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A simple model for the determination of the infiltrated water distribution under basin irrigation is also presented. The model can be applied to both level and sloped basins. Comparison with other more complicated models shows satisfactory agreement in the distributions.

A complete infiltrated water distribution under basin irrigation can be approximated from limited field observations, which are described. However, as only two of a possible four parameters are sufficient to define each distribution, the selection of these parameters should be made on the basis of circumstances under which the field observations were made. General guidelines are given for the selection of the most reliable parameters. The main advantage of the method is its simplicity in both the field measurements required and in the data analysis. The procedure suggested is suitable for most practical cases, especially as a preliminary evaluation procedure when detailed studies are not warranted.

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EVALUATION AND IMPROVEMENT OF BASIN IRRIGATION

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

Water Management Research Project
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Gideon Peri, Gaylord V. Skogerboe,
and Donald I. Norum

ABSTRACT

A comprehensive definition and description of basin irrigation is given. A procedure is outlined for the design and evaluation of basin irrigation systems, showing the interactions between the various basin characteristics, the operational parameters, the management parameters, and the performance parameters. A general model is discussed by considering the various functions upon which it must be based (infiltration, advance, recession).

A simple model for the determination of the infiltrated water distribution under basin irrigation is also presented. It is shown that the actual distribution can be determined from limited field data. The model can be applied to both level and sloped basins. Comparison with other more complicated models shows satisfactory agreement in the distributions.

A complete infiltrated water distribution under basin irrigation can be approximated from limited field observations. Field procedures are described that will provide the parameters required to determine the distributions. However, as only two of a possible four parameters are sufficient to define each distribution, the selection of these parameters should be made on the basis of circumstances under which the field observations were made. General guidelines are given for the selection of the most reliable parameters. The main advantage of the method is its simplicity in both the field measurements required and in the data analysis. The procedure suggested is suitable for most practical cases, especially as a preliminary evaluation procedure when detailed studies are not warranted.

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NOMENCLATURE

A	=	constant in Kostiakov infiltration equation.
A_i	=	areas defined in Fig. 3-2 (i=1, 2, 3, 4).
A_T	=	area of a basin.
AMC	=	available moisture deficiency as volume fraction.
a	=	area fraction in cumulative water distribution.
a	=	area of basin represented by a single measurement station.
a	=	constant in empirical advance equations.
B	=	exponent in Kostiakov infiltration equation.
b	=	exponent in empirical advance equations.
b	=	exponent in the power function distribution.
C	=	constant in modified Kostiakov infiltration rate equation.
CU	=	units coefficient (1 for m-s, 1.49 for ft-sec).
D	=	wetted depth of soil.
d	=	average depth of water on the surface.
d_o	=	depth of flow at upstream end.
d_x	=	depth of water on soil surface at point x.
E_P	=	deep percolation efficiency.
E_S	=	storage efficiency.
F	=	Kiefer shape factor.
FC	=	moisture content at field capacity as mass fraction.
f	=	constant in the nondimensional power function distribution.
f'	=	constant in the dimensional power function distribution.
g	=	constant in the nondimensional power function distribution.
g'	=	constant in the dimensional power function distribution.
H	=	$y/\bar{y}$ = nondimensional depth of water infiltrated.
$\bar{H}$	=	nondimensional mean water application = 1.

H_C = nondimensional mean water deficit relative to $\bar{H}$.

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

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

H_P = nondimensional mean depth of water in surface depressions.

h_B = mean water deficit relative to requirement, h_R.

h_C = mean water deficit relative to mean application, $\bar{y}$.

h_P = mean depth of water in surface depressions.

h_R = required depth of application of water.

I = infiltration rate.

K = shape factor in two linear section (TLS) distribution.

k = constant in Philip's infiltration function.

k = AB = constant in Kostiakov infiltration rate equation.

L = length of basin.

L_n = a nondimensional advance distance.

m = $\bar{y}$ (1-S)/h_P.

n = B-1 = exponent in Kostiakov infiltration rate equation.

n = number of measurement stations in basin.

n = roughness coefficient in Manning's equation.

P = fraction of the area with surface depressions.

PMC = moisture content before irrigation as mass fraction.

P_1 = weighting factor for T.

P_2 = weighting factor for T_{est}.

p = number of stations receiving less than the mean application or required application.

Q = flow rate into basin.

q = flow rate into basin per unit width.

R = ratio of T to T_{est}.

r_y = surface depth shape factor.

r_z = subsurface depth shape factor.

S = ratio of required volume of water to applied volume.

S_o = slope of basin.

S_{op} = optimum slope.

s = constant in Philip's infiltration function.

T = actual volume of water applied to the basin.

T_{est} = estimated volume of water infiltrated.

T_n = a nondimensional time.

TLS = two linear section distribution.

t = time.

t_a = time of advance of water to a station in the basin.

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

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

t_b = infiltration opportunity time at the end of the basin x=L.

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

t_d = time at which surface at x=0 becomes exposed (depletion time).

t_i = time water disappeared on area fraction a_i.

t_{op} = infiltration opportunity time.

t_r = time of recession of water at a station in the basin.

t_{rL} = $t_r(L)$ = recession time at x=L (end of irrigation).

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

U_d = distribution uniformity.

UCC = Christiansen's uniformity coefficient.

V = T/W = final volume of water infiltrated per unit width.

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

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

W = width of basin.

X = fraction of the area not ponded at cutoff time.

X = fraction of the area, as given by a distribution function, receiving more than the mean application.

X_R = fraction of the area receiving excess application.

x = distance from inlet end of basin.

x_m = an intermediate distance.

Y_L = depth of water on surface at x=L at cutoff time.

Y_o = depth of water on surface at x=0 at cutoff time.

y = depth of water infiltrated

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

y_{ad} = adjusted depth of water infiltrated.

y_{KL} = final depth of water infiltrated at x=KL.

$y_{KL}(t)$ = $y_x(t)$ for x=KL.

y_L = final depth of water infiltrated at x=L.

$y_L(t)$ = $y_x(t)$ for x=L.

y_{max} = maximum depth of water infiltrated.

y_{min} = minimum depth of water infiltrated.

y_o = final depth of water infiltrated at x=0.

$y_o(t)$ = $y_x(t)$ for x=0.

$\bar{y}(t)$ = mean depth of water infiltrated at time t.

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

$\bar{y}_{typ}$ = mean depth of water infiltrated, from y_{typ} values.

y_x = $y_x\left[t_r(x)\right]$ = final depth of water infiltrated at point x.

$y_x(t)$ = depth of water infiltrated at point x by time t.

Z = difference in elevation between two ends of basin.

z = cumulative infiltrated depth of water

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

Δ = $y_o(t) - y_L(t)$.

Δ_{co} = difference in infiltrated depths at two basin ends at cutoff.

Δ_f = final difference in infiltrated depths at two basin ends.

Δt = a time period.

γ = soil bulk density as mass per unit volume.

Chapter 1

CHARACTERISTICS OF BASIN IRRIGATION

1. General Description

Surface irrigation methods exhibit two common characteristics: the advance of water over the soil surface is due to gravity where the water is in direct contact with the soil surface, and the absorption of water into the soil is governed by the soil properties, mainly the infiltration.

Surface irrigation methods can be classified into two major groups:

a. Flood irrigation, where the water covers most or all of the irrigated area, and water is infiltrated only vertically over the whole irrigated area.

b. Furrow irrigation, where small channels or furrows are used to convey the water over the soil surface in small individual parallel streams. Infiltration occurs through the sides and bottom of the furrow where the infiltrated water moves both laterally and vertically to moisten the plant root zone. The surface flow of water is confined in the furrow a is dependent on the hydraulic properties of the furrow cross section.

Furrows differ in furrow size (e.g., corrugations) and pattern (e.g., block systems). Flood irrigation methods differ because of slope, discharge, and field size (basin, border strip, border ditch, etc.), however, delineation among various systems is somewhat arbitrary.

To define basin irrigation and to differentiate it from border irrigation, several characteristics will be considered:

a. dike at the low end (closed or open),

b. slope along the run (level, slight, considerable),

c. inlet stream size (small, medium, large),

d. total depth of application (small, medium, high).

These four characteristics may yield as many as 54 possible variations, however, several are impractical and thus limit the feasible combinations (see Fig. 1-1). Of the most feasible combinations of characteristics, four may be defined as those combinations that relate best to the definition of basin (indicated by B in Fig. 1-1):

a. closed end + level slope + medium discharge + high depth of application,

b. closed end + level slope + large discharge + medium depth of application,

c. closed end + level slope + large discharge + high depth of application,

d. closed end + slight slope + large discharge + high depth of application.

The other feasible combinations may best relate to the definition of various types of border irrigation.

Based on the above definition of basin irrigation, further description of this irrigation system is given. The basin is an area completely surrounded by dikes or bunds. The entire desired amount of water is applied in a relatively large stream size so that the basin is quickly covered with water. The water is then ponded in the basin area until absorbed by the soil. Irregularities in the soil surface, which result in peaks (high spots) and depressions (low spots) within the basin, are a major factor in determining the water distribution. Essentially there is no runoff during basin irrigation (unless the dike or bund is overtopped or there is leakage through the dikes or bunds). Basin may be as small as a few square meters around a tree or as large as several hectares. The shape of

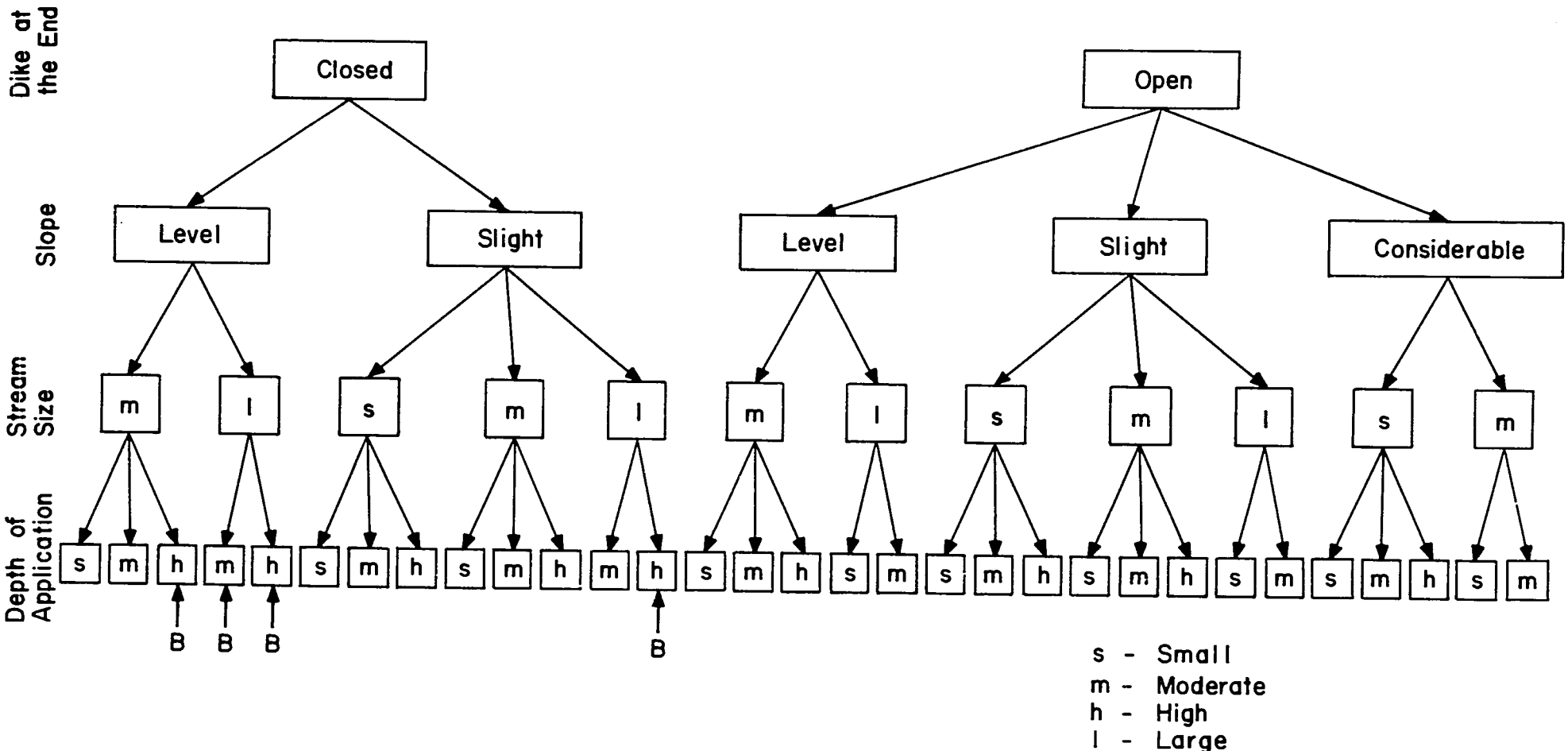


Figure 1-1. Feasible combinations of major characteristics of flood irrigation.

the basin is preferably a regular shape, a square or rectangle. However, basins can also be adapted to local conditions (topography, parcellation, etc.) that require irregular shapes. Since basin irrigation is actually the infiltration of water ponded in the basin, basin irrigation is adaptable to crops that can stand being submerged for short periods, and for pre-irrigation or leaching purposes.

Two classes of basins can be defined:

a. Regular basins (Fig. 1-2a). Regular basins are level borders with closed ends. These basins have regular shapes with a dominant flow direction during the phase in which the water flows to cover the basin surface. The advancing water front is more or less perpendicular to the flow direction. The basin is similar in shape to a level border and has a well recognized water advance phase as in level borders. Most commonly, the difference between regular basins and closed level borders is in the ratio of length, L, to width, W, where $\frac{L}{W}$ is smaller for basin irrigation. However, there is no definite definition of $\frac{L}{W}$ ratio for which the irrigation is regarded as basin. It is common for basin irrigation to have $1.0 \leq \frac{L}{W} \leq 10.0$. In contrast to level borders, after the advance phase (when the basin is completely covered with water) the inflow stream continues so that water is ponded over the basin surface. The irrigation continues until the ponded water is absorbed by the soil. Most models that have been developed for the solution of the water distribution problem for border irrigation (the differential equations of flow or mass conservation equations) can be related also to the specific case of regular basins. However, simplified approaches based on mass conservation may enable practical and accurate solutions.

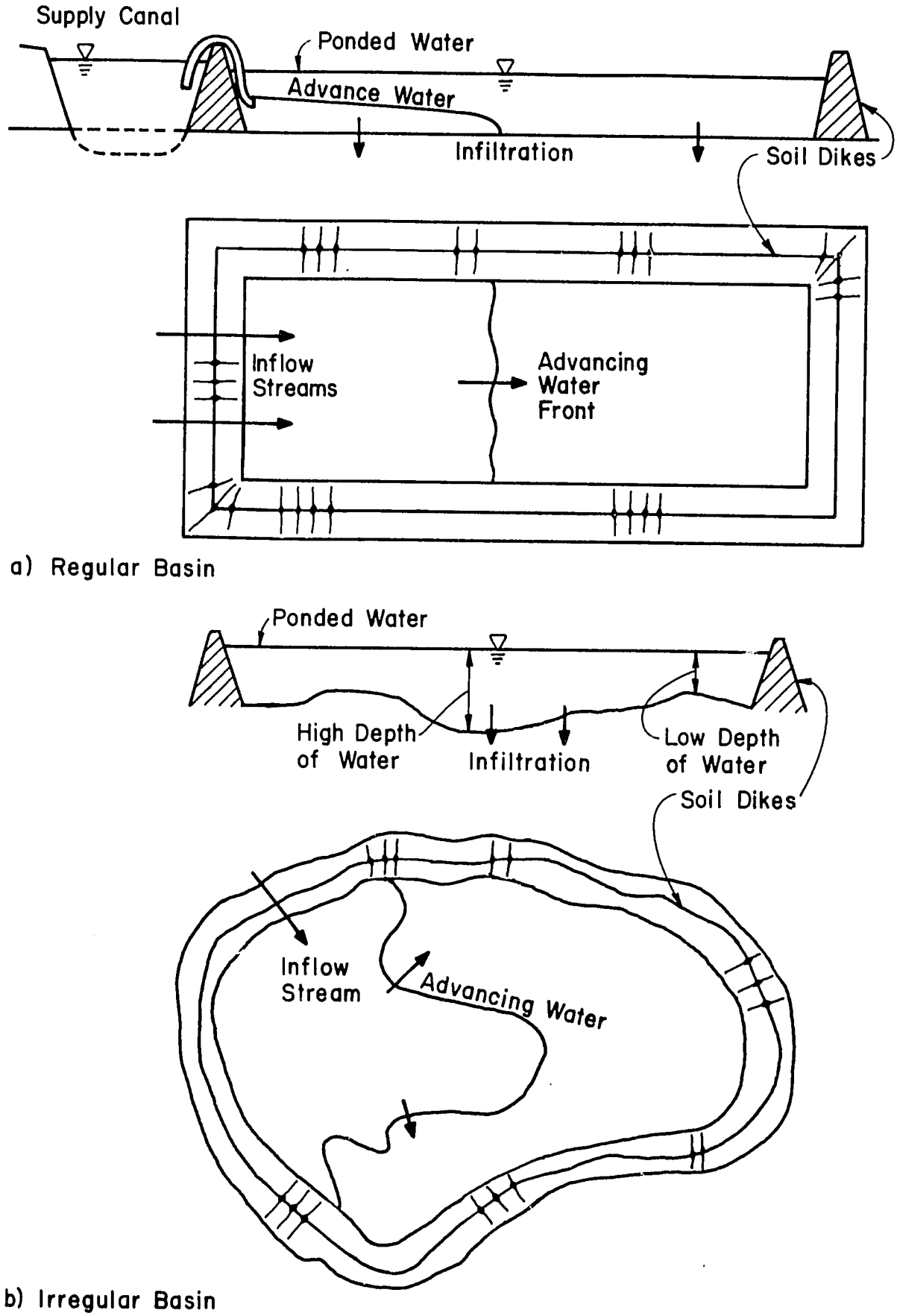


Figure 1-2. Schematic description of regular and irregular basins.

b. Irregular basins (Fig. 1-2b). Irregular basins have irregular shape and slope. During the first phase of irrigation, when water flows over the basin surface, there is no predominant flow direction. The advance of water is governed by the basin microtopography and the location and size of the inlet streams. The advancing water front is irregular.

The main features needed to achieve high quality basin irrigation are:

a. proper dimensions (shape and size),

b. proper grading and smoothening of soil surface,

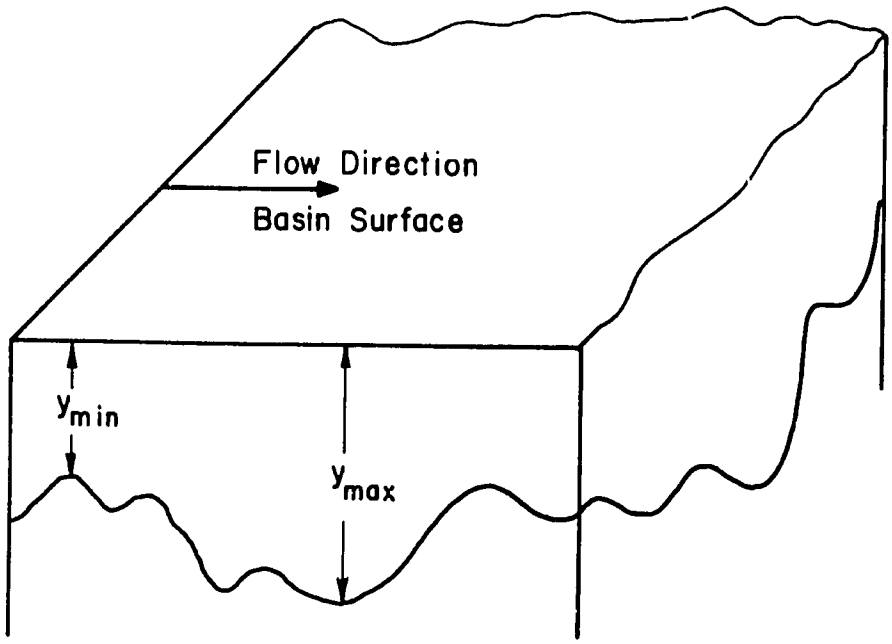
c. uniform soil (mainly uniform infiltration rate) within the basin, and

d. a large enough stream size of water to cover the basin rapidly.

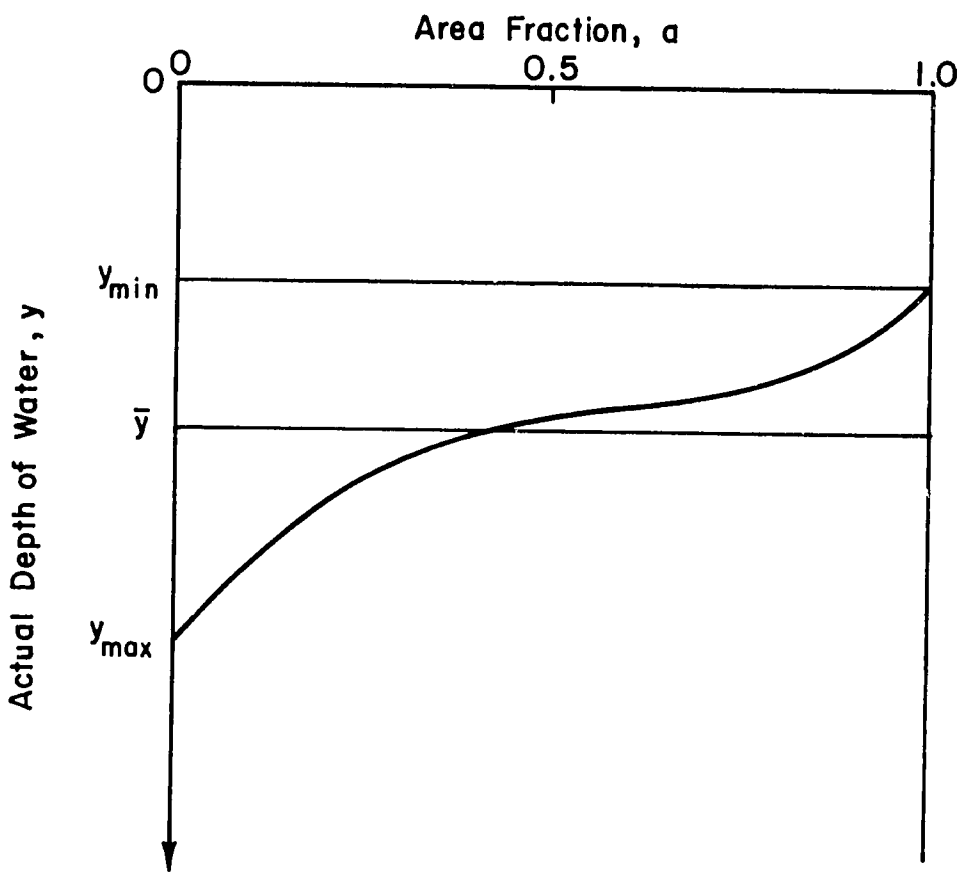
2. Water Distribution Patterns--Basin Irrigation

With irregular basins, the basin does not have a major axis of flow and the water distribution pattern tends to be irregular. Water distribution is described by the actual water distribution (see Part I, Chapter 5), and/or the cumulative frequency distribution of water depths (either dimensional or nondimensional water depths). A typical water distribution pattern for this case is shown in Fig 1-3.

With regular basins, and a well graded soil surface, during the advance phase the water front moves along a well-defined axis, and the water distribution pattern is assumed to be uniform across the basin width so that the actual water distribution is described by the water profile along the flow direction axis (the length of the basin). The actual water distribution profile has a typical shape with a higher depth at the flow inlet end and lower depth at the lower end of the basin. Representation of the water profile in this case is usually done by the actual depth of water distribution along the basin axis or by the cumulative frequency distribution of



a) Water Distribution as Related to the Irrigated Area



b) Cumulative Frequency Distribution of Water Depths

Figure 1-3. Water distribution under irregular basin irrigation.

the water depths (Fig. 1-4). Once the water distribution pattern is established, and the required depth of application, h_R, is established, the irrigation performance can be evaluated by the efficiencies described in Part I, Chapter 5.

Examples of two water distribution patterns under regular basin irrigation and their irrigation performances are given in Table 1-1 and Fig. 1-5 (from Slabbers, 1971).

The representation of water distribution by cumulative frequency is advantageous (Fig. 1-5) since the uniformity of the two distributions can easily be compared.

Based on the water distribution, the deep percolation efficiency, E_p, the storage efficiency, E_s, and the distribution uniformity, U_d, are calculated for each case. The efficiencies and the irrigation performance categories are shown in Table 1-2 and Fig. 1-6 for several required depths of application.

3. Relationships Between Basin Irrigation Performance and System Parameters

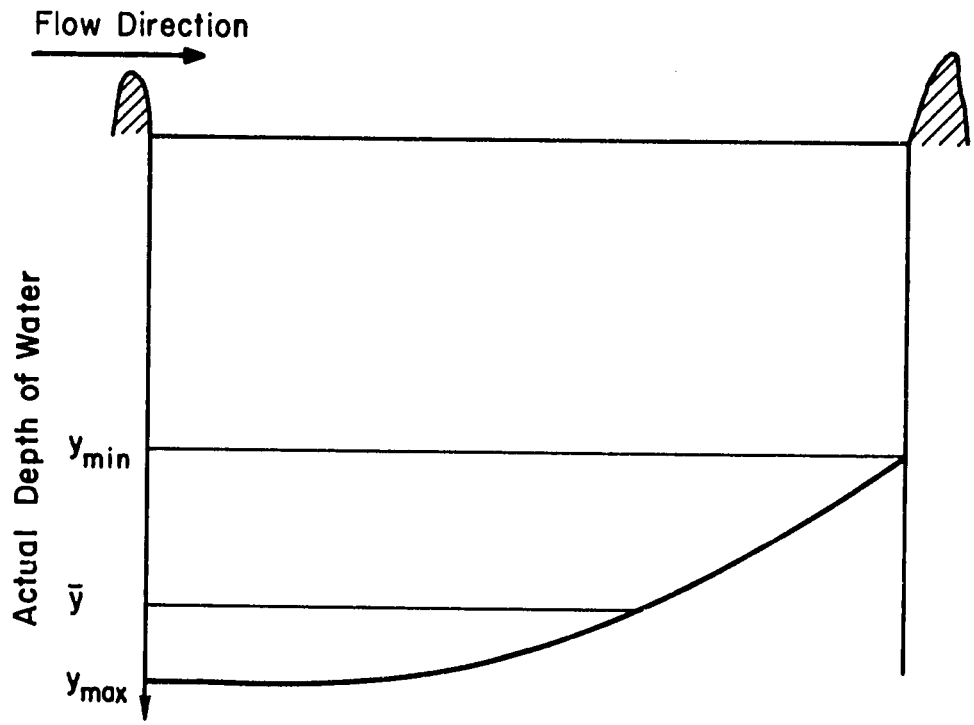
The evaluation of basin irrigation, from the water distribution, requires the use of a model that can relate the system parameters to the irrigation performance. To do this, the model should predict the water distribution pattern from which the efficiencies can then be calculated. The model should also aid in establishing when improvement is required and how this improvement can be accomplished.

The various relationships involved are outlined in Fig. 1-7 and their effect on (a) water distribution within the basin, (b) average depth of application, (c) total water quantity applied to the basin, and (d) economical inputs are shown.

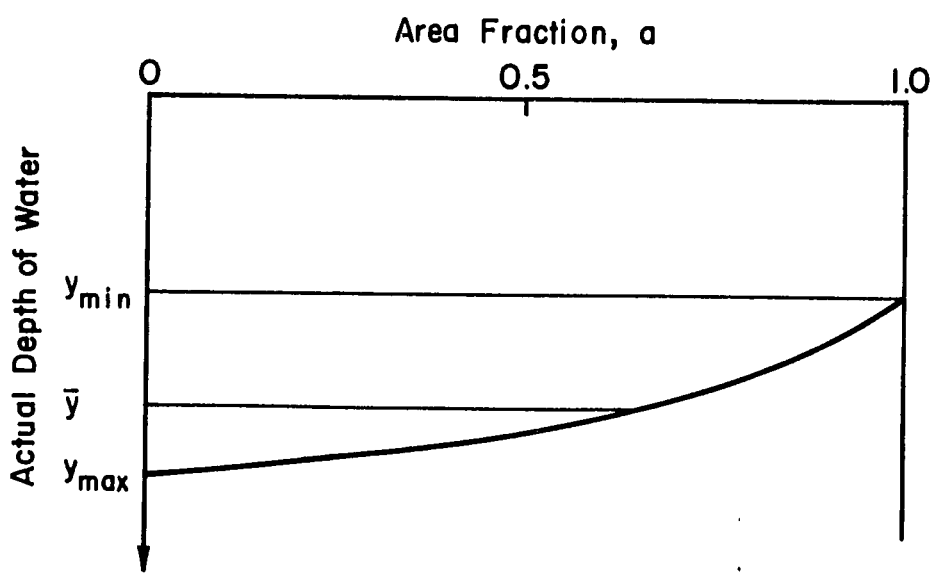
Table 1-1. Water distribution in two regular basins.[1]

Basin B	Length - m	5	10	15	20	25	30	35	40	45	50
	Infiltrated water depth - mm	124	122	119	115	108	99	88	76	62	48
	Fractional area	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
	Cumulative depth - mm	124	122	119	115	108	99	88	76	62	48
	Nondimensional cumulative depth	1.29	1.27	1.24	1.20	1.12	1.03	0.92	0.79	0.65	0.50
	Average depth, mm	96.1									
Basin E	Length - m	2.5	5.0	7.5	10	12.5	15	17.5	20	22.5	25
	Infiltrated water depth - mm	112	111	109	107	105	101	96	91	84	78
	Fractional area	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
	Cumulative depth - mm	112	111	109	107	105	101	96	91	84	78
	Nondimensional cumulative depth	1.13	1.12	1.10	1.08	1.06	1.02	0.97	0.92	0.85	0.78
	Average depth, mm	99.4									

1/After: Slabbers (1971).



a) Water Distribution along the Flow Axis



b) Cumulative Frequency Distribution of Water Depths

Figure 1-4. Water distribution under regular basin irrigation.

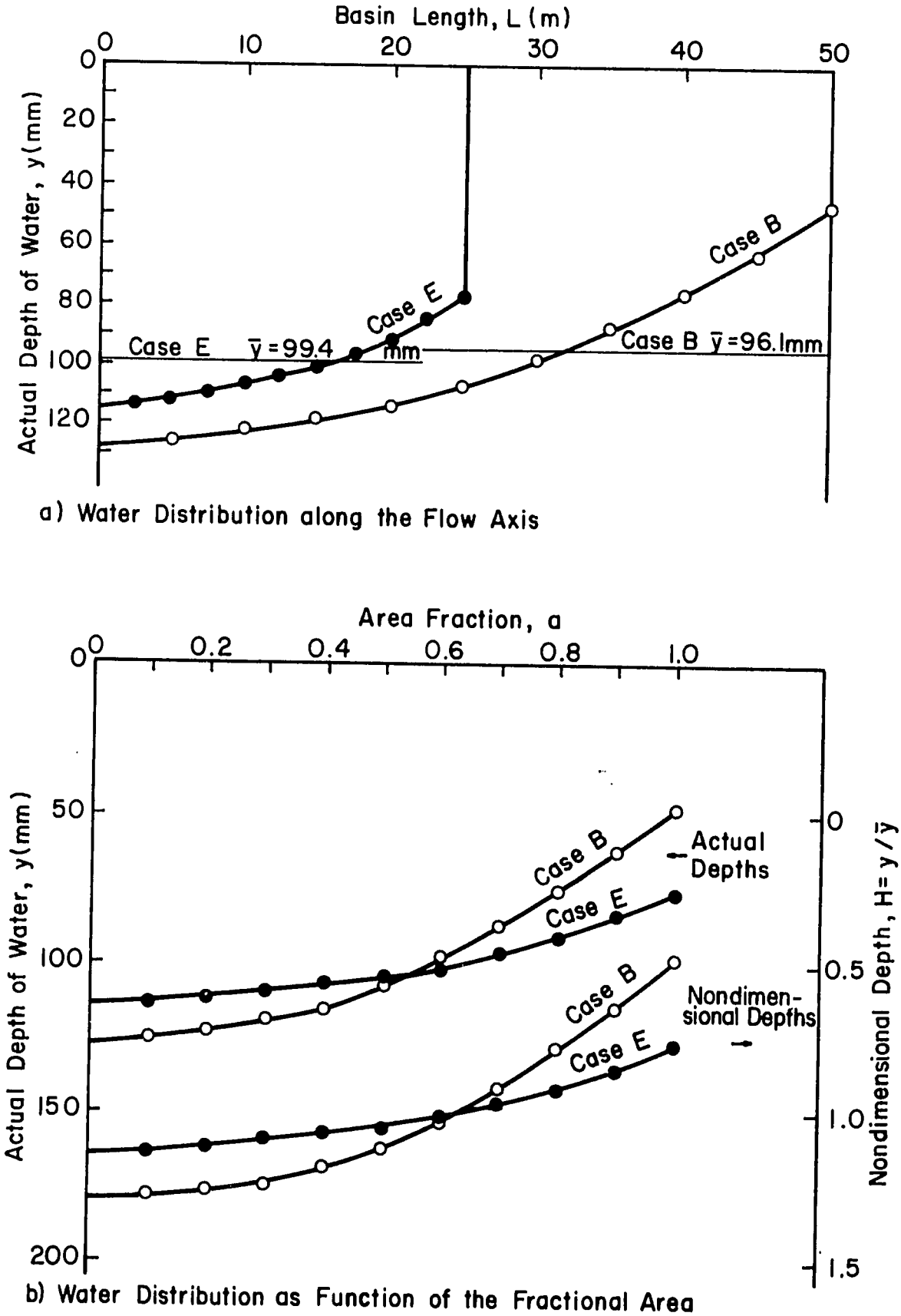


Figure 1-5. Water distribution for two cases of regular basin irrigation.

Table 1-2. Irrigation efficiencies and performance categories for the two study cases.

	h_R mm	B[1] mm x m	E_P	E_S	Performance category[2]	Required Improvement
Basin B	45	0	0.46	1.00	VI	improve both water distribution and decrease depth of application
	50	2x5	0.52	0.996	VI	
	60	12x5	0.61	0.98	VI	
U_d = 0.885	70	30x5	0.70	0.96	VI	
	80	54x5	0.78	0.93	VI	
$\bar{y}$ = 96.1 mm	90	86x5	0.85	0.90	II	improve water distribution only
	100	127x5	0.91	0.87	II	
	110	179x5	0.96	0.84	II	
	120	245x5	0.99	0.79	V	improve water distribution and increase depth of application
	130	339x5	1.00	0.74	V	
	140	439x5	1.00	0.68	V	
Basin E	70	0	0.70	1.0	IV	decrease depth of application
	80	2x2.5	0.80	0.997	I	no improvement needed
	90	18x2.5	0.89	0.98	I	
	100	51x2.5	0.95	0.95	I	
U_d = 0.951	103	65x2.5	0.97	0.94	I	
	106	81x2.5	0.98	0.92	I	
$\bar{y}$ = 99.4 mm	110	109x2.5	0.99	0.90	I	
	113	136x2.5	1.00	0.88	I	
	120	206x2.5	1.00	0.83	I	
	130	306x2.5	1.00	0.76	III	increase depth of application

(1) B is the volume of deficient water after an irrigation (Part I, Chapter 5).
(2) See Part I, Chapter 5.

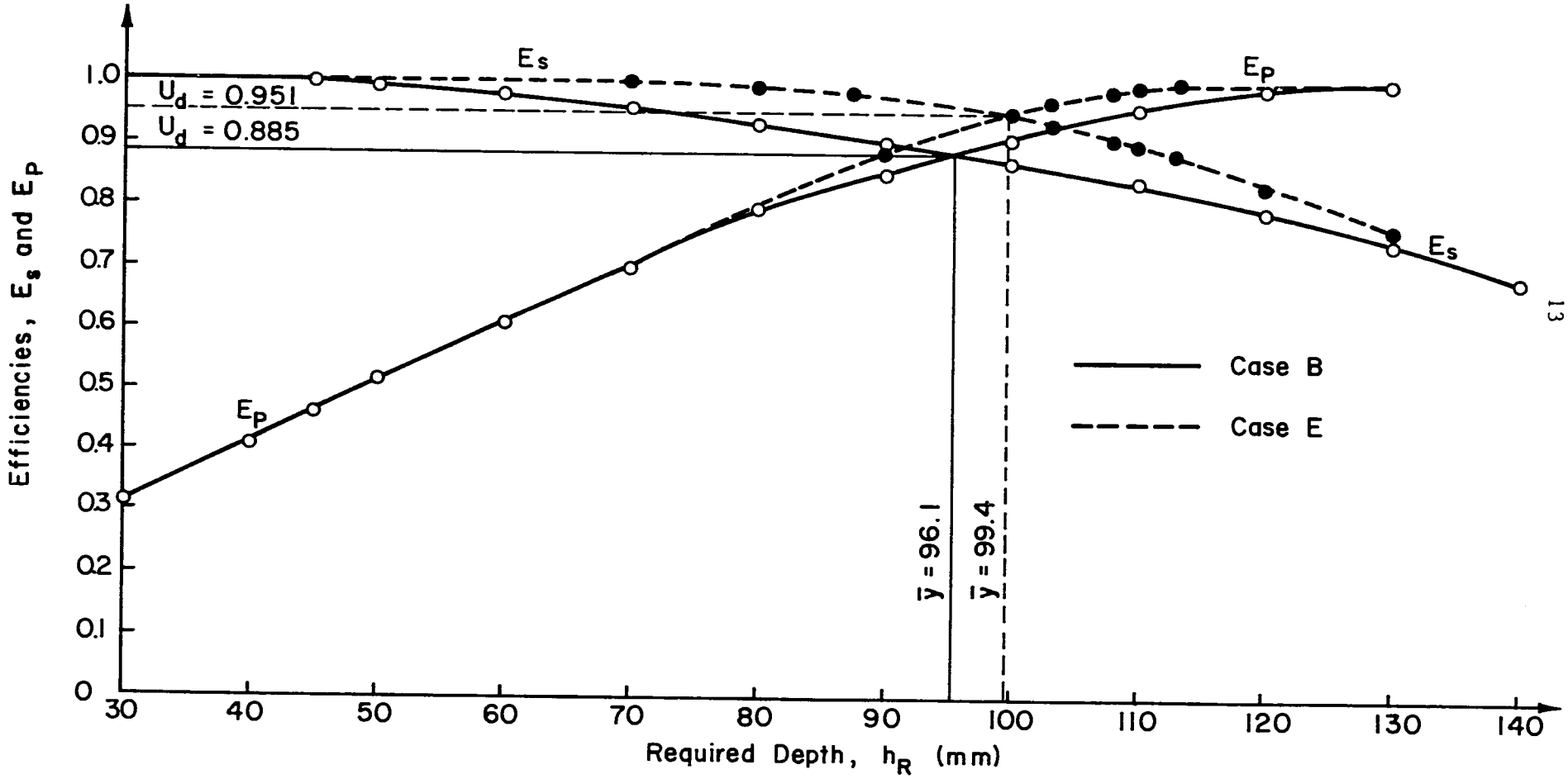


Figure 1-6. Storage efficiency and deep percolation efficiency for two study basins.

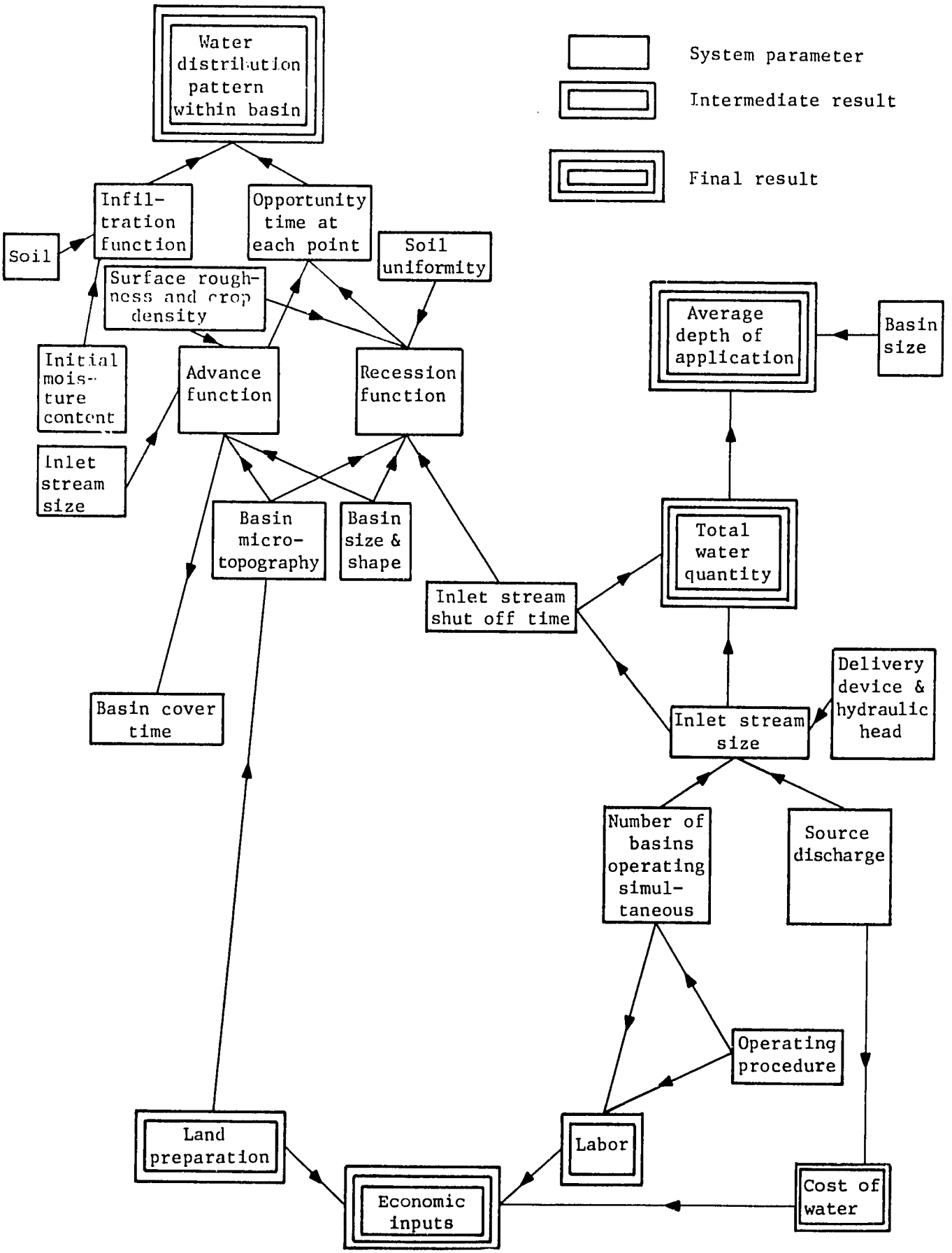


Figure 1-7. Relationships between system parameters and basin irrigation performance.

The water distribution pattern within the basin is a direct function of the opportunity time and the infiltration function. While the infiltration is a function of the soil type and the initial soil moisture content, the opportunity time is derived from the advance and recession contours. For each point in the basin, the opportunity time is the elapsed time between the water arrival and the water disappearance at that point.

The advance function is dependent on several system parameters: the infiltration function, the inlet stream size, the basin microtopography, the basin size and shape, the soil roughness, and the crop density. With irregular basins, the dominant factor is the microtopography and shape, leading to irregular advance contours (Fig. 1-2b). With a regular basin, the shape and size are reduced to the basin length and the microtopography is reduced to the slope along the basin, which is generally uniform and near level. The advance is regular with a uniform water front across the basin. The advance function determines an important system parameter, namely the basin cover time.

Recession is a function of the basin microtopography, basin shape and size, and the shutoff time. With level basins, recession time is negligible and water disappears almost uniformly over the entire basin. Consequently, variations in the opportunity time along a level basin are due mainly to the advance of the water. However, many basins contain many undulations that result in high and low spots, which in turn produce significant differences in the recession time.

The total water quantity applied to the basin is determined by the inlet stream size and the shutoff time. This water quantity, for a given basin size, provides the average depth of application.

The inlet stream size is determined by the delivery structure and the hydraulic head, and may be limited by the discharge at the source and the number of basins operating simultaneously.

The economic inputs are determined by the required method of operation, the labor, the land preparation, and the cost of water. Land preparation affects the basin microtopography and labor is affected by the operating procedure.

In basin irrigation, some of the independent system parameters such as the soil, soil uniformity, crop density, and discharge at the source may be very difficult or impossible to change. Other independent parameters such as basin size and shape, soil surface roughness, delivery device and hydraulic head, operating procedure, and initial moisture content may be changed, but the change must be carried out before (sometimes well before) the irrigation takes place. A third category of independent parameters, namely the inlet stream size and shutoff time can be varied at the time of irrigation. The evaluation and improvement of a basin irrigation system will involve the assessing of the effects of both the latter two categories of independent variables on such dependent parameters as advance function, recession function, opportunity time and basin cover.

Chapter 2

FIELD EVALUATION OF WATER DISTRIBUTION PATTERNS

1. Introduction

In the case of irregular basins, where the water distribution is irregular, an estimation of the water distribution pattern and the evaluation of irrigation performance can be carried out by various field observations and measurements together with suitable assumptions.

Using various methods, the irrigation performance is evaluated and the need for improvement is determined. Improvements should be tested again and be proven by another evaluation of the irrigation performance. Possible methods and procedures for the determination of the water distribution pattern and the evaluation of basin irrigation are:

a. direct field depth estimations;

b. recession pattern measurement;

c. excess application method; and

d. ponded water method.

Each of these methods will be described in more detail. In most cases a linear water distribution is assumed. The possible fitting of a power curve to the field data is described in Appendix 2-B.

2. Direct Field Depth Estimations

A grid is superimposed on the basin that is to be irrigated. Although not mandatory, for simplification it is recommended that each point (station) represent an equal area. That is, if the basin area is A_T and there are n stations, each station should represent an area:

$$a = \frac{A_T}{n} \tag{2-1}$$

A symmetrical and rectangular grid is recommended.

a. Calculation of water depths.

During irrigation the advance and recession contours are plotted for various times (Fig. 2-1). By interpolation, the time of advance, t_a, and the time of recession, t_r, are calculated for each station. The opportunity time, t_{op}, is calculated for each station by:

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

where i denotes the ith station and i = 1, 2, ... n.

Using one of the common procedures, the infiltration equation is determined and the cumulative infiltrated depth is expressed as a function of the opportunity time. On a log-log scale, the cumulative infiltrated depth is close to a linear function of the opportunity time (Fig. 2-2).

Generally, several infiltration tests are carried out and a typical one is then selected. The determination of a typical infiltration curve is carried out by considering local conditions (such as slope, soil heterogeniety, etc.), or when no specific guidance exists, the mean infiltration curve is selected as the typical one.

Using the calculated opportunity time, the depth of water infiltrated at each station is estimated from the typical infiltration curve (Fig. 2-2) and the average depth, $\bar{y}_{typ}$, is calculated (Fig. 2-3) by:

$$\bar{y}_{typ} = \frac{1}{A_T} \sum_{i=1}^{n} \left(a_i y_{typ_i} \right) \tag{2-3}$$

in which y_{typ_i} = estimated depth of water infiltrated at the ith station as calculated from the typical infiltration curve, and a_i = area represented by the ith station.

