CHAPTER 14

DESIGN OF SURFACE SYSTEMS

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Abstract. *Surface irrigation design and operation are a challenge because soil infiltration and soil and crop resistance influence the movement of water over the field and thus the water distribution. In the past, design for 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 chapter, we apply the same basic procedures for the design of all 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. Examples are provided for each method.*

Keywords. *Basin, Border, Design, Furrow, Irrigation, Surface irrigation.*

14.1 INTRODUCTION

Surface irrigation methods are so named because water is distributed across the field by flowing over the field surface. Thus, soil infiltration and soil and crop flow resistance have major influences on the distribution of water. Because these conditions change with both time and space, the design and operation of surface irrigation systems is often more difficult than pressurized irrigation systems. However, surface irrigation systems still comprise more than 90% of the world's irrigated land (FAO, 2005) and almost half of the irrigated land in the U.S. (Hutson, 2004). While some land is continually being converted to pressurized irrigation, a significant portion of land is likely to remain surface irrigated for the foreseeable future. Surface irrigation can be an effective irrigation method and in some cases can equal the efficiencies of pressurized irrigation methods (Kennedy, 1994: 166). Surface irrigation is still the most economical method for many situations.

Table 14.1. 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 cross slope on milder slopes	distributed across upper end	open, blocked, or partially blocked
Level basins/ 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

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 a particular location and the time when standing water no longer occurs on the surface. 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. We distinguish three categories of surface irrigation: sloping furrows, border strips, and level basins and furrows. The main physical differences are given in Table 14.1. These physical differences result in hydraulic differences, primarily the magnitude of the inflow rate (and pattern) and the general shape of the recession curves and runoff hydrographs.

The purpose of this chapter is to present methods for the design of modern surface irrigation systems, where *modern* implies a reasonable control over the water supply. The chapter focuses on the three major categories of surface irrigation; design of rice paddies and methods such as contour levee, contour ditch, wild flooding, etc. are not discussed.

14.2 DESIGN CONSIDERATIONS AND APPROACHES

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 chapter. For more details, see Walker and Skogerboe (1987), Clemmens and Dedrick (1994), or Burt et al. (1999). One advantage of surface irrigation over pressurized irrigation methods is that they often do not require a good, reliable water supply. They 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.

14.2.1 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. Typically, 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 typically 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 include guidance on system operation.

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, E_a. This efficiency is called *potential application efficiency*, PE_a, here to distinguish it from a field-measured E_a. Design is typically based on supplying the target depth of water everywhere in the field. The PE_{min} is the application efficiency when the minimum depth infiltrated just equals the target depth:

$$PE_{min} = \frac{d_{min}}{d_a} = \frac{d_{req}}{d_a} \tag{14.1}$$

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 under-irrigation is usually allowed and operation is based on satisfying the low-quarter depth. Because design does not take into account all of the variability which exists within the field, we use a design based on satisfying the minimum depth, with the expectation that when properly operated the low quarter depth would be satisfied.

14.2.2 Choosing Design Conditions and Parameters

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 and are often very difficult to determine accurately. These factors can change over the course of the growing season, further complicating design. Published values of the Manning roughness coefficient, n, are usually sufficient for design purposes. However, infiltration is much more difficult to estimate. Whenever possible, estimates of these parameters should be obtained from field measurements, preferably from irrigation events. Soil infiltration conditions can change significantly when land is newly converted to surface irrigation.

Once a reasonable estimate of infiltration parameters is obtained and a design selected, a sensitivity analysis should be performed to determine how the design might change under different conditions of infiltration, flow resistance, and target infiltration depth; conditions that will likely be faced by the irrigator.

14.2.3 Equations for Infiltration in Borders and Basins

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. Several equations have been used to define infiltration. Philip (1957) developed equations for ideal soil conditions (e.g., uniform physical properties) for infiltrated depth versus time for short times:

$$d(\tau) = k\tau^{1/2} + b\tau \qquad i(\tau) = 1/2\, k\tau^{-1/2} + b \tag{14.2}$$

and for long times:

$$d(\tau) = c_2 + b\tau \qquad i(\tau) = b \tag{14.3}$$

where τ = infiltration time
k = sorptivity
b is related to the hydraulic conductivity
c_2 = a derived constant.

Unfortunately, soil surface conditions are not ideal and many soils do not follow Equation 14.2 very well at short times. Many do follow Equation 14.3 at long times, but with different values for c_2 than those derived theoretically.

Irrigation engineers have tended to use the Kostiakov or Kostiakov-Lewis equation, or a variation thereof, defined by

$$d(\tau) = k\tau^{a} \qquad i(\tau) = ak\tau^{a-1} \tag{14.4}$$

or in modified form,

$$d(\tau) = c + k\tau^{a} + b\tau \qquad i(\tau) = ak\tau^{a-1} + b \tag{14.5}$$

where b, c and k are all empirically derived. In Equation 14.4, the exponent a is often below $^1/_2$, indicating the poor fit to Equation 14.2. However, with Equation 14.4, the infiltration rate approaches zero as time approaches infinity. Soils generally reach a constant final infiltration rate. For course-textured soils, where the required infiltration time is short, this occurs well after typical irrigation opportunity times. For fine-textured soils, the final infiltration rate is reached during the irrigation and should be included.

In keeping with the theoretical development, a Kostiakov branch function has been used, where the first branch uses Equation 14.4 (or 14.5 with b = 0) for short times, and switches to Equation 14.3 when the infiltration rate (derivative of equation with respect to time) 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:

$$\begin{aligned} d(\tau) &= c + k\,\tau^{a} & i(\tau) &= ak\,\tau^{a\text{-}1} & &for\ \tau \le \tau_B \\ &= c_2 + b\tau & &= b & &for\ \tau \ge \tau_B \end{aligned} \tag{14.6}$$

where τ_B is the time at which the infiltration depth and rate for the two branches match. For cracking-clay soils, the depth infiltrated over the first few minutes of wetting can be large relative to the total depth infiltrated during the irrigation. With Equation 14.4, this could be fit with a very small value of the exponent a. However, this makes the problem unnecessarily difficult, since whether the depth infiltrates immediately or, say, during the first 10 min has very little effect on the hydraulics of the irrigation or on the distribution of infiltrated depths. For these soils, it makes more sense to fit infiltration with an equation such as Equation 14.3 and ignore the initial part of the infiltration curve (e.g., use second half of Equation 14.6).

The U.S. Soil Conservation Service (SCS, currently Natural Resources Conservation Service) developed a series of infiltration families to assist field personnel in determining infiltration relationships. These families are defined according to Equation 14.5 with b = 0 and c = 7 mm. The infiltration relationships for many soils do not follow the SCS families very well. The value of exponent a in this equation is around 0.7 for all the families. Non-cracking soils typically have exponents that range from 0.3 to 0.8. Merriam and Clemmens (1985) developed a relationship between the exponent a and the time to infiltrate 100 mm from a wide range of field data. From this they developed time-rated intake families, based on the time to infiltrate 100 mm, a typical required depth for surface irrigation. These time-rated intake families are given in Table 14.2.

These infiltration relationships are entirely empirical. Emphasis should be placed on the general characteristics of the infiltration relationship and not on the equation chosen or the values of the various parameters. As long as the equation and constants chosen fit the observed infiltration relationship and are useful, this is sufficient justification for their use. In particular, it is most important to have the proper range of infiltrated depths over the range of infiltration opportunity times. Table 14.3 shows the parameters for five infiltration functions, representing the various equations fit to infiltration described by the 6-hr time-rated family (i.e., 6 h to infiltrate 100 mm with a Kostiakov exponent of 0.51). This corresponds to an SCS intake family of about 0.45. These infiltration functions are plotted in Figure 14.1. There is very little difference between the infiltrated depths predicted by the three different Kostiakov equations. A linear equation even fits well between 4 and 8 h.

Table 14.2. Time-rated intake families (from Merriam and Clemmens, 1985).

Time to Infiltrate 100 mm (hours)	Kostiakov Exponent (a)[a]	Kostiakov Constant (k) (mm/hr[a])	Kostiakov Constant (k) (in/hr[a])
0.5	0.739	167.0	6.57
1	0.675	100.0	3.94
2	0.611	65.5	2.58
4	0.547	46.8	1.84
8	0.483	36.6	1.44
16	0.419	31.3	1.23
32	0.355	29.2	1.15

[a] Exponent $a=0.675-0.2125\log_{10}(\tau_{100})$, where τ_{100} is the time (in hours) to infiltrate 100 mm.

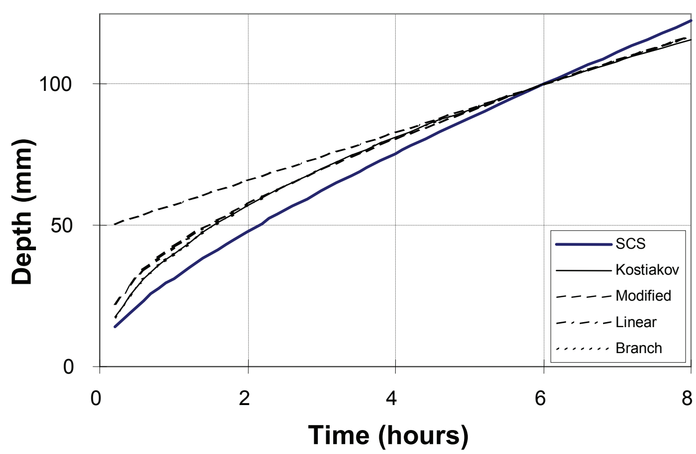

Figure 14.1. Comparison of five infiltration functions.

