} = \frac{\partial v}{\partial X} \Delta X \qquad (2\text{-}4)$$

Similarly, the derivative $\frac{dA}{dt}$ is given by:

$$\frac{dA}{dt} = \frac{\partial A}{\partial t} + \frac{\partial A}{\partial X}\frac{dX}{dt} = \frac{\partial A}{\partial t} + v \frac{\partial A}{\partial X} \qquad (2\text{-}5)$$

In Equations 2-4 and 2-5 one can use the average cross-sectional velocity V from the particle velocity v by using the transformation:

$$\frac{1}{VA} \iint_A v dA = 1 \quad \text{and} \quad \frac{1}{A} \iint_A \frac{\partial V dA}{\partial X} = \frac{\partial V}{\partial X}$$

Thus the equation of continuity can be written by using Equations 2-3, 2-4, 2-5 and 2-6 with their proper transforms as:

$$A \frac{\partial V}{\partial X} + V \frac{\partial A}{\partial X} + \frac{\partial A}{\partial t} + I_X = 0 \qquad (2\text{-}7)$$

or since $VA = Q$ and $\frac{\partial(VA)}{\partial X} = V\frac{\partial A}{\partial X} + A\frac{\partial V}{\partial X}$

$$\frac{\partial (VA)}{\partial X} + \frac{\partial A}{\partial t} + I_x = 0 \qquad (2\text{-}8)$$

or

$$\frac{\partial Q}{\partial X} + \frac{\partial A}{\partial t} + I_x = 0 \qquad (2\text{-}9)$$

or since $\frac{\partial A}{\partial t} = B\frac{\partial y}{\partial t}$ with B as the width at the surface cross-section and y the depth of water above the bottom, Equation 2-9 becomes:

$$A\frac{\partial V}{\partial X} + VB\frac{\partial y}{\partial X} + B\frac{\partial y}{\partial t} + I_x = 0 \qquad (2\text{-}10)$$

Momentum Equation

The momentum (dynamic) equation is an expression of Newton's Second law for the conservation of linear momentum. In a given direction, the law is mathematically described as:

$$F = \frac{d(mv)}{dt} \qquad (2\text{-}11)$$

where m is the mass, v is the velocity of an individual particle and F is the resultant force of all the forces acting on the particle. Taking a control volume, instead of a particle, such as volume abcd in Figure 2, the movement of an elementary slice of water between section X and X + ΔX can be described. Replacing the particle velocity v by the mean velocity V in the cross-section permits the following velocity distribution coefficients to be defined:

$$\alpha = \frac{1}{AV^3} \iint_A v^2 dA \qquad (2\text{-}12)$$

$$\beta = \frac{1}{AV^2} \iint_A v^2 dA \qquad (2\text{-}13)$$

These coefficients α and β depend on the velocity distribution in a cross-section A, and consequently depend on the shape, area, roughness, and the mean flow velocity at the cross-section.

In accordance with Fig. 2, the forces acting on the control volume are given below. Pressure force P is given by:

$$P = \int_0^y \rho g \, (y-r) \, B_r dr \tag{2-14}$$

and since y and B are dependent on X while R is independent of X

$$\frac{\partial P}{\partial X} = \int_0^y \rho g \, \frac{\partial y}{\partial X} \, B_r dr + \int_0^y \rho g \, (y-r) \, \frac{\partial B_r}{\partial X} \, dr \tag{2-15}$$

Since $\int_0^y B_r dr = A$ and $\Delta X \int_0^y \rho g \, (y-r) \frac{\partial B_r}{\partial X} dr = F_1 + F_2$

See Figure 3, then

$$\frac{\partial P}{\partial X} \Delta X = \rho g \, A \, \frac{\partial y}{\partial X} \Delta X - (F_1 + F_2) \tag{2-16}$$

so that the resultant pressure force in the horizontal direction, normal to the cross-section is

$$P - [P + \frac{\partial P}{\partial X} \Delta X + (F_1 + F_2)] = \Delta P \tag{2-17}$$

and the net pressure force in the direction of the bottom slope is given by:

$$\Delta P = -\rho g \, A \frac{\partial y}{\partial X} \, \Delta X \cos \Theta \tag{2-18}$$

The right hand side of Equation 2-11 is expanded as:

$$\frac{d(mv)}{dt} = mv \frac{\partial v}{\partial X} + m \frac{\partial v}{\partial t} + v^2 \frac{\partial m}{\partial X} + v \frac{\partial m}{\partial t} \tag{2-19}$$

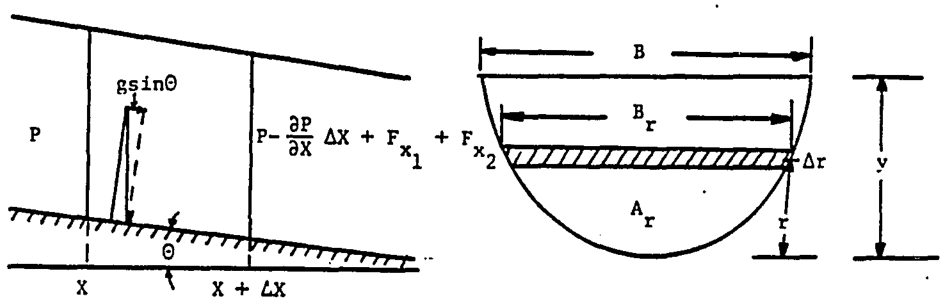

Figure 2. Forces acting on the incremental volume of the channel.

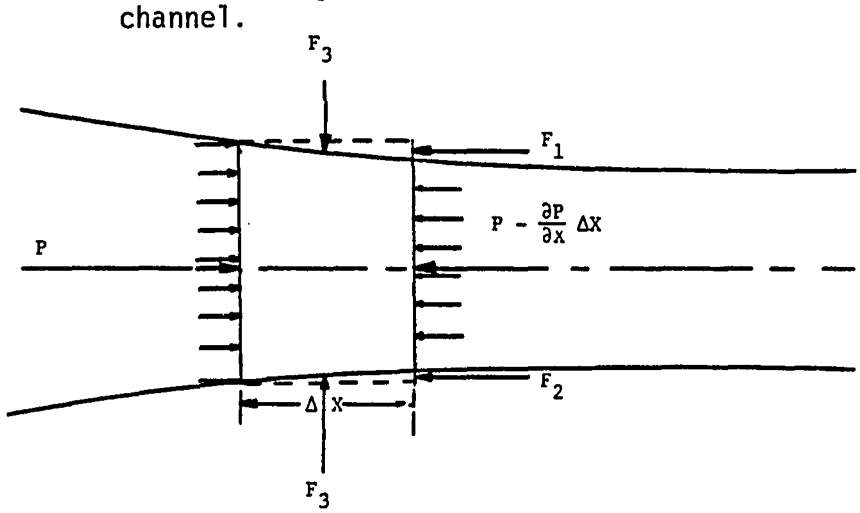

Figure 3. Plan of slice showing the forces acting on the incremental volume.

there the mass m is $m = \rho\Delta X\Delta A = \rho v\Delta t\Delta A$ for a given incremental area ΔA and incremental distance ΔX. Further manipulation of the right hand side of Equation 2-19 gives:

$$mv \frac{\partial v}{\partial X} = \rho\Delta A \frac{1}{2} \frac{\partial(v^2)}{\partial X} \Delta X \tag{2-20}$$

$$m \frac{\partial v}{\partial t} = \rho\Delta A \frac{\partial v}{\partial t} \Delta X \tag{2-21}$$

$$\frac{\partial m}{\partial x} = \rho\Delta A \frac{\partial(\Delta X)}{\partial t} = 0 \text{ as } \Delta X \text{ is independent of X and} \tag{2-22}$$

$$\frac{\partial m}{\partial t} = \rho\Delta A \frac{\partial(\Delta X)}{\partial t} = 0 \quad \text{as } \Delta X \text{ is independent of t} \tag{2-23}$$

Equation 2-19 then becomes (Equation 2-20 plus 2-21):

$$\frac{d(mv)}{dt} = \rho\Delta A\Delta X \left(\frac{1}{2} \frac{\partial(v^2)}{\partial x} + \frac{\partial v}{\partial t}\right) \tag{2-24}$$

Integration of the right side of 2-24 with the coefficients α and β included as defined by Equations 12 and 13 gives

$$\iint_A \rho \frac{\partial v}{\partial t} \Delta X dA = \beta \rho A\Delta X \frac{\partial v}{\partial t} \tag{2-25}$$

and

$$\frac{1}{2}\iint_A \rho \frac{\partial(v^2)}{\partial x} \Delta X dA = \alpha \frac{\rho A\Delta X}{2} \frac{\partial(v^2)}{\partial X} \tag{2-26}$$

Considering the momentum of the total lateral outflow $I_x\Delta X$ as

$$\rho \iint_A I_x \Delta X v dA = \rho \iint_A I_x v^2 \Delta t dA = \beta \rho I_x \Delta X V \qquad (2\text{-}27)$$

since $\Delta X = vdt$ and using 2-13.

Equation 2-11 then becomes

$$\alpha\rho A \frac{\Delta X}{2} \frac{\partial(v^2)}{\partial x} + \beta\rho A\Delta X\frac{\partial v}{\partial t} - \beta\rho I_x \Delta X V = \rho g A\Delta X \sin\theta$$

$$-\rho g A\Delta X S_f - \rho g A\Delta X \frac{\partial y}{\partial X} \cos\theta \qquad (2\text{-}28)$$

where $-\rho g A\Delta X \sin\theta$ represents the weight component, $-\rho g A\Delta X S_f$ is the friction forces with the head loss ΔH_f along the channel bottom where in the limit $\Delta H_f/\Delta X = dH_f/dX = S_f$ as $\Delta X \to 0$. For small angle θ sin θ is approximately equal to $\tan\theta$ which equals S_o or the ratio of the component in the direction of the gravitational force on the element to the weight of the element. Also $\cos\theta$ is approximately equal to unity for a small angle θ. Equation 2-28 then becomes:

$$\frac{\partial(\alpha V^2/2g)}{\partial x} + \frac{\beta}{g}\frac{\partial V}{\partial t} + \frac{\partial y}{\partial t} - S_o - S_f + \frac{\beta V I_x}{gA} = 0 \qquad (2\text{-}29)$$

or

$$\alpha\frac{V}{g}\frac{\partial V}{\partial X} + \beta\frac{\partial V}{\partial t} + \frac{\partial y}{\partial X} = S_o - S_f + \frac{\beta V I_x}{gA} \qquad (2\text{-}30)$$

which is the same as Equation 2-2. In Equation 2-30 $\partial V/\partial t$ is the consequence of unsteadiness and is known as local acceleration, $V\,\partial V/\partial X$ represent non-uniformity and is known as a convective acceleration. The term $\partial y/\partial X$ represents the unbalanced hydrostatic pressure force per unit length, per unit cross-section area, and per unit weight of the surface water in the element.

Some of the basic assumptions used in deriving the two partial differential equations for gradually varied flow of the type in border irrigation are:

1. Vertical acceleration can be neglected in comparison with the horizontal acceleration, or vertical acceleration normal to the channel in comparison with the acceleration along the channel, because they are very small due to the gradual change of depth and discharge with time and distance.
2. Pressure distribution along a vertical plane is hydrostatic.
3. Flow patterns in vertical planes parallel to the longitudinal axis of the channel are the same.
4. Velocity distribution along a vertical plane in unsteady flow is the same as the velocity distribution in steady flow for the same water depth. This implies α and β are constants for given values of discharge, depth, and velocity, or that unsteady flow does not influence these coefficients.
5. Friction resistance in slightly unsteady flow is the same as friction resistance in steady flow.
6. Channel slope is so small that $\cos\theta$ can be replaced by unity and $\sin\theta$ by $\tan\theta$.

The effects of each of these assumptions are not easy to evaluate in detail, but the total influence of all the above assumptions is small in the case of gradually varied flow of border irrigation.

The last term of the right side of Equation 2-2 and 2-30 according to Strelkoff (1969) in border irrigation is the result of net acceleration stemming from removal of zero velocity components of the surface stream at the bed of infiltration. With respect to the term I_x, if the Kostiakov-Lewis power law function is used, a reasonably good approximation for infiltration rate is given by

$$Z = kt^a \tag{2-31}$$

where Z is the cumulative infiltrated depth of water, k and a are constants, and t is the time of infiltration, i.e. the time the water has been in contact with the soil. The rate I_x is given by dividing the volumetric infiltration rate I_v by the width B, i.e. $I_x = I_v/B$. The rate I_x is then evaluated using Equation 2-31 by taking the derivative with respect to time

$$I_x = \frac{\partial Z}{\partial t} = akt^{a-1} \tag{2-32}$$

For S_f in Equation 2-30, the Manning resistance formula, which is widely used in most open channel hydraulics, can be used. Thus, S_f is given by:

$$S_f = \frac{q^2 n^2}{C_u^2 y^{10/3}} \tag{2-33}$$

where q is the flow rate per unit width, n is the Manning roughness coefficient, y is the depth of flow and C_u is a constant depending on the units used.

The Darcy-Weisbach equation in the form

$$S_f = \frac{fV^2}{8gR} \tag{2-34}$$

or the Chezy formula in the form

$$S_f = \frac{V^2}{C^2R} \quad \text{where } C = \frac{C_u y^{1/6}}{n} \tag{2-35}$$

can equally be used. In Equation 2-34, f is the friction coefficient, g is acceleration due to gravity, and R is the hydraulic radius. The f and n values in Equation 2-33 and 2-34 are related by

$$f = kn^2/D^{1/3} \tag{2-36}$$

where k is a constant equal to 185 in the English system of units and 124.45 in metric. The D in Equation 2-36 is diameter of a circular section and can be replaced by 4R for any other cross-section.

It should be noted that the use of the Kostiakov-Lewis function for infiltration and the Manning resistance formula for S_f are not necessary conditions but sufficient for this study since any relationship for infiltration or resistance can be equally used.

Equations 2-1 and 2-2, which are quasilinear hyperbolic partial differential equations of first the order, don't permit a closed form analytical solution because of their nonlinearity and nonhomogenity unless many simplifications are introduced. In irrigation channels the diversity in slope, roughness, free surface condition, and complexity of lateral flow further complicates the analytical expressions that approxi mate these conditions to an extent that it is practically impossible to integrate the equations. Due to these difficulties, approaches used to quanitify runoff from border irrigation have been limited to empirical methods. The next chapter reviews earlier works as well as recent efforts to obtain solutions of the two equations. Review is also made of on runoff determination and reuse systems to integrate it with the basic objectives of the study.

CHAPTER III

PREVIOUS WORK

Solution of Describing Equations

This section deals with the research efforts directed towards solving the governing equations in hydraulics of border irrigation and their use in estimating runoff volume and rates from a given irrigation system. Most of earlier works concentrated on solving the kinematic equation (Equation 2-1) alone in estimating runoff while later works used both kinematic and dynamic equations.

Perhaps the first original work with respect to volume balance was made by Lewis and Milne (1938) who developed an integral equation for solving Equation 2-1 relating constant inflow, average depth, cumulative infiltration, advance distance, and time, with depth of flow accomodating the effects of slope and roughness. Their equation uses a given average depth of surface water storage to replace the equation of motion governing the surface flow and uses this average value in the kinematic equation. Their equation, besides treating the advance phase only, is difficult to use but has been the basis for most later works.

Hall (1956) after studying the Lewis-Milne equation presented a numerical technique which was computationally simpler for solving the integral equation. He used a recurssive relationship to determine successive advance increments. His scheme included the effects of slope and roughness and gives flexibility for using infiltration data without any predetermined relationship. An analytical solution of the integral equation was also presented by Philips and Farrell (1964) who used Laplace transformation and presented their solution in dimensionless forms. Wilke and Smerdon (1965) used a computer algorithm to get a direct solution for a given case of infiltration using Phillips and Farrell's general equation and presented it in dimensionless curves. Fox and Bishop (1965) related advance time, inflow rate, normal depth and some kind of empirical constants to get a solution of the volume balance equation. Hart, Bassett, and Strelkoff (1968) solved the integral equation for the first time using the Kostiakov infiltration function and presented the advance trajectories in dimensionless graphs. All the above work dealt only with the advance phase of irrigation using the volume balance principle.

Several studies considered both dynamic and kinematic aspects of flow to accomodate the recession phase, too. Kruger and Bassett (1965) using an irrigation advance model they developed, related advance distance and surface depth to time, a constant inflow rate, a constant infiltration rate, constant slope and roughness. This effort was followed by a recession model (Schreiber and Bassett, 1967) that related recession distance and flow depth to time, a constant infiltration rate, constant slope and roughness to make it a complete model that takes care of all phases of an irrigation. The condition set by the advance model was used to initiate the recession model.

Kincaid, Heermann, and Kruse (1972) documented the original work by Kincaid (1970), an advance model that gives a relationship between advance and flow depth to time, inflow rate, a variable infiltration rate and a constant slope and roughness. In the same period Powell, Jensen, and King (1972) presented a model that takes care of the three phases of surface irrigation for non-uniform slopes. Bassett (1972) also published a model of irrigation advance that used the method of characteristics to solve the differential equations and tested the model using the works of Roth, et. al. (1974) who collected actual field data for evaluation of pertinent variables at the University of Arizona irrigation laboratory.

Fonken (1974), using an iterative method of solution and finite element analysis to derive an equation for the solution, presented a mathematical model that can predict depth and velocity of flow, and the subsurface profile for any given slope, length, roughness, infiltration rate, and inflow rate. His model satisfies equations of continuity and of momentum at all times throughout the advance, continuing and recession phases of an irrigation. He used field data from the University of Arizona field laboratory to test his model and obtained a reasonable agreement.

Advances in numerical methods and the recent increased capabiity of computers has made solutions of Equations 2-1 and 2-2 easier than when the old method of graphical solution by the method of characteristics was used to integrate the equations. These advances also have made it possible for researchers to treat the complete irrigation.

The later works which used the capability to solve both equations include Bassett and Fitzsimmon (1976), Katopodes and Strelkoff (1977a), and Yevjivich and Barnes (1970). In these works most of the terms in the two Equations 2-1 and 2-2 were used. The degree of approximation was the basic difference between these works. It should be realized, however, where all the terms are included, the models were complex and subject to limitations of computational stability.

Strelkoff and Katopodes (1977a) in line with the approximation proposed by Brakensick, Heath and Comer (1966) in a flood routing context, developed a mathematical model known hereafter as a zero-inertia model. The basic assumption used in the development was that the water velocities in border irrigation are generally low and the acceleration terms in Equation 2-1 and 2-2 are neglected. They demonstrated that indeed for Froude numbers at normal depth and discharge below 0.3, the forces on the surface streams are essentially balanced. Accordingly, Equation 2-2 takes the form:

$$\frac{dy}{dx} = S_o - S_f \tag{3-1}$$

This in effect shows that there is a balance between the net hydrostatic forces on the water and the component of its weight down the channel and the hydraulic drag exerted by bed and vegetation on the water in the direction of flow. This assumption leads to a pair of parabolic equations rather than hyperbolic functions when all the terms were included which makes the solution computationally less complicated and faster.

Solution of Equations 2-1 and 3-1 was made using a fully implicit scheme by Strelkoff and Katopodes (1977b). Local linearizations of the non-linear systems of equations were made to avoid the unnecessary iterative solution and to cut the execution time. The applicability and capability of the model was established by comparing with field results as reported by Strelkoff and Katopodes (1977b) and Clemmens (1977). The effect of local linearization was also verified by Fangmeier and Clemmens (1978) using field measurement with results in good agreement to zero-inertia outputs.

Fangmeier and Strelkoff (1979) and Al-Hassan (1978) independently evaluated the U.S. Soil Conservation Services (SCS) border irrigation design criteria for free outflow sloping borders and found good agreement between the SCS design criteria and zero-inertia model results. Other works using this model include studies of advance functions for level borders (Clemmens and Strelkoff, 1979), determination of the ultimate, post irrigation subsurface profile (Shatanawi, 1980), and development of design charts for ponded borders (Abdel-Rahman, 1981). All these works obtained satisfactory results using the model for the different studies made.

Recently Clemmens (1981) verified the model's capability to predict runoff from border irrigation using actual field data collected by Roth et al (1974) at the University of Arizona irrigation laboratory and obtained a very good agreement between the hydrograph produced by the model and the field measurements.

Based on the above works to justify the capability of the model to treat a wide range of variables and ability to simulate field conditions, the zero-inertia mathematical model was used for studying the runoff characteristics. It was also used to develop design curves which are used to develop reuse system design criteria.

Runoff determination

Design of reuse systems is influenced by the method used to predict runoff from irrigated lands. The solution methods presented earlier by different investigators can be used to estimate runoff. In fact, volumetric development has been the basis for most of the attempts made to quantify runoff. Empirical procedures were the most widely used approach for lack of any analytical solutions. Methods used to estimate precipitation induced runoff are based on probabilistic approaches (e.g. frequency analysis), double mass analysis, rating curves and others. These methods, of course, are not applicable for runoff from irrigation borders which is more or less deterministic. Even most of the available runoff predicting methods from irrigated fields were made mainly for furrow irrigation though they can possibly be used for border systems with some modifications. When applied to furrows these methods are also limited to very narrow ranges of pertinent variables which limit their applications. With the above as a background, this study tries to cover wide ranges of variables and avoid some of the limitations experienced by earlier studies in developing runoff determination procedures. Some of the works done in this respect are reviewed below.

Most of the earlier works dealt with direct measurements. Marsh (1956) made 32 separate measurements and reported an average runoff of 31 percent of the water applied to the farm. Shockley (1959) used a 660-foot row and 12-hour set applying 5.67 inches of water and measured the runoff. He found surface runoff losses of 35 percent.

Tyler (1964) studied three large farms and reported an average runoff of 18.5 percent of the water delivered to the farms. At the same period Davis (1964) reported 10 to 20 percent runoff from farms averaging 160 acres in California.

Several investigators concentrated their efforts on developing their own empirical equation for predicting runoff. Among them was notably Bondurant (1969) who made an extensive contribution in this respect. He made direct measurement of runoff from 105 irrigated farms and reported an average runoff of 11.6 percent. At the same time he used a volume balance approach and got an equation that predicted runoff given infiltration and irrigation advance data for furrows. Willardson and Bishop (1967) presented a method that requires intake rate, rate of advance of the furrow stream, depth of irrigation and physical dimensions of the field to estimate runoff amount. Their method predicted a minimum of 20 percent runoff with stream advance time to total irrigation time ratio of 0.20.

Ohmes and Manges (1972) measured inflow and outflow from a number of furrows during irrigation and expressed the runoff discharge in terms of maximum runoff discharge, runoff period, and some arbitrary field coefficients. They basically integrated the runoff equation they got for the different zones of the hydrograph and determined the volume of runoff.

