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EVALUATION AND IMPROVEMENT OF BORDER IRRIGATION

By Gideon Peri,
Donald I. Norum and
Gaylord V. Skogerboe

Water Management Research Project
Colorado State University
Fort Collins, Colorado

July 1979

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OF BORDER IRRIGATION

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ABSTRACT

The evaluation of border irrigation requires the use of a model that relates the operating conditions to the irrigation results, such as: (a) total water quantity delivered to the border; (b) water losses outside the border by runoff; (c) average depth of application; and (d) water distribution within the border. Evaluation for improving border irrigation can be carried out in two major ways: (a) field tests; and (b) theoretical models.

The improvement of border irrigation based on field tests requires the estimation of the infiltration equation and the advance and recession for a specific inlet stream and border parameters (slope, length, surface). The performance parameters can then be calculated to determine the irrigation performance and the need for improvements. Then, the operating conditions that are to be changed can be determined. This analysis comprises two stages: (a) utilization of the field test data already available; and (b) implementation of the recommended changes for further field tests.

Theoretical models for the evaluation and improvement of border irrigation performance enable one to predict the water distribution and losses for a given set of conditions, without direct field measurements. Only limited amounts of preliminary field work are required to predict the irrigation performance for a wide range of parameters and variables. Available models are described, as well as the process for applying these models.

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NOMENCLATURE

A = constant in Kostiakov infiltration equation.

a = area fraction in cumulative water distribution.

a_p = fraction of the area infiltrating more than the mean infiltration depth.

a_R = fraction of the area infiltrating more than the required depth.

B = exponent in Kostiakov infiltration equation.

b = exponent in the power function distribution.

C = a parameter (see Eq. 4-11).

CU = units coefficient (1.0 for m-s, 1.49 for ft-sec).

D = volume of water applied to the border.

d_F = depth of surface water at advancing front.

$d_L(t)$ = flow depth at end of the border.

$d_o(t)$ = depth of flow at upstream end.

E_d = delivery efficiency.

E_P = deep percolation efficiency.

E_S = storage efficiency.

EV = volume of water lost to evaporation.

f = constant in the nondimensional power function distribution.

G = volume of deficit relative to the required depth.

g = constant in the nondimensional power function distribution.

H = $y/\bar{y}$ = nondimensional depth of water infiltrated.

$\bar{H}$ = nondimensional mean water application = 1.

H_D = nondimensional mean depth of water applied.

H_E = nondimensional mean excess relative to the required depth.

H_F = nondimensional mean deficiency relative to the mean depth infiltrated.

H_G = nondimensional mean deficiency relative to the required depth.

H_{max} = nondimensional maximum depth of water infiltrated.

H_{min} = nondimensional minimum depth of water infiltrated.

H_R = $h_R/\bar{y}$ = nondimensional depth of water required.

H_W = nondimensional mean depth of runoff.

h_D = mean depth of water applied.

h_G = mean deficit relative to the required depth.

h_R = required depth of application of water.

I = average infiltration rate.

K = shape factor in two linear section distribution.

L = length of border.

ℓ = L - x = distance from lower end of border.

n = roughness coefficient in Manning's equation.

P_1 = weighting factor for T.

P_2 = weighting factor for T_{est}.

p = number of stations or observations.

q = flow rate onto border.

$q_L(t)$ = runoff rate.

R = ratio of T to T_{est}.

$R_{ro}(u)$ = runoff factor.

$R_{rec}(u)$ = recession factor.

r_y = surface depth shape factor.

r_z = subsurface depth shape factor.

S_o = slope of border.

S_y = change in surface depth with distance.

T = volume of water infiltrated.

T_{est} = estimated volume of water infiltrated.

t = time.

t_a = time of advance of water to a station on the border.

t_{aL} = $t_a(L)$ = advance time to end of border.

$t_a(x)$ = time that water arrived at point x (advance time).

t_{co} = water delivery time to border, cutoff time.

t_{op} = infiltration opportunity time.

t_r = time of recession of water at a station along the border.

t_ℓ = $t_{ro} - t_{co}$ = lag time.

t_{ro} = $t_r(o)$ = time for water to recede from border at x = 0.

$t_r(x)$ = time that water receded from point x (recession time).

U_d = distribution uniformity.

UCC = Christiansen's uniformity coefficient.

u = dummy limit of integration.

$V_p(t)$ = volume of water ponded on the surface at time t.

$V(t)$ = volume of water infiltrated to time t.

v_F = normal velocity associated with d_F.

W = total volume of runoff.

$W(t)$ = volume of runoff to time t.

$W_r(t)$ = volume of runoff from beginning of recession to time t.

x = distance from upper end of border.

$x_a(t)$ = advance distance at time t.

x_i = length of the border represented by point i.

$x_r(t)$ = point at which recession is taking place at time, t.

x_{ri} = point where recession starts when border is not completely ponded.

Y_o = depth of ponded water at x = 0.

y = depth of water infiltrated.

$\bar{y}$ = mean depth of water infiltrated at end of irrigation.

y_{max} = maximum depth of water infiltrated.

y_{min} = minimum depth of water infiltrated.

$y_o(t)$ = depth of water infiltrated at $x = 0$.

y_{typ} = depth of water infiltrated as estimated from typical infiltration curve.

y_x = depth of water infiltrated at a distance x from the head end of the border.

$z(t_{op})$ = cumulative infiltrated depth of water in infiltration equation.

β = degree of monomial power curve describing surface water profile.

Δt = a time increment.

Δt_r = $t_r(L) - t_{ro}$ = time interval between recessions at lower and upper ends of border.

ΔV = increase in subsurface storage during time increment Δt.

ΔV_p = decrease in volume ponded on the surface during time increment Δt.

Δz = difference between depth of water infiltrated at upper and lower ends of border.

Δz_L = increase in depth of water infiltrated at the lower end of the border during time increment Δt.

Δz_x = increase in depth of water infiltrated at point x during time increment Δt.

ζ = variable of integration.

Chapter 1

CHARACTERISTICS OF BORDER IRRIGATION

1. Description of Border Irrigation

Border irrigation is a method of controlled surface flooding. The border strip is rectangular with a relatively long dimension in the flow direction and a narrow width. It is bounded by dikes or levees along its longer dimension. The dikes restrict the lateral movement of water, causing it to flow to the end of the field between the dikes.

There are three common types of border irrigation: level border, graded border, and guide border. This report is concerned with the graded border type in which there is a gentle and uniform slope in the direction of irrigation. The water flows from the top end of the border in an elongating thin shallow sheet. The border is leveled across and the width is small enough to ensure a relatively uniform distribution of the stream across the border.

The desired volume of water is applied in a time equal to or slightly less than that needed for the soil to absorb the required depth of water. Consequently, to minimize end runoff, the water must be near the lower end of the border by the time that an adequate depth of water has infiltrated at the upper end. Practically this means that the water should be shut off at the upper end before the flow has reached the lower end. Following the water shutoff, there is a recession of the water over the surface. However, generally some runoff occurs at the end of the border, unless it is diked.

2. Water Distribution Pattern Under Graded Border Irrigation

The water distribution pattern associated with a graded border is a direct result of the opportunity time, that is the contact time of water and soil at any point along the border. Two assumptions concerning the water distribution pattern are often made:

a. There is a relatively uniform infiltrated depth of water across the border. Therefore, the water distribution is sufficiently described by the distribution along the border's long axis.

b. The maximum depth infiltrated is usually somewhere along the border and not at the upper end. Most commonly, the infiltrated water depth at the upper end is equal or close to the theoretical required depth of application, and the infiltrated depth at the lower end is the minimal depth infiltrated.

A typical water distribution is shown in Fig. 1-1 by three common methods of representation (for more details and notations, see Peri and Skogerboe (1979)).

When the water distribution profile of the graded border is established, the irrigation performance can be evaluated.

Example

Two irrigation tests were carried out with borders of 320m length but with different stream sizes and shutoff times. The water distribution was estimated after each irrigation and is given in Table 1-1 and Fig. 1-2 in a form of actual water depth distribution (see Chapter 2 for details of method of evaluation).

Table 1-2 contains measured and calculated parameters for the two tests, where

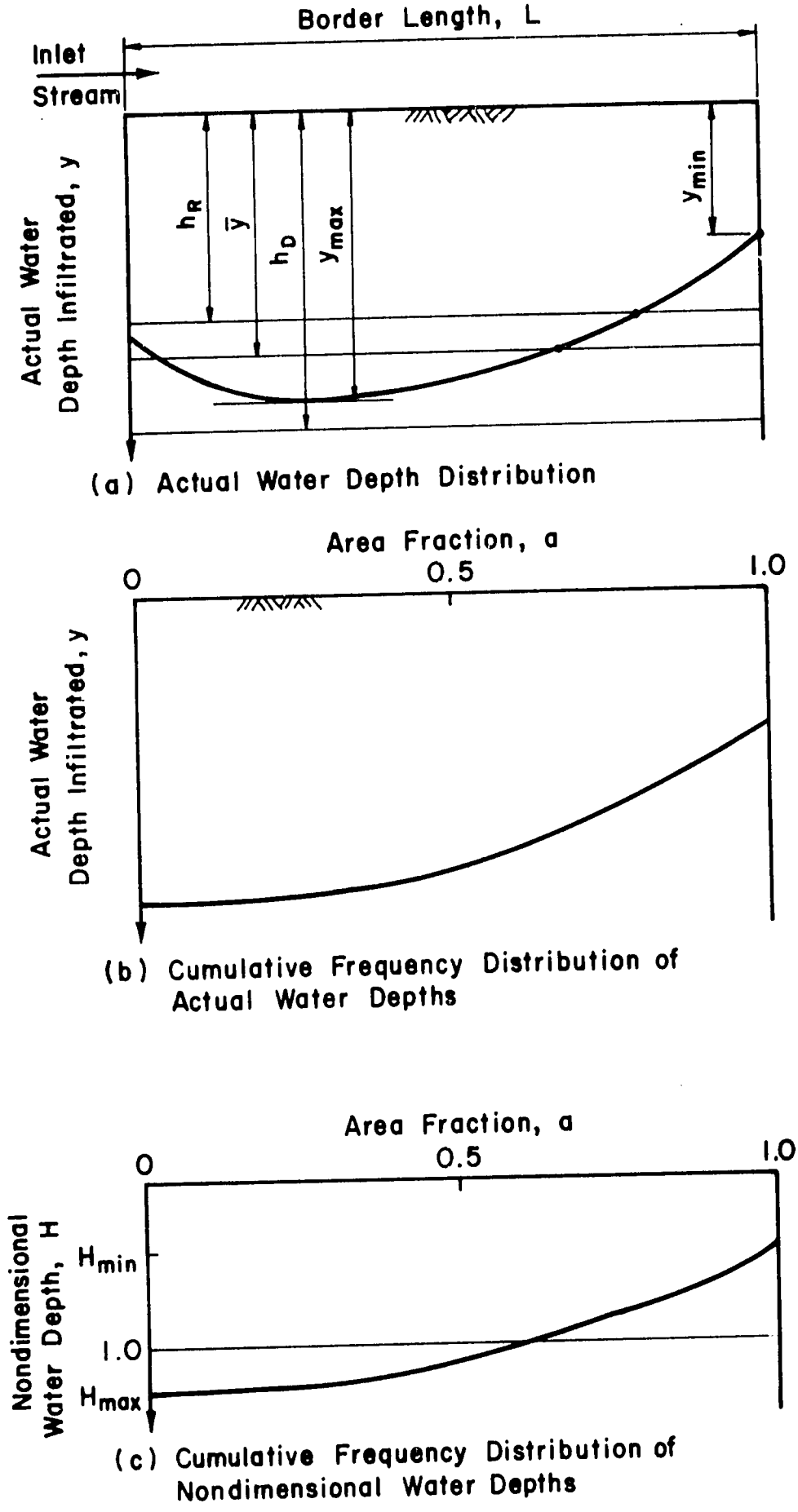


Fig. 1-1. Water distribution under border irrigation.

Table 1-1. Water distribution in two tests of border irrigation.

Station No.	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Length (m)	10	30	50	70	90	110	130	150	110	190	210	230	250	270	290	310
Water depth Test a - (mm)	50	54	57	59	60	60	59	58	57	55	54	51	49	46	43	40
Water depth Test b -(mm)	56	68	76	80	84	86	85	83	80	74	68	60	52	43	32	20

Table 1-2. Data for two border irrigation water distributions.

Parameter	Volume applied D m^3/m	Volume infiltrated T m^3/m	Volume of runoff W=D-T m^3/m	Depth applied h_D mm	Depth infiltrated $\bar{y}$ mm	Distribution uniformity U_d	Delivery efficiency E_d
Test a	19.2	17.0	2.2	60	53.2	0.952	0.888
Test b	24.0	20.9	3.1	75	65.4	0.876	0.873

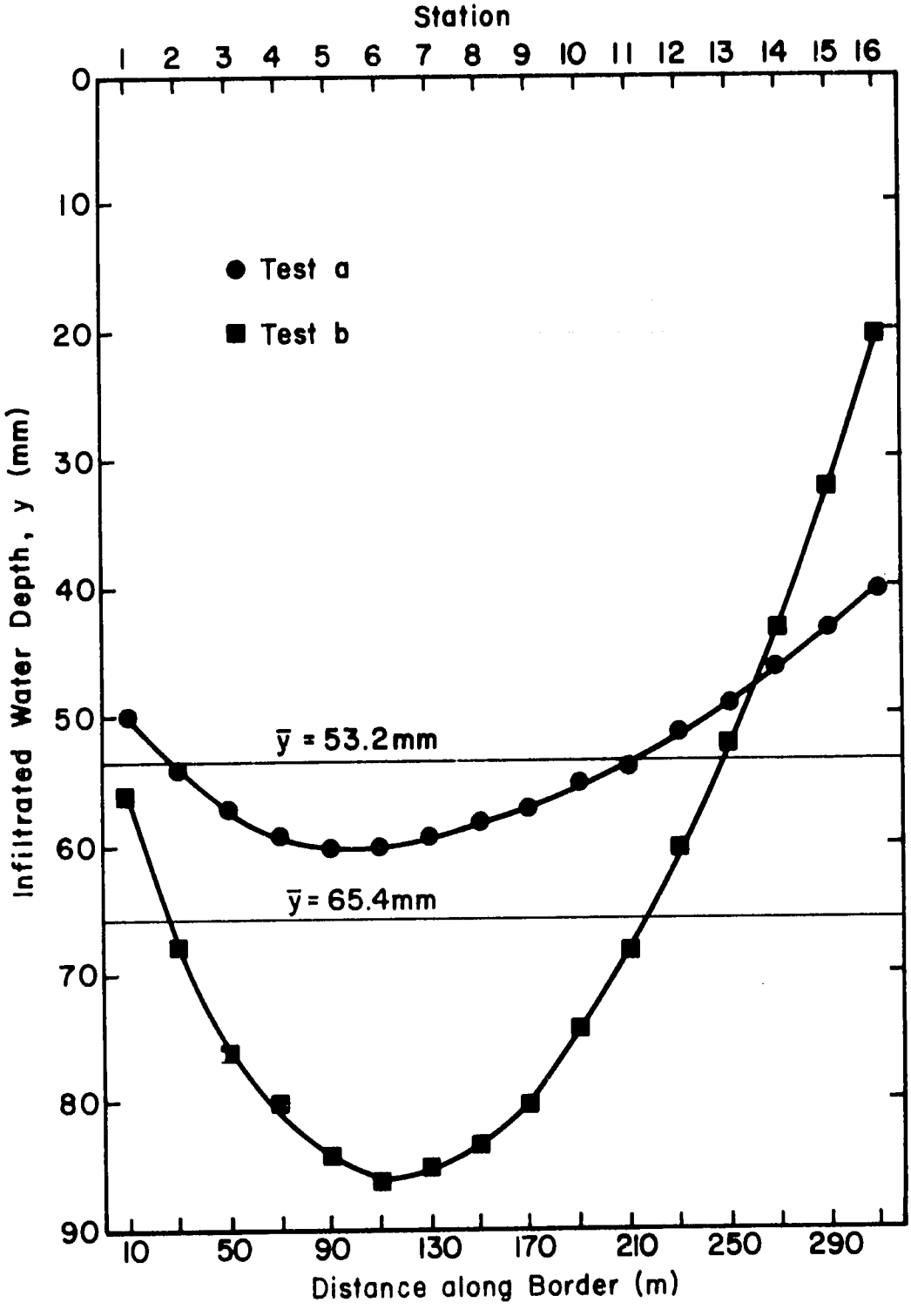


Fig. 1-2. Water distribution for two border irrigation tests. (Example)

D = volume of water applied to the border

T = volume of water infiltrated

W = volume of runoff

$\bar{y}$ = mean depth of water infiltrated

h_D = mean depth of water applied

U_d = distribution uniformity, and

E_d = delivery efficiency.

All volumes above and elsewhere in this report are per unit width of border.

The deep percolation efficiency, E_p, and the storage efficiency, E_S, for various required depths of application, h_R, for the two tests are shown in Figs. 1-3a and 1-3b. The deep percolation efficiency is the fraction of the total water absorbed in the irrigated area which contributes to fulfilling the requirement, and is a measure of the water lost to deep percolation. The storage efficiency is the fraction of the available root zone water storage (at the time of irrigation) which is filled by the irrigation, and is a measure of the adequacy of the irrigation. For more details see Peri and Skogerboe (1979).

If the border irrigation of Test a was aimed to provide a depth of application of 50 mm, the irrigation performance can be regarded as satisfactory, when E_p and E_S are considered satisfactory at the 0.8 level and $U_d > 0.9$. The irrigation performance is in category I (for definitions of categories see Peri and Skogerboe (1979)) and no improvements are needed. The delivery efficiency is also satisfactory with $E_d > 0.8$ so that the runoff is also acceptable with $W = 2.16\ m^3/m$.

If, however, the designed application depth for this border was 40 mm, $E_p = 0.751$, thus the irrigation performance belongs to category IV which requires a decrease in the depth of water actually applied (average

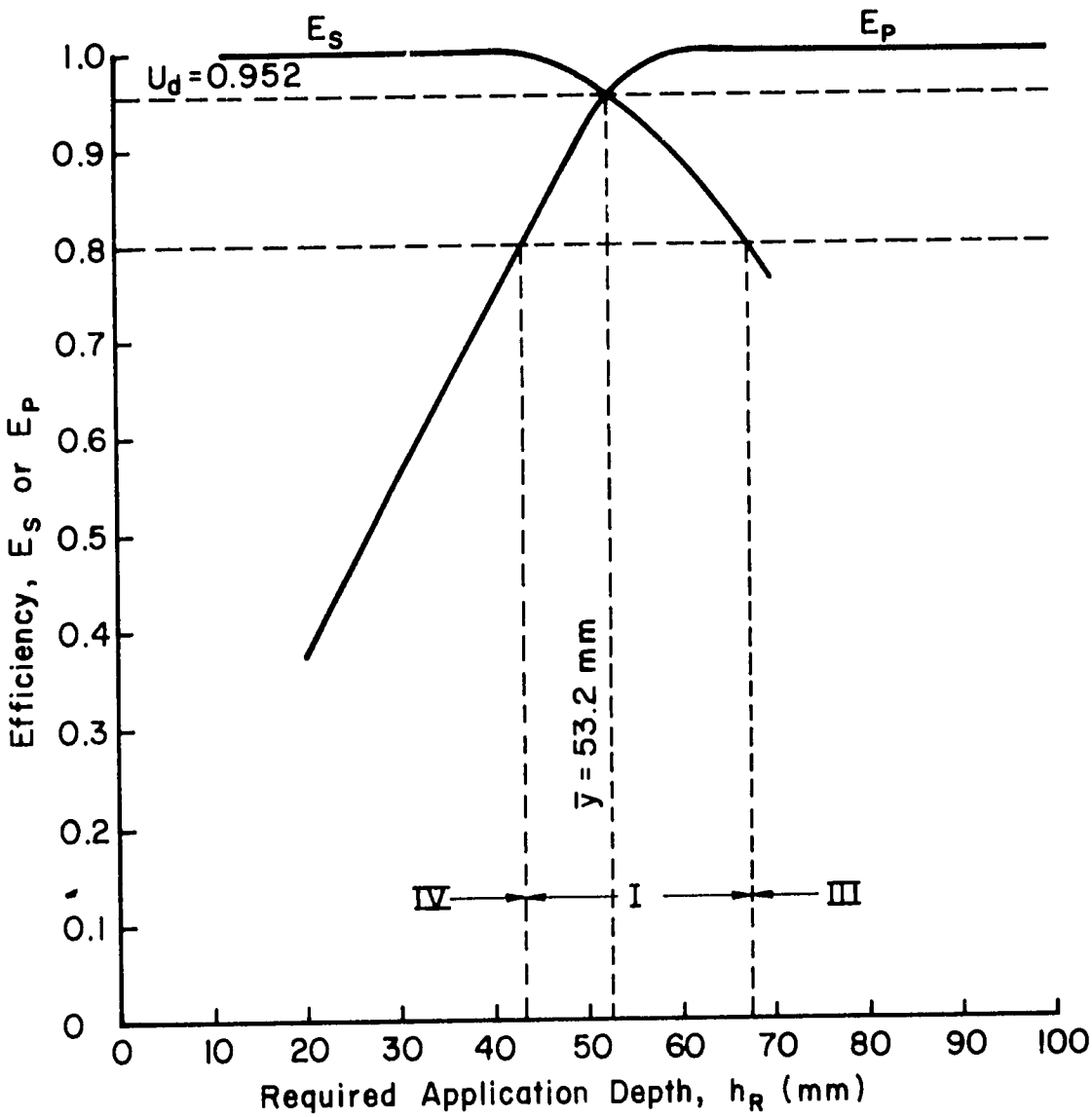


Fig. 1-3a. Irrigation efficiencies for border Test a. Roman numerals refer to irrigation performance categories (see Peri and Skogerboe (1979)).

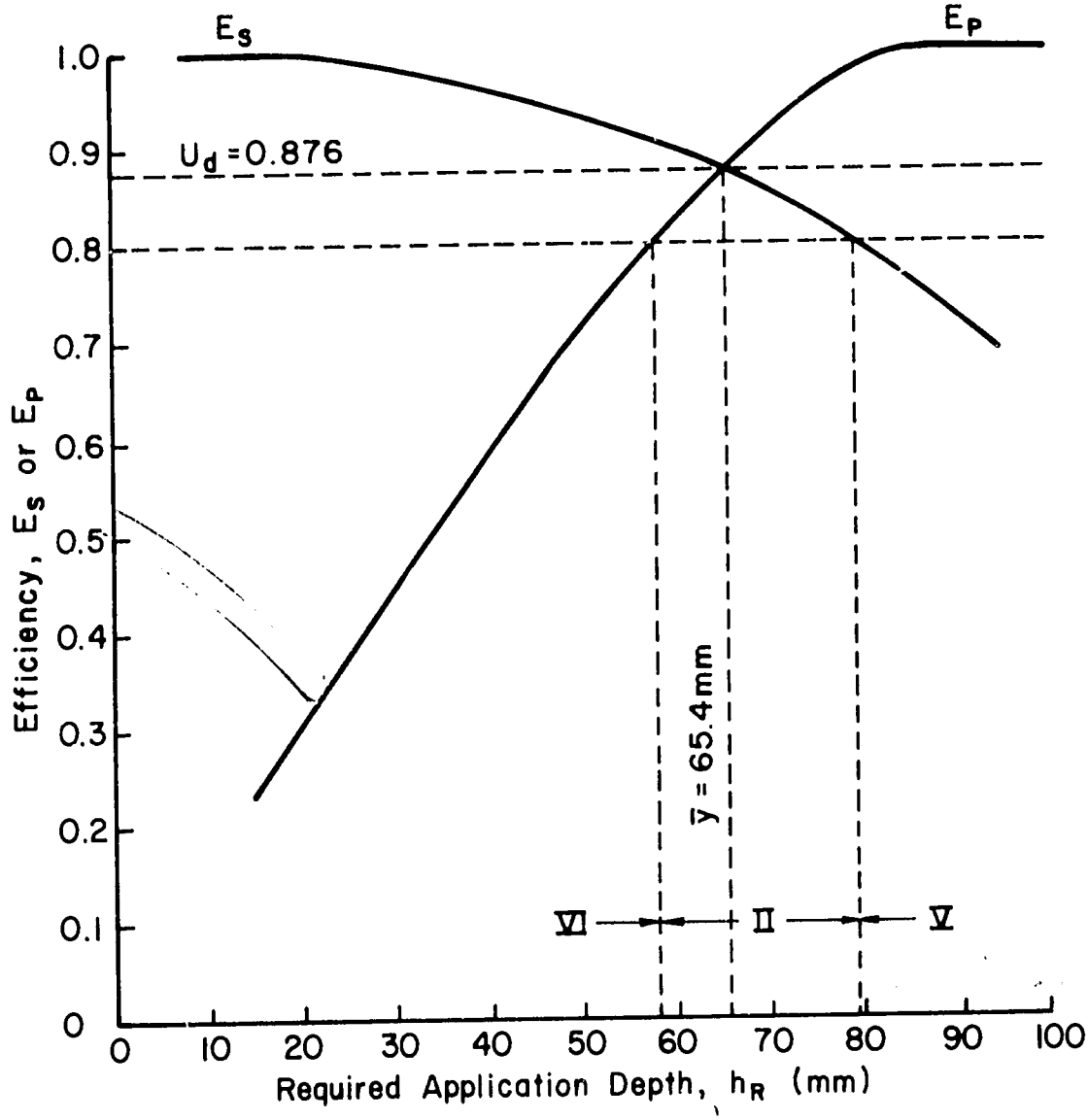


Fig. 1-3b. Irrigation efficiencies for border Test b. Roman numerals refer to irrigation performance categories (see Peri and Skogerboe (1979)).

depth of water applied is 53.2 mm). If the required depth for this border was 68 mm, E_S = 0.783 and the irrigation performance belongs to category III, which requires an increase in the application depth.

The runoff problem is satisfactory and it is independent of the depth of application.

For the border irrigation of Test b, the distribution uniformity is too low (U_d = 0.876, Christiansen's uniformity coefficient, UCC = 0.752), hence, the water distribution should be improved. If the required depth of application is 70 mm, the irrigation performance belongs to category II, for which only the distribution of water should be improved.

If, however, the required depth of application is 50 mm, E_p = 0.711 and the irrigation performance belongs to category VI, which means that the water distribution should be improved along with a decrease in the depth of application.

If on the other hand, the required depth of application for this irrigation is 85 mm, the irrigation performance belongs to category V, for which the water distribution should be improved together with an increase in the depth of application. For the border irrigation of Test b, the delivery efficiency is satisfactory which means that no runoff decrease is essential.

The various possible irrigation and runoff performances for the two border irrigation tests are summarized in Table 1-3.

Table 1-3. Irrigation performances and runoff problem for the two border test irrigations.

Irriga. Perform. Cat.	I	II	III	IV	V	VI	Runoff $E_d<0.8$	Runoff $E_d\geq0.8$
border test a	$43mm \leq h_R \leq 67mm$	not possible ($U_d>0.9$)	$h_R > 67mm$	$h_R < 43mm$	not possible ($U_d>0.9$)	not possible ($U_d>0.9$)	$D>21.3$ m^3/m	$D<21.3$ m^3/m
border test b	not possible ($U_d<0.9$)	$58mm \leq h_R \leq 82mm$	not possible ($U_d<0.9$)	not possible ($U_d<0.9$)	$h_R \geq 82mm$	$h_R \leq 58mm$	$D>27.6$ m^3/m	$D<27.6$ m^3/m

3. Nondimensional representation of water depth distribution in border irrigation.

Water distribution, under border irrigation, as demonstrated in Fig. 1-1, is usually not given as a continuous wetting profile, but as a set of discrete values at points distance x along the border. The discrete points of the wetting profile are obtained by field measurements, theoretical calculations or both (see following chapters).

The calculation of the irrigation efficiencies is carried out with the individual discrete values, as shown in the example of Fig. 1-2. However, it is sometimes preferable to define the whole wetting profile as a continuous function rather than by using individual values of y. This is mainly for the following reasons:

a. The calculation of each of the efficiencies and the associated coefficients is more accurate when it is based on the whole wetting profile (this is equivalent to increasing the number of observations).

b. The calculation is simpler and no corrections for area size represented by each measurement point is needed and there is no need for many individual computations.

