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Simple approach to surface irrigation design: Spreadsheet applications [1]

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Abstract
A companion paper provides equations and procedures for the analysis of surface irrigation systems as an aid to design. These procedures use continuity to determine advance, based on an assumed surface shape factor. Recession in border strips requires empirical expressions, while simple procedures can be used to estimate recession in sloping furrows, level basins and level furrows. The sloping furrow method is extended to include cutback or reuse. The sloping border method is extended to include a blocked end. In this paper examples are provided to demonstrate the application of these design procedures. Performance contours are provided to show the types of solutions that can be developed. Spreadsheets to perform these calculations are available on the Sakia.org e-journal web site http://www.sakia.org/ejlw

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

1. Introduction

In a companion paper (Clemmens 2007), a series of simple procedures were presented for calculating the advance, recession, and performance of surface irrigation systems. The equations are included in Appendix I for completeness. These equations form the basis for design analysis, however they do not directly provide design recommendations. In design, one has to have a larger view of how the system can be adapted to different cropping systems, how the irrigation system interfaces with other farm operations, what labor is required and available, economic viability, and many other considerations. A more detailed discussion of these topics is included in a Burt *et al.* (2000). In this paper, the Clemmens (2007) equations are applied to design examples with the aid of spreadsheet programs.

2. Design considerations

2.1 Design objectives

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

The design objectives are typically stated in terms of achieving some desired application efficiency, *AE*. This efficiency is called potential application efficiency, *PAE*, here to distinguish it

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from a field measured *AE*. Design is typically based on supplying the target depth of water everywhere in the field. In practice, some under irrigation is usually allowed and operation is based on satisfying the low-quarter depth. Since design does not take into account all of the variability which exists within the field, design is based herein on satisfying the minimum depth, with the expectation that when operated the low quarter depth would be satisfied.

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.

It is recognized that soil properties vary spatially. The design should attempt to put soils with similar properties within the same irrigation set or bay. Design often uses average conditions over an area and it is recognized that irrigation performance will degrade when soils are highly nonuniform. Some adjustments can be made in the layout (e.g. different basin length or different border widths) or during the irrigation event (e.g., different furrow flow rates). Design should attempt to minimize the impact of this variability on performance.

2.3 Flow resistance

In theory, the Manning *n* should change with flow depth when vegetation is present. However, the changes in Manning n over the season and with changes in planting density are as large as changes caused by flow depth. For design purposes, the Manning *n* seems to be satisfactory. Published values of Manning *n* for various crops are shown in Table 1.

Table 1:
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 (1991a)	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

Manning *n* values have been observed as low as 0.03 on smooth furrows and as high as 0.4 on closely planted wheat, a value significantly higher than that given in the table. Also, furrow irrigated

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crops with vegetation which hangs into the water can cause much greater resistance than the Manning *n* of 0.04 given in Table 1. Here again, field observations of advance and recession can be used to determine whether design estimates of Manning *n* are reasonable. Field measurements of water depth can also be used to verify Manning *n* values. However, undulations in field slope and the difficulty of making such measurements, except near the field edges, make field measurements somewhat unreliable for routine management.

2.4 Infiltration

Infiltration is one of the most important factors affecting the design and performance of surface irrigation systems. Unfortunately, estimating infiltration conditions is one of the most difficult things to do in the field. Several equations have been used to define infiltration. Irrigation engineers have tended to use the Kostiakov Equation, or a variation thereof. This equation gives infiltration rates that approach 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 may be reached during the irrigation and should be included. Both early high infiltration and late constant infiltration suggest use of the modified Kostiakov or branch Kostiakov equation.

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 (USDA 1974). These families are defined according to Equation 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 which 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 2.

Table 2:
Time-rate intake families (from Merriam and Clemmens 1985).

Time to infiltrate 100 mm hours	Kostiakov Exponent (a)[1]	Kostiakov Constant (k) mm/hr[a]	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

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

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

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opportunity times. Table 3 shows the parameters for five infiltration functions, representing the various equations fit to infiltration described by the 6-hour Time-rated family (i.e., 6 hours 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 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 hours.

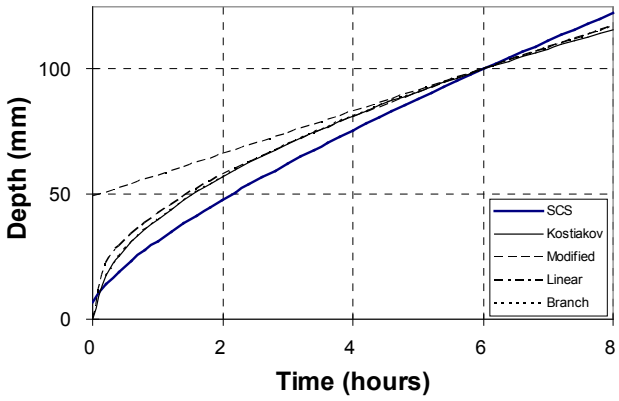

Figure 1:
Comparison of infiltration equations representing 6-hour family, given in Table 3

Table 3:
Infiltration functions matching 6 hour Time-rated family.

