Analysis of surface irrigation systems with WinSRFR—Example application

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ABSTRACT

WinSRFR is an integrated software package for analyzing surface irrigation systems. Software functionalities and technical features are described in a companion article. This article documents an example application. The analyzed field is a graded basin (close-ended border) irrigation system. The event analysis tools of WinSRFR are used first to evaluate performance of the irrigation system and estimate its infiltration and hydraulic roughness properties. Performance contours in the Operations Analysis World are then used to optimize irrigation system inflow rate and cutoff time. The adequacy of the existing design is examined with the performance contours provided in the Physical Design World. Hydraulic and practical constraints are considered in finding an optimal operation or design solution. Finally, sensitivity analyses are used to demonstrate the robustness of the solutions.

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1. Introduction

WinSRFR is an integrated software package for analyzing surface irrigation systems. The software was developed primarily as a tool for irrigation practitioners, but also to serve as a foundation for continued research and development in surface irrigation hydraulics. Bautista et al. (2009) provide an overview of the software and discuss its key technical elements. WinSRFR is structured around four main functionalities, referred to as Worlds in the software. Users can analyze field evaluation data, estimate field infiltration properties, and assess the performance of an observed irrigation event with tools of the Event Analysis World. A wide range of design and operational alternatives can be easily examined with the tools of the Physical Design and the Operations Analysis Worlds. The Simulation World provides access to the simulation engine, which can be used to test individual scenarios or to conduct sensitivity analyses. This article presents an example application, partly with the objective of demonstrating the use of the programs' functionalities. An important constraint in any type of surface irrigation system analysis is the uncertainty of field soil and crop hydraulic properties, which are difficult to measure and variable in space and time. Therefore, the example also analyzes the robustness of the optimized operational recommendation. The example is based on the functionalities of WinSRFR 3.1, released in 2009.

2. Problem data

Niblack (2005) reported several irrigation evaluations conducted on graded basin irrigation systems in Yuma-Mesa Irrigation and Drainage District (YMIDD), Yuma, Arizona. One of those tests, labeled GC/2-9-05, was selected for the example. Field dimension, slope, and flow management data are given in Table 1. The given inflow rate Q is considered relatively accurate because it was measured during the test at regular intervals. The field was leveled at the time that the orchard was originally established to the slope value given in Table 1, according to information provided by the landowner. However, the field was not surveyed as part of the evaluation because of the difficulties of surveying an established orchard. Soils typical of this region are described as a Superstition-Rositas association (Typic Calciorthid: sandy mixed, hyperthermic; Typic Torripsamments: mixed, hyperthermic) (Hendricks, 1985) and are very sandy. Because of the porous soils, high inflow rates are typical of surface irrigation practices in the area. Based on the low water holding capacity of the Superstition-Rositas soils and typical irrigation practices of the area, a 50 mm irrigation depth requirement (d_{req}) was estimated for the example.

3. Event analysis

The primary objective of this part of the analysis is to evaluate the application efficiency and distribution uniformity for the observed irrigation event. As part of the analysis, the program will be used to estimate the infiltration properties of the field and the

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Table 1
Data for the example border.

Variable	Value
Test_ID	GC/2-9-05
Type	Basin
Crop	Citrus
Soil	Loamy sand
Length, L (m)	190.5
Width, W (m)	56.4
Downstream boundary	Closed
Bottom slope, S_0	0.001
Manning's n	0.08
Inflow rate, Q (l/s)	498
Cutoff time, t_{co} (min)	30

value of Manning roughness coefficient n, which are needed for subsequent design and operations analyses.

Three event analysis procedures are currently supported by the Event Analysis World, one of which is Merriam and Keller's (1978) post-irrigation volume balance (PIVB) method. The method estimates a field-averaged infiltration function from the field-measured geometry, inflow and outflow hydrographs, and advance and recession times. The data can be fitted to one of several empirical infiltration functions provided by the software. Two of those options, the NRCS Infiltration Families (IF) (USDA-SCS, 1974, 1984), and the Modified Kostiakov equation, will be used here. Bautista et al. (2009) provide details on the implementation of the PIVB method and discuss the options provided by the software for computing infiltration.

Fig. 1 depicts the advance–recession data for the example. These data were measured at stations located at 0, 25, 50, 75, and 100% of field length and reveal large differences in opportunity time along the field.