To enable the calculation of the efficiencies directly from the wetting profile, the continuous equation that gives the depth of infiltrated water, as a function of the location along the border, is required:

$$y_x = y(x) \tag{1-1}$$

in which,

y_x = depth of water infiltrated at a distance x from the upper border end.

Establishing the water distribution function of Eq. 1-1 is usually carried out by graphical fitting or least squares fitting techniques where typical functions (linear, power, trigonometrical, statistical, etc.) can be fitted to any of the water distribution representations (Fig. 1-1, a, b, c).

The cumulative frequency distribution of nondimensional water depths is the most convenient presentation of the water profile as developed under surface irrigation. Furthermore, Karmeli (1978) has shown that the water distribution under border irrigation can best be defined by a power function of the form:

$$H = f + ga^b \tag{1-2}$$

where

H = nondimensional water depth,

a = fraction of area receiving a nondimensional depth of H or more, and

f, g, b = constants.

Hence, when a set of individual water depths, y_i, is given, the transformation is made to obtain H and a (dimensionless depth and area, respectively) and then, a function of the form of Eq. 1-2 is fitted by the least squares method.

Eq. 1-2 and the nondimensional water distribution, Fig. 1-4, exhibit several characteristics.

$$\text{For } a = 0 \quad H = H_{max} \text{ and } f = H_{max} \tag{1-3}$$

$$\text{For } a = 1 \quad H = H_{min} \text{ and } g = H_{min} - H_{max} \tag{1-4}$$

The areas abca and cdec are equal, thus

$$\int_{o}^{a_p} H da - a_p = 1 - a_p - \int_{a_p}^{1} H da \tag{1-5}$$

Substitution of Eq. 1-2 into Eq. 1-5, carrying out the integration, and rearranging gives

$$b = \frac{g - 1 + f}{1 - f} \tag{1-6}$$

Substitution of Eqs. 1-3 and 1-4 into Eq. 1-6 results in

$$b = \frac{1 - H_{min}}{H_{max} - 1} \tag{1-7}$$

The exponent b determines the nature of the function (Fig. 1-5). For $0 < b < 1$ the function is convex with more than 50% of the irrigated area receiving less than the mean depth of application. For $b = 1$ the power function becomes a linear function. For $b > 1$ the function is concave with more than 50% of the irrigated area receiving more than the mean depth of application.

From Eqs. 1-3, 1-4, and 1-7 it can be seen that the power function is completely defined when the two nondimensional depths, H_{min} and H_{max}, are known.

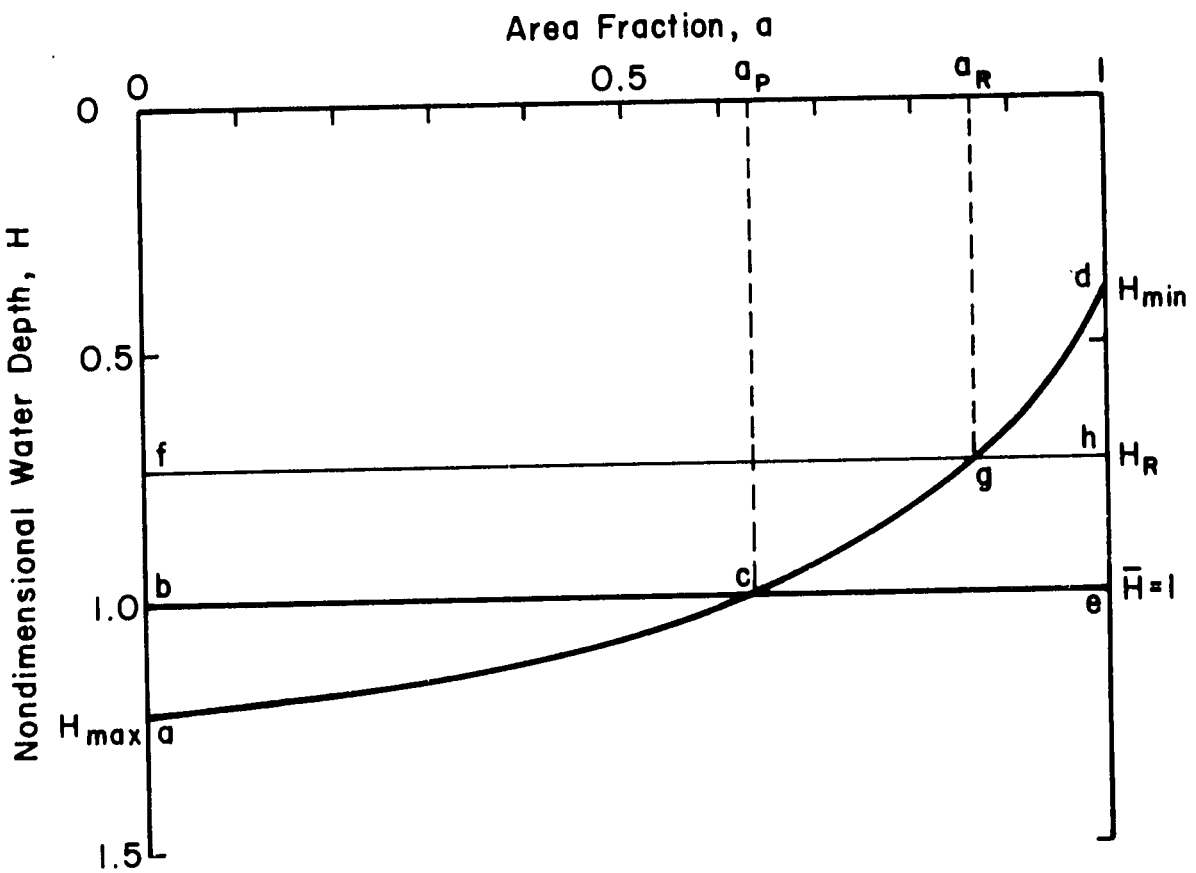


Fig. 1-4. Cumulative frequency distribution of nondimensional water depth in border irrigation.

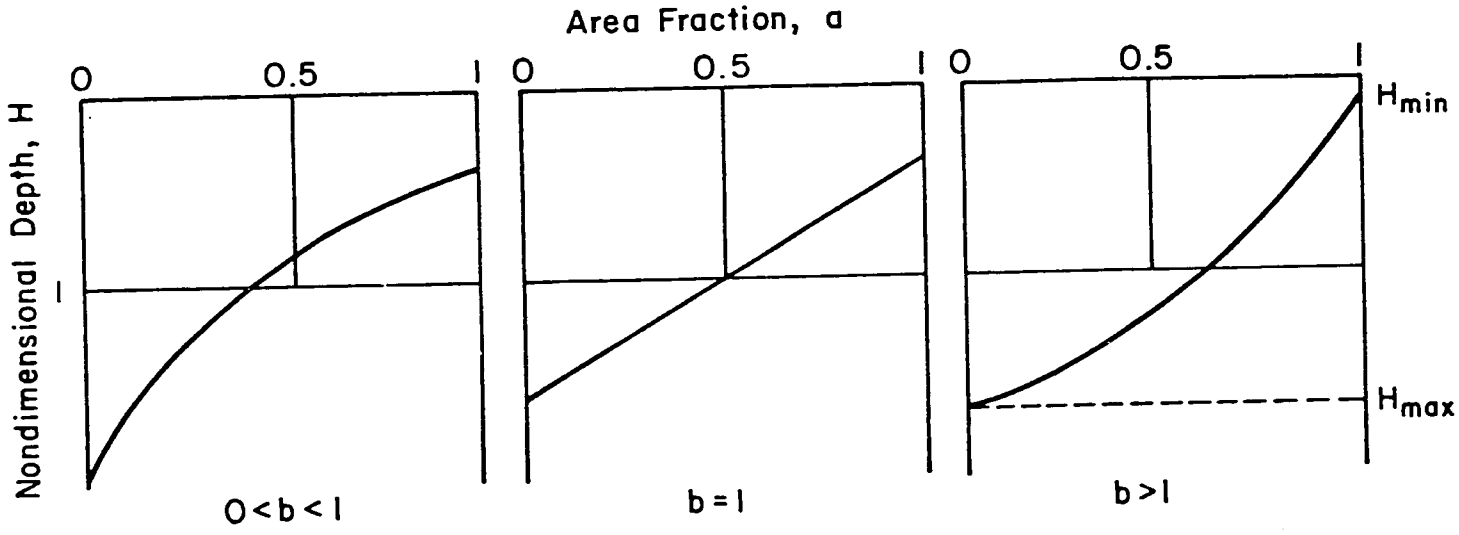


Fig. 1-5. Power water distribution function for various values of the exponent b.

The typical water distribution representing border irrigation (Fig. 1-1a) is concave, therefore the power function describing this distribution has an exponent of $b > 1$. In some cases where the distribution is relatively uniform with only a small area receiving an excessive amount of water, the distribution will be convex and $b < 1$.

When the nondimensional requirement, H_R is defined as

$$H_R = h_R/\bar{y} \tag{1-8}$$

various nondimensional volumes or average depths can be defined and used to calculate the various irrigation efficiencies. These are (refer to Fig. 1-4):

a. the deficiency relative to the required depth (area gdhg)

$$H_G = H_R(1-a_R) - \int_{a_R}^{1} H da \tag{1-9}$$

b. the deficiency relative to the mean depth infiltrated (area cdec)

$$H_F = 1 - a_p - \int_{a_p}^{1} H da \tag{1-10}$$

c. the excess application relative to the required depth (area afga)

$$H_E = \int_{o}^{a_R} H da - a_R H_R \tag{1-11}$$

By substituting Eq. 1-2 into Eqs. 1-9, 1-10 and 1-11 and carrying out the integrations the nondimensional volumes become

$$H_G = H_R(1 - a_R) - 1 + H_{max} a_R - (H_{max} - 1) a_R^{b+1} \tag{1-12}$$

where

$$a_R = \left(\frac{H_{max} - H_R}{H_{max} - H_{min}} \right)^{1/b} \tag{1-13}$$

$$H_F = a_p (H_{max} - 1) \left(\frac{b}{b + 1} \right) \tag{1-14}$$

where

$$a_p = \left(\frac{1}{b + 1} \right)^{1/b} \tag{1-15}$$

$$H_E = (H_{max} - H_R) a_R - (H_{max} - 1) a_R^{\,b + 1} \tag{1-16}$$

With a nondimensional total quantity of water delivered of H_D, the nondimensional runoff loss is

$$H_W = H_D - 1$$

Thus, the efficiencies are:

a. deep percolation efficiency

$$E_P = \frac{H_R - H_G}{\bar{H}} = H_R - H_G \tag{1-17}$$

b. storage efficiency

$$E_S = \frac{H_R - H_G}{H_R} = 1 - \frac{H_G}{H_R} \tag{1-18}$$

c. delivery efficiency

$$E_d = \frac{H_D - H_W}{H_D} = \frac{1}{H_D} \tag{1-19}$$

d. distribution uniformity

$$U_d = 1 - H_F \tag{1-20}$$

<u>Example</u>

Seven water distributions representing seven irrigation tests are given as a frequency distribution of the dimensionless depths in Fig. 1-6 and Table 1-4.

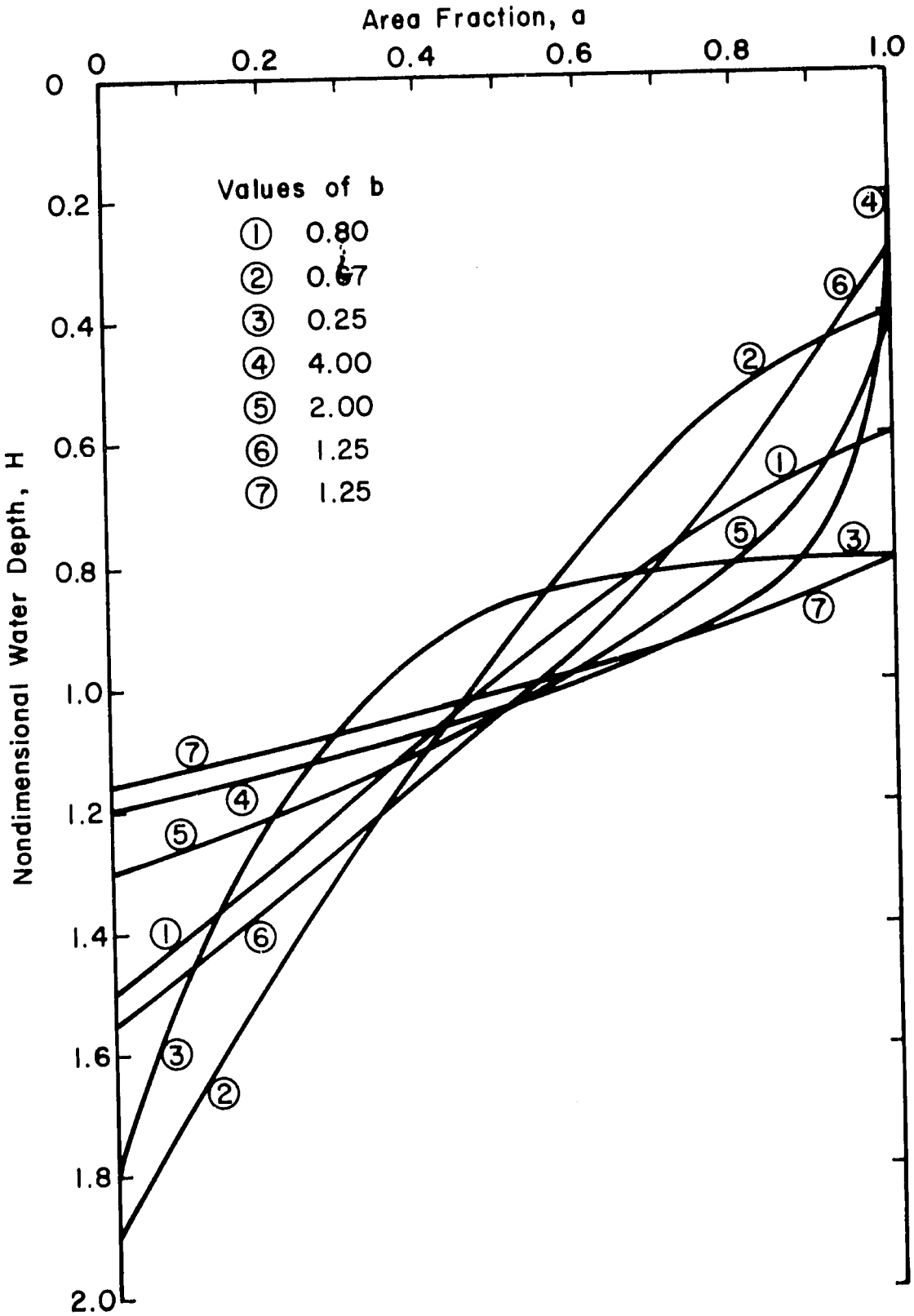


Fig. 1-6. Water distribution for seven tests of graded borders (example).

Table 1-4. Cumulative frequency distribution of seven border irrigation tests.

Test No.	Nondimensional Water Depths for Area Fraction a											Distribution parameters for $H = f + g\ a^b$		
	Fractional Area													
	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	f	g	b
1	1.500	1.389	1.281	1.176	1.075	0.978	0.886	0.799	0.720	0.650	0.6	1.5	-0.9	0.80
2	1.900	1.68	1.473	1.278	1.097	0.930	0.779	0.646	0.534	0.447	0.4	1.9	-1.5	0.67
3	1.800	1.456	1.210	1.040	0.930	0.863	0.826	0.808	0.802	0.800	0.8	1.8	-1.0	0.25
4	1.200	1.174	1.145	1.114	1.080	1.041	0.995	0.940	0.868	0.762	0.2	1.2	-1.0	4.00
5	1.300	1.254	1.205	1.153	1.097	1.036	0.969	0.893	0.802	0.685	0.4	1.3	-0.9	2.00
6	1.560	1.458	1.354	1.247	1.137	1.024	0.905	0.781	0.648	0.500	0.3	1.56	-1.26	1.25
7	1.16	1.131	1.101	1.070	1.039	1.006	0.972	0.937	0.899	0.857	0.8	1.16	-0.36	1.25

The efficiencies of four border irrigation tests are given in Table 1-5 and described in Fig. 1-7.

Table 1-5. Irrigation efficiencies for the border irrigation tests.

H_R	Test No. 7 $U_d = 0.957$ $E_d = 0.85$		Test No. 6 $U_d = 0.851$ $E_d = 0.83$		Test No. 2 $U_d = 0.804$ $E_d = 0.82$		Test No. 3 $U_d = 0.893$ $E_d = 0.90$	
	E_p	E_s	E_p	E_s	E_p	E_s	E_p	E_s
0.3			0.3	1.0				
0.4			0.395	0.989	0.4	1.0		
0.5			0.494	0.982				
0.6			0.58	0.96	.57	.95		
0.7			0.657	0.939				
0.8	0.8	1.0	0.730	0.912	.70	.87	0.8	1.0
0.9	0.89	0.99	0.794	0.882			.855	.950
1.0	0.951	0.957	.851	0.851	.804	.804	.893	.893
1.1	0.994	0.903	.899	0.817				
1.2	1.00	0.833	.937	0.781	.884	.737	.945	.788
1.3			.967	0.744				
1.4			.987	.705	.942	.673	.977	.698
1.5			.998	.665				
1.6			1.00	.625	.979	.612	.995	.622
1.7								
1.8					.997	.554	1.0	.555
1.9					1.0	.526	1.0	0.50
2.0					1.0	0.50		

The irrigation quality, as indicated by the performance category, for several of the tests of Table 1-4 (tests no. 7, 6, 2, 3), is dependent also on the required depth of application.

The values of the required depth of application for each category for the four tests are given in Table 1-6 in nondimensional expressions.

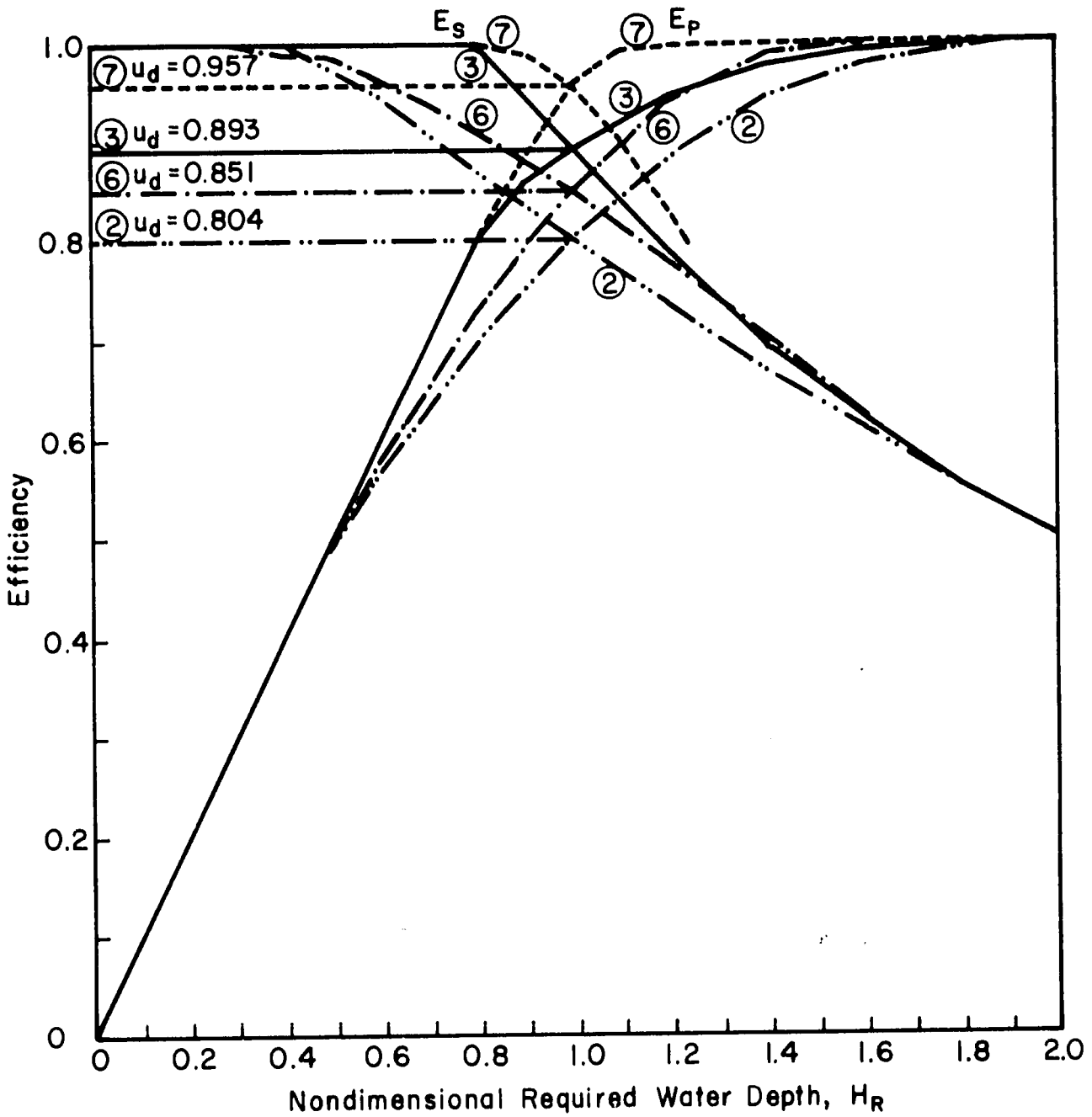


Fig. 1-7. Irrigation efficiencies for four border irrigation tests.

Table 1-6. Irrigation performance category for four irrigation tests.

Test No.	I	II	III	IV	V	VI
7	$0.8<H_R<1.24$	not possible $U_d>0.9$	$H_R>1.24$	$H_R<0.8$	not possible $U_d>0.9$	not possible $E_d>0.9$
6	not possible $U_d<0.9$	$0.9<H_R<1.15$	not possible $U_d<0.9$	not possible $U_d<0.9$	$H_R>1.15$	$H_R<0.9$
3	not possible $U_d<0.9$	$0.8<H_R<1.18$	not possible $U_d<0.9$	not possible $U_d<0.9$	$H_R>1.18$	$H_R<0.8$
2	not possible $U_d<0.9$	$H_R\neq1.0$	not possible $U_d<0.9$	not possible $U_d<0.9$	$H_R>1.0$	$H_R<1.0$

Analysis of the date in Table 1-6 and Fig. 1-7 enables an additional comparison between the various tests. The following are the main points.

(i) The best irrigation quality is achieved in test No. 7. The irrigation performance is satisfactory for a relatively wide range of the required depth. Even if the required depth is in the range of 80 to 124% of the average depth applied, the irrigation performance is satisfactory. The water distribution is highly satisfactory with

$$U_d = 0.957 \quad (UCC = 0.914)$$

(ii) Tests 2, 3 and 6 have low distribution uniformities so the water distributions should be improved. However, comparing tests 3 and 6, test No. 3 exhibits a better irrigation performance because U_d for test No. 3 is greater than that of tests No. 6 and 2, and the range of H_R, for which the irrigation performance is in category II is wider. This means

that the larger deviations in the required depth are permitted before E_p or E_s become unsatisfactory.

(iii) The irrigation performance of test No. 2 is very poor. Not only the distribution uniformity is low, but also for any slight deviation of the required depth from the average actually applied depth, either E_p or E_s will be unsatisfactory. If $H_R = 0.95$, $E_p < 0.8$. If $h_R = 1.05$ $E_s < 0.8$. In this case, both the distribution and the application depth should be improved.

4. General relationships between the border irrigation results and the operating conditions.

The evaluation of border irrigation as derived from the water distribution pattern requires the use of a model that relates the operating conditions of border irrigation to the irrigation results.

Such a model should be aimed to provide some or all of the following:

a. to predict the water distribution pattern so that the necessary and sufficient efficiencies can be derived;

b. when improvement of the border irrigation performance is required, to determine which operating condition is to be changed; and

c. to determine further what is the required change in the operating conditions and the expected improvement in the irrigation performance.

As a basis for a more detailed model, the general relationships and connections between the various operational conditions and the irrigation performance are described. In this description, only the qualitative relationships are defined and the emphasis is on the connections between the operating conditions and the irrigation performance. The basic model is described in Fig. 1-8 in the form of a flow-chart diagram relating the

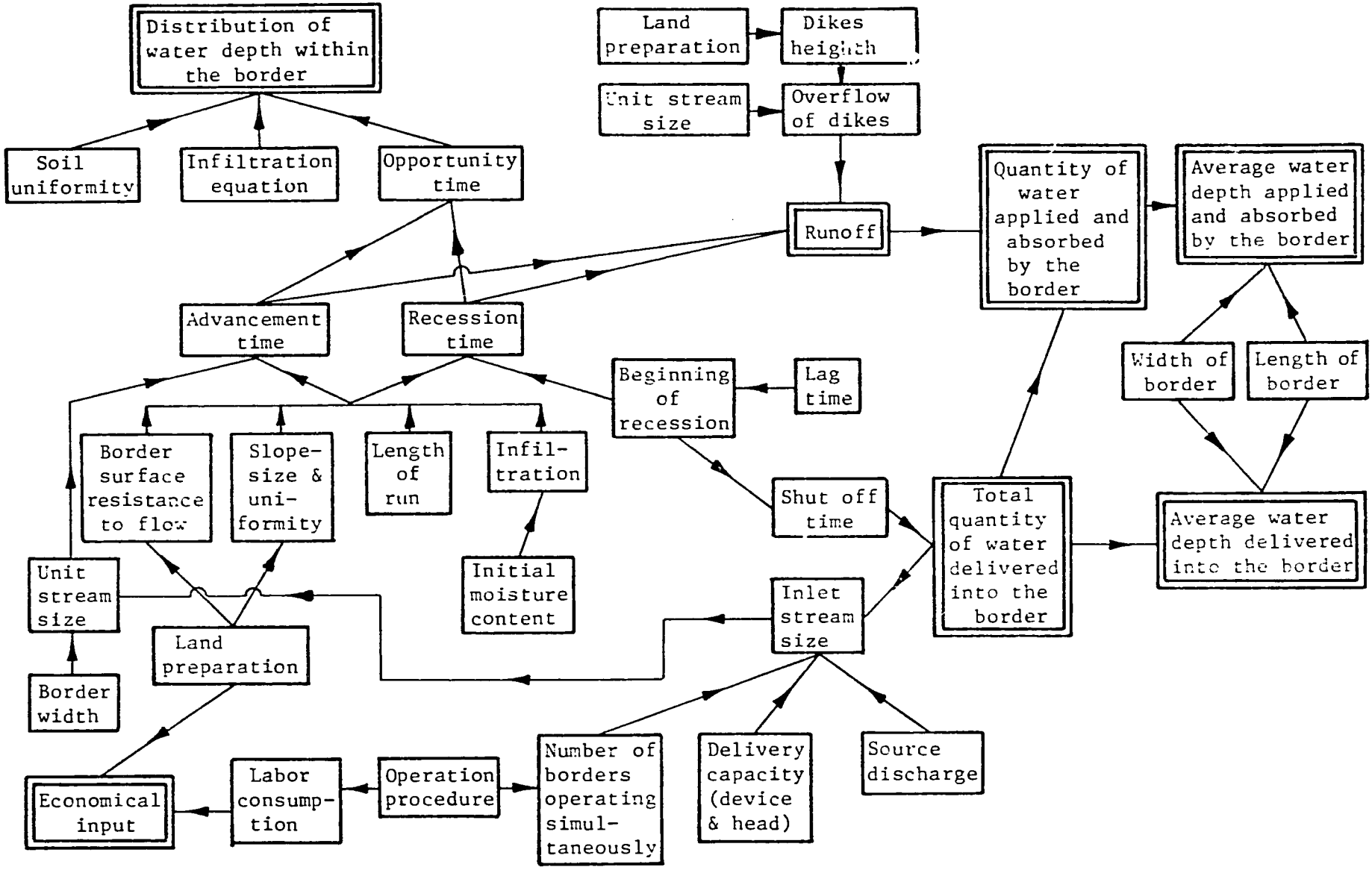


Fig. 1-8. Relationships among operating conditions and border irrigation performance.

connections between the various operating conditions without specifying quantitatively the expression and formulation of these connections. The model provides the irrigation results as a function of the operating conditions. The irrigation results, which are related to each other are water distribution within the border, average depth of application, total water quantity applied and absorbed by the border, total water quantity delivered to the border, and water losses outside the border by runoff.