Parameters	SCS Equation	Kostiakov Equation	Modified Kostiakov	Kostiakov Branch	Linear Equation
k (mm/hr[a])	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

2.5 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. The SCS design procedures for sloping furrows (USDA1984) 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. The width chosen is 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.

In this paper, infiltration constants are applied to the furrow spacing, assuming one-dimensional flow, as in borders and basins. The designer must assure that the infiltrated volume per unit length is appropriate for the field conditions. When compared to standard SCS design practice, the infiltration constants (*k*, *b*, and *c*) may be proportionately in error due to differences between furrow spacing and SCS computed wetted perimeter.

3. Design approach

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.

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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. These adjustments are typically 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.

Irrigation system design is a complex process that includes economics, water supplies, crop selection, etc. See Burt *et al.* (2000) for additional discussion on these issues. Property boundaries, soil texture and its variation, and topography have a significant influence on surface irrigation design. Costs for land grading and construction of canals are a major consideration. The design becomes a compromise between the best dimensions and layouts for each soil texture, how the fields fit within the physical constraints, and economics. The design procedures presented here provide the designer with a relatively quick method for exploring alternatives. Final selection of a specific design includes far more issues than can be covered here.

The examples given in the next section utilize the design procedure discussed in the companion paper (Clemmens 2007). The relevant equations are given, briefly, in Appendix I.

4. Spreadsheet implementation of design equations

The design equations and procedures have been programmed into spreadsheets for simple calculations. These include files *(available on-line)*

- *http://www.sakia.org/ejlw_2007_01_02_furrow_design.xls*,
- *http://www.sakia.org/ejlw_2007_01_02_border_design.xls* and
- *http://www.sakia.org/ejlw_2007_01_02_basin_design.xls*.

These spreadsheets have been set up so that each type of data input is color coded. Input data required for the procedure has a blue/aqua background. Many of these solutions require iterations through trial and error. Cells that require estimates for the trial and error procedures have light green backgrounds. Additional cells are provided that need to be made equal to zero through the iteration process. These cells have red backgrounds with bold letters. The user needs to change the values in the light green cells until the red cells are zero, or reasonably close. There are several cases where the convergence of one light-green/red iteration pair influences another iteration pair. For example, the two advance times can each be iterated on separately, but these change the advance exponent which changes other calculations. There are a few other cases where this occurs, which will be discussed in the examples below. More details on the calculation procedures are provided for the sloping furrow example since it is given first. Following through the examples with the attached spreadsheet should help with understanding the calculations required and the impact of various design choices.

5. Design example

5.1 Design data

Consider a 72 ha field with the following properties:

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- Slope in direction of length = 0.002 m/m
- Width = 600 m
- Slope in direction of width = 0.000 m/m
- Available flow rate, Q = 200 *l*/s

An evaluation of options for field design will be provided for sloping furrows, sloping border strips, level basins and level furrows. Soils are assumed to be uniform. Sloping irrigation systems will be oriented down the existing slope. Level irrigation systems will be oriented across the slope, in benches. In all cases, the length of run for irrigation will be less than the field dimension, requiring that the field length be broken up into different irrigation units of equal size. Both flat planted and furrowed crops should be accommodated by the design, regardless of whether the fields are sloping or level (i.e., same length of run for sloping furrows or border strips).

For simplicity, it is assumed that the target application depth is 90 mm. For furrows, the following geometry is assumed

- furrow spacing, W_f = 1 m
- furrow bottom width, W_b = 100 mm
- furrow side slope, Z_b = 2:1 (horizontal:vertical)

Infiltration in border strips and basins is assumed to follow a 6 hour intake family (Table 3), with parameters k = 40.1 mm/hr^a and a = 0.51. Infiltration in furrows is expected to be lower due to a wetted perimeter that is much less than the furrow spacing. If we assume the effective wetted width is ¾ of the furrow spacing, then the infiltration parameters are k = 30.1 mm/hr^a and a = 0.51, when expressed on the basis of a 1 meter furrow spacing. For level furrows, the infiltration is assumed the same as for level basins, that is, the effective wetted width is the same as the furrow spacing.

In furrows, a Manning n of 0.04 is used, and for border strips and basins, design must consider both n = 0.04 and 0.15, assuming these conditions will both occur during the growing season. For furrows and border strips, the surface shape factor will be assumed as σ_y = 0.75. For level systems σ_y = 0.80.

5.2 Sloping furrows

5.2.1 Standard inflow

The design process starts by assigning a length and flow rate to each furrow. This calculation procedure is followed for a series of values for the length and flow rate. These values are compared in making design choices. It is initially assumed that the overall field is broken into three irrigation lengths of 400 m. The field is also divided into three widths, the 200 *l*/s inflow rate would provide 1 *l*/s per furrow. The steps are outlined below.