Pope and Barefoot (1973) investigated the variation in runoff volume and rate from irrigated sets on the same field and found that runoff percentages for the irrigation sets follow a log normal distribution. They have also studied the time distribution of the runoff water from individual furrows and the economic feasibility of using irrigation runoff recovery system based on the time distribution and probability analysis.

Wilke (1973) using volume balance and a stream advance parameter developed a theoretical runoff predicting equation and presented the relationship in graphical form. He also studied the effect of time on both volume of runoff and irrigation uniformity for furrow method.

Merriam (1975) used general furrow irrigation information including texture, intake rate, slope, and furrow length, to develop curves that predict an approximate runoff rate.

Reuse System Design

Davis (1964) was probably the first to come out with a reuse system design. He used equations developed by Larson and Allred (1956) for drainage pumps to relate inflow, pump capacity, sump volume and cycle time to come up with the minimum storage required for a given maximum cycles per hour. Larson and Manbeck (1961) had studied the effect of cycle length on pumping plant efficiency and recommended a design cycle length of 4 to 8 minutes with a median value of 6 minutes and 10 cycles per hour for most farms.

Bondurant (1969) developed some techniques to design an irrigation runoff recovery system (IRRS) with continuous or intermittent pumping operations. He proposed a method of using runoff water to develop a "cutback" system, i.e., an operating system to achieve a reduced furrow stream input.

Fischbach and Somerhalder (1971) studied irrigation efficiency with automated irrigation systems with and without IRRS and found that irrigation efficiencies and uniformity coefficients of 92 percent could be attained when runoff water was used on adjacent plots. The IRRS system they used was a cut-back system similar to the one described by Bondurant (1969). Davis (1964) and Bondurant (1969) concluded that the size of the sump depends on the value of land and the degree of control required to return runoff water back to the field.

Stringham and Hamad (1975) demonstrated design of an IRRS that eliminates the need for "cutback" type of irrigation by using a constant supply of stream discharge and a constant number of furrow sets. Their system makes use of all the water applied to the field without putting any on adjacent fields.

A theoretical analysis neglected by all the above investigators was made by Schneider (1976) who considered the magnitude of irrigation tailwater losses from a reuse system. He also presented tailwater utilization curves which indicate the basic parameters to consider in design of reuse systems.

Cost Estimates

A limited number of investigators considered cost analysis in their development of reuse systems for furrow irrigation. Those works found in the literature are old and can be used only if proper price indexing is made before use. Davis (1964) estimated annual costs for tailwater systems surveyed in California and gave $1.50 per acre-foot pumped to a nearby outlet or field and $3.00 per acre-foot for pumping back into the upper end of the same field.

Bondurant (1969) used a system to deliver runoff water to a lower field at the farm delivery point for different pipe sizes and pumping rates, and showed the most economical pipe size as a function of runoff rate.

Cost analysis for an irrigation reuse system in Arizona was made by Halderman (1967). He showed a total annual cost of $5.00 per acre-foot of water delivered is an example using a 20-year life and 7 percent interest for a system with 450 gpm pump, a 5 horsepower electric motor, 2 acre-foot storage capacity and 2000 feet of 8 inch asbestos cement pipe. His cost included construction and installation costs.

CHAPTER IV

ZERO-INERTIA MODEL DESCRIPTION AND ANALYSIS PROCEDURES

The availability of computers with more storage capacity and considerable speed made possible the use of mathematical models to solve the Saint Venant equations with full retention of all the terms. Use of all the terms gives a more accurate result than using only a few terms. The problem with this is the expense involved and sometimes computational instability. Bassett and Fitzsimmons (1976) presented a fully hydrodynamic model that was tested and proved to give accurate results. But it was too expensive and sometimes computationally unstable to be used extensively.

The model developed by Katopodes and Strelkoff (1977a) is a hydrodynamic model that has been verified to give accurate results. The model as mentioned earlier is based on the assumption that the acceleration terms in Equation 2-2 can be neglected for border irrigation. This leads to a pair of equations of parabolic rather than hyperbolic type. The solutions are less complicated and computationally more stable. The cost of computation is found to be much less than the full dynamic model mentioned above. For these reasons this model was adopted for this study. The subsequent discussions summarizes the numerical techniques used in solving the two equations.

By the assumption of negligible inertial terms, Equations 2-1 and 2-2 are reduced to

$$\frac{\partial y}{\partial t} + \frac{\partial q}{\partial X} + \frac{\partial Z}{\partial t} = 0 \tag{4-1}$$

$$\frac{\partial y}{\partial X} = S_o - S_f \tag{4-2}$$

with two unknowns, y and q.

While the reader is advised to refer to Strelkoff and Katopodes (1977) for detail, the information presented here is sufficient to introduce the basic numerical technique used by the developers of the model.

Refering to Figure 4, the solution to these equations is sought in that region of the x - t plane, lying between the axis of ordinates, x = 0 and the wave-front trajectory, X(t). The solution is obtained on a sequence of constant-time lines, $t_i(i = 1, 2, 3, ...)$ separated by constant increment of time, dt. On each time line, the solution is obtained at a sequence of node points s_K, (K = 0, 1, 2, ...N, N=i), defined by the location of the wave front at the successive instants of time. In the course of obtaining the depth and discharge profiles in the interior of the wave $(0 - s - X_i)$, at each t_i, these locations $X_i(i = 1, 2, 3, ...)$ are found.

The solution is built up on the basis of the solution for the preceding time step, and on the known discharge, q_O at S = 0. The profiles are thus found at consecutive times t_1, t_2, t_3 ...etc. To advance the solution from one time step to another, e.g., t_{i-1} to t_i, Equations (4-1) and (4-2) are numerically integrated over a sequence of

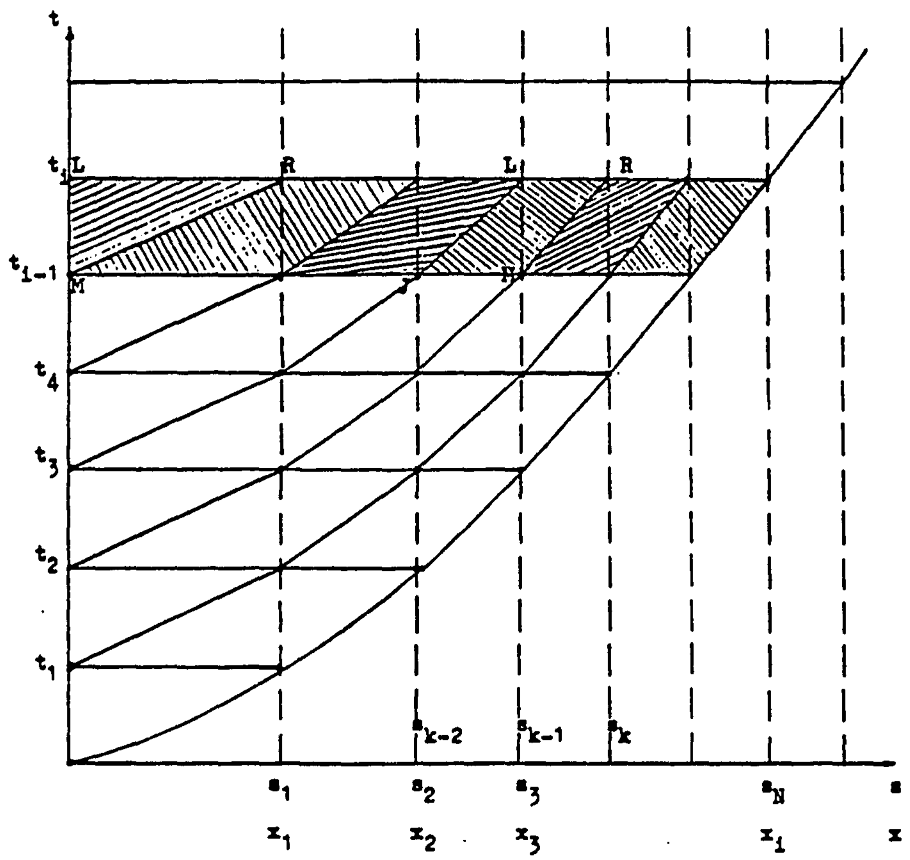

Figure 4. Computational grid during advance.

small cells of the s-t plane shown shaded in Figure 4. The integrals are expressed approximately in terms of the known values of depth and discharge on the time line t_{i-1} and in terms of the unknown values at the new time t_i. This gives a set of simultaneous non-linear algebraic equations in the unknown values. To reduce execution expense, the equations are locally linearized, avoiding the need for an iterative solution. The resulting set of simultaneous linear algebraic equations is solved by a double-sweep technique.

During the advance phase the cells are taken in the oblique form in the x-t plane as shown in Figure 4. The continuity equation is integrated over the cell MJLR in Figure 4, with reference made to the deforming region expected in Figure 4. At time t_{i-1}, the flow region bounded by stations S_{k-2} and S_{k-1}, has a surface depth of Y_j and a subsurface depth Z_j at its left boundary, and surface and subsurface depths of Y_m and Z_m, respectively at its right boundary.

The variables Y_j, Z_j, Y_m, Z_m, Y_l, Z_l, Y_R, Z_R and q_j, q_l, q_m, q_R on Figure 5 are all related through the equation of continuity integrated over the deforming region of space. This has the form (Strelkoff, 1972)

$$\frac{dV}{dt} + \int_s (\overline{V}-\overline{W}) \cdot \overline{n}\, ds = 0 \tag{4-3}$$

Integrated over time period dt, Equation 4-3 has the form

$$\forall_{t_i} - \forall_{t_{i-1}} = -\int_{t_{i-1}}^{t_i} \left\{ \int_s (\overline{V} - \overline{W}) \cdot \overline{n}\, ds \right\} dt \tag{4-4}$$

where $\forall$ = volume of water-filled region of space, v = water-velocity vector at a point on the surface of the region, $\overline{W}$ = velocity of the surface at the point, $\overline{n}$ = outward-directed unit normal to the surface, where $(\overline{V} - \overline{W})\overline{n}$ multiplied by the element of surface area dS is the elementary discharge crossing the surface there. The surface integral is taken over the entire surface area bounding the region. The region is bounded at the top by the free-surface profile and on the bottom, by the profile of the infiltrated water.

For the interior cells, the integrands in Equation 4-4 are represented in the numerical scheme used by the present version of the model by the weighted average of their values at the grid points, so that the integrated equation of continuity appears as

$$[(y_L + z_L)\emptyset + (y_R + z_R)(1 - \emptyset)]\delta s_k - [y_j - z_j)\emptyset + (y_M + z_M)(1 - \emptyset)]$$

$$(\delta s_{k-1}) = (\Theta \{[q_L - (y_L + z_L)\frac{\delta s_{k-1}}{\delta t}] - [q_R - (y_R + z_R)\frac{\delta s_k}{\delta t}]\} \qquad (4\text{-}5)$$

$$+ (1 - \Theta \{[q_j = (y_j + z_j)\frac{\delta s_{k-1}}{\delta t}] - [q_M - (y_M + z_M)\frac{\delta s_k}{\delta t}]\})\delta t$$

Equation (4-4) and (4-5) are modified according to the boundary cell used; that is whether it is the right boundary cell or left boundary cell.

The flow is nearly stationary as soon as the runoff begins when the irrigation stream arrives at the end of the field. For the

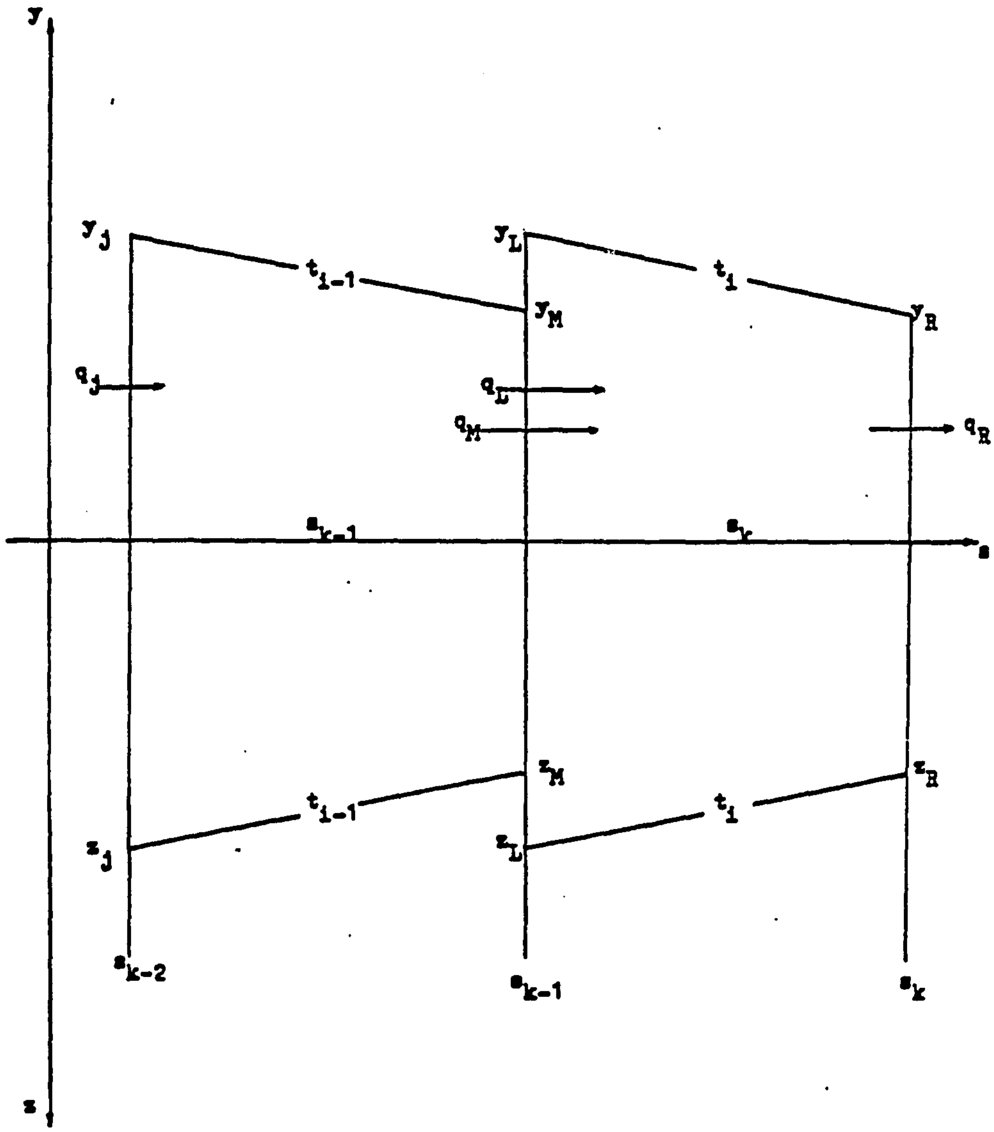

Figure 5. Deforming fluid element during advance phase.

stationary phase, the computational cells are changed from oblique cells to rectangular cells, as shown in Figure 6. The equation of continuity for the end cells is modified to reflect the free-draining end of the channel. The end depth is zero while the velocity increases without bound so that their product, the runoff, remains finite.

In a more general form Equation (4-4) can be written as

$$(\overline{q}_{r_D} - \overline{q}_{r_U})\delta t = [(\tilde{y} + \tilde{Z})\delta x]_t - [(\tilde{y} + \tilde{Z})\delta X]_{t+\delta t} \tag{4-6}$$

without changing its meaning. In the same manner the equilibrium of forces per unit weight of water Equation 2-29 acting on the region can be written as:

$$y_U - y_D = (\tilde{S}_o - \tilde{S}_f)\delta X \tag{4-7}$$

In Equations 4-6 and 4-7, q = unit discharge crossing the face of an element of the stream, y = surface flow depth, z = subsurface depth (volume infiltrated per unit plan area of border), δX = length of element, and δt = time increment. The subscripts U and D refer to the upstream and downstream faces of an element respectively. The bars over a variable indicate a time average over δt and the tilde signifies a distance average over δX.

Equations 4-6 and 4-7 were the basic finite difference equations used in the zero-inertia model subject to the following boundary conditions:

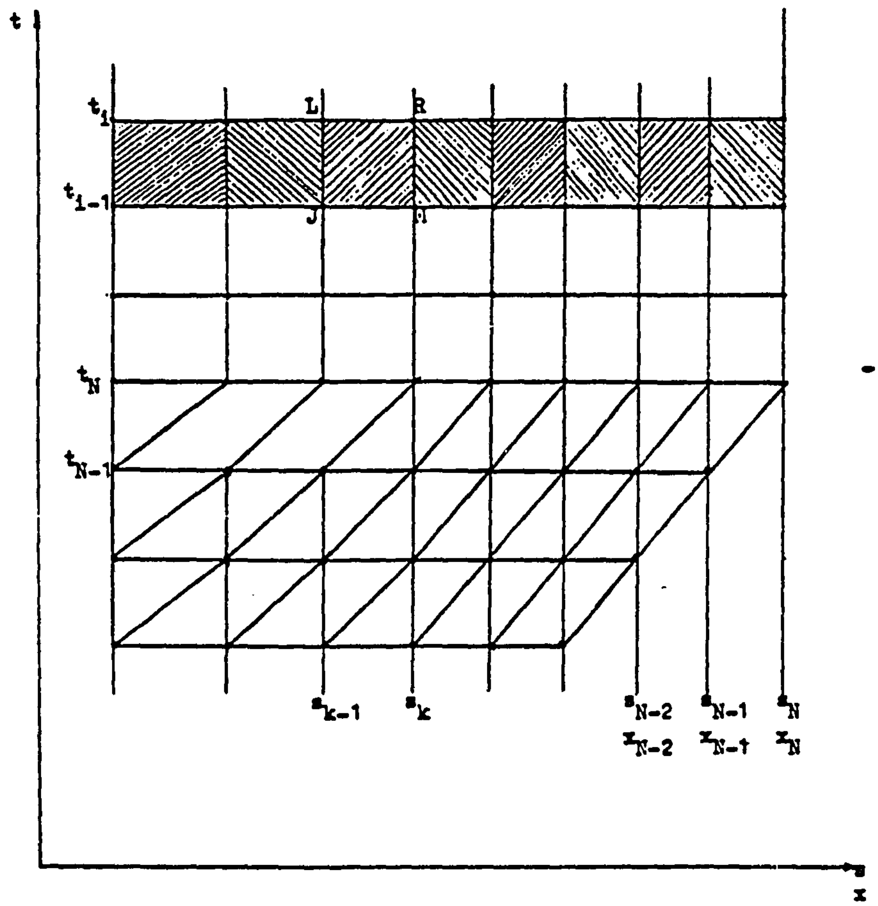

Figure 6. Computational grid nearly stationary phase.

$$q_o = q_{in} \qquad 0 < t < t_{co} \qquad (4\text{-}8)$$

$$q_o = 0 \qquad t > t_{co} \qquad (4\text{-}9)$$

$$q = o = y \qquad X = X_A \qquad (4\text{-}10)$$

$$q = y = o \qquad t = o \qquad (4\text{-}11)$$

where q_0 = flow rate at the upstream and of the border, t_{co} = cutoff time, q_{in} = constant unit discharge (inflow rate), and X_A = length of advancing stream at any time.

Equations 4-6 and 4-7, subject to the boundary conditions given by Equation 4-8 to 4-11, were integrated using a fully implicit numerical solution discussed earlier. Both dimensional and non-dimensional solutions were used in this study. The dimensional solution was utilized in studying the runoff characteristics and evaluating the pertinent design variables with respect to runoff in sloping borders and are discussed in the next chapter. Non-dimensional solutions were used in the development of the runoff curves. Discussion of the analysis is the subject treated below.

Dimensional Analysis

Considering the number of variables affecting runoff characteristics from a given irrigation it is straightforward to see that a thorough examination of each of the variables in dimensional terms is practically impossible. Besides, presentation of the results would require too many graphs. To reduce the number of physical variables, to make possible presentation of a large number of solutions, and to help understand the theoretical limits of the physical variables in a cheaper and faster way, dimensional analysis was used to develop dimensionless runoff curves which are used in designing reuse systems.

Equations 4-6 and 4-7 can be put in dimensionless forms by choosing non-zero reference variables Q, T, X, Y and Z which have the same units as their counterpart physical dimensional variables, i.e. length^2/time, time, length, length and length, respectively. The dimensionless variables are then defined by:

$$q^* = q/Q; \quad t^* = t/T; \quad y^* = y/Y; \quad x^* = x/X; \quad z^* = z/Z;$$
$$t_{co}^* = t_{co}/T; \quad L^* = L/X; \quad q^*_{in} = q_{in}/Q \qquad (4\text{-}12)$$

Substituting these variables into Equations 4-6 and 4-7 and dividing Equation 4-6 by XY and Equation 4-7 by Y yields:

$$V^* (\overline{q}^*_{r_D} - \overline{q}^*_{r_U})\delta t^* = [(\tilde{y}^* + K^*\tilde{z}^*)\delta x^*]_{t^*} - [(\tilde{y} + K\tilde{z}^*)\delta x^*]_{t^* + \delta t^*} \qquad (4\text{-}13)$$

and

$$y^*_U - y^*_D = (S^*_o - S^*_f)\ \delta x^* \qquad (4\text{-}14)$$

also

$$q^*_o = q^*_{in} \qquad o<t^*\leq t^*_{co} \qquad (4\text{-}15)$$

$$q^* = 0 \qquad t^*>t^*_{co} \qquad (4\text{-}16)$$

$$q^* = 0 = y^* \qquad X^* = X^*_A \qquad (4\text{-}17)$$

in which

$$V^* = \frac{QT}{XY} \qquad (4\text{-}18)$$

$$K^* = Z/Y = \frac{kT^a}{Y} \tag{4-19}$$

$$S_o^* = S_o X/Y \tag{4-20}$$

$$q^*_{in} = q_{in}/Q \tag{4-21}$$

$$S_f^* = S_f\ X/Y \tag{4-22}$$

$$t_{co}^* = t_{co}/T \tag{4-23}$$

$$L^* = L/X \tag{4-24}$$

and

$$Z = kT^a \tag{4-25}$$

The friction slope is computed using the reference depth and discharge using the Manning formula as:

$$S_f = \frac{Q^2 n^2}{Cu^2 Y^{10/3}} \tag{4-26}$$

Simplification of Equations 4-13 and 4-14 is possible if the reference variables are properly selected. Katopodes and Strelkoff (1976) after choosing first the characteristic discharge equal to the inflow discharge per unit width at the upstream end of the field, defined a characteristic depth equal to the normal depth for the characteristic discharge using the Manning equation:

$$Y = Y_n = \left(\frac{q_{in} n}{Cu\sqrt{S_o}} \right)^{3/5} \tag{4-27}$$

Then the characteristic values T and X are related through the normal velocity $V_n = q/Y_n$, and by $X = V_n t$. As the result of this selection the dimensionless parameters in the governing equations are found to be K^*, a, and S_0^*.

