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Simple approach to surface irrigation design: Theory [(1)]

A.J. Clemmens [(A)]

[(A)] U.S. Arid-Land Agricultural Research Center, 21881 North Cardon Lane, Maricopa, AZ 85238, USA.

Abstract

The movement of water over the soil surface under surface irrigation has been studied extensively over the last century. However, irrigators are still faced with significant challenges in making surface irrigation systems efficient. In the past, each surface irrigation method was treated differently because of differences in the simplicity with which different phases of the irrigation could be described. This has tended to make surface irrigation analysis and design appear disjointed. In this paper, the same basic procedures are applied to the design of various surface systems, deviating where needed to make the procedures both straightforward and sufficiently accurate. The basis for these designs is the ability to predict advance, recession, the distribution of infiltrated water, and the performance for a given set of conditions. Conservation of mass is the main concept, with empirical approximations used where needed. This paper presents the relevant equations. A companion paper provides solution procedures for hand calculation and discussion of how to apply these in a design setting. Spreadsheets to perform these calculations are available on the Sakia.org e-journal web site http://www.sakia.org/ejlw

Keywords

surface irrigation, design, management, application efficiency, irrigation performance

1. Introduction

Many surface irrigation systems are ineffective and inefficient. This can be caused by physical constraints (e.g., steep land slopes, shallow soils, poor water supplies, etc.), by poor design and layout, or by improper operation and management. A thorough discussion of the constraints and limitations of surface irrigation systems is beyond the scope of this paper. For more details, see Walker and Skogerboe (1987), Clemmens and Dedrick (1994), or Burt *et al.* (2000). One advantage of surface irrigation over pressurized irrigation methods is that it often does not require a good, reliable water supply. It can be adapted to different rates of flow, flows that vary randomly, and flows with poor water quality (sediment, debris, etc.). Efforts in surface irrigation research and extension have focused on methods for providing better water control – control over flow rate or control over volume applied. These generally focus on how the system is operated. Of equal importance is field design and layout. Good operation cannot make up for a poor field design. However, when surface irrigation systems are properly designed and more modern operating procedures are used, irrigation efficiencies and uniformities can be high (Kennedy, 1994, p 166).

Surface irrigation methods can be categorized according to how they function hydraulically. Distinctions can be made by advance and recession curves, which describe the time when the advancing stream reaches particular locations and the time when standing water no longer occurs there. This hydraulic comparison assumes that water enters the field or irrigation set along one end and flows to the other end uniformly across the set width. The following categories of surface irrigation are considered: sloping furrows, border strips, level basin, and level furrows (Table 1).

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The main differences in hydraulic performance among these categories are related to the magnitude and pattern of the inflow rate, the general shape of the recession curves, and runoff hydrographs. Unfortunately, past methods for design have used widely different approaches and assumptions, making the design process somewhat confusing.

Table 1:
Categories of Surface Irrigation.

Method	Control of lateral flow	Slope	Inflow control	End conditions
Sloping furrow	Furrows	Steep or Low-gradient Either can have cross slope	To individual furrows	Open, Blocked, or Group of furrows blocked
Border strip	Flat planted or Corrugations	Steep or Low-gradient Either can have cross slope	Distributed across upper end	Open, Blocked, or Partially blocked
Level basin/ Level furrows	Flat planted Furrowed or bedded	Zero in all directions	Can be point inflow	Blocked If furrowed, all interconnected
	Furrows	Zero in direction of run, can have cross slope	To individual furrows	Blocked, or Group of furrows blocked

The purpose of this paper is to present simple, consistent calculation procedures that can be used in the design of *modern* surface irrigation systems, which can be easily calculated and implemented in a spreadsheet. Here, *modern* implies a reasonable control over the water supply. Design of rice paddies and methods such as contour levee, contour ditch, wild flooding, etc. are not discussed. The equations and calculation procedures are based on continuity, but include some empirical expressions for convenience. These calculations are used to determine advance and application times and the resulting distribution of infiltrated water for a specific set of conditions. Design requires a trial and error process to determine appropriate field dimensions and operational recommendations. The examples in the companion paper lay the groundwork for extending these simple equations to design.

2. Design considerations

2.1 Design objectives

The amount of water to be supplied during an irrigation event, referred to as the target or required depth of application, is a major design consideration. Surface irrigation systems have a narrow range of target depths for which they are reasonably efficient and uniform. The irrigator must adjust irrigation practices (typically flow rate and application time) to account for changing field conditions (infiltration and roughness). Irrigators often develop rules of operation that they use to make adjustments. A poor field design will make these judgments difficult for the operator. A good design will provide guidance on system operation.

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Surface irrigation systems are most applicable on mild to level slopes. On steeper slopes, erosion can become excessive and the range of operating conditions can be narrow (e.g., narrow range of target depths).

The design objectives are typically stated in terms of achieving some desired application efficiency, *AE*. This efficiency is called potential application efficiency, *PAE*, here to distinguish it from a field measured *AE*. Design is typically based on supplying the target depth of water everywhere in the field. The PAE_{min} is the application efficiency when the minimum depth infiltrated just equals the target depth.

$$PAE_{\min} = \frac{d_{\min}}{d_a} = \frac{d_{req}}{d_a} \quad [1]$$

where d_{req} is the required depth, d_{min} is the minimum infiltrated depth, and d_a is the depth of applied irrigation water, in this case, the depth applied that results in the minimum depth just equal to the required depth. In practice, some underirrigation is usually allowed and operation is based on satisfying the low-quarter depth. Since design does not take into account all of the variability which exists within the field, design is based herein on satisfying the minimum depth, with the expectation that when operated the low quarter depth would be satisfied.

