SIMULATION MODEL FOR LEVEL FURROWS:
II. DESCRIPTION, VALIDATION AND APPLICATION

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ABSTRACT

In a companion paper, experimental evidence was elaborated to confirm that in particular circumstances the performance of level furrow irrigation can exceed that of level basin irrigation. The application of a single furrow simulation model to an irrigation event in a level furrow field resulted in large estimation errors. To overcome them, the development and validation of a numerical model of level-furrow irrigation is reported in this work. The model is based on the interconnection of a number of one-dimensional channels. The individual channels are connected using confluence or bifurcation points. Furrow infiltration is modelled through a Kostiakov infiltration

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equation including the furrow discharge as an independent variable. The proposed model is validated using the experimental level furrow evaluation presented in the companion paper. Finally, the model is applied to explore the conditions in which level furrow irrigation can outperform level basin irrigation. The proposed model stands as a valuable tool in the design and management of level furrow irrigation systems.

INTRODUCTION

Two field experiments were presented and analyzed in a companion paper. Infiltration equations were derived from the first experiment to characterize infiltration in the experimental soil for both level basin and level furrow conditions. An effort was made to characterize infiltration in furrows as a function of discharge or wetted perimeter, thus taking into account the effect of flow level on furrow infiltration. In the second experiment a level furrow setup was evaluated, paying particular attention to the evolution of the advance process. A simulation approach was further used to conclude that in the particular conditions of the second experiment: 1) level basin irrigation would require six times more water than level furrow irrigation; and 2) a single furrow simulation model applied to the simulation of the level furrow field would underestimate the time of advance by 24 %.

In this paper, a specific model for level furrow irrigation is presented. The model consists of a network of one-dimensional furrow simulation models, connected via bifurcation and confluence nodes. The level furrow irrigation evaluation presented in the companion paper is used to validate the proposed model. The model is applied to

explore the dependence of level furrow performance on several design and management variables, in an attempt to identify conditions favoring level furrows over level basins.

NUMERICAL MODELS OF SURFACE IRRIGATION

Development of surface irrigation models based on the one-dimensional shallow water equations started early in the 1970's (Chen, 1970; Strelkoff, 1970). Several numerical procedures and discretization techniques have been used to solve the governing equations subject to pertinent boundary conditions. These models have evolved over the years and currently can simulate many situations typical of one-dimensional irrigation flow. Among these models, SRFR (Clemmens and Strelkoff, 1999) and SIRMOD (Walker, 2001) have been maintained over the years, and current versions are stable, robust and fast.

Many level basin and level furrow configurations cannot be adequately simulated with one-dimensional models. In level basins irrigated from a single point, the flow pattern is radial, and therefore a two-dimensional model is needed to track the advancing front and to handle irregular field geometries. Playán et al. (1994a) reported errors of up to 20 % in the time of advance when a one-dimensional model was used to simulate certain configurations of level basin irrigation. Two-dimensional level basin irrigation models have been under development in the last decade (Playán et al., 1994a; Playán et al., 1994b; Playán et al., 1996; Zapata and Playán, 2000; Brufau et al., 2002). These models have been used to assess the effect of spatially variable soil surface elevation, as related to the benefits of using laser guided land leveling.

Flow in level furrows is also two-dimensional as water from entirely flooded furrows often enters adjacent furrows from the downstream end. Still, application of one-dimensional models is possible but only if the system is treated as a network of open channels. Current one-dimensional models (like SRFR or SIRMOD) lack the capabilities required to reproduce the complicated flows derived from an interconnected network of channels. A new model is required, whose elements are one-dimensional reaches of a looped network. In the following section the development of such model is described.

MODEL DESCRIPTION

The numerical model developed in this work for the simulation of level furrow irrigation is based on a looped network of open channels. In these channels flow is assumed to be one-dimensional and unsteady, hence governed by the St. Venant or one-dimensional shallow water equations that are frequently formulated as:

$$\begin{aligned} \frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} + i &= 0 \\ \frac{1}{gA} \frac{\partial Q}{\partial t} + \frac{2Q}{gA^2} \frac{\partial Q}{\partial x} + (1 - Fr^2) \frac{\partial h}{\partial x} &= S_0 - S_f \end{aligned} \quad [1]$$

Where A is the wetted cross section, Q is the discharge, i is the infiltration rate, g is the acceleration due to gravity, Fr is the Froude number, h is the water depth, S_0 is the bed slope and S_f is the friction slope.