The final results of the irrigation, which determine the irrigation performance, include the water distribution infiltrated into the border, the total quantity of water delivered onto the border, the total water applied and absorbed in the border, and the amount of runoff.

The final distribution of water depth within the border is a result of the opportunity time and infiltration equation. Nonuniformity of soil properties, mainly infiltration and moisture capacity, may also affect the distribution pattern. The opportunity time is a result of advance and recession times.

The total quantity of water delivered into the border is the product of the inlet stream and shutoff time. However, depending on the advance and recession, and to a smaller extent on overdike flow, part of the quantity delivered is lost outside the border through runoff. Consequently, the quantity of water applied and absorbed by the border is reduced.

Chapter 2

EVALUATION OF BORDER IRRIGATION BY FIELD TESTS

1. Introduction

Evaluation of border irrigation can be carried out in two major ways:

a. evaluation based on field tests that provide the data needed to estimate the final water distribution for a known set of operating conditions.

b. evaluation based on theoretical models which permit prediction of the final water distribution as a function of the various operating conditions.

This chapter deals with evaluation based on field tests. Most of the required data are directly measured during irrigation tests.

The information to be collected during irrigation tests can be determined from Fig. 1-8 considering two principles:

a. the measured factor should be reasonably easily measured, and

b. the measured factor should be as near as possible to the final irrigation results as described in Fig. 1-8.

These two principles can be further explained as follows. To evaluate the distribution of water depths, the three nearest factors are: soil uniformity, infiltration and opportunity time. However, direct measurement of opportunity time is impractical, therefore, the successive factors to be measured are advance and recession. To determine the total quantity of water delivered into the border, the two nearest factors are shutoff time and inlet stream size; both are easy to measure.

Evaluation of the quantity of water applied and absorbed within the border requires the measurement of the runoff, which can be carried out during field tests.

Determination of the water quantities in terms of depths requires the measurement of the dimensions of the border.

Finally, a field test for border evaluation includes direct data collection of the following:

infiltration depth	shutoff time
soil uniformity	inlet stream size
advance	runoff
recession	width and length.

2. Utilization of data measured during field tests.

The data that are obtained during field tests are utilized in the following steps:

a. Infiltration.

The cumulative infiltration is measured in several locations within the irrigated field (see Peri et al. (1979)). The relationship between the infiltrated water depth and the opportunity time can be assumed to be of the general form

$$z(t_{op}) = At_{op}^{B} \tag{2-1}$$

in which

$z(t_{op})$ = cumulative infiltrated water depth

t_{op} = opportunity time

A, B = constants with $0 < B < 1.0$

The measured values of z and t_{op} are plotted, for each test separately, to log-log scale. Generally, the several tests produce curves which are nearly parallel lines on the log-log scale, but the lines may be different in positions.

After plotting the various infiltration curves, the probable irregularities should be considered. Deviations from a straight line,

mainly in the lower part of the line, are likely to appear as a result of various factors (trapped air, open cracks, plow pan, measuring failures, etc.). Considering the deviations among the several infiltration lines, a representative line is plotted. This line is regarded as the typical infiltration line for the irrigated area. Its position is determined to reflect the actual amount of water that has been delivered into the border. Further description and demonstration is given in the following example.

b. Advance, recession and water depth.

For a given border (slope, surface roughness, width and length) and inlet stream, the advance of the water front is recorded. Since this measurement is fairly reliable, no repetitions are needed and one test for a single discharge is sufficient. However, if more tests are possible, the accuracy can be improved. When recession begins, the time of water disappearance is also recorded. The advance and recession are plotted together. At each point the opportunity time can be determined by:

$$t_{op_i} = t_{r_i} - t_{a_i} \tag{2-2}$$

in which

t_{op_i} = opportunity time at the ith point

t_{r_i} = time when water disappeared at the ith point

t_{a_i} = time when water front reached the ith point.

Using the typical infiltration curve, the water depth that has been infiltrated at each point, i, can be calculated by substituting the opportunity time:

$$y_{typ_i} = A(t_{op_i})^B \tag{2-3}$$

in which

y_{typ_i} = the infiltrated water depth at point i as determined by the typical infiltration curve.

The total volume of water applied and absorbed by the border is estimated by:

$$T_{est} = \sum_{i=1}^{p} x_i y_{typ_i} = A \sum_{i=1}^{p} x_i \, (t_{op_i})^B \qquad (2\text{-}4)$$

where

p = number of measuring points

x_i = length of the border represented by point i

c. Water quantities and adjustment of infiltrated water depths.

The total volume of water that is delivered into the border, D, is calculated by:

$$D = qt_{co} \qquad (2\text{-}5)$$

in which

q = the inlet stream size

t_{co} = the shutoff or cutoff time

With a direct measurement or estimation of the runoff, the volume of water that has been applied and absorbed by the border is given by:

$$T = D - W - EV = qt_{co} - W - EV \qquad (2\text{-}6)$$

in which

EV = volume of water evaporated during irrigation

In most practical cases, especially when the opportunity time is relatively short (less than four hours), EV is relatively small and can be neglected.

Theoretically, the volume of water infiltrated into the border, T, as determined by Eq. 2-6, should be equal to that calculated by Eq. 2-4,

T_{est}. Practically, however, the two estimates are different, most likely due to the position of the typical infiltration line.

Each water depth, y_{typ_i}, can be adjusted to the final water depth using a correction factor R:

$$y_i = Ry_{typ_i} \tag{2-7}$$

in which

y_i = the final infiltrated water depth at the ith point, used for the evaluation of the irrigation peformance

R = correction factor to adjust for the differences between T_{est} and T

The correction factor is given by:

$$R = \left(\frac{P_1T + P_2T_{est}}{P_1 + P_2}\right) x \frac{1}{T_{est}} \tag{2-8}$$

in which

P_1, P_2 = weighting factors that represent the reliability and accuracy of the determination of T and T_{est}, respectively.

Usually, the estimation of the applied quantity by Eq. 2-6 is a more reliable figure than T_{est}, so that $P_1 > P_2$ and R is often calculated with $P_2 = 0$ which gives:

$$R = \frac{T}{T_{est}} \tag{2-9}$$

The correction factor is used to adjust the estimated infiltrated depths. Also, the typical infiltration curve can be readjusted by the factor R.

With the final infiltrated water depths at p points along the border and with the known runoff, the water distribution can be plotted and the irrigation performance efficiencies can be determined.

Example

Given a border of 320 m length. The cumulative infiltration was measured at four different locations within the border as given in Table 2-1 and Fig. 2-1.

The advance and recession were measured at 16 points, 20 m apart (the two extreme points were located 10 m from the border ends) and are given in Table 2-2 and Fig. 2-2.

The water depths infiltrated into the soil are calculated on the basis of the typical infiltration curve and are given in Table 2-2.

The total volume of water applied and absorbed by the border as esimated by the typical infiltration line is (Eq. 2-4):

$$T_{est} = 20 \Sigma y_{typ_i} = 20m \ (1998 \text{ mm}) = 40 \text{ m}^3/\text{m}.$$

The inlet discharge was 37 m^3/hr/m and the shutoff time was 90 minutes. The runoff was estimated at 7 m^3/m. The evaporation during irrigation can be neglected.

The volume of water absorbed by the border, from water quantity measurements, is (Eq. 2-6)

$$T = 37 \cdot \frac{90}{60} - 7 = 48.5 \text{ m}^3/\text{m}$$

Assuming that the water quantity measurements are reliable and represent accurate information, the correction factor, R, is (Eq. 2-8)

$$R = \frac{48.5}{40.0} = 1.21$$

The corrected and final water depths, y_i, are given in Table 2-2.

The average depth applied and absorbed by the border is

$$\bar{y} = \frac{T}{L} = \frac{48.5}{320} = 0.1515 \text{ m} = 151.5 \text{ mm}$$

Table 2-1. Cumulative infiltration at four different locations for example border irrigation test.

Time (min) / Location No.	Water depths (mm)												Cumulative infiltration equation
	1	2	5	10	30	50	80	100	200	400	500	800	
1	5.5	8.2	16.2	25.7	53.7	75.6	103.6	120.3	191.4	304.6	353.7	484.6	$5.5\ t^{0.670}$
2	6.3			29.5		86.6		137.8			405.2	555.2	$6.3\ t^{0.670}$
3	4.9			22.9		67.4		107.2			315.1	431.8	$4.9\ t^{0.670}$
4	4.5			21.0		61.9		98.4	156.6				$4.5\ t^{0.670}$
typical				24.8		72.9		116.0			340.9	467.0	$5.3\ t^{0.670}$

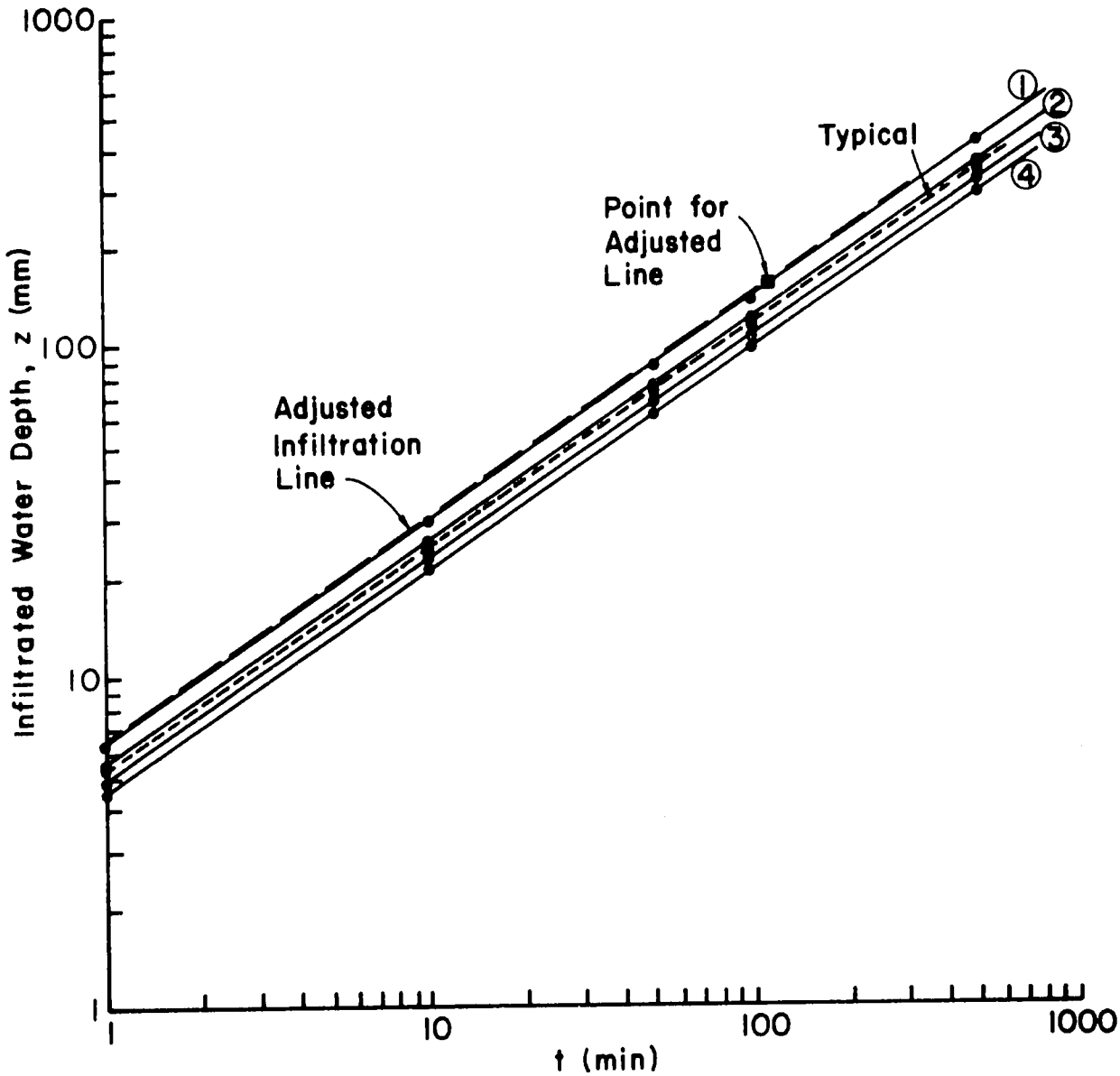


Fig. 2-1. Cumulative infiltration for four different locations for example border irrigation test.

Table 2-2. Advance, recession and opportunity time, for example border irrigation test.

Distance (m) / station / Factor	10 1	30 2	50 3	70 4	90 5	110 6	130 7	150 8	170 9	190 10	210 11	230 12	250 13	270 14	290 15	310 16
Advance time (min)	1	2	3	4	7	11	17	25	32	40	49	60	70	83	95	109
Recession time (min)	100	112	122	130	137	145	150	153	157	161	165	169	172	175	177	179
Opportunity time (min)	99	110	119	126	130	134	133	128	125	121	116	109	102	92	82	70
y_{typ_i} (mm)	115.2	123.6	130.3	135.4	138.2	141.1	140.3	136.8	134.6	131.7	128.1	122.8	117.5	109.6	101.5	91.3
y_i (mm) ($y_{typ_i} \times R$)	139.8	150.0	158.1	164.3	167.7	171.2	170.3	166.0	163.4	159.8	155.5	149.0	142.6	133.0	123.2	110.8

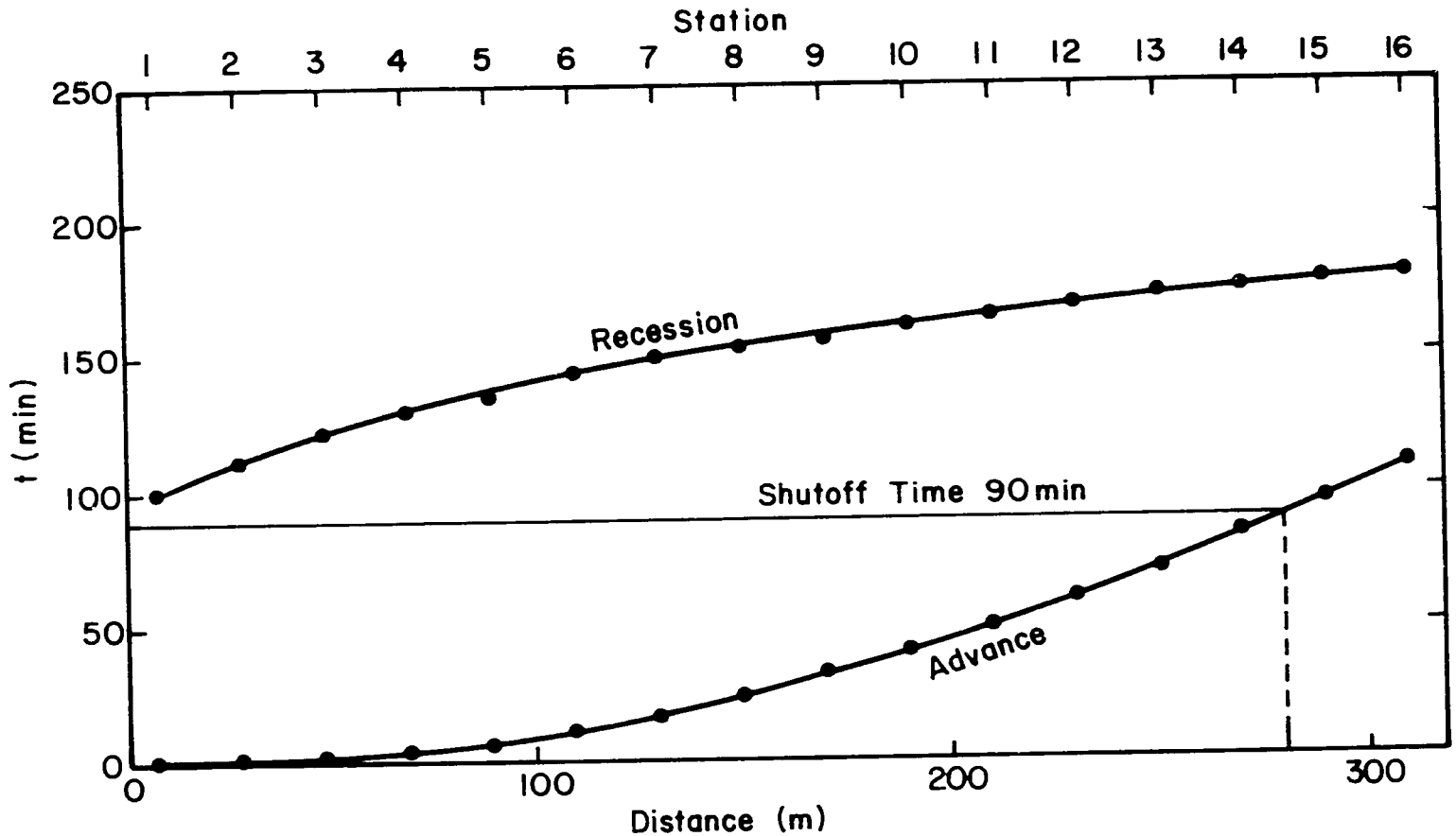


Fig. 2-2. Advance and recession curves for example border irrigation test.

The final infiltration equation is obtained by multiplying the typical equation by R to give

$$z = 6.41\ t_{op}^{0.670} \tag{2-10}$$

This equation describes the adjusted infiltration line of Fig. 2-1.

The actual depth infiltrated at each station is obtained from the infiltration opportunity time for the station and Eq. 2-10. These values are shown in Table 2-2 and on Fig. 2-3.

With these data the various parameters that describe the irrigation performance can be calculated. The delivery efficiency, E_d, is

$$E_d = \frac{T}{D} = \frac{48.5}{55.5} = 0.874$$

The distribution uniformity is obtained from F, the volume of deficit (or excess) of infiltrated water in relation to the mean depth applied. Thus

$$F = \sum_{i=1}^{p} x_i\ (y_i - \bar{y}) \quad \text{when } y_i > \bar{y} \tag{2-11}$$

From Table 2-2, using stations 3 to 11 and x_i = 20 m, F = 2.25 m^3/m and the distribution uniformity is

$$U_d = \frac{T - F}{T} = \frac{48.5 - 2.25}{48.5} = 0.954$$

The deep percolation efficiency, E_p, and the storage efficiency, E_S, are dependent upon the requirement, h_R. When G is the volume of deficit irrigation and h_G is the mean deficit as a depth,

$$G = \sum_{i=1}^{p} x_i(h_R - y_i) \quad \text{when } h_R > y_i \tag{2-12}$$

$$h_G = \frac{G}{L} \tag{2-13}$$

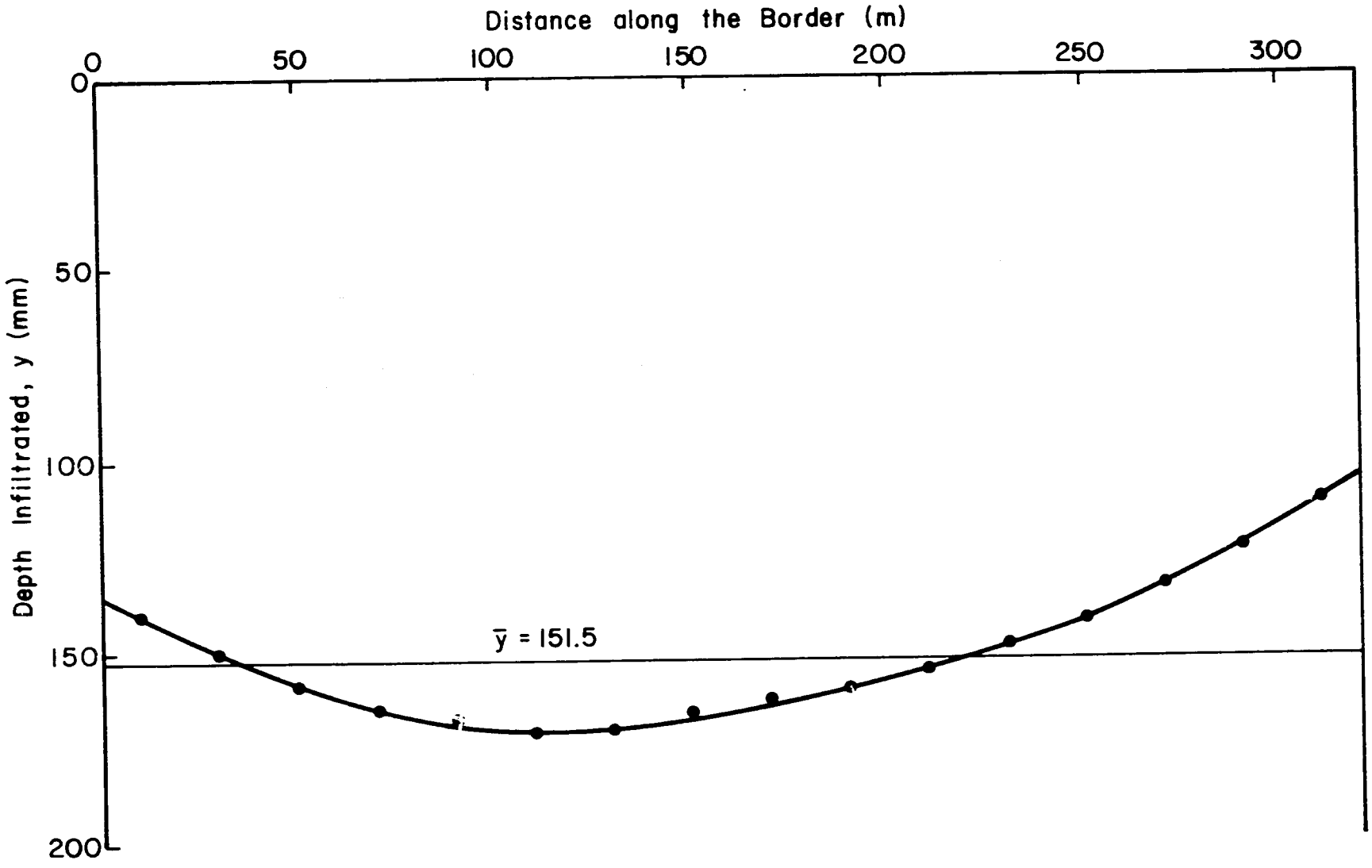


Fig. 2-3. Final subsurface water distribution for example border irrigation test.

$$E_P = \frac{h_R - h_G}{\bar{y}} \tag{2-14}$$

and

$$E_S = \frac{h_R - h_G}{h_R} \tag{2-15}$$

Table 2-3 and Fig. 2-4 show the efficiencies for various requirements.

Table 2-3 Irrigation efficiencies for varying required depths of water application for example border irrigation test.

h_R mm	G m^3/m	h_G mm	E_P	E_S
50	0	0	0.330	1.0
70	0	0	0.462	1.0
100	0	0	0.660	1.0
120	0.18	0.6	0.788	0.995
130	0.52	1.6	0.847	0.988
140	1.06	3.3	0.902	0.976
150	2.03	6.4	0.948	0.958
160	3.56	11.1	0.982	0.930
170	5.94	18.6	0.999	0.891
180	9.11	28.5	1.00	0.842
200	15.51	48.5	1.00	0.758

The irrigation performance can be evaluated and required improvements can be defined.

The distribution uniformity, U_d, is found to be satisfactory with $U_d = 0.954$ (UCC = 0.908).

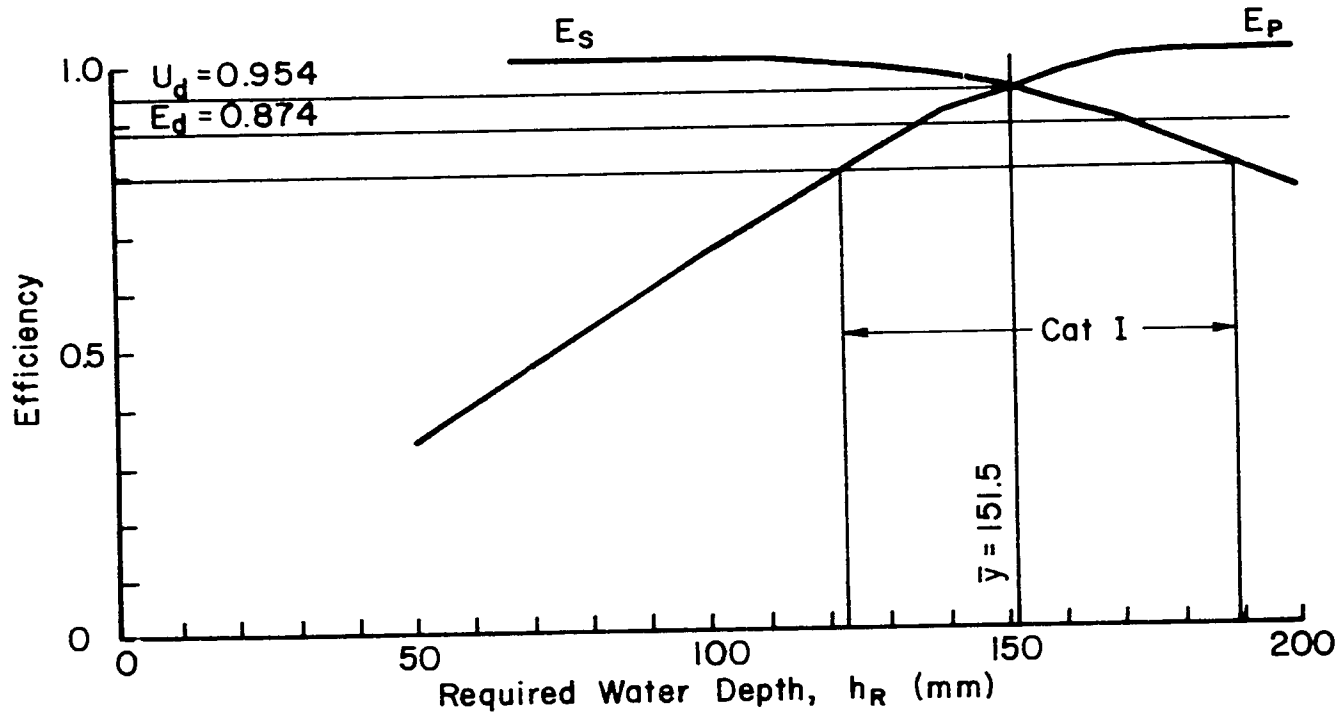


Fig. 2-4. Irrigation efficiencies for example border irrigation test.

The delivery efficiency is $E_d = 0.874$ which is also satisfactory, however, some reduction in runoff should be considered.

The other efficiencies are analyzed with the required depth of application. If, for example, an efficiency value for E_p and E_s is considered satisfactory when $E_p > 0.8$; $E_s > 0.8$, the irrigation performance is satisfactory (category I), when the required depth is within the range: $123 \text{ mm} < h_R < 188 \text{ mm}$. For $h_R > 188$ mm, the irrigation is inadequate and the depth of application should be increased. For $h_R < 123$ mm, the irrigation is excessive and the depth of application should be decreased.

Chapter 3

IMPROVEMENT OF BORDER IRRIGATION PERFORMANCE BY FIELD TESTS

1. Introduction.

The evaluation of border irrigation by field tests requires the estimation of the infiltration equation and the advance and recession for a specific inlet stream and border parameters (slope, length, surface). Based on the advance, recession and infiltration, the water distribution pattern is determined and the performance parameters are calculated to determine the irrigation performance and the need for improvements. From Fig. 1-8, the operating conditions that are to be changed can be determined. However, determination of the magnitude of change for each operating condition requires further analysis. This analysis, when based on field tests, comprises two stages: utilization of the field test data already available, and implementation of the recommended changes for further field tests.

2. Identification of land nonuniformity in border irrigation.

Field evaluation of border irrigation depends mainly on the border parameters (dimensions, slope, surface roughness), advance, recession, infiltration, and inlet stream size and duration.