Surface irrigation design requires the estimation of parameters that define the infiltration of water into the soil and the resistance to water movement caused by the soil surface and vegetation. These are key factors in the design.

2.2 Equations for flow resistance

Resistance to flow is usually described by the Manning equation, which relates the flow rate, *Q*, to the flow area, *A*, the hydraulic radius, *R* (area over wetted perimeter), the friction slope, S_f, and the Manning roughness coefficient, *n*. A units coefficient, C_u, is required to make this dimensionally homogeneous, where C_u = 1 $m^{1/2}$ /s.

$$Q = \frac{A R^{2/3} S_f^{1/2}}{n/C_u} \quad [2]$$

Expressed in terms of flow rate per unit width, *q*, for border strips and basins, this becomes

$$q = \frac{y^{5/3} S_f^{1/2}}{n / C_u} \quad [3]$$

where y is the flow depth and $q = Q/W$, where *W* is the basin or border width. The base units in Equation 2 and 3 are meters and seconds.

2.3 Equations for infiltration

Infiltration is one of the most important factors affecting the design and performance of surface irrigation systems. Unfortunately, estimating infiltration conditions is one of the most difficult things to do in the field. Irrigation engineers have tended to use the Kostiakov Equation, or a variation thereof, defined by

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$$d(\tau) = k\ \tau^{a} \qquad i(\tau) = ak\ \tau^{a-1} \tag{4}$$

or in modified form

$$d(\tau) = c + k\,\tau^{a} + b\tau \qquad i(\tau) = ak\,\tau^{a-1} + b \tag{5}$$

where *d* is the infiltrated depth; *i* is the infiltration rate; τ is the infiltration opportunity time; and *a*, *b*, *c*, and *k* are empirical coefficients.

An alternative to the Modified Kostiakov equation is the Kostiakov branch function. The first branch uses Equation 5 (with *b*=0) for short times, and switches to a constant infiltration rate when the infiltration rate equals *b*. For soils that reach a nearly constant, final infiltration rate during the irrigation, design and evaluation can be greatly simplified with use of this equation. The Kostiakov branch function equations are

$$\begin{aligned} d(\tau) &= c + k\,\tau^{a} \qquad i(\tau) = ak\,\tau^{a-1} \qquad for\ \tau \le \tau_B \\ &= c_2 + b\tau \qquad\quad = b \qquad\quad for\ \tau \ge \tau_B \end{aligned} \tag{6}$$

where τ_B is the time when the infiltration rates for the two branches match.

Infiltration in furrows can be significantly more complicated than in flat borders or basins due to the two-dimensional nature of the furrow cross section. Infiltration in borders and basins is generally considered to be one dimensional-downward. Infiltration in furrows can be influenced by the wetted width of the stream and lateral flow into the furrow bed. For design, it is generally sufficient to express infiltration as infiltrated volume per unit length per unit width. Then, calculated infiltration is not influenced by actual differences in wetted width over the length of run. Infiltration could be expressed as a function of furrow spacing, the wetted width at the upstream end under normal depth, or some related depth. Then, for example, infiltration could change with inflow rate. Further discussion is beyond the scope of this paper.

3. Design approach

3.1 Opportunity time criteria

The amount of water to be supplied during an irrigation event, referred to as the target or required depth of application, d_{req}, is a major design consideration. Surface irrigation systems have a narrow range of target depths for which they are reasonably efficient and uniform. Design approaches are often based on assuming that one end of the field or the other will receive the least infiltrated depth. Then, the inflow and application time are adjusted such that the required depth is infiltrated at that location. The time to infiltrate the required depth, τ_{req}, becomes an important design parameter. Typically, it has more influence on the design than the constants in an infiltration formula themselves.

The infiltration opportunity time at any location, *x*, along the length-of-run, $\tau_{opp}(x)$, is defined as the time between advance, $t_A(x)$, and recession, $t_R(x)$ or

$$\tau_{opp}(x) = t_R(x) - t_A(x) \tag{7}$$

At the head end of the field (x = 0), the opportunity time is equal to the recession time, or

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$$\tau_{opp}(0) = t_R(0) = t_{co} + t_{lag} \quad [8]$$

where t_{co} is the time of cutoff or application time and t_{lag} is the recession lag time, or the time required for the water depth at the upstream end to drop to zero after cutoff (Figure 1). Design based on meeting the requirement at the upstream end does not require advance and recession curves to be computed. However, an estimate of PAE_{min} is required, along with a method for estimating recession lag time (USDA, 1974). For short border-strips and steep slopes, the minimum depth infiltrated can be at the head end of the strip. However, numerous runs with the BORDER design program (Strelkoff *et al.* 1996) indicated that for most situations near the highest potential efficiency, the minimum depth was at the downstream end. Even when the minimum was at the upstream end, the extent of low quarter depths was split between the upper and lower ends. Thus design based on the downstream end should give more consistent results over the range of typical design conditions. Design based on satisfying the requirement at the upstream end is no longer recommended, except under ponded conditions, discussed later.

With the minimum depth at the downstream end of the field, furthest from the water source ($x = L$), advance and recession curves must be computed. Estimating the appropriate inflow rate and time required to achieve τ_{req} at the downstream end is more difficult. Specifically (Figure 1),

$$t_R(L) = t_A(L) + \tau_{req} \quad [9]$$

Precise solutions of advance and recession are possible with solution of the Saint Venant equations of continuity and momentum. However, this approach requires a numerical solution, done on a case by case basis. It does not directly produce general equations for advance and recession. The time of cutoff that will produce the target depth at the downstream end is

$$t_{co} = t_A(L) + \tau_{req} - [t_R(L) - t_R(0) + t_{lag}] \quad [10]$$

where the term in brackets is the time between cutoff and recession at the downstream end (Figure 1). A more or less uniform approach can be used for computing advance curves for the various surface irrigation methods. Recession curves are more difficult to estimate and thus different approaches are used to solve the last term in Equation 10 for the different methods. These approaches range from assuming the term can be neglected, to the use of empirical equations. In this paper, calculation procedures are provided for advance, cutoff time, recession, infiltrated water distribution, and runoff volume that will satisfy the minimum depth at the downstream end, which can subsequently be used in the design process.