1. Calculate normal depth. Equation 2 is solved for depth. Since area and hydraulic radius are a function of depth, this is an iterative solution. See worksheet "*Furrow*" for specific details. The manual trial-and-error solution converges to a normal depth of 0.048 m and an upstream cross sectional flow area of 0.00932 m^2. In this case, the furrow wetted perimeter is 0.313 m, or roughly 30% of the furrow spacing, while the infiltration was chosen as roughly 75% of the border value. Whether or not this is a reasonable approximation depends on the soil. At higher furrow flow rates, the depth and wetted perimeter will be greater, whereas no adjustment for this is made in this example.
2. Calculate the opportunity time required to infiltrate the desired application depth. Solving Equation 4 for opportunity time gives 514 minutes.

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3. Compute advance times to 200 and 400 m. Since surface volume during advance is a function of distance and not time, the surface volumes can be computed directly from the first term on the right-hand side of Equation 12. Several methods can be used to solve for advance times from the combination of Equations 11-16. The one provided in the spreadsheets is for users to guess initial values of the two advance times. From these, the spreadsheet calculates the inflow volume (left-hand side of Equation 12), the advance exponent (Equation 16), the subsurface shape factor (Equation 15), the subsurface volume (Equation 13), the total volume (Equation 11), and the volume error, or difference between the inflow volume and the total volume (surface plus subsurface). New estimates of advance time are computed as the total volume divided by the inflow rate. The user can enter these new estimates as the next guess for advance times, and continuing to iterate. For this example, advance times to 200 and 400 m converge at 141.1 and 491.9 minutes, respectively. The significant digits shown here are not meant to imply accuracy of the calculation, but rather, a sufficient number of significant figures are required to assure proper convergence. The other results are shown in Table 4.

Table 4:
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**
User input	User input	Trial & error	Eq. 11 & 12	normal area	Eq. 11 & 12	Eq. 16	Eq. 15	Eq 13
1.0	200	141.05	8.46	0.0093	1.40	1.80	0.759	7.07
1.0	400	491.96	29.52	0.0093	2.80	1.80	0.759	26.72

Since there are two advance times that must be estimated, and they influence each other through the advance exponent, automatic convergence routines for a single cell are cumbersome. An alternative solution procedure was developed that provides easier convergence, although it requires circular references. Rather than iterating on the advance times, the spreadsheet can be set up to iterate on the advance exponent. Iteration on the advance time is done through a circular reference, based on the user input advance exponent. The advance time results provide a new estimate for advance exponent. The solution converges rapidly. With this approach, automatic calculation of cell formulas is turned off so that these circular references will not cause difficulties. The user needs to remember to repeatedly force calculation (F9 key in excel) until the solutions converge. Some spreadsheet software programs do not allow circular references, so this method is not used in the spreadsheets provided.

4. If recession is assumed to occur everywhere at once, the cutoff time is simply the advance time plus the required opportunity time, or 1006 min, (16.8 hrs.). The application time can be adjusted to take recession into account with Equation 17. This gives a time of 959 min. (16.0 hrs.) and an infiltrated depth at the upstream end of 124 mm. Recession at the downstream end is still at 1006 minutes, which gives 90 mm infiltrated.
5. The advance curve is used to construct advance at eight equal increments of length. Recession times are interpolated between the values at the ends of the field. Infiltration opportunity times are computed as the difference between recession and advance times (Equation 7), from which infiltrated depths are computed (Equation 5). The infiltrated depths

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are averaged over each distance interval, converted to volume by multiplying average depth by the length and width, and then summed to give total volume infiltrated. The runoff volume is computed as the difference between the inflow volume and the infiltrated volume. Advance, recession, and the distribution of water are shown in Table 5.

Table 5:
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	959	959	124
50	12	965	954	123
100	40	971	930	122
150	84	977	893	119
200	141	983	842	116
250	211	988	778	111
300	293	994	701	106
350	387	1000	613	99
400	492	1006	514	90

6. The infiltrated and runoff volumes are expressed in terms of an equivalent depth infiltrated, 113 and 31 mm, respectively. The potential application efficiency is computed as the required depth divided by the total depth applied (infiltrated plus runoff), which for this example was 62.6%. The infiltrated and runoff volumes are converted to fractions of applied water, 0.784 and 0.216, respectively. The usefully infiltrated fraction is simply the potential application efficiency expressed as a fraction. The deep percolation fraction is the difference between the infiltrated fraction and usefully infiltrated fraction, or 0.158.

These results fully describe the design results for the selected length and flow rate. Solution to the design problem requires that these results are obtained for a range of lengths and flow rates. If the 1200 m field is divided into equal increments, practical irrigation lengths could be 200, 240, 300, 400, and 600 m. The width of the field also must be in even increments, say 100, 120, 150, 200, 300, and 600 m. This gives furrow flow rates of 0.33, 0.67, 1, 1.33, 1.67 and 2 l/s. The normal depths for these flow rates for these furrows are 0.0272, 0.0389, 0.0477, 0.0549, 0.0611, and 0.0666 m, respectively. Note that these variables are not continuous, but are discrete variables based on the need to fit equal lengths within the overall field area. The user is responsible for making sure that the furrow length and flow rate are appropriate. The performance results for this set of conditions are shown in Figure 2. Tabulated results are given in