Table 14.3. Infiltration functions matching 6-hr time-rated family.

Parameters	SCS Equation	Kostiakov Equation	Modified Kostiakov	Kostiakov Branch	Linear Equation
k (mm/hr)	24.26	40.1	37.39	40.1	0
a	0.75	0.51	0.35	0.51	0
b (mm/hr)	0	0	5.0	8.5	8.9
c (mm)	7	0	0	0	49.0

14.2.4 Equations for Infiltration in Furrows

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. If gravitational forces dominate the infiltration process (e.g., a very sandy soil), then infiltration may be directly proportional to the wetted width. If furrows are closely spaced and the soil is very heavy such that capillary forces dominate infiltration, the lateral infiltration from adjacent furrows will meet such that infiltration essentially occurs over the entire set width; i.e., for each furrow, infiltration becomes a function of the furrow spacing. The main problem is that for many situations, infiltration conditions are in between these two extremes, and may differ significantly over time or along the length of run. For example, infiltration at the head end of the field, where the flow rate and water depth are high, may result in infiltration after, say, 12 hours essentially governed by furrow spacing; while at the tail end where the flow rate and wetted width are significantly reduced and the infiltration opportunity time is short, infiltration may be strongly influenced by the wetted width.

The SCS design procedures for sloping furrows (USDA, 1984) use an infiltration function based on the wetted perimeter at normal depth, plus an empirical constant of 213 mm to account for the lateral flow. This is reported as an average value from numerous tests. For irrigation of every furrow, this width should not exceed the furrow spacing. This width is then assumed constant over time and distance. The assumption with this procedure is that the SCS intake family for a given soil used for border-strip irrigation design can also be used for sloping-furrow design. In practice, this approach has not been very successful.

For general use, infiltration really needs to be judged from field performance of the irrigation system. If a constant width is to be chosen for infiltration, it is simpler to express infiltration as a function of furrow spacing; recognizing that significant changes in wetted perimeter (e.g., from furrow shape or flow-rate changes) or furrow spacing (or whether all furrows are irrigated) may change infiltration constants based on furrow spacing. The design procedures presented here do not consider the influence of wetted width on furrow infiltration. Thus, these design procedures should be used with some caution as they may overestimate irrigation uniformity.

Where no furrow infiltration data are available, it may be necessary to estimate furrow infiltration from ring infiltration data or from border irrigation trials. For mild slopes, high flow rates, closely spaced furrows and heavy soils, the infiltration can be assumed to be somewhat similar to that for borders (i.e., one-dimensional and based on furrow spacing), except there is a tendency for the infiltration rates to start higher and drop more quickly (i.e., a smaller value of the exponent a in the Kostiakov-Lewis equation). For steeper slopes, low flow rates and coarse-textured soils, the border infiltration rates can be applied to the wetted width, with perhaps an increase in k to account for some lateral movement and a decrease in the exponent a. Such adjustments must be considered just a guess since the effects of differences in tillage and compaction from tractor traffic between borders and furrows can be substantial.

Furrow irrigation has the additional complexity in that equipment wheels travel through some furrows and not others. Wheel traffic generally causes compaction and smoothing of the soil surface, a reduction in infiltration, and a reduction in flow resistance. In some cases with sloping-furrow systems, only wheel or only non-wheel rows are irrigated. Where both are irrigated, the inflow rates to wheel and non-wheel rows are typically adjusted by the irrigator so that advance in all rows is approximately the same. These adjustments are not needed in level furrow or furrowed level-basin irrigation systems. In other cases, non-wheel furrows are intentionally compacted by dragging heavy objects (sometimes called torpedoes) through them. When considering also changes in properties over the season, design procedures often have to be applied to a wide range of conditions. Because of this, some irrigation evaluations only attempt to determine field-average conditions (e.g., an infiltration function determined for an entire set), which assumes that irrigation and tillage practices will be similar after design.

14.2.5 Equations for Flow Resistance

Resistance to flow is usually described by the Manning roughness coefficient. This equation 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 n:

$$Q = \frac{AR^{2/3} S_f^{1/2} C_u}{n} \tag{14.7}$$

The units coefficient, C_u, allows the same value of the Manning n in both foot-pound and metric units. For units of m and m^3/s, C_u has a value of 1.0. Expressed in terms of flow rate per unit width, q, for border strips and basins, this becomes

$$q = \frac{y^{5/3} S_f^{1/2} C_u}{n} \tag{14.8}$$

where y is the flow depth and $q = Q/W$, where W is the basin or border width. All units in Equations 14.7 and 14.8 are in meters and seconds.

In theory, the Manning n should change with flow depth when vegetation is present. However, the changes in the Manning n over the season and with changes in planting density are as large as changes caused by flow depth. For design purposes, a constant Manning n seems to be satisfactory. Published values of the Manning n for various crops are shown in Table 14.4. Manning n values have been observed as high

Table 14.4. Recommended Manning *n* values.

Source	*n*	Conditions for Use
USDA (1974, 1984)	0.04	smooth, bare soil surfaces and furrows
USDA (1974, 1984)	0.10	drilled, small-grain crops if the drill rows run in the direction of water flow and corrugations
USDA (1974)	0.15	alfalfa, broadcast small grains
Clemmens (1991)	0.20	dense alfalfa or alfalfa on long fields with no secondary ditches
USDA (1974)	0.25	dense sod crops and small grains drilled perpendicular to the flow

as 0.4 on closely planted wheat, a value significantly higher than that given in the table. Also, furrow-irrigated crops with vegetation that hangs into the water can cause much greater resistance than the Manning n of 0.04 given in Table 14.4. Here again, field observations of advance and recession can be used to determine whether design estimates of Manning n are reasonable.

14.2.6 Consideration of Operating Practices and Guidelines

We know that the conditions assumed in design will never be exactly encountered in practice. The irrigator of a surface irrigated field will adapt operations to account for differences from the assumed conditions. In addition to variations in infiltration and roughness, the irrigator must adapt to changes in flow rate and target depth of application. Frequently, the flow rate is only approximately known and changes in application time to account for this are not easily made. The operator frequently uses information about the advancing stream to make these adjustments, either in inflow rate or in application time.

Usually, these adjustments are based solely on experience. However, such experience takes time to accumulate and often experience on one type of soil does not translate to another type of soil. To aid in this process, design should consider the operating criteria to be used; either as recommendations after design or used explicitly in the design procedure to help determine the field layout. In any event, a sensitivity analysis can determine what will happen if the irrigator operates the system with the defined operating criteria over the range of possible conditions.

14.2.7 General Design Approaches

The approach taken for design assumes that repeated trials are needed based on existing conditions, local farming and irrigation practices, and designer experience. The procedures provided in this chapter allow the calculation of irrigation results and performance for each individual trial. It is up to the designer to explore various options in arriving at a final design recommendation. The methods presented here allow one to compute advance and recession curves, the distribution of infiltrated water, the amount of deep percolation, and runoff, if applicable. The approach assumes that the designer knows the infiltration function of the soil. If infiltration is not well known or changes over the season, the designer is responsible for evaluating the impact of variations in infiltration on design and operations. This is beyond the scope of this chapter.

The design procedures assume that the depth to be infiltrated during a particular irrigation event is known by the designer. This required depth of infiltration may be different for different crops and can change over the season. It is the designer's responsibility to take this into account during the design process.

Design is typically 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 infiltration constants 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

$$\tau_{opp}(x) = t_R(x) - t_A(x) \tag{14.9}$$

If the minimum depth is at the head end of the field ($x = 0$), the opportunity time at that point is equal to the recession time, or

$$\tau_{opp}(0) = t_R(0) = t_{co} + t_{lag} \tag{14.10}$$

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. Design based on meeting the requirement at the upstream end does not require advance and recession curves to be computed. However, an estimate of PE_{min} is required, along with a method for estimating recession lag time (USDA, 1974).

For well designed systems, the minimum depth is typically at the downstream end of the field, furthest from the water source ($x = L$). Then advance and recession curves must be computed, and estimation of inflow rate and time required to achieve τ_{req} is more difficult. Specifically,

$$t_R(L) = t_A(L) + \tau_{req} \tag{14.11}$$

Precise solutions of advance and recession are possible with solution of the Saint Venant equations of continuity and momentum. However, this solution requires numerical analysis and is done on a case-by-case basis. It does not directly produce general equations for advance and recession. The time of cutoff is

$$t_{co} = t_A(L) + \tau_{req} - [t_R(L) - t_R(0) - t_{lag}] \tag{14.12}$$

where the term in brackets is the time between cutoff and recession at the downstream end, which is sometimes assumed to be zero (e.g., sloping furrows with runoff). In this chapter, we provide methods for design based on satisfying the minimum depth at the downstream end.