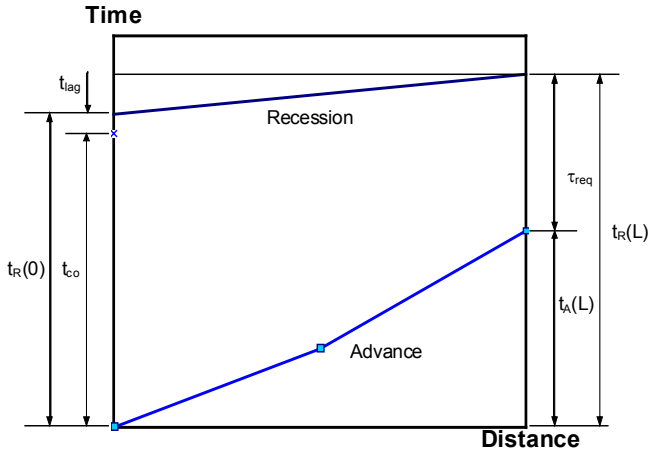

Figure 1:
Advance and recession curves and definitions for design satisfying the target depth at the downstream end.

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3.2 Assumed-surface volume method for advance

All design methods must use procedures that satisfy a volume balance, regardless of how sophisticated they are. In this paper, assumptions are made regarding the surface volume in order to use a volume balance to determine an advance curve. This has advantages over strictly empirical equations since the assumptions regarding the surface volume can be verified with field observations or from computer simulation.

During advance, the cumulative infiltrated volume at any time is equal to the difference between the accumulated inflow volume and the surface storage volume. This volume balance relationship may be expressed as:

$$V_{in}(t) = V_y(t) + V_z(t) \quad [11]$$

where $V_{in}(t)$ is the inflow volume at time t, $V_y(t)$ is the volume in surface storage at time t, and $V_z(t)$ is the infiltrated volume at time t. Using this relationship to determine advance time, t, to distance x requires calculation of surface and subsurface volumes. Typically, Equation 11 is put in the following form

$$Q_{in}t = \sigma_y A_0(t)x + \sigma_z Wz(t)x \quad [12]$$

where $A_o(t)$ is the cross-sectional flow area at the inlet at time t, σ_y is the surface storage shape factor, W is the width, and σ_z is the subsurface shape factor (all units are in meters and seconds). The surface shape factor is the ratio between the average cross-sectional flow area and that at the head of the field. For furrows, it is typically between 0.70 and 0.80.

The subsurface shape factor is the ratio between the average infiltrated cross-sectional area (infiltrated depth times width), and the infiltrated cross-sectional area (depth times width) at the head of the field. When infiltration is defined by Equation 5 or 6, the subsurface shape factor in Equation 12 is difficult to determine. It is easier to rewrite the subsurface volume in the following form (adapted from ASAE 1991)

$$V_z = W(c + \sigma_{z1}\, k\, t_x^a + \frac{h}{1+h} b\, t_x)x \quad [13]$$

where h is the exponent in the advance equation

$$t = s\, x^h \quad [14]$$

where s is a constant. Then σ_{z1} can be found from (ASAE 1991)

$$\sigma_{z1} = \frac{h + a(h - 1) + 1}{(1 + a)(1 + h)} \quad [15]$$

Determining an advance curve requires knowledge of Q_{in}, A_0, σ_y, the infiltration constants (c, k, a, and b), and the width. Defining this curve is an iterative process since the advance exponent h and advance times are interrelated and must be solved for simultaneously.

A common approach has been to use the advance time at two locations to determine the advance exponent in Equation 14, typically the field length and ½ the field length. From these two time-distance pairs, the advance exponent can be computed from

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$$h = \frac{\log(t_{L/2} / t_L)}{\log(1/2)} \quad [16]$$

If the above system of equations are applied for a given situation (with inflow rate, length, width, infiltration and roughness known) at these two advance time-distance pairs, with Equation 13 replacing the second term in the right hand side of Equation 12, there are eight unknowns; σ_y, $A_0(t_L)$, $A_0(t_{L/2})$, σ_{z1}, t_L, $t_{L/2}$, h, and s, with five equations; Equations 12 and 14 at two distances, and Equation 15. Solution of these equations requires external estimates of σ_y, $A_0(t_L)$, and $A_0(t_{L/2})$. The main problem is that the A_0 and σ_y are not known in general. For steep slopes, the surface volume is typically a small fraction of the total inflow volume, such that rough estimates of these parameters give reasonable advance predictions; where typically, A_0 is assumed equal to the flow area at normal depth and σ_y is given a value of 0.7. For milder slopes, the surface volume is often a large portion of the inflow volume, even at the time of cutoff. Also, A_0 changes continuously during the irrigation event, and in some cases never reaches normal depth. Similarly, σ_y can vary. The advance calculation method for the various surface irrigation methods presented herein differ in their assumptions regarding σ_y and $A_0(t)$. In border strips, $A_0 = y_0 W$.