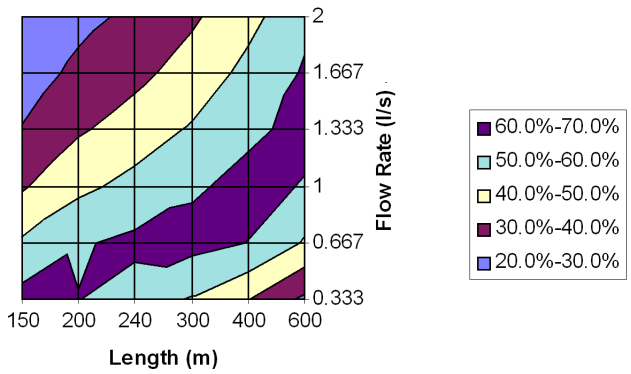

Figure 2:
Performance (PAE_{min}) contours (not to scale) for sloping furrow example.

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the spreadsheet. To aid in the process of recording the performance results for each combination of variables, an additional table is provided at the bottom of the spreadsheet. The row with data provides the results from one design run. This can be copied to another worksheet as values, from which a series of results can then be analyzed or graphed. It is interesting that near the highest efficiencies, runoff and deep percolation volumes are almost equal. The odd shape of the contours in Figure 2 result from interpolation with limited data.

5.2.2 Cutback flow

Potential efficiencies with sloping furrow for the conditions similar to this example are never very high. Cutback flow and tailwater reuse can improve overall performance. For this example, it is assumed that the inflow can be cut back to 50%. Advance is computed as in steps 1 to 3, above. In addition to computing the normal depth and upstream flow area at the inflow discharge, the upstream flow area at the cutback discharge is also needed, since this is used in our recession-time adjustment. Equation 19 or numerical integration is used to determine the time at which infiltration is reduced to half of the inflow. Iteration with Equation 20 (an approximate solution) gives 850 minutes. Iteration with numerical integration (the preferred approach) gives 935 minutes. Adjusting for the volume of water on the field (Equation 20) gives a cutback time of 898 minutes. Computing a new application time from Equation 17 with the cutback flow and volume gives an application time of only 950 minutes. Because of the small difference in time between cutback and cutoff, cut back to 50% of the inflow is probably not feasible for this combination of soils, length, flow rate, etc. since it would occur for only the last 5% of the application time. The potential application efficiency only changed from 62.6% to 64.9%. Runoff was only reduced from 21.6% to 18.9%. Other combinations of conditions can result in substantial reductions in runoff volume.

5.2.3 Tailwater reuse

An alternative to cutback flows for reducing runoff losses is to capture the tailwater and reuse it. This section discusses the design of runoff reuse systems that recycle water at a constant rate, as discussed in Clemmens (*2007*). Runoff is recycled on the same field with limited storage capacity.

For the above example, runoff volume is 21.6% or inflow. The maximum pumpback rate, based on the available runoff volume, is 55.2 *l*/s (Equation 25). With a 200 *l*/s supply, 1800 furrows (600 in each of three sections), and 1 l/s per furrow, the field without runoff reuse is divided into nine segments. This design also requires that pumpback must fit within the constraint of equal widths. With eight irrigation sets, only 25 l/s in pumpback flow is needed. Under this scenario, less than half of the runoff would be pumped back. For this analysis, the runoff not pumped would be lost, and the application efficiency would only increase to 70.4%. (Other strategies could utilize this excess tailwater, but this would take additional management). To divide the field into 7 irrigation sets, (7 does not divide exactly into 1800, one set has an extra furrow), a pumpback rate of 57 l/s would be needed, which is slightly more than is available. This illustrates the difficulty of developing a scheme that balances inflow and runoff to achieve a high efficiency. In the spreadsheet, the user is responsible for providing a pumpback flow that is feasible in terms of both balancing irrigation sets and being available from runoff.

The solution from a design standpoint is to alter the furrow flow rate and repeat the advance and recession calculations. Generally, a higher flow rate should be used to reduce deep percolation losses, since the runoff losses can be recovered. If advance and recession are solved at 1.5 l/s flow rate, the runoff volume is 37.6% of the inflow volume. The maximum allowable pumping rate is 120 *l*/s. Choosing 100 *l*/s for pumping (83% of available volume) allows the field to

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be divided into 9 irrigation sets. This raises the potential application efficiency to 83.4%. However, this requires substantial pumping, equal to one half of the inflow volume.

5.3 Sloping border strips

5.3.1 Open-ended border strips

This example starts by assuming a length of 400 m and a flow rate per unit width of 2 l/s/m. This gives a width of 100 m for the 200 l/s inflow.

1. The normal depth, found from Equation 29, is 0.50 m. This is a direct calculation, as opposed to the iterative solution required for furrows.
2. The required opportunity time determined from inverse solution of Equation 5 (or 4) is 293 min.
3. The advance times are determined iteratively with the same procedure above for furrows, resulting in 135.3 and 368.1 minutes, for the advance times to 200 and 400 m, respectively. Again, the number of significant figures does not imply accuracy, but is used to verify convergence. The results are shown in Table 6.