Improvement of the border irrigation performance is related to changes in the operating conditions. However, the irrigation performance can be affected by factors which practically cannot be changed. Identification of these factors is essential as the first step in analyzing the irrigation performance and determining the required improvements. Commonly, the factors associated with this problem are nonuniform infiltration and longitudinal slope along the border. The presence and the effect of these

factors can be generally identified by irregularities in the advance and recession curves, that is, distortions in the normal shape of advance and recession. Typical changes in the advance and recession, resulting from changes in infiltration and slope, are presented in Table 3-1 and shown in Fig. 3-1.

Table 3-1. Typical changes in advance and recession resulting from changes in infiltration or slope.

Index No.	Changes in infiltration or slope in a specific portion of the border	Effect on advance and recession curves	
		recession	advance
1	faster infiltration	flatter curve (faster)	steeper curve (slower)
2	slower infiltration	steeper curve (slower)	flatter curve (faster)
3	flatter slope	steeper curve (slower)	steeper curve (slower)
4	steeper slope	flatter curve (faster)	flatter curve (faster)
5	depression or peak	steeper noncontinuous curve	steeper noncontinuous curve

Faster infiltration and steeper slope have the same effect on the recession, causing it to be faster (a flatter curve). Also, slower infiltration and flatter slope have the same effect on the recession, causing it to be slower (steeper curve). Faster infiltration and flatter slope have the same effect on the advance, causing it to be slower. Also, slower infiltration and steeper slope have the same effect on the advance, causing it to be faster (flatter curve).

Consequently, a simultaneous study of the irregularities in the advance and recession curves can indicate the possible major factors (infiltration or slope) causing the irregularities.

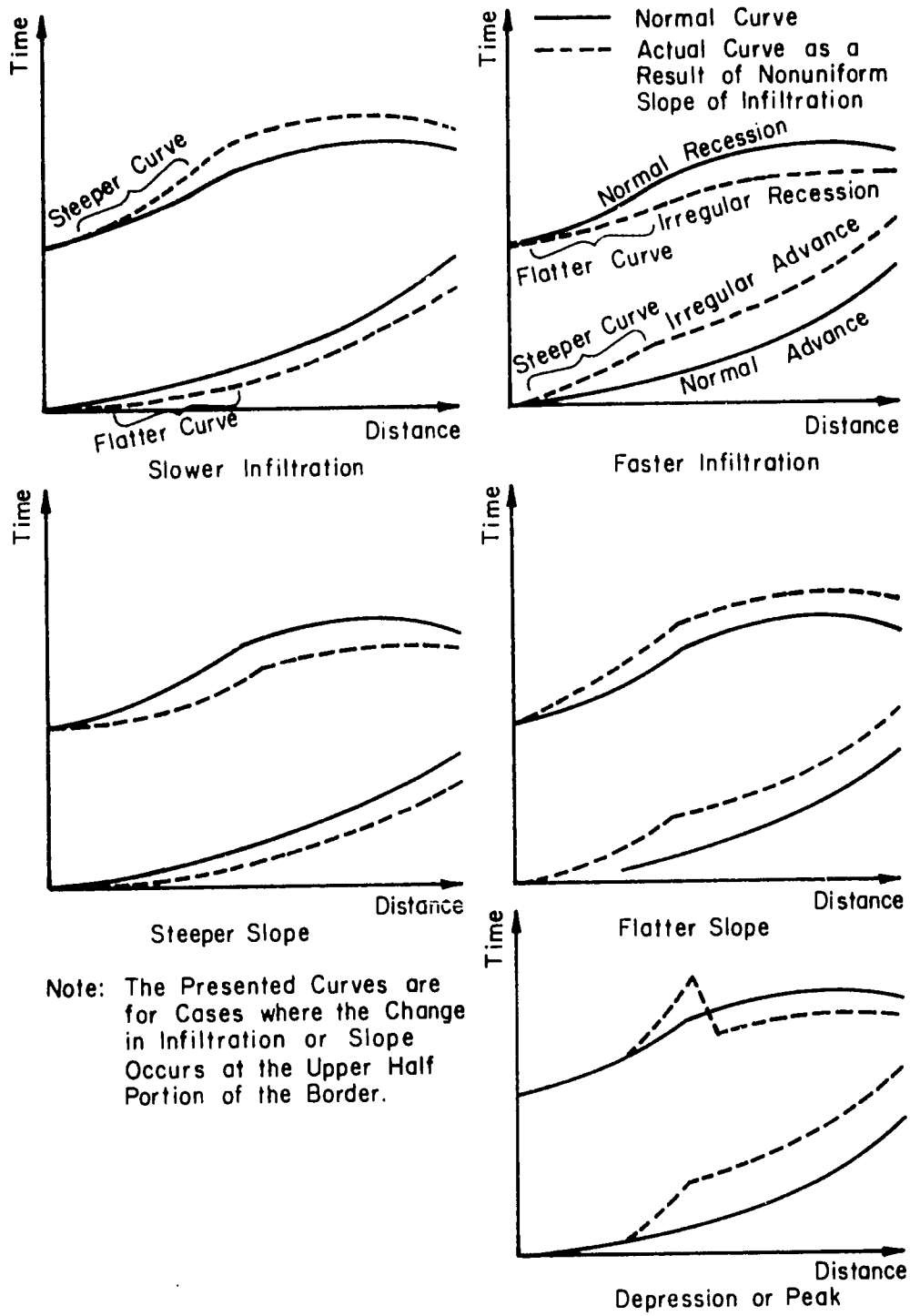


Fig. 3-1. Effects of deviation in infiltration and slope on advance and recession.

Identification of land nonuniformity by the irregularities in recession and advance is given in Table 3-2.

Table 3-2. Identification of nonuniform infiltration or slope by irregularities in the advance and recession curves.

Irregularity in advance curve \ Irregularity in recession curve	Steeper portion (slower recession in some portion of the border)	Flatter portion (faster recession in some portion of the border)
Steeper portion (slow advance in some portion of the border)	flatter land slope	faster infiltration
Flatter portion (faster advance in some portion of the border)	slower infiltration	steeper land slope

Using Fig. 3-1 and Table 3-2, the nonuniformity in infiltration or slope along the border can be detected by irregularities in the advance and recession curves. For existing border irrigation systems, very little can be done to overcome this nonuniformity. However, it should be considered in the evaluation and improvement of the border irrigation performance.

3. Determining and checking of border irrigation improvements.

The performance of border irrigation as determined by field tests is dependent on five main factors:

a. the shape of the advance curve,

b. the shape of the recession curve,

c. the cutoff time,

d. the lag time - the time between cutoff and recession at the head end of the border, and

e. the physical properties of the border, including length, slope, infiltration, surface roughness.

From Fig. 1-8, it can be seen that the advance curve is a function of the border physical parameters (length, slope, surface roughness and infiltration) and the unit stream size. For given border physical parameters, the advance curve is a function of the unit stream size only. The smaller the unit stream, the steeper is the advance curve (as shown in Fig. 3-1). The shape of the recession curve is primarily a function of the border physical parameters so that the starting point of the recession is the main factor in determining the recession curve.

Based on these assumptions, and with a given infiltration, and specific advance and recession curves, the improvement of the irrigation performance is related to changes in inlet unit stream size, cutoff time, and length of border as changing slopes, surface roughness or infiltration is generally impractical.

The procedure for determining the changes of the unit stream size depend upon if the length of the border can be changed.

The steps in the determination of the required changes are as follows:

a. Determine the time at which the recession should begin. It is generally recommended that the recession begin when the upper end of the border has received the required depth of application, h_R. From the typical infiltration equation, the desired opportunity time is:

$$t_{op} = \left[\frac{h_R}{A}\right]^{1/B} \tag{3-1}$$

Therefore the desirable time for recession to begin at x = 0 is

$$t_{ro} = \left(\frac{h_R}{A}\right)^{1/B} \tag{3-2}$$

b. Plot the recession curve obtained by the field tests, but starting at time $t = t_{ro}$.

c. When the length of the border is constant and cannot be changed, determine the required change in the unit stream flow as follows (Fig. 3-2):

if $t_{aL} < \Delta t_r$ decrease the unit stream size

if $t_{aL} > \Delta t_r$ increase the unit stream size

where t_{aL} = the time required for the advancing water front to reach the end of the border, and Δt_r = the time interval between the recession at the lower and upper ends of the field.

This is illustrated in Fig. 3-2. The qualitative effects of the unit stream size (through the advance time) is given in Table 3-3.

Table 3-3. Qualitative effect of the unit stream size on the border irrigation performance.

Relative stream size	Time relations	Uniformity	Deep percolation	Runoff	Deficiency	Location of minimal opportunity time (minimal infiltered depth)
too large	$t_{aL}<\Delta t_r$	poor	large	large	none	upper end
too small	$t_{aL}>\Delta t_r$	poor	low	low	high	lower end
correct	$t_{aL}=\Delta t_r$	good	low	moderate	none	lower and upper ends

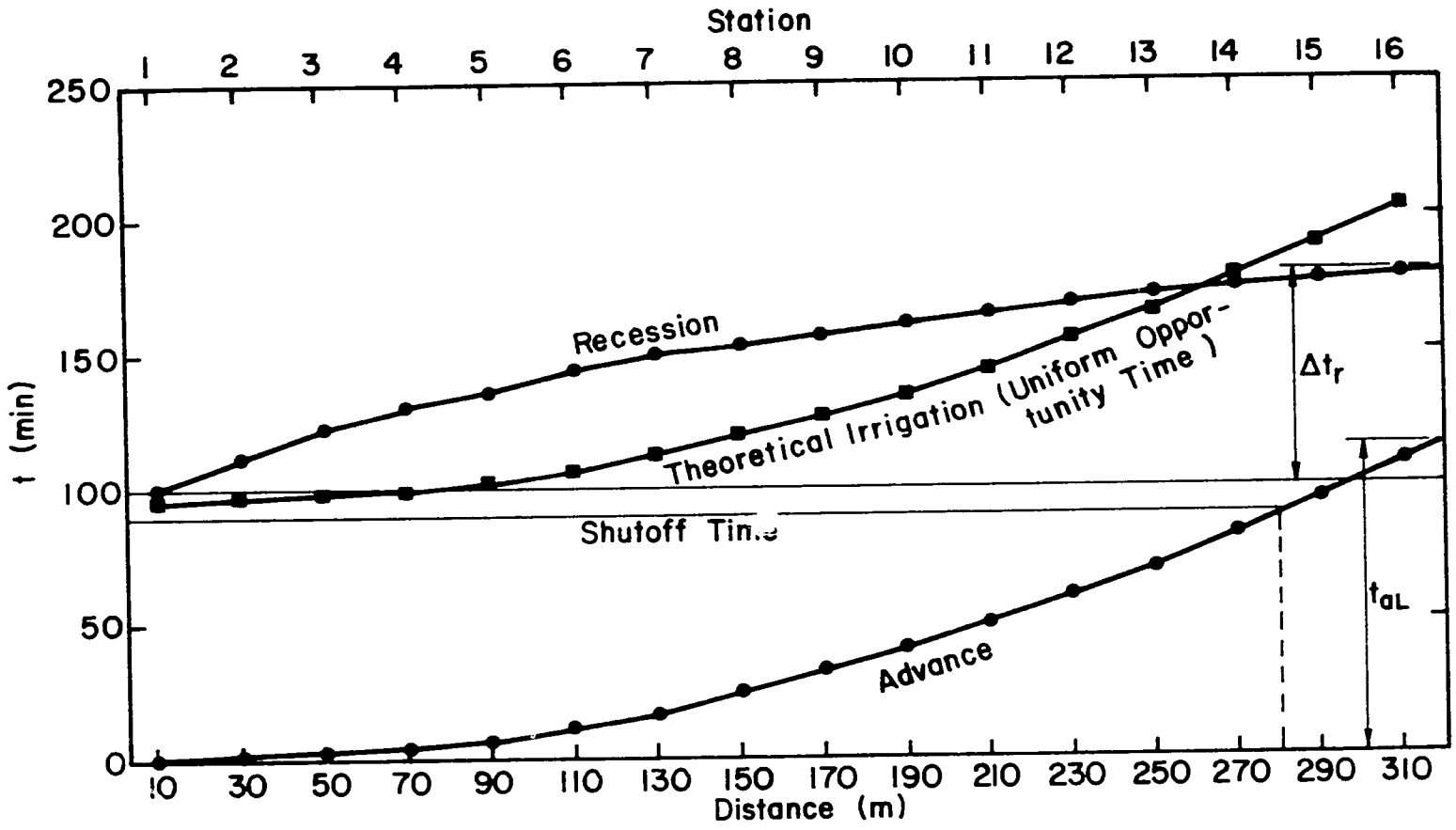


Fig. 3-2. Time parameters for advance and recession curves.

4. Determination of the New Unit Stream.

The new unit stream (increased or decreased from the initial advance curve) is first approximated and then tested by field tests. The approximation of the new unit stream is as follows (Fig. 3-3).

a. Draw the recession curve starting at the required time, t_{ro}, and approximate the curve past x = L, the end of the border.

b. At x = L, mark a point distance t_{ro} below the recession curve. This point is the desirable time for the advance front to reach the end of the border.

c. From the origin, draw a convex curve through the point established in b. If the flow is being reduced this curve will be more convex than the original advance curve, if the flow is being increased, the curve will be flatter.

d. The volume of water included between the recession and advance is calculated from x = 0 to the point where the advance intersects the recession. The calculation of the volume can be carried out by calculating the opportunity time at p discrete points:

$$D = A \sum_{i=1}^{p} x_i t_{op_i}^{B} \tag{3-2}$$

The unit stream, q, can be estimated from:

$$q = \frac{D}{t_{co}} \tag{3-3}$$

in which:

t_{co} = cutoff time

The cutoff time is given by:

$$t_{co} = t_{ro} - t_{\ell} \tag{3-4}$$

in which:

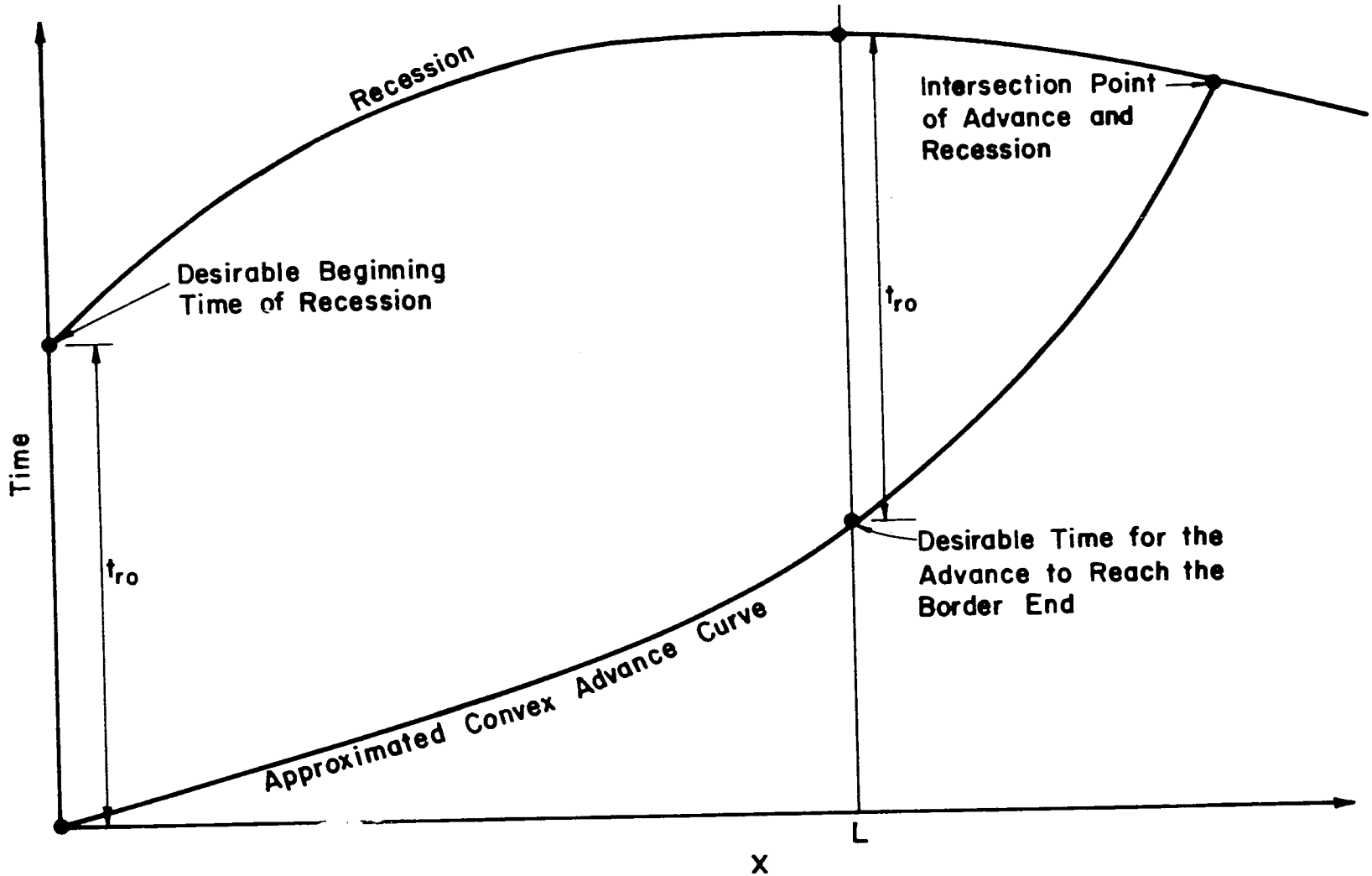


Fig. 3-3. Advance and recession parameters for the approximation of the inlet stream size.

t_ℓ = the lag time between the shutoff of the stream and the beginning of the recession.

The lag time is a function of the stream size, slope and surface roughness of the border. According to the Soil Conservation Service (1974) this relationship is

$$t_\ell = \frac{q^{0.2}\, n^{1.2}}{2\, S_o^{1.6}\, CU^{1.2}} \tag{3-5}$$

where:

n = Manning's roughness coefficient

S_o = the border slope, and

CU = units coefficient (ft-sec CU = 1.49, m-s CU = 1.0)

Combining Eqs. 3-3, 3-4 and 3-5 results in an equation for q of the form

$$q = \frac{D}{t_{ro} - \dfrac{q^{0.2}\, n^{1.2}}{2\, S_o^{1.6}\, CU^{1.2}}} \tag{3-6}$$

For particular values of t_{ro}, D, n and S_o, Eq. 3-6 can be used to iterate for a value of q. Once q has been found, t_ℓ can be obtained from Eq. 3-5.

<u>Example</u>

Given a border that is 200 m long, has a slope of $S_o = 0.0026$, an n = 0.15, and a desired opportunity time of t_{ro} = 48 min (2880 s), find the flow rate and lag time when the total flow onto the border should be 21 m^3/m.

If the Right-Hand-Side (RHS) of Eq. 3-6 is evaluated with q = 0, the resulting Left-Hand-Side (LHS) is q = 0.00729 $m^3/s/m$. Replacing this value into the LHS and iterating, results in successive value of

$q = 0.00802, 0.00804$ and $0.00804\ m^3/s/m$. Substituting into Eq. 3-5 results in $t_{ro} = 267$ or approximately 4.5 min.

5. Retesting in the field.

After the unit stream size has been approximated, a field test should be carried out using:

a. the new inlet stream size; and

b. the cutoff time as calculated by Eq. 3-4 or slightly shorter than the required time, t_{ro}.

During the field test, the advance and recession are measured to check the advance due to the new inlet stream size. The evaluation process can be repeated on the new data. The advance can be improved further by increasing or decreasing the flow.

6. Simultaneous determination of unit stream and border length.

If the length of the border can be changed, the determination of the unit stream size is more flexible since it can be associated with the determination of the border length. Simultaneous determination of unit stream and border length can be carried out so that the opportunity times at the upper and lower ends are close enough to the required opportunity time, t_{ro}.

Improving the border irrigation when both the unit stream size and the length can be changed include the following procedure: (Fig. 3-4)

a. Plot the advance curve as obtained from the field tests (curve 012).

b. Plot the recession curve with shape and slope as obtained by the initial field tests, but starting at time $t = t_{ro}$.

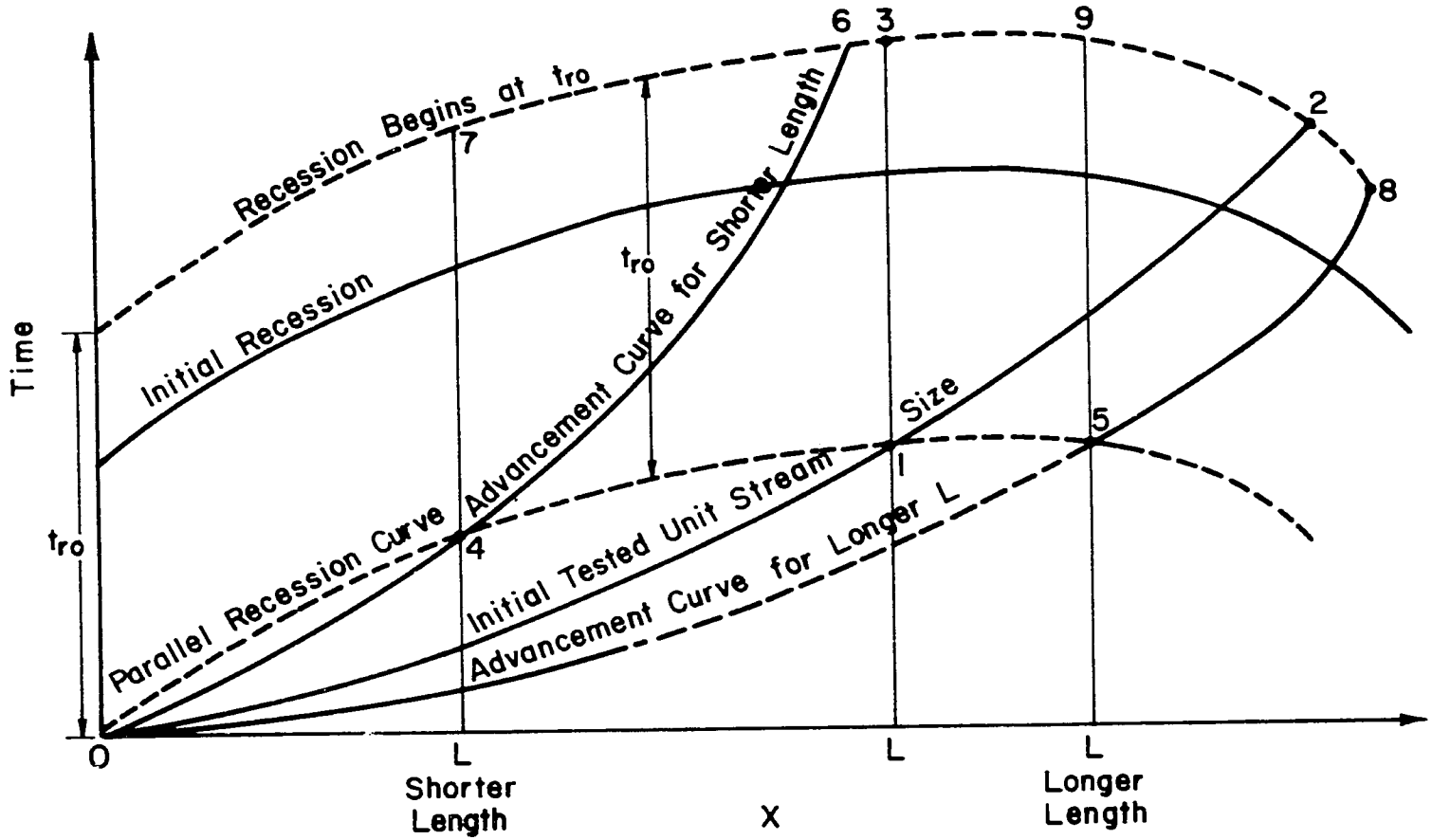


Fig. 3-4. Procedure for the determination of inlet stream size and border length.

c. Draw a curve parallel to the recession curve at a distance of t_{ro} lower (curve 0415).

d. Determine the length of the border where the parallel recession curve meets the advance curve for the initial stream size (point 1). Check the deep percolation (the area between the curves of steps a and c) and runoff (area 1231) for this case.

e. Choose other possible border lengths, one shorter and one longer than that obtained for the initial unit stream size. For each length determine the most desirable time for the advance front to reach the border end (points 4 and 5 on the curve of step c).

f. Through these intersection points and the origin draw convex curves representing approximate advance curves for other unit stream sizes. The unit stream size can be estimated for each curve by means of Eq. 3-6. Theoretical evaluation of the new length-unit stream size combinations can be carried out by analyzing the deep percolation (area confined between recession and advance curves having the common points 0 and 4 and 0 and 5), and the runoff (area confined between points 4, 6, 7 and 5, 8, 9).

g. The three possible combinations of unit stream size and length for the initial stream size (longer length with higher stream size and shorter length with smaller unit stream size) can be compared to each other to select the most favorable operating conditions.

h. Run field tests with the selected length and unit stream size.

The process of improving border irrigation on the basis of field tests is presented in the flow chart of Fig. 3-5.

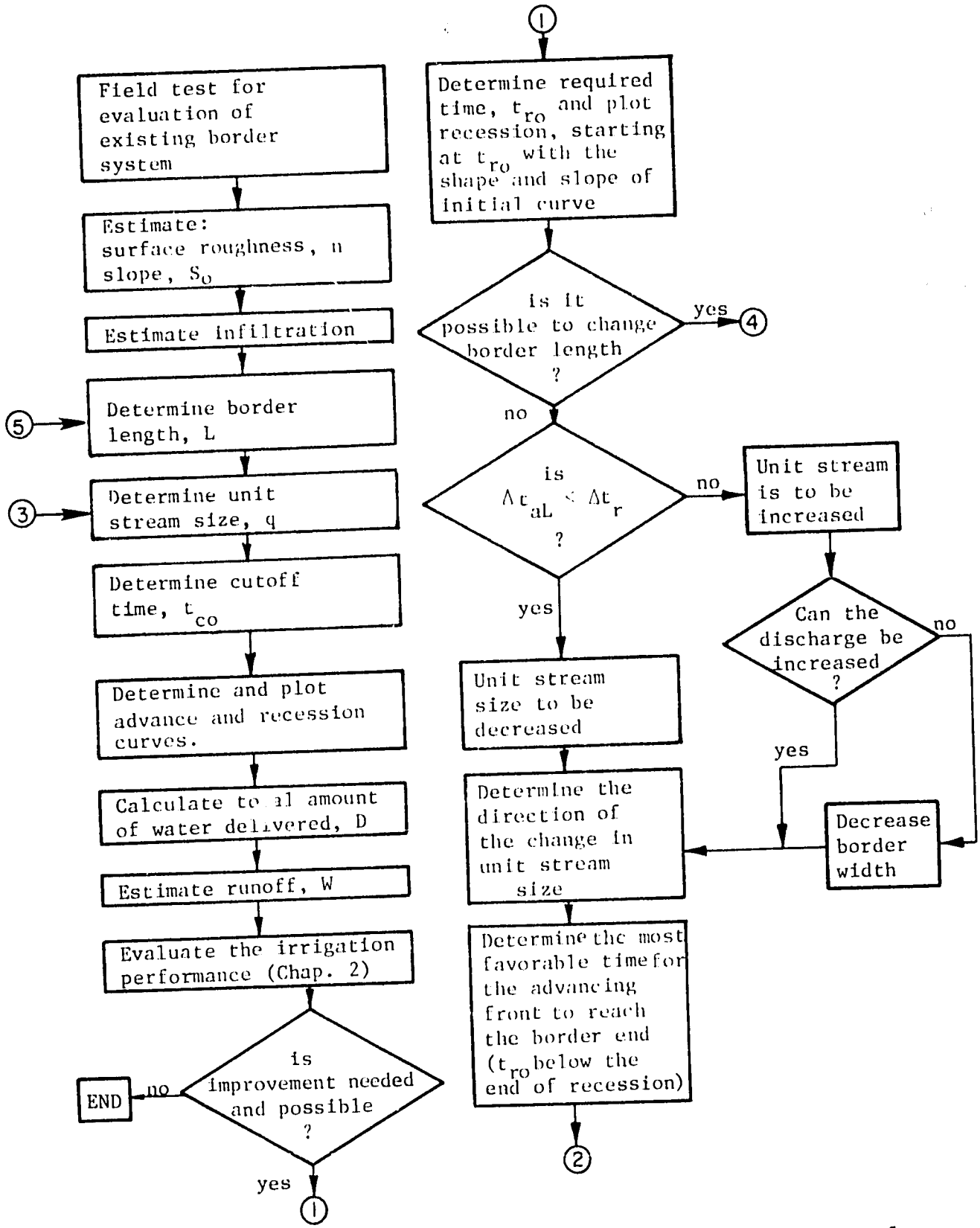


Figure 3-5. Process for improving border irrigation on the basis of field tests.