14.2.8 Assumed Surface-Volume Method

All design methods must use procedures that satisfy a volume balance, regardless of how sophisticated they are. In this chapter, we make assumptions 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 is equal to the difference between the accumulated inflow volume and the surface storage volume at that time. This volume balance relationship may be expressed as:

$$V_{in}(t) = V_y(t) + V_z(t) \tag{14.13}$$

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_x, to distance x requires that the surface and subsurface volumes at any time during advance are known. Typically, Equation 14.13 is put in the following form:

$$Q_{in} t_x = \sigma_y A_0(t_x)x + \sigma_z W z(t_x)x \tag{14.14}$$

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, z is the infiltrated depth, 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 14.5, the subsurface shape factor is difficult to determine. It is easier to rewrite the subsurface volume in the following form:

$$V_z = W\left(c + \sigma_{z1}\, k\, t_x^a + \frac{h}{1+h} b t_x \right) x \tag{14.15}$$

where h is the exponent on the advance equation,

$$t_x = s\, x^h \tag{14.16}$$

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

$$\sigma_{z1} == \frac{h + a(h-1) + 1}{(1+a)(1+h)} \tag{14.17}$$

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 still an iterative process since the advance exponent h and advance times are interrelated and must be solved for simultaneously.

The main problem with determining advance from Equations 14.14 through 14.17 is that the A_0 and σ_y are not known in general. For steep slopes, this method is reasonably good since after a short time, the surface volume is typically a small amount of the total inflow volume. Also, A_0 can be assumed equal to the flow area at normal depth and σ_y can be estimated with sufficient accuracy. 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 (see Chapter 13). Similarly, σ_y can vary (see Chapter 13). On a unit width basis, $A_0 = y_0\, W$.

14.2.9 Simulation Approach

One method for more accurately determining advance is to simulate it with the unsteady flow equations, e.g., with the simulation programs SIRMOD (Walker, 1989) or SRFR (Strelkoff, 1990). Recession can be found either from continued solution of the equations, or from assuming that recession occurs at the same time everywhere. The latter approach was used in the level-basin design program, BASCAD (Boonstra and Juriens, 1988). BASCAD will search through various input parameters to arrive at a useful solution. It does not allow PE_{min} as a design input, but displays it as a design output. The above unsteady-flow simulation programs can also be used through trial and error by the user to arrive at a design solution.

14.2.10 Dimensional-Analysis Approach

Because of the large number of variables involved in surface irrigation, it is virtually impossible to present the results of simulations for the range of possible field conditions. Dimensional analysis methods have been used to reduce the number of parameters so that meaningful results can be displayed on a limited number of graphs. 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). No nondimensional solutions are currently available for furrows. This methodology is presented in some detail in Chapter 13.

14.3 SLOPING-FURROW IRRIGATION

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.

The Soil Conservation Service (USDA, 1984) provided the following guidance for limiting irrigation-induced erosion from furrows. 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. 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-min 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 erosive and non-erosive soils, respectively. The relationship between velocity and flow rate can be obtained from Equation 14.7, since velocity is flow rate, Q, divided by flow area, A.

14.3.1 Design of Sloping Furrows with No Cutback or Tailwater Reuse

Design of sloping furrows with runoff is very straightforward under the following assumptions. First, advance can be computed from Equation 14.13 or 14.14, which give an accurate estimate of advance times when a good estimate of the surface-volume shape factor is made. Second, we can assume 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 0.05%. Next, recession along the entire furrow length is assumed to occur immediately after cutoff for steeply sloped furrows, particularly above 0.5% slope. For flatter slopes, the recession at the downstream end will take some time, resulting in a greater infiltration depth there and underprediction of PE_a. Adjustment to the application time and recession curve are made based on the surface volume at the completion of advance.

Finally, 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, wetted width can cause a significant reduction in infiltration along the furrow 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.

Design for a specific set of field conditions (i.e., infiltration, roughness, and required infiltration depth) is essentially by trial and error for furrow length, slope, and inflow rate. 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 14.7. 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 14.14, 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 14.17, which required a power advance function (Equation 14.16). Estimation of the exponent h in Equation 14.16 requires that at least two points on the advance curve be estimated.

The design procedure starts with estimates for the time for the advancing stream to reach half the field length and the entire field length. From these two, the advance exponent can be computed from

$$h = \frac{\log(t_{L/2}/t_L)}{\log(1/2)} \tag{14.18}$$

With Equations 14.15 and 14.17, we can compute the subsurface volume. A new guess for the advance times can then be estimated from the volume balance (e.g., Equation 14.14) and new values for h calculated. The procedure converges fairly rapidly. An alternate procedure which is computationally faster for spreadsheets is to estimate h, allow the spreadsheet to iterate on the time to achieve a volume balance at both distances independently, and manually iterate on h until it converges.

To provide the required depth at the downstream end, the cutoff time is computed from Equation 14.12, with the last term (in brackets) assumed equal to zero. The application efficiency is computed as the required volume (required depth × furrow spacing × furrow length) divided by the applied depth (inflow rate × cutoff time).

The volume of runoff can be estimated by computing a distribution of infiltrated depths, the associated infiltrated volume, and subtracting this volume from the inflow volume. 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 infiltrated volume can be computed from the infiltrated depths at the head, middle, and tail of the field, since the infiltration opportunity times are known there (i.e., recession is assumed to occur at cutoff). Better estimates can be made by closer numerical integration, for example by computing additional points along the advance curve.

Example 1. Consider a field 400 m long with a slope of 0.002, infiltration (based on a width equal to the 1 m furrow spacing) described by k = 40 mm/hr^a and a = 0.35, and Manning roughness of 0.05. (Carefully smoothed and compacted furrows can have a roughness as low as 0.03.) The furrow spacing is 1 m and the furrow shape is defined as a trapezoid with a bottom width of 100 mm with 2:1 side slopes, horizontal to vertical. Initial design will be made for a required depth of 80 mm. The resulting required opportunity time is 435 min.

This design example starts by assuming an inflow rate of 1.0 L/s. Iterative solution of Equation 14.7 gives; a normal depth of 53 mm, a wetted perimeter of 338 mm, a velocity of 5.5 m/min (well within the acceptable range), and an upstream cross-sectional flow area of 0.011 m^2.

For sloping furrows, we assume that the upstream depth reaches normal depth quickly and remains there. Then the surface volume during advance is not a function of time, and $V_y(t)$ becomes $V_y(x)$. If we assume that $\sigma_y = 0.75$, then from the expression for V_y in Equation 14.14, we get surface volume of 1.65 and 3.29 m^3 for advance to 200 and 400 m, respectively (e.g., $0.75 \times 0.011 \times 200 = 1.65$).

The design procedure starts with a guess for the advance time to half the field length and to the field end. For this example, we arbitrarily guess 100 and 250 min, respectively. For a flow rate of 1 L/s, this represents inflow volumes of 6.0 and 15.0 m^3. From Equation 14.18, we get $h = 1.32$. With $h = 1.32$ and $a = 0.35$, Equation 14.17 gives $\sigma_z = 0.777$. (Note that for the Kostiakov equation σ_z and σ_{z1} are the same.) The infiltrated depths at the head of the field at 100 and 250 min are 47.8 and 65.9 mm, respectively. Substituting these into the expression for V_z in Equation 14.14, we get subsurface volumes of 7.43 and 20.48 m^3 at these two times, respectively.

At 100 min, the computed surface and subsurface volumes total 1.65 + 7.43 = 9.08, which is much larger than the inflow volume of 6.0. Thus the advance time to 200 m must be much longer. Similarly, at 250 min $V_y + V_z = 23.77\ m^3 > V_{in} = 15.0\ m^3$. If we adjust the advance times to remove the volume error, our next guess of advance times would be 100 × (9.08/6.0) = 151 min and 250 × (23.77/15.0) = 396 min, respectively.

If we repeat the calculations, we get $h = 1.39$, $\sigma_z = 0.783$, $V_{in} = 8.65$ and 24.25, $V_y + V_z = 10.30$ and 27.55, for the two advance locations, respectively. Clearly, this linear adjustment is too small. This is due to the fact that the infiltrated depth also changes with the change in time. From this point, if we take $1^1/_2$ times the correction we get advance times of 181 and 490 min, $h = 1.44$, $\sigma_z = 0.787$, $V_{in} = 9.27$ and 26.27, $V_y + V_z = 10.92$ and 29.56, for the two advance locations, respectively. Again the correction is insufficient, but much closer.

The solution for advance finally converges as shown in Table 14.5. The cutoff time is then 494 + 435 = 929 min (15.5 h). The required volume is 32 m^3 (400 m × 1 m × 0.080 m), while the applied volume is 55.7 m^3 (0.0010 m^3/s × 929 min × 60 sec/min), giving an application efficiency of 57.4% (32/55.7). This assumed surface-volume method can be used to compute the advance time to intermediate locations. The advance times computed from the power-function advance relationship derived from the two-point method usually do not exactly satisfy a volume balance, but can be used as a good first guess of the advance time. If we use the value of the advance exponent h from the two-point method, the advance times calculated at intermediate points are shown in Table 14.6.

If we assume that recession occurs everywhere at the time of cutoff, then the depths infiltrated at the head, middle, and tail end of the field are 104, 97, and 80 mm, respect-

Table 14.5. Advance calculations for assumed surface-volume sloping-furrow design example.

Q (L/s)	x (m)	t_x (min)	$V_{in}(t)$ (m^3)	A_0 (mm)	$V_y(t)$ (m^3)	h	σ_z	$V_z(t)$ (m^3)
1.0	200	182.3	10.94	0.011	1.65	1.44	0.787	9.29
1.0	400	494.0	29.64	0.011	3.29	1.44	0.787	26.35

Table 14.6. Advance and recession for sloping-furrow design example (Q_{in} = 1 L/s), with recession times adjusted.