The estimated total volume of water infiltrated into the soil, T_{est}, can be calculated from the depths infiltrated at each station as

$$T_{est} = A_T \cdot \bar{y}_{typ} \tag{2-4}$$

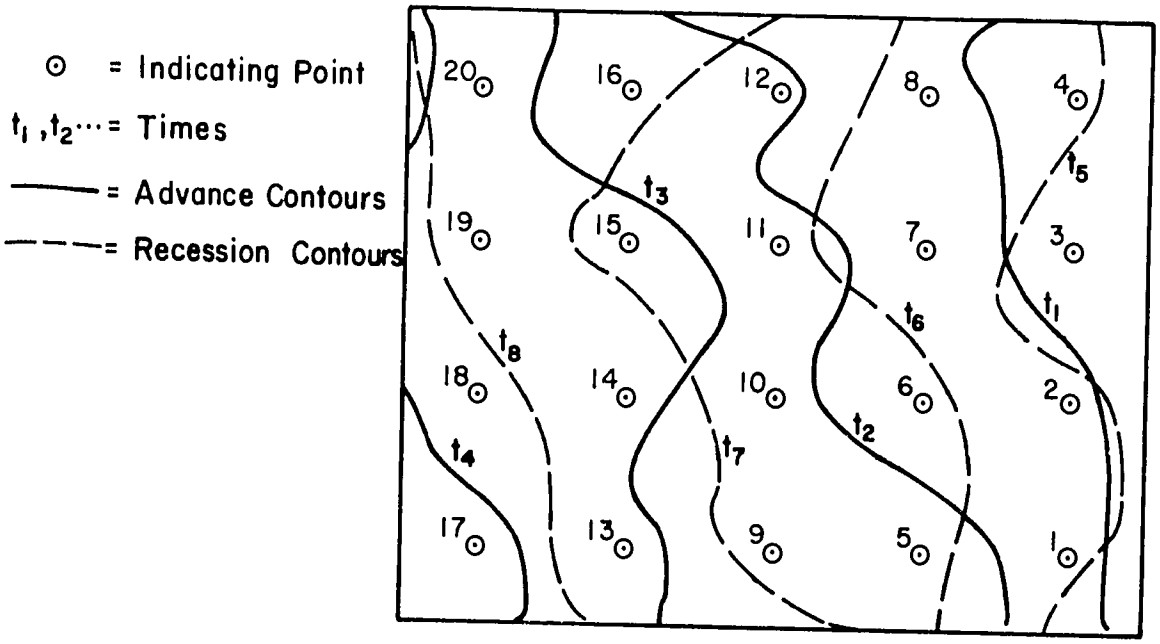


Figure 2-1. Advance and recession contours as measured in the field at various times.

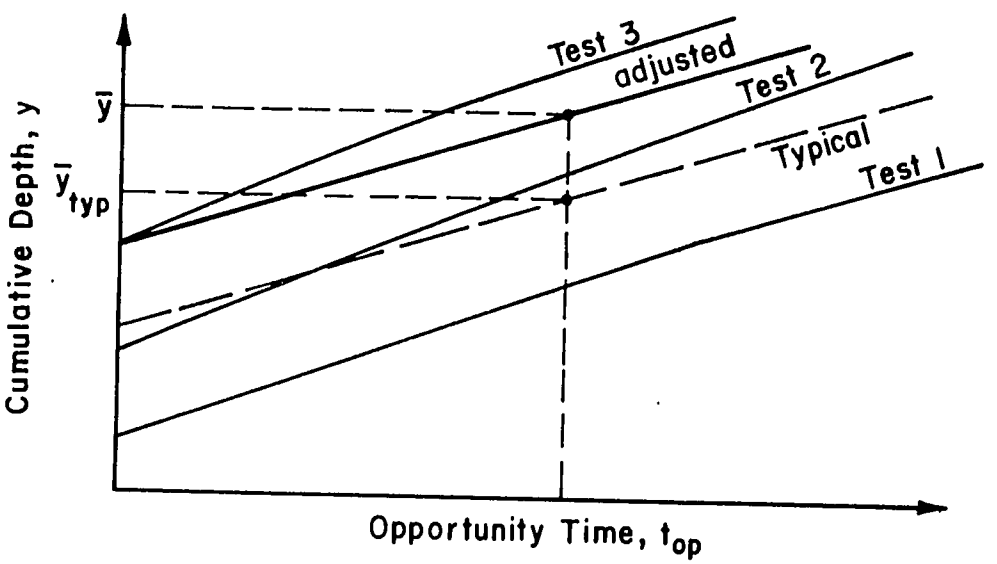


Figure 2-2. Cumulative infiltration curves.

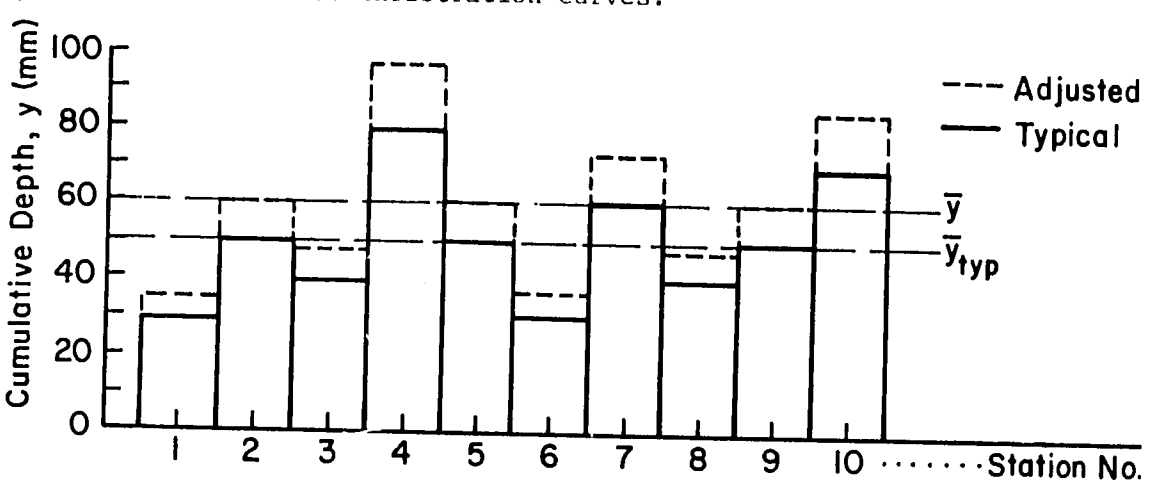


Figure 2-3. Water depths as calculated for the various stations.

The actual volume of water applied during the irrigation is given by:

$$T = \sum_{j=1}^{p} Q_j \Delta t_j \tag{2-5a}$$

where Q_j = a constant inlet discharge for a period of time, Δt_j, and p = number of such time periods. If the inlet discharge is constant at Q, over the total basin delivery time, t_{co}, then

$$T = Q \cdot t_{co} \tag{2-5b}$$

The actual average depth of water applied during the irrigation is:

$$\bar{y}h_D = \frac{T}{A_T} \tag{2-6}$$

Usually, $T_{est} \neq T$; that is $\bar{y} \neq \bar{y}_{typ}$, consequently the cumulative depth infiltrated at each station should be adjusted.

The adjustment of the water depths to the actual applied depth is dependent on the accuracy of the infiltration equation and the discharge measurements. In most practical cases, the measurement of the actual applied volume, as obtained by discharge and time measurement, is significantly more accurate and reliable than the infiltration measurements. Therefore, the actual volume applied is regarded as the correct volume of water and the depth of water, y_i, at each of the stations should be adjusted accordingly. At the same time, the typical infiltration curve is adjusted also. Using the actual delivered volume of water, T, as the real amount of water applied during the irrigation, the water depth at each of the stations is adjusted to (Fig. 2-3):

$$y_{ad_i} = y_{typ_i} \frac{T}{T_{est}} = y_{typ_i} \frac{\bar{y}}{\bar{y}_{typ}} \tag{2-7}$$

The typical infiltration curve is adjusted parallel to the typical curve but crossing through a point having $\bar{y}$ for the same time that the typical

infiltration time provides $\bar{y}_{typ}$. In other words, the adjusted infiltration curve is obtained by plotting $\bar{y}_{typ}$ on the typical infiltration curve, then marking a point on the same vertical line (same time) with cumulative depth, $\bar{y}$, and drawing a line parallel to the typical one through this new point (see Fig. 2-2). The adjusted infiltration curve will provide the same adjusted depths as given by Eq. 2-7.

In cases where the actual volume of water is somewhat doubtful and cannot be relied upon as the real volume of water applied to the field, adjustment of the y_{typ_i} is according to a weighted combination of T and T_{est}. Each water depth y_{typ_i} is adjusted by multiplying it by the ratio, R, where

$$R = \frac{P_1 T + P_2 T_{est}}{(P_1 + P_2) T_{est}} \tag{2-8}$$

where P_1, P_2 are weighting factors for the actual and typical depths, respectively. For example, if $P_1 = P_2$, that is the reliability of the water volume measurement is assumed to be the same as the infiltration measurements, will be:

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

The adjusted water depths for each station are then given by:

$$y_{ad_i} = R y_{typ_i} \tag{2-10}$$

Note that for the case in which the volume of water, T, is considered the real amount of water applied during irrigation, substituting $P_2 = 0$ gives

$$R = \frac{T}{T_{est}} \tag{2-11}$$

The adjusted water depths, y_{ad_i}, are regarded now as the real water depths, y_i, that infiltrated, and the evaluation of the irrigation

can be carried out according to the procedure described in Part I, Chapter 5.

b. Calculation of irrigation performance parameters.

The total quantity of water applied is given by:

$$T = \sum_{i=1}^{n} y_i \cdot a_i \tag{2-12}$$

The average depth of water applied during irrigation is:

$$\bar{y} = \frac{T}{A_T} = \frac{\sum_{i=1}^{n} y_i a_i}{A_T} \tag{2-13}$$

The deficient amount of water in relation to $\bar{y}$, expressed as a depth is:

$$h_C = \frac{1}{A_T} \sum_{i=1}^{p} (\bar{y} - y_i) a_i \text{ for } y_i < \bar{y} \tag{2-14}$$

where p is the number of stations with infiltrated water depth less than $\bar{y}$.

The distribution uniformity is:

$$U_d = \frac{\bar{y} - h_C}{\bar{y}} \tag{2-15}$$

For a required depth of application, h_R, the deficiency, h_B, is given by:

$$h_B = \frac{1}{A_T} \sum_{i=1}^{p} (h_R - y_i) a_i \text{ for } y_i < h_R \tag{2-16}$$

where p = number of stations with infiltrated water depth less than h_R. The deep percolation efficiency is:

$$E_P = \frac{h_R - h_B}{\bar{y}} \tag{2-17}$$

If the areas representing all the stations are equal, E_P can be written as:

$$E_P = \frac{h_R - \frac{1}{n} \cdot \sum_{i=1}^{p} (h_R - y_i)}{\bar{y}} \tag{2-18}$$

The storage efficiency is

$$E_S = \frac{h_R - h_B}{h_R} \tag{2-19}$$

or if the station areas are equal

$$E_S = \frac{h_R - \frac{1}{n} \sum_{i=1}^{p} (h_R - y_i)}{h_R} \tag{2-20}$$

Other ratios, coefficients or efficiencies can be calculated from U_d, E_P, and E_S as is given in Part 1, Chapter 5.

Example

Infiltration curves were established for an irrigated field and the typical infiltration curve was determined (Fig. 2-4). In a test basin, a grid of 12 stations was established and the opportunity time for each station was measured as given in Table 2-1. Each station represents an area of 0.033 ha. The average depth infiltrated, obtained from the opportunity time and the typical infiltration curve, is 694/12 = 57.8 mm. The actual average water depth that was applied to the field was 74.1 mm and is regarded as the real amount of water. The adjusting factor is R = 74.1/57.8 = 1.28. While the adjusted infiltration curve is presented in Fig. 2-4, the adjusted water depths are given in Table 2-1 and are shown in Fig. 2-5.

The total quantity of water applied: (each station represents an area of 0.033 ha), $T = 0.033 \times \sum_{i=1}^{12} y_i = 29.3$ ha-mm (293 m^3). The

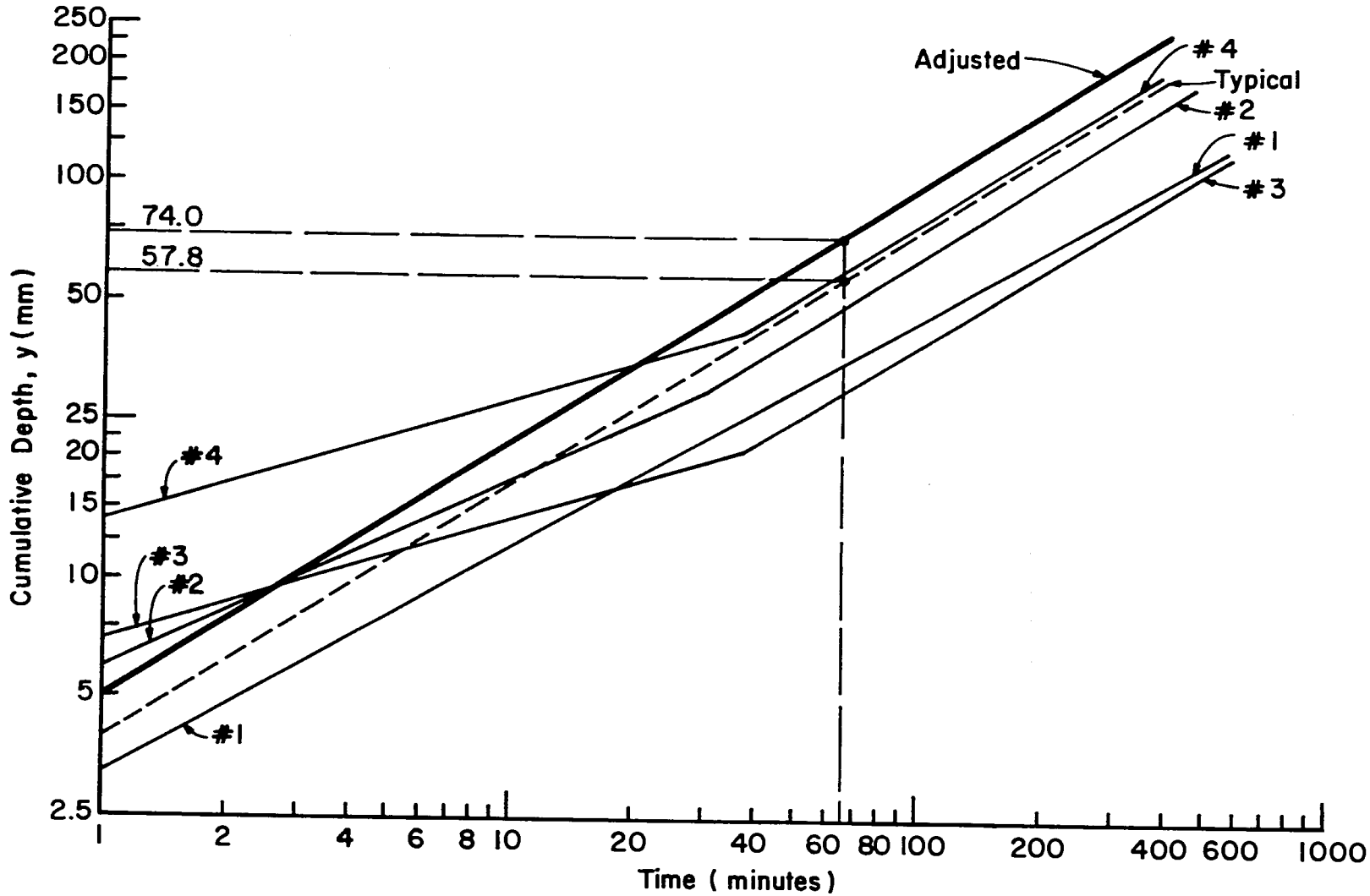


Figure 2-4. Cumulative intake curve (from Merrian, Keller and Alfaro, 1973).

Table 2-1. Opportunity time and applied water depths for example problem.

Station	1	2	3	4	5	6	7	8	9	10	11	12	Total	Av.
t_{op} (min)	30	40	50	30	70	150	110	80	50	100	70	30		
y_{typ} (mm)	35	42	50	35	61	100	82	66	50	77	61	35	694	57.8
y_{ad} (mm)	45	54	64	45	78	128	105	85	64	99	78	45	890	74.1

Table 2-2. Deep percolation and storage efficiencies for various levels of required water depth for example problem.

h_R (mm) (1)	p (2)	$\sum_{i=1}^{p}(h_R-y_i)$ (3)	(3)/12 (4)	$E_P=[h_R-(4)]/\bar{y}$ (5)	$E_S\,[h_R-(4)]/h_R$ (6)
40	0	0	0	0.540	1.000
50	3	15	1.3	0.657	0.974
60	4	51	4.3	0.752	0.929
70	6	103	8.6	0.829	0.877
80	8	167	13.9	0.892	0.826
90	9	252	21.0	0.931	0.767
100	10	343	28.6	0.964	0.714
110	11	448	37.3	0.981	0.661
120	11	558	46.5	0.992	0.613
130	12	670	55.8	1.000	0.571

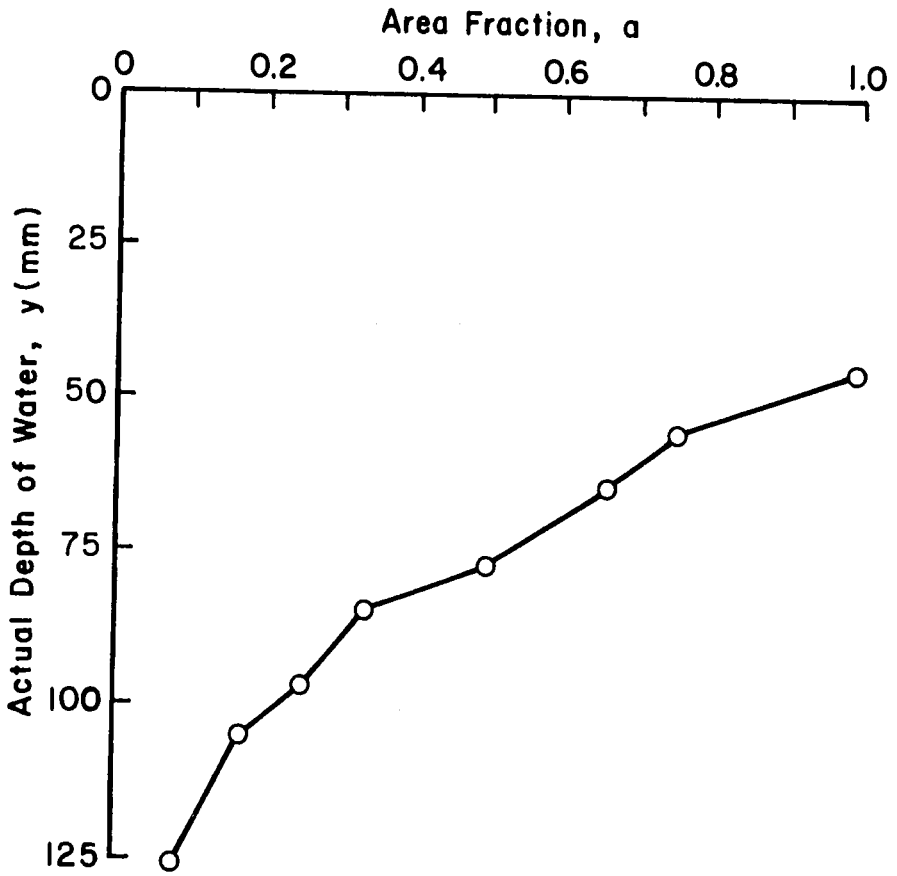


Figure 2-5. Adjusted water depth distribution for example problem.

deficient amount of water in regard to $\bar{y}$ is:

$$h_C = \frac{1}{12}\,[(74.1-45) \times 3 + (74.1-54) + (74.1-64) \times 2] = 10.6 \text{ mm}$$

or a total volume of 12 x 0.033 ha x 10.6 mm = 4.20 ha-mm ($42m^3$).

The distribution uniformity is:

$$U_d = \frac{74.1 - 10.6}{74.1} = 0.857$$

In general this would be regarded as too low and its improvement is recommended.

The values of E_P and E_S are calculated for several possible required depths of application and are given in Table 2-2. The efficiencies as a function of the required depth of application are also shown in Fig. 2-6. Values of h_R were chosen to range from at least y_{min} to y_{max}. Assume that satisfactory levels of irrigation performance are $U_d \geq 0.9$, $E_P \geq 0.8$, and $E_S \geq 0.8$. If the required depth of application is 80 mm, the irrigation performance belongs to category II, (Part I, Chapter 5) because E_P and E_S are satisfactory, but U_d is not. In other words, the distribution of water should be improved for the same average depth of application. If the required depth of application is 100 mm, the irrigation performance is classified as category V. The irrigation should be improved by improving the water distribution and increasing the total depth of application. If the required depth of application is 50 mm, the irrigation performance is classified as category VI. The irrigation should be improved by improving the water distribution and decreasing the total depth of application.

c. Determination of infiltrated water depth by soil moisture measurement.

A grid of water depths for the evaluation of basin irrigation can be obtained also by direct measurement of soil moisture after the

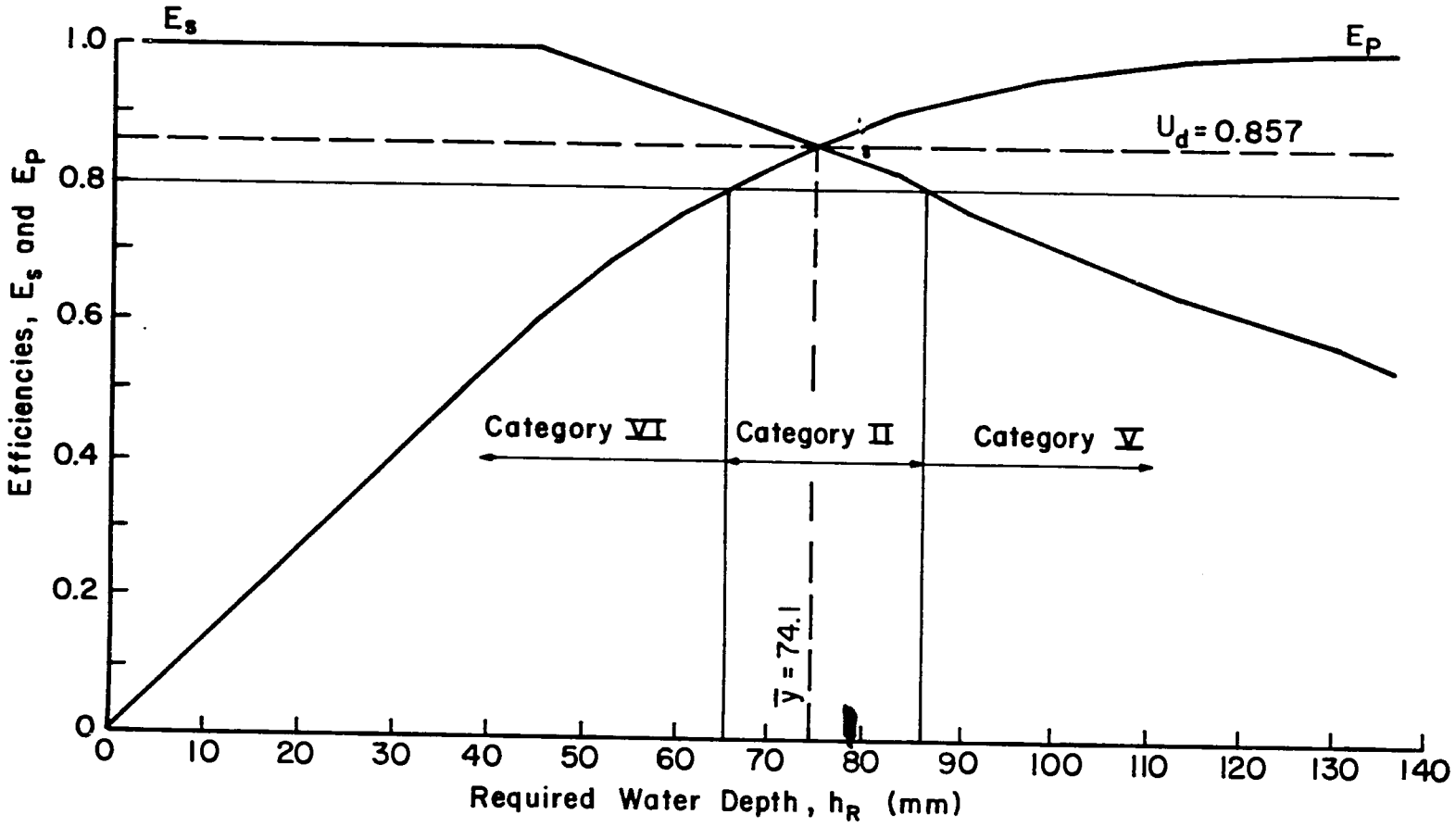


Figure 2-6. Efficiencies and performance categories as function of the required water depth for example problem.

irrigation, rather than using the infiltration characteristics and the opportunity time. The average soil moisture deficiency is determined just before irrigation by:

$$AMC = (FC - PMC)\ \gamma \tag{2-21}$$

where

AMC = available moisture deficiency (volume fraction)

FC = moisture content at field capacity (mass fraction)

PMC = moisture content before irrigation (mass fraction), and

γ = soil bulk density (mass per unit volume)

For each station, the wetted depth D is determined by means of a penetrating probe and the depth of water infiltrated at that point is given by:

$$y_i = D_i \frac{AMC}{100} \tag{2-22}$$

where y_i has the same units as D_i.

The total volume of water infiltrated during irrigation can be estimated by

$$T_{est} = \sum_{i=1}^{n} a_i y_i \tag{2-23}$$

where a_i = the area represented by station i.

Where each point represents an equal area, that is $a_i = a$ for i = 1, 2,...n, the total volume of water is given by:

$$T_{est} = \sum_{i=1}^{n} a\ y_i \tag{2-24}$$

The adjustment coefficient is given by:

$$R = \frac{T}{T_{est}} \tag{2-25}$$

and the adjusted water depths for each station are given by:

$$yad_i = Ry_i \tag{2-26}$$

Eqs. 2-12 through 2-20 can be applied to these values of y_{ad_i} to evaluate the performance of the irrigation.

3. The Recession Method

The water distribution can be estimated, when the total water quantity applied is known and the infiltration function is known, by field measurements of the recession pattern. This method can be applied when the water advance is relatively fast, and the opportunity times vary primarily because of the differences in recession time. This method is best suited to basins with major changes in topography (Fig. 2-7a).

An estimation of the water distribution can be carried out by the determination of the water disappearance time from area fractions of the field. From the infiltration function, the depth of water infiltrated in each area fraction can be determined.

If three area fractions, with areas of a_1, a_2, and a_3, and disappearance times of t_1, t_2, and t_3, respectively, are considered and it is assumed the cumulative infiltration over the basin can be represented by

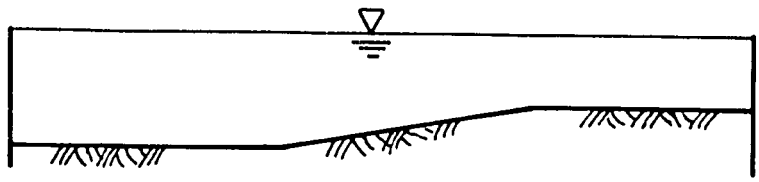
$$y = At_{op}^{B} \tag{2-27}$$

where y = cumulative depth of water infiltrated, t_{op} = infiltration opportunity time, and A and B are constants, the total quantity of water infiltrated will be:

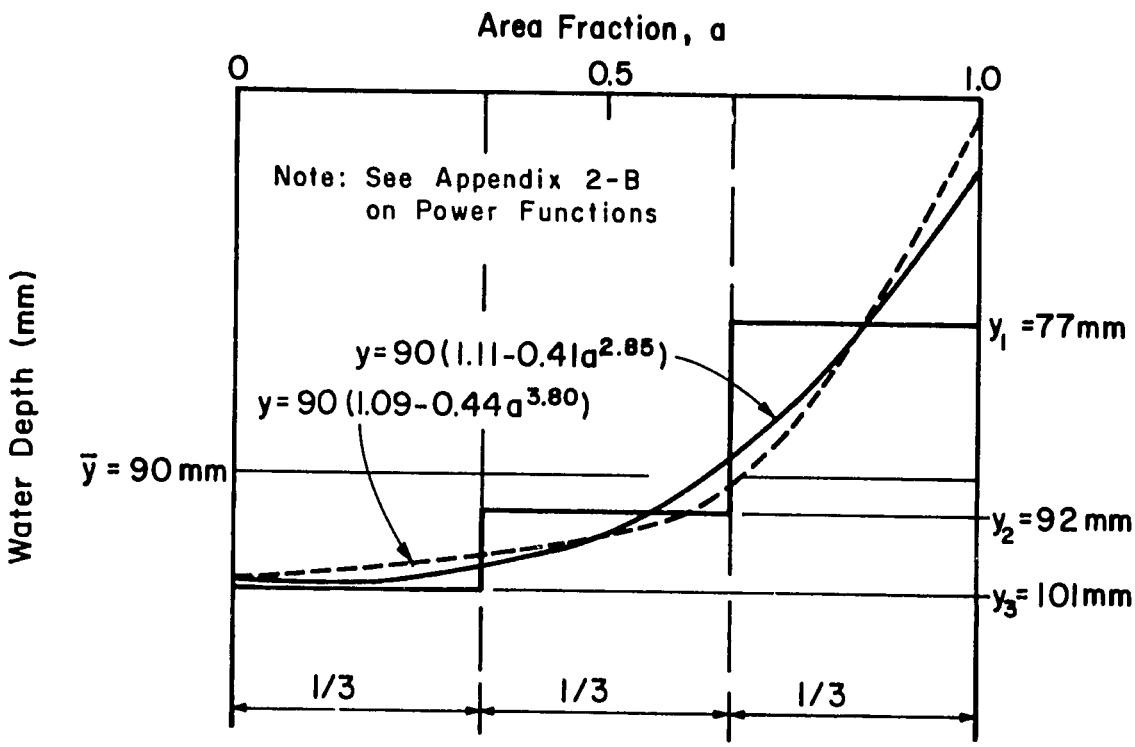
$$T = \bar{y}\, A_T = a_1 y_1 + a_2 y_2 + a_3 y_3 \tag{2-28}$$

where

$$y_1 = At_1^{B} \qquad y_2 = At_2^{B} \qquad y_3 = At_3^{B} \tag{2-29}$$



(a) Schematic Description of Non-leveled Basin



(b) Water Distribution

Figure 2-7. Example of water distribution obtained by recession method.

From Eq. 2-29

$$y_2 = y_1\left(\frac{t_2}{t_1}\right)^B \qquad y_3 = y_1\left(\frac{t_3}{t_1}\right)^B \tag{2-30}$$

and Eq. 2-28 can be written

$$\bar{y}\ (a_1+a_2+a_3) = a_1y_1 + a_2y_1\left(\frac{t_2}{t_1}\right)^B + a_3y_1\left(\frac{t_3}{t_1}\right)^B \tag{2-31}$$

Then

$$y_1 = \frac{\bar{y}(a_1+a_2+a_3)}{a_1 + a_2\left(\frac{t_2}{t_1}\right)^B + a_3\left(\frac{t_3}{t_1}\right)^B} \tag{2-32}$$

and y_2 and y_3 can be calculated from Eq. 2-30. If the area fractions are all equal, Eq. 2-32 reduces to

$$y_1 = \frac{3\bar{y}}{1 + \left(\frac{t_2}{t_1}\right)^B + \left(\frac{t_3}{t_1}\right)^B} \tag{2-33}$$

A typical water distribution obtained by this procedure is shown in Fig. 2-7b.

Once again the irrigation performance can be evaluated by calculatin the irrigation performance parameters U_d, E_p and E_s and the appropriate performance category can be determined.

Example 1

A basin of 1000 m^2 requires a depth of water of 80 mm. A strean of 3.0 m^3/min is delivered to the basin for 30 min. The infiltration equation (Eq. 2-27) has B = 0.677. After total coverage of the basin, which is relatively fast, the water disappearance over the basin is estimated by direct observation as:

the first 1/3 of the area is dry after 100 minutes,

the second 1/3 of the area is dry after 130 minutes, and

the last 1/3 of the area is dry after 150 minutes.

The total volume of water applied is $T = 3.0 \times 30 = 90m^3$ and the average depth of water applied is $\bar{y} = \frac{90m^3}{1000m^2} = 90$ mm. The depths of water infiltrated into the various fractions are:

$$y_1 = \frac{3 \times 90}{1 + \left(\frac{130}{100}\right)^{0.677} + \left(\frac{150}{100}\right)^{0.677}} = \frac{270}{1 + 1.19 + 1.32} = 77 \text{ mm}$$

$$y_2 = y_1 \left(\frac{130}{100}\right)^{0.677} = 77 \times 1.19 = 92 \text{ mm}$$

$$y_3 = y_1 \left(\frac{150}{100}\right)^{0.677} = 150 \times 1.32 = 101 \text{ mm}$$

The water distribution is shown in Fig. 2-7b.

The irrigation performance parameters are:

$h_C = (90-77) \times 0.33 = 4.3$ mm

$U_d = \frac{90 - 4.3}{90} = 0.952$

$h_B = (80-77) \times 0.33 = 1.0$ mm

$E_P = \frac{80 - 1}{90} = 0.877$

$E_S = \frac{80 - 1}{80} = 0.987$

The irrigation performance is satisfactory (category I) with all three efficiencies having relatively high values.

Example 2

A basin requires 80 mm irrigation water. An average depth of 90 mm is applied. The water disappears from the basin surface as follows:

the first 1/3 of the area is dry after 70 minutes,

the second 1/3 of the area is dry after 135 minutes, and

the last 1/3 of the area is dry after 190 minutes.

The infiltration equation power is B = 0.677.

The depths of water infiltrated into the various fractions are:

$$y_1 = \frac{3 \times 90}{1 + \left(\frac{135}{70}\right)^{0.677} + \left(\frac{190}{70}\right)^{0.677}} = 60 \text{ mm}$$

$$y_2 = 60 \times \left(\frac{135}{80}\right)^{0.677} = 93 \text{ mm}$$

$$y_3 = 60 \times \left(\frac{190}{70}\right)^{0.677} = 117 \text{ mm}$$

The water distribution is given in Fig. 2-8. The irrigation performance parameters are:

$$h_C = (90-60)\ 0.333 = 10 \text{ mm}$$

$$U_d = \frac{90 - 10}{90} = 0.88$$

For the required depth of application of h_R = 80 mm:

$$E_P = \frac{80 - 20 \times 0.333}{90} = 0.814$$

$$E_S = \frac{80 - 20 \times 0.333}{80} = 0.915$$

The irrigation performance exhibits a relatively low distribution uniformity that should be improved. No change in the average applied depth is needed.

The water distribution improvement can be obtained by decreasing the differences in the times of water disappearance for the three fractions. For this case, improvement can be achieved only by improved leveling. For example, assume that after improving the leveling of the basin, the water disappearing times are as follows:

The first 1/3 of the area is dry after 80 minutes,

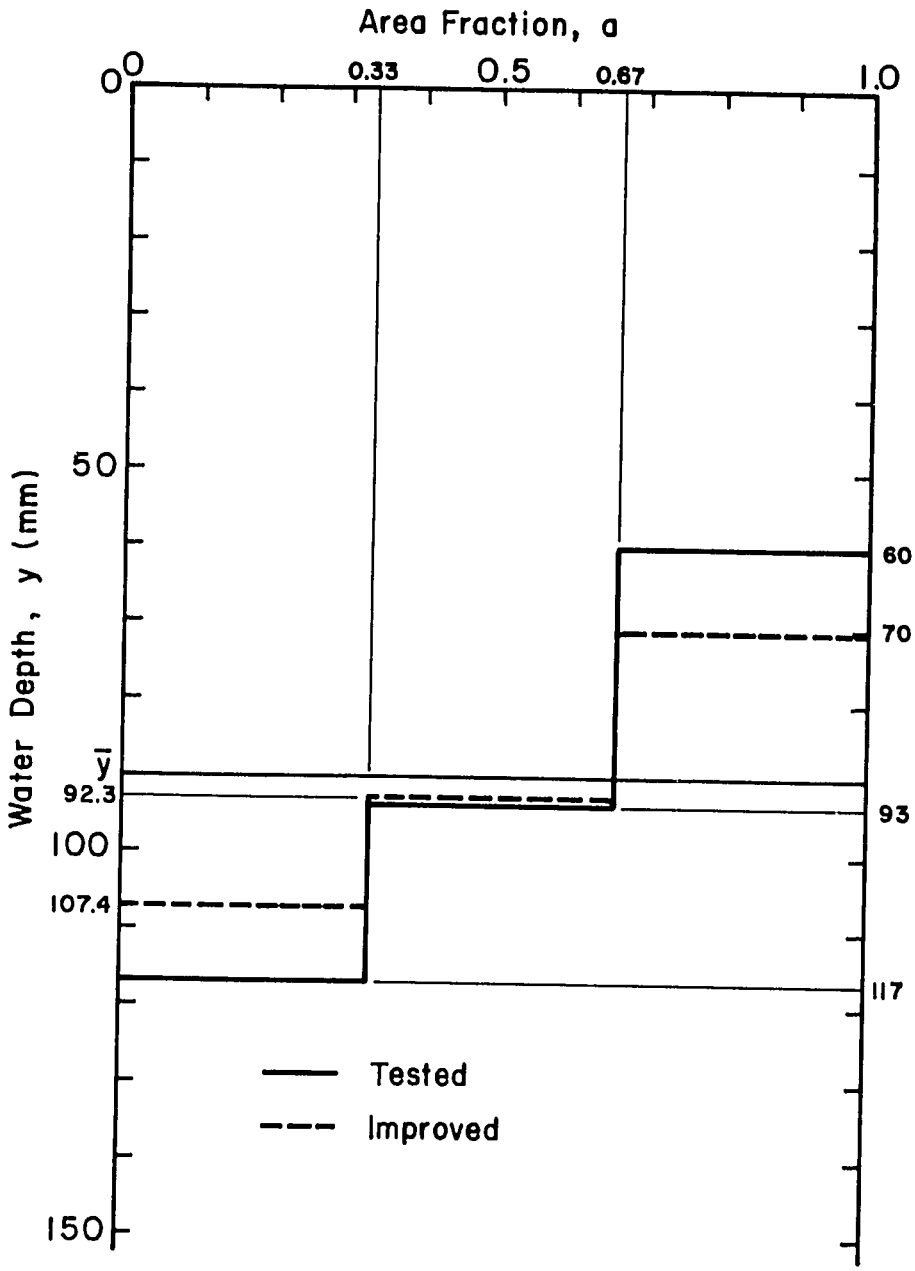


Figure 2-8. Example of water distribution as estimated by disappearance of water from three equal sized areas of a basin.

the second 1/3 of the area is dry after 120 minutes, and the last 1/3 of the area is dry after 150 minutes.

The same amount of water is applied as before, that is $\bar{y}$ = 90 mm. The depths of water infiltered at the various fractions are (Fig. 2-8):

$$y_1 = \frac{30 \times 90}{1 + \left(\frac{120}{80}\right)^{0.677} + \left(\frac{150}{70}\right)^{0.677}} = 70 \text{ mm}$$

$$y_2 = 66 \times \left(\frac{120}{80}\right)^{0.677} = 92 \text{ mm}$$

$$y_3 = 66 \times \left(\frac{150}{80}\right)^{0.677} = 107 \text{ mm}$$

The irrigation performance parameters are:

$$h_C = (90-70)\ 0.33 = 6.6 \text{ mm}$$

$$U_d = \frac{90 - 6.6}{90} = 0.926$$

$$h_B = (80-70)\ 0.33 = 3.3 \text{ mm}$$

$$E_P = \frac{80 - 3.3}{90} = 0.853$$

$$E_S = \frac{80 - 3.3}{80} = 0.959$$

The irrigation efficiencies are all satisfactory so that the irrigation performance for this case is regarded as good.

4. The Excess Application Method

In many cases, the evaluation of basin irrigation is carried out by limited data collection, where the determination of water depths over a particular grid is not possible. In such cases, only certain types of data are collected and reasonable assumptions must be made to estimate the water distribution. One of the simplest procedures is based on the assumption that the minimum depth of water applied during the irrigation is known and is equal to the required depth of application. That is:

$$y_{min} = h_R \qquad (2\text{-}34)$$

The only measured data during this evaluation procedure is the total quantity of water applied, T, which is determined by:

$$T = Qt_{co} \tag{2-35}$$

If T cannot be measured directly, however, it should be estimated by other means (well or pump capacity, canal capacity, etc.).

If S is the ratio of the required quantity of water to the applied quantity of water:

$$S = \frac{h_R}{\bar{y}} \tag{2-36}$$

and, as it is assumed $y_{min} = h_R$, then

$$y_{min} = S\,\bar{y} \tag{2-37}$$

Since the only available data are the total quantity of water applied, T, the irrigated area size, A_T, and the assumption that $h_R = y_{min}$, the distribution of water depths is unknown, and must be estimated. Assuming that the minimum depth of water, y_{min}, is equal to the required water depth, and estimating the average depth applied, $\bar{y}$, the distribution of water depths over the whole area can be approximated in different ways.

a. Linear and symmetrical distribution of water depths.

When the basin has a wide and well distributed spectrum of irregularities (many sizes of depressions and peaks), it can be assumed that the differences between the minimum and average depths, and the maximum and average depths of water infiltrated are approximately equal, that is:

$$y_{max} - \bar{y} = \bar{y} - y_{min} \tag{2-38}$$

If it is also assumed that the water distribution is linear (Fig. 2-9), then the evaluation parameters can be determined as:

$$E_P = \frac{h_R}{\bar{y}} = \frac{y_{min}}{\bar{y}} = S \tag{2-39}$$

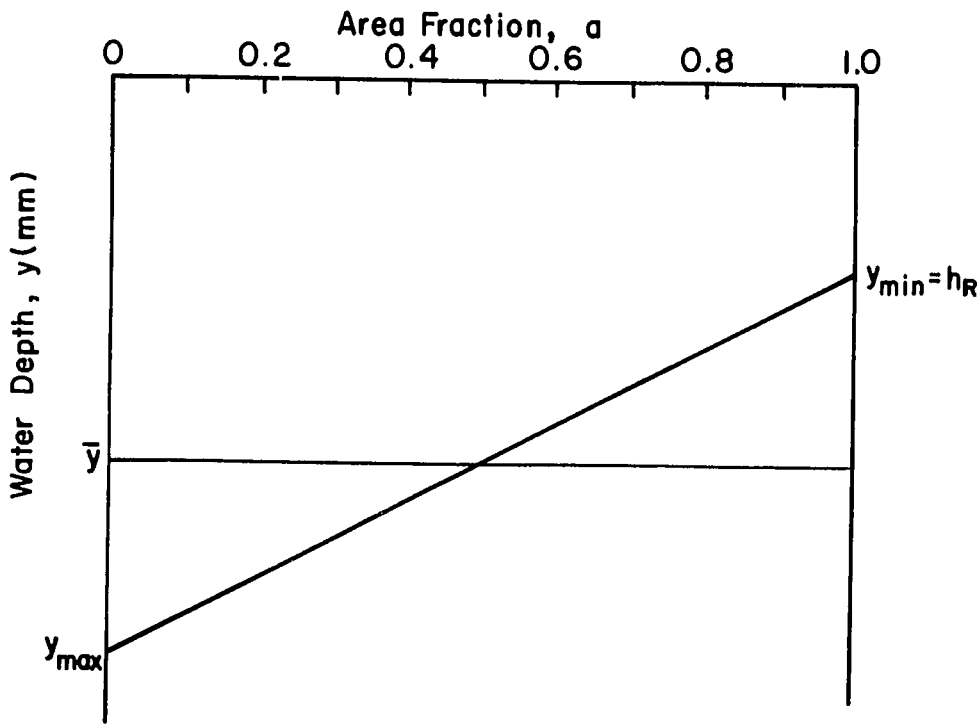


Fig. 2-9. Linear and symmetrical distribution of water depth.

Since the deficiency is zero,

$$E_S = 1 \tag{2-40}$$

The uniformity coefficient becomes,

$$U_d = \frac{\bar{y}\, A_T - \frac{1}{2}\left(\bar{y} - y_{min}\right)\frac{A_T}{2}}{\bar{y}\, A_T} \tag{2-41}$$

Substituting $y_{min} = S\bar{y}$ results in

$$U_d = \frac{3 + S}{4} \tag{2-42}$$

If the irrigation performance is regarded as satisfactory when $U_d \geq 0.9$, $E_P \geq 0.8$ and $E_S \geq 0.8$, then it can be seen that:

(i) If $S \geq 0.8$, then $U_d \geq 0.95$ and the irrigation performance is in category I and is satisfactory.

(ii) If $0.6 \leq S \leq 0.8$, then $U_d \geq 0.9$, but E_P is not satisfactory. The irrigation performance belongs to category IV, which means that the water distribution uniformity is acceptable, however, the total depth of application should be decreased.

(iii) If $S < 0.6$, then $U_d < 0.9$ and $E_P < 0.6$, that is both E_P and U_d are not satisfactory. The irrigation performance belongs to category VI which means that the irrigation should be improved by decreasing deep percolation which, in this case of a linear and symmetrical water distribution, means actually decreasing the total depth of application.

Example

A basin of 1000 m^2 requires a water depth of h_R = 75 mm. Thus, the total required water quantity is 1000 m^2 x 75 mm = 75 m^3.

It is planned to have the required depth infiltrate at the far end of the basin, where the infiltrated depth is minimal. The total amount

of water that is actually applied is measured as $T = 107\ m^3$. Hence, the average depth of water actually applied is $\bar{y} = 107$ mm.

Since no other data are available the water distribution is assumed to be linear with $y_{min} = h_R = 75$ mm and $\bar{y} = 107$ mm. The assumed linear water distribution is shown in Fig. 2-10. The maximum infiltrated depth is $y_{max} = \bar{y} + (\bar{y} - y_{min}) = 107 + 32 = 139$mm.

The ratio between the required and actual applied water quantity is (Eq. 2-36).

$$S = \frac{75}{107} = 0.70$$

Evaluation of the irrigation performance for this case gives a uniformity distribution (Eq. 2-42) of

$$U_d = \frac{3 + 0.70}{4} = 0.925$$

a deep percolation efficiency (Eq. 2-39) of

$$E_p = 0.7$$

and a storage efficiency of

$$E_S = 1.0.$$

If it is considered that $U_d \geq 0.9$, $E_p \geq 0.8$, and $E_S \geq 0.8$ are the satisfactory levels for these efficiencies, it can be concluded that the water distribution is satisfactory ($U_d = 0.92$) but E_p is low and decreasing the total depth actually applied can improve the irrigation performance.