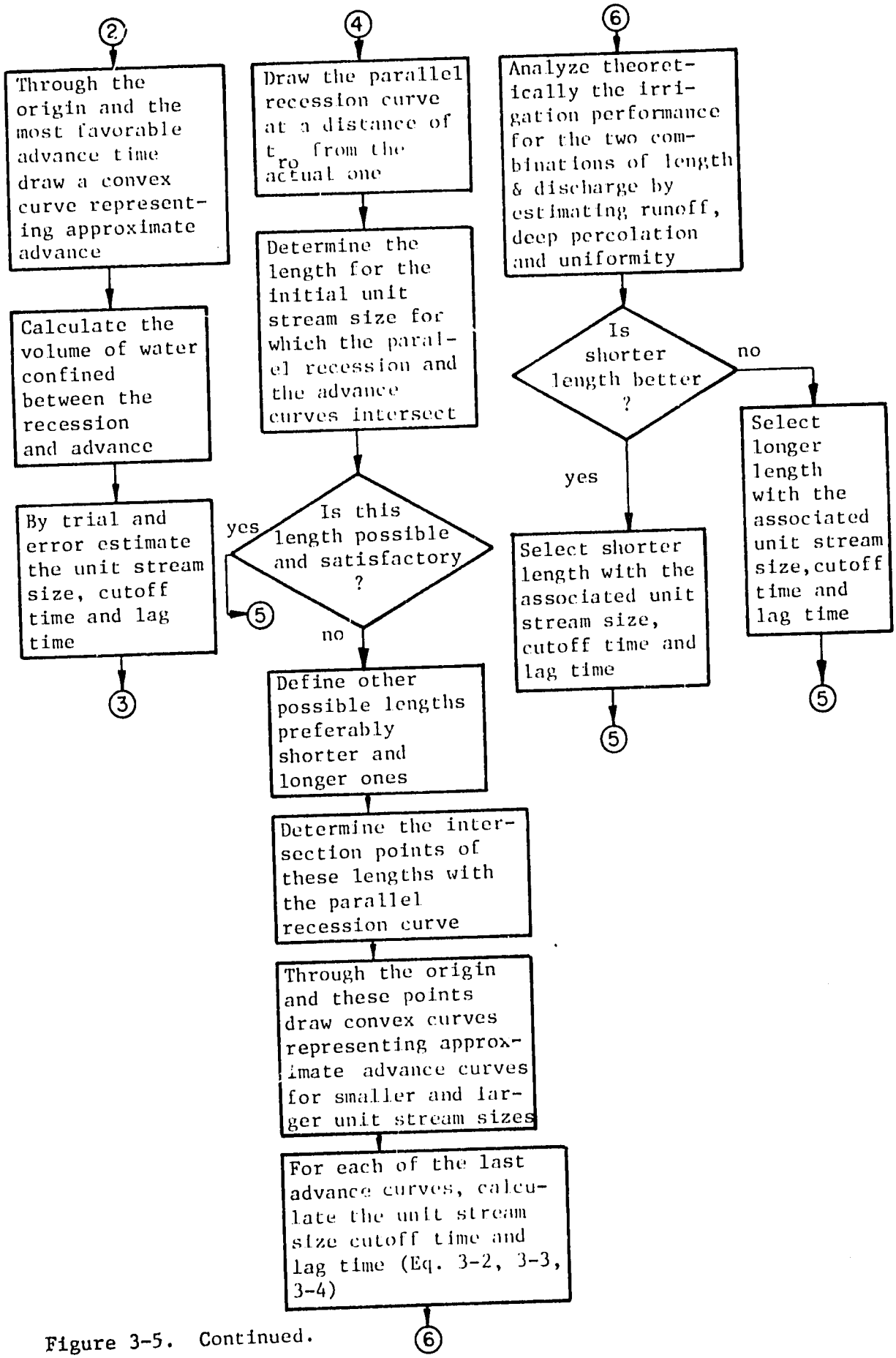


Figure 3-5. Continued.

Chapter 4

MODELS FOR THE EVALUATION AND IMPROVEMENT OF BORDER IRRIGATION PERFORMANCE

1. Introduction.

Theoretical models for the evaluation and improvement of border irrigation performance enable one to predict the water distribution, and losses for a given set of conditions, without direct field measurements. The application of a theoretical model requires only limited amounts of preliminary field work to predict the irrigation performance for a wide range of parameters and variables.

Although there are several different models available, the input data are practically the same for every model and can be divided into two groups:

a. basic data which include:

(i) infiltration characteristics

(ii) border slope

(iii) border length

(iv) border surface roughness

b. operating variables which are:

(i) inlet stream size per unit of border width

(ii) the cutoff time.

The output from each model is different, however, the models can be classified according to their output as follows:

a. models that provide the advance function only

b. models that provide the recession function only

c. complete models that provide the advance, recession, runoff and surface water depths.

With any of the models, an infiltration function is used to estimate the infiltrated water depths over the border for the opportunity time as calculated from the advance and recession.

2. The general process of model application

To use any theoretical model for the evaluation and improvement of border irrigation, the steps are basically the same.

a. Collect the basic data. These data are directly measured by field tests.

b. Record the operating variables (inlet stream size and cutoff time) for the system under study.

c. According to the model output, obtain advance and/or recession curves. With incomplete models, the other required time-related function must be determined by field experiments or by other models.

d. With the model-predicted opportunity time, the infiltrated depths at various points along the border are calculated, and estimation of water losses outside the irrigated field is carried out.

e. The infiltrated water depths are expressed in a form of water distribution from which the irrigation performance is evaluated.

f. The required improvements are determined on the basis of the established irrigation performance (change in applied quantities and/or uniformity). Accordingly, a change of any of the operating variables is carried out and the process of obtaining a new irrigation performance is repeated.

g. For a range of possible values of the operating variables, the irrigation performance can be defined as a function of the inlet

stream size and cutoff time for a given set of basic data. From these relationships the change in operating variables can be determined so that the irrigation performance is improved.

It should be noted that no models provide an explicit value for the irrigation performance for a set of basic data and operating variables. Therefore, the model is used repeatedly with varying input data, to provide the irrigation performance for many sets of input data. No direct determination of the operating variables can be made in order to obtain a particular irrigation performance.

Under specific circumstances, it might happen that several of the basic data parameters can serve as an operating condition. That is, they can be subject to changes. These parameters are generally the border length and slope. Usually when evaluating existing borders it is impractical to change the length and the slope. However, in some cases, these changes may be highly effective in improving the irrigation performance.

3. Available models.

There are several available models for the complete or partial (advance or recession only) solution of border irrigation. Generally speaking, these models can be classified as follows:

a. empirical or semi-empirical models. These models are used primarily to evaluate the advance function. A typical power function is assumed where its constants have been determined empirically. It is common also to use mass conservation principles to derive the function constants.

b. mathematical models for the solution of the continuity and momentum equation of open channel flow. These models are used

for a complete solution of the border irrigation process. They are comprised of numerical solutions of the partial differential equations which with the proper initial and boundary conditions can describe each of the phases of border irrigation.

c. mass conservation models that use the principle of water volume balance, based on a simple water distribution shape, (surface and subsurface) and simple equations to estimate water depths at key points of the border (hydraulic normal depth for the surface depths and infiltrated depths for the subsurface depths).

Some of the most prominent available models are listed in the Appendix. In selecting a model for use in the evaluation and improvement of border irrigation, a few major points should be considered. The mathematical models can yield relatively good accurate results, essentially as accurate as the input data with which they are supplied. These models are flexible and can be used to solve a wide variety of cases. However, they require extensive programming and computer time. The empirical models are relatively simple but they require extensive field work and their accuracy is dependent on the accuracy of measurement, uniformity and repeatability of the measured phenomenon (advance, recession, runoff). The mass conservation models are simple to handle, and can be carried out manually. Since they are based on approximations, their accuracy is expected to be less than that of the mathematical models. However, practically the level of accuracy of the mass conservation models is acceptable under most conditions.

A summary of the main features of these three kinds of models is given in Table 4-1. The flexibility of the various methods to handle various conditions is also mentioned.

Table 4-1. Qualitative comparative evaluation of the three kinds of models for border irrigation.

Kind of Model \ Feature	Overall Simplicity	Programming	Computer Time	Expenditure of Time	Field Tests	Accuracy	Flexibility and Generality
A. Empirical	Medium	Minimal	Minimal	High due to field and lab tests	Many	Poor as a result of soil hetero-genity	Poor
B. Mathematical	Compli-cated	Extensive	High	High	Minimal	High	High
C. Mass Conservation	Simple	Minimal	Minimal	Low	Minimal	Medium	Medium to High

Although each of the models can be used to evaluate the irrigation performance, following the procedure discussed in Chapter 2, a mass conservation model will be discussed in more detail.

The use of mass conservation models is recommended because of the following advantages:

a. The models are simple to apply and the final results can be obtained by manual calculation, although programming can help in the extending of the model application to many cases.

b. With the mass conservation models, it is easy to follow and understand the various phases of the irrigation and to relate specific results (slow advance, cutoff time too short, etc.) to the basic data and operating variables.

c. Although assumptions are made to simplify the models, the final results generally are accurate enough for practical usage.

d. The mass conservation models exhibit a high degree of generality and practicality. They can cover all the common varying cases of border irrigation. Consequently, these models can be applied to any case of border irrigation.

The application of the mass conservation model is further discussed in the next section. However, according to specific circumstances, other models (mathematical or empirical) can be used when their application is favorable.

4. Application of a mass conservation model

The mass conservation model used herein for the evaluation of border irrigation is based on the algebraic model as presented by Strelkoff (1977), with some modifications to simplify the model and to adjust it as a tool in the evaluation of the irrigation performance.

There are four main categories of border irrigation, all of which can be treated by the model. These categories are: (see schematic description in Fig. 4-1)

a. Border irrigation with free drainage at the downstream end.

b. Border irrigation with a downstream dike and a relatively long cutoff time so that the border is completely covered before cutoff occurs and a horizontal pond covers the border at cutoff time.

c. Border irrigation with a downstream dike and a moderately long cutoff time so that the border is completely covered before cutoff occurs but a horizontal pond covers only a portion of the border shortly after cutoff.

d. Border irrigation with a downstream dike and a cutoff time so short that the border is not completely covered by cutoff time. Within this category are three subcategories:

d_1 - recession at the head end starts before the water has advanced to the end of the border.

d_2 - recession at the head end starts after the water has advanced to the end of the border, and there is sufficient water to form a horizontal pond over the entire border.

d_3 - recession at the head end starts after the water has advanced to the end of the border, but there is enough water to form a horizontal pond over only a portion of the border.

Another possibility is that the cutoff time is so short that the water never advances to the end of the border. This situation can be identified by the model, but is not analyzed.

The first stage of the model is the calculation of the advance phase, which is the same for all four categories. After the advance phase is

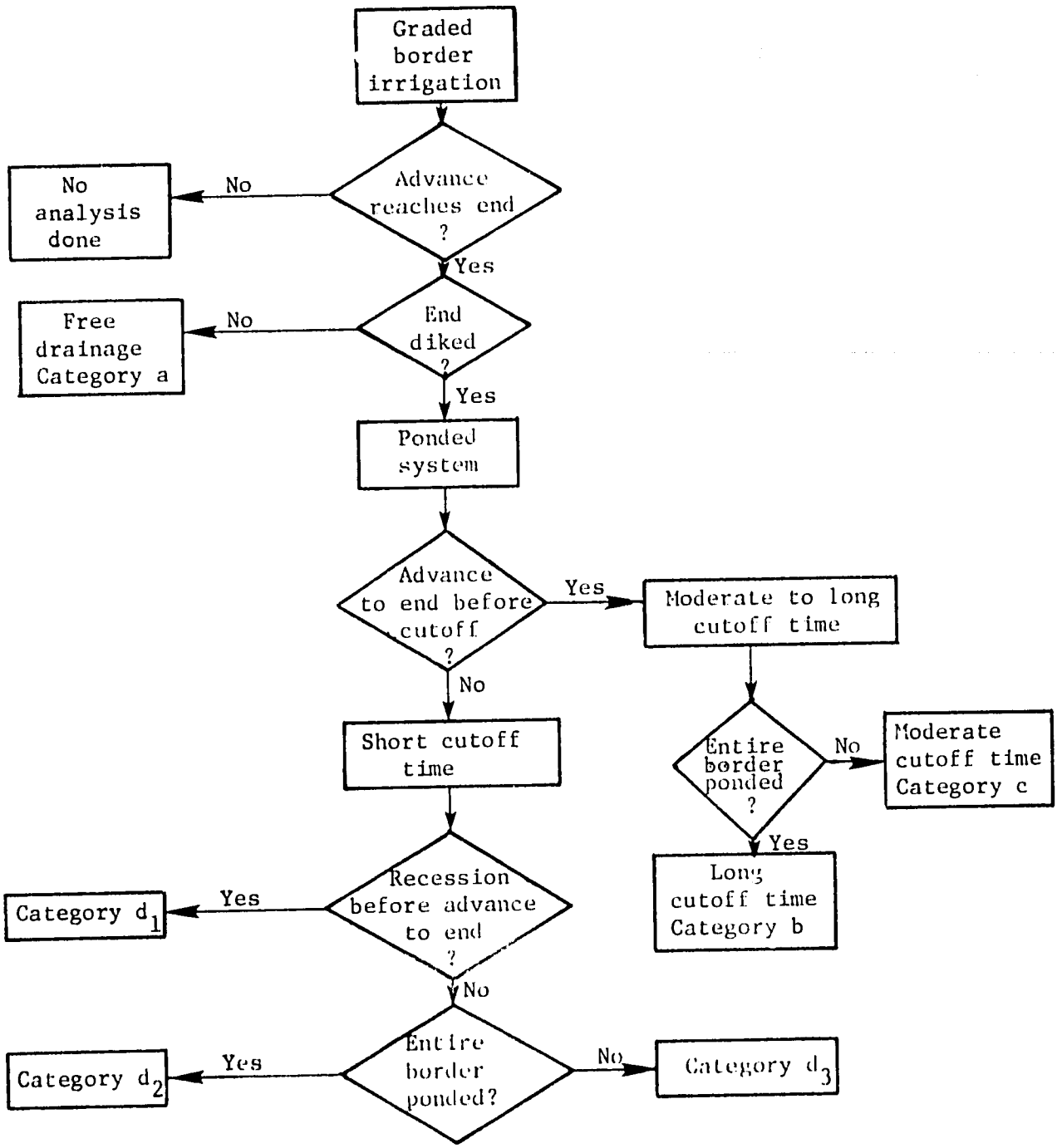


Fig. 4-1. Schematic description of possible categories of border irrigation.

calculated, the model checks and identifies the specific category. For the free drainage category (category a), the model provides the recession starting time and the recession curve together with the runoff before and during recession. For the other three cases with ponded water (categories b, c, d), the model identifies the expected case and provides the recession starting time and recession curve. Finally, when the advance, recession and runoff losses are known, the distribution of the depths of water infiltrated into the soil is established and the irrigation performance is determined.

The detailed procedure is given in Fig. 4-2, in which reference is made to equations which are briefly explained and listed below. For more details see Strelkoff (1977).

a. Surface shape factor (all categories).

The shape factor for the surface flow, r_y, represents the ratio of the volume of water on the surface to the rectangle of area, $d_o(t)x_a(t)$, where

$d_o(t)$ = surface depth of flow at $x = 0$

$x_a(t)$ = advance distance at time t

If the surface profile can be assumed to have the shape of a monomial power curve of degree β (see Fig. 4-3a) with $0 < \beta < 1$, the surface shape factor is

$$r_y = \frac{1}{1 + \beta} \tag{4-1}$$

If no particular value of β is known, it is generally assumed that $\beta = 0.25$, thus $r_y = 0.8$.

b. Subsurface shape factor (all categories).

The subsurface shape factor, r_z, represents the ratio of the volume of water infiltrated to the rectangle of area $y_o(t)x_a(t)$,

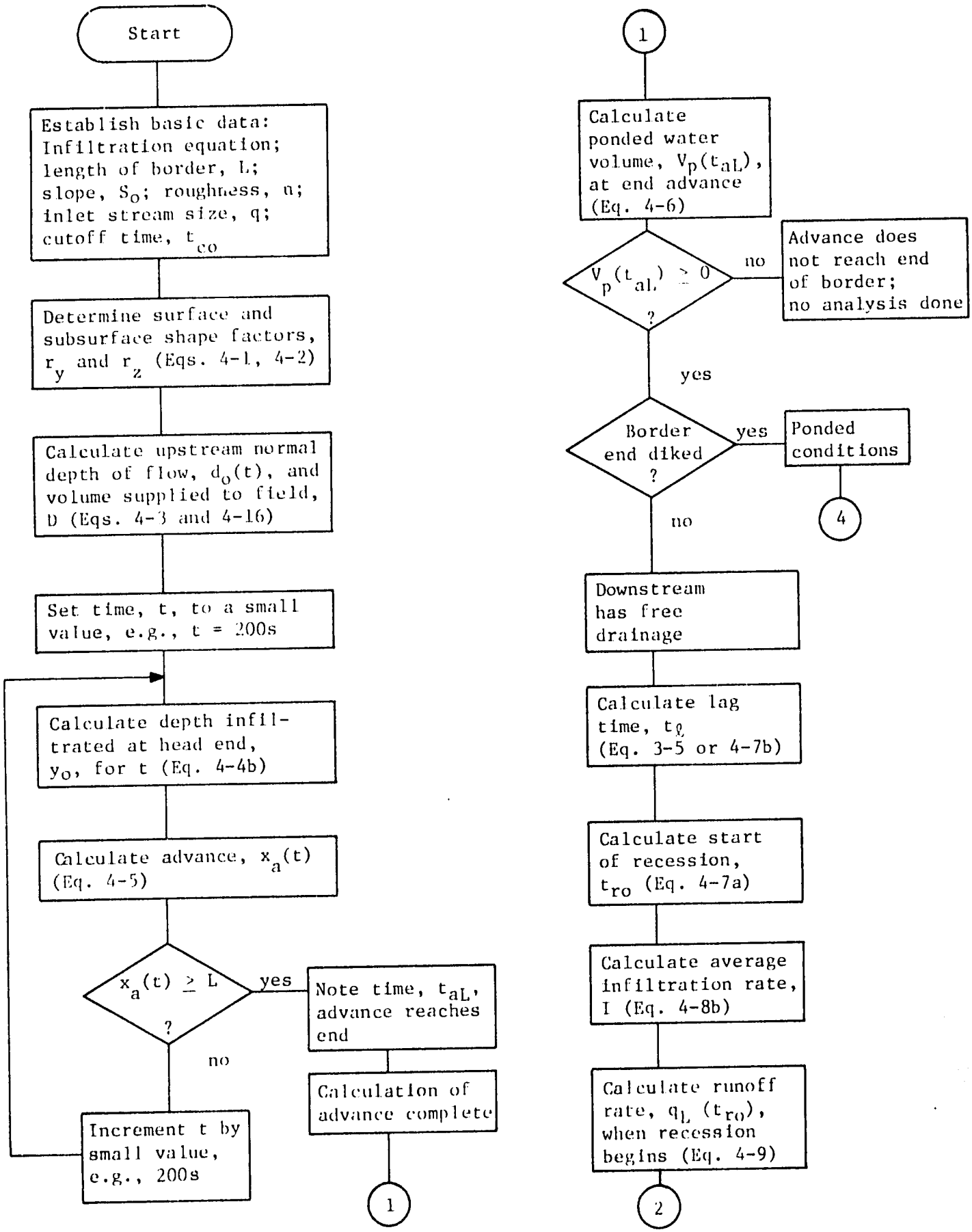
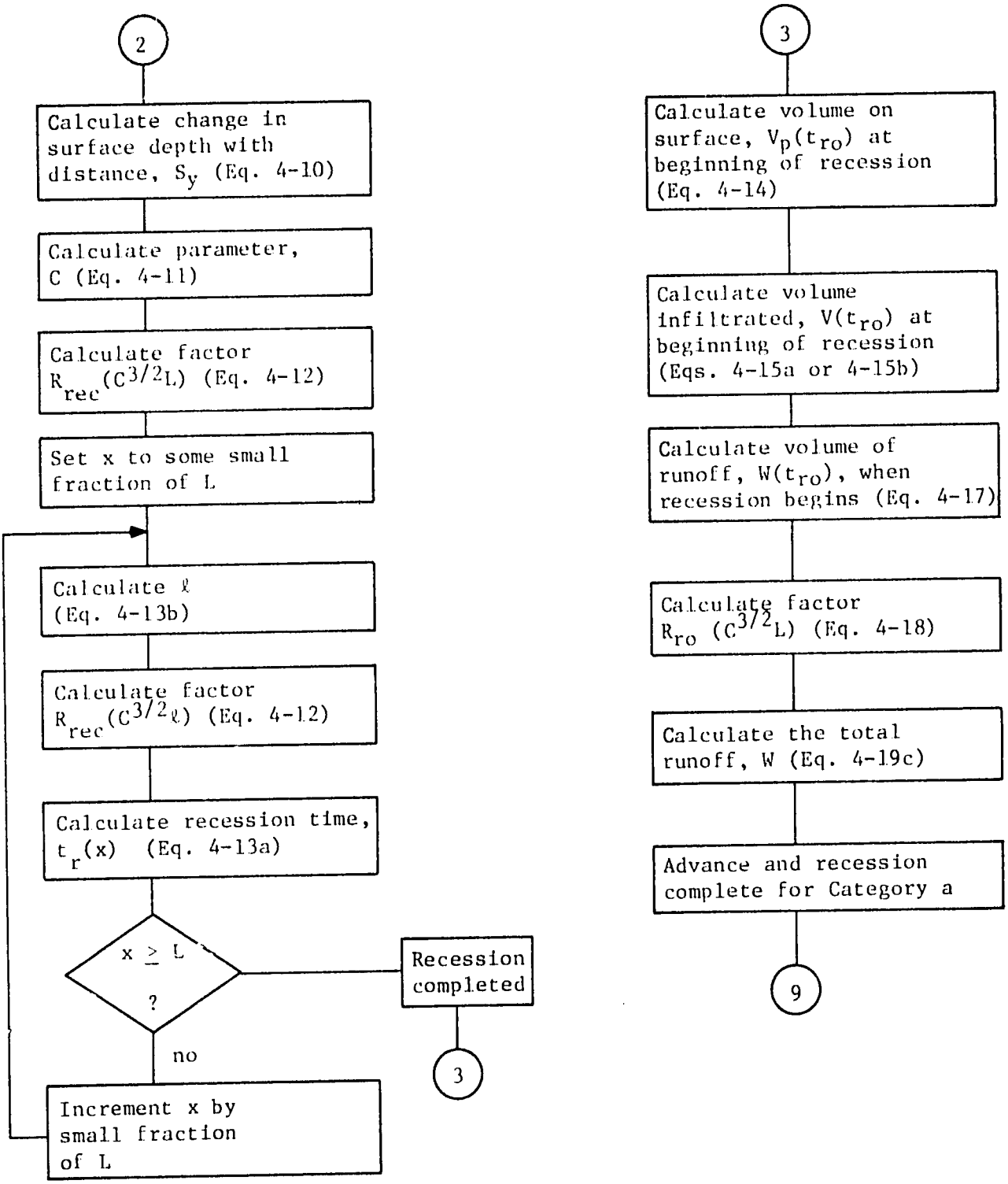
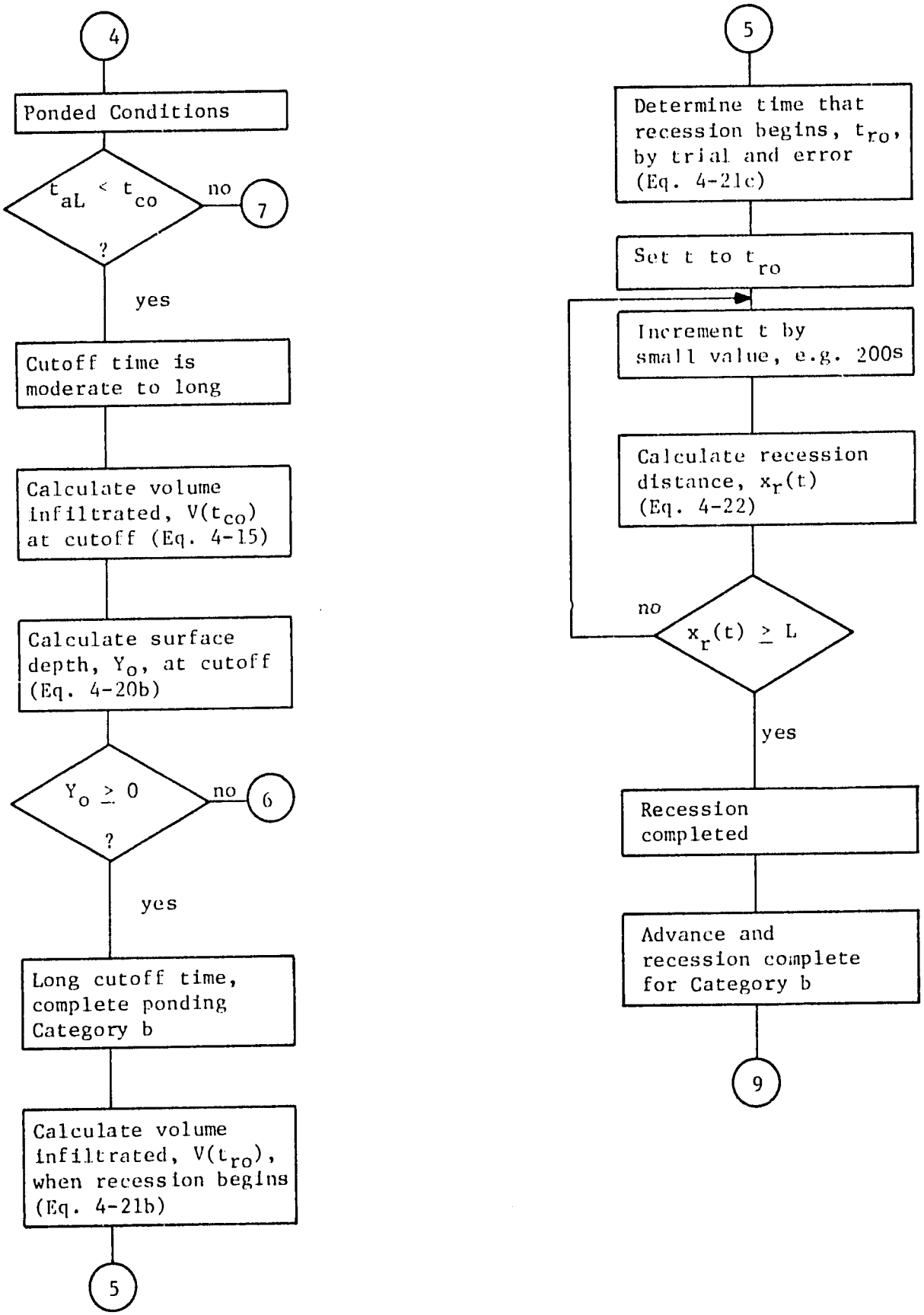
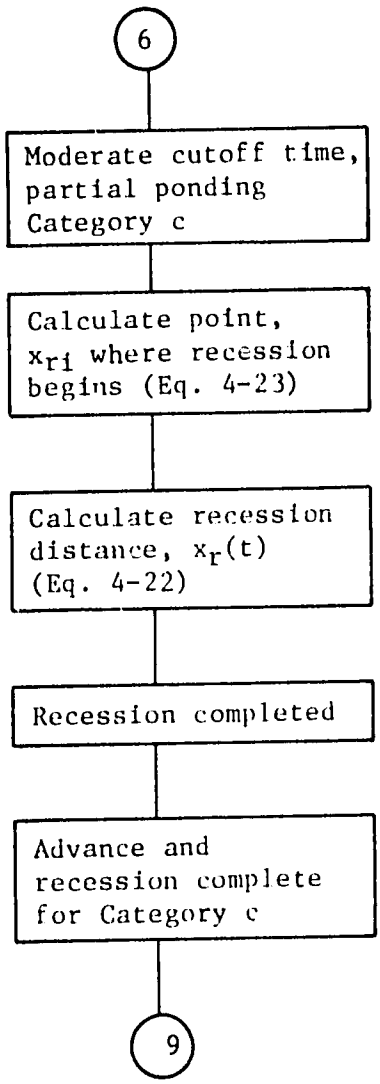
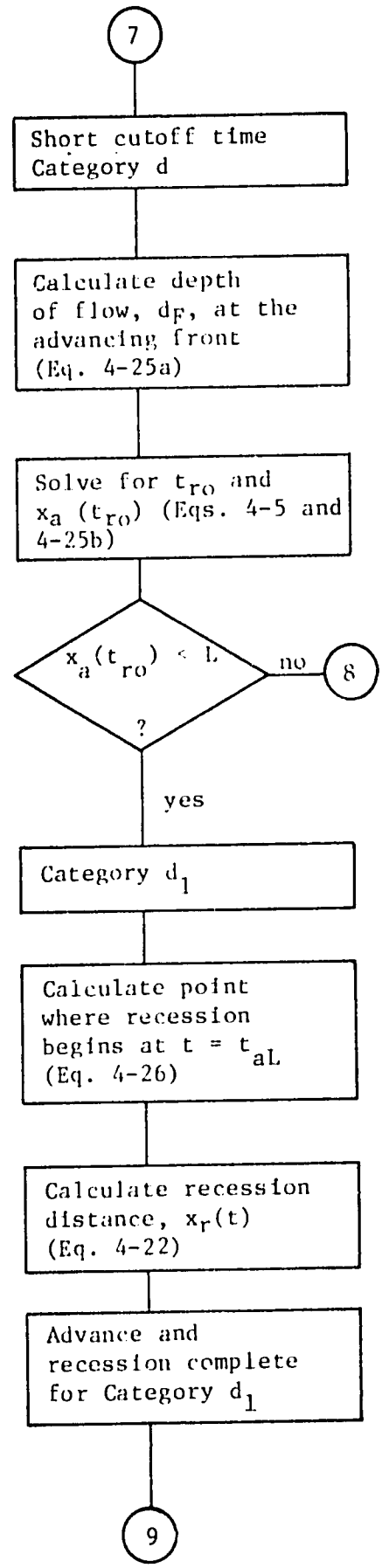