Distance (m)	Advance Time (min)	Recession Time (min)	Opportunity Time (min)	Infiltrated Depth (mm)
0	0	874	874	102
50	25	881	856	101
100	67	888	820	100
150	121	894	774	98
200	182	901	719	95
250	251	908	657	92
300	327	915	580	89
350	408	922	514	85
400	494	929	435	80

tively. Numerical integration with the trapezoidal rule (i.e. numerically averaging depths over the two intervals) gives an average infiltrated depth of 94.5 mm and infiltrated volume of 37.8 m^3. The runoff volume is the difference between the applied and infiltrated volumes, or 55.7 – 37.8 = 17.9 m^3, or 32.1%. The deep percolation volume is 37.8 – 32 = 5.8 m^3, or 10.4%. Better estimates for these volumes can be found by dividing the field into several increments and computing advance either from the volume balance or from Equation 14.16 (the difference is insignificant). In this example, dividing the field into eight increments (as shown for advance in Table 14.6) for advance and recession gave runoff and deep percolation volumes of 31.7% and 10.9% (rather than 32.1% and 10.4%).

Our assumption with the method is that the volume of water on the surface at the time of cutoff is insignificant. In this example, the slope is relatively shallow and the volume on the surface at cutoff from Table 14.5 is 3.3 m^3, or 5.9%, not an insignificant volume. If we compare these results with those from simulation, we find that recession at the downstream end takes more than an hour, implying that we could cut off

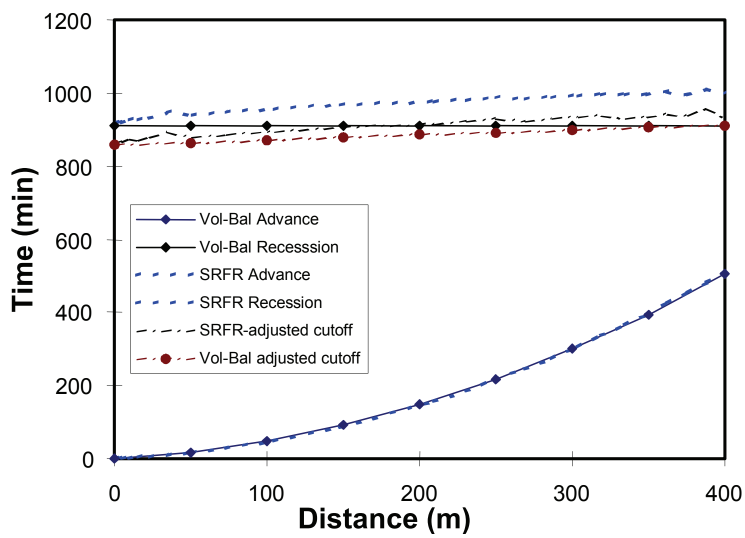

Figure 14.2. Advance and recession curves for sloping-furrow design example.

the stream much earlier. Figure 14.2 shows the advance and recession curves from the volume balance and from simulation with the SRFR program. Note the excellent agreement in the two advance curves, associated with the assumed surface-volume and the unsteady-flow simulation. The computed runoff and deep percolation volumes were 29.4% and 13.3%, respectively. For this relatively mild slope, the effect of the recession curve is significant, as reflected by the shift in runoff and deep percolation volumes.

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

$$t_{co} = t_A(L) + \tau_{req} - \left(\frac{V_y(L)}{Q_{in}} \right) \tag{14.19}$$

where $V_y(L)$ is the steady state surface volume for the inflow rate after advance is complete. For convenience, we use the surface volume computed when advance is just complete. 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}. This results in a cutoff time of 856 min. (14.3 h), an applied volume of 55.7 – 3.3 = 52.4 m^3, and an application efficiency of 61.0%. Here the recession curve is not horizontal, but can be assumed linear between the upstream and downstream ends (Table 14.6). This assumption, with advance and recession computed over eight intervals, gives runoff and deep percolation of 28.3% and 10.7%, respectively. From simulation, this design is closer to matching the required depth at the downstream end, as shown in Figure 14.2, than assuming instantaneous recession. Further, simulation resulted in runoff and deep percolation volumes of 26.9% and 12.2%, respectively. It appears that the inflow volume could be reduced even more than the volume on the surface. However, this simple procedure provides reasonable results and is generally on the conservative side. For furrows with a steeper slopes, this adjustment will be small and is not necessary.

With the runoff percentage significantly higher than deep percolation percentage, one would expect a higher potential application efficiency with a smaller flow rate. Repeating these calculations at 0.7 L/s results in an application efficiency of 64.4%, t_L = 808 min (13.5 h), t_{co} = 1182 min (19.7 h), and runoff and deep percolation of 18.1% and 17.5%, respectively. A field is typically broken up into an integer number of irrigation sets, with the total inflow divided among the furrows. Thus furrow flow rate is not a continuous variable, but typically can only take on discrete values. A field is also broken into an integer number of lengths, making furrow length also not a continuous variable for design.

14.3.2 Design of 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

$$\bar{i}_{CB} \le \frac{Q_{CB}}{WL} \tag{14.20}$$

The average infiltration rate at any time after completion of advance can be computed from numerical integration over the distance (e.g., the eight 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 14.16. If this term is replaced with a truncated series expansion, with higher order terms removed (e.g., $x^h = 1 + h(x - 1)$), an analytical expression for the infiltration rate, averaged over the field length, can be found, namely

$$\bar{i}_{CB} \approx b + \frac{akt_{CB}{}^{(a-1)}}{ah}\left[\left(\left\{\frac{t_{CB}}{t_L}\right\} - 1 + h\right)^a - \left(\left\{\frac{t_{CB}}{t_L}\right\} - 1\right)^a\right] \tag{14.21}$$

where t_{CB} is the time of cutback. The cutback time to achieve the necessary average infiltration rate can be determined from Equation 14.21 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 14.20. The adjusted cutback time is simply

$$t_{CB\,adj} = t_{CB} - \frac{V_y(Q_{in}) - V_y(Q_{CB})}{Q_{CB}} \tag{14.22}$$

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

The cutoff time is computed according to Equation 14.19, but with $V_y(Q_{in})$ replaced with $V_y(Q_{CB})$. Because less volume is on the surface during cutback, the cutoff time is actually slightly longer. For some soils, the reduction in wetted perimeter caused by cutback might require an increase in total application time.

Example 2. For a 50% cutback flow of 0.5 L/s, Equation 14.20 indicates that the average infiltration rate would need to be below 4.5 mm/hr:

$$\frac{0.5\text{L/s}}{400\text{m}\times 1\text{m}} \times \frac{3600\text{sec}}{1\text{hr}} \times \frac{1\text{m}^3}{1000\text{L}} \times \frac{1000\text{mm}}{\text{m}} = 4.5\text{mm/hr}$$

From Equation 14.22, the time of cutback is 582 min. (Trial and error solution with numerical integration of the average infiltration rate over eight intervals gives t_{CB} = 609 min.) The surface volume for the cutback flow is 1.98 m^3. Equation 14.22 gives an adjusted cutback time of 538 min. The adjusted cutoff time, from Equation 14.19

with $V_y(Q_{CB})$ replacing $V_y(L)$, is 863 min. The total inflow volume is now 42.0 m^3 rather than 55.7 m^3. This gives an application efficiency of 76.1%. Runoff is reduced to 4.53 m^3, or 10.8%, while deep percolation is 13.1%. SRFR simulation gave 9.4% and 14.4% for runoff and deep percolation, respectively. Advance, recession and the distribution of infiltrated water are shown in Table 14.7.

Table 14.7. Advance and recession for sloping furrows with cutback design example (Q_{in} = 1 L/s).

Distance (m)	Advance Time (min)	Recession Time (min)	Opportunity Time (min)	Infiltrated Depth (mm)
0	0	863	863	102
50	25	871	845	101
100	67	879	810	100
150	121	888	767	98
200	182	896	713	95
250	251	904	654	92
300	327	912	586	89
350	408	921	513	85
400	494	929	435	80

14.3.3 Design of Sloping Furrows with Tailwater Reuse

Furrow irrigation systems on sloping fields usually need some runoff in order to provide a water depth at the downstream end to get sufficient wetted perimeter and lateral water movement. The design procedures used here ignore this level of detail and can sometimes give misleading results, particularly when cutback flows are used. Tailwater reuse provides another mechanism for providing a sufficient flow rate and water depth at the downstream end, while still improving furrow application efficiency.

Tailwater can be reused in fields which are downstream from the field where the tailwater is generated (e.g. cascaded from field to field). Alternately, the tailwater from a field can be collected in a storage pond, pumped back, and reused on the same field. Improvements in irrigation efficiency resulting from tailwater reuse depend upon the type of system, the amount of the tailwater that is reused, and the number of times it is reused.

Solomon and Davidoff (1997) developed equations for determining the combined efficiencies associated with the reuse of water through a series of areas. Applied to tailwater reuse systems, the application efficiency of the combined system with reuse, E_{a_RU}, can be estimated from the application efficiency from a single irrigation set from

$$E_{a\_RU} = \frac{E_a}{1 - RU\left(\frac{n-1}{n}\right)} \tag{14.23}$$

where RU is the fraction of the applied water that is reused and n is the number of subunits from which water is reused. For a cascaded reuse system, n is the number of systems (e.g., individual fields, farms, projects, etc.) irrigated in sequence. For a recycling reuse system, n is infinity and the term $(n-1)/n$ becomes unity.