If the same uniformity distribution, U_d, is to be maintained with a smaller $\bar{y}$, this requires a change of y_{min} and y_{max}, however, their ratio should remain the same. That is:

$$\frac{y_{min}}{y_{max}} = \frac{75}{139} = .539 \qquad (2\text{-}43)$$

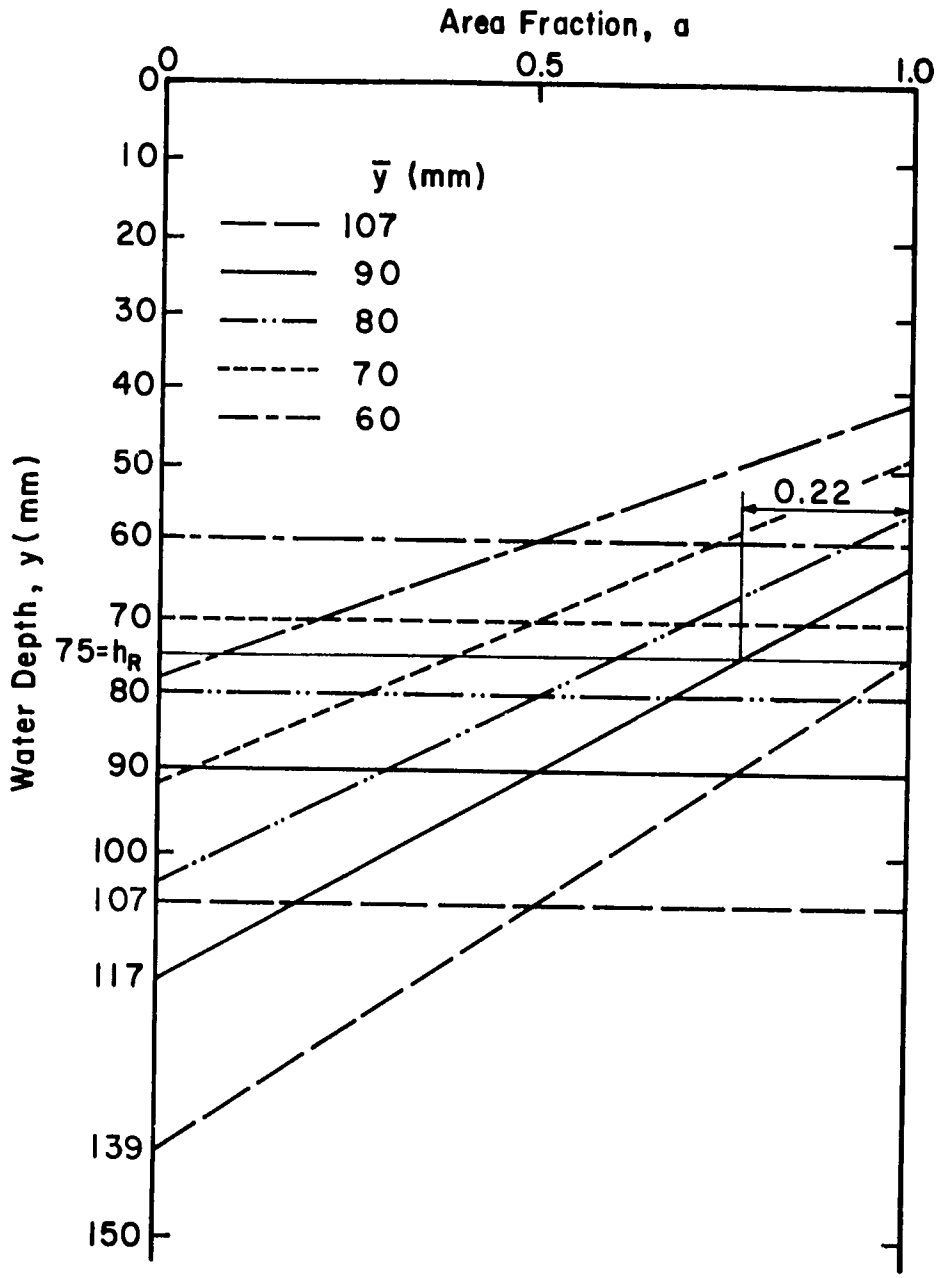


Figure 2-10. Example of water distribution with linear and symmetrical distribution.

In addition, the distribution is linear

$$y_{min} + (y_{max} - y_{min}) \frac{1}{2} = \bar{y} \qquad (2\text{-}44)$$

The operational practices that will provide this improved water distribution are discussed in the following chapter. However, assuming for the moment that another water distribution can be obtained that fulfills Eq. 2-43 and 2-44, the irrigation performance can be improved. If, for example (see Fig. 2-10), it is decided to reduce the total actual applied depth from $\bar{y}$ = 107 mm to $\bar{y}$ = 90 mm, solving Eqs. 2-43 and 2-44 yields: y_{min} = 63 mm, y_{max} = 117 mm and the irrigation deficiencies are:

$$h_C = \frac{117 - 90}{2} \times \frac{1}{2} = 6.8 \text{ mm}$$

$$h_B = (75 - 63) \times \frac{0.22}{2} = 1.3 \text{ mm}$$

and the efficiencies are:

$$U_d = \frac{90 - 6.8}{90} = 0.925$$

$$E_P = \frac{75 - 1.3}{90} = 0.818$$

$$E_S = \frac{75 - 1.3}{75} = 0.982$$

By changing the depth of application from 107 to 90 mm, with the same distribution uniformity (U_d = 0.925), the irrigation performance is improved by significantly increasing E_P from 0.7 to 0.818 (Fig. 2-10). The effect of a change of the average depth of application, $\bar{y}$, for other levels of $\bar{y}$ is given in Table 2-3 and Fig. 2-10.

It can be noticed that even decreasing the average applied depth to $\bar{y}$ = 70 mm, which is below the required depth, h_R, provides satisfactory irrigation while maintaining an acceptable storage efficiency. However, further decreasing the applied depth, say to 60 or 50 mm, results in an unsatisfactory value of E_S.

Table 2-3. Irrigation efficiencies for a linear-symmetrical distribution - test results and suggested improvements.

Parameter/ Irrigation	h_R (mm)	y_{min} (mm)	y_{max} (mm)	$\bar{y}$ (mm)	U_d	E_P	E_S	Category
tested	75	75	139	107	0.925	0.7	1.0	VI
improved	75	63	117	90	0.925	0.818	0.982	I
improved	75	56	104	80	0.925	0.89	0.950	I
improved	75	49	91	70	0.925	0.956	0.89	I
improved	75	42	78	60	0.925	0.978	0.79	II
improved	75	35	65	50	0.925	1.00	0.66	II

b. Linear and nonsymmetrical distribution of water depth.

In cases where the basin surface is not well graded and leveled, and there are small depressions over the surface, an estimation of deep percolation is often carried out. After the surface water has disappeared from most of the basin, and only ponded water remains over portions of the basin surface, the area covered by the ponded water and its average depth, h_P, is estimated by direct measurement of the depth and area of the ponds.

In this case, in addition to knowing $\bar{y}$, and the assumption that $y_{min} = h_R$, the depth of water that infiltrated into the ponded area, y_{max}, and its associated area are also known.

The water distribution can be established by locating point 4 on the $\bar{y}$ line (Fig. 2-11) so that the excess application equals the deficit, in other words area 2342 equals area 4564. The straight lines that are connected to point 4 are also connected to y_{min}, for the deficient zone, and to y_{max} at the middle of its related area (point 1, Fig. 2-11), for the excess zone.

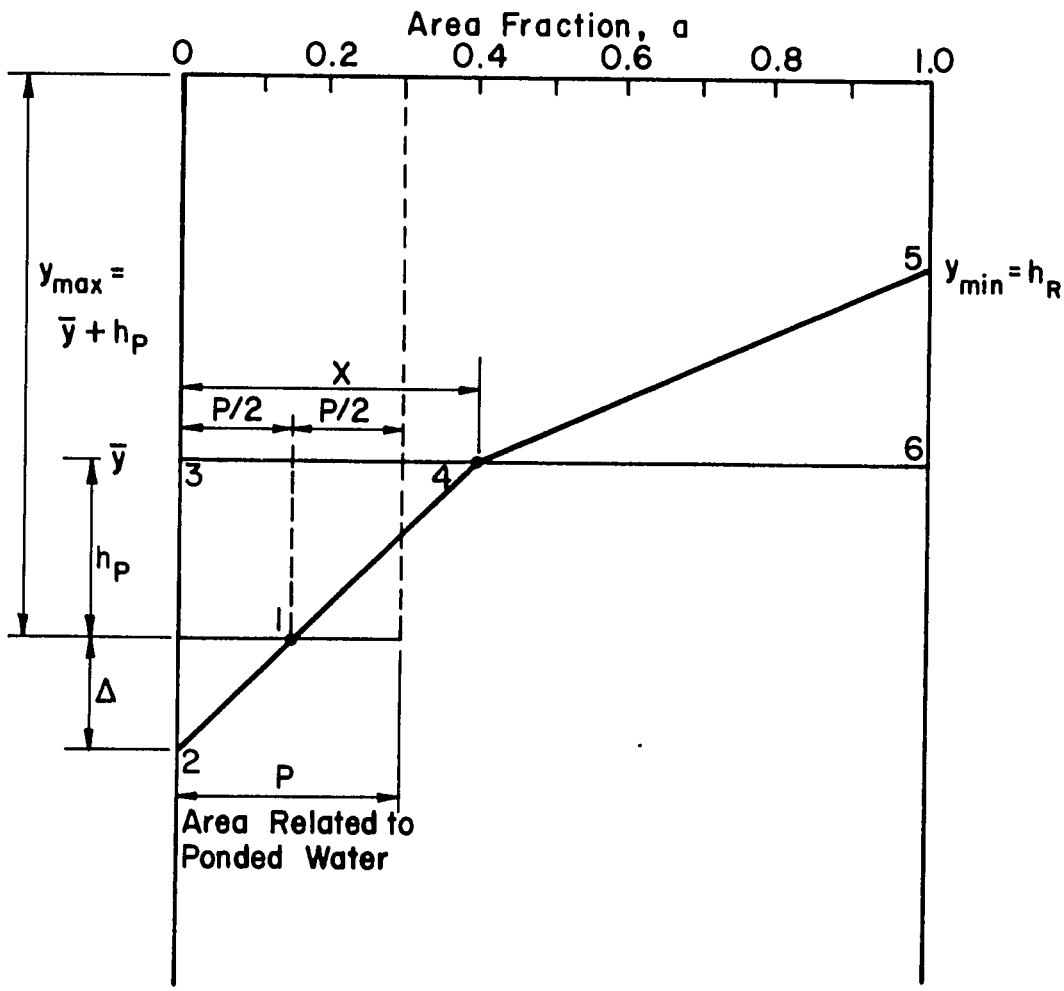


Fig. 2-11. Linear and nonsymmetrical distribution of water depths.

After locating the point 4 (see Appendix 2-I), the evaluation parameters are given by:

$$E_P = \frac{y_{min}}{\bar{y}} = S \tag{2-45}$$

$$E_S = 1$$

$$U_d = 1 - \frac{(1-S)}{2} \quad (1 - X) \tag{2-46}$$

where X is the fraction of the area, as described by the distribution, receiving more than the mean application.

It is clear that comparing this case with the former one, the only change appears in the evaluation of U_d, where instead of assuming $X = 0.5$, as in the case of a linear and symmetrical water depth distribution, X is calculated from the measured excess percolation.

The effect of X on U_d, for various values of S, is shown in Fig. 2-12. For high values of S, the effect of X is less significant. Therefore, it is suggested that X be evaluated by deep percolation measurements only for $S < 0.5$.

For higher levels of S, the estimation of $X = 0.5$, as in case a, is acceptable and there is no practical benefit from determining a more accurate value for X.

Example

A basin of 1000 m^2 requires a depth of $h_R = 75$ mm. Thus, the required water quantity is 75 m^3. The actual applied water quantity is $T = 150$ m^3 with an average depth of application of $\bar{y} = 150$ mm (Fig. 2-13). During irrigation the required depth is applied at the far end of the basin where the infiltrated depth is minimal. Also, excessive percolation is measured and it is estimated that 20% of the area has

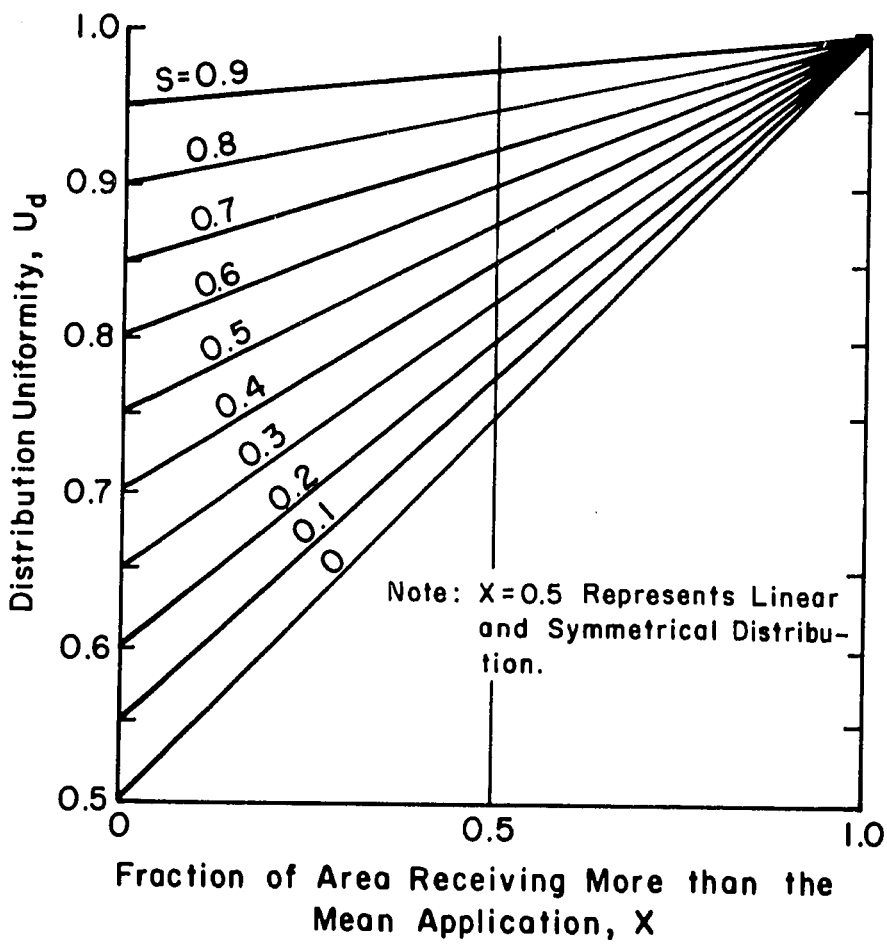


Figure 2-12. Distribution uniformity as a function of fraction of area receiving more than the mean application for various levels of S.

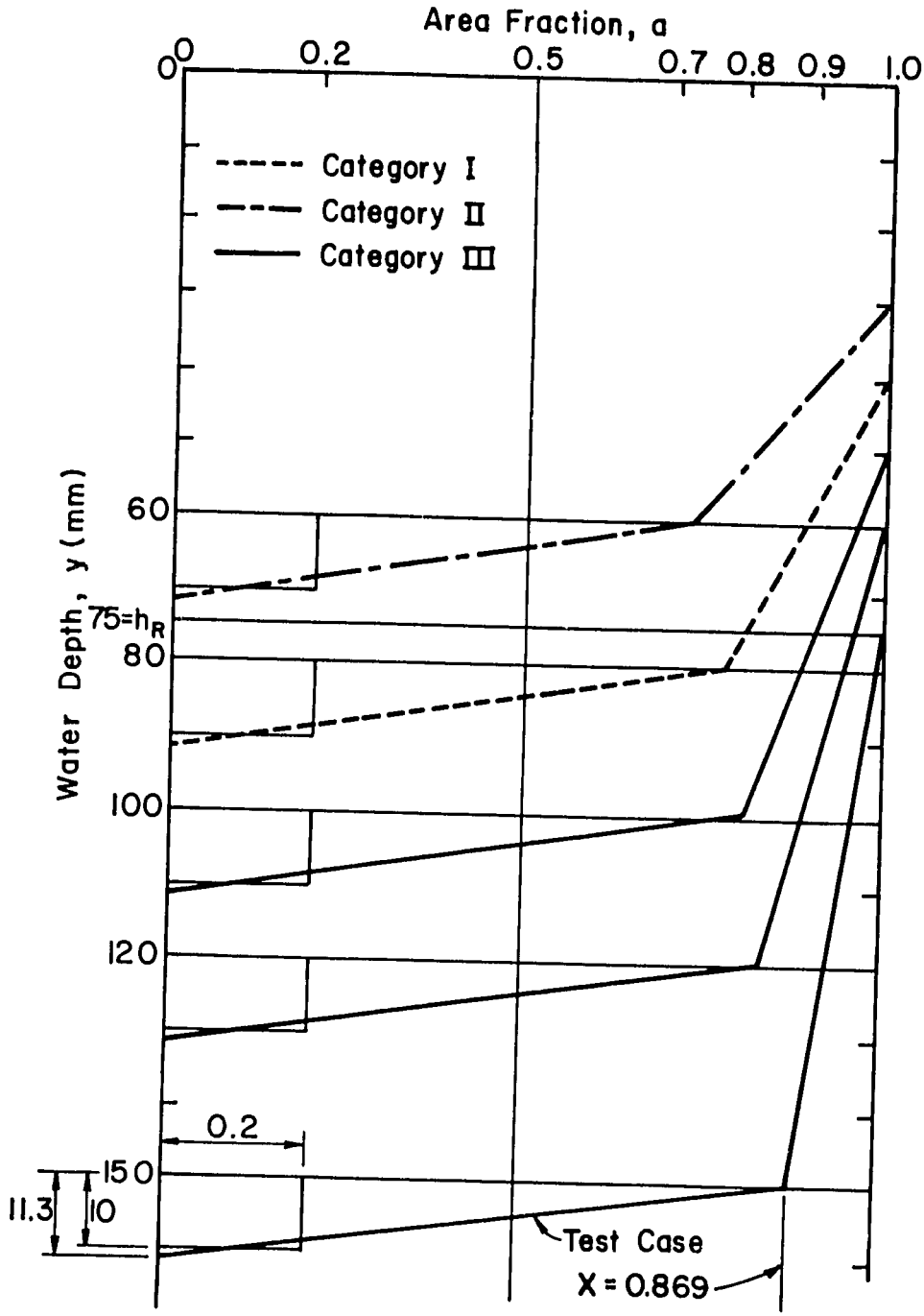


Figure 2-13. Example of water depth distribution as estimated by deep percolation.

depressions that provide an average of 10 mm excessive percolation. The water distribution is assumed to be linear but not symmetric. By referring to Fig. 2-11 and the equations in Appendix 2-A, the distance X is calculated. Given that h_P = 10 mm and $\frac{P}{2}$ = 0.1, by definition from Eq. 2-63 in Appendix 2-A, $m = \frac{\bar{y}}{h_P}(1 - S) = \frac{150\ (1 - 0.5)}{10} = 7.5$ and X = .869 from Eq. 2-65 in Appendix 2-A.

The irrigation efficiencies are:

$$E_P = S = 0.5 \qquad E_S = 1$$

$$h_C = (150 - 75) \times 0.131 \times \frac{1}{2} = 4.9 \text{ mm}$$

$$U_d = \frac{150 - 4.9}{150} = 0.967$$

U_d can be calculated also from Eq. 2-46:

$$U_d = 1 - \frac{(1 - 0.5)}{2}(1 - 0.869) = 0.967$$

If again one considers that $U_d \geq 0.9$, $E_P \geq 0.8$, and $E_S \geq 0.8$ are the satisfactory levels for these efficiencies, it can be concluded that the water distribution is satisfactory (U_d = 0.967) but E_P is low. Decreasing the average depth actually applied will improve the irrigation performance.

While decreasing the average depth of application, it can be assumed that the deep percolation (ponded water) is constant and independent of the total depth. It is further assumed that to maintain the high distribution uniformity, U_d, the ratio $\bar{y}/y_{min}$ must be kept constant also. Changes in the irrigation performance parameters for various average depths of applications are given in Table 2-4 and Fig. 2-13.

From Table 2-4, it can be concluded that decreasing the total applied depth to 120 or 100 mm improves the irrigation performance to a

Table 2-4. Irrigation efficiencies for a nonsymmetrical distribution - test results and suggested improvements.

Irrigation/ Parameter	$\bar{y}$ mm	h_R mm	y_{min} mm	X	U_d	E_P	E_S	Category
tested	150	75	75	0.869	0.967	0.5	1	IV
improved	120	75	60	0.841	0.960	0.625	0.996	IV
improved	100	75	50	0.814	0.953	0.74	0.984	IV
improved	80	75	40	0.777	0.941	0.89	0.952	I
improved	60	75	30	0.736	0.933	1.0	0.80	III

certain degree only. The deep percolation efficiency E_P is increased, but it is still low (below 0.8). With an average depth of 80 mm, the irrigation performance is in category I with satisfactory E_P, E_S and U_d. Further decreasing of the depth of application to 60 mm results in a decrease in the storage efficiency to 0.8, which means that the total depth of application is too small.

5. The Ponded Water Method

The water distribution pattern can be estimated from the total water quantity applied by assuming the excess application (over the average application) is due to ponding in depressions.

The ponded water depth, h_P, is the average depth of water over the ponded area, P (see Fig. 2-14).

The excess of water is given by:

$$h_C = h_P P \tag{2-47}$$

The deficiency of water as related to $\bar{y}$ is:

$$h_B \doteq (\bar{y} - y_{min})\ (1 - P)/2 \tag{2-48}$$

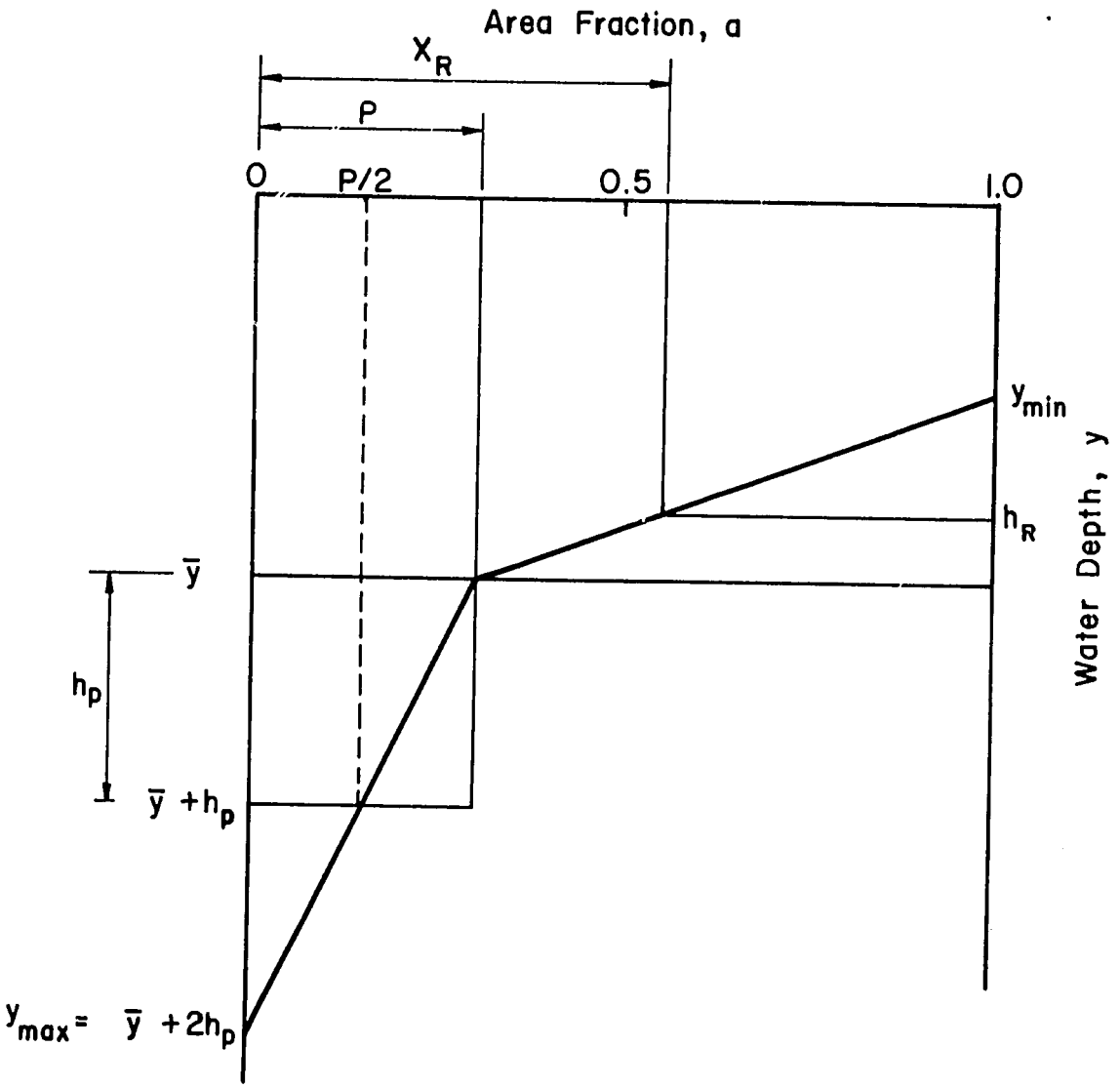


Fig. 2-14. Determination of water depth distribution by ponded water depth.

Since $h_C = h_B$:

$$(\bar{y} - y_{min}) = \frac{2h_P P}{1 - P} \tag{2-49}$$

Solving for y_{min}:

$$y_{min} = \bar{y} - \frac{2h_P P}{1 - P} \tag{2-50}$$

The water distribution in the excess zone is estimated to be linear, which gives $y_{max} = \bar{y} + 2h_P$.

For a given required depth of application, the irrigation performance measured by the water distribution given in Fig. 2-14 can be evaluated as:

$$U_d = \frac{\bar{y} - h_P P}{\bar{y}} \tag{2-51}$$

Assuming for simplicity that $h_R < \bar{y}$, the deficiency of water related to a required depth, h_R, is given by:

$$h_B = (h_R - y_{min})\ (1 - X_R)\ /2 \tag{2-52}$$

where the fractional area with excess application (Fig. 2-14), X_R, can be expressed by:

$$X_R = 1 - \frac{(h_R - y_{min})\ (1 - P)}{(\bar{y} - y_{min})} \tag{2-53}$$

Substituting Eq. 2-53 into Eq. 2-52:

$$h_B = \frac{(h_R - y_{min})^2\ (1 - P)}{2\ (\bar{y} - y_{min})} \tag{2-54}$$

Substituting the value of y_{min} as given by Eq. 2-50:

$$h_B = \left(h_R - \bar{y} + \frac{2h_P P}{1 - P}\right)^2 \frac{(1 - P)^2}{4h_P P} \tag{2-55}$$

with h_B given by Eq. 2-55, E_P and E_S can be expressed by:

$$E_P = \frac{h_R - \left(h_R - \bar{y} + \frac{2h_P P}{1 - P}\right)^2 \frac{(1 - P)^2}{4h_P P}}{h_R} \tag{2-56}$$

$$E_S = \frac{h_R - \left(h_R - \bar{y} + \frac{2h_P P}{1 - P}\right)^2 \frac{(1 - P)^2}{4h_P P}}{\bar{y}} \tag{2-57}$$

Example

A basin of 1000 m^2 is irrigated. During irrigation, a stream of 2.0 m^3/min is delivered into the basin for 50 min. The ponded water is estimated to occur over 30% of the area (P = 0.3) with an average depth of 20 mm.

The total quantity of water applied is: $T = 2.0 \times 50 = 100\ m^3$.

The average depth of water applied is: $\bar{y} = 100$ mm.

The estimated minimal depth infiltrated (Eq. 2-50) is:

$$y_{min} = 100 - \frac{2 \times 20 \times 0.3}{1 - 0.3} = 83 \text{ mm}$$

The assumed water distribution is shown in Fig. 2-15. The distribution uniformity is:

$$U_d = \frac{100 - 20 \times 0.3}{100} = .94$$

The irrigation performance for various required depths of application, h_R, is given in Table 2-5.

If it is assumed that $U_d \geq 0.9$, $E_P \geq 0.8$, and $E_S \geq 0.8$ are satisfactory levels for the irrigation efficiencies, the given water distribution is satisfactory if the required water depth is in the range of 90 - 120 mm. If the required depth is below 90 mm, the total depth should be decreased. If it is above 120 mm, the total depth should be increased.

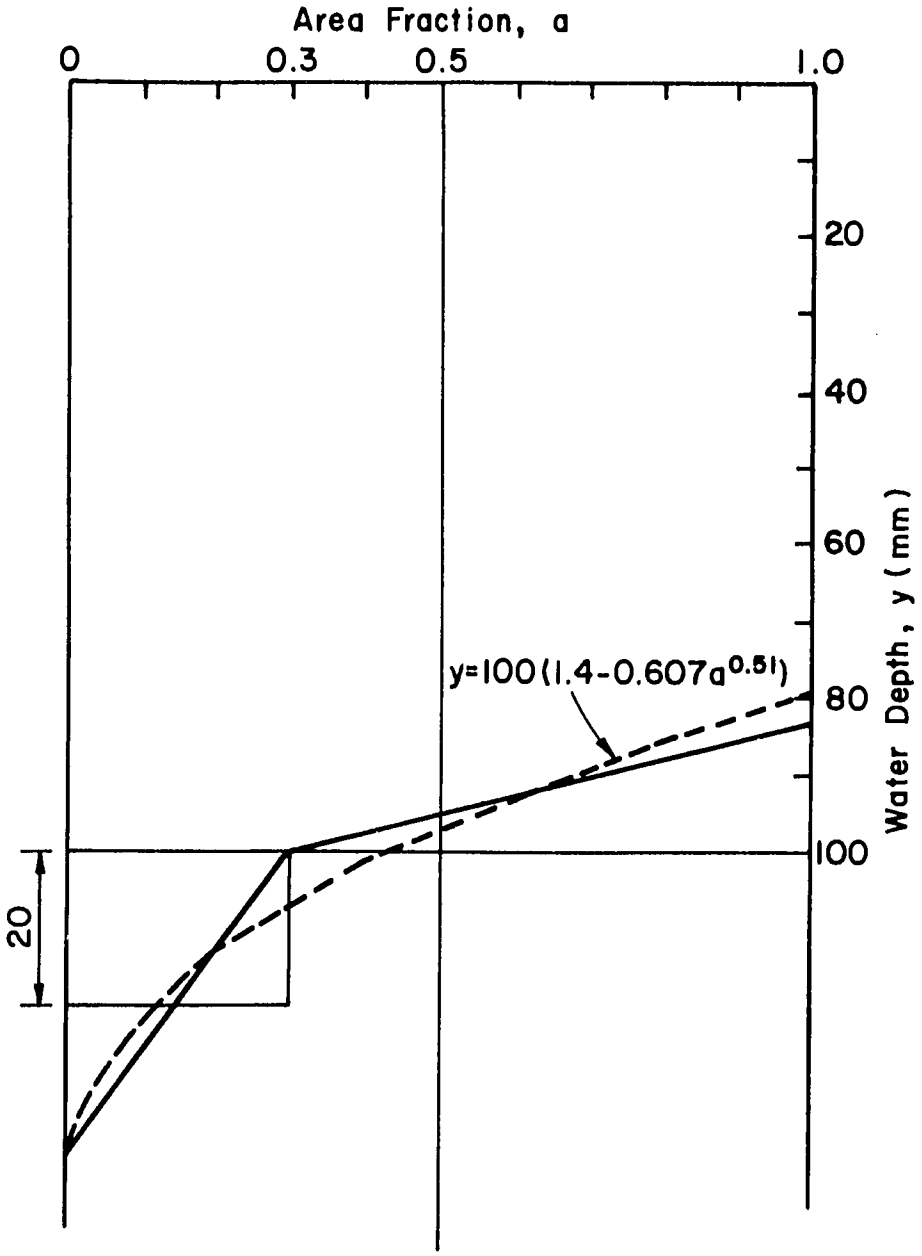


Figure 2-15. Example of water distribution as estimated by the ponded water method.

Table 2-5. Irrigation performance paramenters for various required water depths, for example on page 52.

h_R mm	U_d	E_p	E_S	Category
60	0.94	0.6	1.0	IV
70	0.94	0.7	1.0	IV
80	0.94	0.8	1.0	IV
90	0.94	0.889	0.988	I
100	0.94	0.94	0.94	I
110	0.94	0.966	0.878	I
120	0.94	0.985	0.821	I
130	0.94	0.976	0.766	III
140	0.94	1.0	0.719	III
150	0.94	1.0	0.666	III

APPENDIX 2-A

CALCULATION OF AREA FRACTION X

The notation for this appendix in shown in Fig. 2-11.

The known values are h_P, P, y_{min}, and $\bar{y}$. The excess application (area 2342) is

$$h_C = \frac{X(h_P + \Delta)}{2} \tag{2-58}$$

The deficit (area 4564) is

$$h_C = \frac{(1 - X)\ (\bar{y} - y_{min})}{2} \tag{2-59}$$

By proportions from Fig. 2-11

$$\frac{\Delta}{P/2} = \frac{X}{h_P + \Delta} \tag{2-60}$$

or rearranging

$$\Delta = \frac{h_P \cdot P/2}{X - P/2} \tag{2-61}$$

Setting Eq. 2-58 equal to Eq. 2-59, substituting in Eq. 2-60, and letting $y_{min} = S\bar{y}$ results in

$$\frac{Xh_P}{2}\left[1 + \frac{P/2}{X - P/2}\right] = \frac{(1 - X)\bar{y}\ (1 - S)}{2} \tag{2-62}$$

If one lets

$$\frac{\bar{y}\ (1 - S)}{h_P} = m \tag{2-63}$$

Eq. 2-62 becomes

$$X\left(1 + \frac{P}{2X - P}\right) = m\ (1 - X) \tag{2-64}$$

which can be readily solved for X to give

$$X = \frac{m(P + 2) \pm \sqrt{m^2(2-P)^2 - 8Pm}}{4(m + 1)} \tag{2-65}$$

APPENDIX 2-B

POWER CURVE FITTING TO WATER DISTRIBUTION PATTERNS

In the cases discussed previously, the water distribution has been assumed to be linear. This enables a simple determination of the water distribution and the efficiencies and uniformity coefficient. However, other types of water distributions, which under certain circumstances may better represent the actual infiltrated water depths, may be used also. Karmeli (1978) has suggested a power function as a distribution function for the infiltrated water depth under surface irrigation.

The power function distribution in its dimensional form is given by:

$$y = f' + g' a^b \tag{2-65}$$

Where y is the infiltrated water depth, a is the fraction of the area receiving a depth of water y or more, and f', g' and b are the function parameters. The mean of the distribution is given by:

$$\bar{y} = \int_0^1 y da = f' + g'/(b+1) \tag{2-67}$$

The distribution can be expressed in nondimensional form as:

$$H = f + ga^b \tag{2-68}$$

where

$$H = y/\bar{y} \tag{2-69}$$

$$f = f'/[f' + g'/(b+1)] \tag{2-70}$$

$$g = g'/[f' + g'/(b+1)] \tag{2-71}$$

Examples of power distribution functions are shown in Fig. 2-16. From Eq. 2-68 and Fig. 2-16 the following can be derived:

$$f = H_{max} \tag{2-72}$$

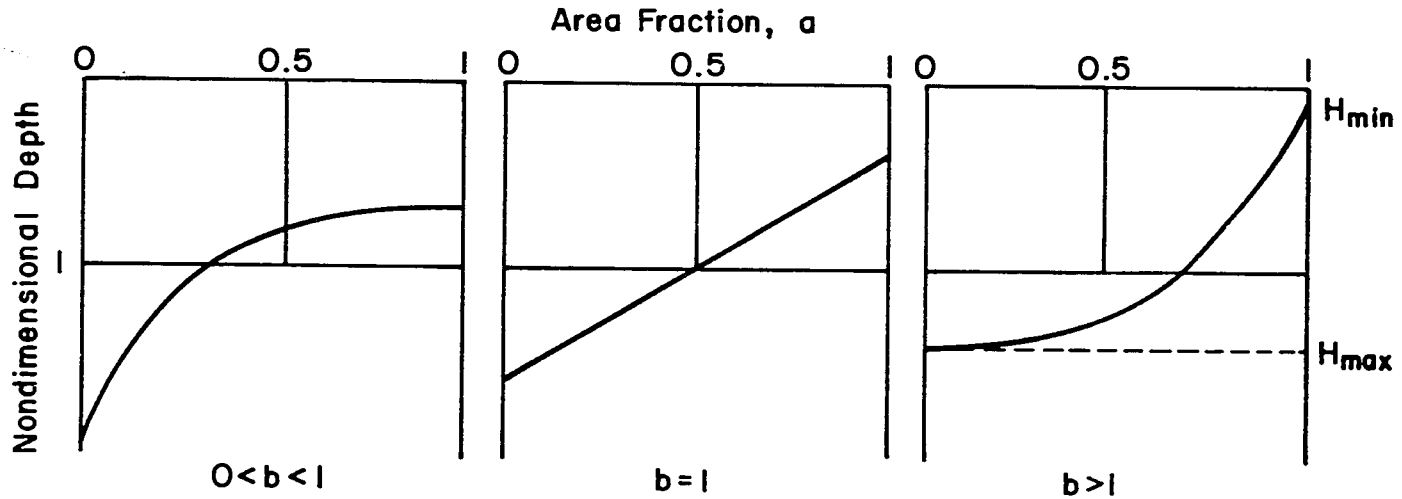


Fig. 2-16. Power water distribution function for various values of the exponent b.

$$g = H_{min} - H_{max} \tag{2-73}$$

Also as

$$\int_{o}^{1} H da = 1 \tag{2-74}$$

the parameter b can be evaluated as:

$$b = \frac{1 - g - f}{f - 1} = \frac{1 - H_{min}}{H_{max} - 1} \tag{2-75}$$

The exponent b determines the nature of the function (Fig. 2-16). 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 the 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.

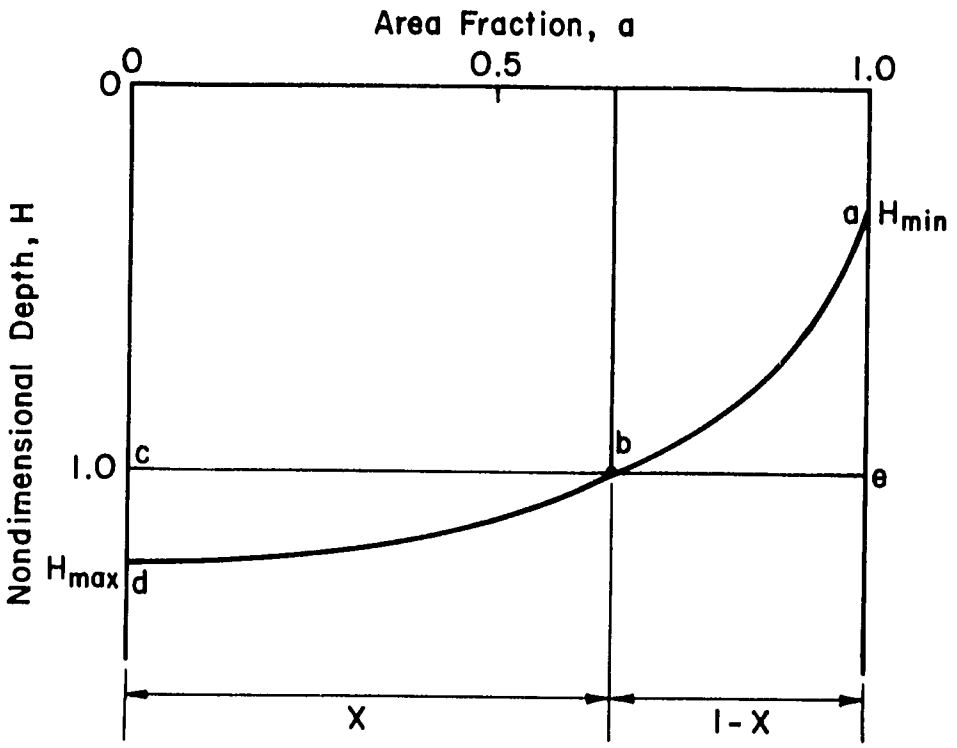
The power function as given in Eq. 2-68 is completely defined when the two nondimensional depths, H_{min} and H_{max}, are known. Therefore, to solve for these two depths, or in other words, to solve for f, g and b, two equations must be available. These equations can be derived from the field data. For the various methods discussed previously, the field data may be used in two different manners to derive the constants in the power equation.

Case a

The minimum depth, H_{min}, (or the maximum depth, H_{max}) is given or estimated, and the deficient or excess quantity is also given (Fig. 2-17). If both H_{min} and H_{max} are known, the coefficients f, g, and b can be derived directly from Eq. 2-72, 2-73, and 2-75.

Case b

The deficit or excess is given and the area fraction, X, that receives the average depth of application or higher is also given (Fig. 2-17).



Case a: H_{min} is Given

H_c = abea is Given

Case b: H_c = abea is Given

X is Given

Fig. 2-17. Definition of terms for the fitting of power distribution function to field data.

1. Curve Fitting for Case a

The power function coefficients are expressed by:

$$f = H_{max} \qquad g = -H_{max} + H_{min} \qquad b = \frac{1 - H_{min}}{H_{max} - 1} \tag{2-76}$$

or expressing g and b in terms of f and H_{min}

$$g = -f + H_{min} \qquad b = \frac{1 - H_{min}}{f - 1} \tag{2-77}$$

The nondimensional deficit, H_C, can be expressed by

$$H_C = 1 - X - \int_X^1 H da \tag{2-78}$$

substitution of Eq. 2-68 into Eq. 2-78 and carrying out the integration results in

$$H_C = (1-X)(1-f) - \frac{g}{b+1}(1-X^{b+1}) \tag{2-79}$$

As $$\bar{H} = 1 = f + gX^b \tag{2-80}$$

$$X = \left[\frac{1 - f}{g}\right]^{1/b} \tag{2-81}$$

But from Eq. 2-77 it can be shown that

$$\frac{1 - f}{g} = \frac{1}{b + 1} \tag{2-82}$$

so that

$$X = \left(\frac{1}{b+1}\right)^{1/b} \tag{2-83}$$

In other words, the fraction of the area receiving the mean depth or greater is a function only of the power parameter in the power distribution. Also, for $b \to 0$ the limit of X is $X = e^{-1}$ so that the minimum value of X is 0.368. Substitution of Eqs. 2-82 and 2-83 into Eq. 2-79 results in

$$\frac{H_C}{f - 1} = b\left[\frac{1}{b+1}\right]^{\frac{1}{b} + 1} \tag{2-84}$$

When H_C and f are known, b can be calculated by iterative means from Eq. 2-84 or by using Fig. 2-18 which shows b as a function of $H_C/(f-1)$.

When H_{min} is known, Eq. 2-84 can be written as

$$\frac{H_C}{1 - H_{min}} = \left[\frac{1}{b+1}\right]^{\frac{1}{b} + 1} \tag{2-85}$$

so that b can be calculated by iterative means. Fig. 2-18 also shows b as a function of $H_C/(1-H_{min})$.

Once b, and either f (H_{max}) or H_{min} are known, the other parameters, g and H_{min} or f, can be calculated from Eq. 2-77.

2. Curve Fitting for Case b

When H_C and X are known, b can be calculated iteratively from Eq. 2-83 or found from Fig. 2-18. (Note that X must be greater than 0.368). Once H_C and b are known, f can be calculated from Eq. 2-84, and H_{min} can be calculated from Eq. 2-85, or the curves from Fig. 2-18 can be used to determine f and H_{min}.

3. Fitting the Power Distribution Curves to the Various Field Evaluation Methods

a. The recession method

For this method both cases a and b may be used. When the minimum depth is taken as y_1 (Eq. 2-29), H_{min} can be calculated from either Eq. 2-23 or Eq. 2-33 as

$$H_{min} = \frac{y_1}{\bar{y}} \tag{2-86}$$

In some situations it may be reasonable to assume that H_{min} is even less than that given by Eq. 2-86.

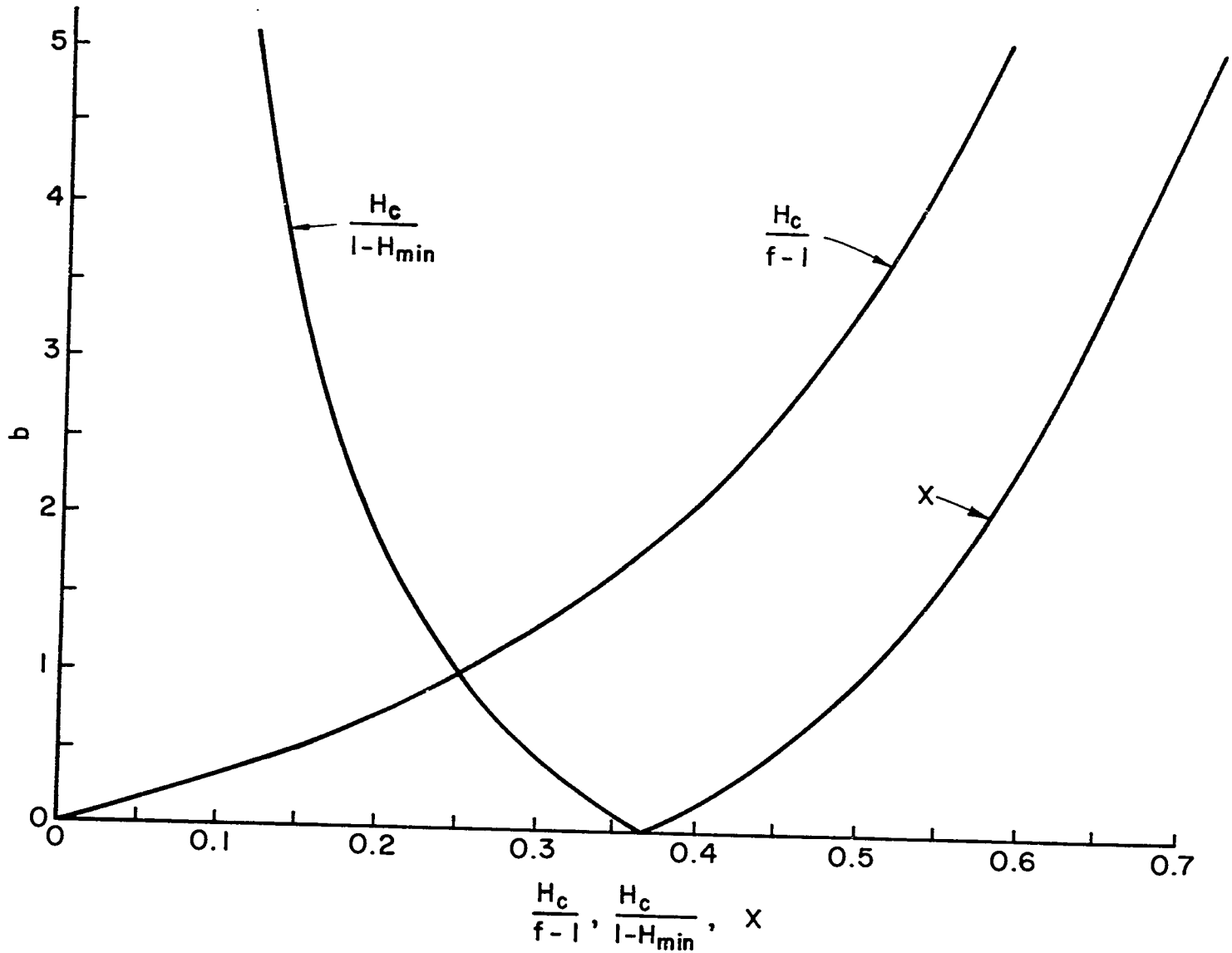


Fig. 2-18. Nondimensional curves for solution of power function distribution.

The nondimensional deficiency, H_C, can be calculated from

$$H_C = \sum_{j=1}^{p} \left(1 - \frac{y_j}{\bar{h}}\right) a_j \qquad (2\text{-}87)$$

where p is the number of area fractions receiving less than the mean application.

Example

Using the data from the first example in section 3 (Fig. 2-7)

$$H_C = \left(1 - \frac{77}{90}\right) \times 0.333 = 0.048$$

If it is assumed that y_{min} = 63 mm, H_{min} - 63/90 = 0.7, and from Fig. 2-18 b = 2.85. Use of Eq. 2-76 results in f = 1.11, and g = -0.41 so that the resulting power distribution equation is

$$H = 1.11 - 0.41a^{2.85}$$

If case b is used for the recession approach, X is the fraction of area receiving greater than the average depth.