Clemmens (1978) working with the same governing equations showed for sloping borders if Y was set equal to normal depth and So, S_f*, V*, and Q_{in}* were all set to unity and the T is defined such that T = Y_n/Y_nS_0 and correspondingly K* = KT^a/Y and X = Y/So, the dimensionless solutions would be governed by K*, a, and t_{co}*.

As one can observe, runoff from a given irrigation is a function of many variables expressed mathematically as:

$$R = f(q_{in}, So, n, tco, k, a, L) \tag{4-28}$$

From dimensional analysis principles five dimensionless terms are needed. The choices are K*, a, L*, q_{in}*, |F, S_0*, t_{co}*, V* and S_f*. Using a |F = 0 by assumption of zero intertia only three need to be selected from K*, L*, Q_{in}*, S_0*, t_{co}*, V*, and S_f* the rest arbitrarily set to constants. For this study q_{in}*, V*, S_f* were set to unity. This leaves K*, L*, S_0*, t_{co}* to choose from based on the range of variation.

Clemmens (1978) showed by letting K* vary instead of S_0*, the range of variation in K* would be about 0.1 to 10 while in S_0* the variation is from 0 to ∞. Strelkoff and Clemmens (1980) indicated that within the important ranges of Kostiakov's constant a, the ranges of other physical parameter vales were found to be $1.858<q_{in}<23.226$-(1/s-m); $0.02<n<0.25\ m^{1/6}$; $1.06<K<12.7(cm/hr^a)$; $0.002<So<0.01$; and $30<L<304(m)$. If the limit on tco is $0.5<t_{co}<5(hr)$ then the practical range for K* is $0.1<K^*<10$ and for t_{co}* is $0.1<t_{co}^*<100$. This was based on the assumption that all extremes will not occur at the same time. Based on these ranges then S_o* was set arbitrarilly to unity and the dimensionless solution for developing the runoff curves in this study was governed by t_{co}*, K*, L*, and a.

The extent of graphical representation is controlled by the number of free parameters. A page of curves relating runoff to t_{co}* and L* would be required for each K* and one value of a. Further simplication was not possible i.e. the importance of each of the parameters K*, t_{co}*, L* and a was such that it was not possible to set any one of them to an arbitrary constant. Thus several curves for each value of K* and one value of a are presented.

Procedure

In this study both dimensional and non-dimensional input data were used with the latest version of the zero-inertia model. The dimensional form was used to study the effects of border irrigation variables on runoff. The input parameters in the dimensional solution are field length (L), field slope in the direction of run with zero cross slope, soil infiltration function as described by Kostiakov's power law function where k and a are used, the Manning roughness coefficient (n), inflow rate (q_{in}) and field end condition which in this case is open.

The model output includes the runoff value, runoff rate as a function of time or the runoff hydrograph, advance and recession distance and times, infiltrated volume and ultimate subsurface profile, volume stored depth of flow, and various measures of efficiencies and uniformity.

Values of k and a for the Kostiakov power law function $Z = kt^a$ were selected to describe the basic intake families given by Soil and Water Conservation Service (1974) design charts. These values were taken from the works of Fangmeier and Strelkoff (1979) and are given in Table 1. Because of the importance of soil infiltration characteristics several runs with different k and a values were made.

Slopes S_o ranging from 0.0005 to 0.01 and Mannings roughness n ranging from 0.04 to 0.35 were studied for each intake family.

The time of application or time of cutoff T_{co} was determined by using the relationship $T_{co} = T_n - T_L$ where T_n is the time needed to infiltrate a given required depth given by SCS (1974) from field studies (Table 2) and T_L is the recession lag time calculated using the equation:

$$T_L = q_{in}^{0.2}/(K_u(Cu/n)^{1.2}S_o^{1.6}) \qquad (4\text{-}29)$$

where K_u = 478 when q_{in} is in l/s-m. This equation was also given by SCS and is an adequate measure of recession-lag time which assures adequate irrigation at the upper end of the field.

As to the nondimensional runs, in accordance with the choice of the system of dimensionless variables that is setting Y to normal depth, S_o^*, S_f^*, V^* and q_{in}^* to unity with K^*, t_{co}^*, a, and L^* as free parameters the physical variables of the border irrigation are related as follows:

$$Q = q_{in} \qquad (4\text{-}30)$$

Table 1. Values of k and a in $Z = kt^a$ derived for the SCS intake families using 50- and 100-mm depths. (Fangmeier and Strelkoff, 1979).

Intake Family	k		a
	in/hr^a	mm/hr^a	
0.1	0.494	12.55	0.595
0.3	0.892	22.66	0.650
0.5	1.252	31.80	0.684
1.0	2.016	51.21	0.706
1.5	2.648	67.26	0.718
2.0	3.244	82.40	0.726
3.0	4.318	109.70	0.735
4.0	5.298	134.60	0.740

Table 2. Intake opportunity time, T_n, in minutes for required depth of infiltration of six intake families (SCS Handbook, 1974).

Intake Family	Required depth of infiltration (mm)				
	50	75	100	125	150
0.3	208	392	604	841	1100
0.5	119	217	328	450	580
1.0	59	106	158	214	273
1.5	40	72	106	143	181
2.0	31	54	80	107	136
3.0	21	37	54	72	91

$$Y = y_n = \left(\frac{q_{in} n/C_u}{\sqrt{S_o}}\right)^{3/5} \quad (4\text{-}31)$$

$$X = Y/S_o \quad (4\text{-}32)$$

$$T = YX/Q \quad (4\text{-}33)$$

$$K^* = kT^a/Y \quad (4\text{-}34)$$

$$L^* = L/X \quad (4\text{-}35)$$

$$t_{co}^* = t_{co}/T \quad (4\text{-}36)$$

where the variables have been defined earlier.

Considering possible values in actual field conditions of the physical variables, the following ranges of the dimensionless variables used as an input were considered; $0.1<K^*<10$, $0.1<t_{co}^*<100$, and $0.1<L^*<100$ for a single value of a. The SCS (1974) description of the intake families define the value of a approximately between 0.6 and 0.8 with an average value of 0.7. This value was used to develop the dimensionless runoff curves such that adjustment for any other value can be made with k to give the same cumulative infiltration depth at the end of the irrigation. Table 3 shows how an a value different from 0.7 can be adjusted using the required depth of infiltration and the intake opportunity time. As an example, a soil with an a value of 0.706 has been changed to different a values by changing the k value as shown in Table 3. There was no significant difference in the runoff percentage as the result of the adjustment to other intake families. This is a great savings both in computer time and money in that the result of the study made for an a value of 0.7 or intake family 1.0 can be used for any other value of a or intake families.

Table 3. Adjustment of k for different a values in $Z = kt^a$.

Intake Family	k mm/hr^a	a	Runoff %
1.0	68.99	.400	21.15
	62.56	.500	20.58
	56.90	.600	19.98
	51.20	.706	20.21
	46.84	.800	18.81
2.0	90.55	.400	13.58
	87.44	.500	13.77
	85.50	.600	12.69
	82.40	.728	12.77
	80.72	.800	12.60

Sensitivity of the result to the time step used was recognized. The smaller time step used the greater the number of steps and resulted in lesser error in estimation, but was more expensive to run. The time step in dimensionless form δt^* is a function of a, K^*, and t_{co}^*; larger values of δt^* should be selected for low values of these parameters. Shatanawi (1980) gave recommended values of the number of cells which controls the computer execution time. The number of cells (N) were related to t_{co}^* and δt^* by $N = t_{co}^*/\delta t^*$. These values were used in this study as guides but an N value of 30 was the minimum used for this study even though his recommendation goes below 30, because experience indicated values less than 30 gave percentage volume errors higher than could be reasonably accepted.

CHAPTER V

ANALYSIS OF RUNOFF IN BORDER IRRIGATION.

Factors Affecting Runoff

The most important variables in the design of reuse systems are the quantity and time distribution of runoff. As indicated by Equation 4-28 runoff from a sloping border is a function of inflow rate (q), time of cutoff (t_{co}), length of run (L), hydraulic drag or surface resistance expressed by the Manning roughness factor (n), border slope (S_0), and soil infiltration characteristics as represented by k and a in the Kostiakov power law function.

Runoff in general increases with increased q, t_{co}, and S_0 and decreases with L, k, a, and n. Several hundred computer simulations were made to study the extent to which these variables affect runoff. The results of these runs are discussed below.

Figure 7 shows the effect of slope on runoff. There is an increase in runoff, as expected, with slope. Interesting to note is that once a certain steepness is reached further, slope increase does not seem to increase runoff. Also for low intake families, the change in runoff per unit change in slope is smaller than for higher intake families.

The effect of roughness is shown in Figure 8 where, as could be foreseen, runoff decreases with an increase in roughness. Again the rate of decrease in runoff per unit increase in roughness is gradual for low intake families. What this means is that the values of both roughness and slope have to be determined carefully when they are to be used as independent variables with soils having a high intake rate.

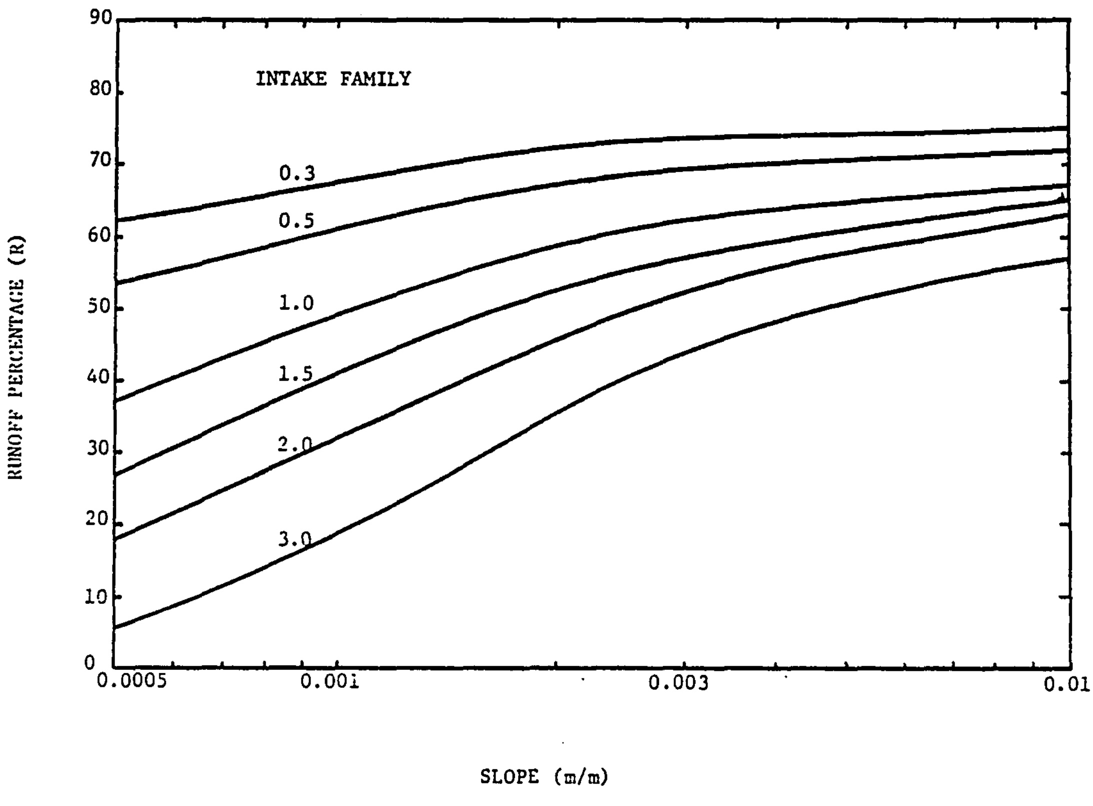

Figure 7. Effect of slope on runoff.

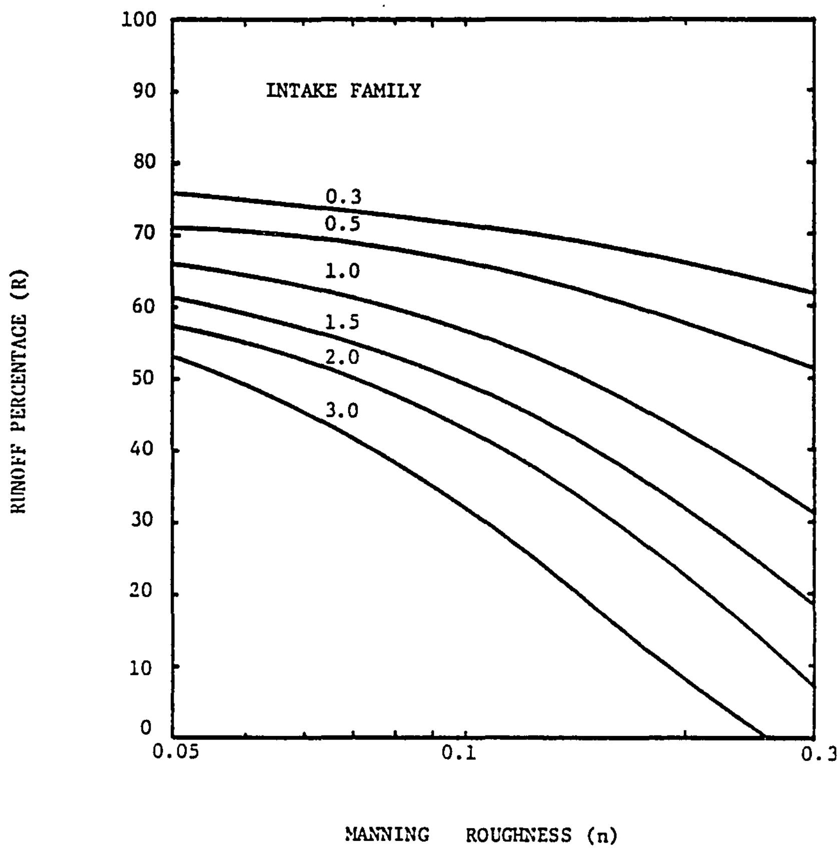

Figure 8. Effect of roughness on runoff

Table 4. Effect of selecting the next higher or lower intake family on runoff. Required depth of infiltration, Z_r = 100 mm; Length ofborder, L = 200 m; Manning's roughness, n = 0.15; Border slope, S_o = 0.001.

Intake Family	k mm/hr	a	Runoff %	Uniformity	Storage Effic., %	Appli. Effic.,%
	12.55	.595	61.68	.939	53.75	38.32
0.3	22.66	.650	28.13	.875	98.59	69.66
	31.80	.684	3.45	.509	98.52	69.05
	22.66	.650	46.76	.947	74.89	53.24
0.5	31.80	.684	26.72	.850	99.13	69.58
	51.21	.706	0.00	.055	94.63	66.17
	31.80	.684	46.96	.908	75.02	53.04
1.0	51.21	.706	22.22	.828	99.52	70.59
	67.26	.718	7.18	.637	99.46	69.76
	51.21	.706	33.66	.904	94.17	66.34
1.5	67.26	.718	19.12	.801	99.71	70.74
	82.40	.726	9.18	.670	99.69	69.78
	67.26	.718	26.01	.862	99.46	70.52
2.0	82.40	.726	15.60	.765	99.83	71.06
	109.70	.735	2.54	.516	98.83	69.89
	82.40	.726	27.45	.883	99.33	69.97
3.0	109.70	.735	14.98	.804	100.00	70.39
	134.60	.740	6.19	.667	99.87	70.35

The most difficult input parameters to determine in the field, but which are very crucial in the design of any irrigation system, are the infiltration constants. Model runs were made to see the effects of an incorrect choice of intake family for a given soil. The results, as given in Table 4, indicate that selecting a higher family decreases runoff while selecting a lower intake family increases runoff. More interesting is the change in Ea (application efficiency). While the changes in runoff are large, the application efficiency changes are small amounts. Therefore, whatever is gained by reducing the runoff is lost to deep percolation. This is also reflected by the change in uniformity. Low uniformity values associated with low runoff in all cases indicate that allowing higher runoff will provide an increased uniformity; the runoff water can then be reused to increase efficiency.

Tables 5 and 6 indicate the results assuming a 20 percent error was made in one or the other of the constants k or a for a given family. In both cases an indirect relationship exists between the estimates of the constant and runoff; i.e., an under estimation of k overly estimates runoff.

No significant change in runoff was observed when the slopes of the infiltration functions were changed while keeping the same cumultive depth of infiltration. That is, changing k and a in $Z = kt^a$ for the same Z and t values. This result indicates that the time to infiltrate a given depth, not the shape of the infiltration function, is more critical in determining the runoff volume for a given soil as shown in Table 7.

Table 5. Effect on runoff percentage of a 20% change in k. Required depth of infiltration, Zr = 100 mm; Length of border, L = 200 m; Manning's roughness, n = 0.15; Border slope, S_0 = 0.001

	RUNOFF (%)					
	Intake Family					
	0.3	0.5	1.0	1.5	2.0	3.0
1.2	17.56	16.11	12.96	9.86	7.71	6.72
1.0	28.60	26.80	23.07	19.12	15.60	14.99
0.8	41.19	39.03	34.78	30.39	26.44	24.11

Table 6. Effect on runoff percentage of a 20% change in a. Required depth of infiltration, Zr = 100 mm; Length of border, L = 200 m; Manning's roughness, n = 0.15; Border slope, S_0 = 0.001

	RUNOFF (%)				
	Intake Family				
	0.3	0.5	1.0	1.5	2.0
1.2 a	8.74	11.92	15.50	15.63	14.57
1.0 a	28.60	26.80	23.07	19.12	15.60
0.8 a	45.55	40.10	30.81	24.21	18.58

Table 7. Effect of changing k and a with the same depth of infiltration for a given time. Required depth of infiltration, Zr = 100 mm; Length of border, L = 200 m; Manning's roughness, n = 0.15; Border slope, S_0 = 0.001

Intake Family	k mm/hr	a	Application Efficiency,%	Uniformity	Storage Efficiency,%	Runoff %
	40.36	.400	69.17	.887	98.14	29.25
	32.05	.500	69.14	.874	98.35	28.78
0.3	25.43	.600	69.35	.869	98.64	28.0
	22.66	.650	69.52	.869	98.84	27.63
	16.00	.800	70.26	.890	99.46	26.18
	51.51	.480	69.70	.884	99.33	26.97
	43.46	.500	69.49	.870	99.24	26.55
0.5	36.88	.600	69.97	.852	99.22	25.38
	31.80	.684	69.96	.851	99.36	24.83
	26.11	.800	70.14	.856	99.58	23.97
	68.99	.400	70.98	.882	99.99	21.15
	62.56	.500	70.90	.852	99.90	20.58
1.0	56.90	.600	70.70	.817	99.78	19.98
	51.21	.706	69.87	.809	99.75	20.21
	46.84	.800	70.64	.784	99.60	18.81
	80.92	.400	70.60	.881	100.00	17.53
	76.43	.500	70.39	.844	100.00	16.99
1.5	72.21	.600	70.12	.807	99.97	16.60
	67.26	.718	70.38	.760	99.83	15.87
	64.44	.800	70.89	.733	99.58	14.77
	90.55	.400	71.06	.873	100.00	13.58
	87.99	.500	70.11	.833	100.00	13.77
2.0	85.50	.600	70.61	.778	99.99	12.69
	82.40	.728	69.91	.720	99.89	12.77
	80.72	.800	69.77	.711	99.76	12.60
	106.10	.400	70.93	.851	100.00	7.94
	107.20	.500	70.04	.783	100.00	8.00
3.0	108.40	.600	70.24	.734	100.00	7.39
	109.70	.735	70.31	.661	99.70	6.79
	110.90	.800	71.12	.587	99.38	5.81

The effect of time of cutoff and length of run are best observed using dimensionless results. Runoff increases with time for a given length while it decreases with length for a given time of cutoff. For a given time of cutoff and other constant input parameters there is a limiting advance length beyond which there is no runoff. This is further explained in the later sections.

Dimensionless Runoff Curves

Allowing runoff past the end of the field as discussed earlier is a practice that will insure uniform irrigation. But unless the runoff is controlled the application efficiency is lowered to an extent that whatever is gained through uniformity is lost through efficiency. Reuse of the runoff on the other hand insures that both application efficiency and uniformity are gained.

One of the most important pieces of information that is required to design a reuse system is the quantity of runoff from a given border. Knowing or being able to predict the volume of runoff from a given irrigation in advance without evaluating actual field work is the idea behind mathematical modelling. Thus the zero inertia models which has been verified to be capable of simulating all phases of border irrigation is utilized in this study.

The model as discussed earlier has the advantage of being inexpensive and reliable. But to get any dependent variable like runoff as an output requires a single run on a computer of considerable speed and memory taking several seconds of computer time. This implies that any user or designer needs to have access to a computer everytime an analysis is needed. Obviously the time has not yet been approached for such an abundant use of computer time by each design engineer. The best that could be done is to make several runs to study the general trend or behavior and making it available in a short, usable form. This section deals with such a method of presenting runoff or any other dependent variable in a dimensionless graphic form. This form is considered because it allows designers to perform their analysis in the field or in offices with a simple pocket calculator.

Dimensional analysis is considered because it makes presentation of results easier in addition to its use in studying trends and behavior of several independent variables in a small amount of time and with less expense than it would be in dimensional form; especially for a variable like runoff which depends on several physical variables.

There are several ways of expressing runoff depending on the purpose for which it is meant to be used. Runoff volume as a percentage of the total applied water at the head end of the border is used in this study. This expression has the merit of being easy to comprehend and utilize in design computation for reuse systems.

Runoff percentage (R) is a function of several border irrigation variables. It increases as shown before with slope, flow rate, application time and decreases with increases in infiltration rate, length of run, and bed and vegetative drag. In dimensionless terms these variables can be studied in terms of K^*, t_{co}^*, L^*, and a as indicated in the last chapter.

Equations 4-18 to 4-25 show the interrelationship among the reference variables chosen for this particular system and the dimensionless parameters mentioned above in terms of the physical parameters of the irrigation. If a different reference variable were to be used the nondimensional independent variables derived through the use of the new reference variables would have to change accordingly to come out with the same dependent variable like runoff. As an example instead of the present selection $Y = Y_n$, $Q = q_{in}$, $S = S_o$ we have $Q = q_{in}$, $T = t_{co}$, the dimensionless variables in the solution would be a, k*, S_o*, L*, for any dependent variable considered with the irrigation parameters for the reference variables being n, tco, q_{in}. This assumes the same initial and boundary condition for solution of Equations 4-13 and 4-14 are valid and the Manning formula for resistance and Kostiakov-Lewis function for infiltration are used.