NRCS Infiltration Families have the following functional form:

$$z = k\tau^a + c \tag{1}$$

In Eq. (1), z is the infiltrated depth computed as a function of opportunity τ, k and a are parameters unique values specific to each family, and c is a constant (7 mm). When this option is selected, WinSRFR searches for the family that will most closely satisfy the volume balance relationship. The solution for this problem is the NRCS 0.9 IF, which is illustrated in Fig. 2.

WinSRFR validates the estimated infiltration function using unsteady flow simulation. The software compares the simulation
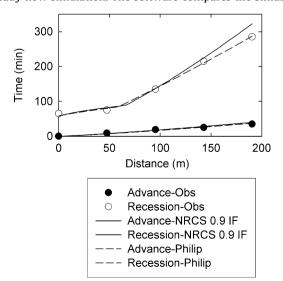

Fig. 1. Observed and predicted advance and recession times. Predicted values computed with the NRCS 0.9 Infiltration Family and the Philip function.
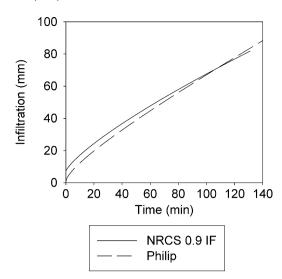

Fig. 2. Infiltrated depth as a function of time predicted with the NRCS 0.9 Infiltration Family and the Philip function.

results with field measurements, and generates performance measures for the irrigation event. Graphical outputs include the measured and predicted advance and recession times, and the corresponding runoff hydrographs. Statistical measures computed by WinSRFR include the root-mean-square error (RMSE) of advance and recession time (displayed in the Goodness-of-Fit Tab). These graphs and statistics can be used to compare the performance of alternative infiltration functions and test the effect of uncertain inputs.

One uncertain input needed for validation is the Manning roughness coefficient n. Detailed depth hydrographs are needed to evaluate n, but such measurements are labor intensive and not collected as part of routine evaluations. Moreover, field studies that have measured n reveal variations in those measurements from one basin, border, or furrow to another, along the field, and over an irrigation season (Esfandiari and Maheshwari, 1997; Li and Zhang, 2001; Walker and Kasilingam, 2004). Hence, any estimate of n has to be considered approximate. Practitioners rely on NRCS recommended values (USDA-SCS, 1974, 1984) to estimate average representative conditions, and potential extreme conditions. Unsteady flow simulation can be used to fine-tune those estimates. For an orchard, a reasonable lower bound for n is the recommended value for bare soil (0.04) while an upper bound, considering weed growth typical, is the value recommended for small grains (0.10). Simulations were conducted to examine the sensitivity of the validation results to n. These test produced relatively poor advance and recession predictions (measured by the RMSE) with n values outside the range 0.04–0.10, best advance predictions (RMSE $\approx$ 2 min) with n in the range 0.06–0.08, and best recession results (RMSE $\approx$ 35 min) with n in the range 0.08–0.10. Consequently, n was set at 0.08 for subsequent analyses. The implications of this selection will be investigated in a later section.

The difference between observed and simulated recession times also suggested a small difference between the actual and stated field slope which, as was explained earlier, was not measured as part of the evaluation, but is known to have been graded based on standard practices in the area. Under the conditions of the problem, a graded field with a closed downstream end, recession is very sensitive to slope and infiltration. A sensitivity analysis confirmed, in fact, that recession times were better simulated with a smaller slope, 0.0008. In a real application, where improvements in performance are expected, the expectation is that field elevations would be carefully measured. However, since the objective is to illustrate the use of the software, the

analysis presented in the following sections will assume that the slope is 0.0008. Advance and recession predictions obtained with this slope and the 0.9 IF are shown in Fig. 1.

A second infiltration solution was generated using the Modified Kostiakov equation:

$$z = k\tau^a + b\tau + c \qquad (2)$$

In this expression, k and a are parameters that describe the transient infiltration behavior, b a parameter associated with the steady-state infiltration behavior, and c a storage term that describes instantaneous infiltration through macropores. When this option is selected, the software displays input boxes for the parameters a, b, and c and calculates the parameter k. Many combinations of parameters can satisfy the mass balance relationship, so the user has to use judgment and simulation results to find a parameter set that will adequately describe the infiltration properties of the evaluated field.