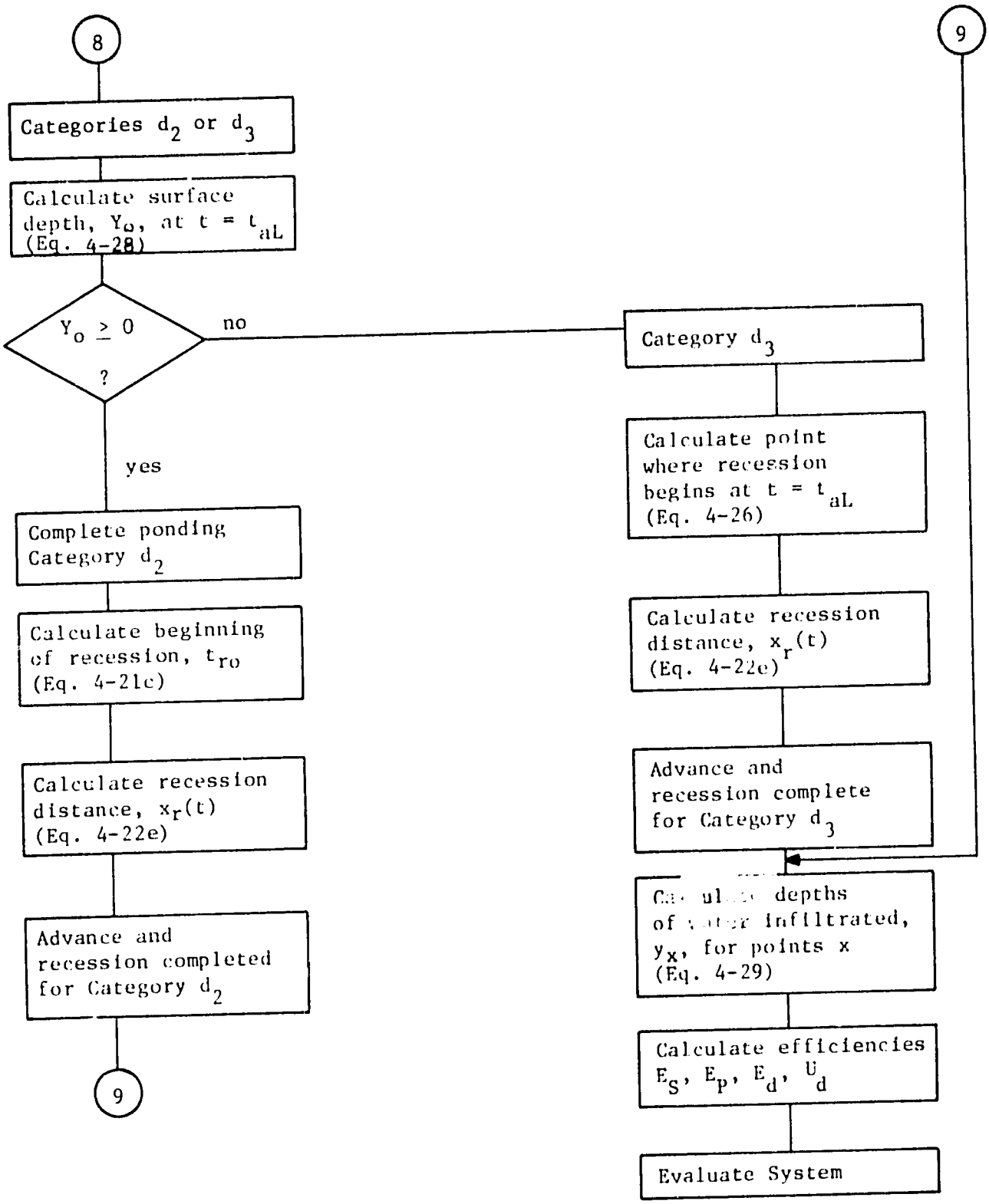
Fig. 4-2 Evaluation of graded border irrigation by mass conservation model (algebraic model)

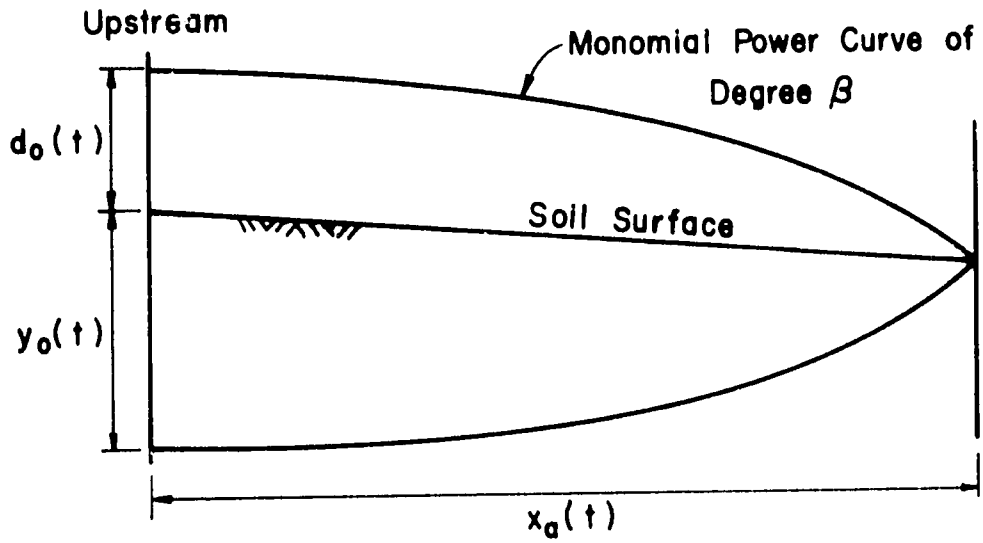

2
Calculate change in surface depth with distance, S_y (Eq. 4-10)
Calculate parameter, C (Eq. 4-11)
Calculate factor $R_{rec}(C^{3/2}L)$ (Eq. 4-12)
Set x to some small fraction of L
Calculate ℓ (Eq. 4-13b)
Calculate factor $R_{rec}(C^{3/2}\ell)$ (Eq. 4-12)
Calculate recession time, $t_r(x)$ (Eq. 4-13a)
$x \geq L$?
no
Recession completed
3
Increment x by small fraction of L
3
Calculate volume on surface, $V_p(t_{ro})$ at beginning of recession (Eq. 4-14)
Calculate volume infiltrated, $V(t_{ro})$ at beginning of recession (Eqs. 4-15a or 4-15b)
Calculate volume of runoff, $W(t_{ro})$, when recession begins (Eq. 4-17)
Calculate factor R_{ro} $(C^{3/2}L)$ (Eq. 4-18)
Calculate the total runoff, W (Eq. 4-19c)
Advance and recession complete for Category a
9


4
Ponded Conditions
$t_{aL} < t_{co}$?
no
7
yes
Cutoff time is moderate to long
Calculate volume infiltrated, $V(t_{co})$ at cutoff (Eq. 4-15)
Calculate surface depth, Y_o, at cutoff (Eq. 4-20b)
$Y_o \geq 0$?
no
6
yes
Long cutoff time, complete ponding Category b
Calculate volume infiltrated, $V(t_{ro})$, when recession begins (Eq. 4-21b)
5
5
Determine time that recession begins, t_{ro}, by trial and error (Eq. 4-21c)
Set t to t_{ro}
Increment t by small value, e.g. 200s
Calculate recession distance, $x_r(t)$ (Eq. 4-22)
$x_r(t) \geq L$
no
yes
Recession completed
Advance and recession complete for Category b
9

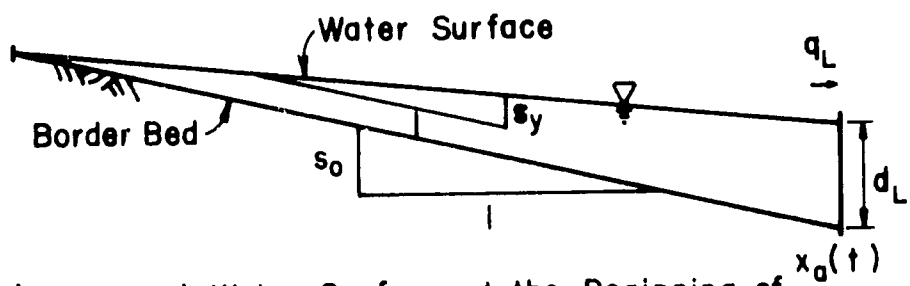

6
Moderate cutoff time, partial ponding Category c
Calculate point, x_{ri} where recession begins (Eq. 4-23)
Calculate recession distance, $x_r(t)$ (Eq. 4-22)
Recession completed
Advance and recession complete for Category c
9


7
Short cutoff time Category d
Calculate depth of flow, d_F, at the advancing front (Eq. 4-25a)
Solve for t_{ro} and $x_a(t_{ro})$ (Eqs. 4-5 and 4-25b)
$x_a(t_{ro}) < L$?
no
8
yes
Category d_1
Calculate point where recession begins at $t = t_{aL}$ (Eq. 4-26)
Calculate recession distance, $x_r(t)$ (Eq. 4-22)
Advance and recession complete for Category d_1
9


8
Categories d_2 or d_3
Calculate surface depth, Y_o, at $t = t_{aL}$ (Eq. 4-28)
$Y_o \geq 0$?
no
yes
Complete ponding Category d_2
Calculate beginning of recession, t_{ro} (Eq. 4-21c)
Calculate recession distance, $x_r(t)$ (Eq. 4-22e)
Advance and recession completed for Category d_2
9
Category d_3
Calculate point where recession begins at $t = t_{aL}$ (Eq. 4-26)
Calculate recession distance, $x_r(t)$ (Eq. 4-22e)
Advance and recession complete for Category d_3
9
Calculate depths of water infiltrated, y_x, for points x (Eq. 4-29)
Calculate efficiencies E_S, E_p, E_d, U_d
Evaluate System



a) General Description of Surface and Subsurface Water Profiles During Advance



b) Assumed Water Surface at the Beginning of Recession-Free Drainage

Fig. 4-3. Description of surface and assumed water profiles in border irrigation.

where

$y_o(t)$ = depth of water infiltrated at $x = 0$

The subsurface shape factor can be approximated by:

$$r_z = \frac{1}{1 + B} \tag{4-2}$$

where B is the exponent in the Kostiakov equation (Eq. 4-4a).

c. Normal depth of flow at head end (all categories).

When water is being applied to the border, it is assumed that the flow depth at the head end of the border, $d_o(t)$ is that of the normal depth of flow for the inlet stream, q. From Manning's equation

$$d_o(t) = \left[\frac{qn}{CU\ S_o^{0.5}}\right]^{0.6} \qquad 0 < t \leq t_{co} \tag{4-3}$$

If ponding occurs, the depth at the head end may increase to some depth, $Y_o > d_o(t)$.

d. Depth of water infiltrated at head end (all categories).

When the infiltration function is of the Kostiakov form

$$z(t_{op}) = At_{op}^{B} \tag{4-4a}$$

the depth of water infiltrated at the head end of the border is

$$y_o(t) = At^{B} \qquad 0 < t < t_r(0) \tag{4-4b}$$

where

$t_r(0)$ = time that water recedes from point $x = 0$

e. Advance function (all categories).

Although there are many methods of calculating the advance distance as a function of time, the volume balance method is often used because of its simplicity and relative accuracy. From the volume balance

$$x_a(t) = \frac{qt}{r_y d_o(t) + r_z y_o(t)} \tag{4-5}$$

f. Surface volume when advance reaches end of border (test for advance reaching end).

If it is assumed that the advance reaches the end of the border after cutoff time, then the surface volume of water when advance is completed is:

$$V_p(t_{aL}) = qt_{co} - r_z L\ z(t_{aL}) \tag{4-6}$$

where

$V_p(t)$ = volume ponded on the surface.

If $V_p(t_{aL})$ is negative, it means that there was not sufficient volume inflow to advance the water to the end of the border. This condition is not considered further.

g. Start of recession time (free drainage, category a).

Recession begins at the head end at time $t_{ro} = t_r(0)$, given by

$$t_{ro} = t_{co} + t_\ell \tag{4-7a}$$

The lag time, t_ℓ, can be calculated from Eq. 3-5 or according to Strelkoff (1977)

$$t_\ell = \frac{d_o(t_{co})L}{2\ q} \tag{4-7b}$$

h. Average infiltration rate at beginning of recession (category a).

From the Kostiakov cumulative infiltration function, the infiltration rate at any time is

$$\frac{dz}{dt} = BAt_{op}^{B-1} \tag{4-8a}$$

If it is assumed that the infiltration rate along the border can be approximated by a straight line at the time that recession begins, the average infiltration rate, I, is given by

$$I = \frac{BA}{2}\left[t_{ro}^{B-1} + (t_{ro} - t_{aL})^{B-1}\right] \tag{4-8b}$$

If the infiltration time at any point is reasonably long, I will also approximate the average infiltration rate at any time $t \geq t_{ro}$.

i. Runoff discharge at beginning of recession (category a).

It is assumed that at the beginning of recession the combined runoff and infiltration rates are equal to the original flow rate onto the border, then

$$q_L(t_{ro}) = q - IL \tag{4-9}$$

where

$q_L(t)$ = the runoff rate

j. Change in surface depth with distance (category a).

If the downstream surface depth, $d_L(t)$, is the normal depth for the flow, $q_L(t)$, the change in surface depth with distance, S_y, is (see Fig. 4-3b)

$$S_y = \frac{d_L(t)}{L} = \frac{1}{L}\left[\frac{nq_L(t)}{CU\ S_o^{0.5}}\right]^{0.6} \tag{4-10}$$

k. Parameter C (category a).

To simplify calculations a parameter, C, is defined as

$$C = \frac{CU\ S_o^{0.5}\ S_y^{5/3}}{nI} \tag{4-11}$$

l. Recession factor, $R_{rec}(u)$ (category a).

An integral of the form

$$R_{rec}(u) = \int_o^u \frac{d\zeta}{1 + \zeta^{2/3}} \tag{4-12a}$$

is used in the calculation of the recession. According to Strelkoff (1977), Eq. 4-12a can be evaluated by

$$R_{rec}(u) = \sum_{i=0}^{\infty} \frac{(-1)^i \, u^{(2i+3)/3}}{(2i+3)/3} \qquad u \leq 1 \tag{4-12b}$$

$$R_{rec}(u) = \sum_{i=0}^{\infty} \frac{(-1)^i \, u^{(1-2i)/3}}{(1-2i)/3} - 4.71238 \qquad u > 1 \tag{4-12c}$$

The constant 4.71238 in Eq. 4-12c arises because of a discontinuity in the value of $R_{rec}(u)$ at $u = 1$ if only the series terms of the equation are used. Fig. 4-4 gives a plot of $R_{rec}(u)$ for $0 \leq u \leq 10$.

m. Recession time (category a).

According to Strelkoff the time that the water recedes at point x is

$$t_r(x) = t_{ro} + \frac{S_y}{IC^{3/2}} \left[R_{rec}(C^{3/2}L) - R_{rec}(C^{3/2}\ell) \right] \tag{4-13a}$$

where

$$\ell = L - x \tag{4-13b}$$

n. Volume of surface water at beginning of recession (category a).

As the water surface is assumed to be a straight line at the beginning of recession, the volume of surface water, $V_p(t_{ro})$ is

$$V_p(t_{ro}) = \frac{S_y L^2}{2} \tag{4-14}$$

o. Volume infiltrated by beginning of recession (category a).

If the subsurface water profile is assumed to be a straight line when recession begins, the volume infiltrated by that time, $V(t_{ro})$, is

$$V(t_{ro}) = \frac{L \, [z(t_{ro}) + z(t_{ro} - t_{aL})]}{2} \tag{4-15a}$$

However, it is clear that the infiltrated depth is a concave line rather than a straight line. This concave line can be approximated by two straight lines meeting at $x = KL$ $(0.5 \leq K \leq 1)$ and $y = z\,(t_{ro} - t_{aL}) + K\,[z(t_{ro}) - z(t_{ro} - t_{aL})]$ (see Fig. 4-5). This approximation results in

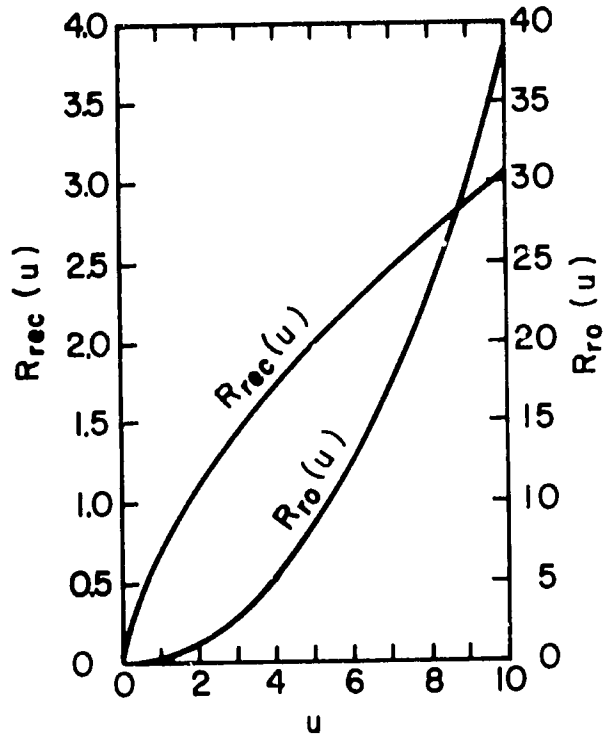


Fig. 4-4. Generalized recession and runoff functions (from Strelkoff, 1977).

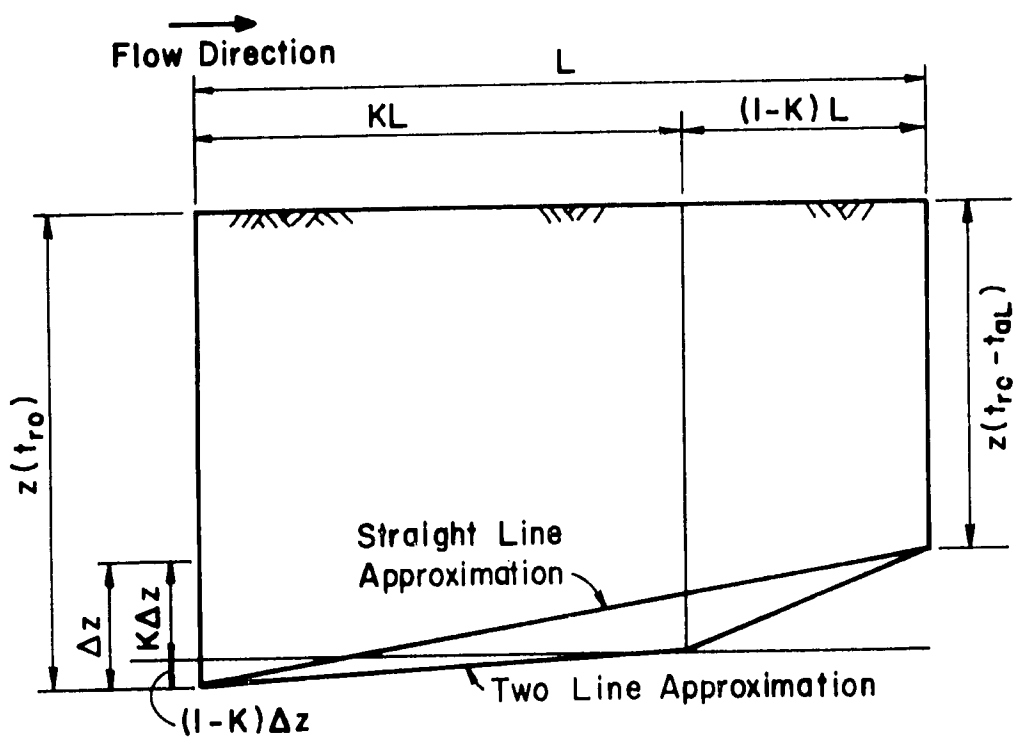


Fig. 4-5. Assumed water distribution profile for the calculation of the infiltrated volume.

$$V(t_{ro}) = L[Kz(t_{ro}) + (1 - K)\ z\ (t_{ro} - t_{aL})] \tag{4-15b}$$

Note that when K = 0.5, Eqs. 4-15a and 4-15b are identical. K can be approximated by:

$$K = \frac{1}{1 + B} \tag{4-15c}$$

where B is the exponent in the Kostiakov equation (Eq. 4-4a). More details about selecting K are given in Peri et al. (1979).

p. Volume of water delivered by beginning of recession (category a).

The volume of water delivered to the field, D, is

$$D = qt_{co} \tag{4-16}$$

q. Volume of runoff before recession (category a).

The volume of water that runs off before recession begins, $W(t_{ro})$, is the difference between the applied volume and the sum of the surface storage and infiltrated volume. Thus

$$W(t_{ro}) = D - V(t_{ro}) - V_p(t_{ro}) \tag{4-17}$$

r. Runoff factor, $R_{ro}(u)$ (category a).

An integral of the form

$$R_{ro}(u) = \int_o^u \frac{\zeta^{5/3} d\zeta}{1 + \zeta^{2/3}} \tag{4-18a}$$

is used in the calculation of the runoff. According to Strelkoff (1977), Eq. 4-18a can be evaluated by

$$R_{ro}(u) = \sum_{i=0}^{\infty} \frac{(-1)^i\ u^{(8 + 2i)/3}}{(8 + 2i)/3} \qquad u \leq 1 \tag{4-18b}$$

$$R_{ro}(u) = \frac{u^2}{2} - \frac{3u^{4/3}}{4} + \frac{3u^{2/3}}{2} - \ln(u) + \sum_{i = 0}^{\infty} \frac{(-1)^i\ u^{(-2 - 2i)/3}}{(-2 - 2i)/3} \qquad u > 1 \tag{4-18c}$$

Fig. 4-4 gives a plot of $R_{ro}(u)$ for $0 \leq u < 10$.

s. Runoff during recession (category a).

The runoff during recession is given by the total runoff less the runoff before recession began, that is

$$W_r(t) = W(t) - W(t_{ro}) \tag{4-19a}$$

or

$$W_r(t) = \frac{S_y}{C^3} \left[R_{ro} (C^{3/2}L) - R_{ro} (C^{3/2}\ell)\right] \tag{4-19b}$$

where

$W_r(t)$ = the runoff volume during recession up to time t

The total runoff for the irrigation, W, is given by

$$W = W(t_{ro}) + \frac{S_y}{C^3} R_{ro} (C^{3/2}L) \tag{4-19c}$$

t. Head end surface depth at cutoff (test for category b or category c).

For ponded conditions it is first assumed that the water surface is horizontal at cutoff time. Thus, the ponded water volume is

$$V_p(t_{co}) = L\left[Y_o + \frac{S_o L}{2}\right] \tag{4-20a}$$

where

Y_o = surface depth at cutoff for x = 0

However, the ponded water volume is the difference between the applied water, D, and the infiltrated volume, $V(t_{co})$ as given by Eq. 4-15a or 4-15b. Therefore, Y_o can be calculated as

$$Y_o = \frac{D - V(t_{co})}{L} - \frac{S_o L}{2} \tag{4-20b}$$

If $Y_o \geq 0$, the cutoff time is considered large and the system is of category b. If $Y_o < 0$, the cutoff time is considered moderate and the system is of category c.

u. Time recession begins (category b).

Recession begins when the surface water depth is zero at the head end of the border, thus

$$V_p(t_{ro}) = \frac{S_o L^2}{2} \tag{4-21a}$$

The infiltrated volume, $V(t_{ro})$, is

$$V(t_{ro}) = D - \frac{S_o L^2}{2} \tag{4-21b}$$

and t_{ro} can be determined by trial and error for t in

$$V(t_{ro}) = L\,[Kz(t) + (1 - K)\,z\,(t - t_{aL})] \tag{4-21c}$$

v. Recession distance as a function of time (category b).