Table 6:
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
User input	User input	Trial & error	Eq. 11 & 12	Eq. 29	Eq. 11 & 12	Eq. 16	Eq. 15	Eq 13
2.0	200	135.3	16.23	49.7	7.45	1.44	0.724	8.79
2.0	400	368.1	44.17	49.7	14.90	1.44	0.724	29.28

4. For recession, first compute the recession time at the downstream end, with is the advance time plus the required opportunity time, 368+ 293 = 661 min. Then start with a guess of the upstream recession lag time, $t_R(0)$'. For this time, compute the infiltration rate from Equation 5, the friction slope associated with this procedure (Walker and Skogerboe 1987) from Equation 31, and a new estimate of upstream recession time from Equation 30. Iterate on this time until the estimated time matches the calculated time. For this example, the solution converges at 514.1 min.
5. Next compute the cutoff time from Equation 32, which is 431 minutes. And finally compute the upstream recession lag time (second term of right hand side of Equation 33), which gives an upstream recession time of 441 min. Note that this time is quite different from the upstream recession time from the Walker and Skogerboe procedure, $t_R(0)$'. This is not an inconsistency in the results. As discussed in the companion paper, the Walker and Skogerboe procedure for determining advance time at the downstream end is based on Strelkoff's rough estimate of upstream recession time. So that estimate of upstream recession time must be used to calculate the downstream recession time. However for computing the recession curve, the upstream recession time from Equation 33 is used as an end point.

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6. Once the recession times are determined at the end points, compute the recession curve from Equation 34, with the addition restriction that the recession time cannot be greater than the downstream recession time. (Note that the spreadsheet does not currently make the check that recession curve is above the straight line between the two end points, as Eq 34 implies). Note that the recession curve for this example is essentially flat for the last half of the field.

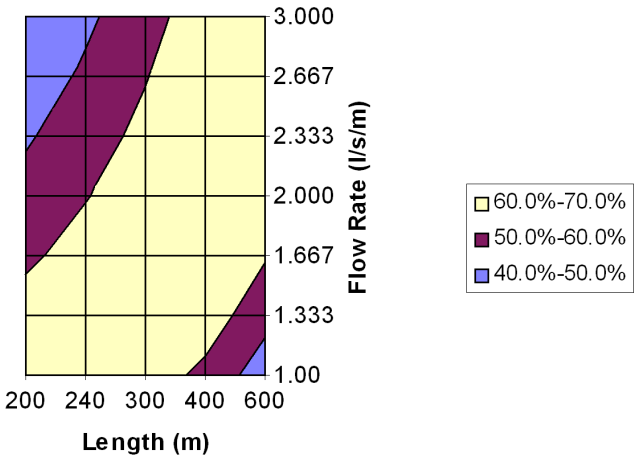

Figure 3:
Performance (PAE_{min}) contours (not to scale) for sloping border strip example.

7. Finally, the advance and recession curves are used with Equation 5 to compute the distribution of infiltrated depths (Table 7). The total infiltrated volume can then be calculated, as well as the runoff and deep percolation volumes. When expressed as a fraction of inflow for this example, these are 0.858 infiltrated, 0.142 runoff, and 0.163 deep percolation. The application efficiency is then calculated as 69.6%.

Table 7:
Advance and recession for border-strip design example (q_{in} = 2 l/s/m), n =0.15.

Distance m	Advance Time min	Recession Time min	Opportunity Time min	Infiltrated Depth mm
0	0	441	441	111
50	18	488	469	114
100	49	534	484	116
150	89	581	491	117
200	135	627	492	117
250	187	661	474	115
300	243	661	418	108
350	304	661	357	100
400	368	661	293	90

For this design scenario, designs must also be developed for different border strip length and width combinations. When this procedure is repeated, the results are shown in Figure 3. Tabulated results are given in the spreadsheet.

5.3.2 Blocked end (ponded) border strips

By blocking the runoff from leaving the field, less water needs to be applied. The calculations for blocked-end border strips start with the assumption that the required depth is satisfied at the downstream end. The amount of runoff is calculated, as above. Then the runoff water is assumed

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to be ponded on the downstream end of the field. The length of ponding is determined from Equation 35. For this example, the initial ponding length is 91 m, or ponding occurs from 309 m to 400 m from the head end. The depth of ponding at the downstream end is just the slope times the length, or 182 mm. The depth of ponding at any location in between goes from zero at 309 m to 182 mm at 400 m, linearly. It is assumed that this water infiltrates vertically. This assumption is good if the infiltration rate does not change significantly during the infiltration of this water, or infiltration is described by Equation 6 (constant infiltration rate).