Given the sandy soil of the example, a reasonable assumption is that $a = 0.5$, $c = 0$, therefore eliminating two unknowns. This solution will be identified in this article as the Philip solution, because its form is similar to that of the Philip infiltration equation (Philip, 1957). The evaluator can rely on experience or published values of saturated hydraulic conductivity to define a likely range for b. The parameter b (and consequently k) can then be adjusted by trial-and-error, by comparing observed and predicted advance and recession times. Fig. 2 compares the computed Philip solution ($k = 19.6$ mm/h^a, $b = 25$ mm/h), with the NRCS 0.9 IF, while Fig. 1 contrasts the corresponding simulation results. Clearly, predictions match the observations more closely with the Philip equation (RMSE of advance and recession times of 2.3 and 4.7 min), than with the NRCS 0.9 IF.

Similarities and differences in the behavior of the NRCS and Philip solutions for this example are worth noting (Fig. 2). For relatively short times the NRCS solution predicts larger initial infiltrated depths than the Philip solution due to the contribution of the c term. The solutions eventually intersect at a time less than the average opportunity time of the test (132 min). Infiltration rates predicted with the NRCS function continuously decay for long times. In contrast, infiltration rates predicted with the Philip function approach the value of the constant b. Because of the sandy soil, one should expect near-steady infiltration rates to be attained rapidly and to be large, as predicted by the Philip solution.

Table 2 summarizes the performance assessment results computed by WinSRFR, based on the two proposed infiltration solutions. Mathematical definitions for these performance indicators are provided in Burt et al. (1997). Both solutions predict low application efficiency (AE) and distribution uniformity of the minimum (DU_{min}), which could have been anticipated based on the differences in opportunity time along the field. Because the system produces no runoff, all losses are as deep percolation (DP) and are nearly 40% with both functions. Simulation results generated with the Philip equation suggest slight under-irrigation ($D_{min} = 48$ mm, $AD_{lq} = 0.96$) while results generated with the NRCS 0.9 IF indicate that the requirement was met everywhere ($D_{min} = 50$, $AD_{lq} = 1.01$). Independently of the infiltration function, results should be useful when discussing the need to optimize operations and/or design with the landowner because they clearly show that a large part of the field is over-irrigated while another part barely gets the required amount.

4. Operations analysis

The Operations Analysis World is used to optimize the inflow rate Q and cutoff time t_{co}. The analysis is conducted with the help of performance contours which depict the variation of selected performance measures as a function of Q and t_{co}. Performance contour plots generated by the software include the application efficiency (AE), distribution uniformity of the minimum (DU_{min}), and runoff (RO) and deep percolation (DP) fractions (Burt et al., 1997). The contour plots are generated by interpolation from simulation results computed at discrete grid points on a rectangular solution region. The solution region is defined by the range of inflow rates and cutoff times that the user wishes to explore.

Fig. 3 is an AE contour plot for the example problem, with infiltration given by the NRCS 0.9 IF and $S_0 = 0.0008$. Design contours for graded basin systems are a new feature of WinSRFR 3.1. As expected, AE is maximized in the lower-left corner of the graph, where the basin is severely under-irrigated, and decreases with increasing Q and t_{co}. Of particular interest to the analysis is the dotted line in Fig. 3, which represents solutions that satisfy the irrigation requirement everywhere, i.e., $D_{min} = D_{req}$. Q–t_{co} combinations to the right and above that line produce a $D_{min} > D_{req}$, while the opposite is true to the left and below the line. The solid star to the left of this line represents the current operation. The predicted AE for this point is consistent with the value determined from the field measurements. Clearly, the contour plot shows that the inflow rate was excessive for the observed irrigation event.

The peculiar shape of the $D_{min} = D_{req}$ curve, which has two points where the slope changes sharply, can be investigated with the help of the water distribution diagram (Fig. 4). When enabled, this diagram displays the predicted final infiltration distribution
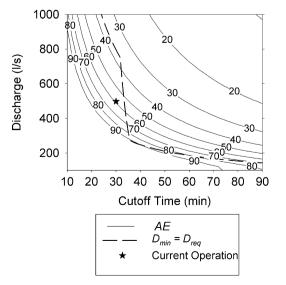
Table 2
Performance of evaluated border with different infiltration solutions.