The infiltration rate is the time derivative of the infiltrated water, Z . Using the Kostiakov approach, Z is calculated in terms of two parameters, k and a , characteristic of the infiltration process:

$$Z = k\tau^a \quad [2]$$

In this equation, the independent variable is τ , defined as the opportunity time, and corresponding to the time interval that a particular point remains wet, or the current time minus the advance time to that point.

Using this equation in furrow irrigation may lead to significant errors in the estimation of the infiltrated volume and the advance curve, since the Kostiakov equation does not include the current furrow wetted perimeter (or the discharge) as an independent variable. In a pioneer work, Fangmeier and Ramsey (1978) pointed out the influence of the wetted perimeter (and therefore of the inflow discharge) on furrow infiltration. In the companion paper the Kostiakov infiltration equation was modified to extend its applicability to furrow irrigation (Rodríguez, 2002). The application of this extended procedure was validated using experimental data. The infiltration equation using discharge as an additional independent variable was used in the proposed numerical model.

The simulated network consists of a short entrance channel and a number of distribution channels and irrigation furrows. The channels connect points at junctions. Junctions may connect two or three network branches depending on their location on the field (two at the non inflow corners; three elsewhere). The model system is sketched in Fig. 1, where the inlet point is represented as a short entrance channel in order to simplify the numerical treatment of the upstream boundary conditions that, otherwise, would be located at a junction point. Each distribution channel or irrigation furrow is bounded by two junctions. In some practical level furrow applications the distribution channels are

substituted by two unfurrowed border strips (Erie and Dedrick, 1979). This particular setup cannot be simulated with the proposed model unless the border strip is narrow enough to be considered one-dimensional. Otherwise, simulation would require linking a two-dimensional overland flow model to a one-dimensional furrow simulation model.

Four kinds of input data have to be supplied to the model: geometric, infiltration, hydraulic and computational data. Field geometry is defined by the number of furrows, the length of both irrigation furrows and distribution channels and the furrow and channel cross sections (side slope and bottom width for a trapezoidal cross section). The model includes an option to assign non-uniform values of these parameters.

Infiltration may take place in the furrows as well as in the channels, but the inlet channel is assumed to be impervious. Different infiltration equations may be assigned to the furrows and channels, since they may differ in the geometry of the cross section. The flow problem is characterized by the applied irrigation discharge and the Manning roughness coefficient. The bottom slope, being a determinant variable in open channel flow problems, is irrelevant in level-furrow simulation due to the assumption of zero slope in the field.

Initial conditions are zero discharge and water depth everywhere except at the entrance point, corresponding to an initially dry field:

$$\begin{aligned} Q_j(x,0) &= 0 & 0 < x < L_j \\ A_j(x,0) &= 0 & 0 < x < L_j \end{aligned} \tag{3}$$

At the inlet point the boundary condition is the applied irrigation discharge hydrograph.

$$Q_1(0,t) = Q(t) \tag{4}$$

Following previous works (Playán et al., 1994a), [4] is enforced so that inflow is always subcritical and, for that purpose, given the inlet discharge, a safety water depth value h_s slightly greater than the critical depth ($h_s = 1.05 h_c$) is imposed as lower limit to the inlet water depth. The critical depth is the water depth corresponding to critical flow, that is, $Fr = 1$. In practice this is calculated from:

$$Fr = \frac{u}{c} = 1 \Rightarrow \frac{Q}{A(h_c)} = \sqrt{g \frac{A(h_c)}{b(h_c)}} \quad [5]$$

where u is the water velocity, c is the celerity of surface waves, and b is the cross sectional top width.

There is no downstream boundary condition in this model since there is no outlet point. All the channels and furrows are interconnected and water is only allowed to disappear from the domain by infiltration. During the advance phase, there are multiple advancing fronts running through the furrows and channels over dry cells but they are all included in the model in a through way without any special treatment.