Monserrat and Barragan (1998) suggest the use of unsteady-flow simulation results to determine values for the average surface water area, $\sigma_y A_0$. They plot values of average surface depth and surface shape factors as a function of dimensionless advance distance for a number of conditions in borders. It has proven useful to compute simulations over a wide range of conditions so that the nature of the problem can be seen and the variation in various shape factors determined. Dimensionless variables have been used to reduce the number of interrelated parameters so that meaningful results can be displayed on a limited number of graphs. Full design solutions have been developed for flat-planted level basins and open-ended sloping border strips, and are available in the form of computer programs BASIN (Clemmens *et al.* 1995) and BORDER (Strelkoff *et al.* 1996). Both are limited in the type of infiltration function and range of conditions considered. However, this does not satisfy our intent here to have simple, spreadsheet-based design methods. The use of simulation for design will be discussed in subsequent papers.

3.3 Performance

The potential application efficiency is computed as the required volume (required depth times furrow spacing times furrow length) divided by the applied volume (inflow rate times cutoff time). No adjustment is needed for under irrigation since this approach assumes the entire field is adequately irrigated. The volume of runoff can be estimated by computing a distribution of infiltrated depths (based on recession minus advance times), the associated infiltrated volume, and subtracting this volume from the inflow volume. Better estimates of these volumes can be made by computing advance, recession, and infiltration at a series of points, i.e. numerical integration. Eight points has proven to be satisfactory in most cases. The deep percolation volume is the infiltrated volume minus the required volume, since it is assumed that all points receive at least the required depth.

The following procedures assume that length, slope, and inflow rate are known. The procedure then determines the minimum application or inflow time required to meet or exceed the required depth everywhere. This provides an estimate of PAE_{min}. Design for a specific set of field conditions (i.e., infiltration, roughness, and required infiltration depth) requires repeated application of these procedures to determine an appropriate furrow length, slope, and inflow rate by trial and error. More systematic application of these relationships is discussed in a companion paper.

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4. Sloping furrow irrigation

4.1 Assumptions

With sloping furrow irrigation, water advance must be sufficiently fast so that the downstream end will receive adequate water while the upstream end is not excessively over irrigated. However advance that is too rapid can result in a large percentage of the applied water running off the field, unless inflow after completion of advance is reduced, for example with a cutback, surge, or cablegation system, or the runoff is collected for reuse.

Furrow slopes in areas of high rainfall should be great enough to allow adequate drainage (>0.03%), yet not so great as to cause significant erosion. The following guidelines are taken from USDA (1984). For erodible soils (e.g., silty soils), the maximum furrow slope should be limited to $60/(P_{30})^{1.3}$, where P_{30} is the 30-minute rainfall in mm for a 2-year frequency. This limit can be exceeded by about 25% for less erodible soils (e.g., sandy and clayey soils). Further, erosion can be limited by placing a limit on the irrigation stream size. The maximum flow velocity should be limited to 8 and 13 m/min for erodible and non-erodible soils, respectively. The relationship between velocity and flow rate can be obtained from Equation 2, since velocity is flow rate, Q, divided by flow area, A.

The furrow cross-sectional area and wetted perimeter must be specified as a function of flow depth (i.e., normal depth). Any function can be useful provided that it properly describes the cross section. Trapezoidal and power-function shapes are commonly used. Then the normal depth for a given discharge can be found by trial and error from Equation 2.

This design approach assumes that the wetted width does not vary over the length of the furrow. That is, the infiltrated volume over a unit length of furrow is dependent only on the infiltration opportunity time and not on the depth of flow or wetted width. For heavy soils with significant lateral sorption, the wetted width from adjacent furrows overlap, making the wetted width per furrow essentially equal to the furrow spacing. For coarse textured soils, a significant reduction in infiltration along the furrow can result from the reduction in wetted width due to a reduction in flow depth as the flow rate gets smaller with distance from the head end. For this latter case, field design should not attempt to minimize runoff since this would drastically reduce furrow flow rates toward the downstream end. Other measures (e.g., return flow) are needed to improve performance in these cases.

4.2 Advance

For sloping furrows, it is often assumed that the flow depth at the upstream end is at normal depth for the entire period of inflow and the friction slope equals the bottom slope, $S_f = S_0$. These assumptions are particularly appropriate for slopes greater than 0.5%. The flow area at the upstream end, A_0, is then computed from the cross-section definition. Since this does not vary during inflow, the surface volume can be computed as a function of distance only.

The assumed-surface-volume method provides a relationship between advance distance x and advance time t_x from Equation 12, where W is the furrow spacing, under the assumption that Q_{in}, W, A_0, σ_z, and σ_y are fixed. Fortunately for most situations of interest, σ_y does not vary significantly, generally falling between 0.7 and 0.8. However, σ_z can vary considerably and should be adjusted for the particular case of interest. For a power infiltration function, it can be estimated from Equation 15. Estimation of the exponent, h, in Equation 14 requires that at least two points on the advance curve be estimated.

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4.3 Recession

For steeply sloping furrows, recession along the entire furrow length is often assumed to occur immediately after cutoff. This is a reasonable assumption for slopes above 0.5%. In this case, the last term (in brackets) in Equation 10 is zero. For flatter slopes, the recession at the downstream end will take some time, resulting in a greater infiltration depth there and underprediction of *PAE*. An adjustment to the recession curve can be made by subtracting the volume of water on the surface at the time of cutoff from the total applied volume. Then the application time is found from

$$t_{co} = t_A(L) + \tau_{req} - \left(\frac{V_y(t_{co})}{Q_{in}}\right) \quad [17]$$

For calculation purposes, the volume in surface storage at the completion of advance is used to estimate the surface volume at cutoff. Experience has shown that this adjustment is reasonable in many cases (Clemmens *et al.* 1998).