Performance indicator	Label	Infiltration function	
		NRCS	Philip
Applied depth (mm)	D_{app}	84	84
Minimum infiltrated depth (mm)	D_{min}	50	48
Low-quarter infiltrated depth (mm)	D_{lq}	51	48
Application efficiency (%)	AE	63	59
Distribution uniformity of the minimum	DU_{min}	0.60	0.57
Distribution uniformity of the low-quarter	DU_{lq}	0.61	0.57
Runoff fraction (%)	RO	0	0
Deep percolation fraction (%)	DP	37	41
Low-quarter adequacy	AD_{lq}	1.01	0.96
Area under-irrigated (%)	AUI	0	48

Fig. 3. Contour plot of application efficiency as a function of discharge and cutoff time for the example problem with plot of the $D_{min} = D_{req}$ line.
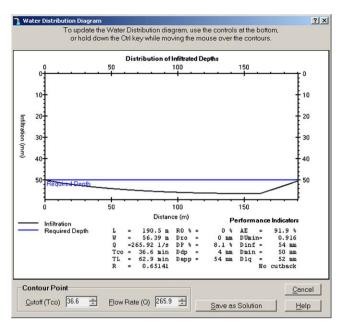
Fig. 4. Water distribution diagram.

Fig. 5. Contour plot of distribution uniformity of the minimum as a function of discharge and cutoff time for the example problem.

and corresponding performance measures. (Note that in the figure, which is generated by the software, Tco = t_{co}, TL = t_L, the final advance time, and Dro, Ddp, and Dinf refer to the runoff, deep percolation depths, and infiltrated depths, respectively; other variables have been previously defined.) The diagram is updated as the cursor navigates over the contour plot, and allows the user to view changes in the infiltrated profile (and performance indicators) with changes in Q and t_{co}. The diagram can be used also to select a particular combination of variables as a solution point. That solution point can later be copied to the Design or Simulation Worlds for further analysis. Fig. 4 illustrates the infiltrated profile produced by the combination $Q \approx 266$ l/s and $t_{co} \approx 36.6$ min, for which AE exceeds 90%. This is a point at which the slope of the $D_{min} = D_{req}$ line is discontinuous and, therefore, where the hydraulic behavior of the system changes. For discharges less than this value, the point of minimum infiltration is located at or near the downstream end of the basin but for larger values the point of minimum infiltration is located upstream. Between this point and the second transition point, at $Q \approx 750$ l/s and $t_{co} \approx 31.8$ min, excess irrigation creates a ponding area that increases in length gradually with increasing Q. In that ponding area, final infiltration depth varies linearly with distance. Beyond 750 l/s, the entire basin is subject to ponding except at the upstream end of the field (i.e., infiltration varies linearly with distance for the entire basin).

An important observation to make at this point is that since the performance contours are based on volume balance calculations tuned with a single unsteady flow simulation, the infiltration profile and performance summary displayed on the water distribution diagram do not exactly match the results computed with unsteady flow simulation (Bautista et al., 2009). As explained in that reference, errors can be controlled by reducing the range of variables under investigation and by adjusting the location of the tuning point. For example, Fig. 3 was generated with the calibration point at the center of the graph ($Q = 550$ l/s and $t_{co} = 50$ min). Despite the large range of Q and t_{co} values used to define the contour region, differences between the volume balance and unsteady flow solutions are not large and vary systematically (performance tends to be underestimated for cases where the minimum infiltration point is located downstream and overestimated when that point is located upstream). Because of the
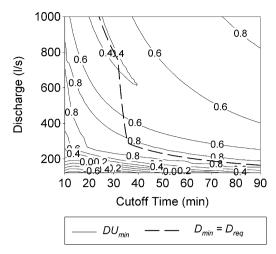
approximation errors, an optimal Q–t_{co} combination does not fall exactly on the displayed $D_{min} = D_{req}$ line, but it is close to that line and can be easily found by trial-and-error.

Returning now to the objective of this section, a desirable Q–t_{co} combination is one that maximizes the application efficiency and distribution uniformity. However, such solutions can represent poorly posed hydraulic problems, i.e., problems for which the predicted advance is very sensitive to small variations in infiltration, roughness, and inflow (which are likely to occur in the real world) and, therefore, undependable. Contours plots of DU_{min} and the ratio R, together with the AE plot, can be used to identify constraints. R is defined as the advance distance at cutoff divided by the field length (X_{co}/L) for cases where cutoff precedes final advance, and as cutoff time over final advance time (t_{co}/t_L), for cases where cutoff follows final advance. Both the DU_{min} and R plots also display the $D_{min} = D_{req}$ line.