With a recycling runoff reuse systems, a very high runoff fraction can result in excessive pumping. The volume of water pumped or recycled, V_{RC}, relative to the inflow volume can be estimated from the reused runoff fraction by

$$V_{RC} = V_{in} \frac{RU}{1 - RU} \tag{14.24}$$

There are two main types of recycling runoff reuse systems: those that recycle runoff at a rate nearly the same as the runoff rate, and those that recycle at a constant rate. The advantage of the runoff-rate type is that storage volume is minimized. Two methods used for these systems are to cycle the pump on and off at short intervals, the exact interval related to the rate of runoff. Maximum recommended cycle rates for these systems are 15 times per hour. Sump surface area depends upon the maximum runoff rate and the allowable depth fluctuations between cycles. The other method for runoff-rate recycling systems is to use variable speed pumps (Trout and Kincaid, 1994).

For these systems, runoff should be applied to another field or a different irrigation set, or pumped to a larger, upstream reservoir. It is not recommended to recycle water back into the supply stream for the field that is creating the runoff, unless additional furrows are started as return flow increases (ASAE, 1997). Using a variable return-flow rate to irrigate additional furrows is difficult to manage and increases labor, and therefore is not recommended. Sump sizes for these systems are on the order of tens of cubic meters.

The intent of a constant flow recycling system is to provide a constant flow rate to all furrows. This can be accomplished by pumping water from a reuse storage reservoir and adding it to the normal irrigation stream from the beginning of the irrigation. The return-flow rate should be such that the return-flow volume for one set is approximately equal to the runoff volume for one set. Then the reservoir will be at approximately the same level at the end of the irrigation as it was at the beginning. This essentially requires a full reservoir at the start of the irrigation. Also, there must be a sufficient volume of water in storage to provide the return flow until the runoff rate from the first set matches or exceeds the pumping rate. This operating scheme might be practical where several fields share the same return-flow system and reservoir. However, for many systems this is not practical.

Rather than starting with the reservoir full, the same general scheme can be used with the reservoir initially empty. This is accomplished by splitting the width of the first set into a portion irrigated with the supply inflow only and a portion irrigated with the reuse portion only. The second part of the first set is actually irrigated after the main supply flow is turned off and irrigated with return flow only (i.e., it is moved to the end of the irrigation and the end of the field). This scheme was first proposed by Stringham and Hamad (1975).

For a given field (length, width, slope, infiltration) and a given inflow rate Q_{in}, the design of these systems is based on balancing three parameters: the desired infiltrated depth (e.g., average, d_i or low quarter, d_{lq}); the individual furrow flow rate, q; and the number of irrigation sets across the field width, N (an integer number). These cannot be selected independently, but must assure that the set runoff and return-flow volumes approximately match. Once these three are determined, the other design parameters can be found. A number of alternative combinations can be examined to find a workable combination (e.g., optimum).

The number of sets, the individual furrow flow rate, Q_f, and relationship between the supply inflow rate, Q_S, and the return-flow or pumpflow rate, Q_P, should satisfy:

$$F = N\left(\frac{Q_s}{Q_f} + \frac{Q_p}{Q_f}\right) = N\left(n_s + n_p\right) \tag{14.25}$$

where F is the total number of furrows in the field, N is the number of sets, and n_s and n_f are the number of furrows in one set irrigated with the supply and pumpback flows, respectively. The selection of return-flow rate must assure that there is sufficient runoff volume which is satisfied when

$$RO \geq \frac{Q_p}{Q_S + Q_P} \quad \text{or} \quad Q_P \leq \left(\frac{RO}{1 - RO}\right)Q_S \tag{14.26}$$

where RO is the runoff fraction (i.e., fraction of inflow that runs off for an *average* furrow). If this equation isn't satisfied, there may not be enough water in the reservoir to irrigate the last set. When this inequality is satisfied as an equality, the pumping rate is the maximum allowable pumping rate, $Q_{P\text{-}max}$. If the pumping rate is less than the maximum allowable, a portion of the runoff volume will not be returned to the same field. Here, we assume that this portion of the runoff volume is unused or a loss.

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,

$$V_R \geq Q_P(1 - RO)t_{co} \leq Q_s ROt_{co} \tag{14.27}$$

where t_{co} is the application time for each set. When Equation 14.26 is an equality, this volume equals the volume of runoff from the first set, irrigated without return flow, and the right inequality of Equation 14.27 is an equality. Note that the volume required in the reservoir at the start of this last set is less than the volume needed to irrigate the last set. For that set, its own runoff is recirculated to provide the necessary volume. The runoff volume lost is equal to the total volume applied times the difference between the actual runoff fraction and the reused runoff fraction $V_{in}(RO - RU)$.

The method presented above for determining sloping-furrow advance and recession curves can be used to determine the runoff fraction for a given furrow flow rate and application time. This information can be used to determine whether this satisfies the desired depth of infiltration and Equations 14.25 and 14.26. The application efficiency can be adjusted according to Equation 14.23 with $n = \infty$ and $RU = RO(Q_P/Q_{P\text{-}max})$, or some fraction thereof to account for return-flow system losses. This assumes that only the water pumped during this irrigation event is reused.

Example 3. Consider the example given in the previous sections. In this case, there are a total of 800 furrows (800 m at 1 m/furrow) and the available flow rate is 80 L/s. An initial trial at 1 L/s/furrow gives the advance and recession curves of Table 14.6 and n_S = 80 furrows. From Equation 14.26, the maximum return-flow pumping rate is 31.6 L/s. Choosing Q_P = 31.6 L/s results in n_P = 31.6 and N = 7.2 (800/[80 +31.6]), which is not an integer and so is not feasible. If the solution were feasible, we would get RO = 28.3%, PE_a = 85.1% (from Equation 14.23), and a reservoir volume of 1188 m^3. One way to make this solution feasible is to choose a smaller pumping rate. At Q_P = 20 L/s, n_f = 20 and N = 8, and a smaller reservoir is needed. However, then only 20/31.6 or 63% of the runoff is reused. If only the reused volume were considered useful, then PE_a would drop to 74.3% with a corresponding runoff volume lost would be 3480 m^3 (Table 14.8). This emphasizes the need to satisfy the inequality in Equation 14.26 as close as possible to an equality.

A number of adjustments can be made in the other variables to produce an integer number of sets, with a better fit to Equation 14.26. Table 14.8 provides the results of changing various parameters on PE_a, V_R, and $V_{RO\text{-}lost}$. If the inflow could be increased to 82 L/s, a feasible solution would result (Table 14.8, run 3). The target application depth could be changed, either increased slightly or decreased slightly (runs 4 and 5). The lower efficiency for run 5 reflects the lower uniformity associated with the reduced target depth. Or, the individual furrow flow rate could be adjusted to arrive at feasible solutions, as in run 6. In run 7, the flow rate decrease did not provide a very useful solution without also a change in the required depth; run 8. Selection of the best alternative from among these and other potential solutions should be based on economic considerations; labor, reservoir construction, pumping plant facilities, etc. Effective operation of these systems requires a matching of runoff to return-flow pumping rate. Monitoring of runoff volume can be used to adjust set duration and/or pumping rate to assure efficient use of the runoff (i.e., a balancing of Equation 14.26).

Table 14.8. Examples of solutions for tailwater recovery system design.

Run	Q_S (L/s)	d_{req} (mm)	Q_f (L/s)	t_{co} (min)	RO (%)	Q_{P_max} (L/s)	Q_P (L/s)	n_S	n_P	N	RU (%)	PE_a (%)	T (hrs)	V_R (m^3)	$V_{RO\text{-}lost}$ (m^3)
1	80	80	1	874	28.3	31.6	31.6	80	32	7.2	28.3	85.1	104	1188	0
2	80	80	1	874	28.3	31.6	20	80	20	8	17.9	74.3	117	752	3480
3	82	80	1	874	28.3	32.4	32	80	32	7	28.0	84.7	102	1203	96
4	80	83	1	922	30.4	35.0	34.0	80	34	7	29.6	85.2	108	1309	256
5	80	68	1	713	20.1	20.1	20	80	20	8	20.0	79.5	95	684	21
6	80	80	1.2	720	34.3	41.8	40	67	33	8	32.8	85.9	103	1135	510
7	80	80	0.97	890	27.5	30.2	17.6	82	18	8	15.9	73.1	119	681	3952
8	80	85	0.97	973	31.0	35.9	31.4	82	32	7	27.1	81.9	114	1265	1268

14.4 BORDER-STRIP IRRIGATION

With border-strip irrigation, application efficiency typically is reduced when advance is either too fast or too slow. Thus design needs to provide a layout such that inflow rate and time can be adjusted within reason to provide satisfactory performance. 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

$$q_{in_{max}} = 0.00018\ S_0^{-0.75} \tag{14.28}$$

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:

$$q_{in_{min}} = \frac{0.000006\ L\ S_0^{1/2}}{n} \tag{14.29}$$

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 14.29 for S_0. This does not consider erosion potential.

14.4.1 Design of Open-Ended Border Strips

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 indicated that for most situations when near the peak potential efficiency, the minimum depth was at the downstream end. Even when the minimum was at the upstream end, the range 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. Since, in addition, estimates for potential efficiency are needed, recession lag-time design (or design based on satisfying the requirement at the upstream end, as in USDA [1974]) is no longer recommended.