Example

Using the same set of data as above, X = 0.667 (note that $X > e^{-1}$). From Fig. 2-18, b = 3.80. Use of Eq. 2-84 results in f = 1.09, and Eq. 2-85 gives H_{min} = 0.65. From Eq. 2-76, g = -0.44 so the power distribution equation is

$$H = 1.09 - 0.44a^{3.80}$$

Since the power function is not sensitive to changes in the value of b for b > 1.0, the difference between b = 2.85 and b = 3.8 as obtained by the two cases, does not significantly affect the distribution function. The water distributions for these two cases are shown in Fig. 2-7.

b. The excess method

With only H_{min} given, and no other reasonable assumption to provide other data, the assumption of $H_{max} - 1 = 1 - H_{min}$ is accepted.

This results in b = 1, which is the linear distribution. With the non-symmetrical distribution (Fig. 2-11) H_{min} is known and the nondimensional deficiency is given by:

$$H_C = P \frac{h_P}{\bar{y}} \tag{2-88}$$

These values of h_{min} and H_C are used to determine the power distribution as in case a.

Example

The excessive percolation was estimated to cover an area fraction of 0.3 with an average nondimensional depth of $H_P = 0.25$. The minimum depth was also found to be $H_{min} = 0.5$.

For this case the nondimensional excessive depth is given by Eq. 2-88.

$$H_C = \frac{0.3 \times 0.25}{1.0} = 0.075$$

With the known H_{min} and H_C, the LHS of Eq. 2-85 is calculated:

$$\frac{H_C}{1 - H_{min}} = \frac{0.075}{1 - 0.5} = 0.15$$

From Eq. 2-85 or Fig. 2-18 the power term b is found to be 3.3.

The LHS of Eq. 2-84 is calculated directly from Eq. 2-84 or Fig. 2-18.

$$\frac{H_C}{f - 1} = 0.49$$

from which f is found to be 1.153.

Finally, g is calculated by

$$g = H_{min} - f = 0.5 - 1.153 = -0.653$$

and the power water distribution is given by:

$$H = 1.153 - 0.653a^{3.3}$$

The water distribution is shown in Fig. 2-19. The fraction of the area receiving the mean depth or greater, in other words, the area with excess application is

$$X = \left[\frac{1.153 - 1}{0.653}\right]^{1/3.3} = 0.644$$

c. The ponded water method

For this method, H_C is given by Eq. 2-47, and X = P (See Fig. 2-14). The maximum nondimensional depth is estimated as

$$H_{max} = \frac{2h_P}{\bar{y}} + 1 \qquad (2\text{-}89)$$

Thus, either case a or case b can be used to determine the power distribution function. However, when $P < e^{-1}$ only the solution for case a is possible and it yields a larger fractional area with excessive irrigation.

Example

Using the data from the example in Section 4 (Fig. 2-15) P = X = 0.3, h_P = 20 mm, $\bar{y}$ = 100 mm.

H_{max} is calculated by Eq. 2-89 as:

$$H_{max} = 2\,\frac{20}{100} + 1 = 1.4$$

Since $X < e^{-1}$, only case a can be used to solve for the water distribution parameters. The power term b is solved by Eq. 2-89 or Fig. 2-18 for $H_C = 0.2 \times 0.3 = 0.06$ and $\frac{H_C}{f-1} = \frac{0.06}{0.4} = 0.15$, so b = 0.51. From Fig. 2-18 $\frac{H_C}{1-H_{min}} = 0.29$ resulting in $H_{min} = 1 - \frac{H_C}{0.29} = 1 - 0.207 = 0.793$.

The water distribution function is:

$$H = 1.4 + (0.793 - 1.4)a^{0.51} = 1.4 - 0.607a^{0.51}$$

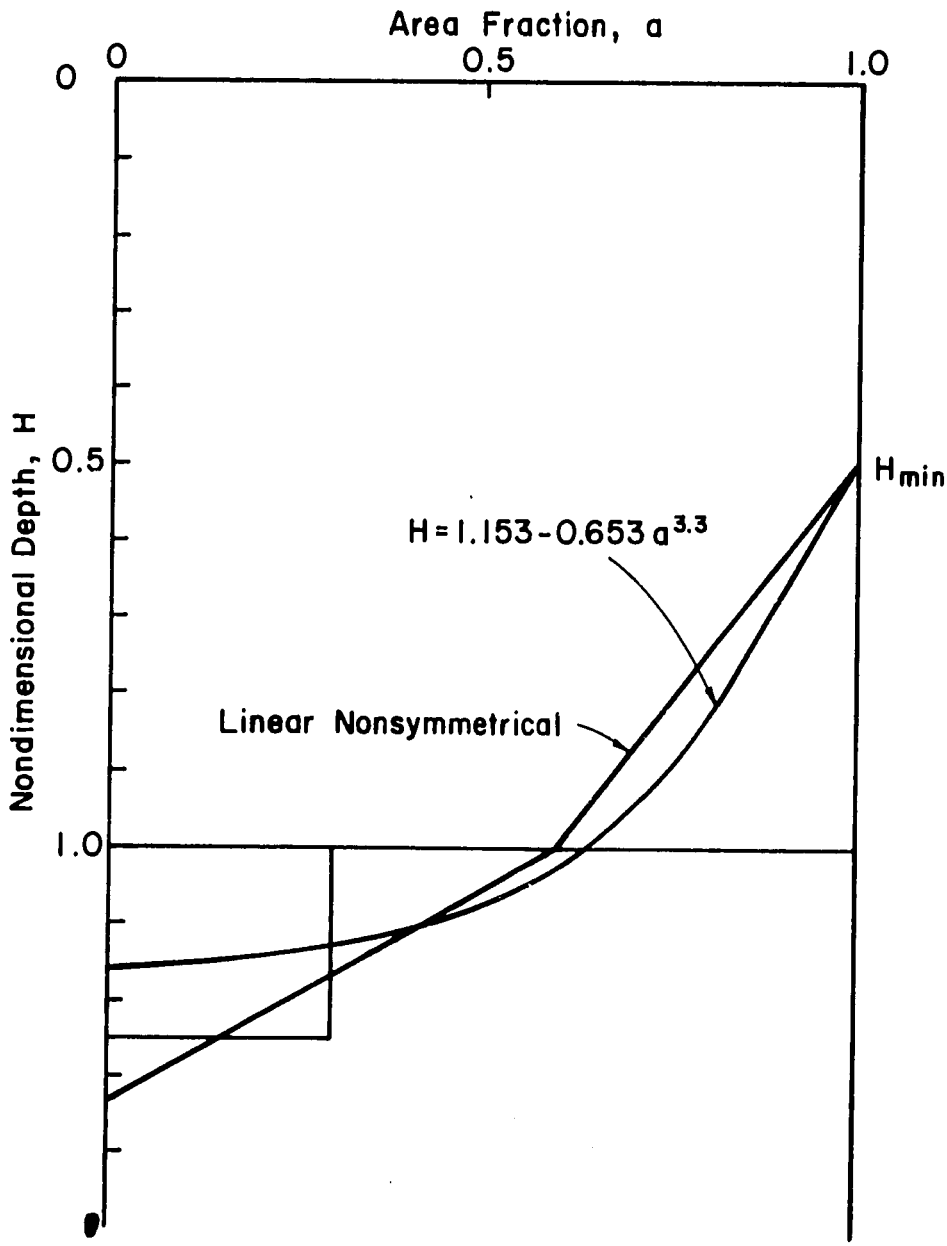


Figure 2-19. Example of water distribution for the excess method.

This distribution function is also shown in Fig. 2-15. The fraction of area receiving the mean depth or greater is

$$X = \left(\frac{1.4 - 1}{0.607}\right)^{1/0.51} = 0.441$$

Chapter 3

THEORETICAL WATER DISTRIBUTION MODELS FOR BASIN IRRIGATION

1. Introduction

Theoretical models for the prediction of water distribution patterns in basin irrigation can be developed and used only for regular basins that are characterized by the following:

a. The basin is almost level with a uniform and smooth graded surface,

b. The water flows along the axis of the basin with a uniform water front across the basin,

c. The irrigation process includes four successive stages:

(i) Advance of water front along the basin. During this stage, the inlet stream flows and water advances until the basin is just covered.

(ii) Ponding of water over the whole basin. In this stage the basin is already covered with water. The inlet stream flows with a flow rate much greater than the overall infiltration into the basin. Consequently, part of the water infiltrates and part is ponded within the basin.

(iii) Depletion. In this stage no more inflow occurs. The ponded water infiltrates into the soil until the upper end of the basin surface is exposed.

(iv) Recession. The remaining water over the basin infiltrates while gradually exposing the basin surface.

Some basic assumptions must be made to develop a theoretical model that will describe the basin water distribution pattern:

a. Recession is negligible with well leveled basins, however, when there is a slope in the basin, recession is a function of the depth of water over the surface, resulting from the slope. (For more details see the Simplified Basin Irrigation Model, Chapter 4.)

b. The depth of water ponded over the surface does not affect infiltration. This assumption is especially true when large portions of the soil profile are already wetted.

c. Evaporation losses are negligible and there is no runoff. Consequently, all the water that has been delivered into the basin is absorbed by it.

A water distribution pattern model must provide a specific function for the depth of water infiltrated, $y_x(t)$, as related to the distance along the basin from the inlet end, x, at time t. However, often the function is not required for all t, but may be required only for $t=t_r(x)$, where $t_r(x)$ is the time that water receded from point x, also known as the recession time. Thus, the final water distribution, y_x, from which the distribution uniformity and irrigation efficiencies are calculated is:

$$y_x = y_x \left[t_r(x)\right] \tag{3-1}$$

The model must provide the water distribution pattern and efficiencies for a specific set of values for the system parameters studied. Then, when a change is desired, the parameters are changed, and a new set of results is obtained. This is an iterative process towards a desired solution (Fig. 3-1).

2. A General Model of y_x

The depth of water infiltrated into the soil is obtained from infiltration equations. Several infiltration equations have been developed,

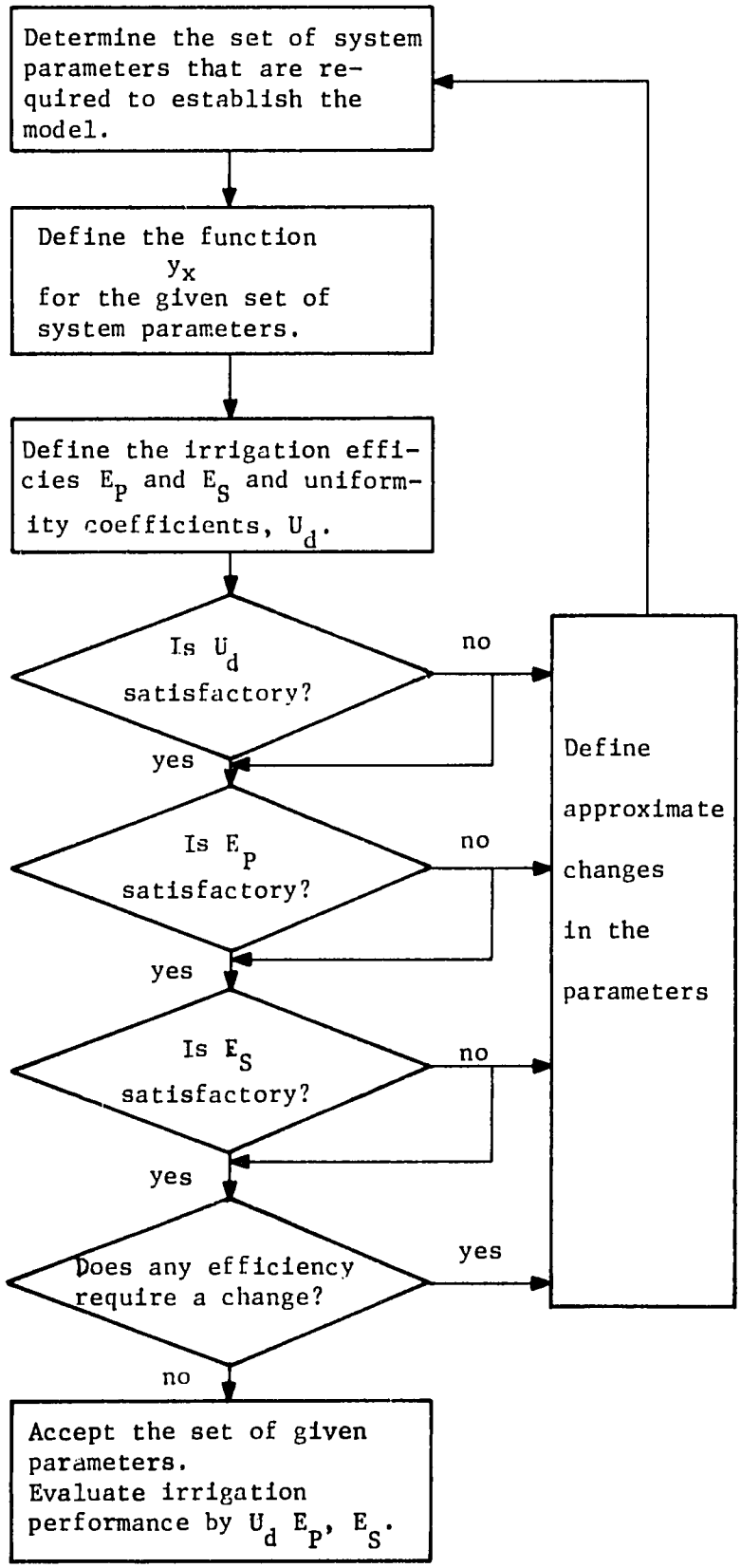


Figure 3-1. Use of the y_x model for the evaluation and improvement of basin irrigation.

but a survey of these equations is beyond the scope of this report. One of the most commonly used infiltration equations is the well known modified Kostiakov equation, in which:

$$I = k\ t_{op}^{n} + C \tag{3-2}$$

$$z = \frac{k}{n+1}\ t_{op}^{n+1} + C\ t_{op} = A\ t_{op}^{B} + C\ t_{op} \tag{3-3}$$

where

I = infiltration rate (LT^{-1})

t_{op} = infiltration opportunity time (T)

C = basic infiltration rate, which is the infiltration rate for large t_{op} (LT^{-1})

k = constant (dependent on soil properties and units)

n = constant (dependent on soil properties) $-1 \leq n \leq 0$

A = constant, $A = \frac{k}{n+1}$

B = constant, $B = n + 1$ $0 \leq B \leq 1.0$

z = cumulative infiltrated depth of water (L)

Usually, irrigation takes place when the infiltration is mainly governed by the power term and C is neglected, so that

$$z = A\ t_{op}^{B} \tag{3-4}$$

For basin irrigation, the opportunity time varies along the basin length, x, so that $y_x(t)$ is a function of x through the opportunity time. The opportunity time at any point at a distance, x, from the upper basin end is given by:

$$t_{op} = t_r(x) - t_a(x) \tag{3-5}$$

where

$t_a(x)$ is the time water first arrived at point x. This is also called the advance time.

When recession can be neglected and it is assumed that the water disappears from the entire basin surface at the same time (an assumption which is accepted with level or almost level basins), the recession time is a constant expressed by

$$t_r(x) = t_b + t_{aL} \text{ for any } 0 < x \leq L \tag{3-6}$$

where

t_b is the time for the infiltration of the water at the far end of the basin

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

Substituting Eqs. 3-6 and 3-5 into the infiltration equation, Eq. 3-4, the infiltrated depth at any point x is given by

$$y_x = A \left[t_b + t_{aL} - t_a(x) \right]^B \tag{3-7}$$

The infiltrated depth, y_x, as given in Eq. 3-7 is a function of other parameters that are related to the time terms as follows:

a. t_b is a function of the infiltrated depth at the lower end. For level basins and no recession, it is related to the minimum depth infiltrated, which sometimes is taken as the required depth. When the water distribution is calculated at a specific time (for example cutoff time or depletion time), t_b is given by this specific time minus the advance time to the end.

b. $t_a(x)$ and t_{aL} are functions of the distance x or length of the basin L, the inlet stream size Q, the infiltration equation, and the hydraulic parameters (bed slope, roughness coefficient and flow cross section).

Assuming that $t_a(x)$ can be defined in terms of x and the other parameters, and t_b can be calculated for a known depth by Eq. 3-4, or taken as a known time, the water distribution profile can be expressed as a function of x and the other parameters. With y_x as given in Eq. 3-7, the following irrigation performance parameters can be derived (for notation refer to Fig. 3-2).

The total volume of water infiltrated (per unit of basin width) is:

$$V = A_1 + A_2 = \int_{x=0}^{L} y_x \, dx \tag{3-8}$$

$$A_2 = y_{min} L \tag{3-9}$$

$$A_1 = \int_{x=0}^{L} y_x \, dx - y_{min} L \tag{3-10}$$

$$A_3 = \bar{y} \, (L-X) - \int_{x=X}^{L} y_x \, dx \tag{3-11}$$

$$A_4 = h_R \, (L-X_R) - \int_{x=X_R}^{L} y_x \, dx \tag{3-12}$$

$$\bar{y} = \frac{V}{L} = \frac{1}{L} \int_{x=0}^{L} y_x \, dx \tag{3-13}$$

where

V = total volume of water infiltrated per unit of width

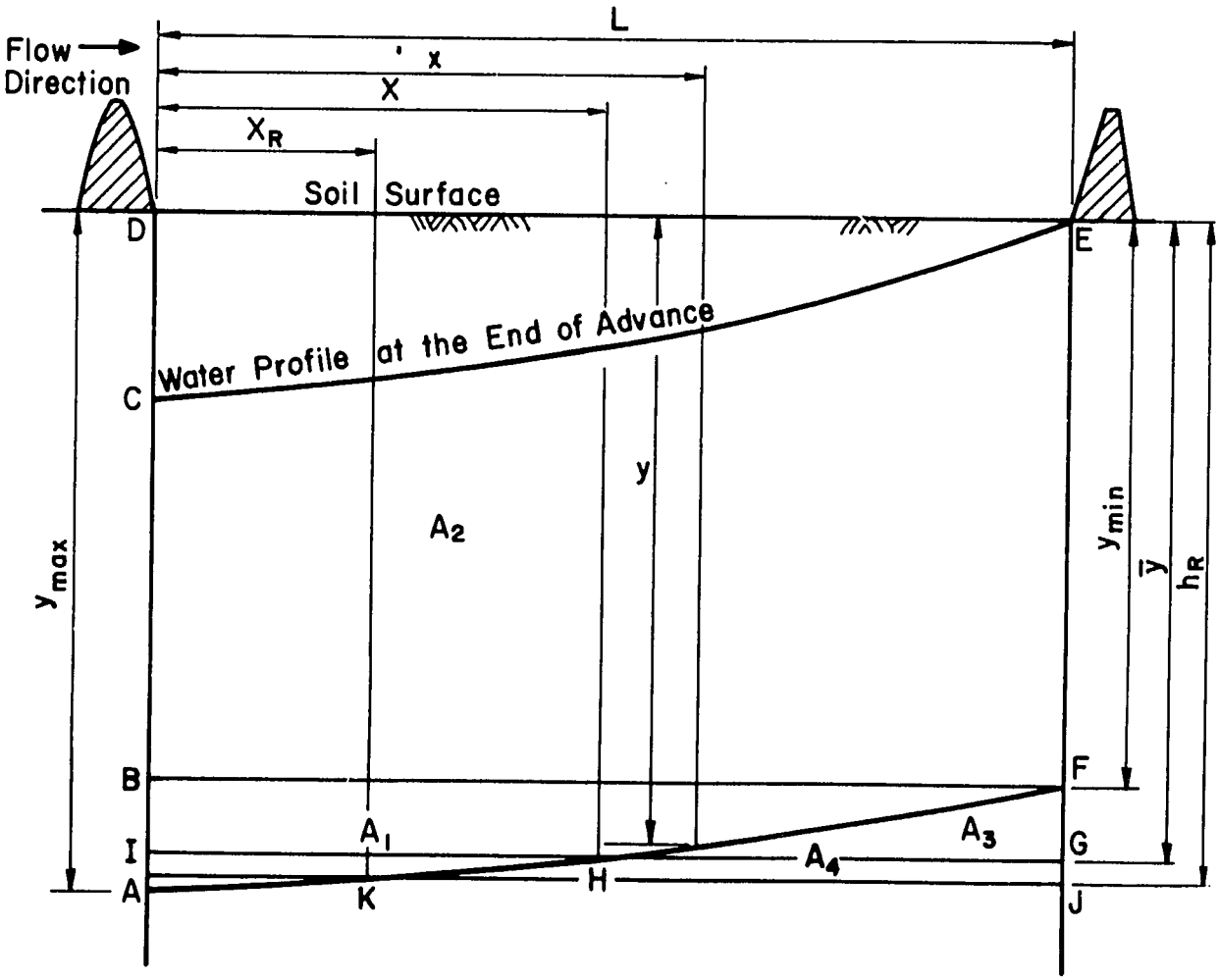
$A_1 + A_2$ = area confined by ADEFA (in Fig. 3-2)

A_1 = area confined by ABFHA

A_2 = area confined by BDEFB

A_3 = area confined by HFGH

A_4 = area confined by KFJK



A_1 = ABFHA

A_2 = BDEFB

A_3 = HFGH

A_4 = KFJK

h_R = Required Depth of Application

Figure 3-2. Water profile under regular basin irrigation.

Since A_1, A_2, A_3, and A_4, are function of $\int y_x \, dx$, they are functions of all the parameters associated with y_x including the time terms t_b, t_{aL}, and $t_a(x)$.

The irrigation performance parameters can be derived from y_x as follows (Part 1, Chapter 5):

$$U_d = \frac{\bar{y} - \frac{A_3}{L}}{\bar{y}} \tag{3-14}$$

$$E_P = \frac{h_R - \frac{A_4}{L}}{\bar{y}} \tag{3-15}$$

$$E_S = \frac{h_R - \frac{A_4}{L}}{h_R} \tag{3-16}$$

3. Use of the General y_x Model

To use the general model, as given in Eq. 3-7, to derive the various parameters and efficiencies (Eqs. 3-8 to 3-16) requires that the infiltration characteristics (A, B) and the times, t_b, t_{aL} and $t_a(x)$ be determined, so that the integrals in Eqs. 3-10 to 3-12 can be evaluated. Most of the procedures used for determining these parameters in border and furrow irrigation problems can be used for basins.

a. The infiltration characteristics.

Obtaining the infiltration characteristics, A and B, is most commonly done through field tests. There are various well known field techniques that enable the estimation of the infiltration function and the determination of A and B (see Appendix A).

b. The time for the infiltration of the minimum depth - t_b.

This time term is a constant and can be determined in several ways.

(i) Direct measurement in the field. This time is measured from the moment the water front arrives at the far end of the basin, to the moment the water disappears from the basin.

(ii) t_b can be estimated through the evaluation of the infiltrated depth, y_{min}, and the infiltration equation: $t_b = \left[\frac{y_{min}}{A}\right]^{1/B}$. In this case, y_{min} is determined by direct field measurement (moisture content and wetted depth) or estimated by other considerations ($y_{min} = h_R$ is a common assumption).

(iii) The total volume of water per unit of width, V, is known by direct measurement of the inlet discharge, Q, and the cutoff time, t_{co} ($V = Qt_{co}$). If t_{aL} and $t_a(x)$ are known (see below), Eq. 3-8 is solved for t_b since V is known also.

c. The advance time terms, t_{aL} and $t_a(x)$.

Often t_{aL} and $t_a(x)$ are assumed to be a power function of x for a given set of parameters (stream size, bed slope, surface roughness, infiltration), although there are advance equations of the form:

$$x = x[t_a(x),\ q,\ S_o,\ A,\ B,\ n] \tag{3-17}$$

in which

x	=	distance of advance
$t_a(x)$	=	time of advance
q	=	inlet discharge per unit width
S_o	=	bed slope
A, B	=	infiltration constants
n	=	roughness coefficient in Manning's equation

Even with a theoretical equation of the form of Eq. 3-17, x is calculated as a function of t for a given set of other parameters and only then is it substituted into y_x of Eq. 3-8.

The most commonly assumed functions to relate $t_a(x)$ and x are:

$$x = a[t_a(x)]^b \tag{3-18}$$

$$t_a(x) = a(e^{bx} - 1) \tag{3-19}$$

in which a, b, = constants.

The relationship between x and t as given by Eq. 3-18 or Eq. 3-19 can be obtained either by:

(i) Direct field measurement. For a known set of operating conditions the advance is measured, and a curve of the form of Eq. 3-18 or 3-19 is then fitted to the experimental data.

(ii) Theoretical models based on hydrodynamics and volume balance. These models allow the solution of the advance curve as a function of the operating conditions. Only a list of some of the most common (although not necessarily widely applied) methods is given.

Hall, 1956

Philip and Farrell, 1964

Fok and Bishop, 1965

Wilke and Smerdon, 1965

Hart, Bassett and Strelkoff, 1968

Chen, 1970

Bassett, 1972

Kincaid, Heermann and Kruse, 1972

Smith, 1972

Katopodes and Strelkoff, 1977a, b

Strelkoff, 1977

If it is assumed that the advance function is given by Eq. 3-18:

$$t_{aL} = \left(\frac{L}{a}\right)^{\frac{1}{b}} \tag{3-20}$$

$$t_a(x) = \left(\frac{x}{a}\right)^{\frac{1}{b}} \tag{3-21}$$

substituting t_{aL} and $t_a(x)$ from Eqs. 3-20 and 3-21 into Eq. 3-7 gives:

$$y_x = A\left[t_b + \left(\frac{L}{a}\right)^{\frac{1}{b}} - \left(\frac{x}{a}\right)^{\frac{1}{b}}\right]^B \tag{3-22}$$

Eq. 3-22 describes the infiltrated water profile as a function of x. Substituting y_x from Eq. 3-22 into any of the parameters of Eq. 3-8 to 3-16 allows the solution of these parameters, however, the main difficulty lies in the solution of the integral:

$$\int_{x_i}^{L} y_x \, dx = \int_{x=x_i}^{L} \left[t_b + \left(\frac{L}{a}\right)^{\frac{1}{b}} - \left(\frac{x}{a}\right)^{\frac{1}{b}}\right]^B dx \tag{3-23}$$

Solving for this integral by any of the available methods enables final evaluation of the irrigation performance.

The total measured volume of water, V, may be used to adjust the calculated V for a given set of A, B, t_b, L, a, b as:

$$V = \int_{x=0}^{L} A \left[t_b + \left(\frac{L}{a}\right)^{\frac{1}{b}} - \left(\frac{x}{a}\right)^{\frac{1}{b}}\right]^B dx \qquad (3\text{-}24$$

In most cases the measured volume of water is a reliable measurement in comparison to other measurements, so the constant, A, is adjusted by letting:

$$A = \frac{V}{\int_{x=0}^{L} \left[t_b + \left(\frac{L}{a}\right)^{\frac{1}{b}} - \left(\frac{x}{a}\right)^{\frac{1}{b}} \right]^{B}} \tag{3-25}$$

In some cases t_b may be adjusted so that Eq. 3-24 is satisfied.

d. Summary.

Use of the y_x model (Eq. 3-7) to evaluate basin irrigation includes the following steps for cases where recession is neglected.

(i) Determine the infiltration equation constants.

(ii) Determine the time required for the infiltration of the minimal depth of water at the basin far end, t_b, for y_{min}.

(iii) Determine the advance function in terms of time and distance (Eqs. 3-18, 3-19) for a given set of q, S_o, A, B, n. This can be done by direct measurements in field tests or by one of the appropriate advance models.

(iv) Determine the cover time, t_{aL} (Eq. 3-20).

(v) With the known values A, B, t_b, t_{aL} and $t_a(x)$, establish the water distribution profile (Eq. 3-22).

(vi) From the water distribution profile, calculate the total volume of water per unit of width (Eq. 3-24). Compare the calculated volume with the measured volume and adjust the value of A if necessary (Eq. 3-25).

(vii) From the water distribution profile with the adjusted A, derive the irrigation performance efficiencies (Eqs. 3-8 to 3-16) and other coefficients if needed. The irrigation performance is evaluated and the need for improvements is considered.

(viii) If improvement of the irrigation performance is needed, change the appropriate system parameters (L, q, S_o, n, t_b).

A graphical and schematic representation is given in Fig. 3-3, in which the relationship among the water distribution parameters are shown. These relationships may be obtained by use of the general model as given in Eq. 3-22.

4. Simplified Basin Irrigation Computational Model

a. Introduction

In many practical cases the basin is not level and the recession phase is significant and cannot be ignored. This further complicates the general model of y_x as given in Eqs. 3-7 and 3-22.

Some surface irrigation models allow for the treatment of all phases of the irrigation; namely, advance, runoff, ponding, depletion and recession. The first models were based mainly on specific laboratory and field experiments (Howe and Heermann,1970; Jobling and Turner, 1973) but their application is limited. The more general models, which are currently available, are based on the partial differential equations of continuity and momentum of flow in an open channel with porous bed. These models provide different numerical solutions for these equations (Bassett and Fitzsimmons, 1976; Katopodes and Strelkoff, 1977a, 1977b; Kincaid, et al., 1972; and Strelkoff and Katopodes, 1977). Although able to solve a wide range of possible cases, these models exhibit two weaknesses:

(i) Extensive programming and computer time is required in the application of these models.

(ii) The accuracy of the results is limited by the input data (infiltration, hydraulic cross section, roughness coefficients, etc.) which generally exhibit uncertainty much beyond the apparent accuracy of the mathematical accuracy of the models.

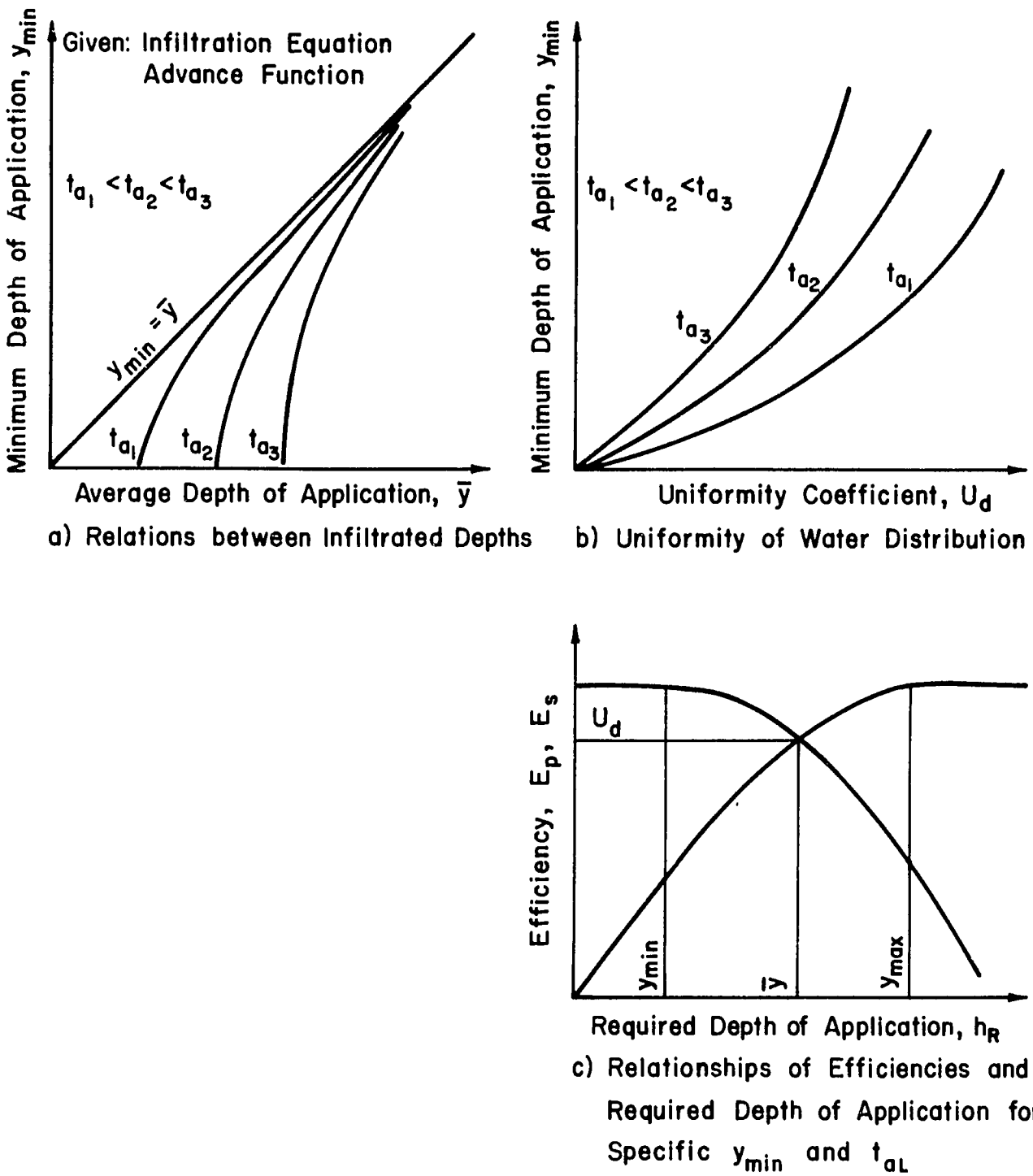


Figure 3-3. Relationship of water distribution parameters as derived from the general y_x basin irrigation model.

An algebraic model (Strelkoff, 1977) based on mass conservation can handle the various phases of border irrigation and cover a wide range of possible cases related to the stream size, slope, and end dike. With some modifications, associated mainly with the shape of the infiltrated water distribution and recession, the mass conservation concept can be used to develop a model for basin irrigation. This model would enable one to calculate the relationships between the water distribution and the basin slope, size, and inlet discharge. Consequently, efficiencies of water application and distribution uniformity coefficients could be calculated and the need for improvement could be defined.

b. Basin irrigation phases and water distribution.

In a regular basin irrigation system the irrigation process is composed of four phases:

(i) Advance of water front. This phase starts with the beginning of irrigation and is terminated at a time, t_{aL}, when the water arrives at the far end of the basin. With regular basins it is assumed that the water front is well distributed laterally across the basin. A schematic description of the infiltrated and surface water at the end of this stage is shown in Fig. 3-4.

(ii) Ponding. After the water has reached the far end, and the basin is completely covered with water, the inflow continues with a stream size greater than the overall infiltration capability of the basin. Thus, while a portion of the inflow water is infiltrated into the soil, the remaining inflow is accumulated over the surface. This phase is terminated with the cutoff time, t_{co}, when water is no longer allowed to flow into the basin.

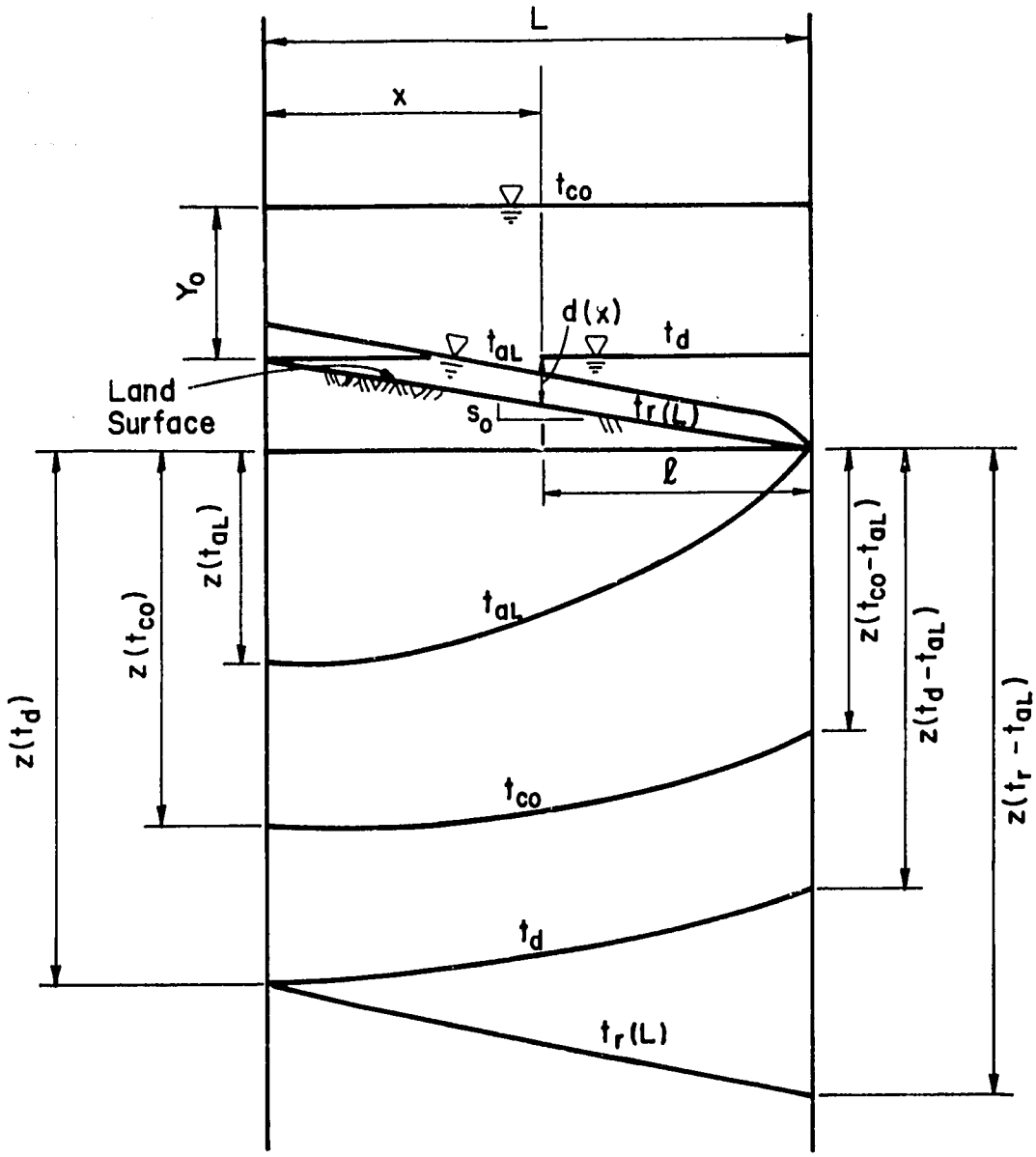


Figure 3-4. Basin irrigation phases and water profiles.

(iii) Depletion of the ponded water. After the cutoff of the inlet stream, the water depletes until the upper end of the basin is exposed, at a time t_d.

(iv) Recession of the remaining water starts at t_d, while the basin area is gradually exposed until the recession ends at time, $t_r(L)$, which is also the end of the irrigation process.

The infiltrated depths of water (Fig. 3-4) can all be calculated from the assumed infiltration equation when the opportunity times are known. To alleviate the problem of determining the opportunity time at every point in the basin, two major assumptions are made to simplify the computation of the water distribution. These assumptions involve the shape of the infiltrated water distribution and the recession equation.

c. The shape of the infiltrated water distribution.

A simplified form of the water distribution profile is assumed which allows for simple, reliable and accurate calculations based on the infiltrated distribution. For the phases of advance, ponding, and depletion, or for any time before recession starts, the infiltrated water distribution profile is assumed to be a two linear sectioned (TLS) distribution, defined by three depths (Fig. 3-5): the infiltrated depth at the lower basin end, $y_L(t)$, the infiltrated depth at the basin upper end, $y_o(t)$, and the average infiltrated depth, $\bar{y}(t)$, which occurs at a distance KL from the upper end and given by:

$$\bar{y}\ (t) = y_L(t) + K\Delta \tag{3-26}$$

where

$$\Delta = y_o(t) - y_L(t) \tag{3-27}$$

and K is a factor in the range $0.5 \leq K \leq 1.0$.

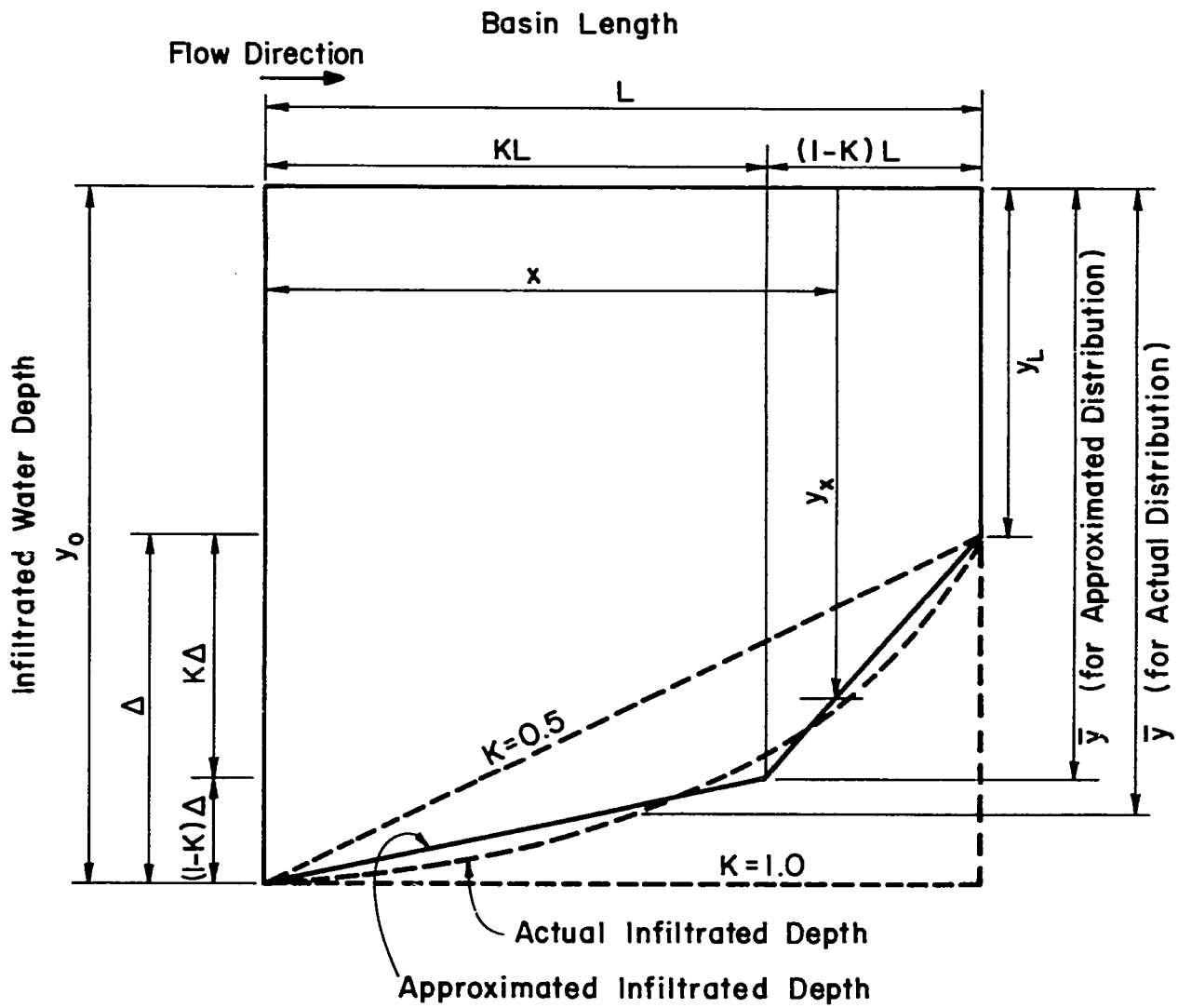


Figure 3-5. Approximated infiltrated water profile for two linear sectioned (TLS) distribution.

This shape of the infiltrated water distribution has the following characteristics:

(i) The two infiltrated water depths at the upstream and downstream ends of the basin are common to the actual distribution and to the assumed distribution.

(ii) With a known value of K, the distribution is completely defined by $y_o(t)$ and $y_L(t)$, the two infiltrated depths, which are easily obtained from calculations. In addition to the infiltration equation, the advance time, t_{aL}, is the only parameter needed to relate $y_L(t)$ to $y_o(t)$ so that the detailed advance function is not needed.

(iii) With the proper selection of K, as described in Appendix 3-A, the TLS distribution is reasonably accurate and provides acceptable agreement with the actual distribution as calculated by more complicated methods.

(iv) The value of K is in the range $0.5 \leq K \leq 1.0$. For $K = 0.5$, the distribution is a single line. For $K = 1.0$, $y_L(t) = y_o(t)$ and the infiltration depth is completely uniform (Fig. 3-5). Procedures for the calculation of K are given in Appendix 3-A.

With the TLS distribution, the infiltrated water depth at any point at a distance x within a basin of length, L, is given by:

$$y_x(t) = y_L(t) + (L-x)\,\frac{K\Delta}{(1-K)L} \qquad \text{for } KL \leq x \leq L \tag{3-28}$$

$$y_x(t) = y_L(t) + \Delta - x\,(1-K)\,\frac{\Delta}{KL} \qquad \text{for } 0 \leq x \leq KL \tag{3-29}$$

where Δ is given by Eq. 3-27.

The volume of infiltrated water, per unit of width, is:

$$V(t) = \bar{y}(t)L = [y_L(t) + K\{y_o(t) - y_L(t)\}]L \tag{3-30}$$

At the end of irrigation, the deficiency of water in relation to the average depth of application is given by:

$$h_C = \frac{K}{2} \Delta (1-K) \tag{3-31}$$

The deficiency of water, h_B, in relation to the required depth of application h_R, is dependent on the value of h_R. The general equation for h_B is given by:

$$h_B = \frac{(h_R - y_L)^2 \quad (1 - K)}{2\Delta K} \qquad y_L \leq h_R \leq \bar{y}$$

$$h_B = \frac{\Delta K(1-K)}{2} + (h_R - \bar{y}) \left[1-K + \frac{K(h_R-\bar{y})}{2\Delta(1-K)}\right] \qquad \bar{y} \leq h_R \leq y_L \tag{3-32}$$

$$h_B = \Delta(1-K) + h_R - y_o \qquad h_R \leq y_0$$

The irrigation performance parameters, U_d, E_P, and E_S can be calculated from Eqs. 5-11, 5-9, and 5-8 of Part I, Chapter 5, respectively, using the values from Eqs. 3-30, 3-31, and 3-32.

Example

A level basin of 50 m length is irrigated. The infiltration equation is $z = 15t^{0.5}$ (z in mm, t in min). The advance function is: $x = 5.86\ [t_a(x)]^{0.63}$ (x in m, t in min). The advance time to the basin end is 30 minutes. The infiltrated depth at the basin end is 80 mm during 28.4 min and the infiltrated depth at the basin upper end is 115 mm during 58.4 min. The water distribution is calculated from the opportunity time, and also by the approximated TLS distribution as shown in Fig. 3-6, for three values of K, using the three suggested procedures for the selection of K (see Appendix 3-A).