Fig. 3 shows that AE > 80% can be achieved with a relatively wide range of inflow rates, between 184 and 315 l/s (equivalent to a unit inflow rate of 3.22–5.64 l s^{-1} m^{-1}). Because the system has no runoff losses, AE = DU_{min} for solutions along the $D_{min} = D_{req}$ line. The DU_{min} contour plot (Fig. 5) confirms this behavior, and also shows that uniformity improves with increasing Q and t_{co} (at the expense of a decreasing AE). In contrast, uniformity can decrease rapidly at low Q. This region of rapidly changing DU_{min} is an indication of a poorly posed hydraulic problem; specifically, it represents a region where the surface volume and advance rate at cutoff time are both small and under which small changes in inputs will cause under-irrigation at the downstream end of the basin. The ratio R (Fig. 6) provides another indication of potential problems with a selected solution. The point of maximum efficiency along the $D_{min} = D_{req}$ line ($Q \approx 266$ l/s) is also the point of minimum R ($R \approx 0.65$). This solution may be risky because cutoff occurs when water reaches 65% of the basin length. For level basins, a minimum recommended value of R is 0.85 (Clemmens and Dedrick, 1982) but a similar criterion has not been developed for graded basins. R can be increased with a different value of Q but, evidently, the tradeoff is a smaller AE. Since AE changes more rapidly with a larger than with a smaller discharge, the recommended Q should be less than 266 l/s, if a value of R greater than 0.65 is required.

Practical factors need to be considered also when recommending a particular Q–t_{co} combination. One such consideration is the maximum available flow rate and whether the irrigator has the ability to adjust the supplied discharge. If the discharge is fixed, then the only option is to use the given flow rate, or to split the flow between two or more borders. For those cases, the range of viable solutions is limited. Work-shift hours will also limit the available
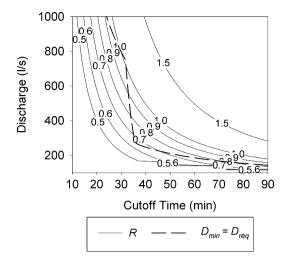
Fig. 6. Contour plot of the cutoff ratio as a function of discharge and cutoff time.

Fig. 7. Discharge–application efficiency and discharge–cutoff time relationships for solutions along the $D_{min} = D_{req}$ line.

solutions, as irrigators try to fit a predetermined number of irrigation sets within their work-shift hours or a 24-h period. For the example problem, water is supplied through a canal and the delivery rate is adjustable. Also, the duration of each set can be easily adjusted because the irrigation sets currently are short (30 min).

Given these constraints, two possible solutions are to split the available flow between 2 or 3 basins. This translates into a basin inflow rate of 250 or 166 l/s. A third possible solution is an intermediate value, e.g. 200 l/s. Table 3 summarizes the values of t_{co} required for each of these solutions and the resulting performance indicators. In all cases, the t_{co} value obtained from the volume balance solution had to be adjusted slightly to satisfy $D_{min} = D_{req}$ because the solution extracted from the contour plots is based on volume balance, as was explained earlier. These results show that $Q = 200$ l/s is a reasonable compromise, considering the high AE, DU_{min}, and R. (However, note that such recommendation ignores potential non-uniformities in bottom slope, an issue that is briefly addressed in Section 7.)

Fig. 7 compares the Q–AE relationship for solutions along the $D_{min} = D_{req}$ line for the two infiltration equations developed in Section 3. The graph also shows the corresponding relationship between Q and t_{co}. These curves, which are also generated by the Design World, were developed based on a narrower discharge range than used in Fig. 3. The graphs show that the NRCS 0.9 IF and the Philip solution yield similar performance and cutoff time relationships for inflow rates less than approximately 200 l/s but not for larger flows. For $Q > 200$ l/s, AE will be overestimated if the analysis is conducted with the NRCS infiltration solution but the true infiltration is given by the Philip equation. Furthermore, a longer cutoff time (about 5 min) will be needed to satisfy the
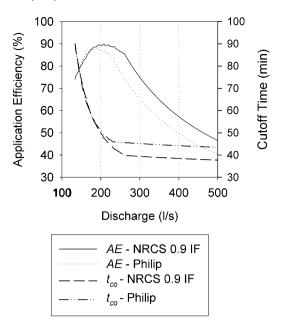
irrigation requirement over that inflow range. These results give greater confidence to the operational recommendation developed in the previous analysis.

5. Design analysis

Performance can be improved for the example problem by optimizing operations. If the existing irrigation system cannot attain reasonable performance even if the operation is optimized, an alternative design needs to be investigated. Although an alternative design is not needed here, the example problem will be used to discuss the WinSRFR design procedures.

The Design World is used to optimize the length and width of a system for a given field slope, infiltration and hydraulic roughness characteristics, target infiltration depth, and available inflow rate. As an alternative, the design analysis can optimize the length and flow rate of the system for a given field width. This option is particularly useful when the user wants to analyze the design based on inflow rate per unit width (in which case the basin width would be set to 1 m). For cases where potential field slope changes have to be investigated as part of the design, then separate analyses have to be conducted at user-selected slope values.