Using the system under consideration for this study, that is, setting V=1, S_f=1, $Q=q_{in}$, $S=S_o$ and $Y=Y_n$ gives QT=XY and Y=SX which yields $X=Y_n/S_o$, $T=Y_n^2/q_{in}S_o$, where $Y_n=(q_{in}n/Cu)^{6/10}/S_0^{3/10}$ will have K*, t_{co}* and L* as governing dimensionless variables along with the parameter a from the infiltration function. No dimensionless parameter comes out of the Manning formula although it should be noted if a different resistance formula was used there would have been a possibiity of having another parameter introduced which would have made it more difficult to present the results graphically.

It should also be noted at this point that if the dimensional analysis were made using the Buckingham Pi theorem, completely different terms could have resulted which may not be of practical use to the problem being addressed. Thus, the graphical representation by use of reference variables, is used for its simplicity as well as for the wider range of choices it gives for analysis.

Using the four governing free parameters, i.e., a, K*, t_{co}*, L*, a generalized runoff curve is presented herein for free outflowing borders. The only requirement for displaying with these terms is that the relationship established between the dimensional and dimensionless variables with the reference variables be maintained.

The procedure followed in developing the curves is as follows. The runoff percentage R was first obtained by making several hundred computer runs for a single a value, different K* values ranging from 0.1 to 10.0 and L* and t_{co}* varying from 0.1 to 100. The percentage values obtained are then plotted against L* for the different values of t_{co}* and one value of K* and a. This plot was then transformed graphically to a plot of t_{co}* versus L* by connecting points of equal runoff percentage but still for the same K* and a values. Graphical transformation has the advantage over transformation through the use of computer runs in that the latter method uses a trial and error approach which requires many computer runs to get the right percentage values.

Only one value of a = 0.7 is used in this study. This value is the one calculated by Fangmeier and Strelkoff (1979) and considered by the SCS as an average value for different soils. Any other a value can be transformed to the same value of a = 0.7 by adjusting the k values to yield the given cumulative infiltration at the end of the irrigation for a given infiltration time for that soil. The end result, i.e. the independent variable has been checked to be the same without the slope a having any effect as long as the k value in the Kostiakov Lewis function is adjusted. This is clearly shown by Table 3 for different intake families. Within the ranges of a values for which the SCS design charts are made, the transformation as can be observed in Table 3 can be used without introducing any error in runoff.

Figures 9 to 24 are the dimensionless runoff curves developed for the ranges of values discussed above for a, K*, L*, t_{co}*. The importance of R in the design of reuse system is discussed in the next chapter. An example to show how to use the dimensionless curves to get the runoff percentage for a given hypothetical set of data is given in Table 8

Length of Run Consideration

Maximum Advance Distance

As a secondary objective for the study, two important design aids were obtained with respect to length of run of border irrigation. The first part deals with the maximum dimensionless advance distance. This distance is defined as the distance from the upstream end of a given border to the location at which the water front ceases to advance. As observed by Shatanawi (1980) and Clemmens and Strelkoff (1980) this distance increases with increase in flow rate, application

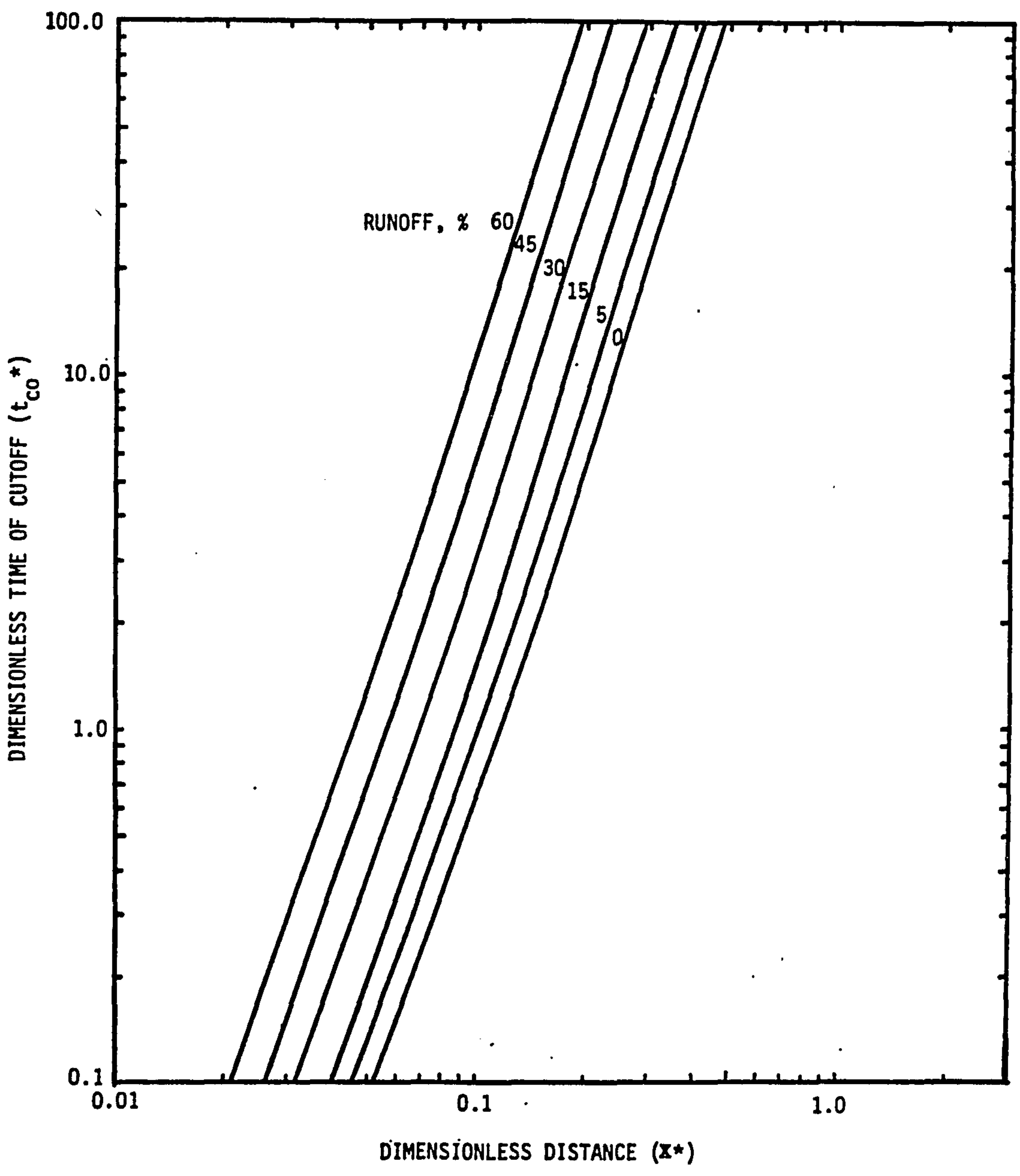

Figure 9. Runoff percentage (a = 0.7, K* = 10.0)

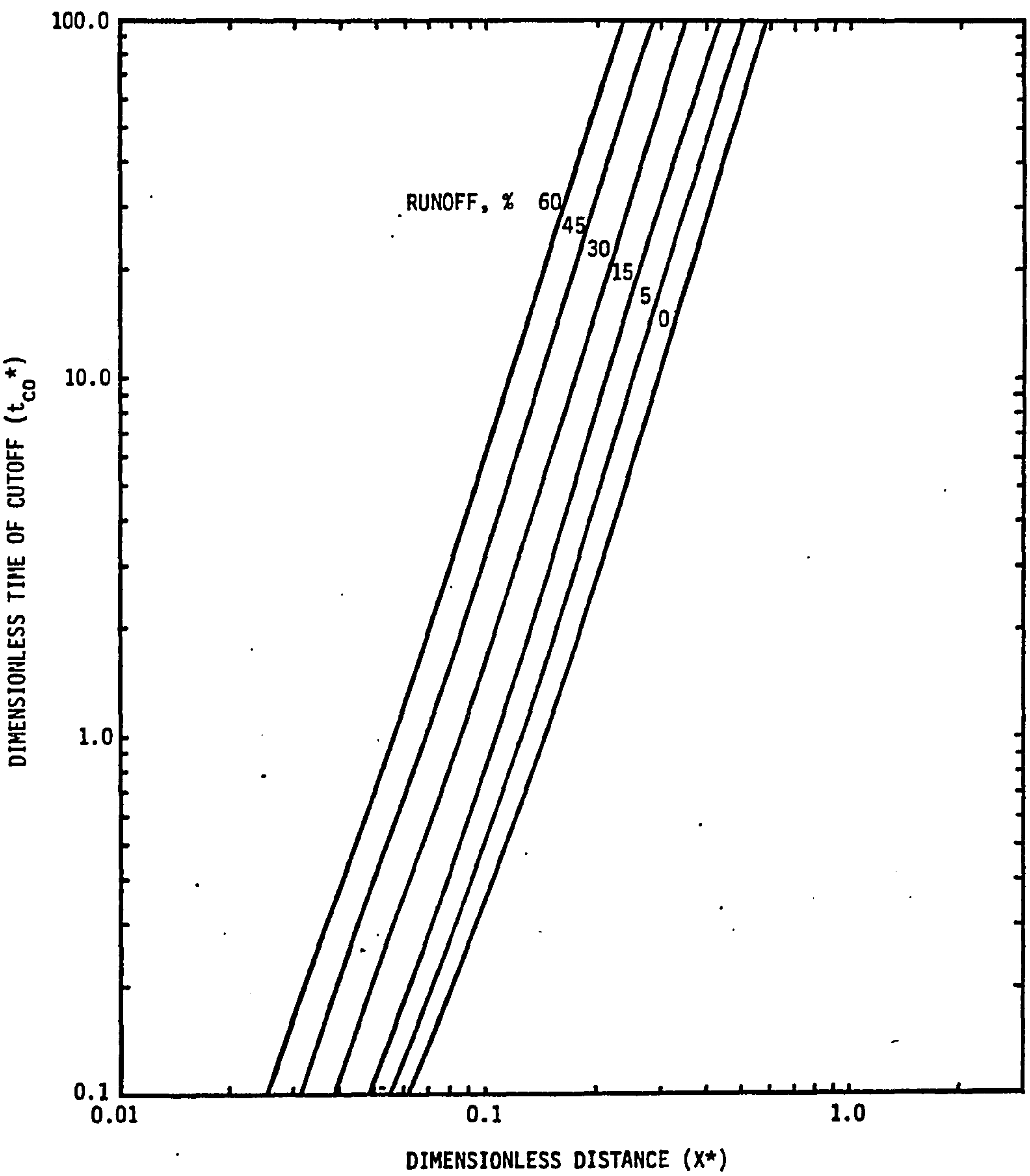

Figure 10. Runoff percentage (a = 0.7, K* = 8.0).

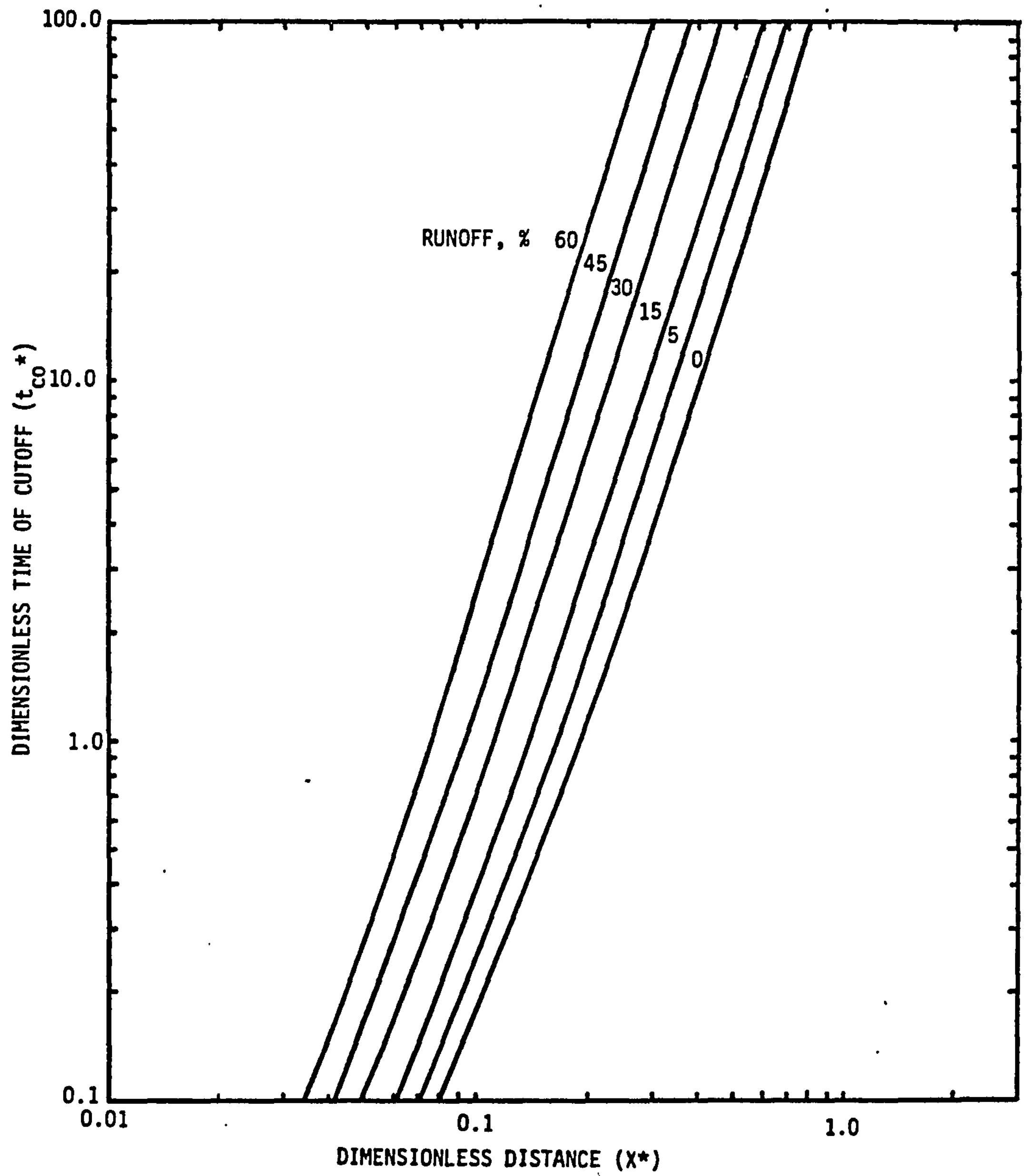

Figure 11. Runoff percentage (a = 0.7, K* = 6.0).

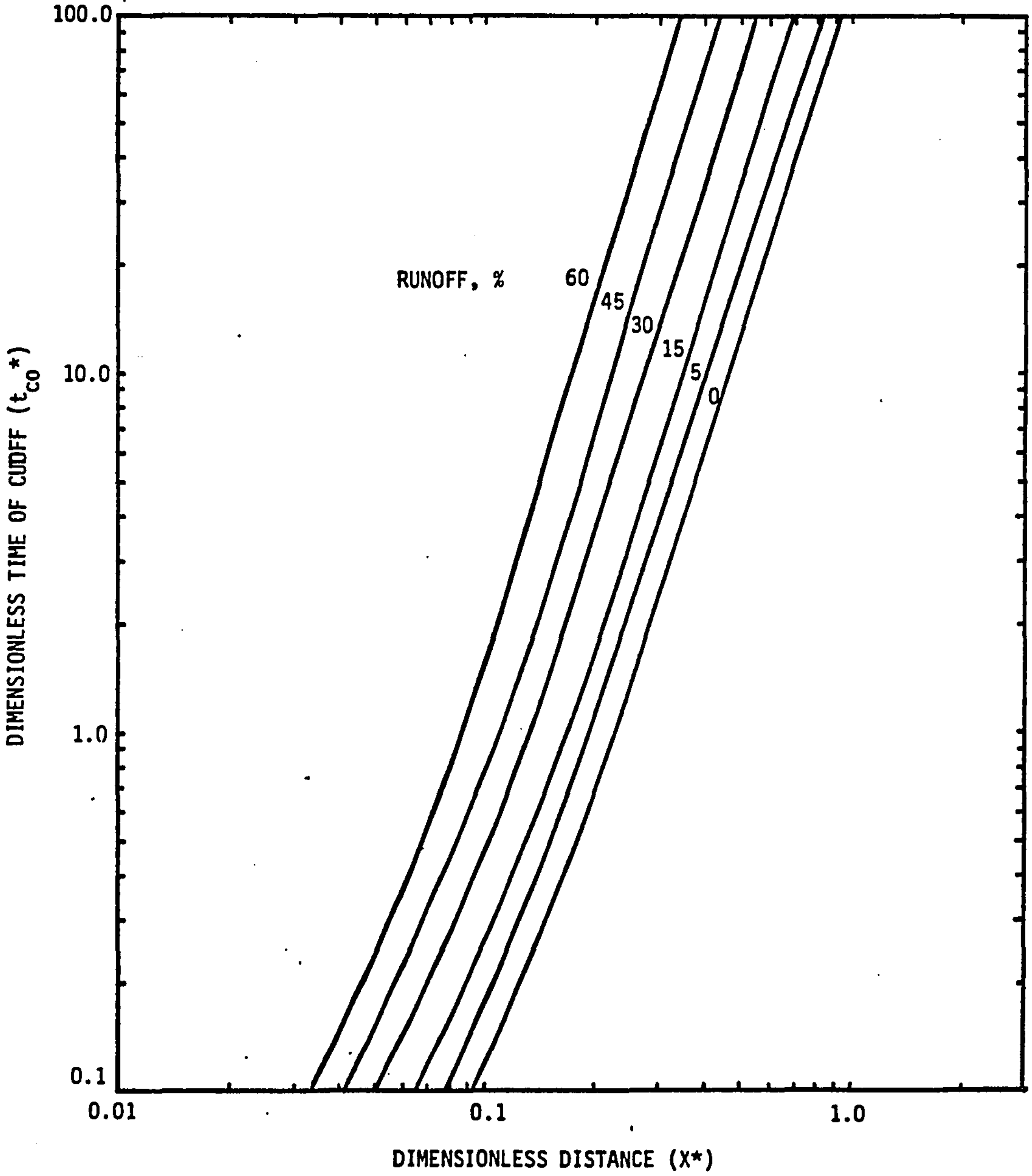

Figure 12. Runoff percentage (a = 0.7, K* = 5.0).

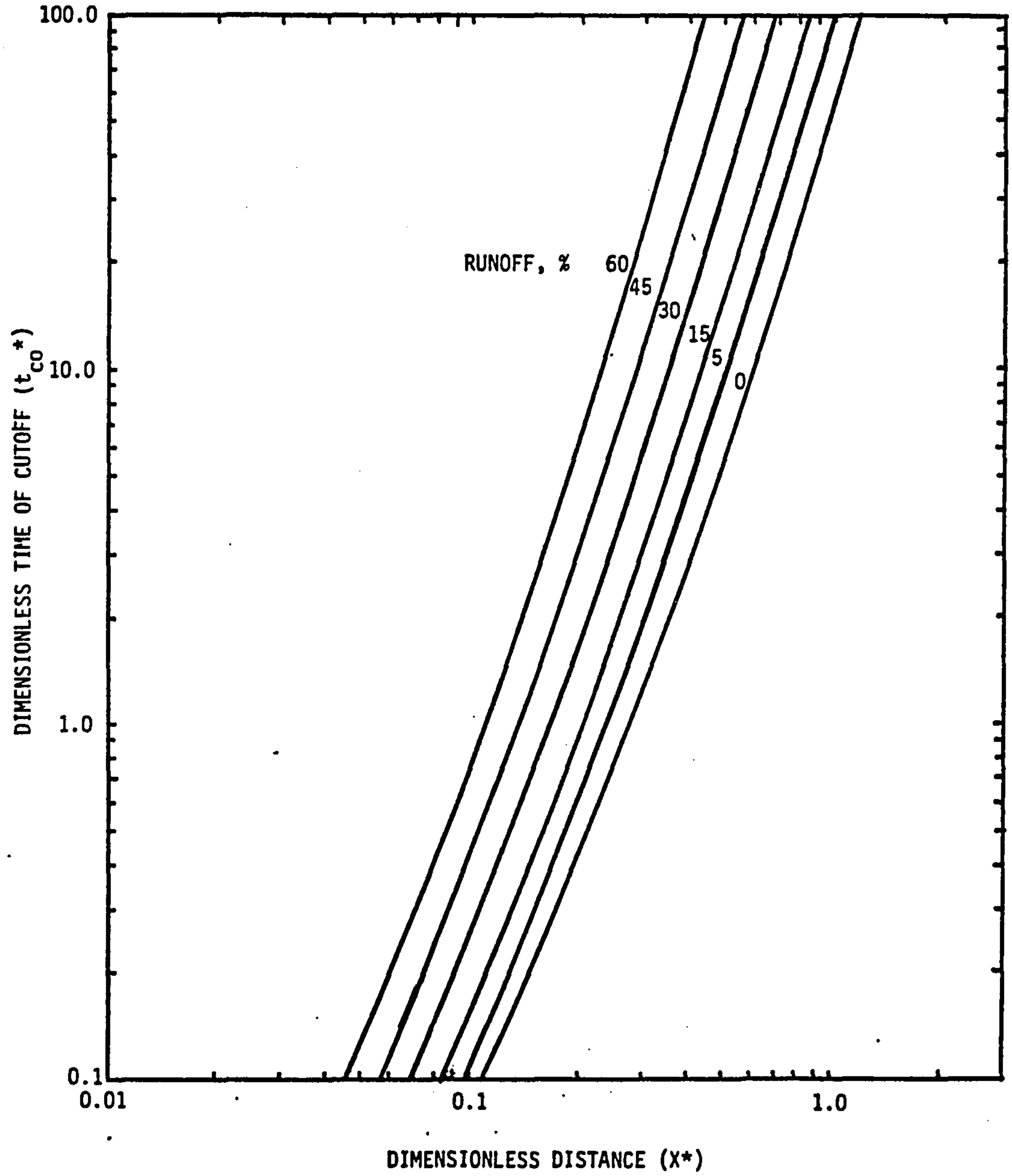

Figure 13. Runoff percentage (a = 0.7, K* = 4.0).