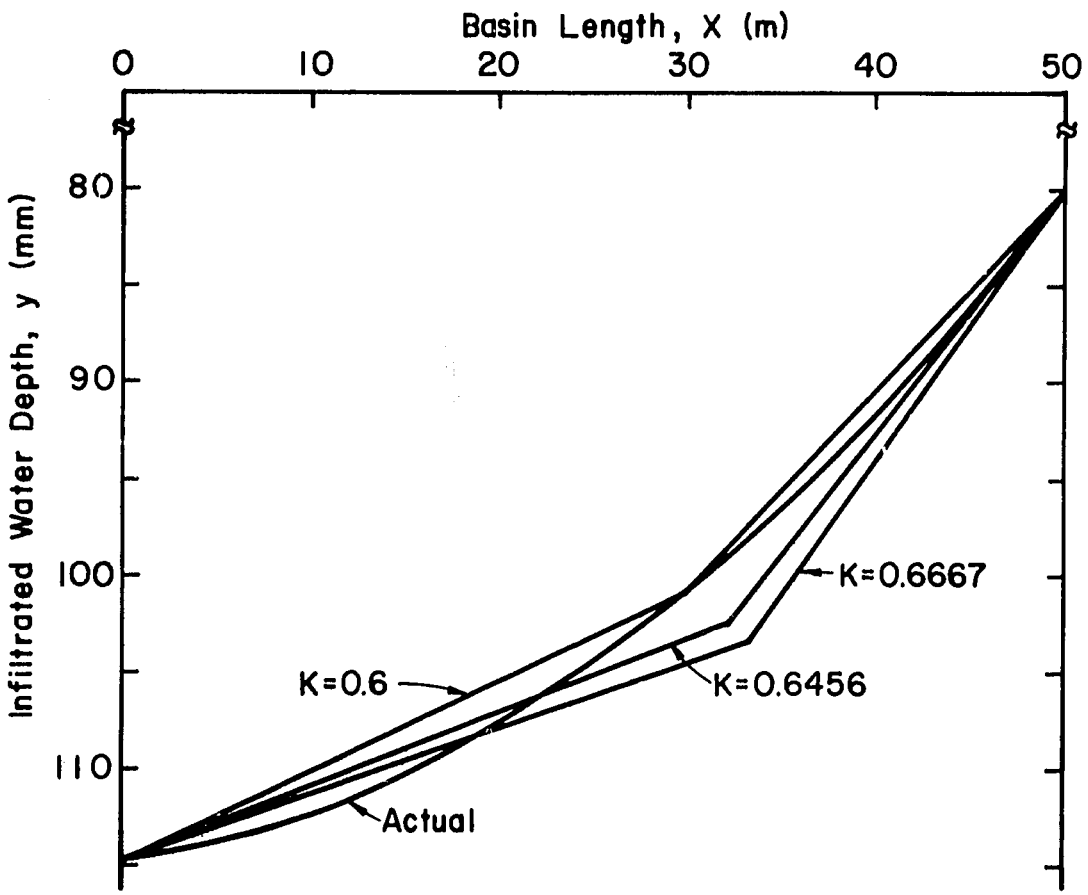


Figure 3-6. Example of actual and approximated water distribution profile.

The three TLS distributions are in satisfactory agreement with the actual distribution with maximum differences in infiltrated depths of 2.3% for K = 0.6 (Eq. 3-78), 4.5% for K = 0.667 (Eq. 3-72), and 3.23% for K = 0.645 (Eq. 3-68).

d. Simplified recession function.

The recession begins when the basin is fully ponded with zero water depth over the surface at the upper basin end. When recession begins, the water depth over the basin surface, at any distance, x, is given by:

$$d(x) = S_o x \tag{3-33}$$

where

$d(x)$ = depth of water over the basin surface at point x.

A simplified recession function can be obtained by assuming that from the beginning of recession to the end of the irrigation, the infiltrated depth at any point is d(x). If the water disappeared at point x after a time, $t_r(x)$, the infiltrated depth during the recession which is also the stored depth, d(x), can be defined by:

$$d(x) = z[t_r(x) - t_a(x)] - z[t_d - t_a(x)] \tag{3-34}$$

Using the infiltration equation (Eq. 3-4) results in:

$$d(x) = A[t_r(x) - t_a(x)]^B = A[t_d - t_a(x)]^B \tag{3-35}$$

Substituting d(x) from Eq. 3-3 and solving for x and $t_r(x)$ gives:

$$x(t_r) = \frac{A[t_r(x) - t_a(x)]^B - A[t_d - t_a(x)]^B}{S_o} \tag{3-36}$$

or

$$t_r(x) = \frac{\{xS_o + A[t_d - t_a(x)]^B\}^{\frac{1}{B}}}{A} + t_a(x) \tag{3-37}$$

Fig. 3-7 shows an example of the recession curve calculated from Eq. 3-37 compared to a recession curve calculated by Bassett (1972) and by Strelkoff (1977). The maximum difference in recession time is less than 4% and the maximum difference in infiltrated depth is 2%. Appendix 3-B contains a comparison of the recession equation of Strelkoff (1977) and Eq. 3-37.

As will be shown later, using this simplified recession curve also simplifies the calculation of the final water distribution profile, since the stored water depth, d(x), when recession begins, can be directly added to the infiltrated water depth, $y_x(t_a)$, to give the final water distribution.

e. Computation of water distribution profile under basin irrigation.

With the two approximations, namely that the shape of the water distribution can be represented by two linear sections, and that the recession is given by Eqs. 3-36 and 3-37, the water distribution under basin irrigation can be computed when the infiltration equation, inlet stream size, basin slope and cutoff time are known.

(i) Advance time. Only the cover time, t_{aL}, need be known as a detailed advance function is not needed. Any of the advance functions can be used to compute the advance time. However, with this simplified model, the volume balance is considered most suitable:

$$L[r_y d_o + r_z y_o(t_{aL})] = qt_{aL} \tag{3-38}$$

where

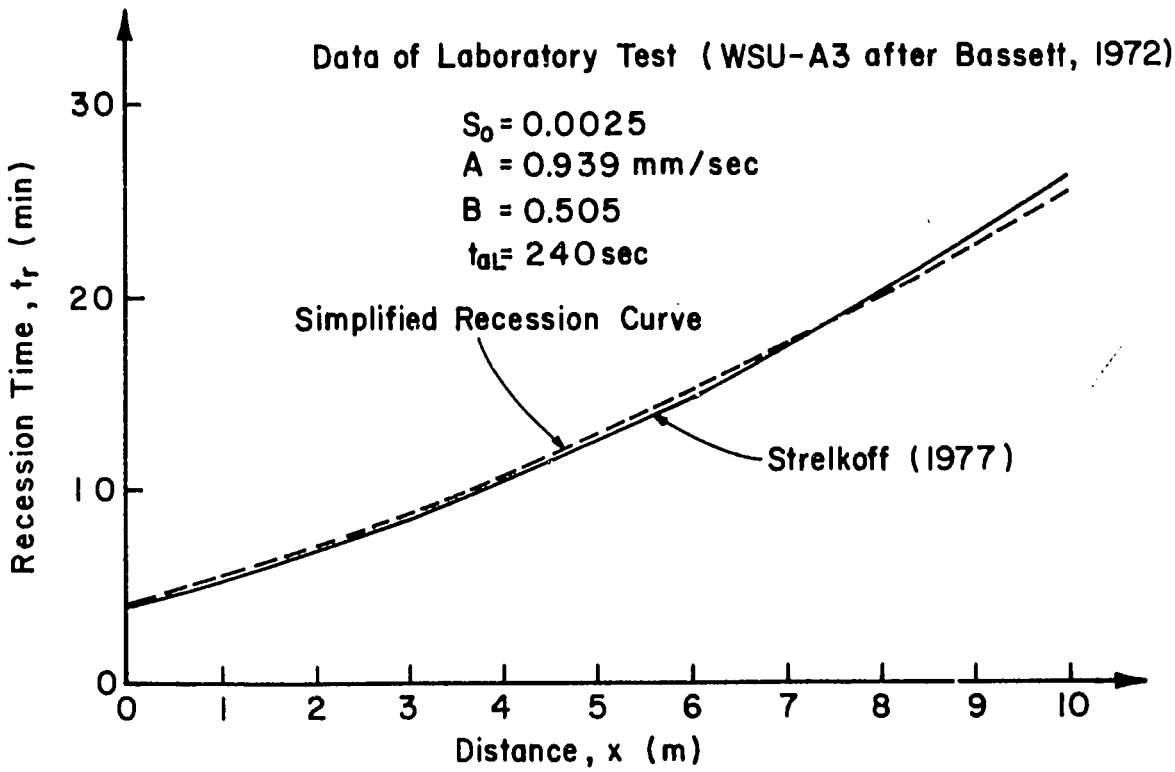


Figure 3-7. Comparison of recession curves.

q = inlet unit discharge per unit width of basin ($L^3T^{-1}L^{-1}$)

d_o = upstream surface water depth, assumed to be the normal depth of flow for the inlet discharge

r_y, r_z = shape factors for surface and infiltrated depths, respectively.

The upstream normal depth, d_o, is calculated by the Manning equation for flow in a wide channel:

$$d_o = \left(\frac{qn}{CU\ S_o^{1/2}} \right)^{3/5} \tag{3-39}$$

where n is Manning's roughness coefficient and CU is a coefficient depending on the system of units (CU = 1 for m-s units, 1.49 for ft-sec units).

The upstream infiltrated depth, $y_o(t_{aL})$, can be derived from Eq. 3-4. The surface shape factor is determined on an experimental basis and most commonly it is taken as $r_y = 0.8$. The infiltrated shape factor can be obtained from Fig. 3-15 (Appendix 3-A).

For known unit inflow stream size, q, basin length, L, slope, S_o, roughness coefficient, n, the advance time, t_{aL}, can be determined from Eq. 3-38. Direct solution of t_{aL} may be difficult, thus L as a function of t_{aL} is calculated for various t_{aL} values and t_{aL} is found in this manner.

(ii) Infiltrated water distribution at the end of depletion.

It is assumed that with basin irrigation the inlet stream size and the cut-off time are large enough to cause complete ponding. That is:

$$qt_{co} \geq V(t_{co}) + \frac{S_o L^2}{2} \tag{3-40}$$

As a consequence of this, there will be a depletion stage.

The volume balance equation at the end of depletion (Fig. 3-4) is:

$$qt_{co} = \frac{S_o L^2}{2} + V\ (t_d) \tag{3-41}$$

where $V\ (t_d)$ is the volume of water infiltrated in the basin at time, t_d, the time depletion ends, and t_{co} is the cutoff time. With the approximated water distribution, the infiltrated volume can be expressed by:

$$V(t_d) = \{z(t_d-t_{aL}) + K[z(t_d) - z(t_d-t_{aL})]\}L \tag{3-42}$$

For t_{aL} known from the advance equation (Eq. 3-38), t_d can be calculated by substituting Eq. 3-42 into Eq. 3-41. Direct solution of t_d is not possible, thus $V(t_d)$ is calculated for various values of t_d. By using the correct t_d, the approximated water distribution (Eqs. 3-28 and 3-29) is established with the three depths required to completely define it:

$$y_L(t_d) = z(t_d-t_{aL}) = A(t_d-t_{aL})^B \tag{3-43a}$$

$$y_o(t_d) = z(t_d) = At_d^B \tag{3-43b}$$

$$y_{KL}(t_d) = y_L(t_d) + K[y_o(t_d) - y(t_d)] \tag{3-43c}$$

(iii) The final water distribution. With the recession as given by Eqs. 3-36 and 3-37, the final water distribution is obtained by adding the surface stored water depths $d(x) = S_o x$ at the end of depletion, to the infiltrated water depths at the end of depletion.

The three infiltrated water depths that define the final water distribution are as follows (Fig. 3-8):

$$y_L = z(t_d-t_{aL}) + S_o L = A(t_d-t_{aL})^B + S_o L \tag{3-44a}$$

$$y_o = z(t_d) = At_d^B \tag{3-44b}$$

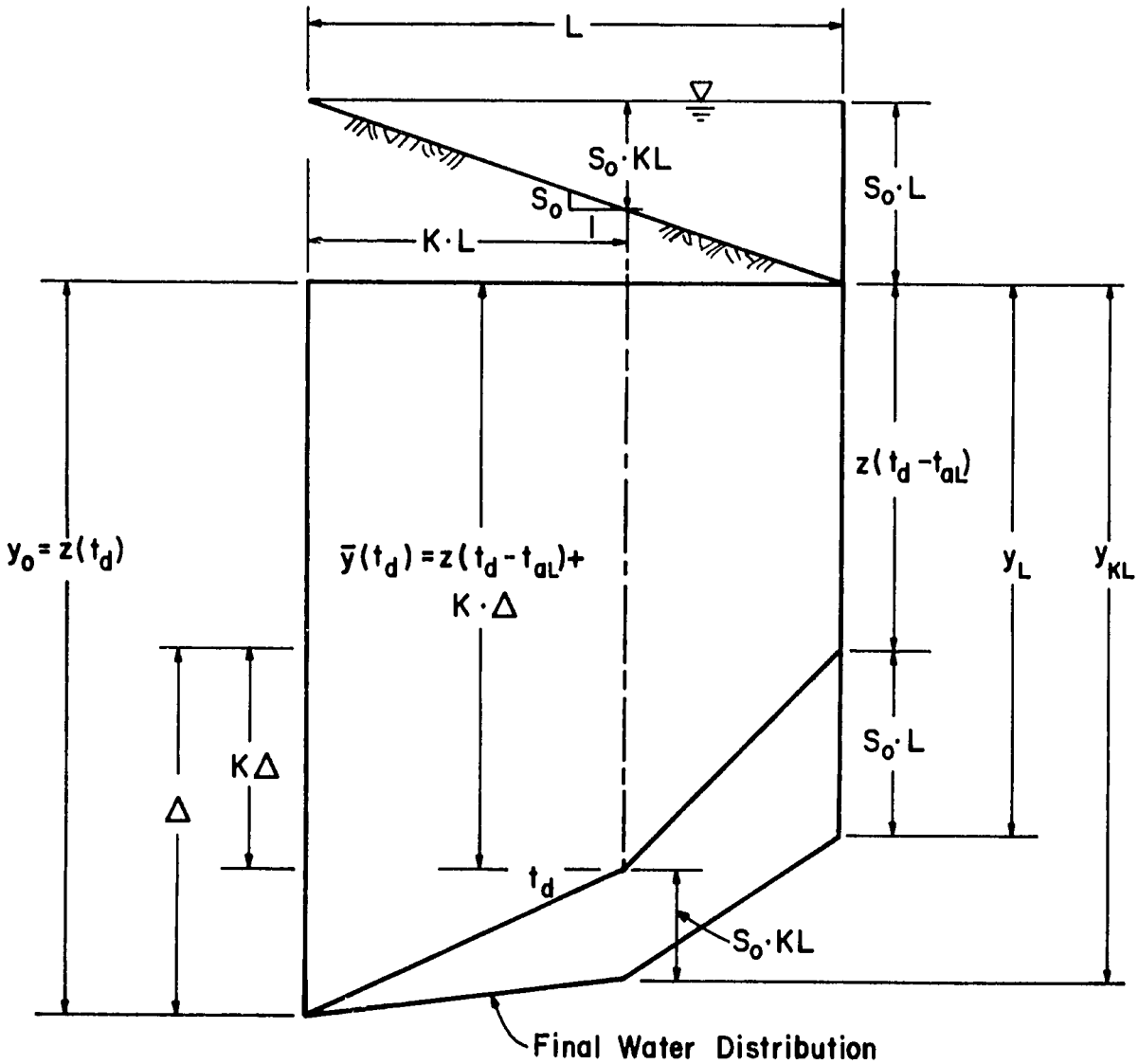


Figure 3-8. Schematic description of the final water profile for two linear sectioned (TLS) distribution.

$$y_{KL} = y_L + K(y_o - y_L) + S_o KL = A(t_d - t_{aL})^B +$$

$$K\left(At_d^B - A(t_d - t_{aL})^B\right) + K\, S_o L \tag{3-44c}$$

Note that

$$\bar{y}(t_d) = y_{KL}(t_d) \tag{3-45}$$

However, as

$$\bar{y} = \bar{y}\ (t_d) + \frac{S_o L}{2} \tag{3-46}$$

and

$$y_{KL} = y_{KL}(t_d) + K\, S_o L \tag{3-47}$$

in general

$$\bar{y} \neq y_{KL} \tag{3-48}$$

unless $S_o = 0$, or $K = 0.5$. Therefore, although the mean depth of water infiltrated at the end of depletion is represented by the infiltrated depth at $x = KL$ in the TLS distribution, the final mean depth of water infiltrated is generally not represented by the final depth at $x = KL$.

Example

Data first given by Bassett (1972) and further discussed by Strelkoff (1977) will be used, however, a longer cutoff time will be considered so that a higher average depth is applied. The data are:

q = 8.39 m^3/m/hr (2.33 x $10^{-3} m^3$/m/sec)

S_o = 0.001

L = 82.3 m

n = 0.035

A = 2.01 mm/sec^B

B = 0.33

t_{co} = 59.0 min

(i) Advance. The surface shape factor, r_y, is selected to be $r_y = 0.8$ (based on experimental data).

The infiltration water distribution shape factor, r_z, is 0.78 (Wilke and Smerdon, Fig. 3-15).

The upstream surface water depth is (Eq. 3-39):

$$d_o = \left[\frac{2.33 \times 10^{-3} \times 0.035}{(0.001)^{1/2}}\right]^{3/5} = 0.0280 \text{ m}$$

The advance equation (Eq. 3-38) is:

$$82.3 = \frac{2.33 \times 10^{-3} t_{aL}}{0.8 \times 0.0280 + 2.01 \times 10^{-3}\ t_{aL}^{0.33} \times 0.78}$$

Solving for t_{aL}: $t_{aL} = 1410$ sec = 23.5 min

(ii) Infiltrated water distribution at the end of depletion

$$V(t_d) = qt_{co} - \frac{SL^2}{2} = 2.33 \times 10^{-3} \times 59.0 - 0.001 \times \frac{82.3^2}{2} = 4.84 \text{ m}^3/\text{m}$$

Using $K = \frac{1}{1 + B} = \frac{1}{1 + .33} = 0.75$

The time that depletion ends, t_d, is calculated from Eq. 3-42.

$$4.84 = \left\{2.01(t_d-1410)^{0.33} + 0.75\left[2.01t_d^{0.33} - 2.01(t_d-1410)^{0.33}\right]\right\} 82.3 \times 10^{-3}$$

Solving for t_d: $t_d = 28{,}100$ sec = 7.80 hrs.

The infiltrated water depths at the end of depletion are (Fig. 3-9):

$$t_L(t_d) = 2.01\ (28{,}100 - 1{,}380)^{0.33} = 58.1 \text{ mm}$$

$$y_o(t_d) = 2.01 \times 28{,}100^{0.33} = 59.0 \text{ mm}$$

$$y_{KL}(t_d) = 58.1 + 0.750\ (59.0 - 58.1) = 58.8 \text{ mm}$$

Note that the infiltrated water distribution at this stage is highly uniform since the advance is relatively fast and is about 1/20 of the depletion

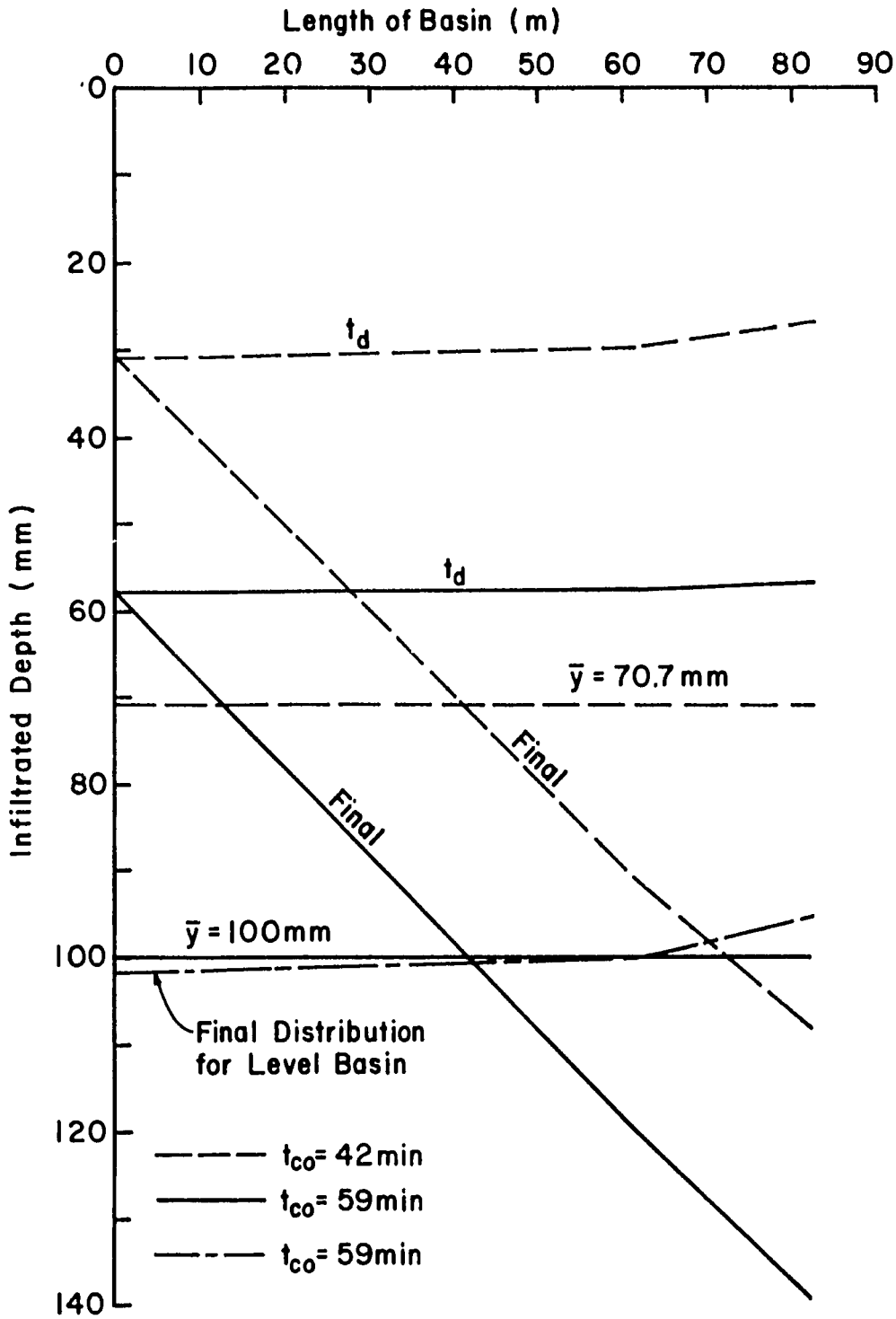


Figure 3-9. Water distribution profiles with sloped basin. (Example)

time. The effect of time differences on differences in infiltration depth is also relatively limited since the infiltrated depth is related to the infiltrated time by a power of B = 0.33.

(iii) The final water distribution (Fig. 3-9).

$$y_L = 58.1 + 0.001 \times 82.3 \times 10^{-3} = 140.3 \text{ mm}$$

$$y_{KL} = 58.8 + 0.001 \times 82.3 \times 10^{-3} \times 0.75 = 120.5 \text{ mm}$$

$$y_o = 59.0 \text{ mm}$$

The average depth applied is

$$\bar{y} = \frac{qt_{co}}{L} = \frac{8.39 \times \frac{59}{60}}{82.3} = 0.1 \text{ m} = 100 \text{ mm}$$

A similar solution for a smaller quantity of water, obtained with a cutoff time of 42 minutes, is also given in Fig. 3-9, for which the end of depletion, t_d = 3670 sec, the infiltrated water depths at the end of depletion are $y_L(t_d)$ = 26.2 mm, $y_{KL}(t_d)$ = 29.6 and $y_o(t_d)$ = 30.7 mm. The final water depths are y_L = 108.5 mm, y_{KL} = 91.3 and y_o = 30.7 mm. The average depth applied is $\bar{y}$ = 70.7 mm.

In both cases, the uniformity of water distribution is poor because of the slope of the basin. Generally speaking, it is recommended that the elevation difference between the basin ends, $Z = S_oL$, be such that

$$Z \leq 2\Delta \tag{3-49}$$

where Δ is the difference between the upstream and downstream infiltrated depths at the end of depletion.

f. Further simplification in calculating the final water distribution.

In the establishment of the final water distribution, the infiltrated water distribution at the end of the depletion phase must be calculated and the solution of t_d and the infiltrated water depths is indirect.

A further simplification will allow for direct calculation of the final water depth. This simplification is the assumption that at the cutoff of the inlet stream, the water ponded at any point in the basin infiltrates at that point. At the cutoff time, the volume balance equation is:

$$q_o t_{co} = V(t_{co}) + \frac{S_o L^2}{2} + Y_o L \tag{3-50}$$

where $V(t_{co})$ is the infiltrated volume at time, t_{co}, and Y_o is the surface water depth at the basin upper end (Fig. 3-4).

For given t_{co}, the infiltrated depth is directly calculated (see Eq. 3-42 replacing t_d by t_{co}) so that Y_o is readily available from Eq. 3-50.

With the calculated Y_o, the three water depths that completely define the final water distribution are:

$$y_L = z(t_{co} - t_{aL}) + S_o \cdot L + Y_o \tag{3-51}$$

$$y_o = z(t_{co}) + Y_o \tag{3-52}$$

$$y_{KL} = z(t_{co} - t_{aL}) + K[z(t_{co}) - z(t_{co} - t_{aL})] + K \cdot S_o \cdot L + Y_o \tag{3-53}$$

If the difference between y_L as calculated by Eq. 3-44a and that calculated by Eq. 3-51 is denoted by DEV_L, and the deviation between y_o as calculated by Eq. 3-44b and that calculated by Eq. 3-52 is denoted by DEV_o, these deviations are expressed by:

$$DEV_L = K[\Delta(t_{co}) - \Delta(t_d)] \tag{3-54}$$

$$DEV_o = (1 - K)[\Delta(t_d) - \Delta(t_{co})] \tag{3-55}$$

where $\Delta(t_{co})$ and $\Delta(t_d)$ are the differences between the infiltrated upstream and downstream depths (Fig. 3-5) at time t_{co} and t_d, respectively. Since $\Delta(t_{co}) > \Delta(t_d)$, DEV_o is always negative. In most practical cases, DEV_L and DEV_o are relatively small, a few millimeters only, so that the accuracy of this assumption is expected to be good. As an example, the value of Δ

is given as a function of time for the example studied by Bassett (1972) and Strelkoff (1977) (Fig. 3-10). For this example, in the case of t_{co} = 59 min and t_d = 442 min, the deviations in y_L and y_o are:

$$DEV_L = 0.75\ (4.7 - 1.0) = 2.8 \text{ mm}$$

$$DEV_o = (1 - 0.75)(1.0 - 4.7) = -0.9 \text{ mm}$$

In the case of t_{co} = 42 min and t_d = 61.2 min, the deviations are:

$$DEV_L = 0.75\ (6.2 - 4.4) = 1.4 \text{ mm}$$

$$DEV_o = 0.25\ (4.4 - 6.2) = -0.5 \text{ mm}$$

In both cases the deviations are negligible and the assumption that the ponded water at the cutoff time is the depth that is further infiltrated at each point by the end of recession, is acceptable.

g. Application of the simplified model with level basins.

Higher irrigation efficiencies can be obtained with level basins, eliminating the unfavorable effect of high ponding water depth at the downstream end of sloped basins.

The simplified model can be easily applied for this case, but the calculation of advance time, t_{aL}, requires some modifications. Since the normal depth, d_o, (Eq. 3-39) cannot be calculated for $S_o = 0$, it can be calculated as the depth at the head end of a wide channel with $S_o = 0$, in which the flow is gradually varied. However, the calculation of d_o can be simplified with the assumption that the hydraulic gradient is approximately the ratio of the surface water depth to the basin length as suggested by the Soil Conservation Service (1974).

In this case the slope is approximated by

$$S_o \simeq \frac{d_o}{L} \tag{3-56}$$

Substituting Eq. 3-58 into Eq. 3-39 and solving for d_o:

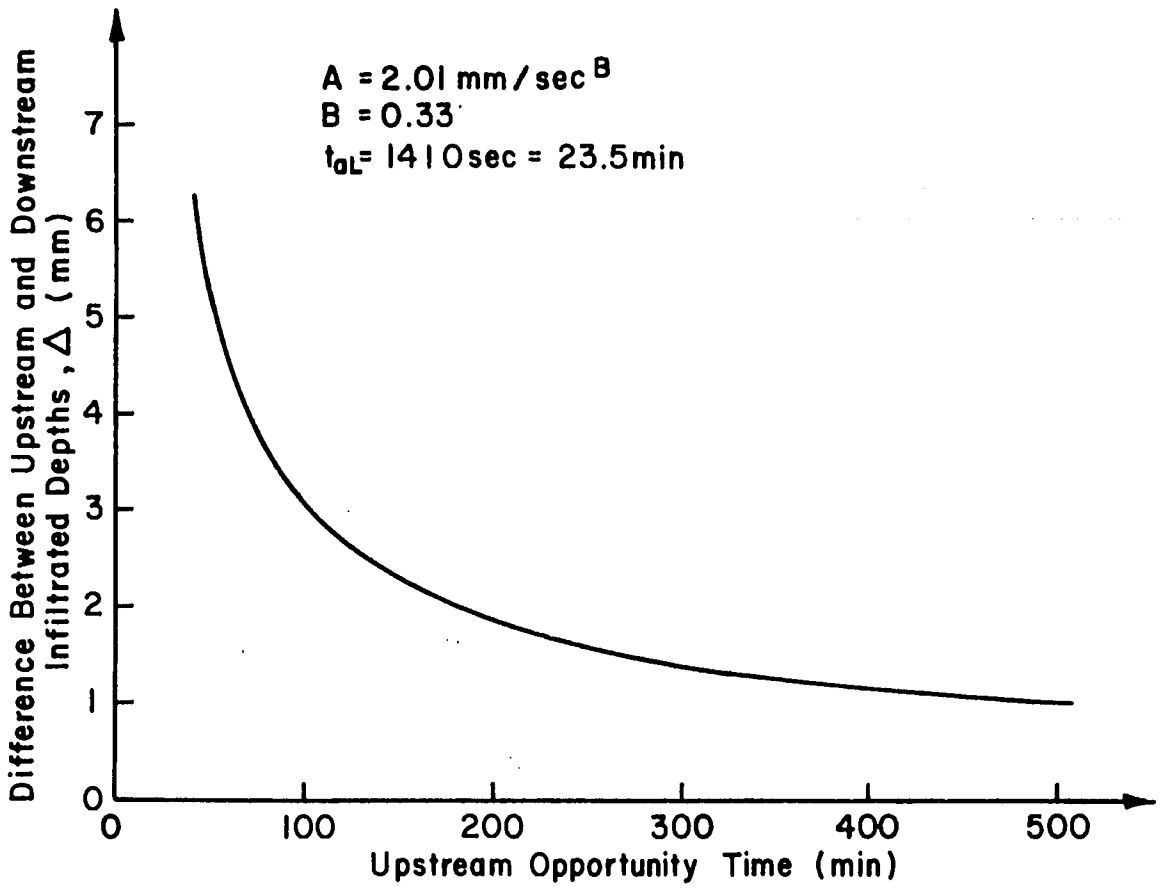


Figure 3-10. Difference between upstream and downstream infiltrated depths as function of upstream opportunity time.

$$d_o = \left(\frac{q\ nL^{1/2}}{CU}\right)^{6/13} \tag{3-57}$$

Assuming that the surface water depth varies linearly from a depth of d_o at the upper basin end to a depth of $0.7d_o$ at a distance L and then sharply drops to 0, the hydraulic gradient can be estimated by:

$$S_o = \frac{0.3d_o}{L} \tag{3-58}$$

Substituting Eq. 3-58 into Eq. 3-39 and solving for d_o:

$$d_o = 1.32\left(\frac{q\ nL^{1/2}}{CU}\right)^{6/13} \tag{3-59}$$

Based on experimental results, as well as functions that describe the surface water profile, Eq. 3-59, which gives a larger depth of water than Eq. 3-57, seems to be more reasonable. However, the difference between the two depths as calculated by Eqs. 3-57 and 3-59 has only limited effect on the advance equation as given by Eq. 3-38.

The advance time, t_{aL}, is then solved by substituting Eq. 3-57 or Eq. 3-59 into Eq. 3-38. However, indirect solution of t_{aL} is needed for known values of L.

For given unit inlet stream size and cutoff time, the surface depth of water at cutoff time, Y_o, which is uniform along the basin, is given by:

$$Y_o = \frac{qt_{co}}{L} - \frac{V(t_{co})}{L} \tag{3-60}$$

The infiltrated water quantity at cutoff time, t_{co}, is given by:

$$V(t_{co}) = \left(z(t_{co} - t_{aL}) + K\left[z(t_{co}) - z(t_{co} - t_{aL})\right]\right)L \tag{3-61}$$

Solving for Y_o from Eqs. 3-60 and 2-61, the three water depths that define the final water distribution are:

$$y_L = z(t_{co} - t_{aL}) + Y_o \tag{3-62}$$

$$y_o = z(t_{co}) + Y_o \tag{3-63}$$

$$y_{KL} = z(t_{co} - t_{aL}) + K[z(t_{co}) - z(t_{co} - t_{aL}] + Y_o \tag{3-64}$$

Note that:

$$y_{KL} = \frac{V}{L} + Y_o = \frac{qt_{co}}{L} = \bar{y}$$

Example

The case discussed by Strelkoff (1977) for a level basin, that is $S_o = 0$, will be used (for details see example following Section 4e).

The surface upstream depth is approximated by Eq. 3-57:

$$d_o = \left(2.33 \times 10^{-3} \times 0.035 \times 82.3^{1/2}\right)^{6/13} = 0.0359 \text{ m}$$

Solving for t_{aL} after substituting the value of d_o into Eq. 3-38 gives:

$$t_{aL} = 1650 \text{ seconds}$$

At the cutoff time, the infiltrated depths from Eqs. 3-43a, 3-43b, and 3-43c, are:

$$y_L(t_{co}) = 2.01\ (3540 - 1650)^{0.33} = 24.6 \text{ mm}$$

$$y_o(t_{co}) = 2.01 \times 3540^{0.33} = 30.3 \text{ mm}$$

$$y_{KL}(t_{co}) = 24.6 + 0.75\ (30.3 - 24.6) = 28.8 \text{ mm}$$

The infiltrated volume of water at the time of cutoff:

$$V(t_{co}) = 0.0289 \times 82.3 = 2.37 \text{ m}^3/\text{m}$$

Solving for Y_o from Eq. 3-60:

$$Y_o = \frac{8.39 \cdot \frac{59}{60} - 2.37}{82.3} = 0.0711 \text{ m} = 71.1 \text{ mm}$$

The three depths that determine the final water distribution are: (Fig. 3-9):

$y_L = 24.6 + 71.1 = 95.7$ mm

$y_o = 30.3 + 71.1 = 101.4$ mm

$y_{KL} = 28.9 + 71.1 = 100.0$ mm (Note: $y_{KL} = \bar{y}$)

With the level basins, the water distribution profile is significantly more uniform, when other conditions are kept constant (q, t_{co}, L, n, A, B), despite an increase in the advance time.

Further application of the simplified model is given by a numerical example for a level basin. In this example, the time of advance, t_{aL}, and the minimal infiltrated depth at the basin far end are taken as variables. The cumulative infiltration is taken as

$$z = 15\ t_{op}^{0.5} \qquad (z = \text{mm},\ t_{op} = \text{min})$$

The shape factor is estimated as K = 0.6. For these data the relationships among t_{aL}, y_L, y_o and Δ are calculated for various selected t_{aL} and y_L values and U_d is also calculated. Results are given in Table 3-1 and Figs. 3-11, 3-12 and 3-13.

The relationships of t_{aL}, y_L, $\bar{y}$, Δ, and U_d as obtained by the simplified model for a given A, B and K, and as presented in Table 3-1 and Figs. 3-11, 3-12 and 3-13 enable estimation of the water profile and the uniformity distribution coefficients. Efficiencies, E_p and E_s, can be calculated for a given h_R. A typical case in which $h_R = y_L$ is used to give the values of E_p (in this case $E_s = 1.0$) in Table 3-1. If, for example, for the given basin the average depth was found to be 100 mm and the cover time was 30 minutes, the estimated minimal depth is $y_L = 79$ mm; the

Table 3-1: Relationships between water distribution parameters as obtained by the simplified model ($K = 0.6$, $z = 15t^{0.3}$).

	y_L mm	$t(y_L)$ (min)	$t_a+t(y_L)$ (min)	y_o (mm)	$\Delta=y_o-y_L$ (mm)	h_c (mm)	$\bar{y}=y_L+0.6\Delta$	u_d	$E_p=\frac{y_L}{\bar{y}}$ (for $y_L=h_R$)
$t_a=5$ min	0	0	5	33.5	33.5	4.0	20.1	0.800	0
	20	1.8	6.8	39.0	19.0	2.3	31.4	0.927	0.636
	50	11.1	16.1	60.2	10.2	1.2	56.1	0.978	0.871
	60	16.0	21.0	68.7	8.7	1.0	65.2	0.984	0.920
	70	21.8	26.8	77.6	7.6	0.9	74.6	0.988	0.939
	80	28.4	33.4	86.7	6.7	0.8	84.0	0.990	0.952
	90	36.0	41.0	96.0	6.0	0.7	93.6	0.992	0.961
	100	44.4	49.4	105.5	5.5	0.6	103.3	0.993	0.968
$t_a=10$ min	0	0	10	47.4	47.4	5.1	28.5	0.800	0
	20	1.8	11.8	57.5	31.5	3.8	38.9	0.900	0.514
	50	11.1	21.1	68.9	18.9	2.3	61.4	0.963	0.815
	60	16.0	26.0	76.5	16.5	2.0	69.9	0.972	0.858
	70	21.8	31.8	84.6	14.6	1.8	78.7	0.9777	0.889
	80	28.4	38.4	93.0	13.0	1.6	87.8	0.982	0.911
	90	36.0	46.0	101.7	11.7	1.4	97.0	0.985	0.927
	100	44.4	54.4	110.7	10.7	1.3	106.4	0.988	0.940
$t_a=20$ min	0	0	0.	67.1	67.1	8.0	40.2	0.800	0
	20	1.8	21.8	70.0	50.0	6.0	50.0	0.880	0.400
	50	11.1	31.1	83.7	33.7	4.0	70.2	0.942	0.712
	60	16.0	36.0	90.0	30.0	3.6	78.0	0.954	0.769
	70	21.8	41.8	96.9	26.9	3.2	86.2	0.962	0.812
	80	28.4	48.4	104.4	24.4	2.9	94.6	0.969	0.845
	90	36.0	56.0	112.2	22.2	2.7	103.4	0.974	0.871
	100	44.4	64.4	120.4	20.4	2.4	112.2	0.978	0.891
$t_a=30$ min	0	0	30.0	82.2	82.2	9.9	49.3	0.800	0
	20	1.8	31.8	84.6	64.6	7.8	58.7	0.868	0.340
	50	11.1	41.1	96.2	46.2	5.5	77.7	0.929	0.643
	60	16.0	46.0	101.7	41.7	5.0	85.0	0.941	0.705
	70	21.8	51.8	107.9	37.9	4.6	92.8	0.954	0.755
	80	28.4	58.4	114.7	34.7	4.2	100.8	0.959	0.794
	90	36.0	66.0	121.9	31.9	3.8	109.1	0.965	0.825
	100	44.4	74.4	129.4	29.4	3.5	117.6	0.970	0.850
$t_a=40$ min	0	0	40	94.9	94.9	11.4	56.9	0.800	0
	20	1.8	41.8	96.9	76.7	9.6	66.2	0.855	0.302
	50	11.1	51.1	107.2	57.2	6.9	84.3	0.918	0.593
	60	16.0	56.0	112.2	52.2	6.3	91.4	0.931	0.657
	70	21.8	61.8	117.9	47.9	5.7	98.7	0.942	0.709
	80	28.4	68.4	124.1	44.1	5.3	106.4	0.950	0.7
	90	36.0	76.0	130.8	40.8	4.9	114.5	0.957	0.786
	100	44.4	84.4	137.8	37.8	4.5	122.7	0.963	0.815
$t_a=50$ min	0	0	50.0	106.1	106.1	12.8	64.0	0.800	0
	20	1.8	51.8	107.9	87.9	10.6	72.8	0.855	0.275
	50	11.1	61.1	117.3	67.3	8.1	90.4	0.911	0.553
	60	16.0	66.0	121.9	61.9	7.4	97.1	0.923	0.618
	70	21.8	71.8	127.1	57.1	6.8	104.2	0.934	0.671
	80	28.4	78.4	132.8	52.8	6.3	111.7	0.943	0.716
	90	36.0	86.0	139.1	49.1	5.9	119.5	0.951	0.753
	100	44.4	94.4	145.8	45.8	5.5	127.5	0.957	0.784
$t_a=70$ min	0	0	70.0	125.5	125.5	15.0	75.3	0.800	0
	20	1.8	71.8	127.1	107.1	12.8	84.2	0.847	0.237
	50	11.1	81.1	135.1	85.1	10.2	101.0	0.899	0.495
	60	16.0	86.0	139.1	79.1	9.5	107.5	0.912	0.558
	70	21.8	91.8	143.7	73.7	8.8	114.2	0.922	0.613
	80	28.4	98.4	148.8	68.8	8.3	121.3	0.932	0.596
	90	36.0	106.0	154.4	64.4	7.7	128.7	0.940	0.699
	100	44.4	114.4	160.5	60.5	7.2	136.3	0.946	0.734

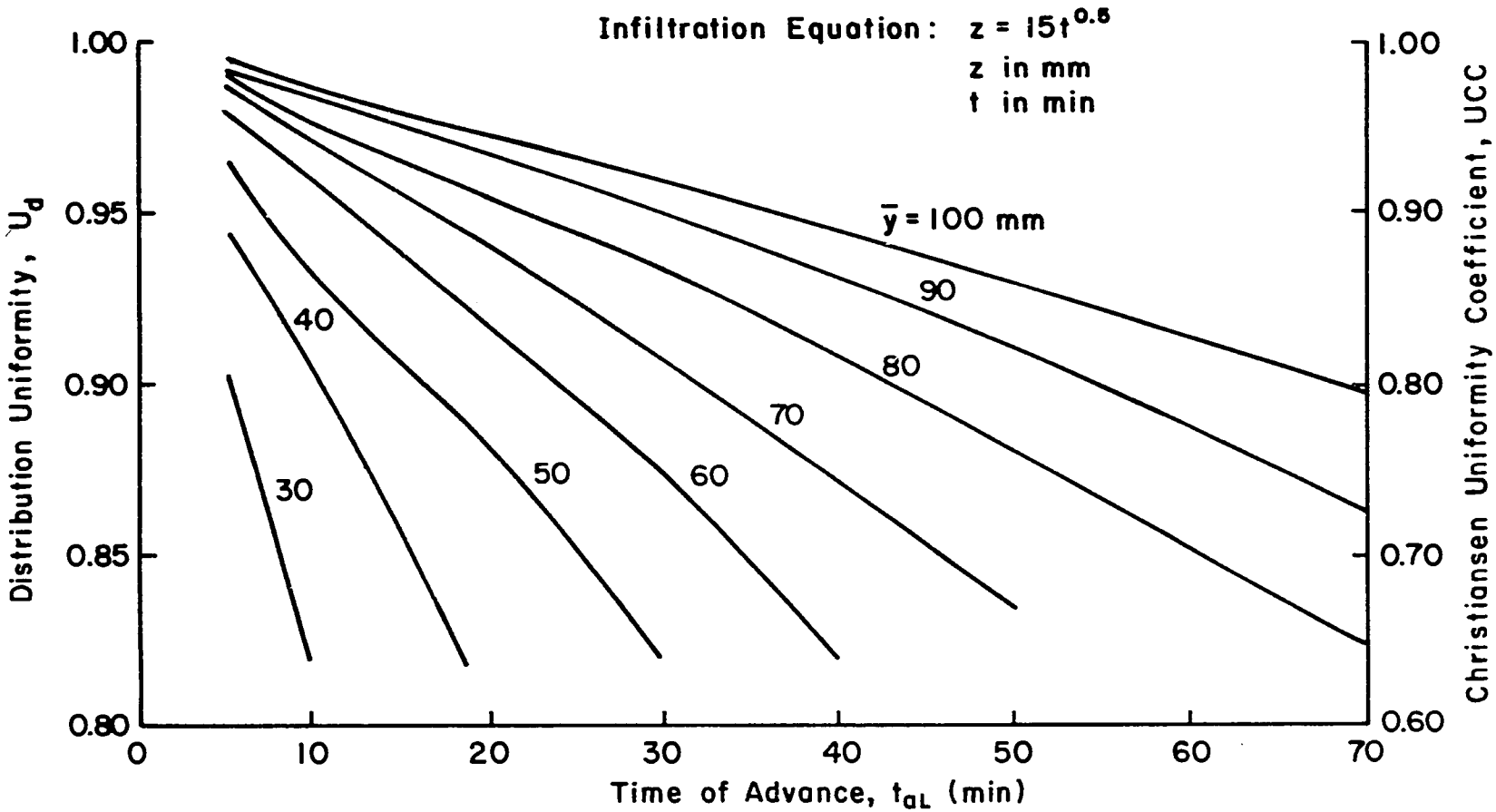


Figure 3-11. Example of distribution uniformity as function of advance time for various average infiltrated depths using simplified model for level basins.

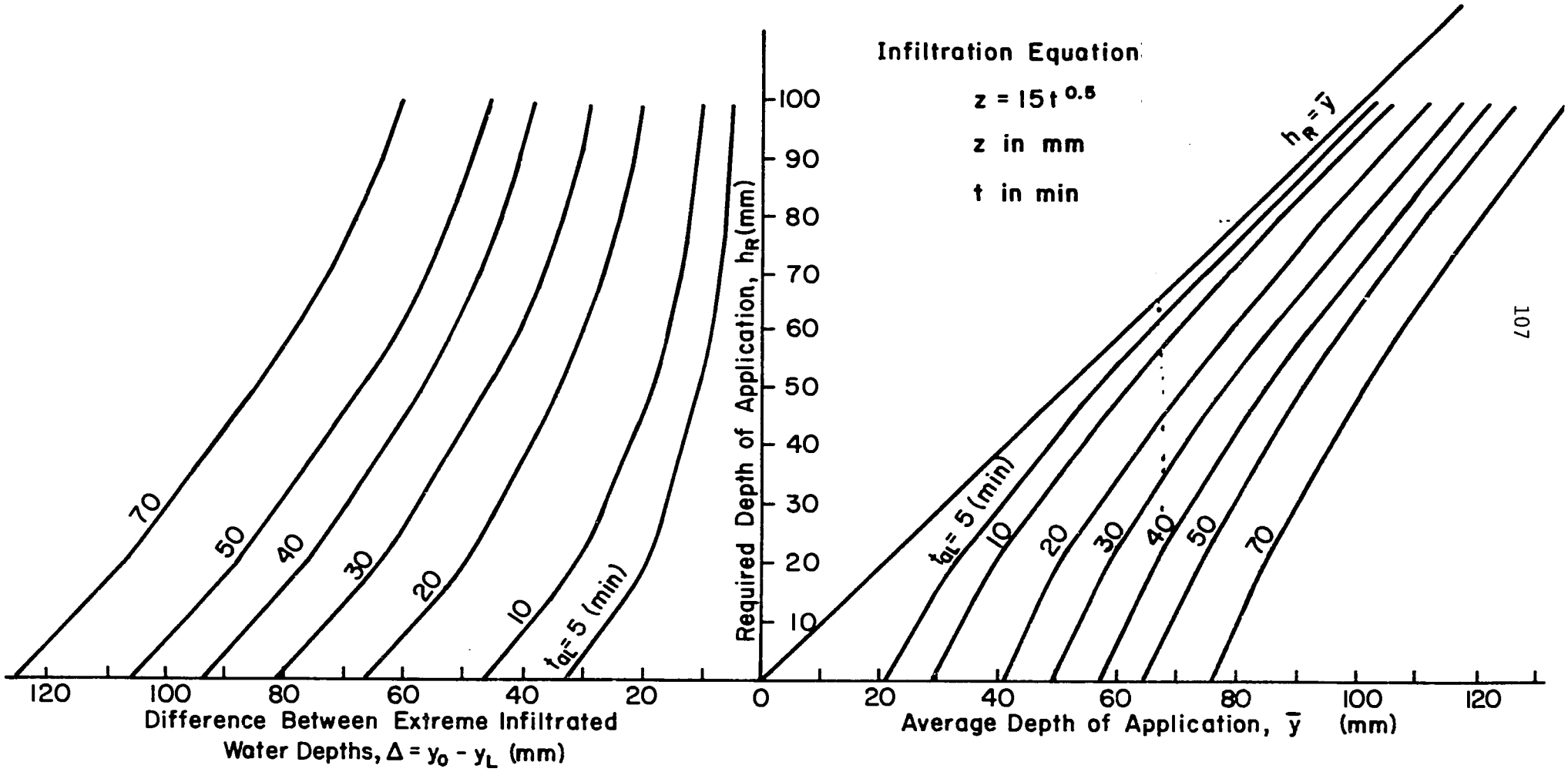


Figure 3-12. Example of relationships between infiltrated depths and advance time for level basin irrigation using simplified model.