The Design World generates design solutions that exclusively satisfy the $D_{min} = D_{req}$ requirement. Hence, instead of displaying an AE contour plot, the Design World generates a contour plot of potential application efficiency of the minimum (PAE_{min}—the maximum efficiency that can be attained assuming $D_{min} = D_{req}$). Other performance contours are like those provided by the Operations World.

Fig. 8 is a PAE_{min} contour plot for the example basin. The graph illustrates the typical variation of PAE_{min}, which improves with increasing length and width, reaches a maximum, and then decreases. For the given conditions, high potential application efficiencies are attainable with different combinations of basin length and width. The combinations are represented by the region within the $PAE_{min} = 80\%$ contour. With this broad range of acceptable basin dimensions, a size that will fit within the total length and width of the field can be easily found. The broad range of solutions also ensures a robust design so that if actual field conditions differ slightly from those assumed in design, the basin will still be able to achieve high levels of performance. The existing

Table 3
Performance of three proposed operational solutions for the example close-ended border.

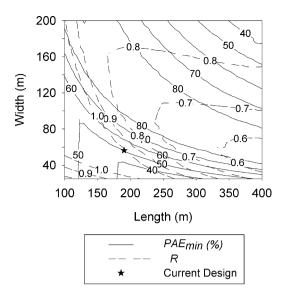| Performance indicator	Q (l/s)		
	250	200	166
t_{co} (min)	41	52	65
D_{app} (mm)	57	58	60
R	0.70	0.75	0.79
AE (%)	87	86	83
DP (%)	13	14	17
D_{min} (mm)	50	51	50
DU_{min}	0.87	0.87	0.83
DU_{lq}	0.91	0.92	0.89
AD_{lq}	1.04	1.06	1.07

Fig. 8. Contour plots of potential application efficiency and cutoff ratio as a function of basin length and width.

system is marked in the plot and has a PAE_{min} of less than 40%. Because the contours change more rapidly with changes in width than length, the graph suggests that performance can be more easily improved with available inflow by increasing the basin width rather than the length.

The PAE_{min} contours of Fig. 8 suggest that high performance can be achieved with a basin twice as long as the current basin and the same width. This can be desirable for a landowner because it reduces the cost and time of tillage operations. However, such a solution has undesirable properties, as shown by the R contours in the same figure. This type of contour overlay can be easily generated with WinSRFR. With such field dimensions, maximum AE can only be achieved with $R \approx 0.55$. Under the given field conditions an R greater than 0.8 cannot be achieved with basin lengths greater than about 175 m. Hence, the current design seems like a reasonable compromise considering PAE_{min} and R.

Fig. 9 is an example of how the software can be used to analyze alternative field slopes for given field dimensions. The graph compares the Q–AE relationships for slopes of 0.0006, 0.0008, and
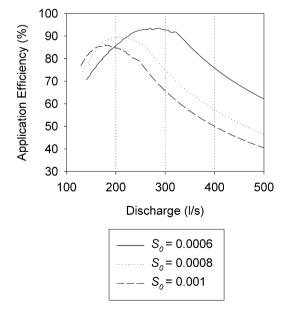
0.001 and were developed in the Operations World assuming the NRCS 0.9 IF. Results suggest minor performance differences if $Q < 200$ l/s, but not for larger flows. They also suggest improvement in performance with the smaller slope, but only at large flows. Results also make evident the importance of properly evaluating field slope when analyzing operational alternatives. The apparent advantages of the case $S_0 = 0.0006$ are tempered again by the relative cutoff distance at cutoff time R, which are not illustrated in the graph. If $S_0 = 0.0006$, a value of $R > 0.7$ can only be achieved with $Q < 200$ l/s. In this Q range, larger values of R and also better AEs can be achieved with $S_0 = 0.001$ or 0.0008.

6. Sensitivity analysis

Because of the uncertainty of field properties and system inputs, systematic sensitivity analyses need to be conducted to assess the robustness of any operation or design recommendation. For typical surface irrigation system analyses, infiltration and Manning n will vary from basin-to-basin on a given field and over the irrigation season. For irrigation systems supplied by open channels, such as in the example, inflow rates can vary during the course of the day. No information is currently available to quantify the potential range of discharge variation for the evaluated basin. In addition, for this particular example there is uncertainty as well about the field slope. The following paragraphs will discuss only sensitivity test results for infiltration, roughness, and inflow.