Simple volume-balance procedures can be used for border strip design, as was done for furrow design, under the assumption of a surface shape factor. For sloping borders, we can assume that the flow depth at the upstream end approaches normal depth. The normal depth can be computed from Equation 14.8 with q as the unit inflow rate and S_f set to the bottom slope, S_0. Solving for y gives

$$y_0 = \frac{q_{in}^{3/5} (n / C_u)^{3/5}}{S_0^{3/10}} \tag{14.30}$$

As with furrows, $V_y = \sigma_y y_0 W$, where $\sigma_y \approx 0.7$. The remaining problem in solving the volume balance is computing the recession curve, where the recession time varies with distance.

The recession time at the downstream end is set so that the required depth is just satisfied there, as in Equation 14.12. An empirical relationship, adapted from Walker and Skogerboe (1987), is used to determine the cutoff time for this required downstream recession time. An approximate upstream recession time is found from

$$\Delta t_R = t_R(L) - t_R(0) = \frac{0.666\, n^{0.4757}\, S_y^{0.2074}\, L^{0.6829}}{I^{0.5244}\, S_0^{0.2378}} \tag{14.31}$$

where all units are in meters and seconds, and S_y is

$$S_y = \frac{1}{L}\left[\frac{(q_{in} - IL)n}{S_0^{0.5}}\right]^{0.6} \tag{14.32}$$

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 14.31 is essentially an empirical fit to 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

$$t_{co} = t_R(0)_S - \frac{y_0 L}{2 q_{in}} \tag{14.33}$$

Solution of Equations 14.31 and 14.32 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 14.31 is needed for the procedure used to compute cutoff time, it does not give a very realistic estimate for the actual upstream recession time at small slopes. The following equation can be used to estimate the upstream recession time (adapted from Hart et al., 1980):

$$t_R(0) = t_{co} + \frac{q_{in}^{0.2} n^{1.2}}{\left[S_0 + \left(\frac{0.345 n q_{in}^{0.175}}{\tau_{req}^{0.88} S_0^{1/2}} \right) \right]^{1.6}} \tag{14.34}$$

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 14.34 generally gives the best estimate (thus it is recommended over Equation 14.33). 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.

Once the advance and recession curves are computed, the infiltrated volume and runoff volume can be determined. The above procedures give estimates for 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, the following procedure can be used.

Compute the slope of the recession curve from

$$\frac{d t_R}{dx} = \max\left[\frac{3 \sigma_y y_n}{q_{in}}, \frac{t_R(L) - t_R(0)}{L} \right] \tag{14.35}$$

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 14.35 is just a straight line between the recession time, $t_R(L)$ computed from Equation 14.12 and $t_R(0)$ computed from Equations 14.31 and 14.32. The first term assumes that recession progresses at a rate which removes the surface volume, linearly, at one-third the inflow rate.

Example 4. Consider the same conditions as in the sloping-furrow design example, but with the infiltration function from Table 14.3, with k = 40.1 mm/hr^a and a = 0.51. With this infiltration function the infiltration opportunity time required to infiltrate 80 mm is 232 min. The other particulars are Manning n = 0.15, σ_y = 0.75, and q = 2.5 L/s/m. With a slope of 0.002 m/m the maximum and minimum recommended flow rates from Equations 14.28 and 14.29 are 19 L/s/m and 0.7 L/s/m, respectively. The same assumed surface-volume method is used to compute advance, as shown in Table 14.9.

Table 14.9. Advance calculations for assumed surface-volume border-strip design example.

Q (L/s/m)	x (m)	t_x (min)	$V_{in}(t)$ (m^3/m)	y_0 (mm)	$V_y(t)$ (m^3/m)	h	σ_z	$V_z(t)$ (m^3/m)
2.5	200	108.6	16.29	56.8	8.52	1.38	0.716	7.77
2.5	400	282.0	42.30	56.8	17.03	1.38	0.716	25.28

The desired recession time at the downstream end from Equation 14.11 is $t_R(L)$ = 232 + 282 = 514 min. The cutoff time is computed from Equations 14.31 to 14.33 as follows. Start by assuming that $t_R(0)_S$ = 514 min. Then compute the infiltration rate at the downstream end of the field with $\tau_{opp}(L)$ = 232 min from Equation 14.4, $I(L)$ = 10.5 mm/hr (or 2.93 × 10^{-6} m/s). Estimate the infiltration rate at the upstream end equal to the infiltration rate at $t_R(0)_S$ from Equation 14.4, $I(0)$ = 7.14 mm/hr (or 1.98 × 10^{-6} m/s). Next compute S_y from Equation 14.32 (with I = 2.45 × 10^{-6} m/s), as

$$S_y = \frac{[(0.025\,\text{m}^2/\text{s} - 2.45\times10^{-6}\,\text{m/s}\times400\,\text{m})0.15]^{0.6}}{400\,\text{m}\times0.002^{0.3}} = 0.000105$$

From this compute the upstream recession time from Equation 14.31 as

$$t_R(0)_S = 514 - \frac{0.666\times0.15^{0.47565}\times0.000105^{0.20735}\times400^{0.6829}}{(2.45\times10^{-6})^{0.52435}\,S_0^{\,0.237825}}\,\frac{1\,\text{min}}{60\,\text{sec}} = 514\,\text{min} - 150\,\text{min} = 364\,\text{min}$$

which is different from our assumed value. We recalculate the average infiltration rate with $\tau(0)$ = 364 min and $\tau(L)$ = 364 – 282 = 82 min, which gives us 8.5 and 17.6 mm/hr, averaging 13.0 mm/hr (or 3.61 × 10^{-6} m/s). Then recompute S_y and $t_R(0)_S$, until they converge. The final solution is I = 11.9 mm/hr, S_y = 0.0000903, and $t_R(0)_S$ = 386 min. (If we integrate the infiltration rate over eight intervals, the final solution is $t_R(0)_S$ = 376 min.)

In these iterations, only the average infiltration rate changes. The method assumes that recession starts at the upstream end after completion of advance. If this is not the case, then the infiltration at the downstream end is zero.

The cutoff time is determined from Equation 14.33 as

$$t_{co} = 386\ \text{min} - \frac{400\ \text{m}\times0.0568\,\text{m}}{2\,0.0025\,\text{m}^2/\text{s}}\times\frac{1\ \text{min}}{60\ \text{sec}} = 386 - 76 = 310\,\text{min}$$

The actual upstream recession time is found from Equation 14.34, as

$$t_R(0) = 310 + \frac{0.0025^{\,0.2}\times0.15^{\,1.2}}{\left[0.002+\left(\dfrac{0.345\times0.15\times0.0025^{\,0.175}}{(232\times60)^{\,0.88}\times0.002^{\,1/2}}\right)\right]^{1.6}}\times\frac{1\,\text{min}}{60\,\text{sec}} = 310+10 = 320\,\text{min}$$

Figure 14.3 shows the advance and recession curves for this example. The advance curves for the assumed surface-volume advance is a reasonably close match to advance from unsteady flow simulation (e.g., SRFR). The recession at the downstream end is well predicted by Equations 14.31 and 14.32. The recession time at the upstream end is well predicted by Equation 14.34. However, the recession curve itself is not all well predicted. Using a straight line from the recession lag time from Equation 14.33 (i.e., suggested by Walker) to the downstream recession time matches the curve fairly well except at the upstream end. Equations 14.34 and 14.35 are reasonable for this example, but are not satisfactory for other conditions. If the difference in the upstream recession time computed by the two methods is less than 10% to 15%, use the simplified straight line, otherwise the recession curve should be computed.

The volume applied for this example is 46.6 m^3/m, whereas the target volume is 32 m^3/m, giving an application efficiency of 68.7% (32/46.6). Use of Equations 14.34 and 14.35 for advance and recession computed over eight intervals (Table 14.10) resulted in deep percolation and runoff percentages of 14.4% and 16.9%, respectively. (The simplified procedure of Walker resulted in 13.3% and 18% for these two percentages.) Unsteady flow simulation with a cutoff time of 312 min gave these same percentages for deep percolation and runoff, however even with slightly more water applied, the target depth was not quite satisfied at the downstream end (E_a was 68.4% and minimum depth was approximately 79 mm). Either procedure produces reasonable results in most cases.

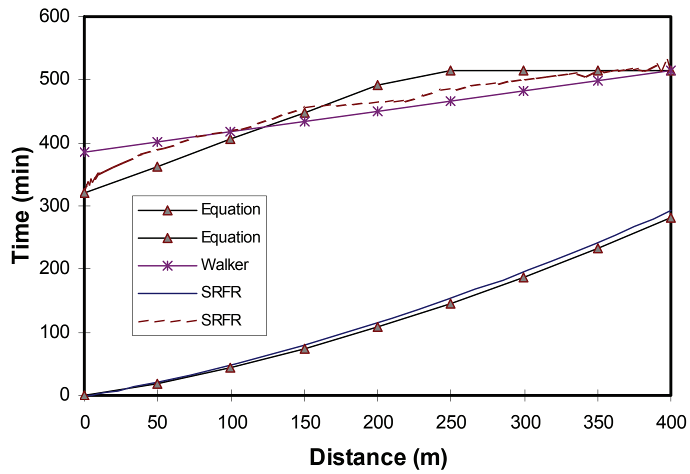

Figure 14.3. Advance and recession curves for border-strip design example.