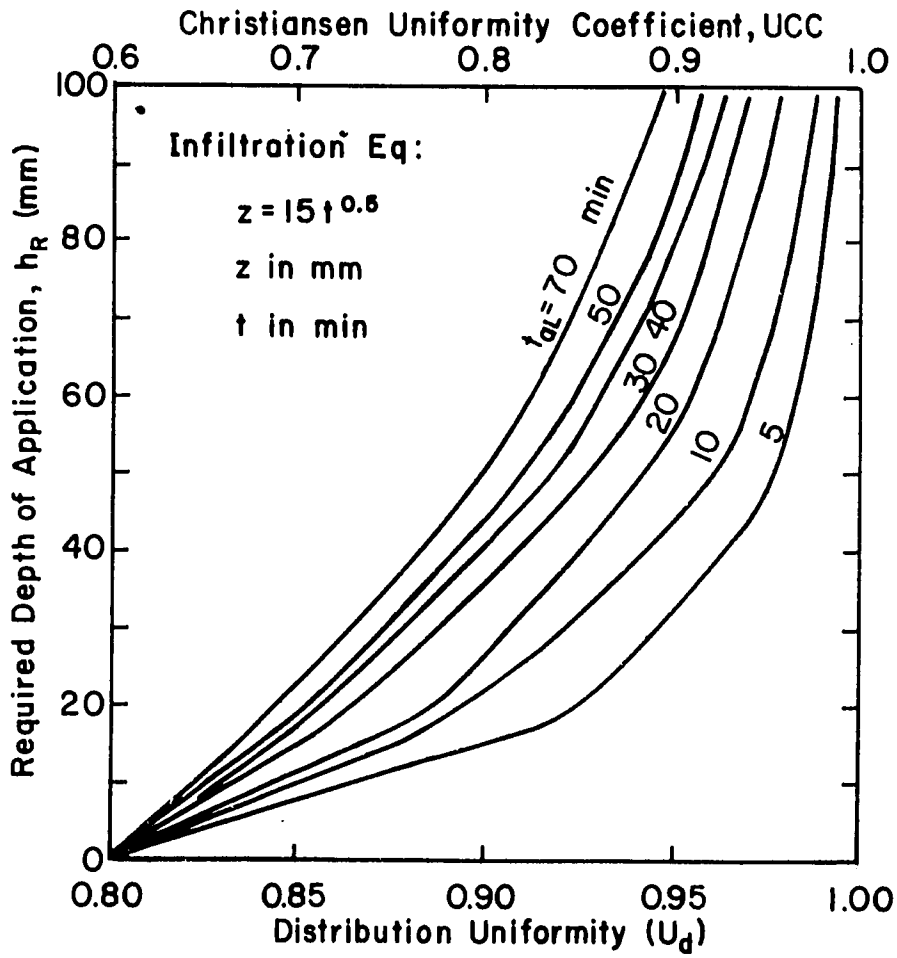


Figure 3-13. Example of relationships between required depth of application and uniformity coefficients for varying time of advance.

maximum depth is $y_{max} = 79 + 34 = 113$ mm; the distribution uniformity is $U_d = 0.958$ and the deep percolation efficiency for $h_R = y_L$ is $E_P = 0.793$.

A further application of the simplified model includes the relationship between the time of advance, t_{aL}, and the operating conditions (infiltration constants A, B, bed slope, S_o, surface roughness, n, discharge, Q, length of basin, L). By establishing these relationships among all the other parameters provided by the model (y_o, $\bar{y}$, y_L, Δ, U_d), the final water distribution can be related to the operating conditions.

Thus, a further advantage of the simplified model is that it can provide direct relationships between the main distribution parameters and the operating conditions. If a change is required, the parameters, and variables that should be changed can be determined directly, along with the required change.

5. Actual Application of the Theoretical Models

Theoretical models allow the prediction of the water distribution and irrigation performance, but some field tests are essential to provide basic data in order to check the accuracy of data used and the accuracy of the predictions. Four major field measurements are recommended, as described below.

a. Infiltration.

Field evaluation of infiltration is necessary since there are no other acceptable means to obtain the infiltration function. If t_d can be measured in the field, adjustment of the infiltration coefficient A can be done using:

$$\left[A(t_d-t_{aL})^B + K\left(At_d^{\,B} - A(t_d-t_{aL})^B\right)\right] L = qt_{co} - \frac{S_o L^2}{2} \qquad (3\text{-}66)$$

b. Advance.

It is recommended that for a given basin (soil, slope, surface roughness) the advance of the water front be measured with time. If a continuous measurement cannot be conducted, the cover time, t_{aL}, should be measured, and be used in the simplified model. Without field measurements of advancement, advance functions can be established by theoretical equations, however, these are less accurate and more complicated.

c. Total water delivered to the basin.

This is relatively easy and a reliable measurement of either inlet discharge and cutoff time, or volume applied and cutoff time should be made. With the volume, discharge and the known cutoff time, the distribution of the infiltrated water can be established.

d. Slope of the field.

The slope along the field should be determined by direct measurement to allow the calculation of recession and for the calculation of advance time by theoretical models, if necessary.

The procedure for the calculation of the water distribution and the irrigation performance parameters is given in Fig. 3-14.

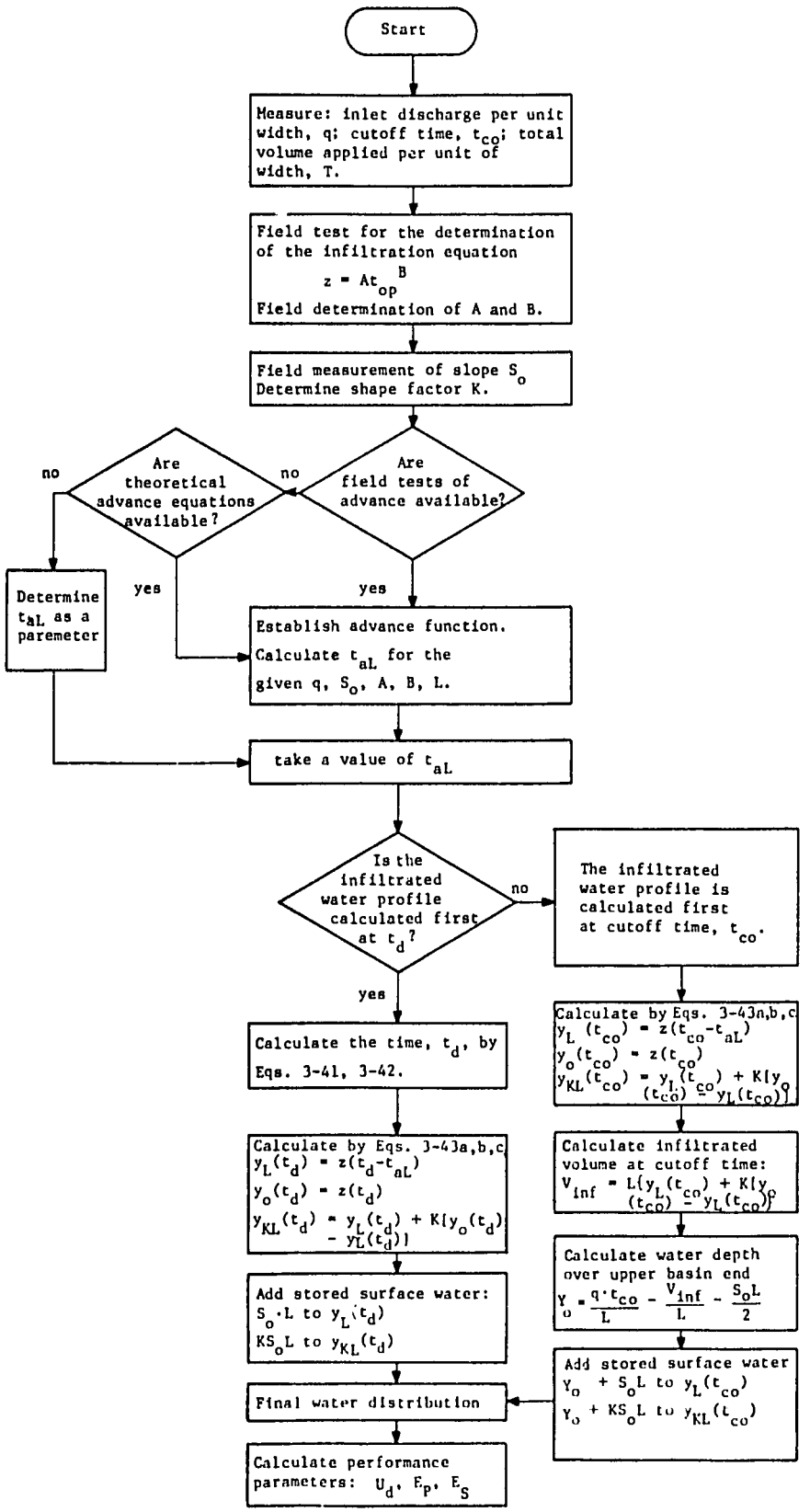


Fig. 3-14. Procedure for the determination of water distribution under basin irrigation using the simplified model.

APPENDIX 3-A

ESTIMATION OF K FOR THE TLS DISTRIBUTION

The estimation of K should be such that the TLS distribution will be as close as possible to the actual infiltrated water distribution. This can be defined by a minimum difference between the infiltrated water volumes. Three possible ways for the selection of K are suggested.

1. Equal Infiltrated Volumes

The shape factor, K, can be determined so that the infiltrated volume of water of the actual distribution is equal to the infiltrated volume of water from the TLS approximation. This means that the area below the mean depth line (Fig. 3-5) must equal the area above, so that

$$\frac{KL(y_o-\bar{y})}{2} = \frac{(1-K)L(\bar{y}-y_L)}{2} \tag{3-67}$$

Thus

$$K = \frac{\bar{y} - y_L}{y_o - y_L} = \frac{\bar{y} - y_L}{\Delta} \tag{3-68}$$

This, of course, means that these infiltrated depths must be known before K can be calculated.

2. Advance Shape Factor

During the advance phase, the volume of water infiltrated can be calculated from

$$V(t) = r_z y_o(t)x \tag{3-69}$$

where

$V(t)$ = volume infiltrated,

$y_o(t)$ = depth infiltrated at the top of the basin,

x = length of advance, and

r_z = shape factor for infiltrated depths.

According to Strelkoff (1977), when the infiltration is described by the Kostiakov equation (Eq. 3-4), the shape factor for infiltrated depths can be approximated by

$$r_z = \frac{1}{1 + B} \tag{3-70}$$

where B is the exponent in the Kostiakov equation. From Eq. 3-70, $0.5 \leq r_z \leq 1$.

From the work of Wilke and Smerdon (1965), it can be shown that r_z actually has the range of $0.6 \leq r_z \leq 1$ (see Fig. 3-15), when the infiltration is described by the Kostiakov equation, and the advance time is less than two times the time it would take to infiltrate a depth of water equal to the average surface depth. Wilke and Smerdon speculated that this also would hold true for much longer advance times.

For the TLS distribution

$$V(t) = \{K[y_o(t) - y_L(t)] + y_L(t)\}L \tag{3-71}$$

When the advance front has just reached the end of the basin, $y_L(t) = 0$, consequently (equating Eqs. 3-69 and 3-71)

$$K = r_z \tag{3-72}$$

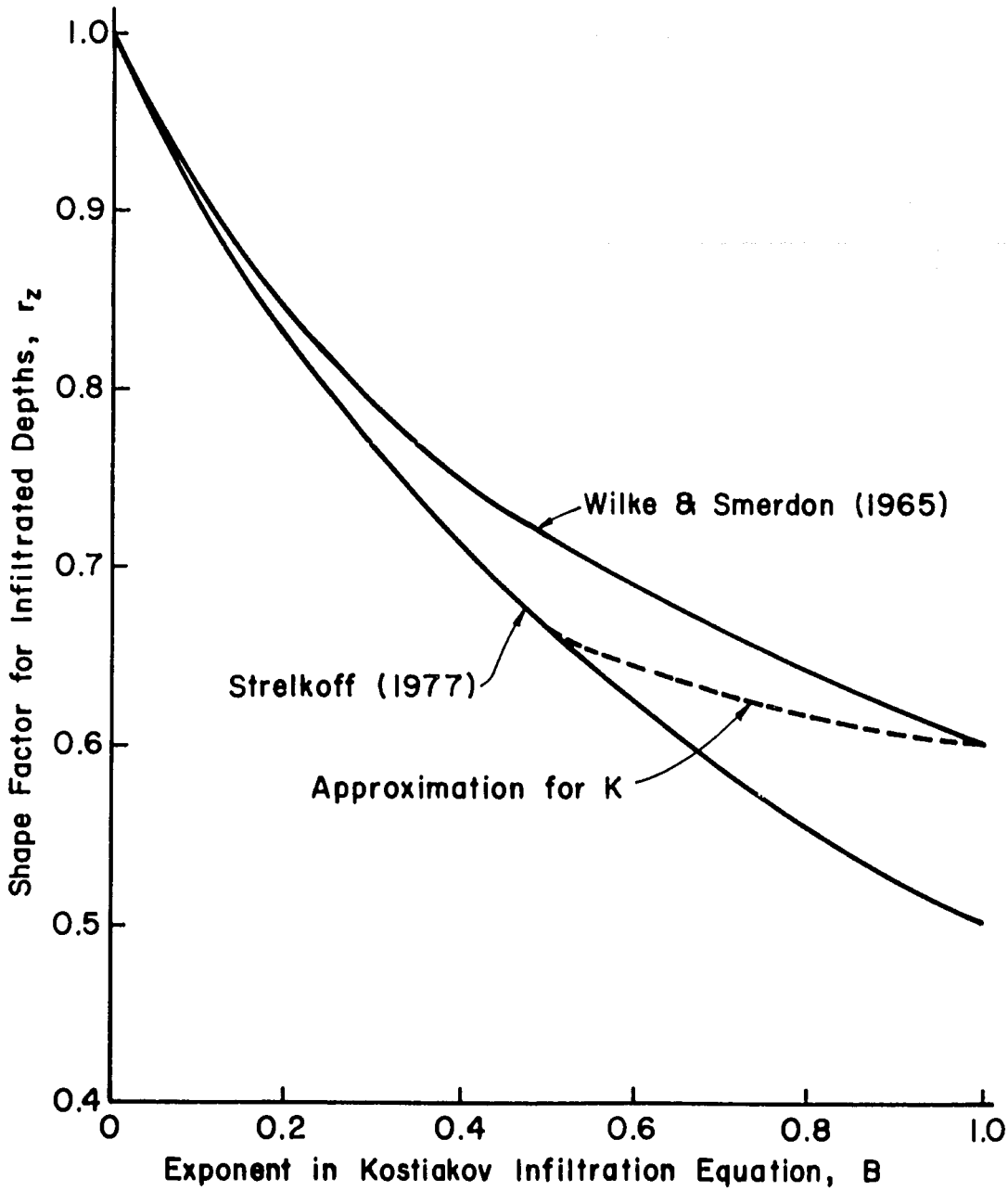


Figure 3-15. Shape factor for infiltrated depths, and K factors for Kostiakov infiltration function $z = At^B$.

When $y_L(t) \neq 0$, and $B \neq 1$, K will be somewhat less than the shape factor for infiltrated depths, in order that Eq. 3-71 hold true. When B = 1, Eq. 3-72 defines K exactly.

The example in Section 3-c shows that when B = 0.5, the approximation of $K = 1/(1+B) = 0.667$ is reasonable. Therefore, a reasonable approximation for K is probably that of

$$K = \frac{1}{1 + B} \qquad 0 \leq B \leq 0.5 \tag{3-73}$$

and some function (such as that shown by a dashed line in Fig. 3-15) that monotonically decreases from K = 0.667 to K = 0.6 for B = 0.5 to B = 1. This solution of K is, however, less accurate than the value of K as given in Eq. 3-68, however, this method allows K to be estimated from the infiltration function only.

3. Three Common Points

The factor K can be selected so that the average depth, $\bar{y} = y_L + K\Delta$, lies on the actual infiltrated depth curve at a distance of x = KL (Fig. 3-16). In this case, the actual infiltrated distribution and the TLS distribution will have three common points so that the difference between the two distributions is reduced, especially in comparison with the commonly used assumption of K = 0.5.

According to the above relationship

$$y_L + \Delta K = A[t_o - t_a(KL)]^B \tag{3-74}$$

where $t_a(KL)$ is the time of advance to a distance, KL, and t_o is the infiltration time for y_o. With an advance function of the form

$$x = a[t(x)]^b \tag{3-75}$$

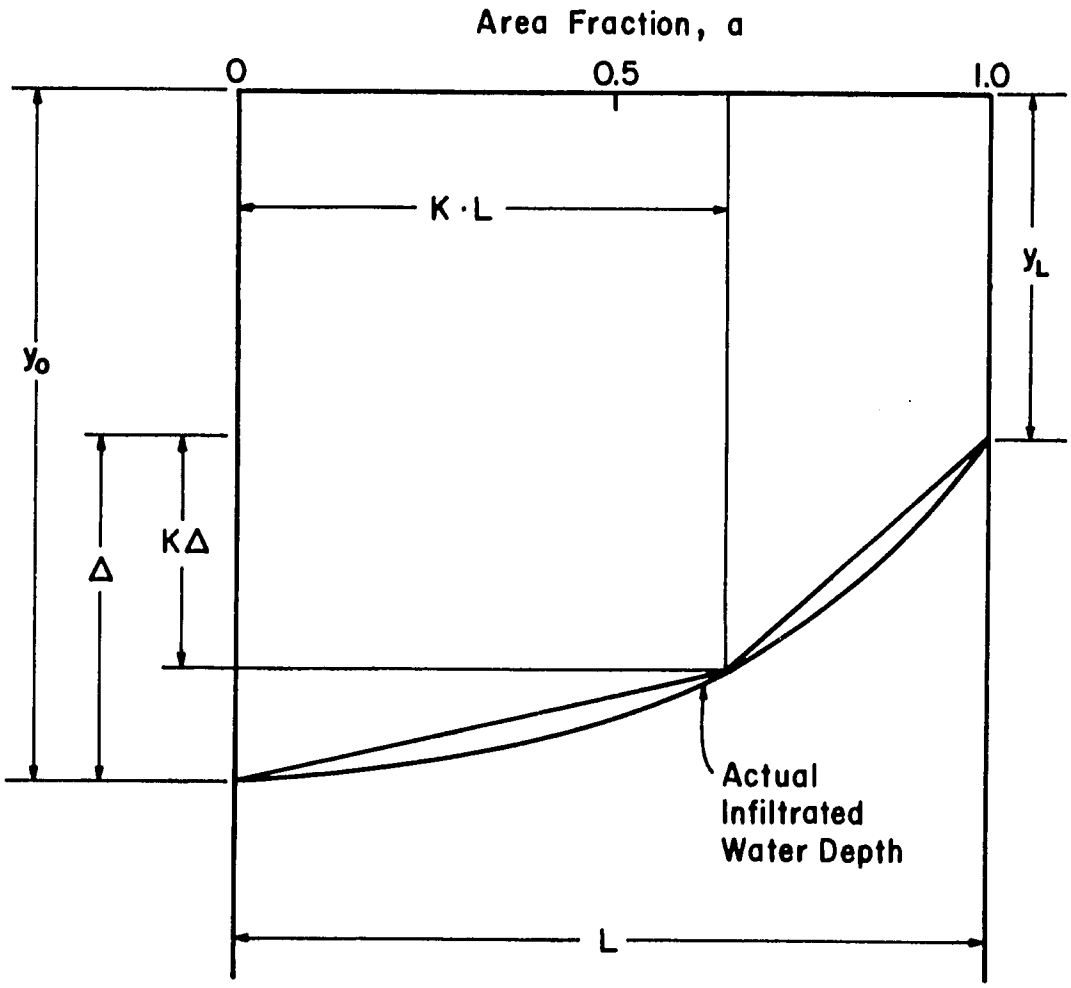


Figure 3-16. Schematic description for the evaluation of K by three common points.

where a and b are the advance function constants, the time $t_a(KL)$ can be expressed by:

$$t_a(KL) = C\ K^d L^d \tag{3-76}$$

with

$$C \doteq \left(\frac{1}{a}\right)^{\frac{1}{b}} \text{ and } d = \frac{1}{b} \tag{3-77}$$

Substituting Eq. 3-76 into Eq. 3-74:

$$y_L + \Delta K = A(t_o - C\ K^d L^d)^B \tag{3-78}$$

For a known infiltration function (A,B) and advance (a,b), K can be solved for any set of y_L and y_o. The solution of K from Eq. 3-78 is however, iterative.

This procedure, for calculating K, is more complicated than the shape factor procedure as it requires that the advance function and the infiltration function be known.

APPENDIX 3-B

ANALYSIS OF THE SIMPLIFIED RECESSION

Based on mass balance, Strelkoff (1977) has developed a recession equation for a sloped basin ("ponded case").

Assuming a linear variation in the infiltration rate, I, over the length, L - x, the change in the volume of the ponded water is given by:

$$\frac{dV_p(t)}{dt} = \frac{I[t_r(x) - t_{aL}] + I[t_r(x) - t_a(x)]}{2} (L-x) \tag{3-79}$$

where $V_p(t)$ is the surface ponded water volume per unit width, t_{aL} is the advance time to the basin end, and $t_a(x)$ is the advance time to point x, where x is the point at which recession is taking place.

As the ponded water volume is given by

$$V_p(t) = \frac{S_o(L-x)^2}{2} \tag{3-80}$$

Eq. 3-79 can be used to obtain

$$\frac{dx}{dt} = \frac{I[t_r(x) - t_{aL}] + I[t_r(x) - t_a(x)]}{2S_o} \tag{3-81}$$

If it is assumed that $t_a(x)$ can be treated as a constant, integration of Eq. 3-81 provides the recession equation:

$$x(t_r) = \frac{z[t_r(x) - t_{aL}] + z[t_r(x) - t_a(x)]}{2S_o} - \frac{z[t_d - t_{aL}] + z[t_d - t_a(x)]}{2S_o} \tag{3-82}$$

Strelkoff (1977) assumed that $t_a(x)$ could be considered as a constant:

$$t_a(x) = \frac{1}{3} t_{aL}.$$

If it is assumed that:

$$z[t_r(x) - t_{aL}] - z[t_r(x) - t_a(x)] = z[t_d - t_{aL}] - z[t_d - t_a(x)] \quad (3\text{-}83)$$

substitution of Eq. 3-83 into Eq. 3-82 results in

$$x(t_r) = \frac{z[t_r(x) - t_a(x)] - z[t_d - t_a(x)]}{S_o} \quad (3\text{-}84)$$

However, Eq. 3-84 is identical to Eq. 3-36, which was derived from the assumption that during the recession stage, the depth of water that infiltrated at a point is equal to the surface depth of water at that point when recession started. Consequently, the use of a recession equation, such as Eq. 3-82, is not necessary to determine the depth of water infiltrated during recession.

Chapter 4

USE OF WATER DISTRIBUTION MODELS FOR IMPROVING IRRIGATION

1. Introduction

Once the irrigation is evaluated, the need for improvements can be defined, however, changes in the system parameters to improve irrigation are different for regular and irregular basins.

Performance of irregular basins can be evaluated only by field measurements (Chapter 2) where improvement is achieved mainly by changes in the microtopography, by obtaining uniform, smooth and level basin surfaces with a geometry that allows rapid coverage of the area with water. No general functions can be formulated to relate microtopography and irrigation performance. A schematic description of the effect of microtopography and its improvement on the irrigation performance is given in Fig. 4-1.

Performance of regular basins can be evaluated by theoretical models which include some field tests (Chapter 3). Based on the theoretical models, the water distribution and the various efficiencies (E_p, E_s, U_d) can be expressed as a function of the cover time, t_{aL}, and the average depth, $\bar{y}$ (or the minimal depth, y_{min}). With the known relationships of t_{aL} and the system parameters (q, L, S_o, A, B, C) the water distribution and its derived efficiencies can be related to the system parameters. An example is given in Fig. 4-2.

In connection with Fig. 4-2, the following points should be considered:

a. In most practical cases, the advance curve is generally expressed as a relationship between the time of advance and the length of advance for various inlet stream sizes, while other factors are kept constant, (slope, soil, surface roughness). These relationships are expressed in the upper

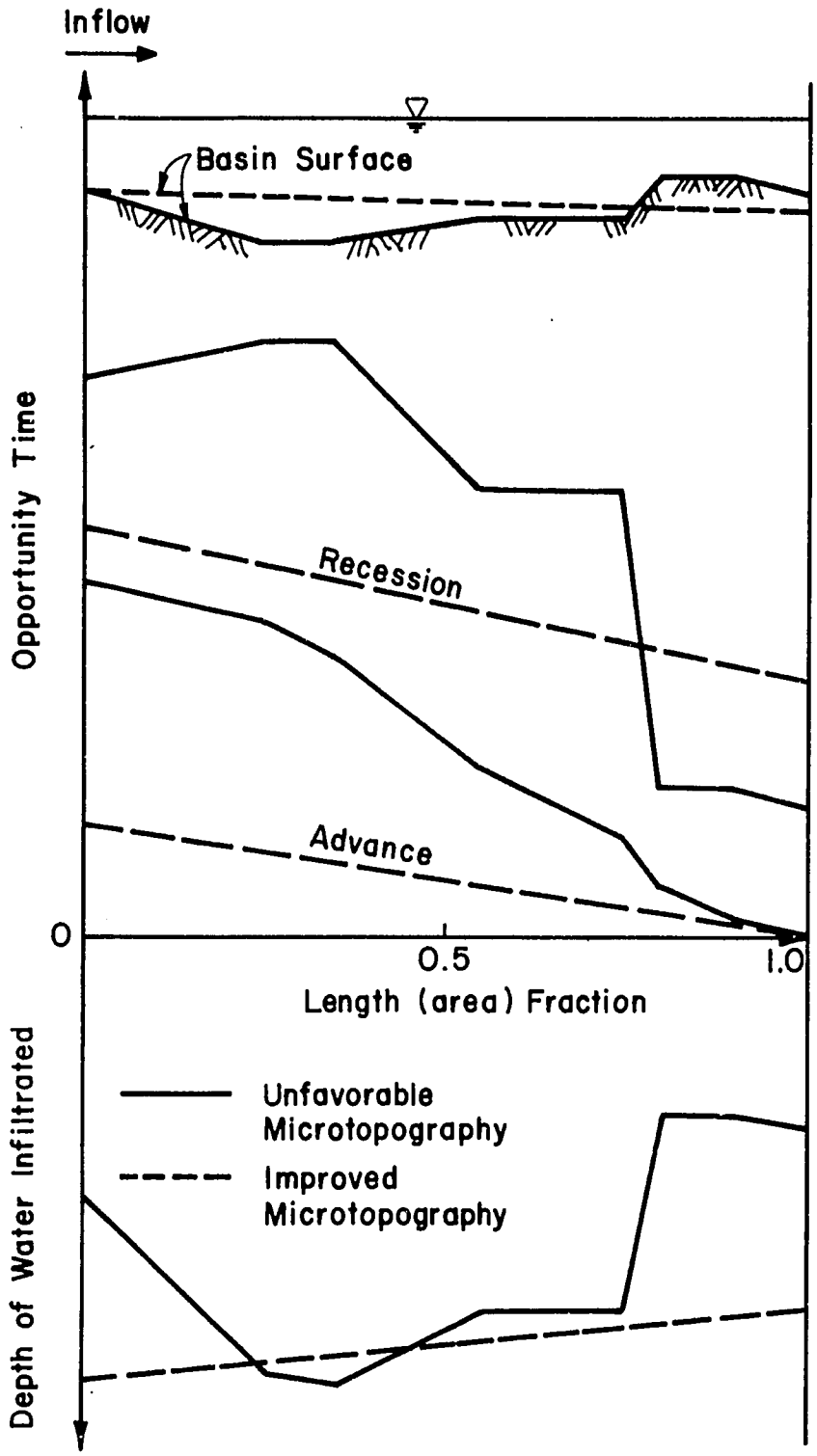


Figure 4-1. Schematic description of the effect of microtopography on water distribution under basin irrigation.

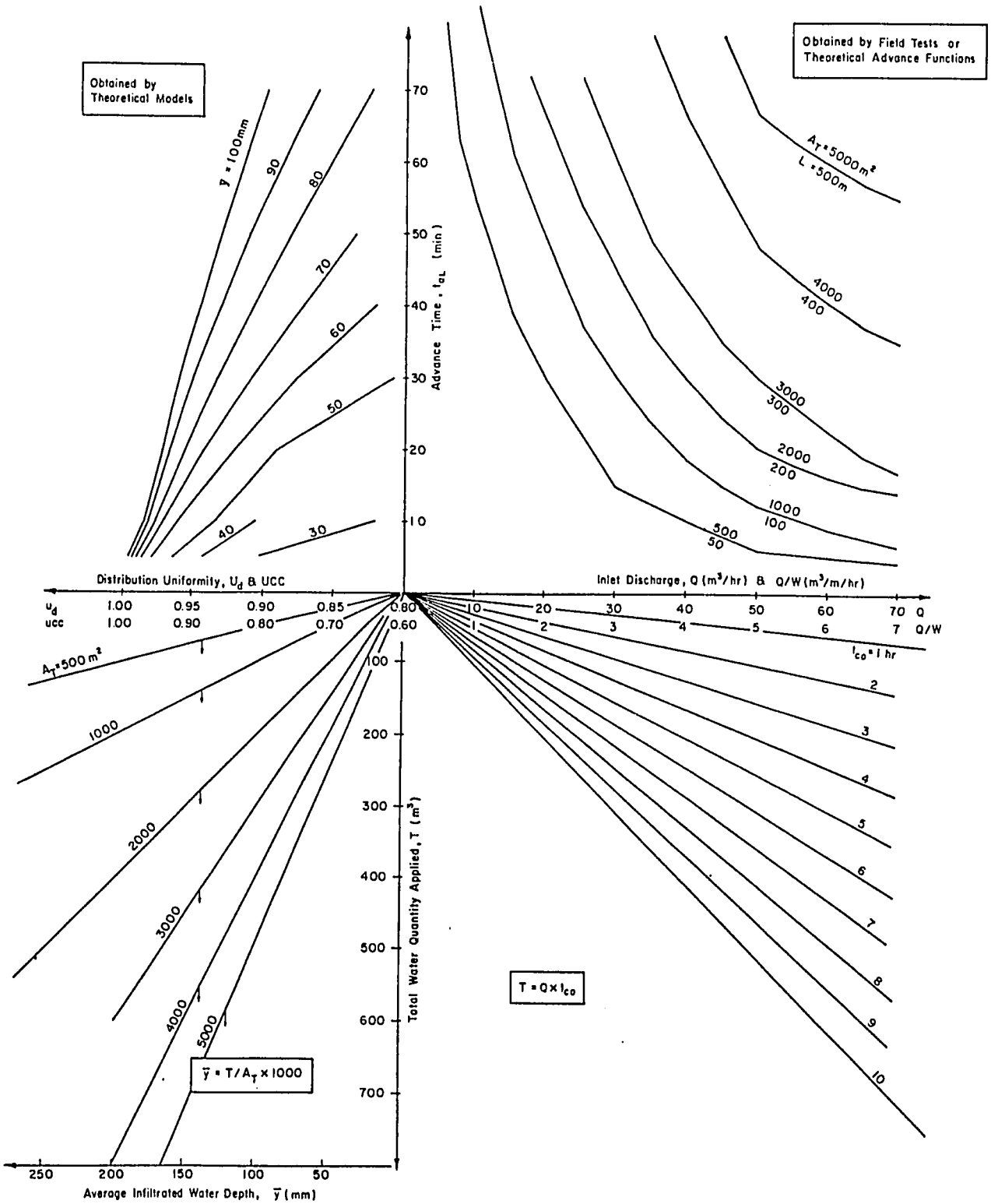


Figure 4-2. Relationships among system parameters for the improvement of basin irrigation performance.

right quadrant. The discharge can be expressed also as the stream size per unit of width.

b. The relationship between the efficiency and the cover time, t_{aL}, for various average depths of application, is derived from the model as described in Chapter 3 (see for example Table 3-1 and Figs. 3-11, 3-12, and 3-13 in Chapter 3). The uniformity distribution, U_d, is defined for any t_{aL} and $\bar{y}$, however, the other efficiencies, E_P and E_S, are dependent on h_R also; thus, they are calculated for any t_{aL} and $\bar{y}$ for a specific h_R.

c. From the upper two quadrants of Fig. 4-2, changes of basin length, L, (or basin area A_T) and inlet discharge (or q = Q/W) can be determined so that the efficiency is improved. However, any change in the discharge or the basin length will affect the average depth of application, $\bar{y}$. For a given and known h_R, this may affect E_P and E_S. Maintaining the same $\bar{y}$ requires a change in the cutoff time. These relations are given in the lower quadrants of Fig. 4-2 in which:

$$\frac{T}{W} = \frac{Q}{W} \times t_{co} \tag{4-1}$$

$$\bar{y} = \frac{T/W}{L} \tag{4-2}$$

where:

T = total quantity of water delivered intc the basin, $[L^3]$.

Q = inlet discharge delivered into the basin $[L^3T^{-1}]$.

W = basin width,

L = basin length,

$\bar{y}$ = the average water depth applied to the basin,

t_{co} = cutoff time, and

A_T = basin size (area).

d. The use of Fig. 4-2 to define the possible changes in the basin system parameters allows for simultaneous analysis of all the factors that are involved. Further details and examples will be given.

For the examples that follow, the simplified model was used for the case in which $z = 15\ t_{op}^{0.5}$ (z in mm, t_{op} in min), $K = 0.6$, and the basin is level.

The water distribution for various parameters is given in Chapter 3, Table 3-1 and Figs. 3-11, 3-12 and 3-13. The relationships between the advance time, t_{aL}, the inlet discharge Q, and the basin size A_T have been determined experimentally. The upper two quadrants of Fig. 4-2 have been established accordingly for this example. The lower two quadrants have been directly derived from Eqs. 4-1 and 4-2.

Possible improvement of the irrigation performance for the various performance categories will be analyzed assuming regular basin irrigation.

2. Improvement of Irrigation Performance where Distribution is Non-satisfactory while Other Efficiencies are Acceptable (Category II)

While the total water applied may be unchanged, the goal is to improve the distribution pattern. The minimal depth, y_{min}, is increased while the maximum depth, y_{max}, is decreased while keeping $\bar{y}$ constant. In the process, E_P and E_S will be improved also (Fig. 4-3).

A Study Case

The inlet stream size is $Q = 40 m^3/hr$; basin size $A_T = 2000\ m^2$; cover time, $t_{aL} = 30$ min; average depth applied, $\bar{y} = 60$ mm; distribution uniformity $U_d = 0.88$ (UCC = 0.76); total quantity of water delivered, $T = A_T\bar{y} = 120\ m^3$; and cutoff time $t_{co} = \frac{T}{Q} = 3$ hr. To determine possible improvements, Fig. 4-2 is used. Improvement of U_d with about the same $\bar{y} = 60$ mm is possible in any of the following typical combinations summarized in Table 4-1.

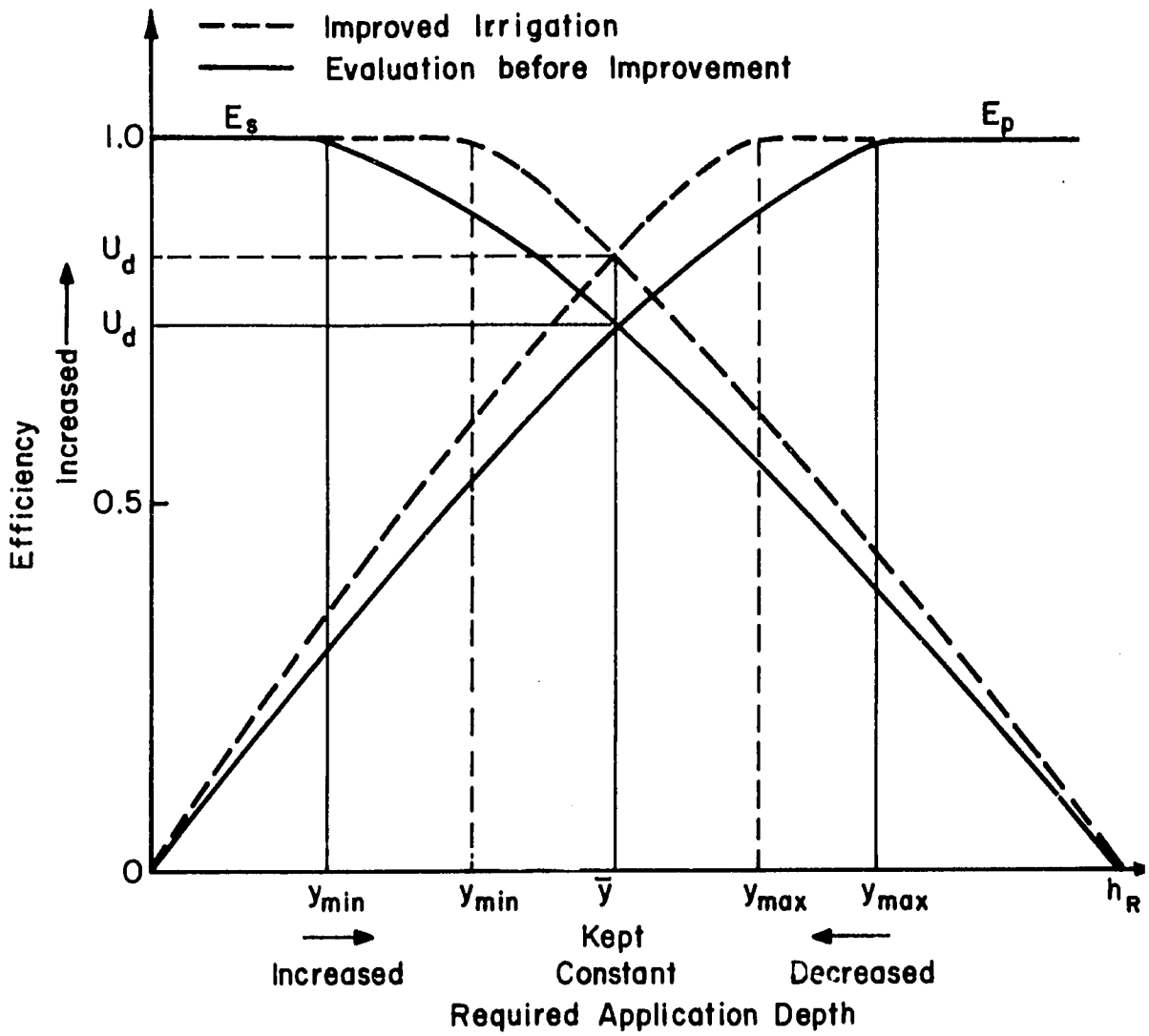


Figure 4-3. Improved distribution uniformity for the same average applied depth.

Table 4-1. System parameter changes for improving Category II of basin irrigation.

Case		$\bar{y}$ mm	Q m^3/hr	A m^2	t_{aL} min	U_d (UCC)	T m^3	t_{co} hr	Remarks
studied		60	40	2000	30	0.88 (.76)	120	3	E_p and E_s satisfactory higher U_d required
Improvement possibilities	1	60	50	2000	21	0.92 (.84)	120	2.4	
	2	60	40	1000	19	0.93 (.86)	60	1.5	
	3	60	50	1000	13	0.95 (.90)	60	1.2	

Table 4-2. System parameter changes for improving Category III of basin irrigation.

Case		$\bar{y}$ mm	Q m^3/hr	A m^2	t_{aL} min	U_d (UCC)	T m^3	t_{co} hr	Remarks
studied		80	50	1000	12	0.97 (0.94)	80	1.6	U_d satisfactory required depth $\bar{y}$ = 100 mm
Improvement possibilities	1	100	50	1000	12	0.98 (0.96)	100	2.0	
	2	100	60	1000	9	0.98 (0.96)	100	1.66	
	3	100	50	500	6	0.99 (0.98)	50	1.0	

a. Increasing stream size to $Q = 50\ m^3/hr$ with the same basin size $A_T = 2000\ m^2$. This results in $t_{aL} = 21$ min and $U_d = 0.92$. The total quantity of water delivered is unchanged at $120\ m^3$ and the cutoff time is $t_{co} = \frac{T}{Q} = 2.4$ hr.

b. The same stream size of $Q = 40\ m^3/hr$ with a smaller basin size of $A_T = 1000\ m^2$. This results in $t_{aL} = 19$ min and $U_d = 0.93$. The total quantity of water delivered is $T = 60\ m^3$ and the cutoff time is $t_{co} = 1.5$ hr.

c. Increasing stream size to $Q = 50\ m^3/hr$ and decreasing basin size to $A_T = 1000\ m^2$. This results in $t_{aL} = 13$ min and $U_d = 0.95$. The total quantity of water delivered is $60\ m^3$ and the cutoff time is 1.2 hr.

3. Improvement of Irrigation Performance where Distribution is Satisfactory and Applied Depth of Water, $\bar{y}$, is Smaller than the Required Depth (Category III)

The applied depth of water should be increased while maintaining the satisfactory distribution. Increasing the average depth of application can be carried out by the change of one of two system parameters: decreasing the basin size and/or increasing total basin water quantity.

A Study Case

Study case data together with three typical possibilities for changing the system parameters resulting from Fig. 4-2 are given in Table 4-2.

4. Improvement of Irrigation Performance where Distribution is Satisfactory and the Applied Depth of Water, $\bar{y}$, is Larger than the Required Depth (Category IV)

The applied depth of water, $\bar{y}$, should be decreased while maintaining the satisfactory distribution. Decreasing the average depth of application can be carried out by a change of one of two system parameters: increasing the basin size and/or decreasing total water quantity. However, these changes may result also in lower uniformity; therefore, a proper combination

of system parameter changes must be found, so that together with the required decrease in the applied water depth, the uniformity will be maintained at its satisfactory level.

A Study Case

Study case data together with some typical possibilities for changing the system parameters as derived from Fig. 4-2 are given in Table 4-3. It should be noted that in the cases where only one system parameter is changed, the smaller depth of water is associated with a decreased U_d (Cases 1, 2, 3 in Table 4-3). Therefore, in order to decrease the average water depth without decreasing U_d, a proper combination of simultaneous changes of several parameters is required (Cases 4 and 5 in Table 4-3).

5. Improvement of Irrigation Performance where Distribution is Nonsatisfactory and the Applied Depth of Water is Smaller than the Required Depth (Category V).

The storage efficiency, E_s, and the distribution uniformity, U_d, are not satisfactory so that the applied depth of water should be increased together with improvement in the distribution pattern. A combination of increased stream size and/or decreased basin size is needed.

A Study Case

Study case data together with four typical possibilities for changing the system parameters as derived from Fig. 4-2 are given in Table 4-4.

6. Improvement of Irrigation Performance where Distribution is Nonsatisfactory and the Applied Depth of Water is Larger than the Required Depth (Category VI)

The deep percolation efficiency, E_p, and the distribution uniformity, U_d, are both nonsatisfactory so that the applied depth of water should be decreased together with improvement in the distribution pattern. A combination of changes in stream size, basin size, and cutoff time is needed.

Table 4-3. System parameter changes for improving Category IV of basin irrigation performance.

Case		$\bar{y}$ mm	Q m^3/hr	A m^2	t_{aL} min	U_d (UCC)	T m^3	t_{co} hr	Remarks
studied		100	50	2000	20	0.97 (0.94)	200	4	U_d satisfactory required $\bar{y}$ = 80 mm
Improvement possibilities	1	80	50	2000	20	0.94 (0.88)	160	3.2	U_d is worse (decreased shutoff time)
	2	80	40	2000	30	0.93 (0.86)	160	4	U_d is worse (decreased discharge)
	3	80	50	3000	30	0.93 (0.86)	240	4.8	U_d is worse (increased basin size)
	4	80	60	2000	16	0.96 (0.92)	160	2.66	Sharp decrease in t_{co} compensates for increase in Q
	5	80	30	500	15	0.97 (0.94)	40	1.33	Decrease of Q compensates for decrease in basin size

Table 4-4. System parameter changes for improving Category V of basin irrigation performance.

Case		$\bar{y}$ mm	Q m_3/hr	A m^2	t_{aL} min	U_d (UCC)	T m^3	t_{co} hr	Remarks
studied		80	40	2000	30	0.93 (0.86)	160	4	U_d should be improved and $\bar{y}$ should be increased to 100 mm
Improvement possibilities	1	100	40	2000	30	0.96 (0.92)	200	5	Increasing shutoff time to 5 hr. Provide $\bar{y}$ = 100 mm. The U_d should be improved further.
	2	100	50	2000	20	0.97 (0.94)	200	4	Increasing inlet stream alone improves U_d for the required depth.
	3	100	40	1000	19	0.97 (0.94)	100	2.5	Decreasing basin size with the same discharge improves U_d for the required depth.
	4	100	50	1000	11	0.98 (0.96)	100	2.0	Increasing inlet stream size with decreasing basin size improves U_d for the required depth.

Other combinations of simultaneous changes of operating conditions are possible.

A Study Case

Study case data together with four typical possibilities for changing the system parameters as derived from Fig. 4-2 are given in Table 4-5.

It should be noted, for example, that for a case where $A_T \geq 1000\ m^2$ and $Q \leq 60\ m^3/hr$, there is no possible solution that can provide simultaneously an average applied depth of $\bar{y} = 70$ mm and distribution uniformity of $U_d \geq 0.97$. To achieve a distribution uniformity greater than 0.97, the average depth applied must be greater than 70 mm. The required depth, h_R, should be adjusted to the average depth by extending the interval between irrigations.

7. Summary of Case Studies

The results of the case studies are listed in Table 4-6. Categories II, III, IV, V and VI of basin irrigation performance are summarized in Table 4-6.

Table 4-5. System parameter changes for improving Category VI of basin irrigation performance.

Case		$\bar{y}$ mm	Q m^3/hr	A m^2	t_{aL} min	U_d (UCC)	T m^2	t_{co} hr	Remarks
studied		90	40	3000	42	0.93 (0.86)	270	6.75	The U_d should be improved together with decreasing $\bar{y}$ to 70 mm
Improvement possibilities	1	70	40	3000	42	0.87 (0.74)	210	5.25	Decreasing only the depth of application causes worse U_d.
	2	90	60	3000	22.5	0.95 (0.90)	270	4.5	Increasing stream size only with decreased shutoff time improves U_d but not to the required level.
	3	70	60	3000	22.5	0.93 (0.86)	210	3.5	Increasing stream size for the lower $\bar{y}$ = 70 exhibits only slight improvement in U_d.
	4	70	60	2000	46	0.95 (0.90)	140	2.33	Increasing stream size in decreasing basin size improves U_d with the smaller $\bar{y}$.