The sensitivity of the optimized operational strategy, $Q = 200$ l/s and $t_{co} = 53$ min, was tested for deviations in infiltration properties from the estimated 0.9 NRCS IF. Tests were conducted using NRCS Infiltration Families in the range 0.7–1.5 and also the Philip infiltration function developed in Section 3. The range of NRCS Infiltration Families considered here is consistent with the range of infiltration properties found in the YMIDD area and represents an 80–140% variation in infiltrated depth for the average opportunity time of the evaluated irrigation event. Fig. 10 illustrates the resulting infiltration profiles and Table 4 summarizes the predicted irrigation performance. If actual infiltration is much larger than expected (i.e., the NRCS 1.5 IF), water will not reach the end of the field and performance will drop severely. For the other infiltration conditions assumed in the analysis, performance will not change substantially from the optimized results and, therefore, the solution is relatively robust. Under the extreme infiltration
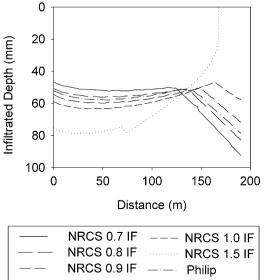

Fig. 9. Discharge–application efficiency relationships for the existing field design with alternative slopes.

Fig. 10. Combined plot of infiltrated profiles calculated with different infiltration functions.

Table 4
Sensitivity of operational solution to infiltration properties.

Performance indicator	Infiltration function					
	NRCS 0.7 IF	NRCS 0.8 IF	NRCS 0.9 IF	NRCS 1.0 IF	NRCS 1.5 IF	Philip
R	0.77	0.76	0.75	0.73	0.68	0.80
t_L (min)	72.2	74.9	78.3	83.1	NA	70.5
DP (%)	14	12	12	14	26	14
AE (%)	86	87	87	87	74	86
D_{min}	47	51	51	47	0	51
DU_{min}	0.81	0.88	0.87	0.81	0	0.88
DU_{lq}	0.86	0.90	0.91	0.87	0.34	0.91
AD_{lq}	1.00	1.05	1.06	1.01	0.39	1.05

Table 6
Sensitivity of operational solution to discharge.

Performance indicator	Discharge Q (l/s)			
	NRCS 0.9 IF		Philip	
	160	240	160	240
R	0.75	0.75	0.75	0.75
t_{co} (min)	63	46	57	42
t_L (min)	99	67	91	61
AE (%)	87	82	92	89
DU_{min}	0.75	0.85	0.67	0.84
D_{min} (mm)	42	52	34	47
AD_{lq}	0.90	1.10	0.77	0.99

conditions (NRCS 1.5 IF), the irrigator still has the option to irrigate at the maximum available flow rate (498 l/s), but with an adjusted t_{co}, to maintain an application efficiency $\geq 80\%$. For all other cases, the optimal Q–t_{co} combination will essentially meet the requirement everywhere ($D_{min} \geq 47$ mm) and yield both a high AE and DU.

The sensitivity of the optimal solution to deviations in Manning n from the assumed value is examined in Table 5. The results of this table consider the case where the actual infiltration is as assumed in the analysis (NRCS 0.9 IF) and where it is given by the Philip solution. Variations in Manning n in the range 0.04–0.1 have a significant impact on advance times, but not on performance. In this n range, deep percolation losses and AE remain unchanged while distribution uniformity indicators change only slightly. These results are explained by changes in the location of the point of minimum infiltrated depth which moves upstream with increasing n and downstream when n decreases. While results show that adequate performance can be attained as long as the right volume of water is applied, these potential variations in roughness complicate cutoff decisions for the irrigator because of the relatively large differences in final advance time. If the actual Manning n is 0.12 and infiltration is given by the NRCS 0.9 IF, advance will slow down substantially and cause increased deep percolation losses upstream and under-irrigation at the downstream end, but not if infiltration behaves as predicted by the Philip solution. Hence, the analysis is more conservative if using the 0.9 IF, at least with regards to the effect of hydraulic roughness.