The point at which recession is taking place at any time $t > t_{ro}$ can be calculated in a stepwise fashion by equating the decrease in surface volume to the increase in subsurface storage over a time increment Δt. Fig. 4-6 shows the surface and subsurface water profiles at two times, t and $t - \Delta t$. The decrease in surface storage over the time increment Δt is

$$\Delta V_p = \frac{S_o\,[x_r(t) - x_r(t - \Delta t)]^2}{2} + S_o[L - x_r(t)][x_r(t) - x_r(t - \Delta t)] \tag{4-22a}$$

The increase in subsurface storage over the same time increment can be approximated by

$$\Delta V = \frac{\Delta z_x\,[x_r(t) - x_r(t - \Delta t)]}{2} + [L - x_r(t)]\frac{[\Delta z_x + \Delta z_L]}{2} \tag{4-22b}$$

where $\Delta z_x = z[t - t_a(x)] - z[t - \Delta t - t_a(x)]$ (4-22c)

$$\Delta z_L = z[t - t_{aL}] - z[t - \Delta t - t_{aL}] \tag{4-22d}$$

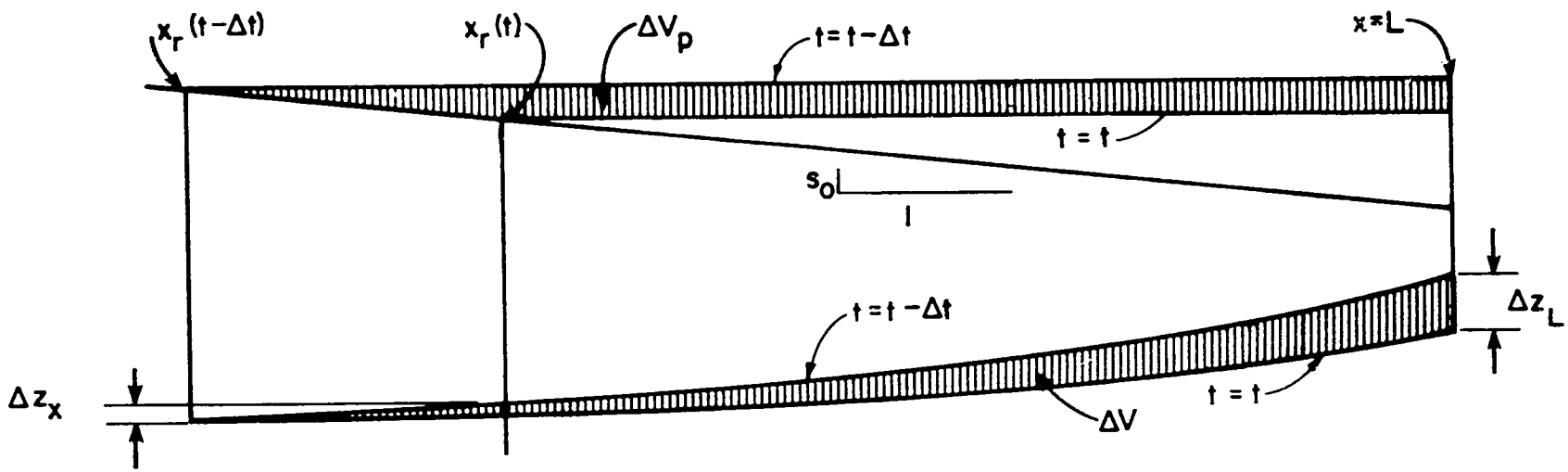


Fig. 4-6. Surface and subsurface water profiles at two times, t and t-Δt, for ponded conditions.

Setting Eq. 4-22a equal to Eq. 4-22 b and solving for $x_r(t)$ results in

$$x_r(t) = L + \frac{\Delta z_L - \sqrt{4S_o^2[L-x_r(t-\Delta t)]^2 - 4S_o[L-x_r(t-\Delta t)]\Delta z_x + \Delta z_L^2}}{2\,S_o} \qquad (4\text{-}22e)$$

The RHS of Eq. 4-22e does contain one unknown term, that is $t_a(x)$ [via Δz_x], however a reasonable approximation can be made for $t_a(x)$. If it is deemed necessary, an improved value for $t_a(x)$ can be inserted after making a first calculation for $x_r(t)$ and the equation can be iterated. In most practical cases this iteration will not be necessary, in fact Strelkoff (1977) has assumed

$$t_a(x) = \frac{t_{aL}}{3} \qquad (4\text{-}22f)$$

for all x. A better approximation can be made by assuming the advance was linear and letting the advance time to $x = x_r(t)$ be that for $x = x_r(t - \Delta t)$, that is

$$t_a(x) = \frac{x_r(t - \Delta t)\ t_{aL}}{L} \qquad (4\text{-}22g)$$

By assuming the advance to be linear, the advance time to any point $x < L$ is overestimated, however, using the advance time to point $x = x_r(t - \Delta t)$ underestimates the advance time to point $x = x_r(t)$, therefore, the errors will be compensating. Eq. 4-22e is solved by starting at $t = t_{ro} + \Delta t$ so that $t - \Delta t = t_{ro}$ and $x_r(t - \Delta t) = 0$.

w. Point where recession begins when recession begins shortly after cutoff (category c).

When the cutoff time is moderate, there is not a sufficient volume of water applied to the border to form a pond with a horizontal surface over the entire border and Y_o of Eq. 4-20b has a negative value.

However, shortly after cutoff, a horizontal water surface does form downfield beginning at a point x_{ri}. This point can be determined from a volume balance between the applied volume, D (Eq. 4-16), the infiltrated volume, $V(t_{co})$ ($t_{ro} = t_{co}$ in Eq. 4-15a or 4-15b) and the surface volume. That is

$$x_{ri} = L - \left\{ \frac{2[D - V(t_{co})]}{S_o} \right\}^{0.5} \tag{4-23}$$

x. Recession distance as a function of time (category c).

The recession distance for this category is calculated similar to that for category b (Eq. 4-22e), however, the first calculation will be for time, $t = t_{ro} + \Delta t$, so that $t - \Delta t = t_{ro}$ and

$$x_r(t - \Delta t) = x_{ri} \tag{4-24}$$

y. Advance distance when recession begins at x = 0 (test for category d_1 or categories d_2 or d_3).

If it is assumed that the depth of water at the advance front, d_F, is that depth which corresponds to a normal velocity of v_F, which in turn is the average advance rate, then

$$d_F = \left[\frac{Ln}{t_{aL} CU\ S_o^{0.5}} \right]^{1.5} \tag{4-25a}$$

If it is assumed further, that the surface water profile, when recession begins at x = 0 is that of a triangular wedge with depth of zero at x = 0 and depth d_F at $x_a(t_{ro})$, then the volume balance equation at $t = t_{ro}$ is

$$\left[r_z\ z(t_{ro}) + \frac{d_F}{2} \right] x_a(t_{ro}) = D \tag{4-25b}$$

Eq. 4-25b can be solved for t_{ro} (using Eq. 4-5 to evaluate $x_a(t_{ro})$) by trial and error. If $x_a(t_{ro}) > L$, a physical impossibility, recession must start after advance has reached the end of the field, that is $t_{ro} > t_{aL}$ and the system is category d_2 or d_3. If $x_a(t_{ro}) < L$, recession starts before advance has reached the end of the field and the system is category d_1.

z. Point where recession begins (category d_1).

When recession begins before advance has reached the end of the border, it is assumed that a horizontal pond develops at or shortly after time, t_{aL}. The point where this pond begins is found from a volume balance as

$$x_{ri} = L - \left\{ \frac{2[D - Lr_z\ z(t_{aL})]}{S_o} \right\}^{0.5} \tag{4-26}$$

aa. Recession distance as a function of time (category d_1).

The recession distance for this category is calculated in a in a manner similar to that for categories b and c, however, the recession starts at $x = x_{ri}$ at $t = t_{aL}$ thus the first calculation is for

$$t = t_{aL} + \Delta t \tag{4-27}$$

bb. Head end surface depth when advance reaches end (test for category d_2 or d_3).

When the advance reaches the end of the border, a horizontal pond is formed soon thereafter. From a volume balance, the depth of water at the head end can be determined as

$$Y_o = \frac{D}{L} - \frac{S_o L}{2} - r_z\ z(t_{aL}) \tag{4-28}$$

If $Y_o \geq 0$, the entire border is covered by the pond, and the system is of category d_2. If $Y_o < 0$, only a portion is ponded and the system is of category d_3.

cc. Recession distance as a function of time (category d_2).

The time that recession begins at $x = 0$, t_{ro}, can be found from Eq. 4-21c and the recession distance $x_r(t)$ can be obtained from Eq. 4-22e.

dd. Recession distance as a function of time (category d_3).

The horizontal pond begins at x_{ri} as given by Eq. 4-26 and the recession distance can be obtained from Eq. 4-22e.

ee. Final subsurface water profile (all categories).

The final depth of water infiltrated at any point is given by

$$y_x = z[t_r(x) - t_a(x)] \tag{4-29}$$

<u>Example (Category a)</u>

A border with the following physical characteristics is irrigated with free drainage at the end.

$A = 0.0005\ m/s^{0.6}$

$B = 0.6$

$S_o = 0.001$

$L = 200\ m$

$n = 0.05\ m^{1/6}$

The cumulative infiltration curve for this soil is shown in Fig. 4-7.

The operating variables are chosen as

$q = 0.005\ m^3/s \cdot m$

$t_{co} = 2500\ s$

Following the scheme given in Fig. 4-2:

$r_y = 0.8$ (assumed)

$r_z = \dfrac{1}{1 + 0.6} = 0.625$ (Eq. 4-2)

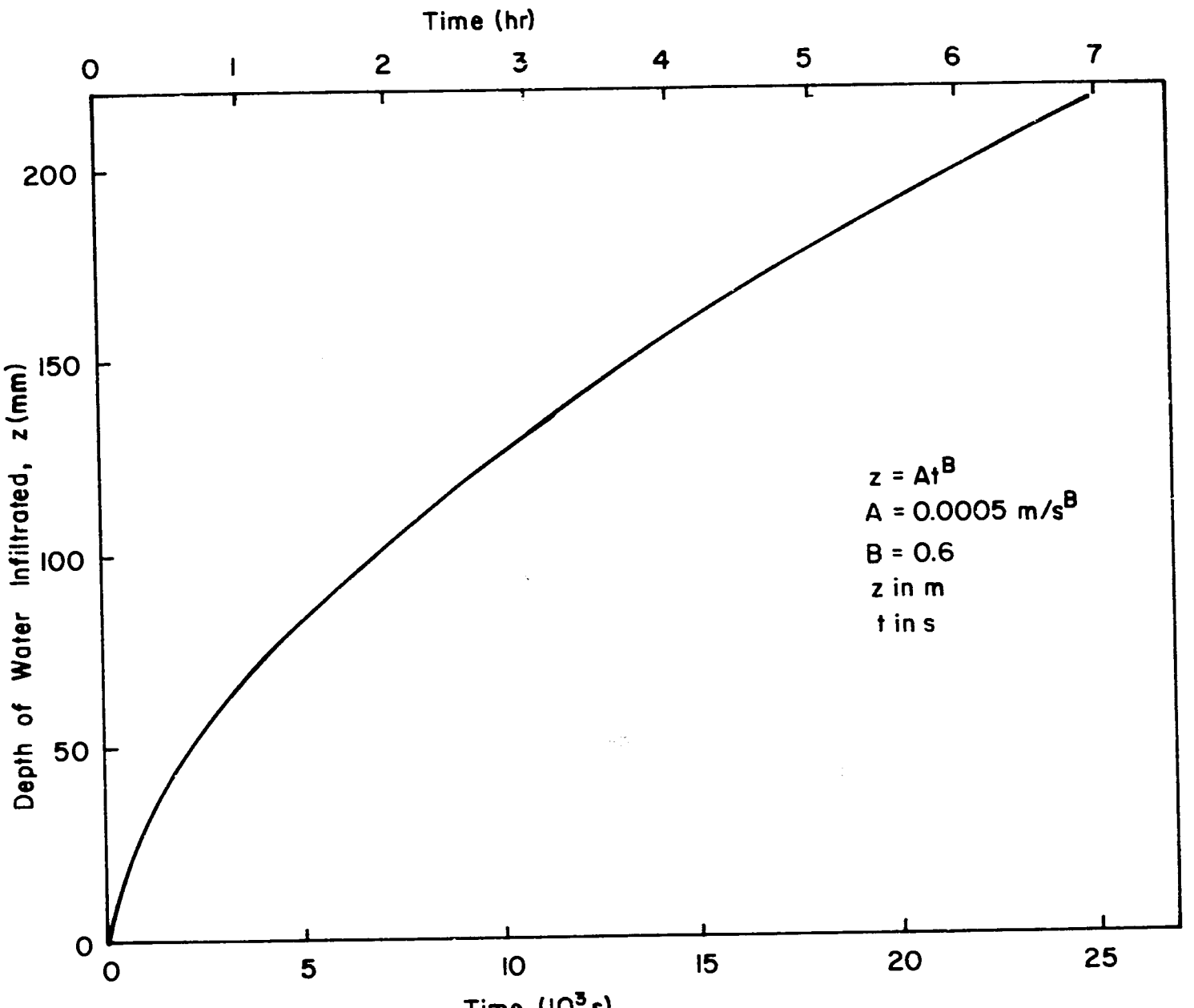


Fig. 4-7. Cumulative depth of water infiltrated (z in m, t in s).

$$d_o(t) = \left[\frac{0.005 \cdot 0.05}{1 \cdot 0.001^{0.5}}\right]^{0.6} = 0.0548 \text{ m} \qquad \text{(Eq. 4-3)}$$

$$D = 0.005 \cdot 2500 = 12.5 \text{ m}^3/\text{m} \qquad \text{(Eq. 4-16)}$$

Use time increments of 200s to determine the advance. For example, when $t = 200$s

$$y_o(200) = 0.0005 \cdot 200^{0.6} = 0.0120 \text{ m} \qquad \text{(Eq. 4-4)}$$

$$x_a(200) = \frac{0.005 \cdot 200}{0.8 \cdot 0.548 + 0.625 \cdot 0.0120} = 19.5 \text{ m} \qquad \text{(Eq. 4-5)}$$

Table 4-2 shows $y_o(t)$ and $x_a(t)$ up to the end of advance. By linear interpolation, $t_{aL} = 3396$s. It should be noted that although cutoff takes place at $t = 2500$ s, it is assumed that the advance continues as if the water had not been shut off. This is a reasonable approximation as nearly 80% of the border is actually covered by cutoff time.

$$V_P(t_{aL}) = 0.005 \cdot 2500 - 0.625 \cdot 200 \cdot 0.0005(3396)^{0.6}$$
$$= 4.29 \text{ m}^3/\text{m} \qquad \text{(Eq. 4-6)}$$

$V_P(t_{aL})$ is positive, therefore, there is sufficient water to reach the end of the border.

$$t_\ell = \frac{0.0548 \cdot 200}{2 \cdot 0.005} = 1096 \text{ s} \qquad \text{(Eq. 4-7b)}$$

$$t_{ro} = 2500 + 1096 = 3596 \text{ s} \qquad \text{(Eq. 4-7a)}$$

$$I = \frac{0.6 \cdot 0.0005}{2}\left[3596^{-0.4} + (3596 - 3396)^{-0.4)}\right]$$
$$= 23.7 \times 10^{-6} \text{ m/s} \qquad \text{(Eq. 4-8b)}$$

$$q_L(t_{ro}) = 0.005 - 23.7 \times 10^{-6} \cdot 200$$
$$= 0.000262 \text{ m}^3/\text{s}\cdot\text{m} \qquad \text{(Eq. 4-9)}$$

$$S_y = \frac{1}{200}\left[\frac{0.05 \cdot 0.000262}{1 \cdot 0.001^{0.5}}\right]^{0.6} = 0.0000467 \qquad \text{(Eq. 4-10)}$$

Table 4-2. Infiltrated depth at upper end of border and advance distance as a function of time, for examples of category a and category d.

Time (s)	0	200	400	600	800	1000	1200	1400	1600	1800	2000	2200	2400	2600	2800	3000	3200	3400
$y_o(t)$ (mm)	0.0	12.0	18.2	23.2	27.6	31.5	35.2	38.6	41.8	44.9	47.8	50.6	53.3	56.0	58.5	61.0	63.4	65.7
$x_a(t)$ (m)	10	19.5	36.2	51.4	65.5	78.7	91.1	103.0	114.3	125.2	135.6	145.7	155.5	164.9	174.1	183.0	191.7	200.2

Table 4-3. Recession time as a function of distance, for example of category a.

$x_r(t)$ (m)	0	10	20	30	40	50	60	70	80	90	100	110	120	130	140	150	160	170	180	190	200
$t_r(x)$ (s)	3596	3614	3633	3652	3671	3689	3708	3727	3746	3765	3784	3803	3822	3841	3860	3880	3899	3918	3938	3957	3977

$$C = \frac{1 \cdot 0.005^{0.5}(0.0000467)^{5/3}}{0.05 \cdot 23.7 \times 10^{-6}} = 0.00361 \text{ m}^{-2/3} \qquad \text{(Eq. 4-11)}$$

$$R_{rec}\,(C^{3/2}L) = R_{rec}\,(0.0434) = 0.0404 \qquad \text{(4 terms of Eq. 4-12b)}$$

Use increments of 10 m to calculate the recession time. For x = 10

$$\ell = 200 - 10 = 190 \text{ m} \qquad \text{(Eq. 4-13b)}$$

$$R_{rec}\,(C^{3/2}\ell) = R_{rec}\,(0.0412) = 0.0385 \qquad \text{(4 terms of Eq. 4-12b)}$$

$$t_r(10) = 3596 + \frac{46.7 \times 10^{-6}}{23.7 \times 10^{-6}(0.00361)^{3/2}}\,[0.0404 - 0.0385]$$

$$= 3614 \text{ s} \qquad \text{(Eq. 4-13a)}$$

Table 4-3 shows the recession distance versus time for the entire irrigation. The data are also plotted in Fig. 4-8.

$$V_p(t_{ro}) = \frac{0.0000467(200)^2}{2} = 0.934 \text{ m}^3/\text{m} \qquad \text{(Eq. 4-14)}$$

$$V(t_{ro}) = 200[0.625 \cdot 0.0005(3596)^{0.6} + 0.375 \cdot 0.005(3596 - 3396)^{0.6}] = 9.40 \text{ m}^3/\text{m} \qquad \text{(Eq. 4-15b)}$$

$$W(t_{ro}) = 12.5 - 9.40 - 0.934 = 2.17 \text{ m}^3/\text{m} \qquad \text{(Eq. 4-17)}$$

$$R_{ro}(C^{3/2}L) = R_{ro}(0.0434) = 0.0000794 \qquad \text{(4 terms of Eq. 4-18b)}$$

$$W = 2.17 + \frac{0.0000467}{(0.00361)^3}\,(0.0000794) = 2.25 \text{ m}^3/\text{m} \qquad \text{(Eq. 4-19c)}$$

From the advance and recession curves of Fig. 4-8 or the data of Tables 4-2 and 4-3, interpolation can be done to give the infiltration opportunity times at the midpoint of each 10 m section of border. From these times the depth of water infiltrated at each point can be calculated (as given in Table 4-4) and plotted as a subsurface water profile (Fig. 4-9).

The mean depth infiltrated is 51.2 mm. Using Eq. 2-11 to calculate the volume of deficit of infiltrated water in relation to the

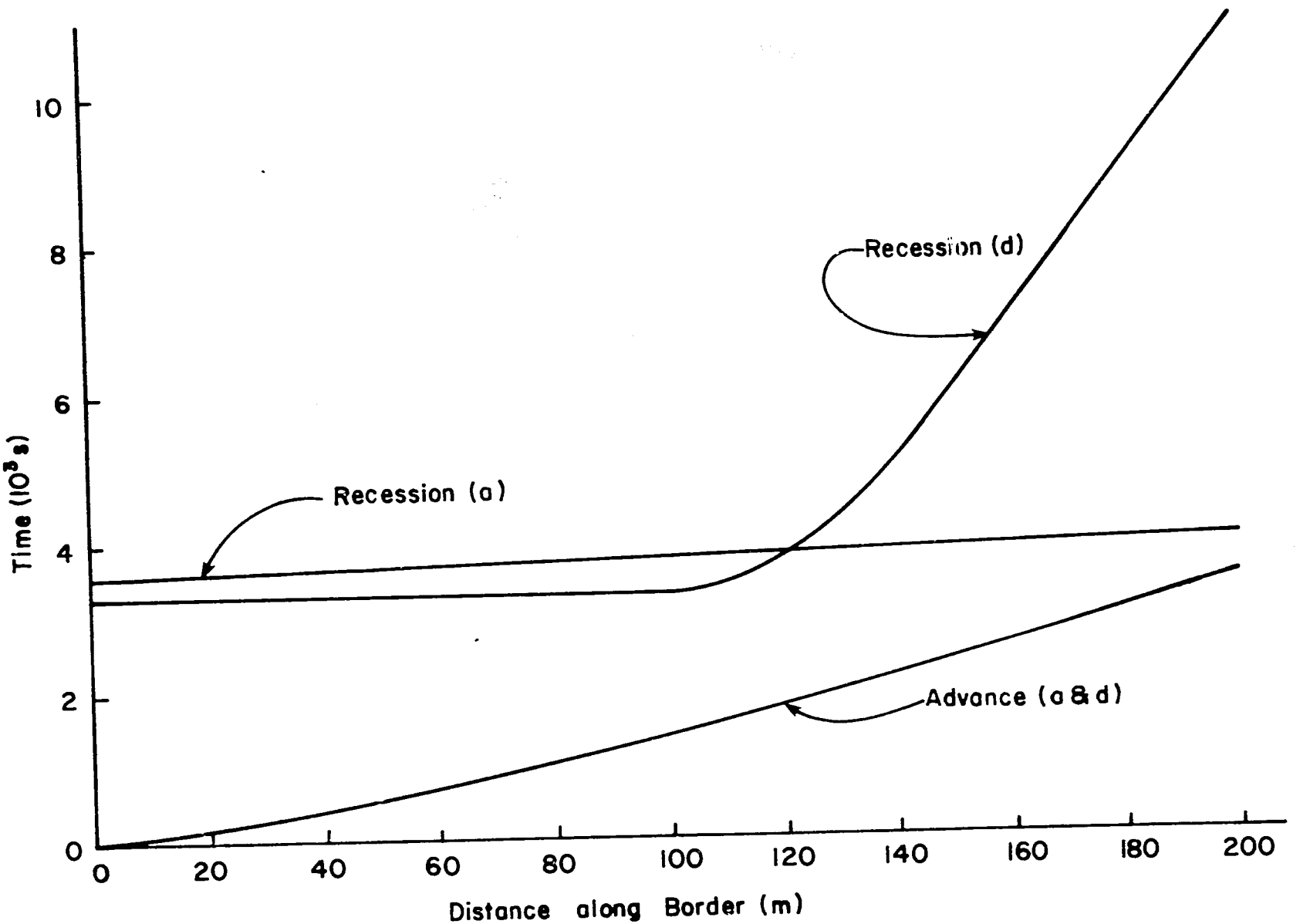


Fig. 4-8. Advance and recession curves for examples of categories a and d.

Table 4-4. Infiltration opportunity time and depth of water infiltrated along the border, for examples of category a and category d.

	x(m)	5	15	25	35	45	55	65	75	85	95	105	115	125	135	145	155	165	175	185	195	$\bar{y}$
Category a (free drainage)	top(s)	3554	3469	3376	3276	3165	3038	2924	2792	2654	2509	2358	2200	2035	1863	1685	1500	1307	1108	902	690	
	y_x(mm)	67.5	66.6	65.4	64.3	63.0	61.6	60.1	58.4	56.7	54.8	52.8	50.6	48.3	45.8	43.1	40.2	37.0	33.6	30.0	25.2	51.2
Category d (ponded)	top(s)	3345	3242	3130	3010	2880	2745	2603	2452	2294	2131	1961	2063	2327	2751	3267	3870	4546	5292	6106	6985	
	y_x(mm)	65.1	63.9	62.6	61.1	59.5	57.8	56.0	54.0	51.9	49.7	47.2	48.7	52.4	57.9	64.2	71.1	78.3	85.7	93.4	101.3	64.1

Table 4-5. Recession distance as a function of time, for example of category d.

$t_r(x)$ (s)	3396	3896	4396	4896	5396	5896	6396	6896	7396	7896	8396	8896	9396	9896	10396	10896
$x_r(t)$ (m)	107.4	121.0	129.8	137.4	144.3	150.6	156.6	162.3	167.7	172.9	177.8	182.6	187.3	191.8	196.2	200.4

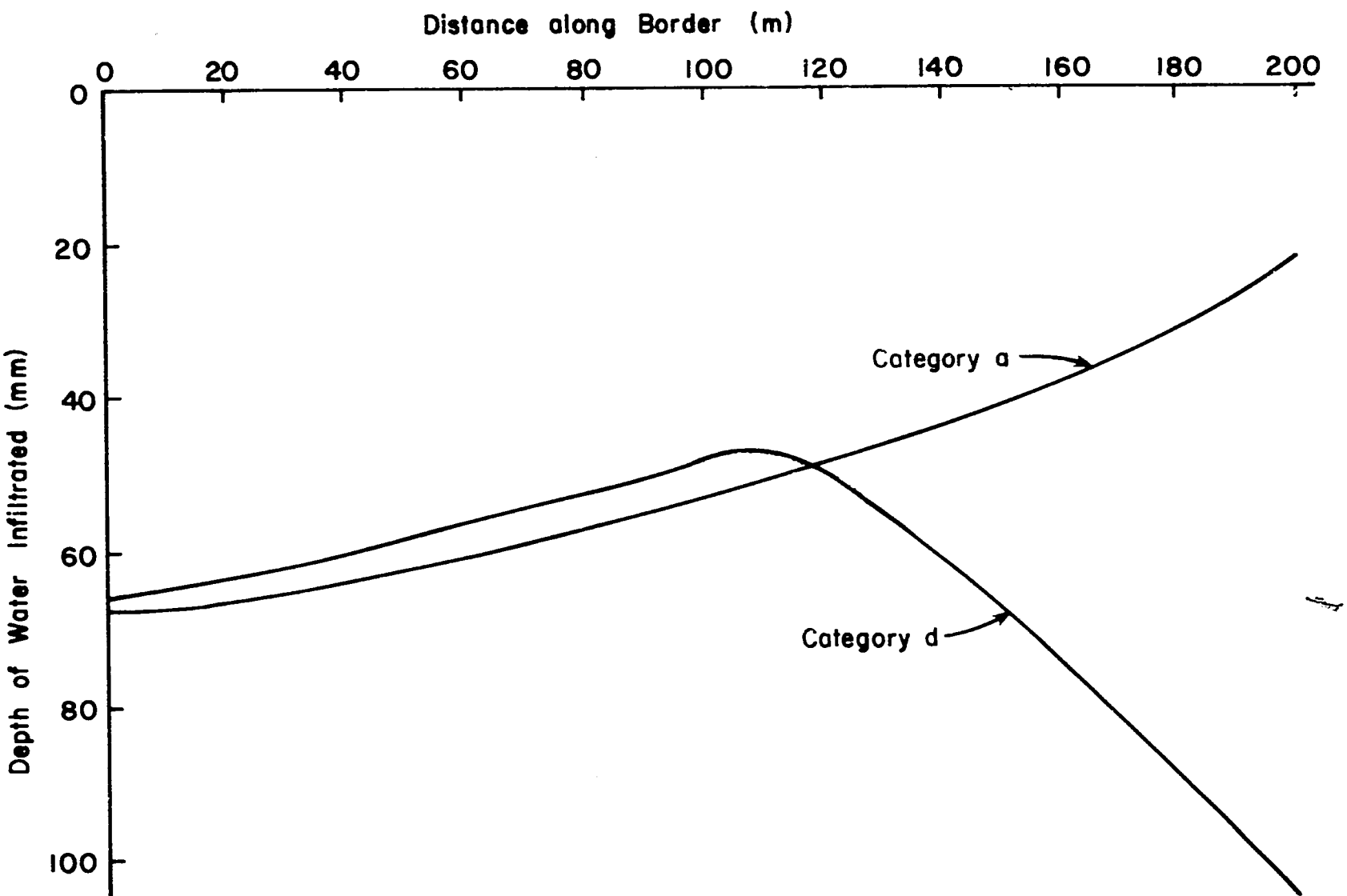


Fig. 4-9. Subsurface water profiles for examples of category a and category d.

mean depth applied, results in a uniformity coefficient of 0.895 (UCC = 0.79). The delivery efficiency, E_d can be calculated as

$$E_d = \frac{D - W}{D} = \frac{12.5 - 2.25}{12.5} = 0.82$$

The irrigation performance can be evaluated by calculating E_p and E_s for various requirements, h_R, from Eqs. 2-12 to 2-15. Plots of E_p and E_s are shown in Fig. 4-10.

The uniformity coefficient for this irrigation is near the satisfactory minimum of 0.90. The uniformity could be improved by increasing the flow rate so that the advance curve was flatter. From Fig. 4-10 it is apparent that the irrigation would be satisfactory (E_p and $E_s \geq 0.80$) if the requirement was in the range 44 mm to 63 mm. If the requirement is less than 44 mm, there will be too much deep percolation, and if the requirement is greater than 63 mm, too much area will be receiving deficit irrigation.

Example (Category d)

The same border and flow conditions will be used as in the example for category a, however, the border will be assumed to be diked at the end.