Table 4-6. Improvements of basin irrigation--summary of study cases.

Parameter	Category	Study case	Required improvement	Changes and results				
				1	2	3	4	5
$Q(m^3/hr)$		40		50	40	50		
$A\ (m^2)$		2000		2000	1000	1000		
t_{aL} (hr)	II	0.50		0.35	0.32	0.22		
t_{co} (hr)		3.0		2.4	1.5	1.2		
U_d		0.88	≥ 0.90	0.92	0.93	.95		
$\bar{y}$		60	60	60	60	60		
$Q(m^3/hr)$		50		50	60	50		
$A\ (m^2)$		1000		1000	1000	500		
t_{aL} (hr)	III	0.2		0.2	0.15	0.10		
t_{co} (hr)		1.6		2.0	1.66	1.0		
U_d		0.97	≥ 0.90	0.98	0.98	0.99		
$\bar{y}$		80	100	100	100	100		
$Q(m^3/hr)$		50		50	40	50	60	30
$A\ (m^2)$		2000		2000	2000	3000	2000	500
t_{aL} (hr)	IV	0.33		0.33	0.5	0.5	0.27	0.25
t_{co} (hr)		4		3.2	4.0	4.8	2.66	1.33
U_d		0.97	≥ 0.95	0.94	0.93	0.93	0.96	0.97
$\bar{y}$		100	80	80	80	80	80	80
$Q(m^3/hr)$		40		40	50	40	50	
$A\ (m^2)$		2000		2000	2000	1000	1000	
t_{aL} (hr)	V	0.5		0.5	0.33	0.32	0.183	
t_{co} (hr)		4.0		5.0	4.0	2.5	2.0	
U_d		0.93	≥ 0.95	0.96	0.97	0.97	0.98	
$\bar{y}$		80	100	100	100	100	100	
$Q(m^3/hr)$		40		40	60	60	60	
$A\ (m^2)$		3000		3000	3000	3000	2000	
t_{aL} (hr)	VI	0.7		0.7	0.375	0.375	0.27	
t_{co} (hr)		6.75		5.25	4.5	3.5	2.33	
U_d		0.93	≥ 0.95	0.87	0.95	0.93	0.95	
$\bar{y}$		90	70	70	90	70	70	

Chapter 5

IMPROVEMENT OF BASIN IRRIGATION PERFORMANCE BY LAND PREPARATION

1. Introduction

With irregular basins, or when the basin bed is not level, the irrigation performance is affected primarily by the basin microtopography and shape. Changes in other system parameters such as stream size, cutoff time, etc., may have only limited effects on the improvement of the irrigation performance. In such a case, land preparation is the most and only effective measure in improving the basin irrigation performance. However, land preparation is an expensive operation and its effectiveness can be determined only by comparing the cost for improved basin microtopography to the resulting benefits in terms of the irrigation performance.

2. Extent of Land Preparation

The need for land preparation as a means for improving basin irrigation performance, can be determined by considering the following:

a. the topography of the basin surface--a survey of the basin surface provides the information about necessary land leveling;

b. appearance of dry spots during the filling of the basin and/or soon after the ponded water is absorbed by the soil, which is related to the existence of knolls, mounds or ridges (high spots) in the basin area;

c. appearance of ponded areas in several spots when most of the basin is already exposed, which is related to pockets or swales (low spots) in the basin area;

d. ponded water at the far end of the basin while most of the basin area is already exposed, which is related to the slope of the basin;

e. nonuniform advancement of the water front during the filling of the basin, although the stream size is large enough; and

f. nonuniform depths of wetted profile.

Land preparation for already existing basins is composed of two stages:

a. The first stage is the minimal and necessary land preparation that should be carried out in any case, which includes:

(i) rough grading to remove knolls, mounds and ridges and filling pockets and swales; and

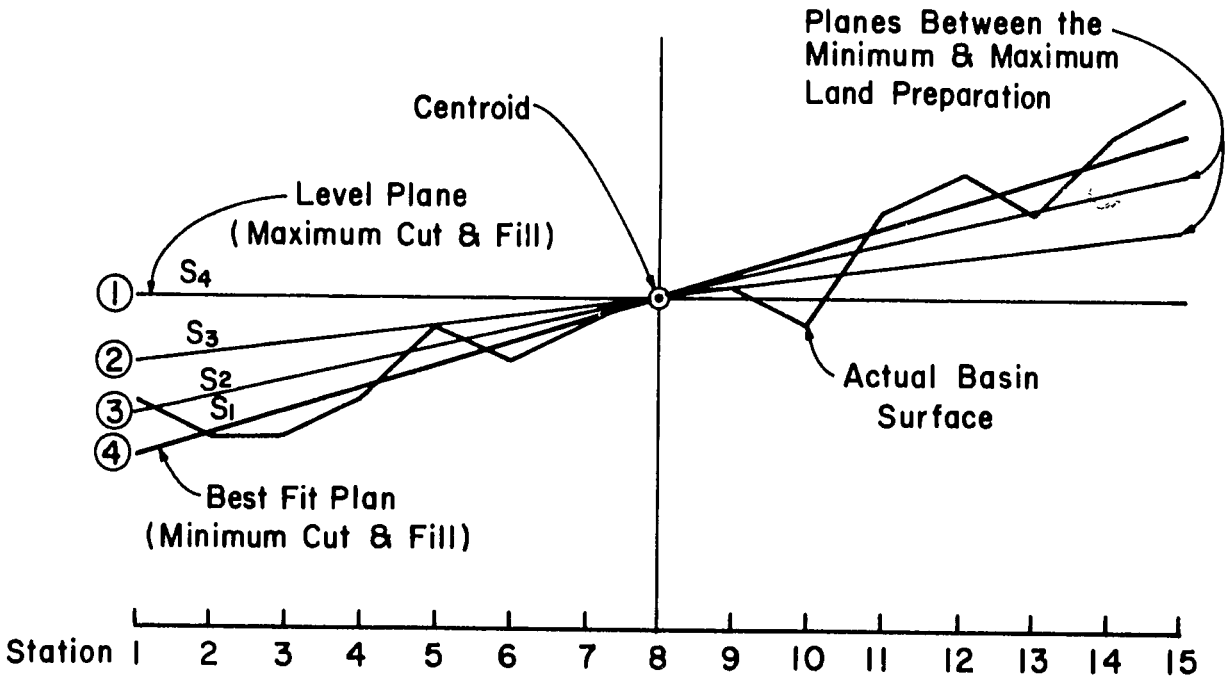
(ii) establishing the proper cross basin slope, as close as possible to level; and

b. The second stage is to establish the grade line level of the basin (the extent of this stage can be determined according to the cost and expected benefits).

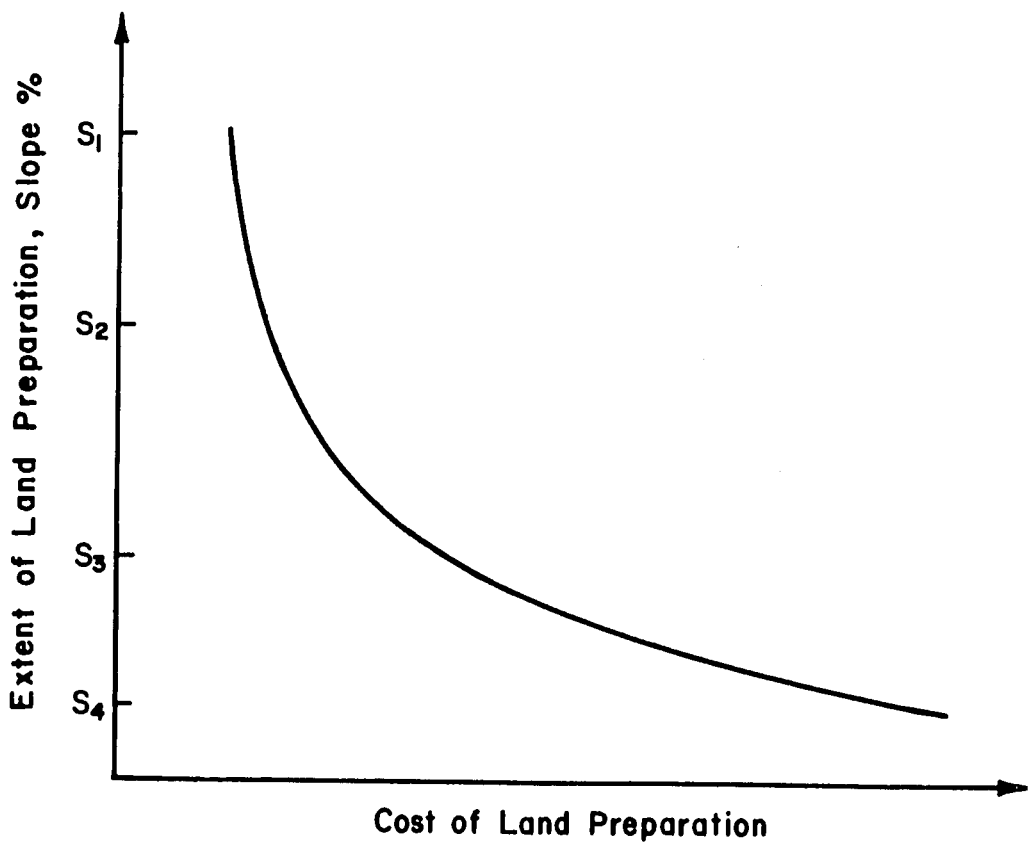
The optimal land slope for basin irrigation is assumed to be level or almost level, although proper management and operating conditions can lead to highly efficient irrigation with slightly sloped basins. However, for a given topography, leveling to zero slope is the most expensive land preparation, therefore, other degrees of land preparation should be considered.

The minimal land preparation can be obtained using the "Plane Method" leveling in which a plane passes through the centroid of the area with slopes that provide minimum cut and fill. Any other plane that passes through the centroid but with a different slope will provide cut volume equal to fill volume, but the volumes are greater than that obtained by the "Plane Method," therefore, preparing this plane is more expensive. The alternatives are illustrated schematically in Fig. 5-1.

If the best fit plane (which is obtained by the Plane Method) exhibits a downward slope which is too high, other leveling degrees should be



(a) Possible Slopes



(b) Cost Function

Fig. 5-1. Schematic description of the extent of land preparation as a function of its cost.

considered, as is shown by Planes No. 2, 3 and 4 of Fig. 5-1a. For each plane, the cost of land preparation can be evaluated so that relationships between the final land leveling, as expressed by the downward slope, and the land preparation cost, can be established. This is demonstrated in Fig. 5-1b.

3. Effect of the Extent of Land Leveling on Basin Irrigation Performance

A complete rational procedure to evaluate the effect of land leveling on the irrigation performance is not feasible, since several of the most important hydraulic characteristics cannot be defined as a function of the basin microtopography. However, with a few assumptions, a rational or quasi-rational procedure can be developed. A theoretical model will be presented together with a simplified model.

The various extents of land preparation are presented by a slope along the basin, when the closer the slope to level, the more land preparation is needed. With a slope along the basin, the irrigation process includes three successive phases as shown in Fig. 5-2.

a. Initially, water advances along the basin until the basin is completely covered. As stated in Chapter 3, the advance can be defined theoretically or empirically.

Assuming an advance function of the form:

$$x = a\left[t_a(x)\right]^b \tag{5-1}$$

where:

x = distance of the water front from basin upper end

$t_a(x)$ = time that the water front arrives at a distance x, and

a, b = constants

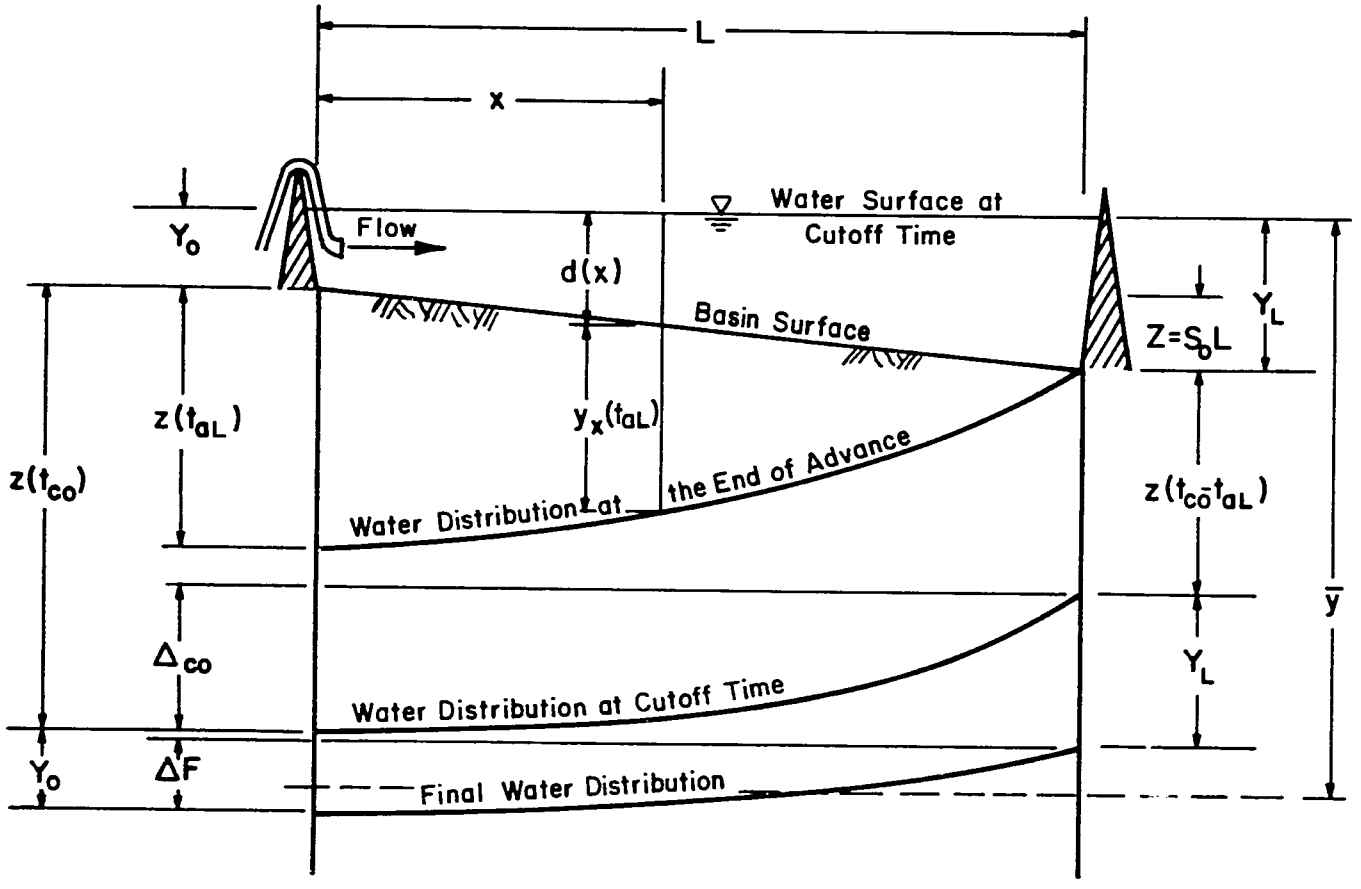


Figure 5-2. Schematic description of basin irrigation model with bed slope.

Constant b, and to a smaller extent constant a, is dependent on the basin slope.

b. After the basin is covered, the inlet stream, which is larger than the infiltration capacity of the basin, causes ponding of water over the surface. This stage terminates at time t_{co}, by which time the gross volume of water delivered to the basin is

$$T = Qt_{co} \tag{5-2}$$

where:

T = gross volume of water delivered to the basin (L^3)

Q = inlet stream size (L^3T^{-1})

t_{co} = cutoff time

At the end of this phase, a portion of the water that has been delivered to the basin has infiltrated while the other portion has accumulated on the surface. If the effect of the water depth over the surface on the infiltration rate is neglected, the extent of land preparation (the basin slope) affects the soil water distribution mainly by the ponded water depth distribution over the surface.

c. When the inlet stream is turned off, the third phase is the absorption of the ponded water. In more general cases, this phase is regarded as two separate phases; depletion and recession as described in Chapter 3. As shown in Chapter 3 (the simplified recession), it can be assumed that during this phase the ponded depth of water, which varies along the basin due to the surface slope, infiltrates into the soil. Consequently, during this phase the ponded water over each point is the additional infiltrated depth at that point (Fig. 5-2).

4. Calculation of Soil Water Distribution (theoretical model)

Referring to Figure 5-2, the water distribution in the soil, and the irrigation performance parameters (E_P, E_S, U_d) can be evaluated as follows:

a. At the end of the advance phase.

The water depth infiltrated at any distance x is given by:

$$y_x(t_{aL}) = A\left[t_{aL} - t_a(x)\right]^B \tag{5-3}$$

where

$$t_{aL} = \left(\frac{L}{a}\right)^{\frac{1}{b}} \qquad t_a(x) = \left(\frac{x}{a}\right)^{\frac{1}{b}} \tag{5-4}$$

Substituting Eq. 5-4 into Eq. 5-3, the water depth infiltrated at the end of advance is given by:

$$y_x(t_{aL}) = A\left[\left(\frac{L}{a}\right)^{\frac{1}{b}} - \left(\frac{x}{a}\right)^{\frac{1}{b}}\right]^B \tag{5-5}$$

The volume of water infiltrated at the end of the advance is:

$$V(t_{aL}) = \int_{x=o}^{L} y_x(t_{aL})dx = A\int_{o}^{L}\left[\left(\frac{L}{a}\right)^{\frac{1}{b}} - \left(\frac{x}{a}\right)^{\frac{1}{b}}\right]^B dx \tag{5-6}$$

b. At the end of the delivery phase.

The water depth infiltrated at any distance x is given by:

$$y_x(t_{co}) = A\left[t_{co} - t_a(x)\right]^B \tag{5-7}$$

Substituting Eq. 5-4 into Eq. 5-7, the water depth infiltrated at the end of this phase is given by:

$$y_x(t_{co}) = A\left[t_{co} - \left(\frac{x}{a}\right)^{\frac{1}{b}}\right]^B \tag{5-8}$$

The volume of water infiltrated at the end of the delivery phase (at the cutoff time) is:

$$V(t_{co}) = \int_{x=o}^{L} y_x(t_{co})dx = A\int_{o}^{L}\left[t_{co} - \left(\frac{x}{a}\right)^{\frac{1}{b}}\right]^B dx \tag{5-9}$$

The depth of water ponded over the surface, d(x), at a distance x is dependent upon the relationship between the depth of water ponded over the surface at the lower end of the basin at cutoff, Y_L, and the elevation difference in the basin, Z (see Fig. 5-2). That relationship is

$$d(x) = Y_L - Z\left(1 - \frac{x}{L}\right) \qquad x \geq L\left(1 - \frac{Y_L}{Z}\right) \tag{5-10a}$$

$$d(x) = 0 \qquad x \leq L\left(1 - \frac{Y_L}{Z}\right) \tag{5-10b}$$

The volume of water ponded on the surface, $V_P(t_{co})$, is

$$V_P(t_{co}) = L\left(Y_L - \frac{Z}{2}\right) \qquad Y_L \geq Z \tag{5-11a}$$

$$V_P(t_{co}) = \frac{L\,Y_L^{\,2}}{2Z} \qquad Y_L \leq Z \tag{5-11b}$$

c. At the end of the irrigation (the end of absorption of the ponded water).

The water depth infiltrated at any distance x is

$$y_x = A\left[t_{co} - \left(\frac{x}{a}\right)\frac{1}{b}\right]^B + Y_L + Z\left(\frac{x}{L} - 1\right) \qquad \text{for } x \geq L\left(1 - \frac{Y_L}{Z}\right) \tag{5-12a}$$

$$y_x = A\left[t_{co} - \left(\frac{x}{a}\right)\frac{1}{b}\right]^B \qquad \text{for } x \leq L\left(1 - \frac{Y_L}{Z}\right) \tag{5-12b}$$

The volume of water infiltrated at the end of the irrigation, V, is

$$V = A\int_0^L\left[t_{co} - \left(\frac{x}{a}\right)\frac{1}{b}\right]^B dx + \int_{x_m}^L\left[Y_L - Z\left(1 - \frac{x}{L}\right)\right]dx \tag{5-13a}$$

or

$$V = A\int_0^L\left[t_{co} - \left(\frac{x}{a}\right)\frac{1}{b}\right]^B dx + (L - x_m)\left[Y_L - Z + \frac{L - x_m}{2L}\right] \tag{5-13b}$$

in which

$$x_m = 0 \qquad \text{for } Y_L \geq Z$$

$$x_m = L\left(1 - \frac{Y_L}{Z}\right) \qquad \text{for } Y_L \leq Z$$

For a known inlet unit stream size, q, and delivery time, t_{co}, Y_L can be determined from Eq. 5-13 for any Z by substituting:

$$V = qt_{co} \tag{5-14}$$

With Y_L known as a function of Z, y_x in Eq. 5-12 can be defined as a function of Z so that for the known infiltration equation (A, B), advancement equation (a, b), inlet discharge (q), delivery time (t_{co}), and basin dimensions (L, W), the water distribution y_x can be determined for any Z (that is for any slope) and the irrigation performance parameters can be derived. The use of the general equations as given in Eqs. 5-12 and 5-13 is associated with two major difficulties; the need to express and use the advance equation, and the solution of the integrals. These difficulties can be eliminated with the use of the simplified model as described in Chapter 3.

5. Calculation of Soil Water Distribution (simplified model)

The infiltrated water distribution as derived from the simplified model of Chapter 3 is shown in Fig. 5-3. Changing the slope of the basin changes the advance function and cover time, and the infiltrated water distribution over the basin surface. If the advance function and cover time were independent of slope (this assumption can be justified only for dense crops or rough surfaces), the infiltrated water distribution at the end of the delivery stage and the volume of the ponded water would be independent of slope.

Using the simplified model of Fig. 5-3, and the infiltration equation, the water distribution at the end of the irrigation is expressed by y_L,

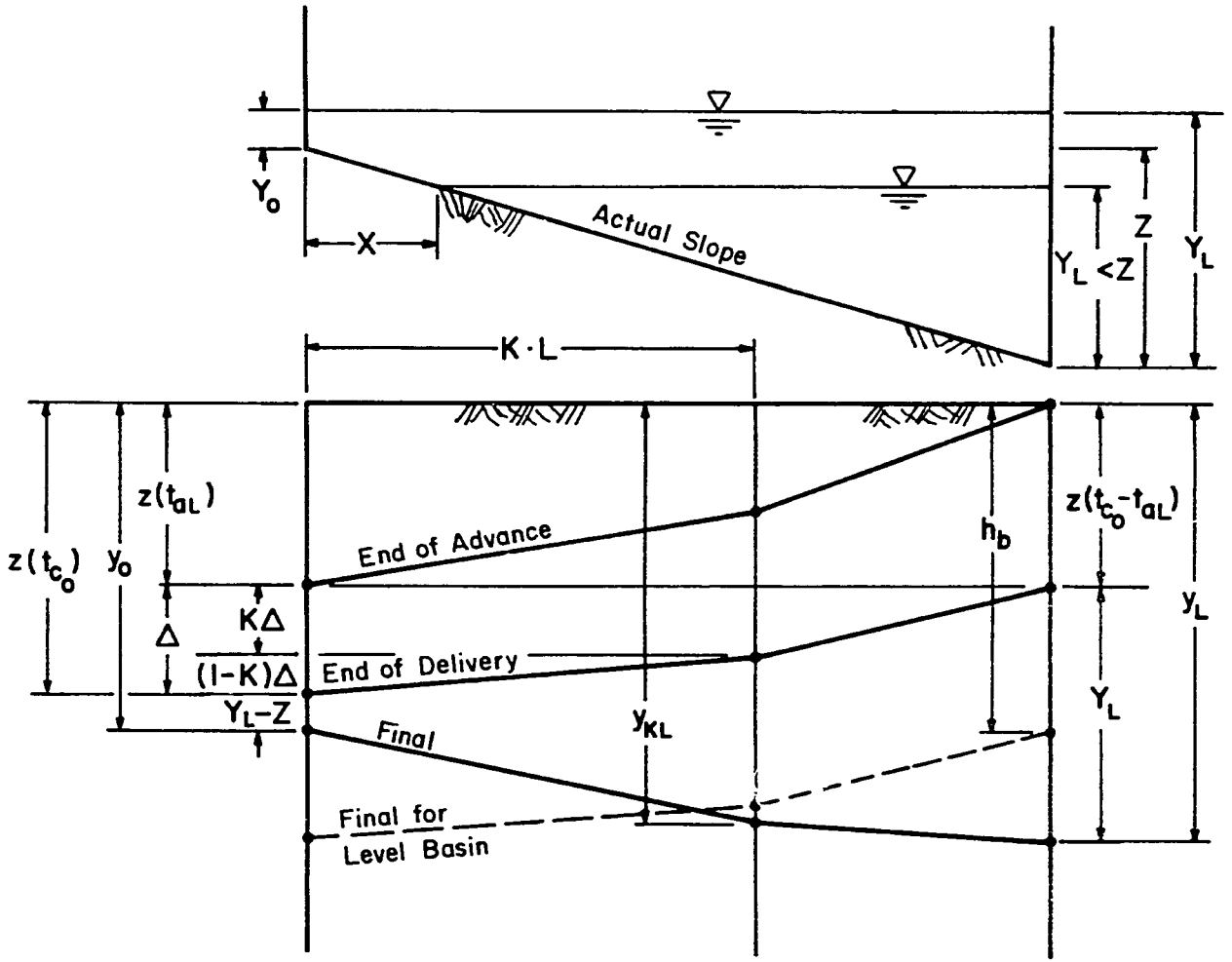


Fig. 5-3. Simplified model for the determination of infiltrated water distribution under basin irrigation.

y_{KL} and y_o (the infiltrated depths at the lower end, at K·L and at the upper end, respectively). When $Y_L < Z$, the minimal infiltrated depth, y_{min}, and the distance, x, at which it occurs are required also. The infiltrated water distribution can be established with the use of the simplified model as follows:

The depth of water infiltrated at the lower basin end at the cutoff time is:

$$y_L(t_{co}) = A[t_{co} - t_{aL}]^B \tag{5-15}$$

The depth of water infiltrated at the upper basin end at the cutoff time is:

$$y_o(t_{co}) = At_{co}^{\ B} \tag{5-16}$$

The difference in infiltrated depths at the upper and lower ends at the cutoff time is:

$$\Delta_{co} = y_o(t_{co}) - y_L(t_{co}) = A\left[t_{co}^{\ B} - (t_{co} - t_{aL})^{\ B}\right] \tag{5-17}$$

The infiltrated volume at the end of the delivery phase (at cutoff) is:

$$V(t_{co}) = A\left[Kt_{co}^{\ B} + (1 - K)(t_{co} - t_{aL})^B\right] L\cdot W \tag{5-18}$$

The volume of water ponded over the surface at the cutoff time, V_p, is calculated by:

$$V_p = Qt - V(t_{co}) \tag{5-19}$$

For a known infiltration function, advance time, t_{aL}, cutoff time, t_{co}, and unit inlet discharge, Q, the water profile at the end of the delivery phase and the ponded volume of water can be calculated by Eqs. 5-18 and 5-19.

The depth of the ponded water at the lower end, Y_L, can be calculated as (Fig. 5-4).

$$Y_L = \frac{V_P(t_{co})}{LW} + \frac{Z}{2} \qquad \text{for } \frac{V_P(t_{co})}{LW} \geq \frac{Z}{2} \tag{5-20a}$$

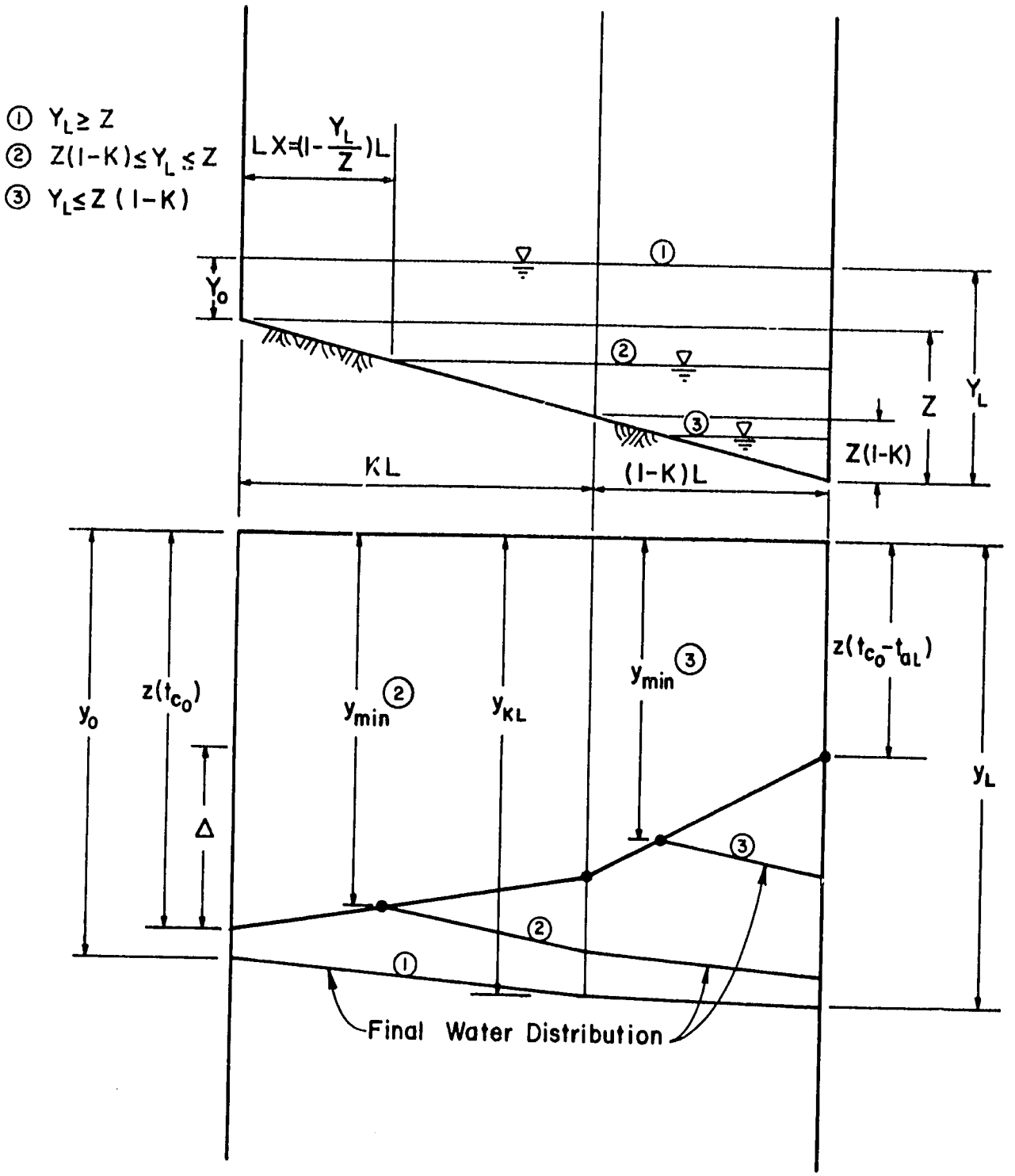


Figure 5-4. Water distribution profiles for sloped basins under three levels of ponded water.

$$Y_L = \sqrt{\frac{2ZV_p(t_{co})}{LW}} \qquad \text{for } \frac{V_p(t_{co})}{LW} \leq \frac{Z}{2} \tag{5-20b}$$

For known V_p (from Eqs. 5-18, 5-19), Y_L can be determined for any slope. With Y_L known, and with Eq. 5-10, the three water depths that form the water distribution, y_L, y_{KL}, and y_o can be determined as follows (Fig. 5-4).

$$y_L = z\ (t_{co}-t_{aL}) + Y_L \tag{5-21}$$

$$y_{KL} = z\ (t_{co}-t_{aL}) + K\Delta + (Y_L-Z) + KZ \qquad \text{for } Y_L \geq Z \tag{5-22a}$$

$$y_{KL} = z\ (t_{co}-t_{aL}) + K\Delta + Y_L\left(\frac{K-X}{1-X}\right) \qquad \text{for } Z(1-K) \leq Y_L < Z \tag{5-22b}$$

$$y_{KL} = z\ (t_{co}-t_{aL}) + K\Delta \qquad \text{for } Y_L < Z(1-K) \tag{5-22c}$$

$$y_o = z\ (t_{co}) + Y_L - Z \qquad \text{for } Y_L \geq Z \tag{5-23a}$$

$$y_o = z\ (t_{co}) \qquad \text{for } Y_L < Z \tag{5-23b}$$

$$y_{min} = \text{minimum of } (y_o,\ y_L) \qquad \text{for } Y_L \geq Z \tag{5-24a}$$

$$y_{min} = z\ (t_{co}-t_{aL}) + K\Delta + (1-K)\Delta\frac{K-X}{X} \qquad \text{for } Z(1-K) \leq Y_L < Z \tag{5-24b}$$

$$y_{min} = z\ (t_{co}-t_{aL}) + K\Delta\left(\frac{1-X}{1-K}\right) \qquad \text{for } Y_L < Z(1-K) \tag{5-24c}$$

where

$$X = 1 - \frac{Y_L}{Z}$$

With Eq. 5-18 to Eq. 5-24, the final infiltrated water distribution can be calculated for various surface slopes. The required data are: infiltration characteristics (A, B), inlet stream size (Q), cutoff time (t_{co}) or average depth of application ($\bar{y}$), and advance time (t_{aL}), (either measured or calculated from a theoretical equation). With these data, the infiltrated water volume at cutoff time is calculated from Eq. 5-18, and the surface ponded water is calculated from Eq. 5-19. The ponded water depth at the basin lower end (Y_L) is calculated by

Eq. 5-20 for various values of the slope and is used to calculate the infiltrated water distribution for the various slopes (Eq. 5-21 to Eq. 5-24). The effect of the slope on the final water distribution is illustrated in the following examples.

Example 1

Given: a basin of 5 m width and 20 m length, a cumulative infiltration equation $z = 10t^{0.5}$ (z = mm, t = min), an inlet stream size Q = 7.92 m^3/hr, a cutoff time t_{co} = 100 min, and a required depth of application of 118.5 mm.

Assume: an average advance time of t_{aL} = 40 min for all slopes and $K = 0.6$.

Find: the approximate water distributions and irrigation efficiencies for various slopes.

The average depth of application is 132 mm. The calculations for the water distribution and irrigation performance parameters are shown in Table 5-1. The approximated water profiles for the various slopes are shown in Fig. 5-5. The following should be noted:

a. Since $z(t_{co})$ is 100 mm and is smaller than h_R(118 mm), with the increase of Z, a deficiency zone is created at the upper end of the basin (Z = 60 mm and more).

b. Since $z(t_{co})$ is relatively close to h_R and both are relatively high, the values of E_P and E_S will remain high even for large Z. The maximum possible value of h_B is approximately 0.6 x (118.5 - 95.5) = 13.8 mm so that the minimal value of E_P is $\frac{118.5 - 13.8}{132} = 0.793$ and the minimal value of E_S is $\frac{118.5 - 13.8}{118.5} = 0.883$. However, the value of U_d is decreased and an excessive depth of application is obtained at the basin lower end. Fig. 5-6 shows the relationship between the uniformity and the slope.

Table 5-1. Calculation of water distribution and irrigation performance (example 1).

S_o z	Z mm	t_{aL} min	$z(t_{co}-t_{aL})$ mm	$z(t_{co})$ mm	Δ mm	$K\Delta$ mm	$z(t_{co}+t_{aL})$ $+K\Delta$ mm	$V(t_{co})$ m^3	V_p m^3	$\frac{V_p}{WL}$ mm	$\frac{Z}{2}$ mm	Y_L (mm) Case*	Y_L (mm) value	y_L mm	Z(1-K) mm	y_{KL} (mm) Case*	y_{KL} (mm) value	X	y_o (mm) Case*	y_o (mm) value	y_{min} (mm) Case*	y_{min} (mm) value	h_B mm	h_C mm	E_P	E_S	U_d	UCC
0	0	40	77.5	100	22.6	13.5	91.0	9.1	4.1	41	0	a	41	118.4	0	a	131.9	0.0	a	141.0	a	118.4	0	2.7	0.895	1.0	0.979	0.958
0.05	10	40	77.5	100	22.6	13.5	91.0	9.1	4.1	41	5	a	46	123.4	4	a	132.9	0.0	a	136.0	a	123.4	0	1.5	0.895	1.0	0.989	0.978
0.1	20	40	77.5	100	22.6	13.5	91.0	9.1	4.1	41	10	a	51	128.4	8	a	133.9	0.0	a	131.0	a	128.4	0	0.5	0.895	1.0	0.996	0.992
0.2	40	40	77.5	100	22.6	13.5	91.0	9.1	4.1	41	20	a	61	138.4	16	a	135.9	0.0	a	121.0	a	121.0	0	2.4	0.895	1.0	0.981	0.962
0.3	60	40	77.5	100	22.6	13.5	91.0	9.1	4.1	41	30	a	71	148.4	24	a	137.9	0.0	a	111.0	a	111.0	0.6	4.9	0.892	0.995	0.962	0.924
0.4	80	40	77.5	100	22.6	13.5	91.0	9.1	4.1	41	40	a	81	158.4	32	a	139.9	0.0	a	101.0	a	101.0	2.5	7.4	0.877	0.978	0.943	0.886
0.5	100	40	77.5	100	22.6	13.5	91.0	9.1	4.1	41	50	b	90.5	167.9	40	b	141.4	0.095	b	100	b	98.6	4.3	9.9	0.864	0.963	0.924	0.348
0.6	120	40	77.5	100	22.6	13.5	91.0	9.1	4.1	41	60	b	99.2	176.6	48	b	142.2	0.173	b	100	b	97.4	5.7	12.6	0.853	0.951	0.904	0.808
1.0	200	40	77.5	100	22.6	13.5	91.0	9.1	4.1	41	100	b	128.1	205.5	80	b	138.9	0.360	b	100	b	94.5	9.2	16.0	0.827	0.922	0.878	0.756

*Case a, b, or c refers to the particular form of Eq. 5-21, 5-23, 5-24, or 5-25.

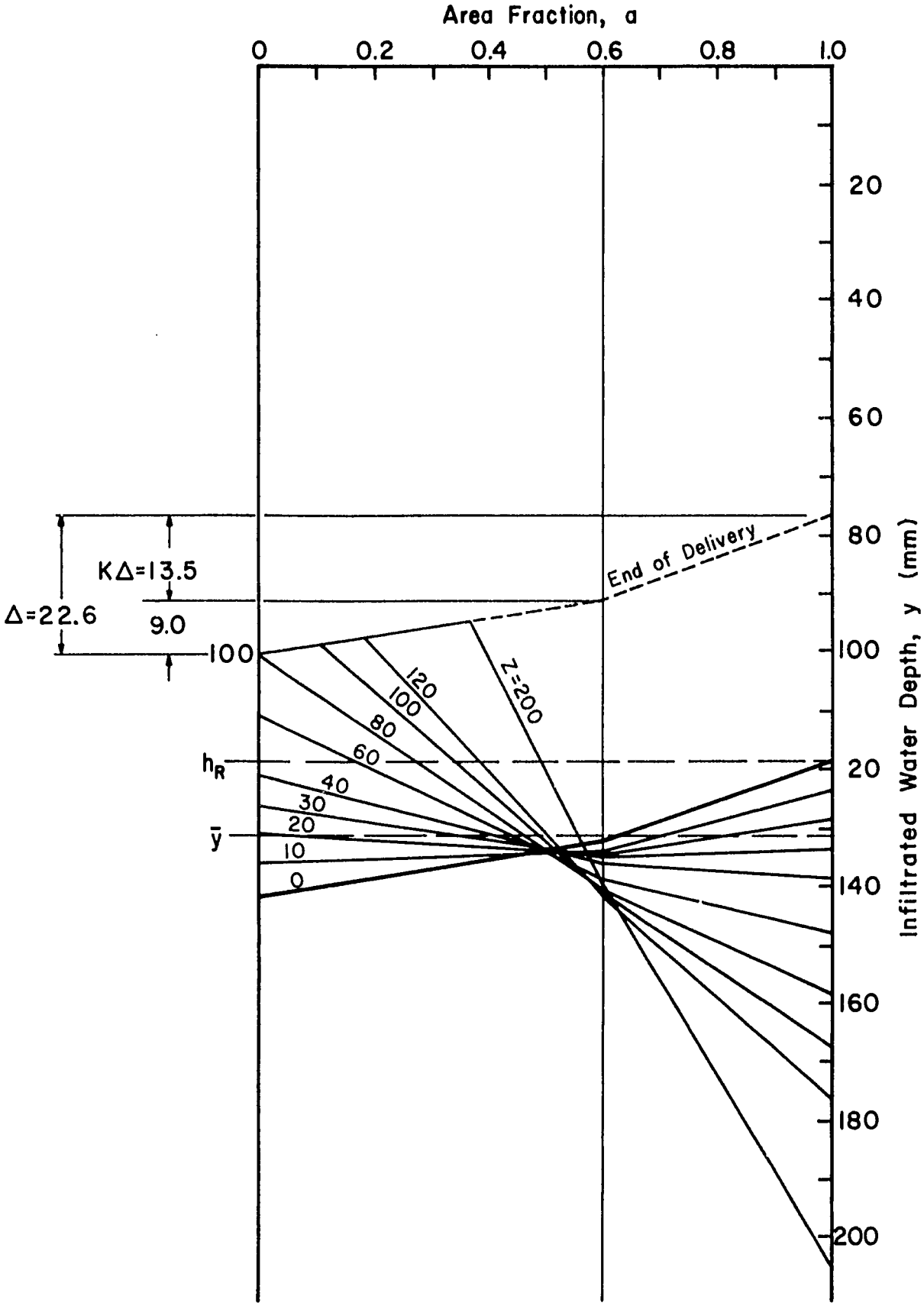


Fig. 5-5. Approximate infiltrated water profiles for various elevation differences (Example 1).

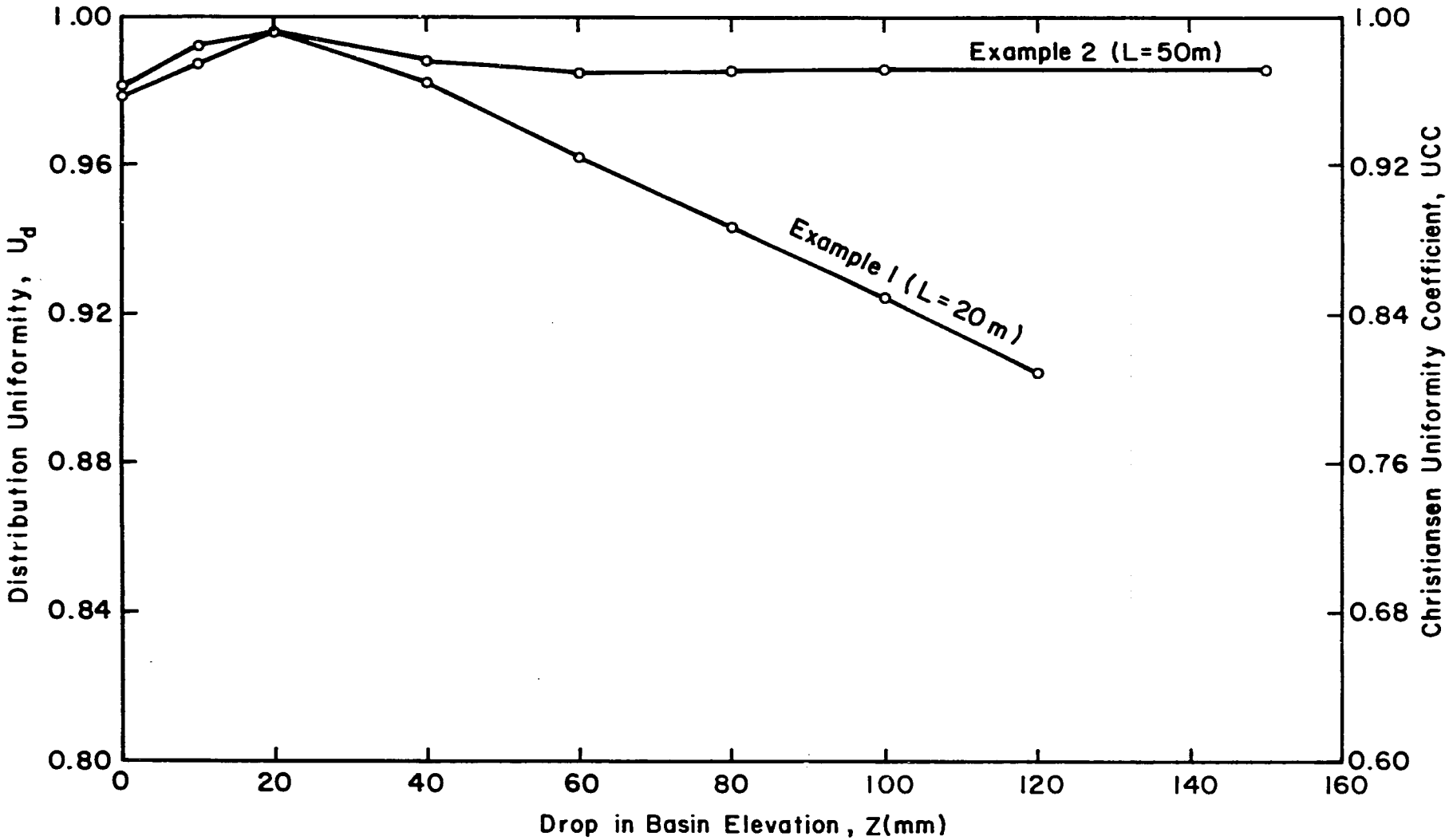


Figure 5-6. Relationship between slope and distribution uniformity (Examples 1 and 2).

Example 2

Given: a basin of 10 m width and 20 m length, a cumulative, infiltration operation $z = 10t^{0.5}$ (y = mm, t = min), an inlet stream size Q = 30 m^3/hr, a cutoff time t_{co} = 100 min, and a required depth of application of 90 mm.

Assume: K = 0.6 and the time of advance changes with the slope according to the following empirical relationship:

$t_{aL}(50) = 30$ min $\qquad S \leq 0.02\%$

$t_{aL}(50) = 5.1/S_o^{1/2}$ min $\qquad S \geq 0.04\%$

where $t_{aL}(50)$ is the cover time for the 50 m basin, and S is the basin slope.

Find: the approximate water distributions and irrigation efficiencies for various slopes. The cover times for various elevation differences are:

Z(mm)	0	10	20	40	60	80	100	150
S_o(%)	0	0.02	0.04	0.08	0.12	0.16	0.2	0.3
t_{aL}(min)	30.0	30.0	25.4	18.0	14.7	12.7	11.4	9.3

The calculations of the water distribution and irrigation performance are shown in Table 5-2. The approximated water profiles for the various slopes are shown in Fig. 5-7. In regard to Table 5-2 and Fig. 5-7, the following should be noted.

a. With the steeper slope, the advance time is shorter, improving the water distribution at cutoff time.

b. Since $z(t_{co})$ is 100 mm and is larger than the required depth of application, h_R = 90 mm, no deficiency, h_B, occurs so that E_P and E_S are constants, independent of the slope. E_S equals 1.0 and E_P equals $h_R/\bar{y}$. For this case, since $h_R/\bar{y}$ = 0.9, E_P has a relatively high value.

Table 5-2. Calculation of water distribution and irrigation performance (example 2).