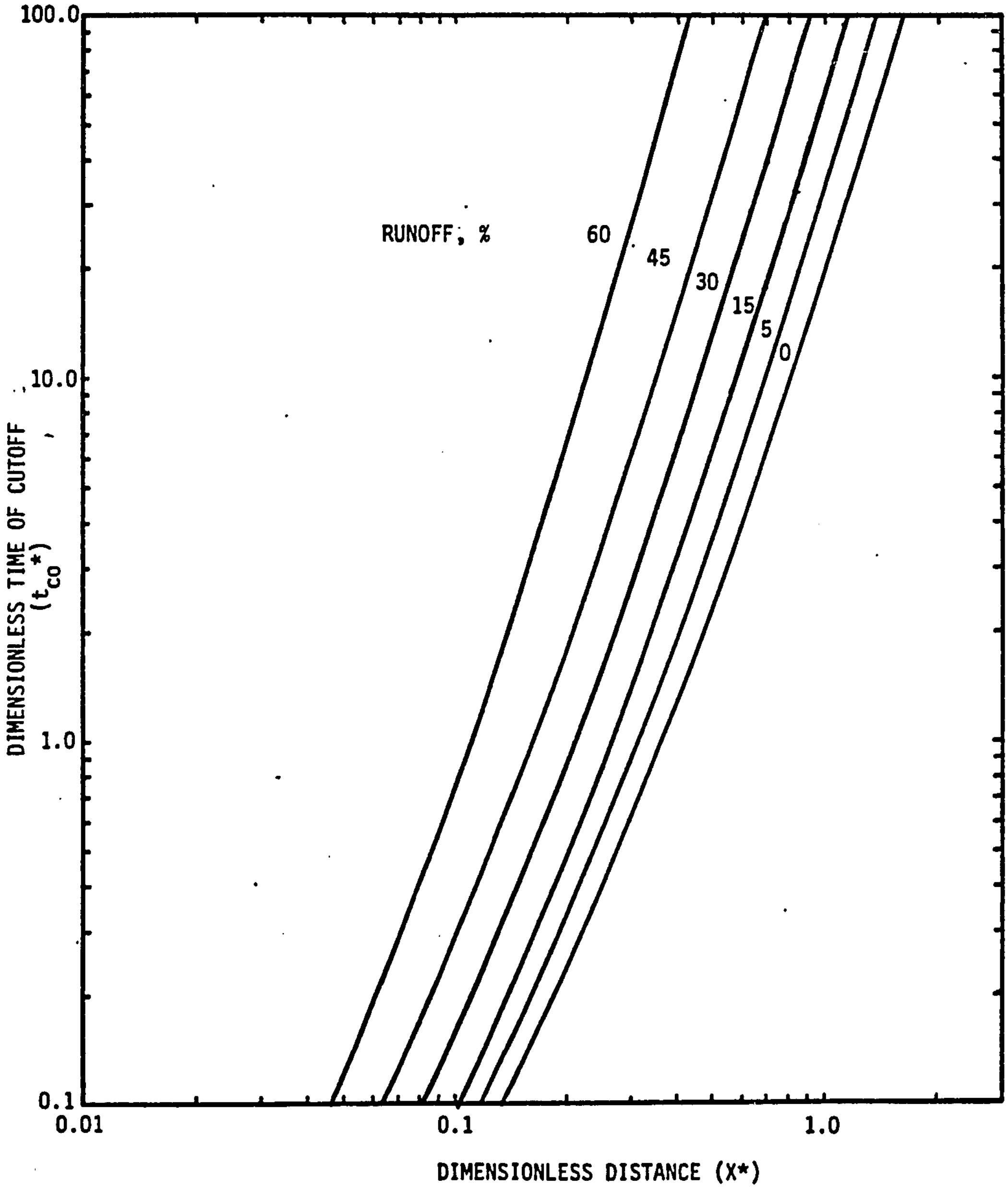

Figure 14. Runoff percentage (a = 0.7, K* = 3.0).

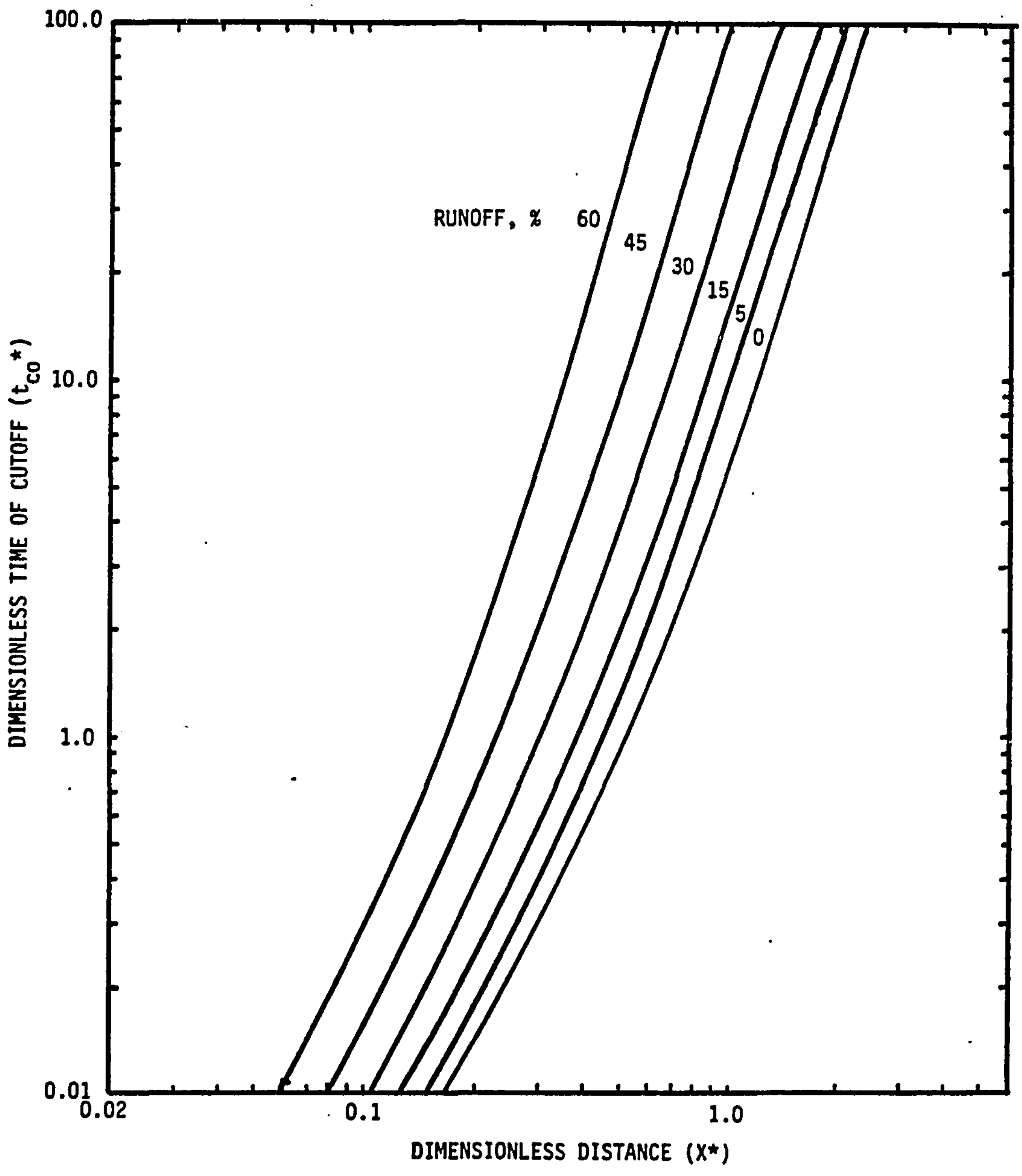

Figure 15. Runoff percentage (a = 0.7, K* = 2.0).

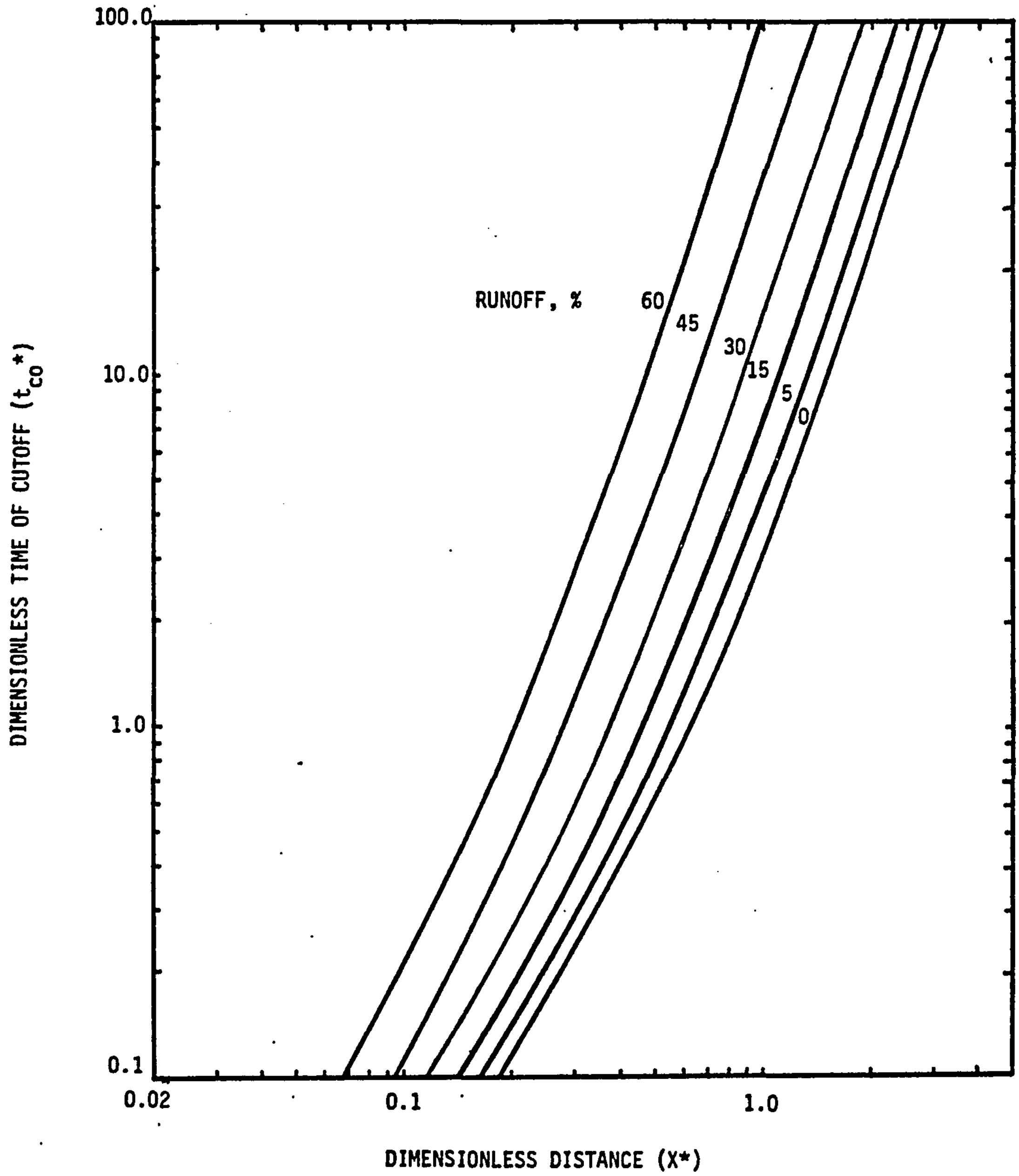

Figure 16. Runoff percentage (a = 0.7, K* = 1.50).

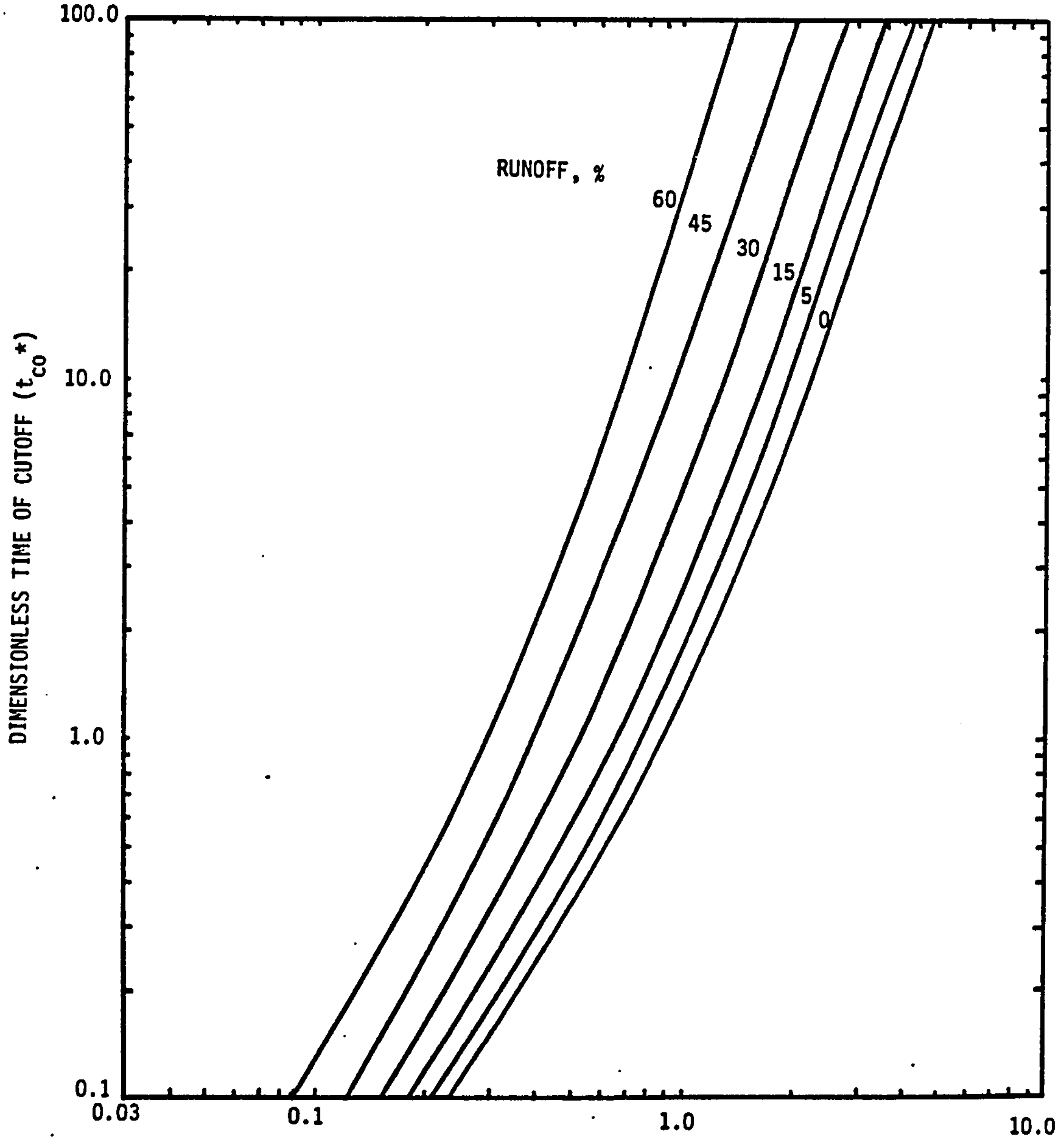

Figure 17. Runoff percentage (a = 0.7, K* = 0.80).

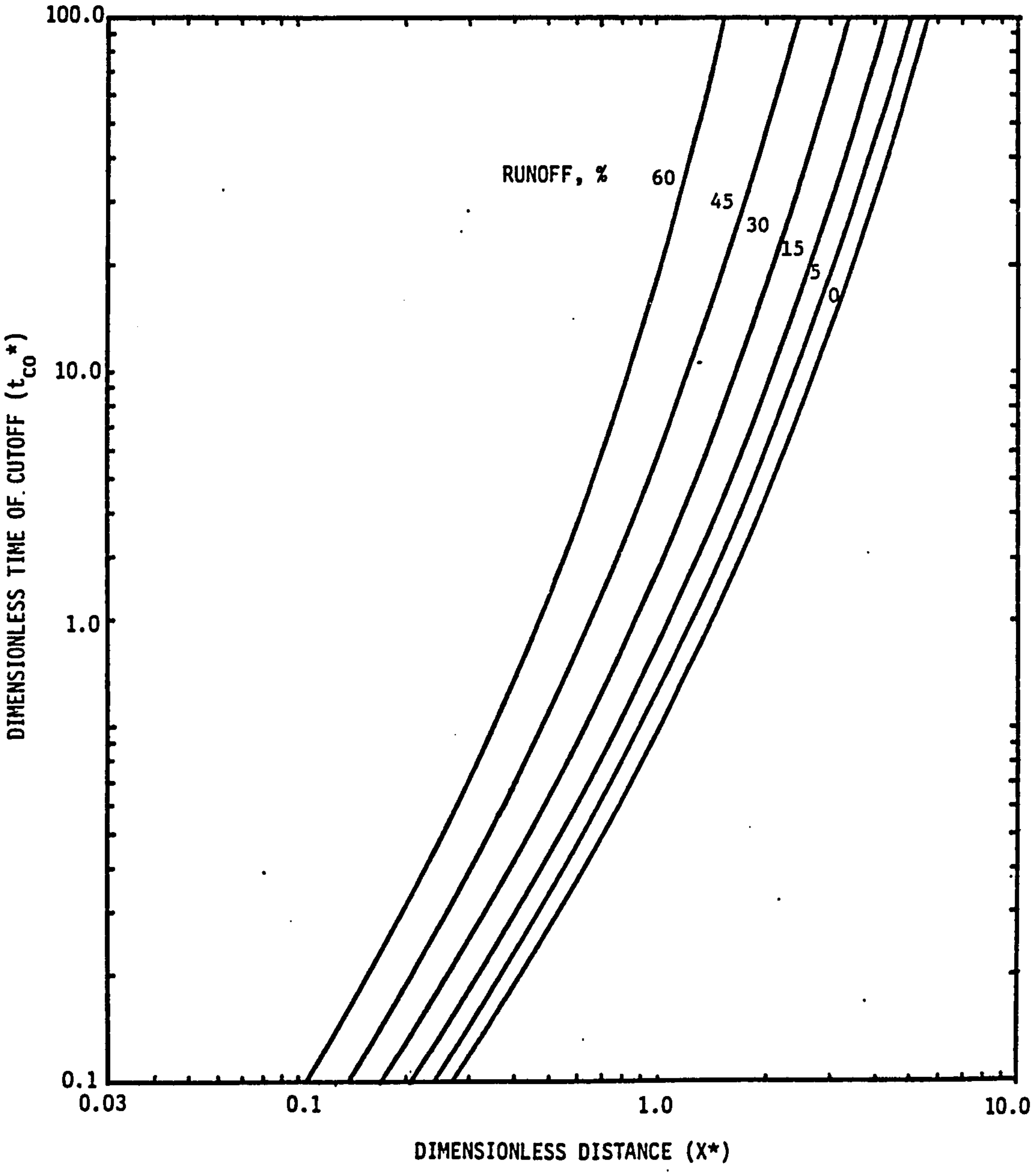

Figure 18.- Runoff percentage (a = 0.7, K* = 0.80).

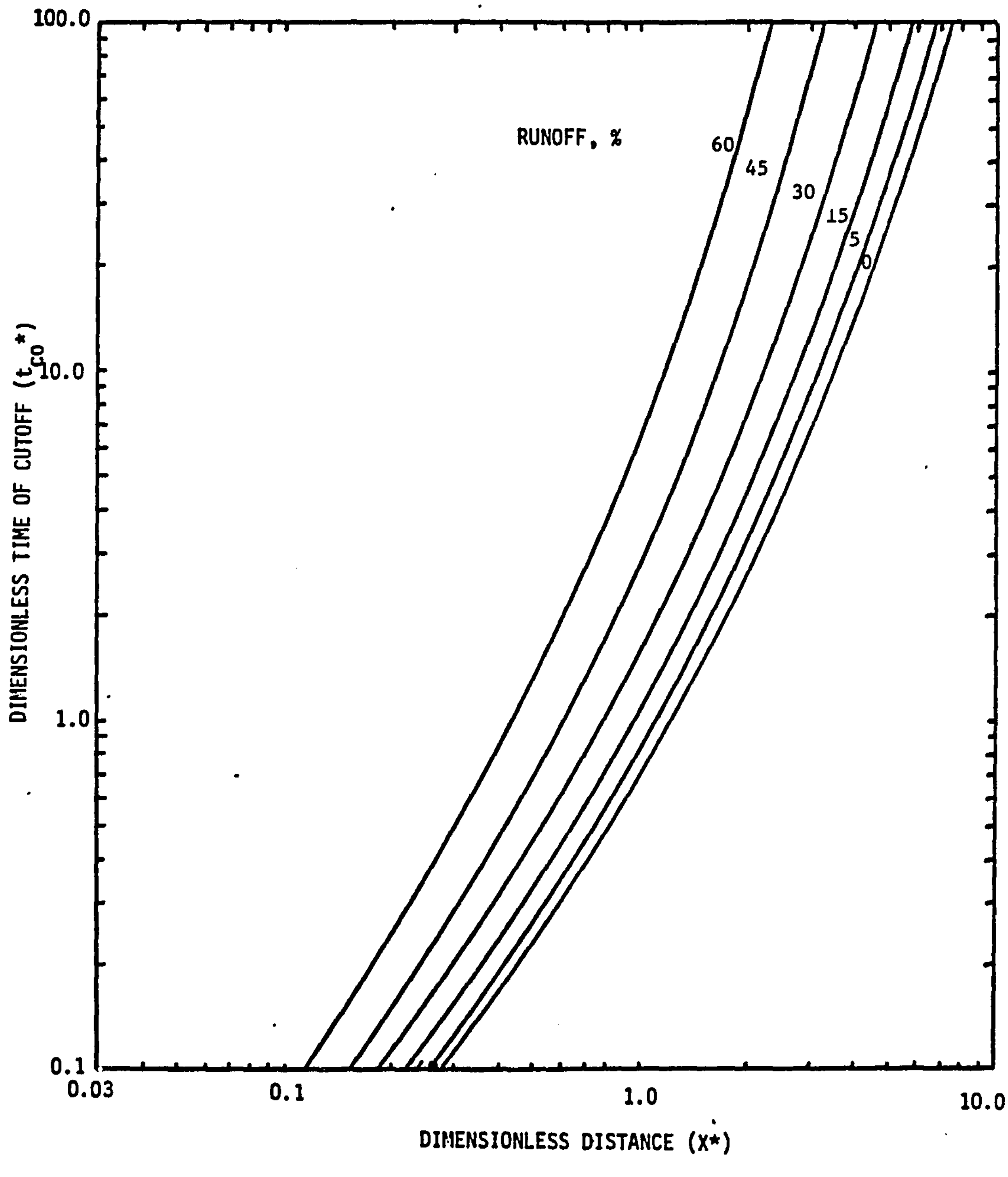

Figure 19. Runoff percentage (a = 0.7, K* = 0.60).