The calculation procedure is to reduce the application time so that the minimum depth of water infiltrated is the required depth, here 90 mm. This can only be done by trial and error since the location of this minimum depth is not known. In the spreadsheet, a "ponding time" adjustment is made by adjusting the target opportunity time, which then modifies our calculations of recession time, and eventually cutoff time. This is easier than modifying the cutoff time directly since the recession times are used to compute the recession curve. When this ponding time is adjusted, the recession time is recalculated. Several iterations with the ponding time and recession time are needed before the solution converges. If for this example, the target opportunity time is reduced by 63.5 minutes, the upstream recession time (for Walker & Skogerboe curve, not actual) is 443.5 min, runoff (for downstream ponding) is 5.6%, ponding length is 49.4 m, ponding location is 350.6 m, and ponding depth is 99 mm. This gives 90 mm infiltrated depth at 350 m, just downstream from where ponding starts. An extra column is provided that computes an estimate of the depth infiltrated at the location where ponding starts based on linear interpolation between each pair of stations. The best estimate is that given by the interval within which the ponding location occurs. The rest are computed for simplicity, since the worksheet is protected to avoid accidental corruption. Based on interpolation, the infiltrated depth at the start of ponding is 90.0 mm. The spreadsheet calculations used to provide convergence assume that the minimum depth is at the start of ponding. If the minimum depth is at the upstream end, the user should iterate to satisfy the required depth there. The deep percolation and efficiency are computed as before. In this case, the application efficiency improved from 67.9% to 83.2%, a substantial improvement.

5.4 Level basins

For level basins, start with a length of 400 m, and a relatively high flow rate, 3 *l*/s/m.

1. Since normal depth has no meaning for zero slope conditions, the flow depth at the upstream end is computed as a function of the advance distance. For the 200 and 400 m distances, from Equation 37 the depths are 97 and 114, respectively. This directly gives surface volumes of 15.5 and 36.4 m^3 per meter width (or m^2) for the two distances, respectively.
2. Next, estimate advance times. From these estimates, compute the advance exponent, subsurface shape factor, and infiltrated volumes. From the infiltrated volume, plus the previously computed surface volume, compute the time needed to apply that volume of water. Manually input a new estimate of the advance times and continue to

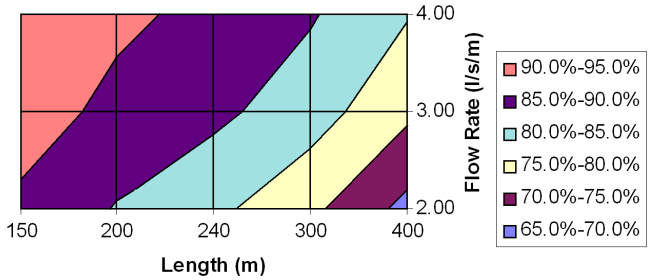

Figure 4:
Performance (PAE_{min}) contours (not to scale) for level basin example.

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iterate until the manual and computed values match. In this case, this occurs at 134.7 and 363.4 minutes, respectively. The results are shown in Table 8.

Table 8: Advance calculations for assumed-surface-volume level basin 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
User input	User input	Trial & error	Eq. 11 & 12	Eq. 37	Eq. 11 & 12	Eq. 16	Eq. 15	Eq 13
3.0	200	134.7	24.25	96.9	15.5	1.43	0.722	8.74
3.0	400	363.4	65.41	113.7	36.4	1.43	0.722	29.02

3. Next, compute an estimate of the final volume infiltrated from Equation 38, which is 49.3 m^2. Dividing this by the inflow rate gives the cut off time, 274 min.
4. Finally, assume that recession occurs everywhere at the same time to provide the needed depth at the downstream end. The distribution of water is then computed, from which a better estimate of the final subsurface water distribution is found (Table 9). Integration gives the final infiltrated volume of 47.3 m^2, corresponding to a cut off time of 263 minutes, or an application efficiency of 76.1%. Of interest is that the application time is less than the advance time. From the advance curve, compute that the water is cut off when it has advanced 0.80 of the distance. At this distance from the end, experience suggests that advance may be slowed significantly after the inflow stream is cut off, such that the water may reach the end of the field later than predicted, if at all. While a reasonably high application efficiency is predicted, it may not be achievable.

Table 9:
Advance and recession for level basin design example (q_{in} = 3 l/s/m), n =0.15.

Distance m	Advance Time min	Recession Time min	Opportunity Time min	Infiltrated Depth mm
0	0	656	656	136
50	19	656	637	134
100	50	656	606	130
150	89	656	567	126
200	135	656	521	121
250	185	656	470	115
300	241	656	415	108
350	300	656	356	99
400	363	656	293	90

Further analysis provides estimates for application efficiency for a range of lengths and widths, as shown in Figure 4. Tabled values are provided in the spreadsheet. Note that performance is improved with shorter basin lengths.

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5.5 Level furrow

For the level furrow, use the same length as the border and basin examples, 400 m, and a flow rate of 2 *l*/s per furrow.

1. As with level basins, the flow depth and cross sectional area at the upstream end is dependent only on the length, from Equation 40. This has to be solved iteratively. For these lengths of 200 and 400 m, the cross sectional areas are 0.031 and 0.038 m^2, respectively (Table 10). The corresponding wetted perimeters are 0.55 and 0.61 m, respectively.