An important constraint to the optimal operation of the example basin is the variability of inflow, and the fact that inflow is not measured routinely except at the farm turnout. If flow is variable, then t_{co} needs to be adjusted to deliver the desired application volume and performance. In the absence of adequate flow measurement, the irrigator can use advance distance as a cutoff criteria, if the system has been optimized. If the inflow rate is

less than expected, then t_{co} will have to be increased relative to the optimized value. Since water will advance more slowly with a smaller inflow rate, cutoff time will increase by default if based on advance distance. By the same logic, cutoff time will be reduced if based on distance if the inflow rate is greater than the design value. This advance cutoff strategy was tested using the R value calculated for the $Q = 200$ l/s and $t_{co} = 52$ min optimal solution, $R = 0.75$ (Table 3). This R value was applied to two different inflow rate scenarios representing $\pm 20\%$ of the optimized Q. Again, the analysis considers the two estimated infiltration functions and results are summarized in Table 6. Performance is still reasonable with this distance-based cutoff strategy, but only if the actual infiltration follows the 0.9 IF. In that case, the downstream end of the field will be slightly under-irrigated if discharge is less than ordered ($AD_{lq} = 0.90$), while downstream ponding will increase along with deep percolation losses if the flow is greater than ordered ($DP = 18\%$). If the actual infiltration is given by the Philip solution, then the selected R value will be inadequate and will not satisfy the irrigation requirement with either a high or low Q. A more conservative approach for management purposes would be to use the R calculated for the Philip solution, 0.8. This choice will still produce a high AE and DU if the actual infiltration is the 0.9 IF and will substantially improve performance if infiltration is given by the Philip solution.

For the irrigator, the difficulty is that inflow is not the only uncertain input and different operational strategies need to be applied when dealing with variable infiltration and n (time based cutoff) than when dealing with uncertain inflow (distance base cutoff) (Bautista et al., 2002, 2003). Because of the difficulties of measuring infiltration, any effort to improve surface irrigation design and operation should emphasize the importance of improving flow measurement.

7. Discussion

Although the main focus of this article is to illustrate the use of the WinSRFR package, limitations to the above presented analysis need to be highlighted. Those limitations could require additional simulation studies and/or field testing of the proposed operational strategy. One limitation is the range of infiltration, roughness, and inflow conditions considered in the analysis, which could be narrower than the range that could be encountered in practice. Also, the analysis is based on simulations with a uniform bottom slope, while the actual field elevations can vary considerably from design values due to imprecise land grading, soil transport from the upstream end of the field to lower parts with the irrigation flows, and tillage practices. Moreover, elevation variations can occur both along and across the direction of flow while the analysis assumes strictly one-dimensional flow. Two-dimensional simulation studies may be needed to properly analyze the impact of soil elevation variations and determine a minimum inflow rate and, therefore, minimum upstream depth requirement under the particular conditions of the example. (Generic inflow rate/

Table 5
Sensitivity of operational solution to hydraulic resistance.

Performance indicator		Manning n				
		0.04	0.06	0.08	0.10	0.12
AE (%)	NRCS 0.9 IF	86	86	86	86	84
	Philip	85	86	86	86	86
DU_{min}	NRCS 0.9 IF	0.84	0.86	0.87	0.84	0.50
	Philip	0.80	0.85	0.88	0.86	0.81
t_L (min)	NRCS 0.9 IF	57	68	78	91	113
	Philip	50	61	71	80	92
R	NRCS 0.9 IF	0.94	0.82	0.75	0.69	0.65
	Philip	1.04	0.89	0.80	0.74	0.69
D_{min} (mm)	NRCS 0.9 IF	49	50	51	49	29
	Philip	46	49	51	50	47
AD_{lq}	NRCS 0.9 IF	1.00	1.04	1.06	1.03	0.89
	Philip	0.96	1.02	1.05	1.05	1.01

upstream depth recommendations are provided in USDA-SCS, 1974.)

Irrigation practices in an area are often the result of particular constraints and traditions that can be difficult to resolve. In the YMIDD area, mounds form over time around the trees and at the end of the field because of tillage practices. These mounds can keep the water from wetting the area immediately around the tree if the flow depth is too low. High flow rates are needed in these orchards partly to overcome the high permeability of the soil, but also as a result of the perceived need to wet the surface within the tree mounds, even if only for a short time. Hence, field testing would be needed to determine, first, if the improved operational strategy would result in flow depths greater than the height of the mounds, and if not, if productivity would be affected.

8. Conclusions

Optimized design and operation of surface irrigation systems translate into high levels of performance. With WinSRFR, the analyst can visualize the range of solutions that will result in near optimal performance, find one that will meet practical constraints, and study the sensitivity of the recommended design or operational strategy. For those solutions, performance will be reasonably robust, i.e., will tolerate variations in field conditions relative to the assumed ones.

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