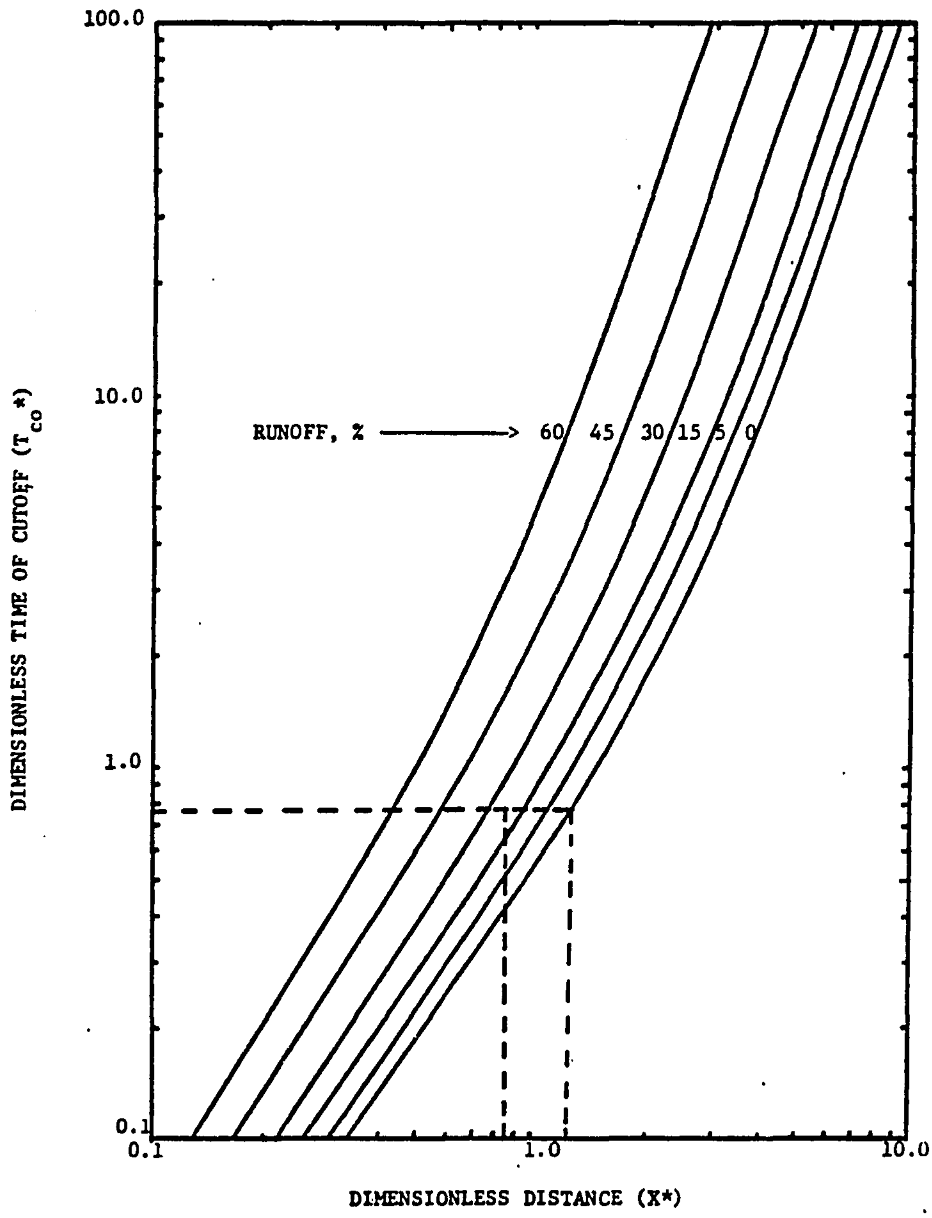

Figure 20. Runoff percentage (a = 0.7, K* = 0.50).

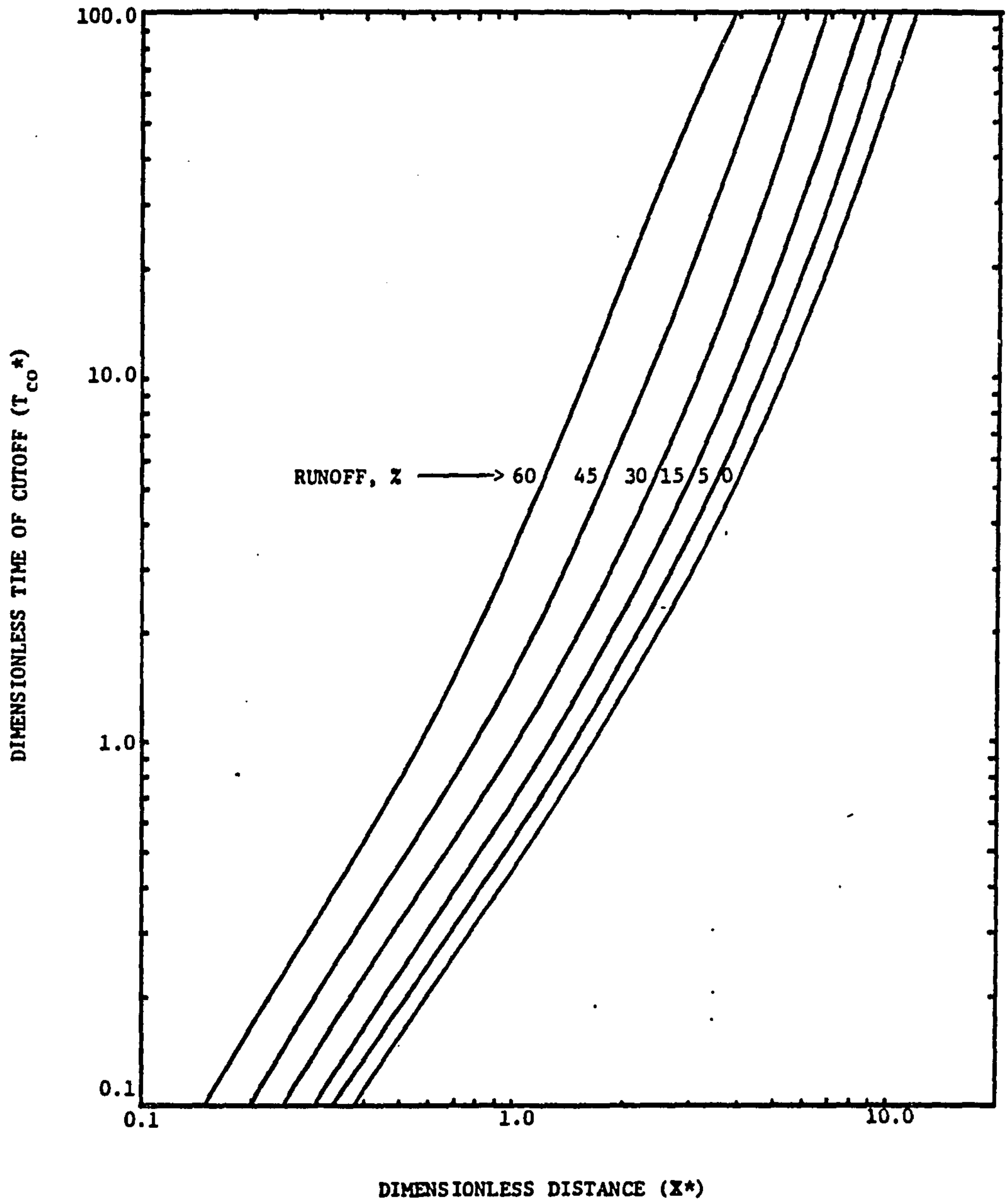

Figure 21. Runoff percentage (a = 0.7, K* = 0.40).

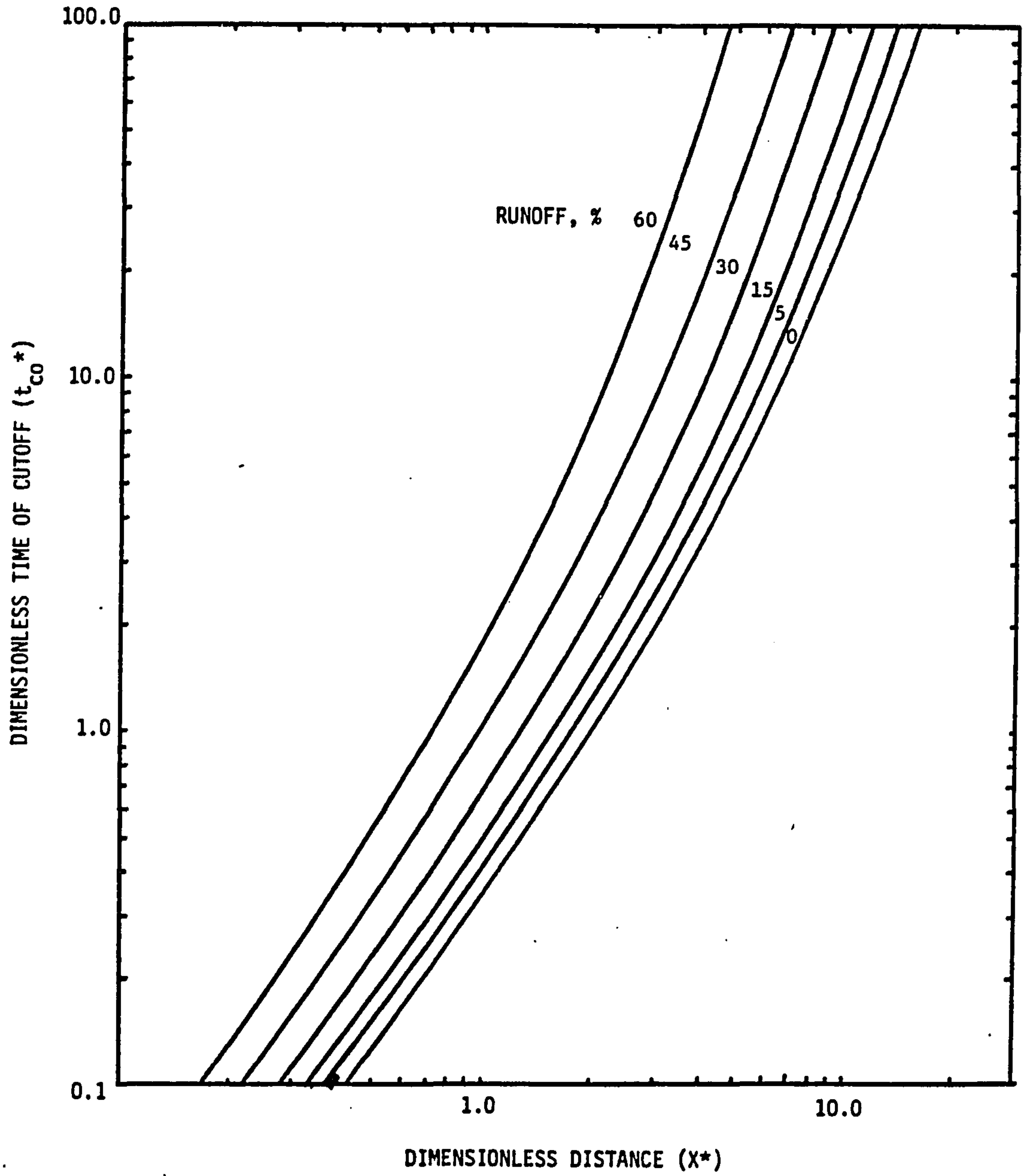

Figure 22. Runoff percentage (a = 0.7, K* = 0.30).

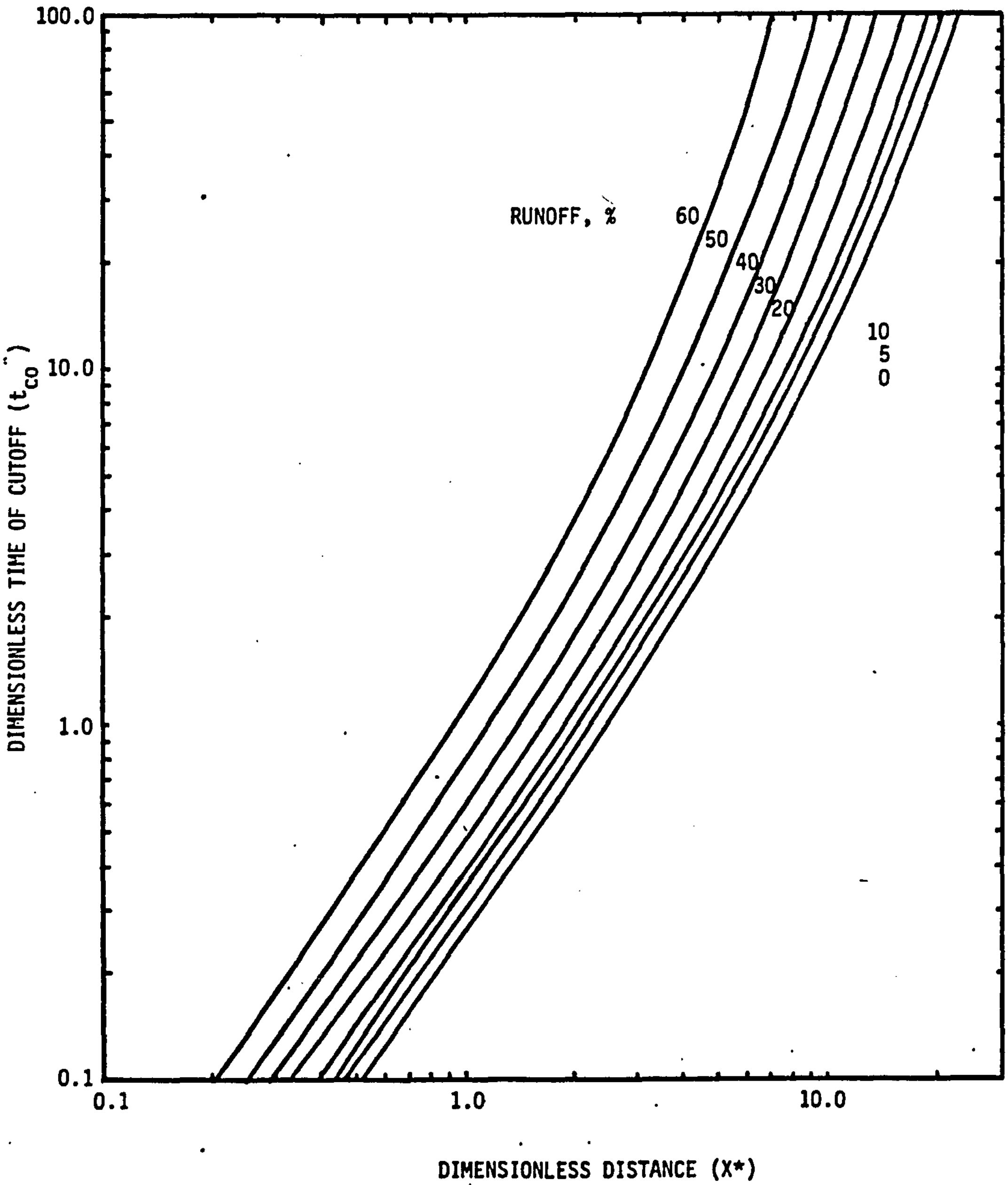

Figure 23. Runoff percentage (a = 0.7, K* = 0.20).

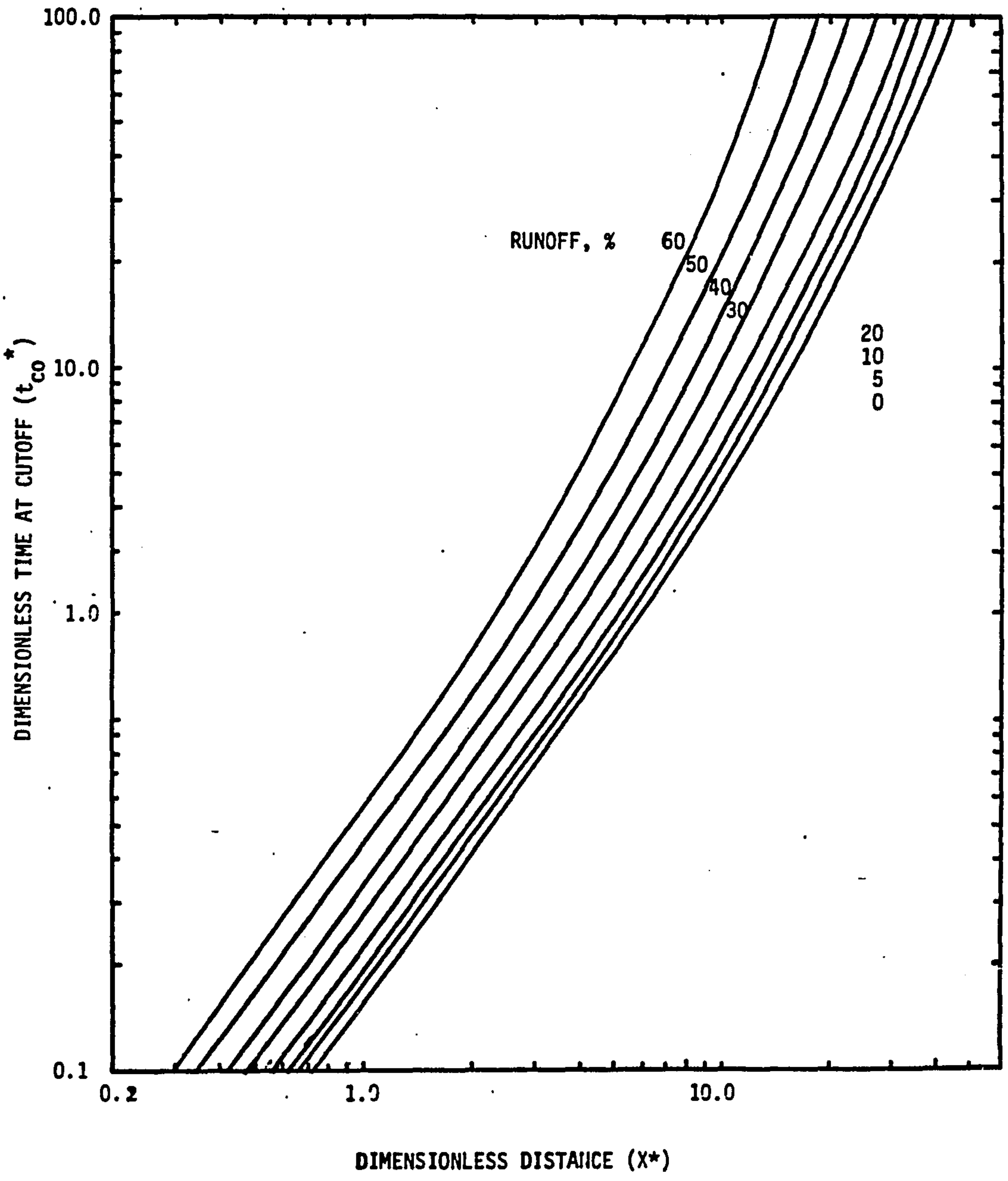

Figure 24. Runoff percentage (a = 0.7, K* = 0.10).

Table 8. Example of the use of dimensionless runoff curves in sloping borders.

Given: q_{in} = 2.995 l/s-m; L = 91.4 m; n = 0.25; S_o = 0.001;

a = 0.7; k = 5.12 cm/hr^a; t_{co} = 47 min.;

Required:

Runoff percentage

Maximum advance distance

Solution:

Reference and dimensionless variables are:

$$Y = \left(\frac{q_{in} n/Cu}{S_0}\right)^{3/5} = \left(\frac{0.002995 \ X \ 0.25}{0.001}\right)^{3/5} = 0.1058\text{m}$$

$$X = Y/So = \frac{0.1058}{0.001} = 105.8 \text{ m}$$

$$T = \frac{XY}{q_{in}} = \frac{105.8 \ X \ 0.1058}{0.002995} = 3739.5 \text{ sec} = 62.32 \text{ min} = 1.04 \text{ hrs.}$$

$$Z = kT^a \rightarrow K^* = \frac{kT^a}{Y} = \frac{(0.0512)(1.04)^{0.7}}{0.1058} = 0.497 = 0.5$$

$$t^*_{co} = \frac{t_{co}}{T} = \frac{47}{62.32} = 0.754$$

$$L^* = \frac{L}{X} = \frac{91.4}{105.8} = 0.864$$

Using K*=0.5, L*=0.864 and t_{co}*=0.754 from Figure 20, R equals 22%. This matches the computer results for the same t_{co}*, L*, K* and a values. The maximum advance distance is obtained for the same t_{co} value using the zero runoff line and indicates L* = 1.24 from which L is computed to be 131 meters (i.e. 1.24 x 105.8).

time and bed slope, and decreases with increase in infiltration rate, and bed and vegetative drag. In dimensionless terms it can be viewed as a function of a, K* and t_{co}*.

The importance of this distance is that for design purposes the length L* should be less than the maximum advance distance X^*_{max}. Otherwise the stream will halt before reaching the downstream end of the field. For X^*_{max} greater than L* there will be runoff in free outflow borders. The advantage of having runoff in free outflow border was observed from previous discussions.

Both Shatanawi (1980) and Clemmens and Strelkoff (1980) presented maximum advance distance as a separate dependent variable by itself using several pages of graphs. The author's attempt to develop one curve which would be used to get runoff and maximum advance distance in this study was successful.

If one accepts that at maximum advance distance equal to the length of the border no runoff will exist, then it is obvious to see that the zero runoff curve represents the maximum advance distance for a given dimensionless time of cutoff. Thus using the data for the given a, K*, t_{co}* and L* the graphs from Figures 9 to 24 are used to get both the runoff percentage R and the maximum advance length. To obtain the maximum advance distance, the user needs to extend the line for a given t_{co}* up to the zero runoff curve. The dimensionless distance that corresponds to the point on the zero runoff curve describes the maximum distance. The method was checked with the other two works and gives exactly the same results.

The method has obviously an advantage in that it presents both the runoff and the maximum distance in one page of curves rather than having two separate pages.

Optimum Length

Another result from this study, which is also helpful in design, is finding the optimum length of the border. The optimum length is used here as the length associated with the maximum application efficiency. For given a, K* and t_{co}* the length was varied to the maximum advance distance and the efficiencies checked until the maximum point is reached. The distances corresponding to these maximum efficiency points are then plotted against dimensionless time of cutoff tco* for different K* and an a value of 0.7 and are presented in Figure 25.

The curves represent the length that would give the maximum application efficiencies without regard to the other dependent variables. The curves help designers check how close the length used for the particular system is to the length that would give maximum application efficiencies.