The recession time at the upstream end is t_{co} and the infiltration opportunity time at the downstream end is assumed equal to τ_{req}, with recession at the downstream end from Equation 9. For intermediate points, the recession curve is assumed to be a straight line between the end points. Figure 2 shows advance and recession curves generated with the above equations and from simulation with SRFR (Strelkoff et al. 1990). Specifics include L = 400m, S_0 = 0.002, k = 30.1 mm/hr[a], a = 0.35, n = 0.05, d_{req} = 80 mm, furrow spacing = 1 m, Q_f = 1.0 l/s, and trapezoidal furrow shape with bottom width = 100 mm and side slopes = 2:1, horizontal to vertical. Clearly the adjustment to the recession time from Equation 17 is reasonable in this case.

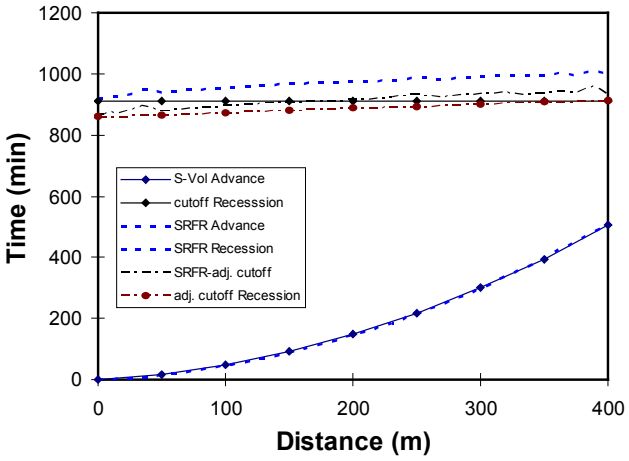

Figure 2:
Comparison of furrow advance (from surface volume method and from SRFR simulation) and recession (from cutoff time, from adjusted cutoff based on furrow volume at time of cutoff, and from simulation with SRFR with original and adjusted cutoff times) curves.

4.4 Sloping furrows with cutback

The efficiency of furrow irrigation systems can often be improved by reducing the inflow rate after water has advanced to the end of the field. A high initial flow rate can provide rapid advance, and thus more uniform opportunity time, while cutting back the stream will reduce the amount of runoff. If the cutback stream is too small to keep up with infiltration, recession can occur at the downstream end and move back up the field. Rather than cutting off at the completion of advance, cutoff is typically delayed until infiltration is somewhat reduced. This will also help assure that the downstream end receives sufficient flow depth and wetted perimeter.

A common practice is to cut back to 50% of the inflow. Dividing the cutback inflow rate by the wetted field area gives the average infiltration rate that matches the cutback inflow

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$$\bar{i}_{CB} \le \frac{Q_{CB}}{WL} \quad [18]$$

The average infiltration rate at any time after completion of advance can be computed from numerical integration over the distance (e.g., the 8 intervals used above). A direct analytical solution for average infiltration rate is not possible, but a numerical approximation can be found by integrating infiltration rate over distance. Infiltration at any point and any time is a function of the infiltration time, or the current time minus advance time. The advance time is a function of x^h, from Equation 14. If this term is replaced with a truncated series expansion, with higher order terms removed (i.e. $x^h = 1 + h(x-1)$), an analytical expression for the infiltration rate, averaged over the field length, can be found, namely

$$\bar{i}_{CB} \approx b + \frac{akt_{CB}^{(a-1)}}{ah}\left[\left(\left\{\frac{t_{CB}}{t_L}\right\} - 1 + h\right)^a - \left(\left\{\frac{t_{CB}}{t_L}\right\} - 1\right)^a\right] \quad [19]$$

where t_{CB} is the time of cutback. The cutback time to achieve the necessary average infiltration rate can be determined from Equation 19 by trial and error.

This is a very conservative estimate of cutback time since the reduction in flow results in less surface storage on the field, and this change in surface storage can contribute to infiltration. A conservative estimate of the correction in cutoff time can be found by dividing the change in surface volume by the cutback flow rate, which is related to the distance averaged infiltration rate through Equation 18. The adjusted cutback time is simply

$$t_{CB\,adj} = t_{CB} - \frac{V_y(Q_{in}) - V_y(Q_{CB})}{Q_{CB}} \quad [20]$$

where the surface volume after completion of advance is now a function only of flow rate.

The cutoff time is computed according to Equation 17, but with $V_y(t_{co})$ replaced with $V_y(Q_{CB})$ and Q_{in} replaced with Q_{CB}. The cutoff time is actually slightly longer, since the ratio of volume to flow rate (last term in Equation 17) is larger. For some soils, the reduction in wetted perimeter caused by cutback might require an increase in total application time.

4.5 Sloping furrows with runoff reuse

This section discusses the design of runoff reuse systems that recycle water at a constant rate. The intent is to provide the same flow rate to all furrows during the entire duration. Assuming that the reservoir is empty at the beginning and end, the *first set* can be split into a portion irrigated with the supply inflow only at the beginning and a portion irrigated with the reuse portion only at the end. The second part of the *first set* is actually irrigated after the main supply flow is turned off (i.e., it is moved to the end of the irrigation and the last set in the field). This scheme was first proposed by Stringham and Hamad (1975).

The number of irrigation sets, N, the individual furrow flow rate, Q_f, and relationship between the supply inflow rate, Q_s, and the return flow or pump flow rate, Q_p, should satisfy the following;

$$F = N\left(\frac{Q_s}{Q_f} + \frac{Q_p}{Q_f}\right) = N\left(n_s + n_p\right) \quad [21]$$

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where F is the total number of furrows in the field, N is the number of sets, and n_s and n_p are the number of furrows in one set irrigated with the supply and pumpback flows, respectively. F, N, n_s, and n_p are integers.