The advance will be the same as that for category a as given in Table 4-2. New calculations begin at ④ in Fig. 4-2. As the cutoff time t_{co} = 2500 s is less than the total advance time t_{aL} = 3396 s, the system is of category d.

$$d_F = \left[\frac{200 \cdot 0.05}{3396 \cdot 1 \cdot (0.001)^{0.5}}\right] = 0.00931 \text{ m} \qquad \text{(Eq. 4-25a)}$$

From Eqs. 4-5 and 4-25b, t_{ro} and $x_a(t_{ro})$ can be found by trial and error to be 3778 s and 142.7 m, respectively. As $x_a(t_{ro}) < L$, recession starts before advance has finished and the system is of category d_1.

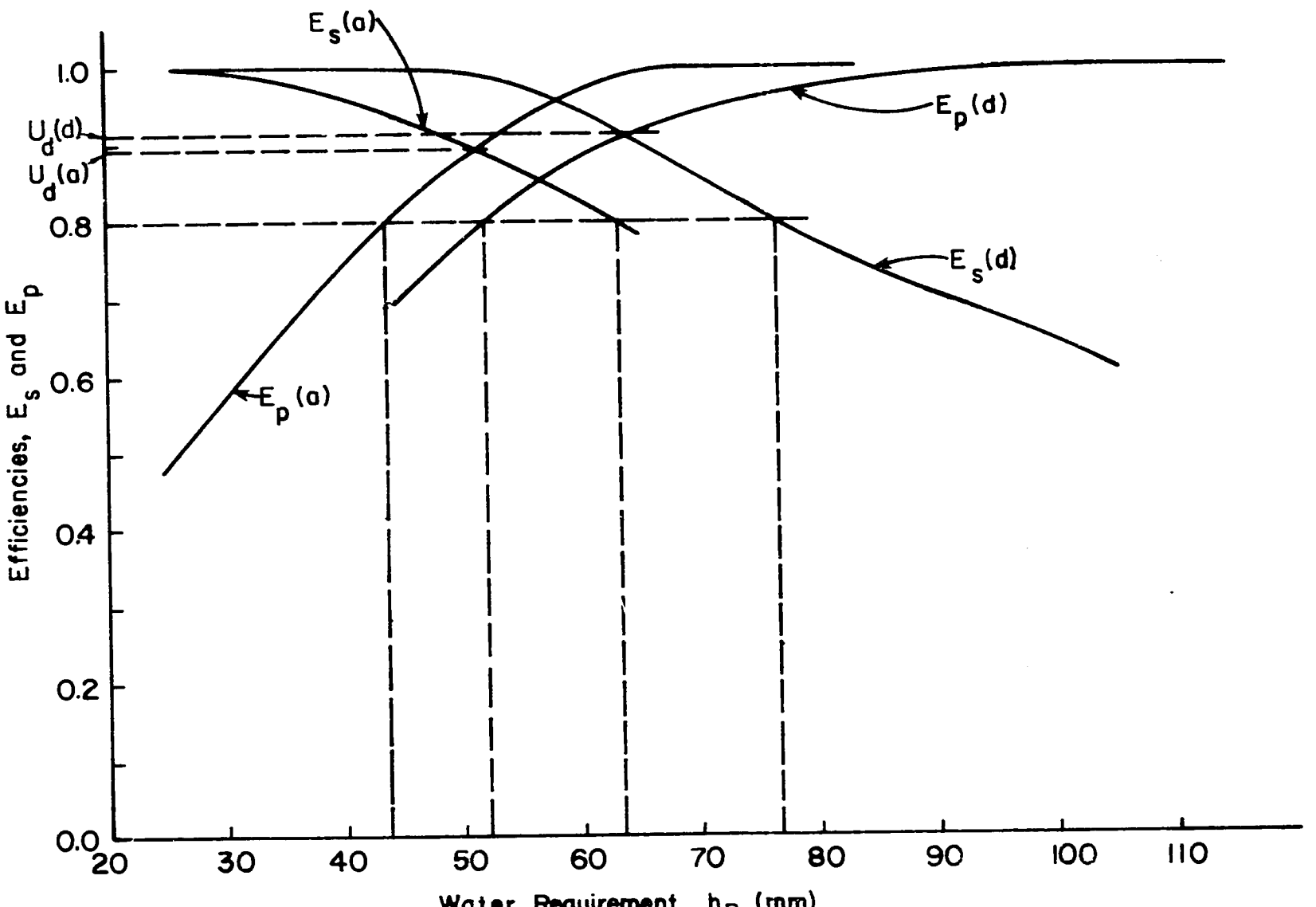


Fig. 4-10. Storage efficiency, E_S, and deep percolation efficiency, E_P, for various requirements for examples of category a (a), and category d (d).

$$x_{ri} = 200 - \left\{\frac{2[12.5 - 200 \cdot 0.625 \cdot 0.0005(3396)^{0.6}]}{0.001}\right\}^{0.5}$$

$$= 107.4 \text{ m} \qquad \text{(Eq. 4-26)}$$

If the recession distance is calculated for every 500s after the beginning of recession, the point where recession is taking place at $t = t_{aL} + \Delta t =$ 3896s can be calculated using Eq. 4-22g for

$$t_a(x) = \frac{107.4 \cdot 3396}{200} = 1824 \text{ s} \qquad \text{(Eq. 4-22g)}$$

and

$$\Delta z_x = 0.0005(3896 - 1824)^{0.6} - 0.0005(3896 - 500 - 1824)^{0.6}$$

$$= 0.00690 \text{ m} \qquad \text{(Eq. 4-22c)}$$

$$\Delta z_L = 0.0005(3896 - 3396)^{0.6} - 0.0005(3896 - 500 - 3396)^{0.6}$$

$$= 0.02081 \text{ m} \qquad \text{(Eq. 4-22d)}$$

Then as $L - x_r(t - \Delta t) = 200 - 107.4 = 92.6$ m

$$x_r(3896) = 200 + \frac{0.02081 - \sqrt{4(0.001)^2(92.6)^2 - 4(0.001)(92.6) \cdot 0.00690 + (0.02081)^2}}{2 \cdot 0.001}$$

$$= 120.7 \text{ m} \qquad \text{(Eq. 4-22e)}$$

Table 4-5 shows the recession distance as a function of time to completion of irrigation. The data are also plotted in Fig. 4-8.

As in the previous example (category a), the advance and recession curves of Fig. 4-8 or data of Tables 4-2 and 4-5 can be used to calculate infiltration opportunity times and depths of water infiltrated. These are also given in Table 4-4 and the subsurface water profile is plotted in Fig. 4-9.

The mean depth infiltrated, as calculated from the data of Table 4-4, is 64.1 mm. This is approximately 2.6% greater than the actual application of 12.5 m^3/m but is well within practical limits.

The uniformity coefficient is U_d = 0.914 (UCC = 0.828). Irrigation performance parameters E_P and E_S, for various water requirements, h_R, are shown in Fig. 4-10.

By ponding the water on the surface, rather than letting it runoff (category a) the uniformity coefficient has been improved slightly. However, the main difference between categories a and d is that the range of water requirements for which the irrigation is satisfactory has been changed to 52 mm to 77 mm from the 44 mm to 63 mm of category a.

Example (Category b)

The same border as used in the example for category d (end diked) will be used, however, the operating conditions will be chosen as

$q = 0.018\ m^3/s \cdot m$

$t_{co} = 1389\ s$

Following the scheme given in Fig. 4-2:

$d_o(t) = 0.1182\ m$ (Eq. 4-3)

$D = 25.0\ m^3/m$ (Eq. 4-16)

Table 4-6 shows $y_o(t)$ and $x_a(t)$ as calculated from Eqs. 4-4 and 4-5 for 200 s increments of time. By linear interpolation, t_{aL} = 1309 s. The advance is also shown in Fig. 4-11.

$V_P(t_{aL}) = 20.4\ m^3/m$ (Eq. 4-6)

As $V_P(t_{aL})$ is positive, there is sufficient water to reach the end of the border. The cutoff time is greater than the advance time, therefore, the cutoff is considered moderate to long.

$V(t_{co}) = 4.54\ m^3/m$ (Eq. 4-15a)

$Y_o = 0.0023\ m$ (Eq. 4-20b)

As Y_o is positive, the cutoff time is considered long and the border is completely ponded (category b).

Table 4-6. Infiltrated depth at upper end of border and advance distance as a function of time, for examples of category b and category c.

Time (s)	0	200	400	600	800	1000	1200	1400	1600	1800	2000
y_o (t) (mm)	0.0	12.0	18.2	23.2	27.6	31.5	35.2	38.6	41.8	44.9	47.8
Category b-x_a(t) (m)	0.0	35.3	68.0	99.0	128.8	157.5	185.3	212.3			
Category c-x_a(t) (m)	0.0	27.0	51.4	74.1	95.6	116.1	135.7	154.6	172.8	190.5	207.6

Table 4-7. Recession distance as a function of time, for example category b.

t_r(x) (s)	1462	3462	5462	7462	9462	11462	13462	15462	17462	19462	21462	23462	25462	27462
x_r(t) (m)	0	32.7	55.4	74.4	91.2	106.6	120.9	134.3	147.0	159.1	170.7	181.8	192.6	203.0

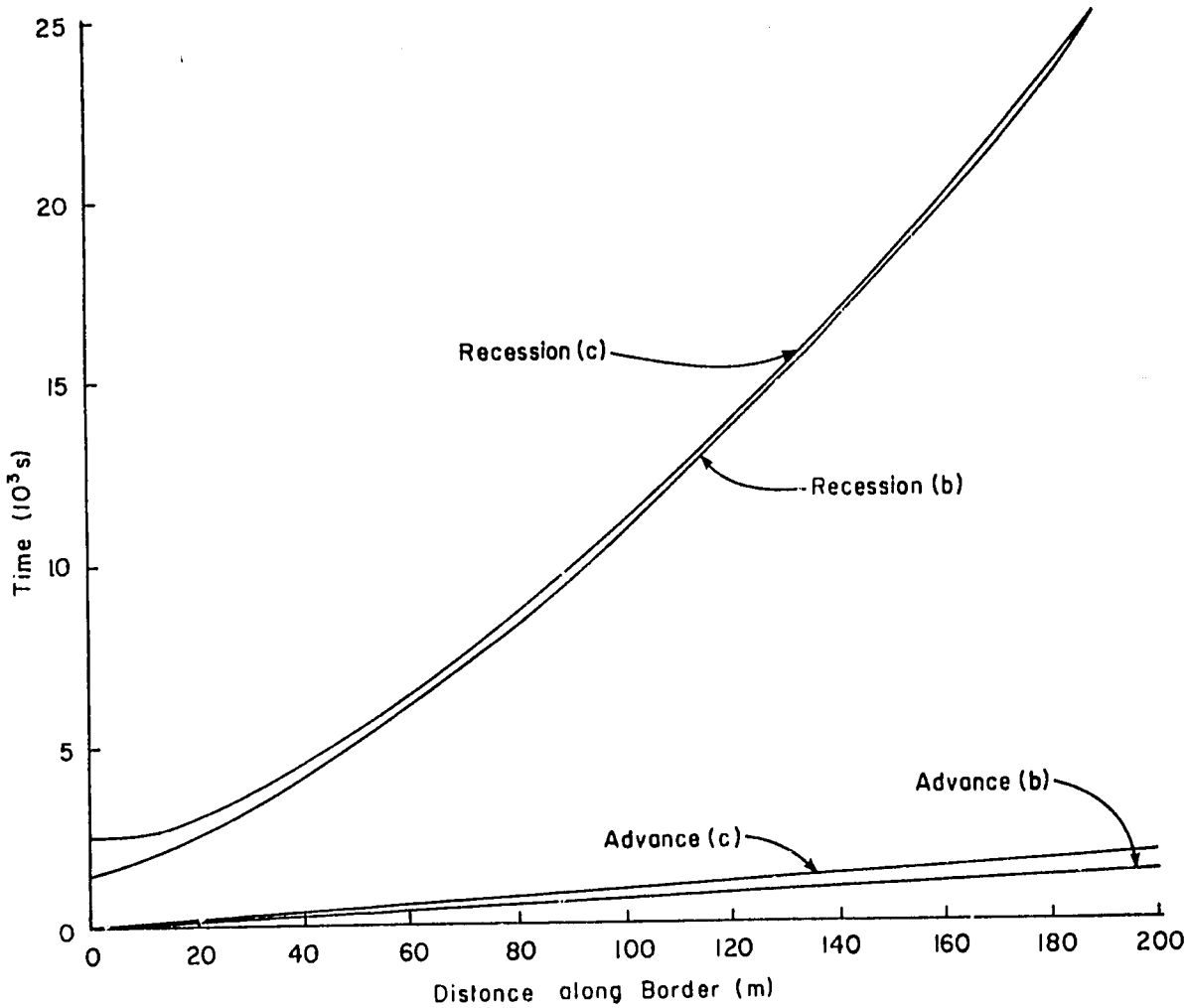


Fig. 4-11. Advance and recession curves for examples of categories b and c.

$$V(t_{ro}) = 25.0 - \frac{0.001(200)^2}{2} = 5.0 \text{ m}^3/\text{m} \qquad \text{(Eq. 4-21b)}$$

By trial and error, using Eq. 4-21c, t_{ro} = 1462s. The recession is calculated using Eqs. 4-22c to 4-22g, with the results being listed in Table 4-7.

The advance and recession data of Fig. 4-11 or data of Tables 4-6 and 4-7 can be used to calculate infiltration opportunity times and depths of water infiltrated. These are given in Table 4-8 and the subsurface water profile is plotted in Fig. 4-12.

The mean depth infiltrated, as calculated from the data of Table 4-8, is 126.9 mm. This is approximately 1.5% greater than the actual application of 25.0 m^3/m, but is once again well within practical limits.

The uniformity coefficient is U_d = 0.819 (UCC = 0.638), a value which is too low for efficient irrigation. Irrigation performance parameters, E_p and E_s, for various water requirements, h_R, are shown in Fig. 4-13. The requirement range for which this irrigation would be satisfactory, as far as E_p and E_s are concerned, is limited to 121 mm to 135 mm. One could assume that the entire irrigation could be improved by reducing the flow rate and increasing the cutoff time so that the same amount of water is applied, but it takes a longer time to reach the end of the border. This is the situation covered by the following example.

Example (Category c)

Once again the same end diked border will be used, however, the flow rate will be reduced and the cutoff time increased to:

$$q = 0.01 \text{ m}^3/\text{s}\cdot\text{m}$$

$$t_{co} = 2500 \text{ s}$$

The result is that the same quantity of water is applied as in the example for category b. That is

Table 4-8. Infiltration opportunity time and depth of water infiltrated along the border, for examples of category b and category c.

	x (m)	5	15	25	35	45	55	65	75	85	95	105	115	125	135	145	155	165	175	185	195	$\bar{y}$
Category b	t_{op}(s)	1740	2294	2849	3466	4288	5110	6095	7091	8213	9377	10611	11929	13301	14730	16237	17806	19431	21115	22858	24658	
	y_x(mm)	44.0	51.9	59.1	66.5	75.6	84.0	93.3	102.2	111.6	120.8	130.1	139.6	149.0	158.4	168.0	177.5	187.1	196.7	206.2	215.8	126.9
Category c	t_{op}(s)	2463	2437	3083	3722	4423	5280	6160	7167	8213	9342	10543	11796	13122	14519	15968	17476	19041	20674	22363	24108	
	y_x(mm)	54.2	53.8	62.0	69.4	77.0	85.6	93.9	102.9	111.6	120.6	129.6	138.7	147.8	157.1	166.3	175.6	184.8	194.2	203.6	212.9	127.1

Table 4-9. Recession distance as a function of time, for example of category c.

$t_r(x)$ (s)	2500	4500	6500	8500	10500	12500	14500	16500	18500	20500	22500	24500	26500	28500
$x_r(t)$ (m)	14.3	42.1	63.4	81.6	98.0	113.0	127.0	140.2	152.7	164.7	176.2	187.2	197.9	202.1

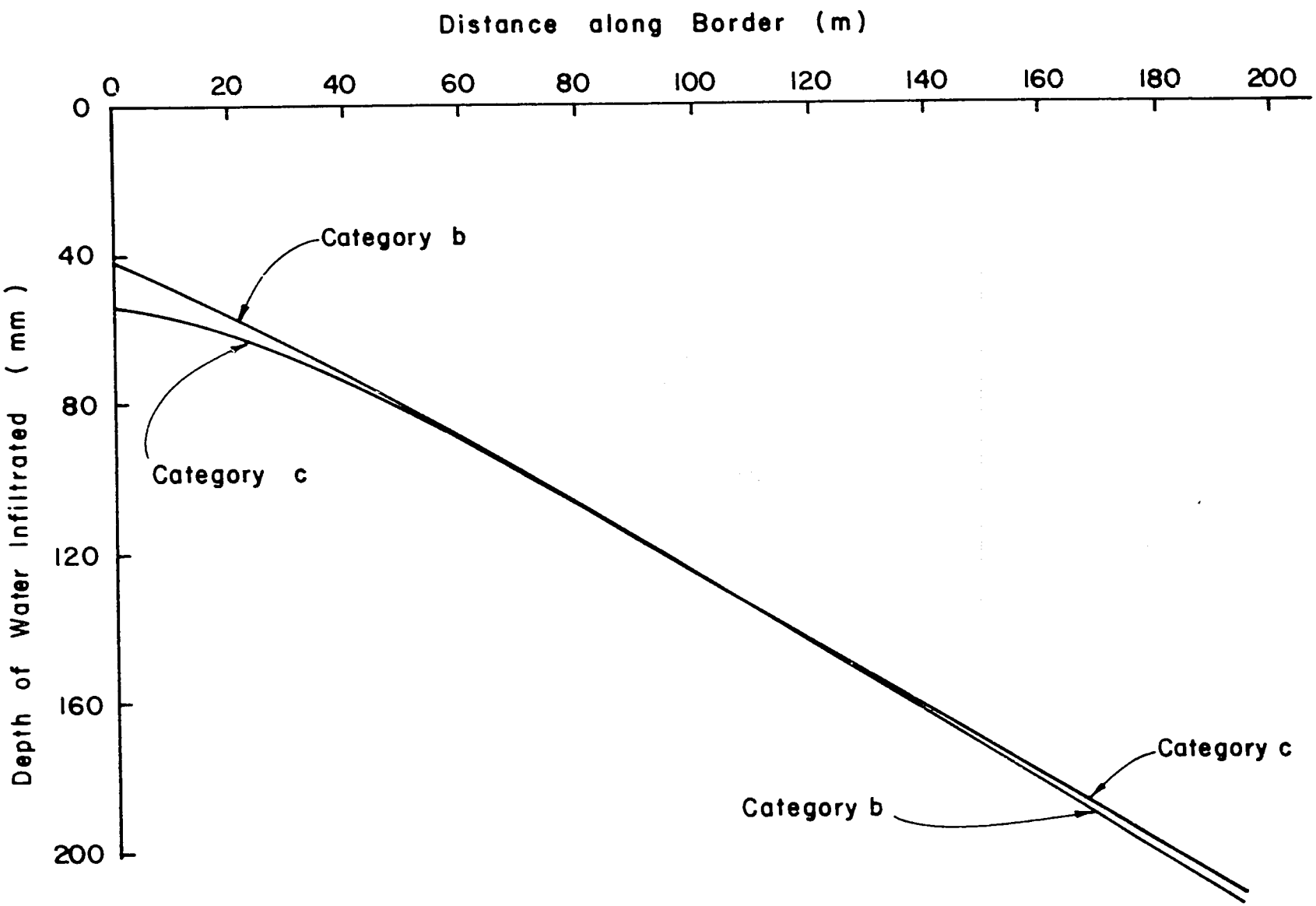


Fig. 4-12. Subsurface water profiles for examples of category b and category c.

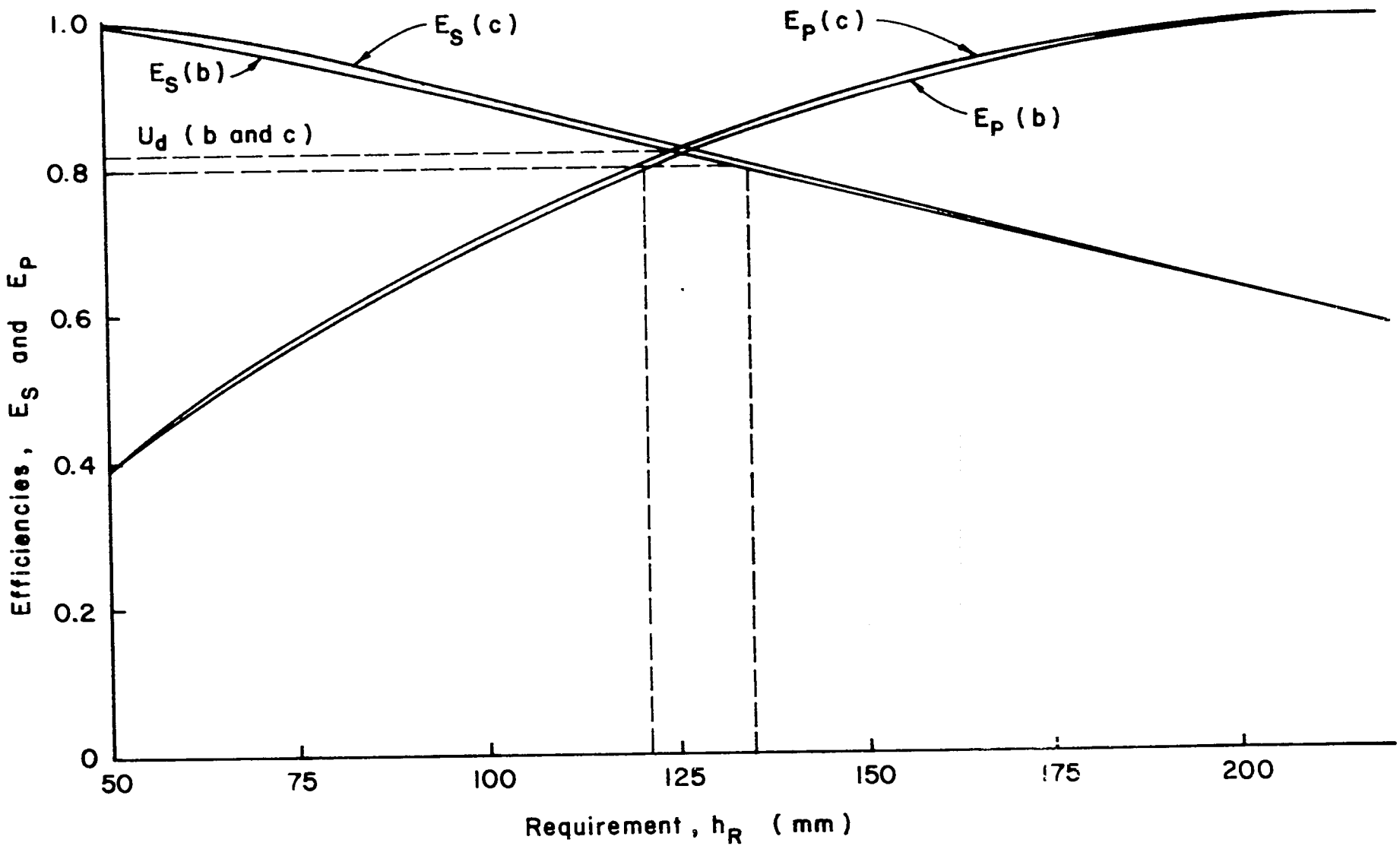


Fig. 4-13. Storage efficiency, E_S, and deep percolation efficiency, E_P, for various requirements for examples of category b (b), and category c (c).

$D = 25.0\ m^3/m$ (Eq. 4-16)

Following the scheme given in Fig. 4-2:

$d_o(t) = 0.0831\ m$ (Eq. 4-3)

Table 4-6 shows $y_o(t)$ and $x_a(t)$ as calculated from Eqs. 4-4 and 4-5 for 200 s increments of time. By linear interpolation, $t_{aL} = 1911s$. The advance is shown in Fig. 4-11.

$V_P(t_{aL}) = 19.2\ m^3/m$ (Eq. 4-6)

$V(t_{co}) = 7.76\ m^3/m$ (Eq. 4-15a)

$Y_o = -0.0138\ m$ (Eq. 4-20b)

As Y_o is negative, the border is only partially ponded (category b).

$$x_{ri} = 200 - \left\{\frac{2[25.0 - 7.76]}{0.001}\right\}^{0.5} = 14.3\ m \qquad \text{(Eq. 4-23)}$$

The recession is calculated using Eqs. 4-22c to 4-22g and given in Table 4-9.

The advance and recession data of Fig. 4-11 or data of Tables 4-6 and 4-9 can be used to calculate infiltration opportunity times and depths of water infiltrated. These are given in Table 4-8 and the subsurface water profile is plotted in Fig. 4-12.

The mean depth infiltrated, as calculated from the data of Table 4-8, is 127.1 mm. This is approximately 1.7% greater than the actual application of 25.0 m^3/m.

The uniformity coefficient is $U_d = 0.827$ (UCC = 0.654), once again an unsatisfactory value. Irrigation performance parameters, E_P and E_S are shown in Fig. 4-13.

It is apparent from Figs. 4-12 and 4-13 that the reduced flow rate and increased cutoff time have done little to improve the irrigation. If this amount of water is to be applied to a soil of this type, the border must have a shallower slope.

APPENDIX

MODELS AVAILABLE FOR BORDER IRRIGATION

Empirical and Semi-empirical Models

Wilke and Smerdon (1965). A Solution of the Irrigation Advance Problem. Provides the advance phase in which x is related to t. Regression coefficients are used, as derived by calcuation of the Philip and Farrel equation.

Sastry and Agarwal (1972). Prediction of Irrigation Advance Function by Dimensional Analysis. Provides advance phase.

Singh and Chauhan (1972). Shape Factors in Irrigation Water Advance Equation. Provides advance phase as calculated with shape factors. The shape factors were derived using empirical equations for the water profile and mass conservation.

Wu (1972). Recession Flow in Surface Irrigation. Provides recession in which the distance and the time are related through a recession factor.

Jobling and Turner (1973). Physical Model Study of Border-Strip Irrigation. Provides surface profile, advance and recession.

Karmeli (1978). Distribution Patterns and Losses for Furrow Irrigation. Provides power function to define the water distribution ($y = f + ga^b$). The coefficients f, g, b are determined by fitting to data obtained by field tests or theoretically by other models.

Mathematical Models for the Solution of the Continuity and Momentum Equations of Open Channel Flow

Chen (1970). Surface Irrigation using Kinematic-Wave Method. Provides the advance phase.

Bassett (1972). Mathematical Model of Water Advance in Border Irrigation. Provides the advance phase.

Kincaid, Heerman and Kruse (1972). Hydrodynamics of Border Irrigation Advance. Provides description of time variation of surface depths and rate of advance. With modifications, can be used to calculate recession with specified boundary conditions at the downstream.

Smith (1972). Border Irrigation Advance and Ephemeral Flood Waves. Provides the advance phase and surface water profile.

Sakkas and Strelkoff (1974). Hydrodynamics of Surface Irrigation - Advance Phase. Provides the advance phase.

Bassett and Fitzsimmons (1976). Simulating Overland Flow in Border Irrigation. Provides description of water movement over and into the soil surface for either free drainage or ponding downstream.

Katapodes and Strelkoff (1977). Dimensionless Solutions of Border Irrigation - Advance. Provides the advance phase.

Strelkoff and Katapodes (1977). Border Irrigation Hydraulics with Zero Inertia. Provides solution of advance and recession.

Mass Conservation Models

Fok and Bishop (1965). Analysis of Water Advance in Surface Irrigation. Provides advance phase with a correction factor which is derived from empirical advance function.

Hart, Bassett and Strelkoff (1968). Surface Irrigation Hydraulics Kinematics. Provides the advance phase derived from volume balance consideration with shape factors.

Strelkoff (1977). Algebraic Computation of Flow In Border Irrigation. Provides advance and recession for free drainage and end diked cases. Calculations of runoff.

Peri, Skogerboe and Norum (1979). Simplified Model for the Evaluation and Improvement of Basin Irrigation. Subsurface profile approximated by two linear sections.

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Bassett, D. L., 1972. Mathematical Model of Water Advance in Border Irrigation. Trans. ASAE 15(5):992-995.

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