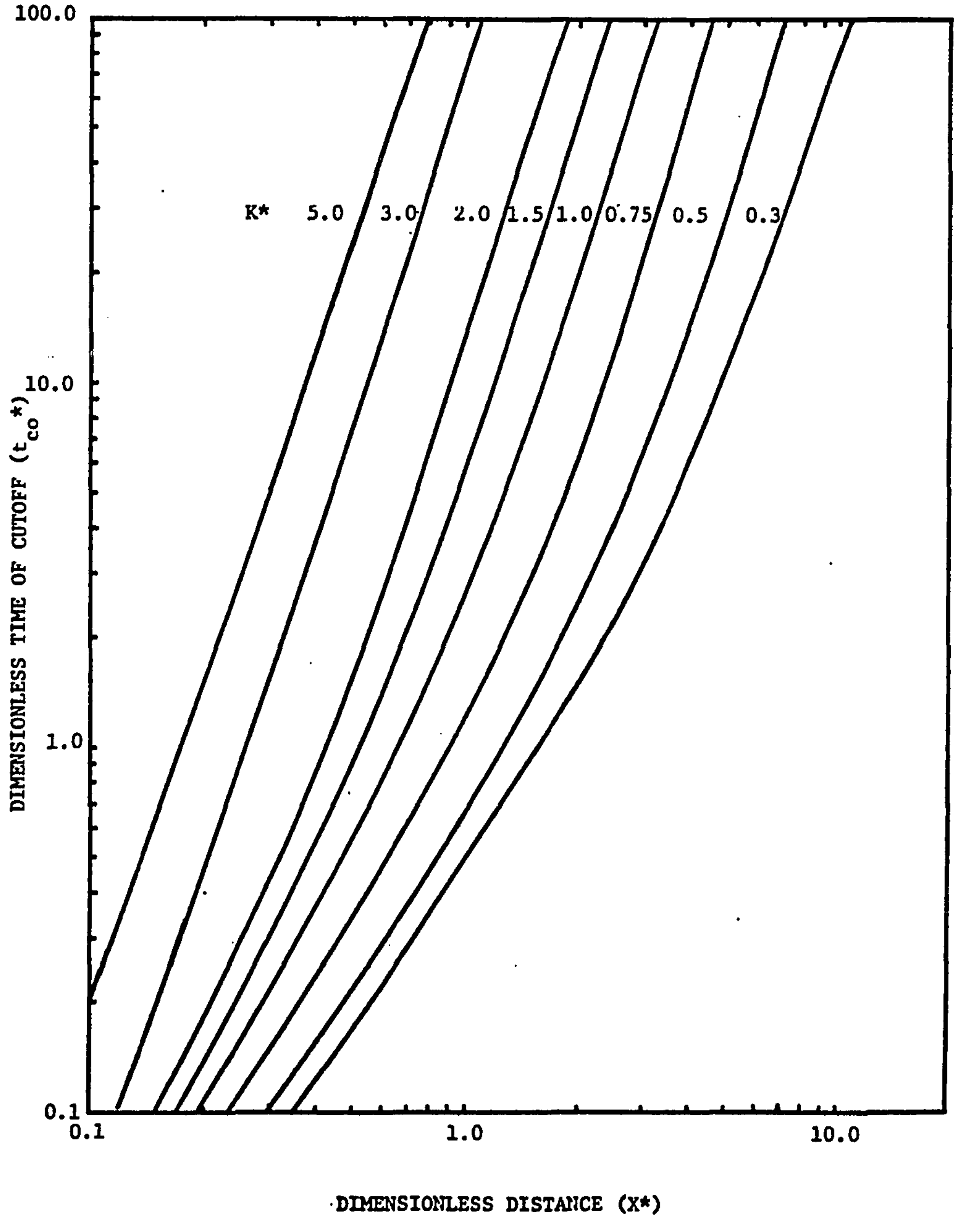

Figure 25. Dimensionless optimum length of run.

CHAPTER VI

REUSE SYSTEM

Pressurized irrigation systems like sprinkler and drip systems are recommendsized as a means to increse efficiency and uniformity of application, and yet surface irrigation methods are still the predominant methods in the world today. It is then obvious to see that significant and rapid improvement in on-farm irrigation application has to be made to avoid critical water shortages in the long run.

There is a great potential for improving surface irrigation systems without affecting the farmer's ways of using water. This can be done through the use of reuse systems, although at some additional cost.

The primary purpose of a reuse system is to collect irrigation runoff water and control it for further use. The system requires a sump or reservoir to collect and store the runoff water, pump facilities for pumping back the runoff, and a pipeline or other method for conveying water back to the irrigation system.

The last chapter dealt with the estimate of runoff because it is the key factor in design of these systems. This chapter deals with the development of design procedures for proper sizing of these components of reuse systems.

Reuse systems may be classified as reservoir or cycling sump systems depending on whether or not they accumulate and store runoff water. They are also classified according to method of handling runoff water as return flow system or sequence system. The return flow system delivers runoff water to the field at a higher elevation than the collecting point while the sequence system delivers water to a field at a lower elevation than the collection point.

Regardless of classification, reuse systems have to function properly to be effective. (Bondurant 1969) summarized functional analysis of reuse systems as follows:

1) Runoff water should be applied to a different field or portion of the field than that on which runoff occurs. Recirculating runoff to the same irrigation set that is generating runoff results only in temporarily storing water on the field. This will not increase the infiltration rate, but will increase the rate of runoff and probably increase erosion in the furrow;

2) When computed over the time interval required to irrigate the area contributing to the recirculating system, runoff water will have to be returned to the system at the same rate it is accumulated if all of the runoff is to be reused. If temporary storage is provided, stored runoff will eventually have to be recirculated at a rate equal to storage accumulation to prevent loss by overflow;

3) Maximum improvement in total water use on the farm will result from using stored runoff water to achieve a reduced stream size for cutback irrigation, i.e., stored runoff water is pumped to increase the stream size during the advance period and pumping is stopped after the field has

started to produce runoff. This reduces deep percolation and runoff so that a minimum amount of water must be recirculated.

This analysis was made in reference to furrow irrigation systems. Except for use of cutback system which is not practiced in border irrigation, the rest of the analysis applies to borders as well. With this as a background the volume balance or conservation of mass principle was used in this study to develop design formulas for proper sizing of the reuse system components.

System Design

Design of reuse systems involves the determination of the various components mentioned earlier. The design depends on the mode of operation considered. Four approaches are used here to develop equations that go with different operational requirements and are given below:

1) Continuous Pumping with Variable Rates: The law of conservation of mass applied to runoff from borders having the same slope, length width, retardance characteristics, and soil infiltration characteristics assuming continuous pumping of runoff water collected from these borders to another border of the same characteristics yields a design equation for sizing sumps as:

$$V = Q_{in}\, t_{co}\, R + (N-1)(\overline{Q}T_R - Q_p T_p) \qquad (6\text{-}1)$$

where V is the volume of sump; Q_{in} is inflow rate to the border assumed to be constant; t_{co} is the cutoff time; R the runoff percentage which is equal to the ratio of the amount of runoff to the amount of applied water to the border; Q_R is the average runoff rate, i.e. $Q_R = Q_{in}t_{co}R/T_R$, T_R is the time of runoff from start to the end of runoff, Q_p is the rate of pumping from the sump; T_p is the time of pumping from start of irrigation to time runoff ceases; and N is the number of borders from which runoff is generated. Equation 6-1 shows two important results. First, if N equals one the volume of sump is equal to $Q_{in}t_{co}R$ which is the total runoff from one border which is not pumped back but stored for later use. Second, the minimum sump size occurs if QT_R and Q_pT_p are equal. Only a buffer storage is needed if the total runoff for that particular irrigation is equal to the volume reused. This sytem provides the simplest mode of operation.

The rate Q_p can be estimated by:

$$Q_p = \left(\frac{1}{N-1} - \ell\right) Q_{in}\frac{t_{co}}{T_R} \tag{6-2}$$

where ℓ represents the runoff fraction lost in the reuse system. Equation 6-2 is developed by assuming runoff from (N-1) borders if accumulated in a reservoir would be enough to irrigate one border with the required depth in a time of pumping equal to the time of runoff. This can be adjusted by changing time from T_R to another time t depending on the need of the individual operator.

2. Steady Continous Pumping: Again using the volume balance principle, the size of reservoir for a steady continuous pumping reuse system can be determined by:

$$V = Q_{in} t_{co} R - \frac{Q_{in}{}^2 R^2 t_{co}{}^2}{Q_p T_R} \qquad (6\text{-}3)$$

where the variables are as defined earlier. This assumes runoff to be pumped at constant rate out of the reservoir to another border continuously for a time $T_p = Q_{in} t_{co} R/Qp$.

If loss of the runoff water ℓ percent is assumed due to the reuse system inefficiency, the volume of the sump will be given by:

$$V = (1-\ell)\ Q_{in} t_{co} R\ (1 - Q_{in} t_{co} R/Q_p T_R) \qquad (6\text{-}4)$$

For N borders irrigated at the same time, the right side of Equations 6-3 or 6-4 should be multiplied by N to get the total volume required.

3. Cycling Sump System: Larsen (1959) worked on pumped drainage systems relating inflow, pump capacity, sump volume and cycle time by:

$$CV = 60\ Q_R/Qp\ (Qp - Q_R) \qquad (6\text{-}5)$$

where V is sump storage, C is the number of cycles per hour, Q_R inflow rate to the sump, and Qp is the pumping rate from the sump. When

Equation 6-5 is differentiated with respect to Q_R it gives a maximum storage when Q_R is 1/2 Qp. Substituting this in Equation 6-5 results:

$$\forall = 15\ Qp/C \tag{6-6}$$

This shows that the number of cycles per hour must be maximum to minimize the sump volume needed.

In a later study of number of cycles per hour for single-phase motors, Larsen and Manbeck (1961) obtained from manufacturers a maximum of 15 cycles per hour that won't result in a decrease of pump efficiency. When this is substituted into Equation 6-6 the result is:

$$\forall = Qp = Q_R max \tag{6-7}$$

The best result would be obtained if Qp = Q_R maximum from functional analysis of the system. Using this condition the minimum sump size for a recovery system can be obtained assuming 15 cycles per hour. For number of cycles other than 15, Equation 6-6 should be used. If these equations are to be used for border irrigation, prior knowledge of the runoff hydrograph is needed.

Knowing the depth of storage (H) in the sump between on and off float level, the inside diameter of a circular sump (D) in centimeters can be obtained by:

$$D = 27.6\ \sqrt{Qp/H} \tag{6-8}$$

where Q_p is in liters/sec and H in meters. Davis (1964) gave a table to get the minimum sump diameter which can be generalized by Equation 6-7 or 6-8.

For square sump shape of square surface area the volume of the sump can be calculated using the relationship

$$V = H/6\ (A_{k-1} + 4A_m + A_k) \tag{6-9}$$

where V is the volume of the sump, h is height of storage in the sump between on and off float level; A_{k-1} is bottom surface area; A_k is top surface area, and A_m is the area of the plane surface at H/2 above the bottom surface.

A more generalized approach to get the volume that can be used for any shape assuming that an elevation area curve can be found from the field is given by the following expression.

$$FA(H_x) = [\sqrt{A_{k-1}} + \frac{\sqrt{A(k)} - \sqrt{A_{k-1}}}{H_k - H_{k-1}}\ (H_x - H_{k-1})]^2 \tag{6-10}$$

where these terms are as defined below. Area can be defined in form $A = \alpha D_i^2$.

$$FA = D^2(H) \tag{6-11}$$

then $D_k = \sqrt{A_k}$ and $\alpha = 1$ (6-12)

This assumes area is given by the form A(1), A(2), ...A(N) with H(1), H(2), ...H(N) corresponding. It can be also deduced that

$$D(H_x) = \sqrt{A_{k-1}} + \frac{\sqrt{A_{(k)}} - \sqrt{A_{(k-1)}}}{H_{(k)} H_{k-1}}\ (H_x - H_{k-1}) \tag{6-13}$$

which by Equation 6-11 results in Equation 6-10 since $FA(H_x)$ is a function of $D^2(H_x)$. The volume of the sump is then given by:

$$\forall = \frac{H_x - H_{k-1}}{3} \left(FA\,(H_{k-1}) + \sqrt{FA(H_{k-1}) \times FA(H_x)}\right) = FA(H_x) \qquad (6\text{-}14)$$

Equation 6-14 is a more general form for volume of any shape and is best used for computerized works.

4. Complete Control of Storage: If the entire runoff from the irrigated borders is to be contained with no pumping from the sump, the volume of storage needed is given by:

$$\forall = NQ_{in}t_{co}R \qquad (6\text{-}15)$$

where all the terms are as defined earlier. This sytem has the merit of giving flexibility of use of the runoff water. It can even be used as a separate supply source. A pump of any size can be used. The main problem with the system is that it takes productive land and sometimes evaporation can be significant.

Time of Runoff

In Equations 6-1 to 6-7 the time of runoff is one of the important independent variables needed. Time of runoff T_R is defined as the time from the beginning to the end of runoff. It is the base of the runoff hydrograph that is produced for any one irrigation. The model is capable of giving this time directly. But presentation of time of runoff alone would take another set of graphs varying as function of other irrigation variables.

To avoid the need for other sets of graphs, an attempt was made to find other means of getting the time of runoff. The author made several runs using the model and checked if there was any relationship between the intake opportunity time at the end of the field and the time base of the runoff hydrograph produced by the model. The results were identical with very little difference in time. This leads to a very important result in that the time of runoff can be approximated by the intake opportunity time at the end of the field without any serious effect on the design of the system.

The intake opportunity time is defined by the Kostiakov-Lewis function written as

$$Z = kt^a \qquad (6\text{-}16)$$

where Z is the infiltrated depth, k and a are constants discussed earlier. Determination of time using this equation requires the knowledge of the ultimate profile of the infiltrated depth for the physical variables of a given border irrigation

Shatanawi (1980) devoted the major part of his doctoral work to determining a mathematical expression for the ultimate profile of infiltrated depth. He used a least squares approximation and checked the results with the ultimate profile produced with the zero-inertia model and found complete agreement between the two. His result is used in this study to get the time of runoff as related to the infiltrated depth at the end of the border by the infiltration function.

The ultimate infiltrated depth at the end of the field is given by the expression

$$Z_u(L) = Z' \bar{Z}^* Y \tag{6-17}$$

where $\bar{Z}^*$ is given by t^*_{co}/X^*_{max} and is the dimensionless average depth infiltrated over the maximum distance which resulted from the volume balance relation $q^*_{in}t_{co}^* = \bar{Z}^*X^*_{max}$ and

$$Z' = a_o \ (1-L^*/X^*_{max})^{a/2} + a_1 \ (1-L^*/X^*_{max}) \tag{6-18}$$

where Z' is the dimensionless depth infiltrated at the lower end of the field, and a_0 and a_1 are constants related to the Kostiakov-Lewis constant a by

$$a_1 = \frac{4 + 2a - 4a_o}{a + 2} \tag{6-19}$$

The value of a_0 as a function of the dimensionless border irrigation variables to be used in conjunction with the runoff curves developed is given in Figure 26.

In Equation 6-18, L^* = the dimensionless field length and X^*_{max} is the dimensionless maximum advance distance defined by the zero runoff curves from Figures 9 to 24. Knowing Z at the end of the field the time of runoff is given by

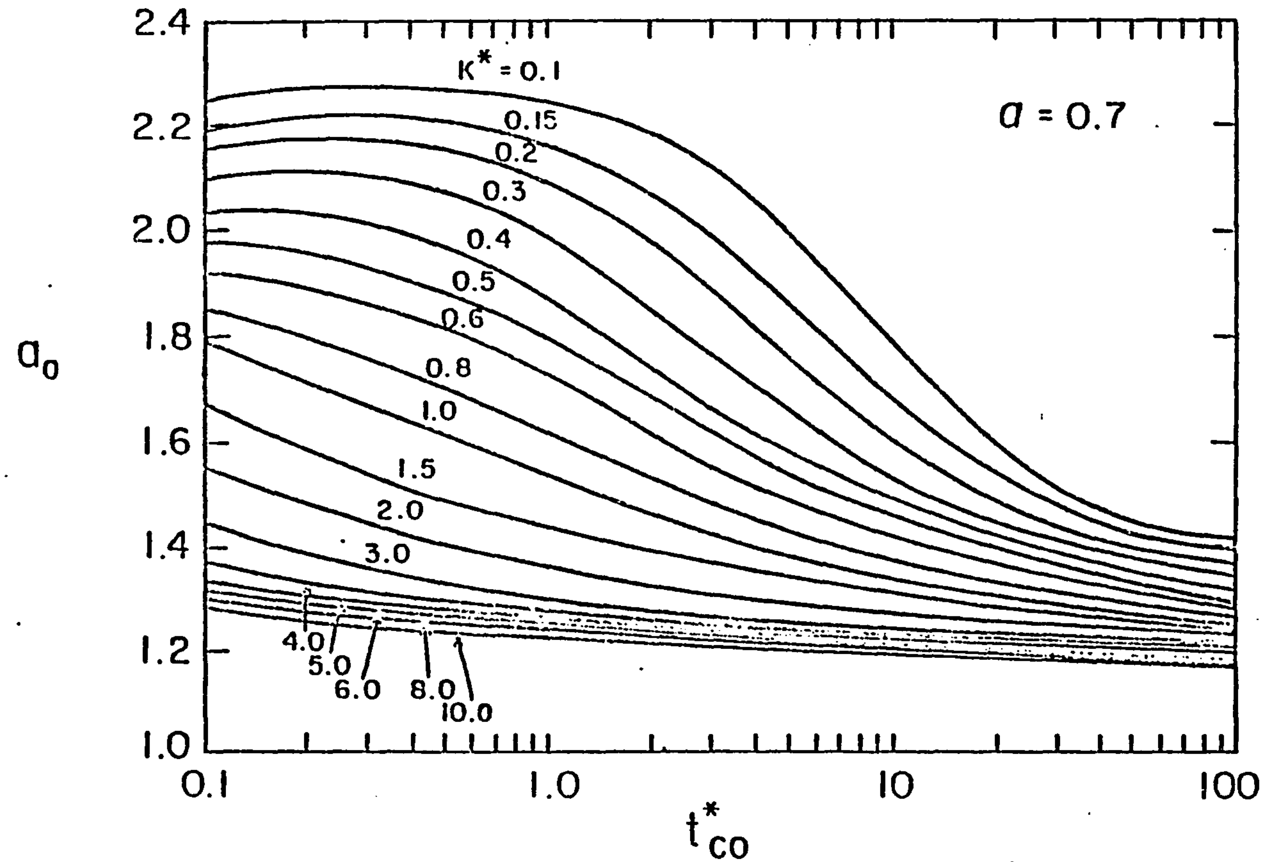

Figure 26. Shape factor variation (a = 0.7).

$$T_R = \left(\frac{Z_u(L)}{k}\right)^{1/a} \tag{6-20}$$

This is the value used in the reuse system design.

Design Data

As stated earlier, in reuse systems we are concerned with the sump, pump capacity, and conveyance mechanism. Several border irrigation parameters need to be known for most of the system design equations to be meaningful. Other variables like the choice of the individual farmer, or institutional factors affect the design. With due consideration to these important factors only the border irrigation variables are discussed here.

The physical variables needed for the design of reuse systems are mainly the slope S_0; length of run, L; type of crop cover to get the retardance coefficient n; the system capacity which is described by the inflow rate, time of cutoff or application time for the given required depth of water to the border and the required level of application efficiency; and soil infiltration characteristics. These variables determine the percentage of runoff R as shown in Chapter 5 by using the dimensionless runoff curves, (Figures 9 to 24). Once the runoff, flow rate to the border, and time of application are defined, the equations developed in this section can be used to determine the sump volume for different pumping rates out of the sump.

Depending on the pumping rate from the sump, the pump size and pipeline system are determined accordingly. The size of the pump is a function of not only the pumping rate, but also the total head it has to operate against.

The total head for a reuse system can be described by:

$$H = H_s + S_o L_1 + 10.28 \times 10^{10}\, n^2 L \frac{Q^2}{D^{16/3}}$$

$$+ \quad 0.80827 \times 10^{6} \sum_{i=1}^{J} C_i \frac{Qp^2}{D^4} \qquad (6\text{-}21)$$

where H is the total head (m); S_o is the border slope, L_1 is length of run of the border (m); n is the Manning regardance coefficient ($m^{1/6}$); D is the diameter pipe used for conveying water back to the supply of the borders (mm); H_s is the suction lift (m); Qp is the pumping rate from the sump (ℓ/sec); L_2 is the length of the pipeline (m); C_i's are the coefficients for each type of minor losses considered where i goes from zero with no minor loss to J total number of minor losses like bend, valve and so forth, considered.

Equation 6-21 was developed using Darcy-Wiesbach's equation for head loss where the friction factor f is expressed in terms of Mannings roughness factor by:

$$f = 124.45\, n^2/D^{1/3} \qquad (6\text{-}22)$$

using metric units.

CHAPTER VII

IRRIGATION EFFICIENCIES

To get information that helps engineers design a system with performance better than others and enable comparison of various systems for better economic decision is the objective of any study dealing with irrigation systems. From all the available criteria, irrigation efficiencies are the best way of evaluating the performance as well as operating conditions that will help for most decision making processes.

If one is to use irrigation efficiencies to indicate the performance of an irrigation system, there must be an understanding of the definition of the terms used to express the efficiencies. Several definitions of irrigation efficiencies are currently available with different people defining them according to their need. As an example, Israelsen and Hansen (1963) defined water application efficiency as the ratio of water stored in the root zone to the volume of water needed in the soil root zone Strelkoff (1972) defined application efficiency differently as the ratio of the useful water volume to the total water delivered. However, useful volume means different things to different users. According to Israelsen and Hansen it means the volume of water stored in the root zone, while to some others it includes the leaching fraction.

To standardize the meaning of the efficiency terms used, the On Farm Irrigation Committee of the Irrigation and Drainage Division of the American Society of Civil Engineers (1978) came out with definitions that received wide acceptance. The efficiency terms as defined by the group are used for this study and are given below.

Application Efficiency (AE) is the ratio of the average depth of the irrigation water infiltrated and stored in the root zone to the average depth of irrigation water applied, expressed as a percent.

$$AE = \frac{\bar{Z}}{\bar{d}} \times 100 \qquad (7\text{-}1)$$

where $\bar{Z}$ is the average depth of water infiltrated and stored in the root zone and $\bar{d}$ is the average depth of water applied. This definition assumes that all the water stored in the root zone is used by the crop which is not always true.

Actual application efficiency of low-quarter (AELQ) is the ratio of the average low quarter (LQ) depth of irrigation water infiltrated and stored in the root zone to the average depth of irrigation water applied. The average low quarter depth infiltrated is the average of the lowest one fourth of the measured values where each value represents an actual unit of area and cannot exceed the soil moisture deficiency. This is a good measure of not only efficiency but also uniformity.

$$AELQ = \frac{\bar{Z}_{lq}}{\bar{d}} \times 100 \qquad (7\text{-}2)$$

where $\bar{Z}_{lq}$ equals the average low quarter depth infiltrated and stored and $\bar{d}$ equals the average depth of water applied.