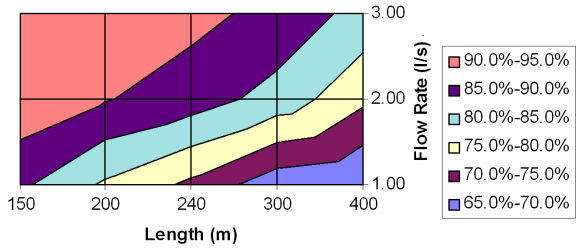

Figure 5:
Performance (PAE_{min}) contours (not to scale) for level furrow example.

Table 10:
Advance calculations for assumed-surface-volume level furrow design example.

Q *l*/s	**x** m	**t_x** min	**$V_{in}(t)$** m^3/m	**A_0** mm	**$V_y(t)$** m^3/m	**h**	**σ_z**	**$V_z(t)$** m^3/m
User input	User input	Trial & error	Eq. 11 & 12	Eq. 40	Eq. 11 & 12	Eq. 16	Eq. 15	Eq 13
2.0	200	108.3	13.0	0.031	4.9	1.67	0.750	8.0
2.0	400	344.2	41.3	0.038	12.1	1.67	0.750	29.1

2. The remaining steps use the same procedures as with level basins. Computed values are given in Table 11. The efficiency in this case is 76.0%. In contrast to the level basin example, in this case advance is complete long before the irrigation stream is cut off (395 min).

Table 11:
Advance and recession for level furrow design example (q_{in} = 2 *l*/s/furrow), n =0.05.

Distance m	**Advance Time** min	**Recession Time** min	**Opportunity Time** min	**Infiltrated Depth** mm
0	0	637	637	134
50	11	637	626	133
100	34	637	603	130
150	67	637	570	126
200	108	637	529	122
250	157	637	480	116
300	213	637	424	109
350	275	637	362	100
400	344	637	293	90

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Performance results for level furrow were computed for the range of conditions, as shown in Figure 5. Tabled values are given in the spreadsheet.

6. Discussion

Detailed designs should consider changes in infiltration, flow resistance, and target application depth during the season. For example, it is well known that infiltration can be quite different for the first irrigation of seasonal crops. Often, the design intent is to develop an irrigation system that has a particular seasonal performance, rather than achieving a target efficiency for some "typical" condition. Further a sensitivity analysis should be performed to determine the sensitivity of the design to changes in these conditions. This is particularly useful for developing advance-distance based cutoff criteria. These exercises are left to the reader.

If soil spatial variability is a significant issue, one can determine the impact of this variability (if known) on irrigation performance with the method described in Clemmens (1991b). As stated earlier, the design intent is to lay out fields to minimize this influence. If performance is severely impacted by soil variability, the designer might consider other irrigation methods.

Several software programs are available for performing surface irrigation design calculations. Shatanawi (1980) made numerous unsteady-flow simulation runs with the zero-inertia model to produce infiltrated depth profiles. These results were utilized within the border-strip design program, BORDER (Strelkoff *et al.* 1996). This software allows the user to examine, among other things, the influence of field length and inflow rate on performance. Unfortunately, the range of conditions available is somewhat limited. It also includes advance distance at cutoff and the ability to pick a point off the contour plot for examination of the distribution of infiltrated water and performance. This software is also limited to only a power infiltration equation.

Clemmens and Dedrick (1982) developed dimensionless design solutions for level basins. This was coded into the BASIN program (Clemmens *et al.* 1995), which includes advance distance at cutoff criteria (Clemmens 1998). This software only allows a power infiltration equation. Boonstra and Juriens (1988) developed a software program for level basins, BASCAD. In this program, advance is computed with unsteady-flow simulation, while recession is assumed to occur everywhere at the same time. They have a similar problem as faced with the approach here in that advance after stream cutoff is questionable. They limit solutions to cutoff after advance is 90% complete.

Several simulation programs are available for making calculation of advance, recession, and the distribution of water (e.g., SRFR: Strelkoff *et al.* 1990, SIRMOD: ISED 1989). These more accurate solutions avoid some of the assumptions with the methods presented here. However with simulation software, one has to perform simulations by adjusting cutoff time by trial and error to obtain a solution that gives the minimum depth equal to the required depth. This would have to be done for each set of conditions to be examined, that is, each point on the performance contour plot. Unless the software assists with this process, this is not convenient for routine application.

Clemmens *et al.* (1998) suggest the use of the above equations for furrow design, combined with simulation runs with SRFR to adjust the approximations made here such that the solutions to the design equations match the simulation results. Then the adjusted equations are used to develop the performance contours. With this approach, the adjusted solution will be correct at the points where simulations are made and will drift away from the true solution as one moves away from these conditions.

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7. Conclusions

A general design approach for surface irrigation was extended to all surface irrigation methods. The approach uses continuity to compute the advance time to field end and half the distance to field end. The irrigation methods differ slightly in how the surface volume is computed, but each utilizes a surface shape factor. The subsurface volume calculations use a modified Kostiakov equation that includes both an initial sink term and a constant final infiltration rate. Recession calculations differ for each method. With sloping methods, adjustments to a straight line recession curve make recession estimates more reasonable. For level methods, recession is assumed to occur at the same time everywhere.