The gain in efficiency for a runoff reuse system can be computed from

$$PAE_{RU} = \frac{PAE}{1 - RU} \quad [22]$$

where RU is the fraction of the applied water that is reused, or

$$RU = \frac{Q_p}{Q_S + Q_P} \quad [23]$$

The volume of water pumped or recycled, V_{RC}, relative to the inflow volume can be estimated from the reused runoff fraction by

$$V_{RC} = V_{in} \frac{RU}{1 - RU} \quad [24]$$

The selection of return flow rate must assure that there is sufficient runoff volume, which is satisfied when

$$RO \geq RU \quad \text{or} \quad Q_P \leq \left(\frac{RO}{1 - RO} \right) Q_S \quad [25]$$

where RO is the runoff fraction (i.e., fraction of inflow that runs off for an *average* furrow). If this inequality isn't satisfied, there may not be enough water in the reservoir to irrigate the last set. If Q_p is too low, then only a small fraction of the runoff can be recovered. The unrecovered runoff is $V_{in}(RO\text{-}RU)$.

The reservoir volume, V_R, needed for operation of such systems is the amount of water required in the reservoir to irrigate the last set with only return flow

$$V_R \geq Q_P (1 - RO) t_{co} \leq Q_s RO t_{co} \quad [26]$$

where t_{co} is the application time for each set.

5. Border-strip irrigation

5.1 Assumptions

With border-strip irrigation, flow resistance from vegetation causes a significant amount of water to be in surface storage. For efficient irrigation, this often requires that the inflow be cut off prior to the completion of advance. The recession in border strips is also much slower than furrows due to the vegetative resistance and the geometry. If resistance changes substantially during the growing of a crop, different flow rates are often required to maintain efficient irrigation over the season. Thus design needs to provide a layout such that inflow rate and time can be adjusted within reason to provide satisfactory performance.

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The Soil Conservation Service (USDA 1974) provided the following recommendations. The maximum recommended inflow rate to limit erosion on non-sod forming crops, such as alfalfa and small grain is found from

$$q_{in_{\max}} = 0.00018\, S_0^{-0.75} \quad [27]$$

where S_0 is in m/m and q_{in} is in m^2/s. For sod forming crops, twice this value can be used. A minimum inflow rate has also been suggested so that the water depth will be sufficient to spread laterally

$$q_{in_{\min}} = \frac{0.000006\, L\, S_0^{1/2}}{n} \quad [28]$$

Border-strip irrigation is typically practiced on slopes less than 0.05 m/m (5%). On fine textured soils, slopes are typically less than 0.01 m/m. The maximum slope based on the criteria for minimum flow depth (and discharge) can be found by solving Equation 28 for S_0. This does not consider erosion potential.

5.2 Open-ended border strips

For sloping borders, it is often assumed that the flow depth at the upstream end approaches normal depth. The normal depth can be computed from Equation 3 with q as the unit inflow rate and S_f set to the bottom slope, S_0. Solving for depth gives

$$y_0 = \frac{q_{in}^{3/5} (n / C_u)^{3/5}}{S_0^{3/10}} \quad [29]$$

As with furrows, simple volume balance procedures can be used to estimate advance, with $V_y = \sigma_y\, y_0\, W$, where $\sigma_y = 0.7$.

For sloping borders, recession can not be assumed to start at cutoff. For design, the recession time at the downstream end is set so that the required depth is just satisfied there, as in Equation 10. An empirical relationship, adapted from Walker and Skogerboe (1987), can be used to estimate the cutoff time for this required downstream recession time. An approximate upstream recession time is found from

$$t_R(0)_S = t_R(L) - \frac{0.666\, n^{0.47565}\, S_y^{0.20735}\, L^{0.6829}}{I^{0.52435}\, S_0^{0.237825}} \quad [30]$$

where all units are in meters and seconds and S_y is

$$S_y = \frac{1}{L}\left[\frac{(q_{in} - IL)n}{S_0^{0.5}}\right]^{0.6} \quad [31]$$

and I is the infiltration rate (m/s) at $t_R(0)_S$, averaged over the length. For the branch infiltration function after the branch point, it is simply b. For the other infiltration functions, the infiltration rate can be numerically integrated or approximated by averaging the values at the upstream and downstream ends, $\tau(0) = t_R(0)_S$ and $\tau(L) = t_R(L) - t_A(L)$. Equation 30 is essentially an empirical fit to

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a series of computer runs over a wide variety of conditions. It is based on an estimate of the upstream recession lag time from (Strelkoff, 1977), which results in a cutoff time of

$$t_{co} = t_R(0)_S \frac{y_0 L}{2 q_{in}} \quad [32]$$

Solution of Equations 30 and 31 for $t_R(0)_S$ is essentially a trial and error process if the average infiltration rate is not fixed (i.e., with the Kostiakov branch function, the trial and error is not needed).

While the upstream recession time from Equation 30 is needed for the procedure used to compute cutoff time, it does not give a very realistic estimate for the actual upstream recession time for border strips on small slopes. The following equation can be used to estimate the upstream recession time (adapted from Hart *et al.* 1980).

$$t_R(0) = t_{co} + \frac{q_{in}^{0.2} n^{1.2}}{[S_0 + (\frac{0.345 n q_{in}^{0.175}}{\tau_{req}^{0.88} S_0^{1/2}})]^{1.6}} \quad [33]$$

where units are in meters and seconds. The recession lag times for steeper slopes are generally very small and either equation gives reasonable results. For smaller slopes (e.g. < 0.004), Equation 33 generally gives the best estimate (i.e., it is recommended over Equation 30). The procedure given above for computing cutoff times appears to be valid over a wide range of slopes even if it gives poor estimates of recession lag time.