Potential Application Efficiency of Low Quarter (PELQ) is the efficiency that is attainable, expressed as percent when the average low quarter of irrigation water infiltrated and stored is just equal to the managment allowed deficiency (MAD)

$$\mathrm{PELQ} = \frac{\bar{Z}_{lq}}{\bar{d}} \times 100 = \frac{\mathrm{MAD}}{\bar{d}} \times 100 \qquad (7\text{-}3)$$

This efficiency term is the basis for this study for the reason that the designer's aim is to get the maximum attainable efficiency assuming management is operating at optimal condition.

Distribution Uniformity Low Quarter (DULQ) is the ratio of the average low quarter depth of irrigation water infiltrated to the average depth of irrigation water infiltrated expressed as a percent.

$$\mathrm{DULQ} = \frac{[(\int_{1-L'}^{1-.75L'} a_0 \tau^{a/2} + a_1 \tau \, d\tau)\, q_{in}\, t^*_{co} T] \,/\, .25L}{[(\int_{1}^{1-L'} a_0 \tau^{a/2} + a_1 \tau \, d\tau) q_{in} t^*_{co}\, T] \,/\, L} \qquad (7\text{-}4)$$

where L' = L*/X*max and t = 1-L'. The DULQ value is an indication of how uniform the water will be applied on the farm. The use of PELQ and DULQ for the design of border irrigation systems is enough for a reasonable prediction of the performance of the system under optimum management condition, and thereafter they are used here.

Accepting the above definition for PELQ a relationship between runoff, DULQ and PELQ is obtained as follows:

Runoff (R) for this study is defined as

$$R = \frac{\bar{d} - \bar{Z}}{\bar{d}} = 1 - \frac{\bar{Z}}{\bar{d}} \quad ; \quad \bar{Z} = (1-R)\,\bar{d} \tag{7-6}$$

and

$$DULQ = \frac{\bar{Z}_{1q}}{\bar{Z}} \;;\; \bar{Z}_{1q} = DULQ(\bar{Z}), \tag{7-7}$$

therefore

$$PELQ = \frac{\bar{Z}_{1q}}{\bar{d}} = \frac{DULQ(\bar{Z})}{\bar{d}} = \frac{DULQ\ (1-R)\bar{d}}{\bar{d}} = DULQ\ (1-R) \tag{7-8}$$

NOTE: $\bar{d} = \dfrac{q_{in} t^{*}_{co} T}{L}$

Thus the basic relationship between the three is

$$PELQ = DULQ\ (1-R) \tag{7-10}$$

Similarly the deep percolation (D_p) can be obtained as follows:

$$D_p = \frac{\bar{Z} - \bar{Z}_{1q}}{\bar{d}} = \frac{\bar{Z}}{\bar{d}} - \frac{\bar{Z}_{1q}}{\bar{d}} = \frac{\bar{Z}}{\bar{d}} - PELQ$$

But from Equation 7-5 $\dfrac{\bar{Z}}{\bar{d}} = 1-R$

Then

$$D_p = (1\text{-}R) - PELQ \times 100 = 1\text{-}(R + PELQ) \times 100 \qquad (7\text{-}12)$$

With reuse system the terms will change as follows:

$$PELQ_{(reuse)} = \frac{\overline{Z}_{1q}}{\overline{Z}} \times 100 \qquad (7\text{-}13)$$

$$R = 0 \qquad (7\text{-}14)$$

$$Dp = \frac{\overline{Z} - \overline{Z1}_q}{\overline{Z}} = 1 - \frac{\overline{Z1}_q}{\overline{Z}} = (1\text{-}DUL_Q) \times 100 \qquad (7\text{-}15)$$

Once this relationship is developed, the remaining work is to define DULQ as a function of the dimensionless variables K*, t_{co}*, L*, and a used to define the runoff percentage (R). This is possible through the use of Equation 6-18.

Integrating Equation 6-18 between the limits of 1-L' and 1-0.75L' and dividing by the dimensionless average depth infiltrated would result in the value of DULQ.

A complete example of the design procedure and performance prediction is given in the next chapter.

CHAPTER VIII

DESIGN PROCEDURE

This chapter summarizes the procedure that is proposed to design a reuse system for a given border. It assumes the basic physical variables of the irrigation system are known. They are the length, slope, type of crop to be grown, and the soil infiltration characteristics. Given a particular crop and location, an engineer can calculate the crop water requirement throughout the growing season using available techniques. Knowing the seasonal requirement and variation with time, one is also able to schedule the irrigation program. That is, at least one can know the depth of water to be applied for a given period of time. The system capacity equation then can be applied in the form

$$qt_{co} = L \cdot \frac{Z_r}{E} \tag{8-1}$$

or, in terms of the volume of water to be infiltrated, can be related

$$q_{in}t_{co} = L \cdot d \tag{8-2}$$

where q_{in} equals the inflow rate, t_{co} equals time of cutoff or application time, Z_r is the average depth of infiltration, E is the application efficiency (Eq. 7-1) Z_r can be calculated from the Kostoiakov Lewis function, $Z = kt^a$, t can be approximated by the infiltration time t_r required at the upper end of the border to infiltrate Z_r. Adjustment can be made to include the time lag at the upper end by using the SCS (1974) experimental results. For most practical purposes as an initial estimate, t_{co} can be equated to t_r. If the level of irrigation efficiency is estimated, the SCS (1974) border design charts can be used which makes equation 7-1 applicable.

The design procedure is outlined as follows:

STEP ONE

Determine the crop water requirement as a function of time.

STEP TWO

Schedule the irrigation to satisfy the water requirement for given period of time and know the depth to be applied for each irrigation.

STEP THREE

Using the system capacity equation or design charts, determine the inflow rate to the border.

STEP FOUR

For the given crop, slope, and soil determine the physical variables n, K and a.

STEP FIVE

Calculate the reference variables Y, X, T, and using these calculate the dimensionless variables L*, K*, and t_{co}*.

STEP SIX

Using figures 9 to 24, determine the runoff percentage for given t_{co}*, K*, L* and a values.

STEP SEVEN

Using the same figures as above for the given t_{co}*, K*, and a, determine the X^*_{max} using the zero runoff curve.

STEP EIGHT

From the given t_{co}*, K* and a, determine the shape function a_o from Figure 26.

STEP NINE

Calculate the infiltrated depth at the downstream end $Z_u(L)$ using Equations 6-17 to 6-19.

STEP TEN

Determine the time of runoff using the infiltrated depth calculated at the downstream end by Equation 6-20.

STEP ELEVEN

Calculate the volume of sump required for reuse system by Equation 6-1 or 6-3.

STEP TWELVE

(a). If cycling sump is adaptable use Equation 6-6 to determine the volume of sump required as a function of the pumping rate from the sump.

(b). Determine the diameter of the circular sump for different on and off float levels by Equation 6-8.

STEP THIRTEEN

If complete control of storage is required calculate the volume using Equation 6-16.

STEP FOURTEEN

If continuous pumping is to be used, select a pump for the rate of flow out of the sump and the total head in the system that can be calculated using Equation 6-21.

STEP FIFTEEN

Calculate the energy required for the system selected and use it for making management decisions.

STEP SIXTEEN

Estimate the potential application efficiency by using Equation 7-3.

Example: Given a field 200 m (about 650 ft) long with a slope S_0 = 0.001. The cumulative infiltration $Z = 5.12\ t^{0.7}$ where Z is in centimeters and t is in hours. The crop to be grown is alfalfa. The Manning roughness for fully grown alfalfa cover is estimated to be 0.15 $m^{1/6}$. The crop is to be grown in the Tucson area. Average border width of 12 m (about 40 ft.) is accepted for most of the farms in the area. Design a reuse system suitable for the given condition.

Solution:

Step One

a) The crop water requirement is calculated using the Modified Blaney-Criddle formula (SCS, 1967) and is given in Table 9 below.

Table 9. Consumptive Use of Alfalfa for Tucson Area

Month	Semi-monthly Consumptive Use (mm)	Cumulative Consumptive use (mm)
February	46.0 45.97	 46.0
March	69.9 81.8	115.9 197.7
April	89.9 102.1	287.6 389.4
May	117.1 136.1	506.8 642.9
June	135.6 139.2	778.5 917.7
July	134.6 115.6	1052.3 1167.9
August	112.5 126.5	1280.4 1406.9
September	109.0 80.8	1515.9 1596.7
October	63.0 47.0	1659.7 1706.7
November	41.7	1748.4

b) For the active root zone depth given by Erie et. al.(1965) the moisture to be added for each irrigation period was calculated based on 65 percent depletion before each irrigation period and is given in Table 10. The soil condition assumed is as given in Table 11.

Step Two

From Table 11 the maximum depth of water that can be applied at any given period is 151.89 (mm) (6.11 inches). Using this we can obtain an estimate of the flow rate by using the system capacity Equation 8-1 or design charts (SCS (1974). The time of cutoff is calculated using (SCS 1974) design charts which considers time lag and was found to be 260 minutes. Thus, using Equation 8-1

$$qt_{co} = L \cdot \frac{Z_r}{E}$$

The flow rate q is found to be 3.029 ℓ/s-m (0.0326 cfs/ft). An entering efficiency of 65% was used.

Step Three

Given q_{in}, S_0, n, k and a the reference variables Y, X and T and the dimensionless variables L*, K* and t_{co}* are calculated as follows.

Table 10. Assumed Soil Conditions and Available Moisture

Soil Depth (m)		Soil Texture	Available Moisture (mm)
0 - 0.3	(0-1 ft)	Loam	48.3
0.3 - 0.61	(1-2 ft)	Loam	48.3
0.61 - 0.91	(2-3 ft)	Fine Sandy Loam	40.6
0.91 - 1.22	(3-4 ft)	Fine Sandy Loam	40.6
1.22 - 1.52	(4-5 ft)	Fine Sandy Loam	40.6
1.52 - 1.83	(5-6 ft)	Loamy Sand	20.3
Total Available Moisture in the Entire root zone			239

Table 11. Moisture to be Added per Irrigation

Number of irrigations	Active Root Depth (m)	Available Moisture (mm)	Moisture to be Added (mm)
1st two	0 - 0.61 (0 - 2 ft)	96.5	62.7
2nd two	0 - 0.91 (0 - 3 ft)	137.2	89.2
3rd two	0 - 1.22 (0 - 4 ft)	177.8	115.6
4th two	0 - 1.52 (0 - 5 ft)	218.4	142.0
5th two	0 - 1.83 (0 - 6 ft)	233.7	151.9
6th - end	0 - 1.83 (0 - 6 ft)	233.7	151.9

$$Y = \left(\frac{q_{in}\ n/Cu}{\sqrt{S_0}}\right)^{3/5} = \left(\frac{0.003029 \times 0.15}{\sqrt{0.001}}\right)^{3/5} = 0.0784 \text{ m } (.253 \text{ ft.})$$

$$X = Y/S_0 = \frac{0.078}{0.001} = 784 \text{ m } (257.3 \text{ ft.})$$

$$T = \frac{XY}{q_{in}} = \frac{0.07842 \times 78.42}{0.003029} = 33.8 \text{ min } (2030.3 \text{ sec.})$$

$$K^* = \frac{kT^a}{Y} = \frac{.051206\ \left(\frac{33.84}{60}\right)^{0.7}}{0.07842} = 0.437$$

$$t^* = \frac{t_{co}}{T} = \frac{260}{33.84} = 7.68$$

$$L^* = \frac{L}{X} = \frac{198.1}{78.42} = 2.53$$

Step Four

Using Figures 20 and 21, the runoff percentage is determined by interpolating for K* = 0.4 and K* = 0.5. An R value of 32 percent is obtained.

Step Five

Using Figures 20 and 21, the maximum dimensionless advance distance, X^*_{max} is obtained by interpolation. This is done by matching the zero runoff lines with the t_{co}^* value of 7.68 for both K* values. An X^*_{max} value of 4.50 is obtained.

Step Six

From Figure 26 for $t_{co}* = 7.68$, $K* = 0.437$, and $a = 0.7$. The slope factor a_0 is found to be 1.51.

Step Seven

The infiltrated depth at the downstream end (minimum for the given length) is determined as follows.

$$Z' (L) = a_0(\tau)^{a/2} + a_1 \tau$$

where $\tau = 1-L'$

$$L' = \frac{L*}{X^*_{max}}$$

$$a_1 = \frac{4 + 2a_0 - 4a_0}{2 + a}$$

thus for $a_0 = 1.51$

$$a_1 = \frac{4 + 2\ (1.51) - 4(1.51)}{2 + 0.7} = -0.237$$

$$L' = \frac{2.53}{4.5} = 0.562$$

$$\tau = 1 - 0.562 = 0.438$$

$$Z'(L) = 1.51\ (.438)^{0.35} - 0.237\ (.438) = 1.027$$

$$Z_u(L) = \overline{Z}* \times Z'(1) \times Y$$

where

$$Z^* = \frac{t^*_{co}}{X^*_{max}} = \frac{7.68}{4.50} = 1.71$$

therefore $Z_u(L) = 1.71 \times 1.027 \times 0.0784 = 0.138$ m

Step Eight

The time of runoff is calculated by using Equation 6-10

$$T_R = \left(\frac{Z_u(L)}{k}\right)^{1/a} = \left(\frac{0.1377}{.0512}\right)^{1/0.7} = 4.11 \text{ hr.} = 247 \text{ min.}$$

Step Nine.

From known R, T_R, T_{co}, Q_{in} (q_{in} x width), a relationship between sump volume V, pumpage rate from the sump Q_p for different time of pumping Tp using Equation 6-1 or 6-3.

Thus using Equation 6-1

$$\forall = Q_{in}t_{co}R + (N-1)(\overline{Q}_R T_R - Q_p T_p)$$

$$\overline{Q}_R = \frac{Q_{in}t_{co}R}{T_R} = \frac{0.00302 \times 12.2 \times 260 \times 0.32}{247} = 0.0124 \text{ m}^3\text{/sec}$$

$$\forall = 0.0369 \times 260 \times 60 \times .32 + (N-1)(184.2 - Q_p T_p)$$

$$\forall = 184.2 + (N-1)(184.2 - Q_p T_p)$$

Assume there are two borders that are contributing to the sump. Then

$$\forall = 184.2 + (184.2 - Q_p T_p)$$

Table 12 gives $\forall$ as a function of Q_p and T_p. It should be noted that for pumpage rates of 0.0499, 0.0249, 0.0166 and 0.0124 m^3/sec there would be no additional storage required than the buffer storage 184 m^3 for time of pumping of 0.25 T_R, 0.5 T_R, 0.75 T_R and T_R, respectively.

Step Ten

If a cycling sump is to be used, Equation 6-6 is applied with Equation 6-8 to obtain the result given in Table 13. It should be recognized that Table 13 gives only the minimum value using 15 cycles per hour but, as pointed out by Davis (1964), there are other restrictions such as:

1. inside diameter being at least five times the inside diameter of the pump column.

2. clearance between the sump floor and the strainer being at least one-half the inside diameter of the pump column.

3. lowest water level being able to provide submergence over the pump strainer, and

4. Any local design requirements.

Table 12. Volume of Sump for Different Pumping Rate and Time.

Pumping Rate	Volume of Sump (V) (m^3) Time of pumping (T_p)			
	0.25 T_R	0.5 T_R	0.75 T_R	T_R
3.15	357	345	333	322
6.31	345	322	298	275
9.41	333	298	263	228
12.4	322	277	231	184
16.6	306	245	184	---
18.9	298	228	---	---
22.1	287	205	---	---
24.9	276	184	---	---
28.2	264	---	---	---
31.5	252	---	---	---
34.7	240	---	---	---
37.8	228	---	---	---
41.0	216	---	---	---
44.2	205	---	---	---
49.9	184	---	---	---

Table 13. Minimum Size of Sump for Reuse System. (C = 15 Cycles per hour).

Inflow to the Sump (liters/sec)	Inside Diameter of Circular Sump (cm)					
	Depth in Storage Between on and Off Levels (m)					
	0.5	1.0	1.5	2.0	2.5	3.0
1.0	39	28	23	20	17	16
2.5	62	44	36	31	28	25
5.0	87	62	50	44	39	36
10.0	124	87	71	62	55	50
15.0	151	107	87	76	68	62
20.0	175	124	101	87	78	71
25.0	195	138	113	98	87	80
30.0	214	151	124	107	96	87
35.0	231	164	134	116	103	94
40.0	247	175	143	124	111	101
45.0	262	185	151	131	117	107

Step Eleven

For complete storage of the total runoff, the reservoir size is calculated by:

$$\forall = N\,Q_{in}\,t_{co}R$$

$$\forall = 2(0.0369)\ 260 \times 60 \times .32 = 368\ m^3.$$

Step Twelve

For the Q_p value given in Table 12, the next job is to calculate the total head required by Equation 6-21. Assume a flow of 0.0124 m^3/sec which is the flow rate that can irrigate a border at a time of pumping equal to T_R is to be used. Assume suction lift of 2 meters, Manning Roughness Coefficient of 0.011 $m^{1/6}$ for aluminum pipe. The length of the pipe will be $\sqrt{12.2^2 + 198.1^2} = 198.5 \approx 200$ meters. This assumes the length of reuse pipe to be the diagonal which is a common lay out. Since the minor losses are very small, they are neglected most often.

Thus, using Equation 6-21 the total head is calculated as a function of the diameter by:

$$H = H_s + S_0L + 10.28 \times 10^{10}\ n^2\ LQ_p{}^2/D^{16/3}$$

$$H = 2 + (0.001)(198.1) + (10.28)(10^{10})(0.011)^2(198.1)(12/4)^2/D^{16/3}$$

$$H = 2.198 + 3.79 \times 10^{11}/D^{16/3}$$

Step Thirteen

The energy required for the given discharge and head is then calculated using

$$\text{Energy (E)} = 9.8 \times 10^{-3} Q_p \cdot H \cdot T_R$$

E is the energy in kilowatt hours; where Q is flow rate in ℓ/sec, H is the total head (m) and T_R is in hours. The energy required assuming an 80% efficient pump is 1.4 KWH with the diameter not making a significant difference.

Step Fourteen

The potential application efficiency is determined by making use of Equations 7-5 to 7-7.

With R given in Step Four, the important component left is the DULQ which can be calculated by using Equation 6-18 as shown below.

$Z_u(X) = a_0(\tau)^{a/2} + a_1 \tau$ where $\tau = 1\text{-}L'$) and $L' = L^*/X^*_{max}$. When $X = 0$, $L' = 0$, $\tau = 1$ and when $X = L$, $\tau = 1\text{-}L'$.

Using Equation 7-5 for $a_0 = 1.51$, $a_1 = -0.237$, $L' = 0.562$ and $\tau = 0.438$, DULQ will come out to be 0.92. Where $\bar{Z}_{lq} = 0.143$ (m) and $\bar{Z} = 0.156$ (m).

By Equation 7-8

$$\begin{aligned} \text{PELQ} &= \text{DULQ (1-R)} \\ &= .92\ (1-0.32) \times 100 \\ &= 63 \text{ percent.} \end{aligned}$$

Table 14 summarizes the efficiency values with and with out re-use system obtained using Equations 7-5 to 7-7.

Table 14. Efficiency Evaluation

Efficiency Term	Without Reuse System	With Reuse System
DULQ	92	92
R	34	0
PELQ	63	92
D_p	3	3

It can be noted by looking at Equation 4-28 that of all the terms affecting runoff, time of cutoff and inflow rate are the only two variables that can be changed during irrigation or in the short term. Thus the designer has to try different values of the two variables until a particular design satisfies the needs.

Also the use of Equation 6-18 makes it possible to check which part is under-irrigated or over-irrigated if the desired depth of infiltration is known. Adjustments can be made by changing the time of cutoff and the inflow rate.

The choice of the system also depends on the personal preference as well as some economic and other constraints like availability of land.

The energy required can be meaningful if the diameter of the pipe is fixed since the head varies with diameter. Cost estimates for reuse system also can be made once the flow rate and the diameter are selected.

CHAPTER IX

SUMMARY AND RECOMMENDATION

Increased use of mathematical models is being made in open channel hydraulics as the result of better understanding of numerical analysis and availability of more capable high speed computers. In surface irrigation the variability of the physical parameters makes it more desirable to use such models. A zero-inertia mathematical model was used to get both dimensional and dimensionless solutions of the governing equations of motion and continuity for shallow, non-uniform unsteady flow over porous media.

Runoff, one of the outputs of the model, was combined with several border irrigation physical variables to observe its behavioral trend with physical variables which are inputs to the model and to present runoff curves which in turn are used to develop design criteria for reuse systems.

The important physical variables identified were inflow rate, time of cutoff, length of run, surface resistance defined by Manning roughness coefficient, slope of the border and the soil infiltration characteristics defined by the Kostoiakov Lewis power law function represented by constants k and a.

Runoff was found to increase with increase in slope, time of cutoff and inflow rate, and decrease with increase in length, intake rate (k and a) and roughness. The effect of slope and roughness was found to be more critical than others with high intake families of soils than with low intake families.

The dimensional solution was also extended to show a very important result with respect to intake families. Studies to date were made using different "a" values for the Kostiakov Lewis function making it necessary to have many computer runs for each "a" value. Runoff was found to be unaffected when the "a" value was set to 0.7 and the "k" value for different families was adjusted for the same value of cumulative depth of infiltration and intake opportunity time. This makes it possible to use the result of runoff study made for one intake family (a = constant) for a different family by simply adjusting the constant k.

Recognizing the practical impossibility of studying behavioral trends of combinations of several irrigation physical variables, and the difficulty of presenting the results of such a study, a dimensionless solution was used for the major part of this study to develop the runoff curves. Significant reduction in the number of curves and simplification of behavioral studies was realized by non-dimensionalization.

Extension of the dimensionless solutions also made it possible to get the maximum advance distance and to present it with the runoff curves adding to the use of the curves for border irrigation design other than just for reuse system design.

A volume balance approach was used to develop equations for designing reuse systems under different operational conditions. These equations make use of the runoff curves developed to give the components of the reuse system, mainly the sump and pump sizes.

Use of a shape function for the ultimate post irrigation subsurface water profile was made to define the various efficiency terms that should be used to check the design of the irrigation system.

A procedure for using the runoff curves and reuse system design equations is proposed. The procedure helps in understanding the relationship between the dimensionless variables with the border irrigation physical variables and in making an actual design of a reuse system.

Further studies should be made to use decision analysis (for example, goal programming) to establish selection criterion among the different reuse systems with cost, environmental quality, labor, etc., as constraints.

Adjustment of k and a values, as used in this study, should be further investigated with respect to other dependent variables (model outputs) to reduce expenses.

A computer model that is capable of simulating cutback in inflow and its effect in reducing runoff should be developed in the future to establish evidence for deciding the need to have, or not to have, a reuse system.

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