Table 14.10. Advance and recession for border-strip design example.

Distance (m)	Advance Time (min)	Recession Time (min)	Opportunity Time (min)	Infiltrated Depth (mm)
0	0	321	321	94
50	20	363	344	98
100	45	406	361	100
150	75	448	374	102
200	109	491	382	103
250	146	514	368	101
300	188	514	326	95
350	233	514	281	88
400	282	514	232	80

14.4.2 Design of 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 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 it is 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

$$L_p = \sqrt{\frac{2V_{RO}}{S_0}} \tag{14.36}$$

where V_{RO} is the runoff volume. The ponded depth is zero at a distance L_p from the downstream end and is S_0L_p at the downstream end.

In the design procedures of the Soil Conservation Service (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.

Example 5. For the example given above, the runoff volume is 7.79 m^3/m. From Equation 14.36, the ponding length is 88.3 m, or 311.7 m from the upstream end. The additional depth of ponding at the downstream end is 177 mm. By interpolation from Table 14.10, the depth infiltrated at this point is 94 mm, much greater than the 80 mm required. Thus a significant decrease in application time can be made. The simplest approach for this design procedure is to decrease the recession time at the downstream end for the open-ended design procedure. If the decrease were proportionate to the minimum depth, the decrease in application time would be roughly 46 min (310 min × (94 – 80)/94 = 46 min). The required recession time at the downstream end is reduced from 514 to 468 min. The recession time at the upstream and downstream ends must now be recomputed, as described above, with the solution $t_R(0)_S$ = 352 min, t_{co} = 277 min, $t_R(0)$ = 287 min. The runoff volume is now 5.11 m^3/m, which results in a ponding length of 71.5 m, and a minimum depth infiltrated there of 84 mm. In this case, the minimum depth is still at the upstream end of the ponded section. In other cases it will be at the upstream end of the field.

After several iterations, the minimum depth matches the requirement with a reduction in open-ended downstream opportunity time of 62 min, with solution $t_R(0)_S$ = 342 min, t_{co} = 266 min, $t_R(0)$ = 276 min, V_{RO} = 4.33 m^3/m, and L_p = 65.8 m. The final distribution of infiltrated water is given in Table 14.11. The very large depth infiltrated at the downstream end, 200 mm, suggests that the slope on this field may be too steep for complete ponding. While the application efficiency in this case is 80.2%, this large ponded depth may be unacceptable.

Table 14.11. Advance and recession for ponded border-strip design example.

Distance (m)	Advance Time (min)	Open-Ended Recession Time (min)	Open-Ended Opportunity Time (min)	Open-Ended Infiltrated Depth (mm)	Ponded Infiltrated Depth (mm)
0	0	276	276	87	87
50	16	318	302	91	91
100	42	361	319	94	94
150	73	404	331	96	96
200	109	446	338	97	97
250	148	452	305	92	92
300	190	452	263	85	85
334	220	452	232	80	80
350	235	452	218	77	109
400	282	452	170	68	200

14.5 LEVEL-BASIN AND LEVEL-FURROW IRRIGATION

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. 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. With level basins, flow depth can become high and needs to be examined in design.

14.5.1 Design of 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. In order to solve the volume balance equation (Equations 14.13 and 14.14), they compute the upstream depth, y_0, for advance to some distance x from the Manning equation by assuming σ_y = 0.8 and by assuming

$$S_f = \frac{y_0}{x} \tag{14.37}$$

Combining Equations 14.8 and 14.37 and solving for y_0 gives the flow depth at the upstream end as

$$y_0 = (q_{in}^2 (n / C_u)^2 x)^{3/13} \tag{14.38}$$

For a given flow rate, advance time can be found iteratively from

$$t_L = \frac{V_y(x)}{Q_{in}} + \frac{V_z(x, t_x)}{Q_{in}} \tag{14.39}$$

where $V_y(L) = \sigma_y y_0 WL$ and V_z is found from Equation 14.15, with $x = L$ and is a function of advance time. The SCS (USDA, 1974) used a similar procedure, but ignored the last term in Equation 14.39. 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.

Yet to be determined are t_{co} and PE_{min}. The SCS assumed a relationship between advance ratio, $A_R = t_L/\tau_{req}$, and PE_{min}. With this assumption, they were able to generate a series of design charts (USDA, 1974). These charts plotted curves of PE_{min} as a function of unit flow rate (ordinate) and length (abscissa). However, Clemmens and Dedrick (1981) showed that PE_{min} varied widely with the advance ratio, particularly when the infiltration exponent varied from that assumed by the SCS intake families. Walker and Skogerboe (1987) assumed that after advance, the increase in infiltrated depth throughout the basin is the same as the required depth. This actually results in a very conservative estimate of PE_{min}.

However, if we assume that recession occurs at the same time throughout the basin, we can integrate the infiltrated volume over distance. A direct solution is not possible, but an approximation can be obtained by representing the power advance function with a series expansion. Taking the first two terms in the expansion results in the following equation for the final infiltrated volume:

$$V_z = LW\left\{c + k\tau_{req}^{a}\frac{[(1+hA_R)^{a+1}-1]}{hA_R(a+1)} + b\tau_{req}\left(1+\frac{hA_R}{1+h}\right)\right\} \tag{14.40}$$

However, if we use the branch infiltration function, we get an exact solution, provided that $\tau_{req} > \tau_B$. The resulting equation is

$$V_z = LW\left\{d_{req} + \frac{bht_A(L)}{(h+1)}\right\} \tag{14.41}$$

The cutoff time is found by dividing this volume by the inflow rate, Q_{in}. For very small values of AR, this equation is not appropriate, however in such cases the cutoff time can be based solely on required volume (i.e., PE_{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.

Example 6. The assumed surface-volume method given above was applied to the design example for border strips, except that the slope is zero. (Such light application depths are more difficult to apply with level basins unless the length is significantly reduced. Variations in field surface elevations also play a role in potential efficiencies if attempts are made to apply too little water.) The length is reduced to 200 m and the flow rate to 2.0 L/(s m). The surface shape factor is 0.80. The advance curve computed with the assumed surface-volume method (Equations 14.38 and 14.39) is given in Table 14.12.

From Equation 14.40, the total volume applied is 20.23 m^3, resulting in a cutoff time of 169 min and PAE_{min} = 79.1%. If we assume that recession occurs at the time required to infiltrated the desired depth at the far end (195.4 + 232.4 = 428 min), opportunity times at various points along the basin can be computed by subtracting the advance time from this recession time, as shown in Table 14.13. Using these computed points to estimate the volume infiltrated gives (with eight-point numerical integration) PAE_{min} = 80.5%, suggesting that Equation 14.40 is a reasonable estimate for the required volume. (Walker and Skogerboe's method gives PAE_{min} = 60.1%.)

Table 14.12. Results for assumed surface-volume level-basin design.

Q (L/(s m))	x (m)	t_x (min)	$V_{in}(t)$ (m^3/m)	y_0 (mm)	$V_y(t)$ (m^3/m)	h	σ_z	$V_z(t)$ (m^3/m)
2.0	100	72.2	8.66	68.5	5.47	1.44	0.723	3.18
2.0	200	195.4	23.45	80.4	12.86	1.44	0.723	10.58

Table 14.13. Advance and recession for level-basin design example.

Distance (m)	Advance Time (min)	Recession Time (min)	Opportunity Time (min)	Infiltrated Depth (mm)
0	0	428	428	109
25	8	428	420	108
50	21	428	407	106
75	38	428	390	104
100	58	428	370	101
125	80	428	348	98
150	104	428	324	95
175	131	428	297	91
200	195	428	232	80

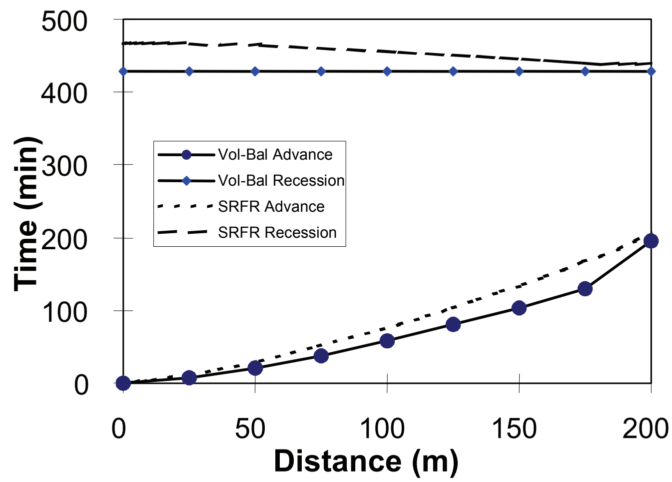

Figure 14.4. Advance and recession curves for level-basin design example.

For this example, the advance time exceeds the application time. This assumed surface-volume method likely overestimates PAE_{min}. The BASIN design program for this set of conditions gives PAE_{min} = 78.4%, $t_A(L)$ = 211 min, t_{co} = 170 min, and cutoff at advance to 88% of the basin length (R = 0.88). This is close to the limit line presented by Clemmens and Dedrick (1982), which occurs at approximately R = 0.85, and beyond the limit in BASCAD for design. While the differences are small in this case (2% change in PAE_{min}), errors increase rapidly as cutoff occurs at shorter advance distances. A comparison of advance and recession curves for the assumed surface-volume method and unsteady-flow simulation are given in Figure 14.4.