The above procedure estimates the two ends of the recession curve. For steeper slopes, a straight line through two points gives reasonable results. For milder slopes, a straight line generally underestimates the volume infiltrated and overestimates the volume of runoff. If a better estimate of the recession curve for smaller slopes is desired, compute the slope of the recession curve from

$$\frac{d t_R}{dx} = \max[\frac{3 \sigma_y y_0}{q_{in}}, \frac{t_R(L) - t_R(0)}{L}] \quad [34]$$

with the restriction that the computed recession time at any distance not exceed the recession time at the downstream end. The second term in the brackets of Equation 34 is just a straight line between the recession times; $t_R(L)$ computed from Equation 9 and $t_R(0)$ computed from Equation 33. The first term assumes that recession progresses at a rate which removes the surface volume, linearly, at one third the inflow rate – a strictly empirical estimate.

Figure 3 shows an example of advance and recession curves with simulation and from the procedures discussed above. Specifics include L = 400m, S_0 = 0.002, k = 40.1 mm/hr^a, a = 0.51, n = 0.15, d_{req} = 80 mm, and q_{in} = 2.5 l/s/m. The recession time at the downstream end is well predicted with the Walker and Skogerboe (1987) procedures. However, Hart *et al.* (1980) provide a better estimate of upstream recession. In between, results are mixed.

5.3 Blocked-end border strips

Improvement in application efficiency can be obtained by blocking the downstream end of the border strip. This should only be done where the ponding depth, and associated infiltration time, will not cause crop damage. To limit crop damage, the end is sometimes partially blocked to limit

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the maximum ponded depth (e.g., by the elevation of overspill) or the maximum ponding time (e.g., by leaving a breach in the dike to allow it to eventually drain).

Where all the runoff is contained, the distribution of infiltrated water can be modified by assuming that the volume that ran off is ponded at the downstream end. Advance, recession and the distribution of infiltrated depths are computed as if for an open-ended border strip, then a ponded depth is simply added to the infiltrated depth at each location. The length of ponding can be found from

$$L_p = \sqrt{\frac{2V_{RO}}{S_0}} \qquad [35]$$

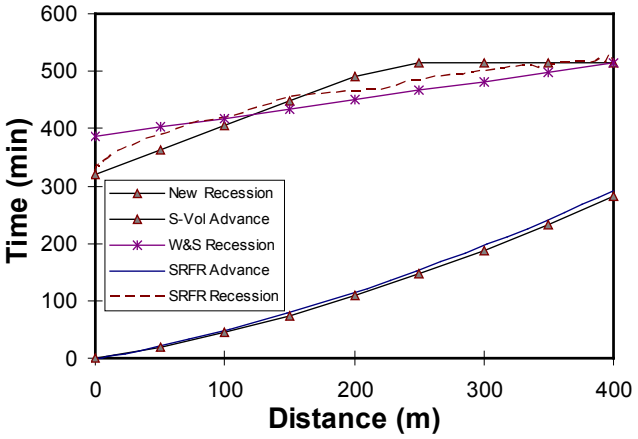

Figure 3:
Comparison of Border-strip advance (from assumed surface volume and from SRFR simulation) and recession (from original Walker and Skogerboe 1987, from adjusted procedure presented here, and from SRFR simulation).

where V_{RO} is the runoff volume. The ponded depth is zero at a distance L_p from the downstream end and is $S_0 L_p$ at the downstream end.

In the design procedures of the USDA (1974), the field length is adjusted to account for the ponding. In this design procedure, the cutoff time is adjusted, by trial and error, until the minimum depth matches the required depth. This will either occur at the upstream end or at a distance L_p from the downstream end.

6. Level-basin and level-furrow irrigation

6.1 Assumptions

With level-basin irrigation, rapid advance will produce a high uniformity. The design of basin irrigation systems is based on providing rapid advance, but without applying excessive amounts of water. In some cases, this requires cutoff prior to completion of advance. But with no field slope and only the water surface gradient to drive the water, cutoff with advance too far from the field end can give unpredictable results. Since the field is level, soil erosion is only a concern where water is turned into the field, i.e. locally. Minimum and maximum flow rates are not specified, but are dictated by the hydraulic conditions, i.e. design. With level basins, flow depth can become high and needs to be examined in design.

6.2 Flat-planted level basins

Walker and Skogerboe (1987) provide a straightforward volume balance approach for level basins, again by assuming a surface volume shape factor. With little or no slope, it is not reasonable to assume that the upstream water depth is at normal depth. In order to solve the volume balance equation (Equations 11 and 12), they compute the upstream depth, y_0, for advance to some distance x from the Manning Equation by assuming $\sigma_y = 0.8$ and by assuming

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$$S_f = \frac{y_0}{x} \quad [36]$$

Combining Equations 3 and 36 and solving for y_0 gives the flow depth at the upstream end

$$y_0 = \{q_{in}^2 (n / C_u)^2 x\}^{3/13} \quad [37]$$

For a given flow rate, advance time can be found iteratively with the same procedures as for furrows or border strips, except that y_0 is now a function of advance distance. As before, two advance distances and times are required so that the advance exponent h can be determined, typically associated with the field length and one half the field length.