It is shown that these procedures can produce reasonable predictions of design performance over a range of conditions. An example is provided to put the method and its application into context. The approach is also conducive to a sensitivity analysis for determining the impact of variations in input parameters on performance.

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Appendix I – Design equations

The equations below are provided here with very brief descriptions for completeness. More details on these equations and their source can be found in Clemmens (2007).

The PAE_{min} is the application efficiency when the minimum depth infiltrated just equals the target depth.

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

where d_{req} is the required depth, d_{min} is the minimum infiltrated depth, and d_a is the depth of applied irrigation water, in this case, the depth applied that results in the minimum depth just equal to the required depth.

Resistance to flow is usually described by the Manning equation, which relates the flow rate, Q, to the flow area, A, the hydraulic radius, R (area over wetted perimeter), the friction slope, S_f, and the Manning n.

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

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

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

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

The Kostiakov-Lewis equation for infiltration is

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

or in modified form

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

where *d* is infiltrated depth, *i* is infiltration rate, τ is the infiltration opportunity time and *b*, *c* and *k* are empirical coefficients. In Equation 4, the exponent *a* is often below ½, which is at odds with soil physics, unless one considers soil cracking, consolidation on wetting, etc.

The Kostiakov branch function equations are

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

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

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

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

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

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

where t_{co} is the time of cutoff or application time and t_{lag} is the recession lag time, or the time required for the water depth at the upstream end to drop to zero after cutoff.

The recession time at the downstream end is

$$t_R(L) = t_A(L) + \tau_{req} \tag{9}$$

where *L* is the field length and τ_{req} is the time required to infiltrate the required depth.

The time of cutoff can be computed from

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

where the term in brackets is the time between cutoff and recession at the downstream end.

The inflow volume is equal to the surface storage volume plus the cumulative infiltrated volume,

$$V_{in}(t) = V_y(t) + V_z(t) \tag{11}$$

where $V_{in}(t)$ is the inflow volume at time *t*, $V_y(t)$ is the volume in surface storage at time *t*, and $V_z(t)$ is the infiltrated volume at time *t*. Typically, Equation 11 is put in the following form

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

where t_x is the advance time to distance x, $A_0(t)$ is the cross-sectional flow area at the inlet at time t, σ_y is the surface storage shape factor, W is the width, and σ_z is the subsurface shape factor (all units are in meters and seconds). The surface shape factor is the ratio between the average cross-sectional flow area and that at the head of the field.

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

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

where h is the exponent on the advance equation

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

where s is a constant. Then σ_{z1} can be found from

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

From two time-distance pairs, the advance exponent can be computed from

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

An adjustment to the recession curve for sloping furrows 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

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

For calculation purposes, the volume in surface storage at the completion of advance, $V_y(L)$ is used to estimate the surface volume at cutoff, $V_y(t_{co})$.

Dividing the cutback inflow rate by the wetted field area gives the average infiltration rate that matches the cutback inflow

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

The average infiltration rate can be computed from numerical integration over the distance (e.g., the 8 intervals suggested above), or estimated from

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

where t_{CB} is the time of cutback.

The cutback time is adjusted to account for the change in surface storage resulting from the reduced flow rate

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

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

The conditions for runoff reuse systems should satisfy the following;

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

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. F, N, n_s, and n_p are integers.

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

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

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

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

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

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

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

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

where RO is the runoff fraction (i.e., fraction of inflow that runs off for an *average* furrow). The unrecovered runoff is $V_{in}(RO\text{-}RU)$.

The reservoir volume, V_R, needed for operation of such systems is the amount of water

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required in the reservoir to irrigate the last set with only return flow

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

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

The maximum recommended inflow rate to limit erosion on non-sod forming crops, such as alfalfa and small grain, for sloping border strips is found from

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

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

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

The normal depth in border strips can be computed from Equation 3 with q as the unit inflow rate and S_f set to the bottom slope, S_0. Solving for depth gives

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

An approximate upstream recession time for border strips is found from (Walker and Skogerboe 1987)

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

where all units are in meters and second and S_y is

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

and I is the infiltration rate (m/s) at $t_R(0)'$, averaged over the length. It is based on an estimate of the upstream recession lag time from (Strelkoff, 1977), which results in a cutoff time of

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

The following equation can be used to estimate the upstream recession time for border strips (adapted from Hart *et al.* 1980).

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

where units are in meters and seconds.

The slope of border strip recession curves can be computed from

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

with the restriction that the computed recession time at any distance not exceed the recession time at the downstream end.

The length of ponding on a border strip with a blocked end can be found from

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

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

For level basins, assume

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

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

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

The volume of infiltrated water can be approximates for the first two terms in the expansion, resulting in the following equation

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

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

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

For level furrows, combining Equations 2 and 36 gives

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

which can be solved for y_0 by trial and error for a given distance, x, since A_0 and R_0 are direct functions of y_0.

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