14.5.2 Design of 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 14.38 is replaced with (from Equations 14.7 and 14.37)

$$A_0^2 R_0^{4/3} y_0 = q_{in}^2 (n / C_u)^2 x \tag{14.42}$$

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 are quite different.

Example 7. The infiltration and roughness conditions for the sloping-furrow irrigation example are used here: k = 40 mm/hr^a, a = 0.35, and n = 0.05. The furrow spacing is 1 m, and as before, the furrow bottom width is 100 mm with 2:1 side slopes (horizontal:vertical). In this case, we start the design assuming 2 L/s per furrow and a length of 200 m.

The design starts with a solution for the depth at the upstream end as a function of distance x, from Equation 14.42. This relationship does not depend on the advance time. At 100 m, we guess a depth of y_0 = 0.1 m. For a trapezoidal shape, this results in an area A_0 of 0.0485 m^2, a wetted perimeter of 0.6935 m, and a hydraulic radius R_0 of 0.0699 m. Solving Equation 14.42 for y_0 gives 0.0148 m. This new estimate of y_0 does not provide a good value for the next guess (i.e., the solution diverges). Instead, we use 0.8 times the original guess plus 0.2 times the value computed from Equation 14.42; 0.8(0.1) + 0.2(0.0148) = 0.1091 m. After three iterations, the solution converges to 0.0891 m. At 200 m, solution with this procedure gives y_0 = 0.1012 m.

The surface volume for advance to these two distances is found from Equation 14.14, with σ_y = 0.80, as 1.983 and 4.897 m^3, respectively. The subsurface volumes are a function of time and the advance exponent. Initial guesses of 60 and 250 min give h = 1.32, and from Equation 14.15, subsurface volumes of 3.107 and 4.897 m^3, respectively. The inflow volumes at these times are 7.2 and 18.0 m^3, respectively, or volume errors of more than 20%. From Equation 14.39, new estimates of advance time are 42 and 122 min, respectively. The solution eventually converges to 39 and 104 min, respectively, h = 1.415 and an advance ratio A_R = 0.239.

From Equation 14.40 with a target depth of 80 mm, the ultimate infiltrated volume is 16.89 m^3, which occurs at a cutoff time of 140.7 min, or approximately 37 min after the completion of advance. The potential application efficiency for this case is 94.7%. Numerical integration with eight points on the advance and recession curve, assuming recession occurs at the time needed to infiltrate 80 mm at the downstream end, gives an infiltrated volume of 16.75 m^3, an application time of 139.5 min and a potential application efficiency of 95.6%. Clearly in this case, either a longer basin or a smaller application depth can be applied. Further application of this procedure at different lengths, furrow flow rates, and application depths should be explored to arrive at an economic design.

14.6 SURFACE IRRIGATION SYSTEM HEADWORKS AND CONTROL OF INFLOW

The headworks for a surface irrigation system are used to convey and distribute water from the supply point to individual irrigation sets, furrows, etc. They provide a transition from the supply to the field, and must be compatible with both the type of supply available and the type of surface irrigation system in use. There are two main types of supply: from a farmer-operated groundwater well and from an irrigation district. The irrigation district supply may be from a pressure pipeline or from an open canal or pipeline. Flow from a groundwater well is typically constant and non-adjustable. Flow from an irrigation district with open canals or pipelines is typically variable and not under control of the irrigator. Flow from an irrigation district pressure pipeline may be adjustable by the irrigator (e.g., by applying backpressure on the outlet).

Farm field conveyance and distribution works may be a pipeline or an open ditch. Water from groundwater can be pumped into a field pipe distribution system or an open canal. Where sufficient head is available, water from canals and open pipelines can also supply field pipe distribution systems (e.g., gated pipe). However, debris must be kept out of the pipeline since it could clog outlets. Turbulent fountain screens are an effective way to achieve this (Bondurant and Kemper, 1985).

Where earthen farm ditches are used, water can be distributed to individual furrows with siphon tubes or spiles; or to individual borders or basins through siphon tubes, pipe outlets (such as large spiles that are closable) or cuts in the canal wall. For concrete ditches, options for water distribution include siphon tubes (for all methods), pipe outlets (for borders and basins), or open gated outlets (e.g., with jack gates). For farm pipe conveyance and distribution systems, alfalfa valves are common for borders and basins, while gated pipe systems are most common for furrows.

The selection from among these various alternatives should be based on compatibility with the supply and water control to the irrigated field. This latter must consider the available head of the water above the field surface, which may change substantially over the farm, so that water will flow out of the distribution system into the desired field area and not elsewhere. Systems should typically have 0.15 to 0.3 m of head available at all points. Outlets should also be non-erosive. This can be a severe problem with steep slopes and erodible soils, or where very high flow rates are used (e.g., > 300 L/s).

The headworks must often supply fields that may irrigate both row and field crops, such that, for example, both furrow and border-strip irrigation are practiced on the same field. The headworks should not limit this possibility.

Selection of headworks is often dictated by common practices in the area and economics. However, modernization or improvement in surface irrigation practices usually requires better control of inflow and may suggest other alternatives. Control of inflow rate is necessary to help balance advance and recession curves and to minimize runoff that is lost. Is some cases, some form of automation will help improve water control.

Cutback furrow irrigation has not been widely practiced because of the labor involved in making effective cutback. Several methods have been developed to accomplish cutback automatically or semi-automatically. Surge flow was developed in attempts to apply cutback flow to gated pipe. With these systems, if the pipe gates are set to give the desired distribution at full flow (i.e., gate openings vary due to slope, wheel vs. non-wheel row, etc.), reducing the pipe flow in half will not give each furrow half of its previous flow. By cycling the full flow at a relatively short interval (e.g., 10 min), a more effective cutback is achieved. Automatic surge valves that cycle water have made surge flow, and the associated cutback, a practical method for improving sloping-furrow irrigation performance.

Cablegation is another method of achieving an effective cutback flow. Under these systems, gated pipe is placed on a uniform slope. A plug moves at a constant speed down the gated pipeline. Immediately upstream from the plug, the outlets have essentially full pressure. Further upstream, outlets have a reduced pressure. By allowing the plug to move downstream, the flow gradually decreases, essentially producing a gradual cutback. Different flow distributions can be produced with different supply inflows and plug speeds (Kincaid, 1984). Automation only requires control over the plug speed. These systems are not in common use.

Cutback flows from field ditches was first proposed by Garton (1966), where spiles for subsequent irrigation sets are located at a lower elevation. When the next set is irrigated, the current set has a lower head on it, producing the cutback flow. These have not proven to be practical in the field, partly because of physical limitations (i.e., they need to have the correct cross-slope), their relative nonadjustability, and plugging of the spiles with debris. Canal side weirs or ditch notches have been used effectively in some areas. Here small weirs, one for each furrow, are made when the concrete ditch is poured. Flow into a set of furrows is controlled by the water surface elevation in the ditch. Blocking the canal gives full flow. Cutback flow can be achieved and controlled by unblocking the canal to allow most of the flow to continue downstream, but by maintaining the water level (e.g., with check boards). This system is also less susceptible to weed plugging. These systems have not seen widespread use due to the cost of construction. (Eftekharzadeh et al., 1987).

Improved management of borders and basins usually requires selection of the right flow rate and duration. With level-basin irrigation, the tendency has been to increase the flow rate to speed advance and reduce labor costs. Level basins as large as 15 ha with flows above 1 m^3/s have been used. Outlet erosion control becomes a major issue for such systems. With fixed-width borders and basins, flow rate adjustments are not always feasible, making control of duration the only operational control over performance. A number of methods have been proposed for the automatic control of duration to such systems, including automatic control of jack-gates, pipe outlets, and drop-open or drop-closed gates within a concrete-lined supply canal. Most of these use simple timers to move water from one set to another. Field sensors have also been developed to sense the arrival of water and signal the change in irrigation set (e.g., based on advance distance). These sensors are also not in common use.

Drain-back level basins were developed to reduce many of the limitations of the more traditional, modern level basins used in southwestern Arizona. With these systems, an earthen ditch is placed below the field grade. When the ditch is blocked, the water level rises and simply flows into the field. When the ditch is opened, the supply water is conveyed past the field, below its surface elevation, to the next field. In addition, some of the applied water drains back off the field just irrigated, thereby reducing the applied depth. The conveyance ditch is used as the turn row. This system significantly reduces the cost of level-basin construction (i.e., no concrete ditch to construct), makes lighter water applications more feasible, and places less restriction on machinery operations (i.e., one can drive though ditches from one basin to the next) (Dedrick, 1984).

More recently, similar systems have been utilized for level-basin irrigation in high rainfall areas, including paddy rice production. A common feature is below-grade ditches on one to three sides. These replace the contour levee system, in that rather than water flowing from levee to levee through the field, the water flows in the side ditches from levee to levee. This actually promotes both more efficient irrigation and fastest surface drainage (Clemmens, 2000).

The headwork facilities are an extremely important aspect in system design since they often represent a major part of the development cost for a surface irrigation system. The surface irrigation design procedures presented herein should be used in conjunction with an economic analysis of all of the factors that influence the cost of development and operations. Consideration should also be given to the future pressure to reduce water applied and environmental protection. Designs that potentially allow better water control offer more flexibility to meet these changing demands. More details on these the considerations in irrigation system selection can be found in Burt et al. (1999).

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