However, if it is assumed that recession occurs at the same time throughout the basin, the infiltrated volume can be determined by integrating depth (times width) over distance. A direct solution is not possible, but an approximation can be obtained by representing the power advance function with a series expansion, as before (i.e. $x^h = 1 + h(x-1)$). Taking the first two terms in the expansion results in the following equation for the final infiltrated volume

$$V_z = LW\{c + k\tau_{req}^a \frac{[(1+hA_R)^{a+1} - 1]}{hA_R(a+1)} + b\tau_{req}(1 + \frac{hA_R}{1+h})\} \quad [38]$$

where A_R is the advance ratio, advance time divided by required opportunity time. However, if the branch infiltration function is used, the solution is exact, provided that $\tau_{req} > \tau_B$. The resulting equation is

$$V_z = LW\{d_{req} + \frac{bh\,t_A(L)}{(h+1)}\} \quad [39]$$

The cutoff time is found by dividing this volume by the inflow rate, Q_{in}. For very small values of A_R, this equation is not appropriate, however in such cases the cutoff time can be based solely on required volume (i.e., assuming PAE_{min} = 100%).

This procedure assumes that advance is complete prior to cutoff. For large level basins as used in the U.S., this is frequently not the case, particularly when flow resistance is high (e.g., alfalfa or grass). One of the biggest errors associated with this procedure is that the surface volume during advance is large. This is particularly true when cutoff precedes completion of advance. To avoid problems with this procedure, it should not be used when the advance is less

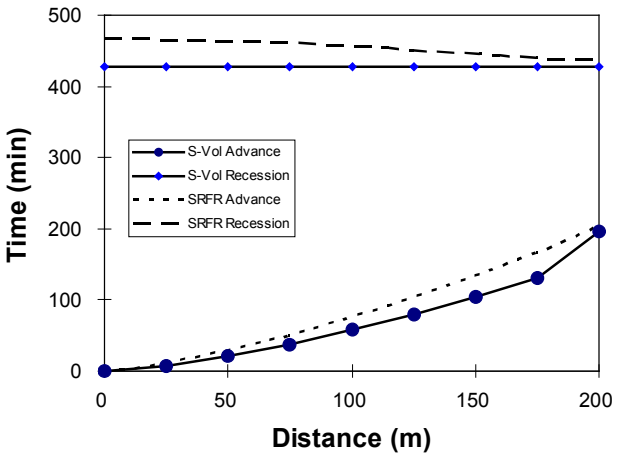

Figure 4:
Comparison of level basin advance (from assumed surface volume and from SRFR simulation) and recession (from uniform recession and from SRFR simulation) curves.

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than 90% complete.

Figure 4 shows an example of advance and recession curves with simulation and from the procedures discussed above. Specific include L = 400m, S_0 = 0.000, k = 40.1 mm/hr^a, a = 0.51, n = 0.15, d_{req} = 80 mm, and q = 2.0 l/s/m. Note that advance is not as well predicted as with the sloping methods. This may result from changes in the surface shape factor over time. Also, note how the recession curve is not flat, but has more opportunity time at the upstream end. This results from higher infiltration toward the downstream end where opportunity times are shorter.

6.3 Level furrows and furrowed level basins

The same assumed-surface-volume method can be used for level furrows or furrowed level basins as was used for flat-planted level basins, except Equation 37 is replaced with (from Equations 2 and 36)

$$A_0^2 R_0^{4/3} y_0 = q_{in}^2 (n / C_u)^2 x \qquad [40]$$

Since A_0 and R_0 are functions of y_0 as defined by the furrow cross section shape, determination of A_0 for the volume balance calculation is iterative rather than direct (i.e., it adds one more internal iteration loop to the procedure). While the design procedures for the two systems are essentially the same (i.e., based on advance and recession of a single furrow), the cultural and irrigation operations for level furrows and furrowed basins can be quite different. For example, cutoff rarely occurs before the completion of advance.

7. Discussion

The set of equations provided above form the basis for computing actual designs. These equations do not lead the user through the design process. They only provide advance, recession and performance results based on preselected field length, width, inflow rate, infiltration, and roughness. The same solution could be obtained by unsteady-flow simulation, where a search was made for the minimum application time that provides the required depth everywhere. Unsteady flow simulations require a relatively sophisticated software program. The intent of this paper is to provide simple procedures that can be used with hand calculators or spreadsheet programs. In a companion paper, the equations developed here are applied to design examples with the aid of spreadsheet programs in which these equations have been programmed.

8. Conclusions

In this paper, equations are presented that can be used to determine advance and recession curves for

- Sloping furrow irrigation, with and without cutback, and with or without runoff reuse
- Sloping border strips, either open or blocked
- Level basins, either flat planted or furrowed

The equations for advance use conservation of mass, but with the Manning equation, and a few coefficients derived from other sources. The advance equations only differ in how the friction slope is computed. For steep slopes, normal depth for the inflow is used, while for level fields, the water surface gradient and upstream depth are a function of the advance distance.

The equations for recession are different for each method. For very steep sloping furrows, one

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can assume that recession occurs everywhere at cutoff with little error. As the slope decreases, one can estimate recession by reducing the cutoff time by the volume of water on the surface at cutoff and assuming a linear recession curve from the time of cutoff at the upstream end to the advance time plus required opportunity time at the downstream end. For level basins and furrows, recession is computed with a volume balance based on assuming that recession occurs at the same time everywhere.

For border strips, recession is more difficult. Here, empirical equations are used for the difference in recession times between the upstream to downstream ends. Empirical relationships are also available for recession at the upstream end. The recession curve between these end points can be assumed a straight line, or can be constructed from an additional empirical relationship.

These equations should be useful for hand calculator and spreadsheet applications.

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Contact address of the author

Albert J. Clemmens
Research Leader
Water Management and Conservation Research Unit
U.S. Arid-Land Agricultural Research Center
USDA-ARS
21881 North Cardon Lane
Maricopa, AZ 85238
USA

E-mail: bert.clemmens@ars.usda.gov
Web: http://www.ars.usda.gov/pwa/maricopa
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