Z mm	t_{aL} min	$z(t_{co}-t_{aL})$ mm	$z(t_{co})$ mm	Δ mm	$K\Delta$ mm	$z(t_{co}+t_{aL})$ $+K\Delta$ mm	$V(t_{co})$ m^3	V_p m^3	$\frac{V_p}{WL}$ mm	$\frac{Z}{2}$ mm	Y_L (mm) Case*	Y_L (mm) value	y_L mm	$Z(1-K)$ mm	y_{KL} (mm) Case*	y_{KL} (mm) value	X	y_o (mm) Case*	y_o (mm) value	y_{min} (mm) Case*	y_{min} (mm) value	h_B mm	h_C mm	E_P	E_S	U_d	UCC
0	30	83.7	100	16.3	9.8	93.5	46.7	3.3	6.6	0	a	6.6	90.3	0	a	100.1	0.0	a	106.6	a	90.3	0	1.94	0.9	1.0	0.981	0.962
10	30	83.7	100	16.3	9.8	93.5	46.7	3.3	6.6	5	a	11.6	95.3	4	a	101.1	0.0	a	101.6	a	95.3	0	0.78	0.9	1.0	0.992	0.984
20	25.4	86.4	100	13.6	8.2	94.6	47.3	2.7	5.4	10	b	14.7	101.1	8	b	101.3	0.265	b	100	b	97.6	0	0.55	0.9	1.0	0.995	0.990
40	18	90.5	100	9.5	5.7	96.2	48.1	1.9	3.8	20	b	17.4	107.9	16	b	97.2	0.565	b	100	b	96.4	0	1.185	0.9	1.0	0.988	0.976
60	14.7	92.3	100	7.7	4.6	96.9	48.4	1.6	3.2	30	b	19.6	111.9	24	c	96.9	0.673	b	100	c	96.1	0	1.487	0.9	1.0	0.985	0.970
80	12.7	93.4	100	6.6	4.0	97.4	48.7	1.3	2.6	40	b	20.4	113.8	32	c	97.4	0.745	b	100	c	95.9	0	1.38	0.9	1.0	0.986	0.972
100	11.4	94.1	100	5.9	3.5	97.6	48.8	1.2	2.4	50	b	21.9	116.0	40	c	97.6	0.781	b	100	c	96.0	0	1.36	0.9	1.0	0.986	0.972
150	9.3	95.2	100	4.8	2.9	98.1	49.0	1.0	2.0	75	b	24.5	119.7	60	c	98.1	0.837	b	100	c	96.4	0	1.38	0.9	1.0	0.986	0.972

*Case a, b, or c refers to the particular form of Eqs. 5-21, 5-23, 5-24, or 5-25.

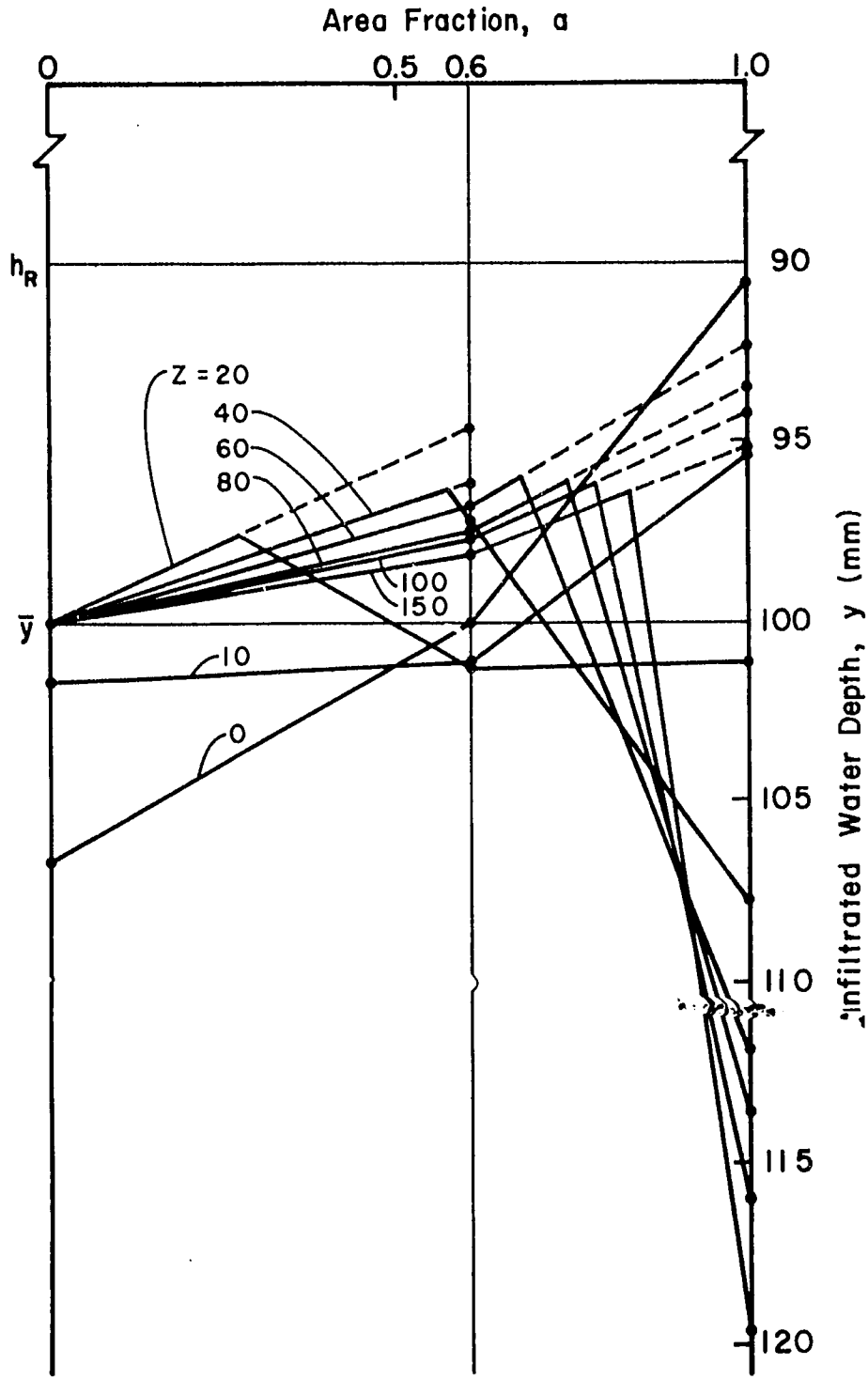


Figure 5-7. Approximate infiltrated water profiles for various elevation differences (Example 2).

c. Since the water ponded over the basin surface at the end of the delivery phase is a relatively small portion of the delivered volume (maximum 6.6%), the slope has only limited effect on the final water distribution. The ponded water is restricted to a relatively small portion of the basin, thus U_d remains relatively high. The relationship between uniformity and the difference in elevation are shown in Fig. 5-6.

6. Effect of Land Slope on Water Distribution Under Basin Irrigation

In the preceding sections, the effect of the slope on the final water distribution has been demonstrated so that for any particular case, the water distribution profile can be calculated and the irrigation performance parameters can be derived. With the simplified model of Chapter 3, however, the effect of the slope on the water distribution can be determined in general terms, enabling one to predict the water distribution profile and to determine the optimal slope. The optimal slope is the slope for which the distribution uniformity is maximum when the other parameters are kept constant.

The slope of the basin affects the advance time, t_{aL}, and the ponded Y_o and Y_L [illegible] Fig. 5-4). For a given set of conditions, the slope, therefore, affects the distribution uniformity.

The advance time, t_{aL}, can be related to the basin slope, S_o, through an advance equation. Using the mass balance equation, Eq. 3-38.

$$L[r_y d_o + r_z y_o(t_{aL})] = q t_{aL} \tag{5-25}$$

The surface water depth at the upper end is assumed to be normal depth (Strelkoff, 1977) and can be expressed by Manning's formula for a wide and shallow channel as

$$d_o = \left(\frac{q \cdot n}{CU}\right)^{0.6} \frac{1}{S_o^{0.3}} \tag{5-26}$$

Substituting d_o from Eq. 5-26, and $y_o(t_{aL}) = z(t_{aL})$, as given by the Kostiakov infiltration equation (Eq. 3-4), into Eq. 5-25 results in

$$L\left[r_y\left(\frac{qn}{CU}\right)^{3/5}\frac{1}{S_o^{0.3}} + r_z A t_{aL}^{B}\right] = q t_{aL} \tag{5-27}$$

Rearranging gives:

$$q t_{aL} - L r_z A t_{aL}^{B} = L r_y\left(\frac{qn}{CU}\right)^{0.6}\frac{1}{S_o^{0.3}} \tag{5-28}$$

The relationship between t_{aL} and S_o can be calculated from Eq. 5-28 when A, B, L, r_y, q, and n are given. The inlet stream size, q, can be used as a parameter. Typical t_{aL} - S_o relationships are shown in Fig. 5-8 for an example described in detail later.

The difference between the infiltrated depths at the upper and lower basin ends at cutoff time can be calculated also as a function of the slope. This difference is:

$$\Delta_{co} = z(t_{co}) - z(t_{co} - t_{aL}) = A t_{co}^{B} - A(t_{co} - t_{aL})^{B} \tag{5-29}$$

At time, t_{co}, the term $A t_{co}^{B}$ is constant, while the other term of the right-hand side of Eq. 5-29 is dependent on t_{aL} which is a function of S_o. Consequently, Δ_{co} is a function of S_o when t_{co} is given, along with A, B, L, r_y, q, n. However, to find the relationship between Δ_{co} and S_o, t_{aL} should be established first as a function of S_o. Δ_{co} as a function of S_o is shown in 5-9 for an example described in detail later.

The effect of the slope can be analyzed for a set of given conditions, among them constant q and t_{co}, so that the total water volume applied with the various slopes is the same. Thus, the average depth of water infiltrated is the same for the various slopes, however, the distribution and the uniformity are different. In order to define the effect of the slope on the final water distribution, a simple parameter is selected as an

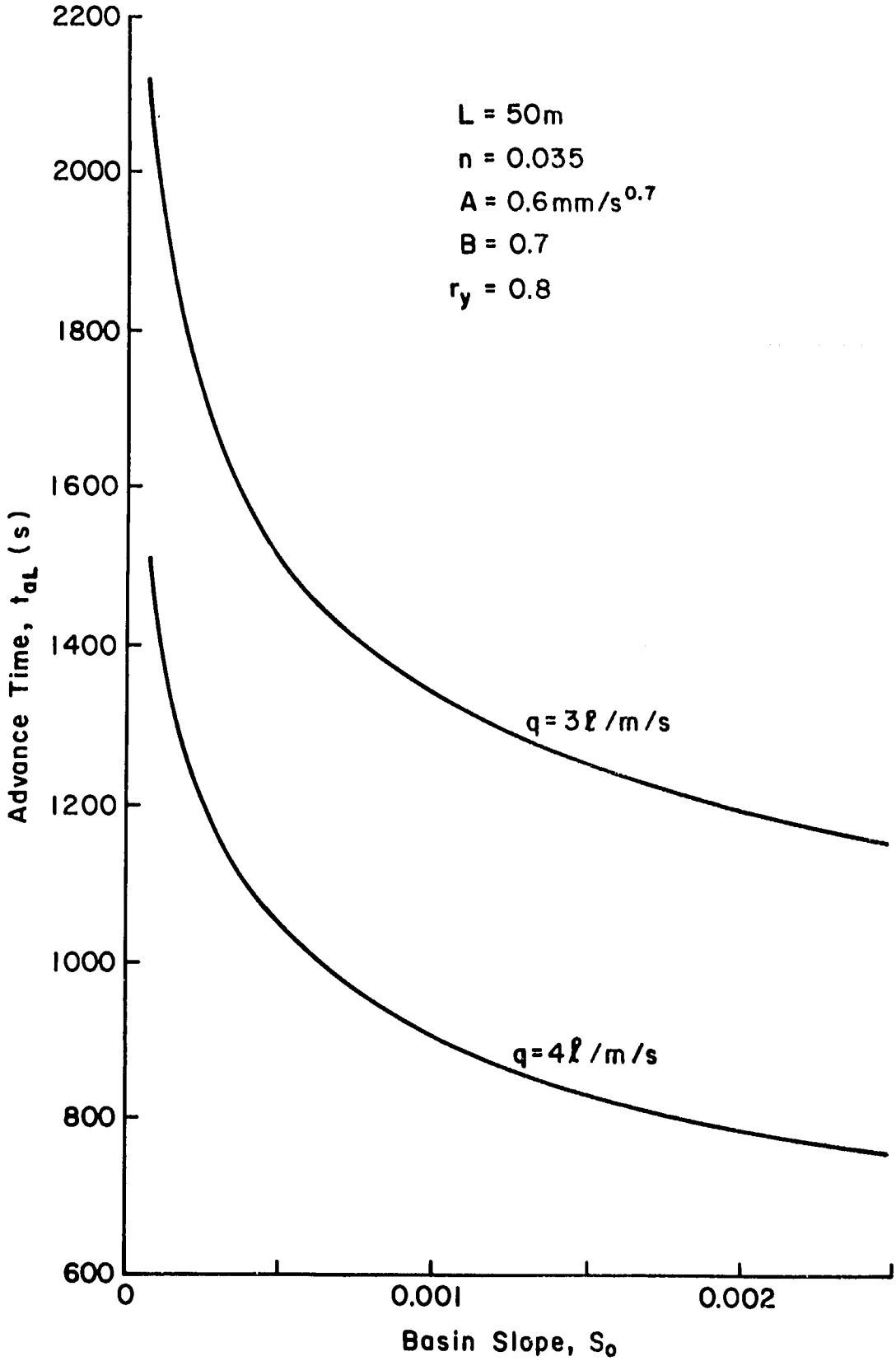


Figure 5-8. Example of advance time as a function of basin slope.

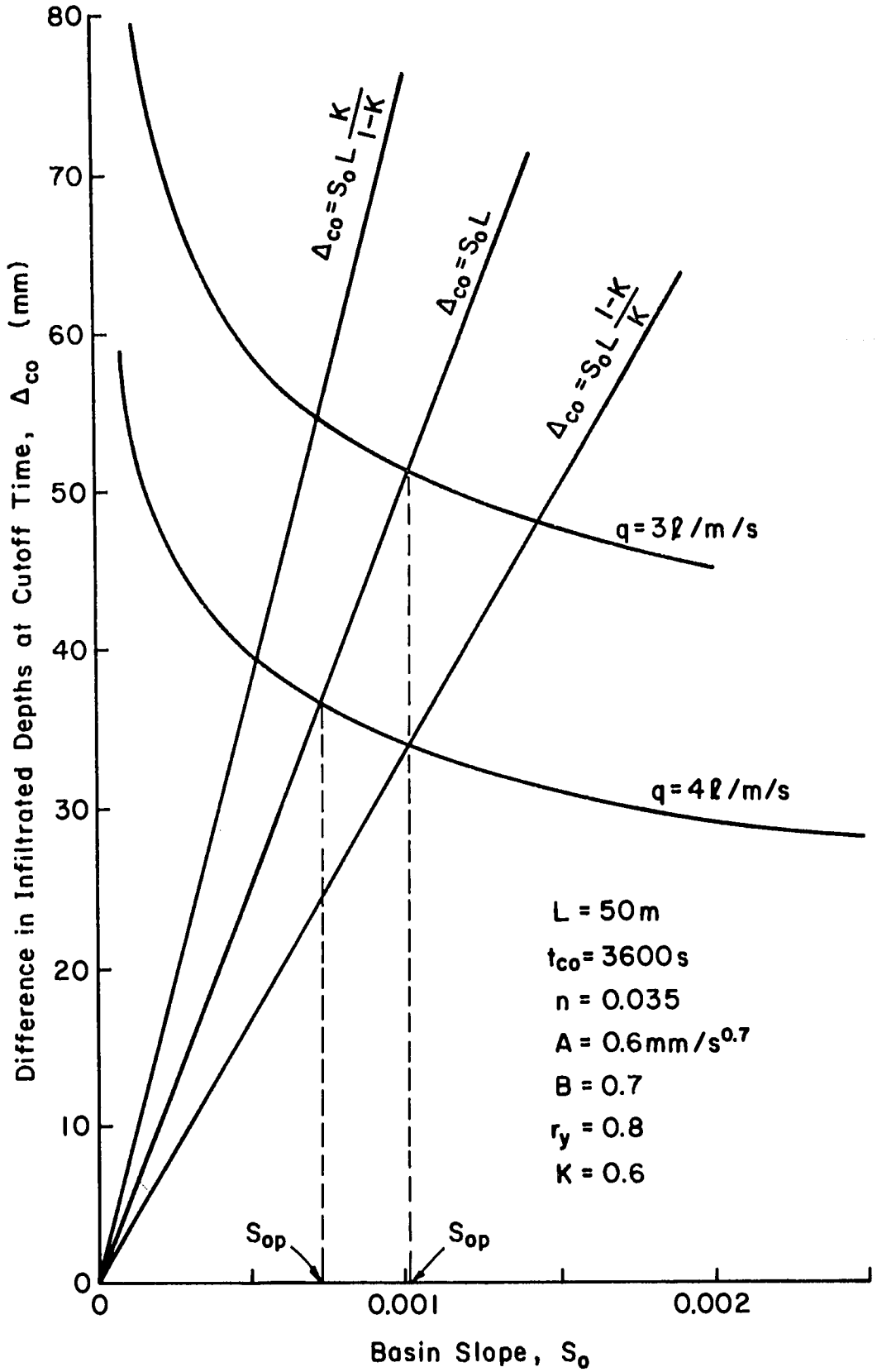


Figure 5-9. Example of difference in infiltrated depth at cutoff time as a function of basin slope.

expression for the water distribution uniformity and is noted by Δf and defined as the maximum difference among the three infiltrated depths, y_L, y_{KL}, and y_o (Fig. 5-3). It is obvious that the smaller Δf, the more uniform is the distribution of water.

An increase in basin slope results in changes in the factors determining Δf: the difference between the infiltrated depths at the upper and lower ends at the cutoff time, Δ_{co}, is decreased, but the term S_oL is increased. Consequently, the pattern of the water distribution changes with a change in the slope.

Four different water distribution patterns can be delineated for various values of the slope. The four distribution patterns, in order starting with the smallest slope, are (Fig. 5-10):

Pattern 1 $\quad y_o > y_{KL} > y_L$ and $\Delta f = y_o - y_L$ (5-30)

Pattern 2 $\quad y_{KL} > y_o > y_L$ and $\Delta f = y_{KL} - y_L$ (5-31)

Pattern 3 $\quad y_{KL} > y_L > y_o$ and $\Delta f = y_{KL} - y_o$ (5-32)

Pattern 4 $\quad y_L > y_{KL} > y_o$ and $\Delta f = y_L - y_o$ (5-33)

For the level basin, or very flat slope, the water distribution is Pattern 1. With a gradual increase in the slope, the patterns change to 2, 3 and 4.

Using Eq. 5-29 to 5-33 and the simplified model (Eqs. 3-46 to 3-48), allows one to calculate Δf for each pattern as a function of S_o and Δ_{co}. Table 5-3 summarizes these values for the four patterns.

The transition between successive patterns is characterized by Δ_{co} as a linear function of S_o with slopes of $LK/(1-K)$, L, and $L(1-K)/K$, respectively, for the three transitions. These transition lines are plotted on Fig. 5-9.

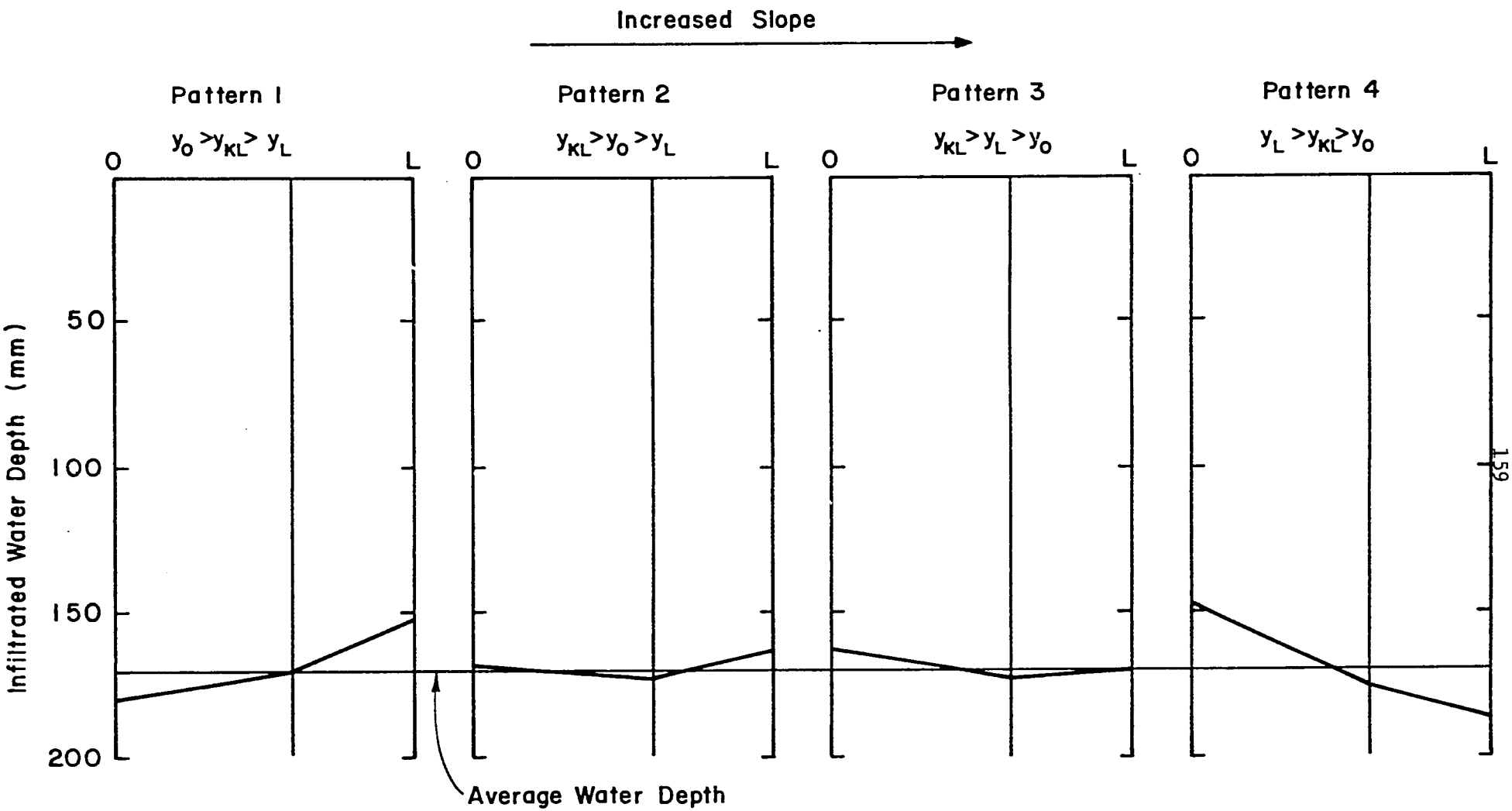


Figure 5-10. Schematic of water distribution patterns for various basin slopes.

Table 5-3. Water distribution patterns and uniformity as related to the basin slope.

Water distribution pattern	Condition	Δf	Transition Δ_{co}	Transition Δf
1	$\Delta_{co} > S_o L \frac{K}{1-K}$	$\Delta_{co} - S_o L$	$S_o L \frac{K}{1-K}$	$S_o L \frac{2K-1}{1-K}$
2	$S_o L \Delta <_{co} < S_o L \frac{K}{1-K}$	$K\Delta_{co} - S_o L(1-K)$	$S_o L$	$S_o L\ (2K-1)$
3	$\frac{1-K}{K} S_o L < \Delta_{co} < S_o L$	$KS_o L - \Delta_{co}(1-K)$	$S_o L \frac{1-K}{K}$	$S_o L \frac{(2K-1)}{K}$
4	$\Delta_{co} < S_o L \frac{1-K}{K}$	$S_o L - \Delta_{co}$		

Analysis of Δf, from Table 5-3, shows that Δf is a minimum at the transition between Patterns 2 and 3, where

$$\Delta f_{min} = S_o L(2K-1) = \Delta_{co}(2K-1) \tag{5-34}$$

and the optimum slope is

$$S_{op} = \frac{\Delta_{co}}{L} \tag{5-35}$$

Fig. 5-11 shows Δf as a function of S_o for two flow rates when the other parameters are held constant.

From Eq. 5-34 it is apparent that $\Delta f < \Delta_{co}$, and if, in fact, a typical value of $K = 0.6$ is assumed, $\Delta f = 0.2\ \Delta_{co}$. In other words, the maximum difference between the final infiltrated depths is only 20% of the difference at the cutoff time.

At the optimal slope, for which Δf is minimal, the water distribution shows

$$y_o = y_L \tag{5-36}$$

and

$$y_{KL} = y_L + \Delta_{co}(2K-1) = y_o + \Delta_{co}(2K-1) \tag{5-37}$$

When the basin slope is less than $\frac{\Delta_{co}}{L}$, the main effect upon uniformity is the difference in opportunity time as a result of the advance time, t_{aL}. When the basin slope is greater than $\frac{\Delta_{co}}{L}$, the main reason for non-uniformity is the difference in ponded water depths, due to the slope.

7. Procedure for the Selection of the Optimum Basin Slope

The selection of the optimum basin slope and the analysis of the effect of slope on the water distribution can not be calculated directly due to the complicated interrelationships among the parameters. A suggested procedure for determining the optimum slope is as follows:

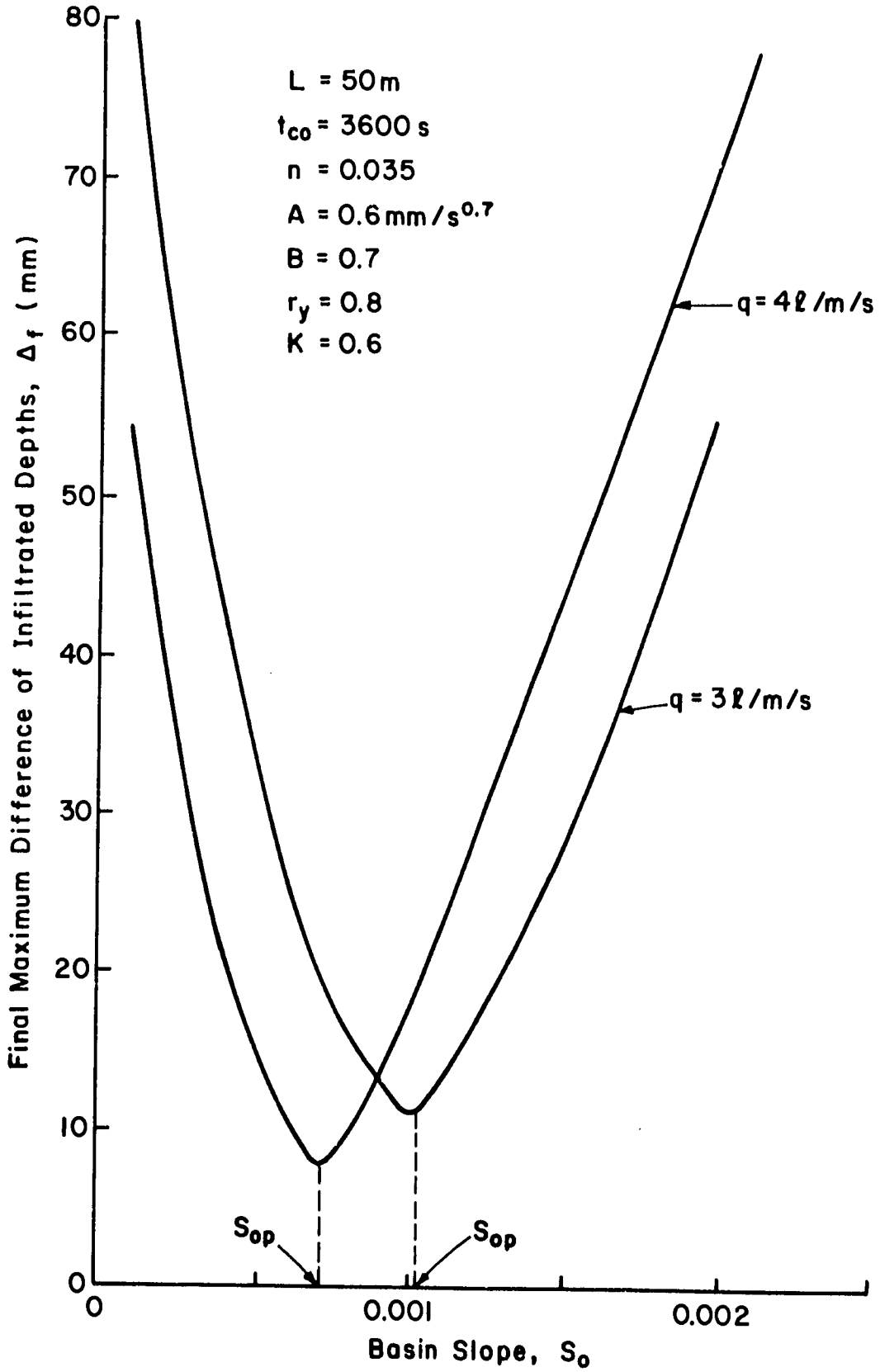


Figure 5-11. Distribution uniformity, characterized by maximum difference of infiltrated depths as a function of basin slope.

a. Calculate the relationship between the slope, S_o, and the time of advance, t_{aL}, for given q, L, n, r_y, A, and B (Eq. 5-28). The inflow stream, q, can be used as a parameter. Plot t_{aL} as function of S_o (Fig. 5-7).

b. Calculate Δ_{co} for various values of t_{aL} (Eq. 5-29) and relate Δ_{co} to S_o through the advance time, t_{aL}. Plot Δ_{co} as function of S_o (Fig. 5-8).

c. On the plot of Δ_{co} and S_o, draw the three transition lines with slopes of LK/(1-K), L, and L(1-K)/K (Fig. 5-9). The intersections of these lines with the Δ_{co} curves give the region of S_o for which different patterns of water distribution appear. In particular, the intersection of the line of slope, L, with the curve, Δ_{co}, gives the slope, $S_o = \frac{\Delta_{co}}{L}$ for which Δf is minimal. This is the optimal slope, at least from the point of view of water distribution uniformity.

d. For further analysis of the effect of the slope, S_o, on the water distribution uniformity, plot the relationship between Δf and S_o, when Δf is calculated as given in Tables 5-3 and 5-4. This enables one to predict the effect of slope on the distribution uniformity.

e. To define the complete water distribution for a particular slope, the ponded water depth at the lower end of the basin at cutoff time, Y_L, is calculated from Eq. 5-20 and the four depths, y_L, y_{KL}, y_{min}, and y_o are calculated from Eqs. 5-21, 5-22, 5-23, and 5-24, respectively.

f. If the cost of land preparation can be determined for various final slopes, the relationship between this cost and the distribution uniformity, Δf, can be defined.

Table 5-4. Calculation of the final maximum difference of infiltrated depths, Δ_f (Example).

S_o		0.0001	0.0002	0.0003	0.0004	0.0005	0.0006	0.0007	0.0008	0.0009	0.0010	0.0011	0.0012	0.0013	0.0014	0.0016	0.0018	0.002	0.0022	0.0024
$S_o L \frac{K}{1-K}$ (mm)		8	15	23	30	38	45	53	60	68	75	83	90	98	105	120	135	150	165	160
$S_o L$ (mm)		5	10	15	20	25	30	35	40	45	50	55	60	65	70	80	90	100	110	120
$S_o L \frac{1-K}{K}$ (mm)		3	7	10	13	17	20	23	27	30	33	37	40	43	47	53	60	67	73	80
q = 4 ℓ/m/s	Δ_{co} (mm)	59	49	45	42	40	38	37	36	35	34	33	32	32	31	30	30	29	28	28
	Pattern	1	1	1	1	1	2	2	3	3	3	4	4	4	4	4	4	4	4	4
	Δ_f (mm)	54	39	30	22	15	11	8	10	13	16	22	28	33	39	50	60	71	82	92
q = 3 ℓ/m/s	Δ_{co} (mm)	86	73	66	62	59	57	55	54	53	52	51	50	49	48	47	46	45		
	Pattern	1	1	1	1	1	1	1	2	2	2	3	3	3	3	4	4	4		
	Δ_f (mm)	81	63	51	42	34	27	20	17	14	11	13	16	19	23	33	44	55		

Example

The basin parameters are: basin length L = 50 m; infiltration equation $z = 0.6t^{0.7}$ (z in mm, t in seconds); Manning coefficient is n = 0.035; surface shape factor is $r_y = 0.8$; infiltrated shape factor r_z = 0.588; the water distribution shape factor is K = 0.6. The effect of the slope on the water distribution and the selected slope is studied for two inlet stream sizes: q = 4ℓ/m/s and q = 3ℓ/m/s. The cutoff time is t_{co} = 3600 seconds so that in the case of q = 4ℓ/m/s a quantity of 14.4 m^3/m is delivered and in the case of q = 3ℓ/m/s a quantity of 10.8 m^3/m is delivered. The advance time, t_{aL}, as a function of S_o is shown in Fig. 5-8 for the two inflow streams. The difference in infiltrated depths at cutoff time, Δ_{co}, is shown in Fig. 5-9 for the two inflow streams. The optimal basin slope is 0.072% when q = 4ℓ/m/s and 0.102% when q = 3ℓ/m/s.

The water distribution patterns for this example are as follows:

Pattern 1 appears when S_o < 0.052% when q = 4ℓ/m/s, and S_o < 0.072% when q = 3ℓ/m/s.

Pattern 2 appears when 0.052% < S_o < 0.072% when q = 4ℓ/m/s, and 0.072% < S_o < 0.102% when q = 3ℓ/m/s.

Pattern 3 appears when 0.072% < S_o < 0.102% when q = 4ℓ/m/s, and 0.102% S_o < 0.143% when q = 3ℓ/m/s.

Pattern 4 appears when S_o > 0.102% when q = 4ℓ/m/s, and S_o > 0.143% when q = 3ℓ/m/s.

The distribution uniformity, Δf, as a function of the basin slope is calculated as shown in Table 5-4 and is shown in Fig. 5-11. Note that in both Table 5-4 and Figs. 5-9 and 5-11 the curves for q = 3ℓ/m/s end at a slope of 0.2%, because the flow rate and cutoff time are not great enough to pond water over the entire basin.

For the optimal slope, the maximum difference between the infiltrated water depth is 8 mm when q = 4ℓ/m/s (average infiltrated depth of 288 mm), while with a near level slope of 0.01% the maximum difference is 54 mm. When q = 3ℓ/m/s (216 mm average application depth), the maximum difference between the infiltrated depths is 11 mm for the optimal slope and 81 mm for a near level slope of 0.01%. In both cases, the optimal slope significantly improves the distribution uniformity as compared to a level basin.

APPENDIX A

FIELD EVALUATION OF THE INFILTRATION EQUATION FOR BASIN IRRIGATION

1. Introduction

In many cases, infiltration equations that have been established by standard methods do not represent the actual infiltration, due to differences between the test conditions and the actual irrigation conditions. For practical design and evaluation of basin irrigation, it is preferable to obtain the infiltration equation as well as the advance time by field tests that resemble the actual conditions of the irrigated fields. A procedure that is similar to that of Christiansen, et al. (1966) is modified for use in basin irrigation. It is based primarily on the measurement of the advancing water front at time intervals. This field measurement is sufficient to define the infiltration equation when the surface water depths are calculated theoretically. However, direct measurements of surface water depths during the advance phase may eliminate the need for surface depth calculations. It should be noted that accurate surface depth measurements are not obtained easily. Therefore, calculated depths may be as accurate as measured depths.

2. Available Techniques for Determining Soil Infiltration Characteristics

All the available techniques for determining soil infiltration characteristics are based on direct field measurement of infiltrated water. The most common techniques are summarized by Kawal[illegible] et al. (1978). The techniques that can most practically be applied to basin irrigation are:

a. Cylinder Infiltrometers (Richards, 1954; Bouwer, 1961). A cylinder of diameter 200-250 mm is installed vertically into the

soil and the portion remaining above the soil surface is used as a water reservoir. The water level in the cylinder is recorded at successive times to give the cumulative infiltrated depth with time. This data is fitted to an infiltration equation such as the Kostiakov equation (Eq. 3-4).

b. Basin Infiltrometer. This is a method similar to the cylinder infiltrometer only instead of a cylinder, a small basin is used. The basin is rectangular or circular with an area of about 1 m^2.

c. Christiansen, et al. (1966) Volume Balance Method. This technique is based on the mass conservation approach which can be expressed by:

$$Qt = dWL + \bar{y}WL \tag{A-1}$$

or equivalently

$$qt = dL + \bar{y}L \tag{A-2}$$

where

L = the final distance of advance (L)

Q = inflow stream size, assumed constant (L^3T^{-1})

W = width of basin or border (L)

d = average depth of water on surface (L)

$\bar{y}$ = average depth of infiltrated water (L)

q = inflow stream size per unit width ($L^3T^{-1}L^{-1}$)

With the infiltration equation of the Kostiakov type:

$$z = At^B \tag{A-3}$$

and an assumed advance function of the form of

$$x = at^b \tag{A-4}$$

the average infiltrated depth can be estimated, as shown by Kiefer (1959), as

$$\bar{y} = bAt^B \left[\frac{1}{b} \left(\frac{1}{b} - \frac{B}{b+1} + \frac{B(B-1)}{2(b+2)} \cdots \right) \right] \tag{A-5}$$

By defining a factor F as

$$F = b(B+1) \left[\frac{1}{b} - \frac{B}{b+1} + \frac{B(B+1)}{2(b+2)} \cdots \right] \cong \frac{2 - (1-B)(1-b)}{1+b} \tag{A-6}$$

Eq. A-5 can be expressed as

$$\bar{y} = At^B \frac{F}{B+1} \tag{A-7}$$

Substituting Eq. A-7 into Eq. A-2 yields:

$$\frac{F}{B+1} At^B = \frac{qt}{L} - d \tag{A-8}$$

The average surface water depth can be measured by field tests or can be evaluated as suggested by Fok and Bishop (1965) as:

$$d = d_o/(1+b) \tag{A-9}$$

where

d_o = water surface depth at the upper end of the basin

In Eq. A-8 the term $\frac{qt}{L} - d$ is calculated from field measurements for various times and is plotted versus t on a log-log scale. A straight line is then fitted with the result that the slope of the line is the value of B and its interception for t = 1 gives the value of $\frac{F}{B+1}$ A. With the known B and b, F can be calculated from Eq. A-6 so that A can be determined.

d. Infiltration Equations from Rate of Advance Data. Evaluation of the infiltration characteristics from field data of advance is suggested by Norum and Gray (1970). With an infiltration equation of the Kostiakov form (Eq. A-3), the rate of advance is given by Philip and Farrell (1964) as

$$\frac{x}{t} = \frac{q}{d} \sum_{n=o}^{\infty} \frac{\left[-At^B \Gamma(1+B)\right]^n}{d^n \Gamma(2+nB)} \tag{A-10}$$

Eq. A-10 is written in nondimensional form as:

$$L_n = \sum_{n=o}^{\infty} \frac{\left[-T_n^B \Gamma(1+B)\right]^n}{\Gamma(2+nB)} \tag{A-11}$$

where

$$L_n = \frac{dx}{qt}$$

$$T_n = t\left(\frac{A}{d}\right)^{1/B} \tag{A-12}$$

With the field measurement of the advance (x and t), the average depth of surface water d, and the known q, Norum and Gray (1970) describe a curve matching procedure that gives the infiltration equation constants A and B. The same method was used to solve the infiltration constants of the Philip infiltration equation given by:

$$z = st^{1/2} + kt \tag{A-13}$$

where s and k are coefficients. This procedure is based on a complete solution of the advance phase and thus may be regarded as accurate and reliable. However, it is more complicated, especially if the advance is not uniform across the basin width.

3. Modified Volume Balance Technique for Basin Irrigation

The Christiansen, et al. (1966) method of evaluating the infiltration equation may exhibit some difficulties in practical cases of basin irrigation as:

a. in many cases of basin irrigation the advancing water front is nonuniform across the basin,

b. the advance of the water front must be expressed as a power function, and

c. the estimation of d by Eq. A-9 is also dependent on the power of the advance equation and its accuracy should be confirmed by field tests and experience.

The modified volume balance technique enables one to evaluate the infiltration characteristics without the explicit advance function, and for a nonuniform advancing water front.

The basin width, W, is divided into p strips, each of width ΔW and in each strip there are n station points spaced a distance Δx. These n points in each strip can be actual points in the field that are used for locating the advancing water front (Fig. A-1). For simplification it is recommended that ΔW and Δx be equal for all the strips.

If subscript i represents a strip, and subscript j represents a station along the strip (Fig. A-1), the advance distance for any strip at time t can be measured and designated as x_i, and the depth of surface water at a station can be designated as y_{ij}.

The total volume of water stored on the surface at time t will be:

$$V_p(t) = \Delta W \, \Delta x \sum_{i=1}^{p} \sum_{j=1}^{r} y_{ij} \tag{A-14}$$

where r is the number of stations along the strip. Obviously some depths, y_{ij}, will be zero when the advance has not yet reached the station (i, j). The surface depths, y_{ij}, can be measured directly during the test.

If, however, the water depths, y_{ij}, cannot be measured, they can be calculated subject to using two assumptions:

a. The surface water depth at the upper basin end is the normal depth, given by the Manning equation:

$$y_{io} = \left(\frac{q \cdot n}{CU \; S_o^{1/2}} \right)^{3/5} \tag{A-15}$$

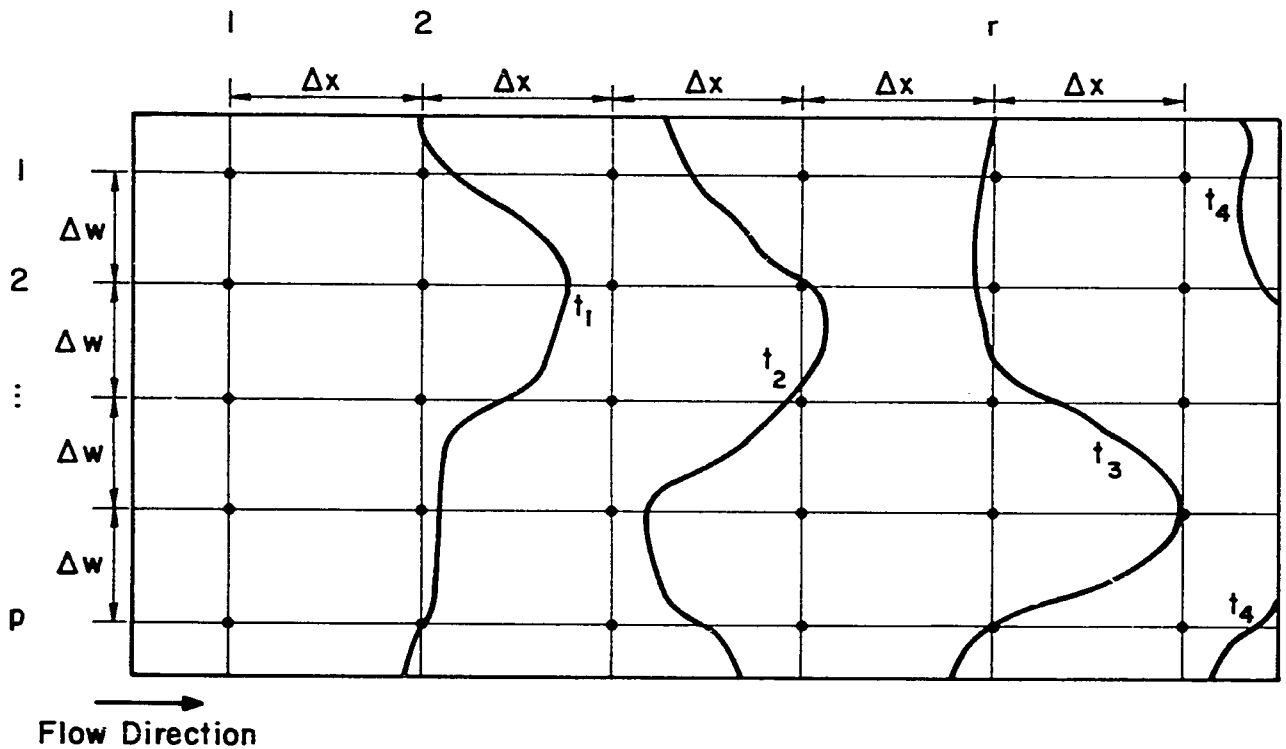


Fig. A-1. Schematic description of grid points for the measurement of the advancing water front.

where

y_{io} is the water depth at the basin upper end (L)

q is the inlet discharge per unit width ($L^3T^{-1}L^{-1}$)

S_o is the basin slope

n is Manning's roughness coefficient

CU is a coefficient depending on the units system (see Eq. 3-39).

b. The surface water depth varies linearly from the normal depth, y_{io}, at the basin upper end to a depth of 0.7 y_{io} at the water front located at a distance x_i, and then sharply drops to 0.

The volume of surface water stored on a strip will be

$$[V_p(t)]_i = 0.85\ \Delta W\ x_i y_{io} \tag{A-16}$$

so the total volume stored on the surface will be

$$V_p(t) = 0.85\ \Delta W \sum_{i=1}^{p} x_i y_{io} \tag{A-17}$$

With the surface volume known as a function of time (Eq. A-14, or Eq. A-16), the total infiltrated volume of water is given by:

$$V(t) = Qt - V_p(t) \tag{A-18}$$

from which the infiltrated volume during advance can be expressed as a function of t.

The infiltrated volume during advance can also be expressed by:

$$V(t) = \Delta W \sum_{i=1}^{p} KAt^B\ x_i \tag{A-19}$$

where K is a shape factor, $0.5 < K < 1.0$.

The determination of K is discussed in Appendix 3-B. For any given time, t is constant for all $i = 1, 2 \ldots p$ and Eq. A-19 becomes:

$$V(t) = \Delta W\ \ KAt^B \sum_{i=1}^{p} x_i \tag{A-20}$$

or rearranging

$$\frac{V(t)}{\Delta W \sum_{i=1}^{p} x_i} = KAt^B \tag{A-21}$$

With $V(t)$ and $\sum_{i=1}^{p} x_i$ known for various values of t, the LHS of Eq. A-21 can be plotted as a function of t on a log-log scale. When a straight line is fitted through the points, the slope of the line is B and the interception of the line with t = 1 gives the value KA.

If the water front advances uniformly across the basin width,

$$\Delta W \sum_{i=1}^{p} x_i = Wx(t) \tag{A-22}$$

and Eq. A-21 becomes

$$\frac{V(t)}{Wx(t)} = KAt^B \tag{A-23}$$

Both Eqs. A-8 and A-23 are expressions for the average depth of water applied, thus equating these equations shows that

$$K = \frac{F}{B + 1} \tag{A-24}$$

which is the result given by Strelkoff (1977) (see Appendix 3-A), if F = 1.

The procedure for determining the infiltration characteristics for an irrigation basin can be summarized as follows:

a. Divide the basin into equal width strips and measure the advance distance in each strip for various times.

b. Measure the surface depths at equally spaced intervals along the strips for the same times as used in a, or estimate the surface storage from Eq. A-17.

c. Calculate the volume of water infiltrated for the various times (Eq. A-18).

d. Calculate the average depth infiltrated (LHS Eq. A-21) and plot as a function of t on log-log graph paper.

e. Fit a straight line to the plotted data. The slope of the line is B and the intercept at t = 1 is KA.

f. Estimate K from Fig. 3-15 and calculate A.

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