The hydrodynamic model is used only to simulate the advance phase, that is, applied to simulate water movement until the advance process covers all the nodes (therefore using a criterion of cutoff at completion of advance). At that moment, the water supply is cut and an algebraic procedure is used to determine the time of recession (the time of advance plus the time required to allow all the enclosed water to infiltrate) and the nodal values of infiltration. This procedure is applied in order to reduce CPU time in the validation of the model and not because the model is unable to continue the simulation of the subsequent flow evolution. The simplification is based on the assumption that, once all furrows and channels are covered with water, the infiltration process is almost

static and therefore a transient model is no longer required. Since the chosen infiltration equation depends on the discharge, it can not be used any more once the flow becomes static. Therefore, during this last phase of the simulation, the equivalent infiltration equation using the wetted perimeter as an independent variable is used. A time stepping scheme is applied starting at the time of advance so that, at each time step, the volume of overland water is determined as the difference between the total irrigation volume and the infiltrated volume, and so is the corresponding wetted perimeter at the canals and furrows. The nodal values of the infiltration rate are determined from the wetted perimeter and the opportunity time. The nodal infiltrated depth is increased by the product of the nodal infiltration rate times the time step. Finally the volume of infiltrated water is determined from the nodal values of infiltration. The process comes to an end when the volume of overland water goes to zero. The current time is the recession time and the current values on infiltrated depth constitute the final infiltrated depth.

Numerical Solution

The governing equations [1] are discretized in finite difference form using the MacCormack numerical scheme. It is a well-known finite explicit scheme second order accurate in space and time. The scheme is able to deal with both gradually and rapidly varying flows. The method is based on a two-step predictor-corrector time stepping procedure (MacCormack, 1971). When applied to a general hyperbolic conservation law like:

$$\frac{\partial U}{\partial t} + \frac{\partial F(U)}{\partial x} + H(U, x, t) = 0 \quad [6]$$

in which U is the conserved variable, F is the flux and H is the source term, the predictor-corrector sequence to go from the known solution at time $n\Delta t$, U_j^n , to the unknown values at time $(n+1)\Delta t$, U_j^{n+1} is as follows:

$$\begin{aligned}
 &\text{predictor - forward difference } U_j^p = U_j^n - \frac{\Delta t}{\Delta x} [F_{j+1}^n - F_j^n] - \Delta t \cdot H_j^n \\
 &\text{corrector - backward difference } U_j^c = U_j^n - \frac{\Delta t}{\Delta x} [F_j^p - F_{j-1}^p] - \Delta t \cdot H_j^p \\
 &\text{updated value } U_j^{n+1} = \frac{1}{2} (U_j^p + U_j^c)
 \end{aligned} \tag{7}$$

The application of this method to the shallow water equations has been described by several authors (García-Navarro and Savirón, 1992, Fennema and Chaudhry, 1986). For that purpose, the system must be rewritten in conservation form:

$$\begin{aligned}
 &\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = -i, \\
 &\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x} \left(\frac{Q^2}{A} + gI_1 \right) = gA(S_0 - S_f) + gI_2
 \end{aligned} \tag{8}$$

in order to identify the vector of conserved variables, $\mathbf{U}$, the vector of fluxes, $\mathbf{F}$, and the source term vector $\mathbf{H}$:

$$\mathbf{U} = \begin{pmatrix} A \\ Q \end{pmatrix}, \quad \mathbf{F} = \begin{pmatrix} Q \\ \frac{Q^2}{A} + gI_1 \end{pmatrix}, \quad \mathbf{H} = \begin{pmatrix} -i \\ gA(S_0 - S_f) + gI_2 \end{pmatrix} \tag{9}$$

Following this identification, the procedure indicated in [7] is followed for the predictor and corrector steps in the solution. In [8], I_1 and I_2 are hydrostatic pressure integrals representing pressure forces exerted by the liquid column and solid walls respectively and are given by:

$$I_1 = \int_0^h b(x, \eta)(h - \eta) d\eta \quad , \quad I_2 = \int_0^h \frac{\partial b(x, \eta)}{\partial x} (h - \eta) d\eta \quad [10]$$

The numerical parameters of interest in the CPU time and stability properties are the nodal distance (space step) Δx and the time step size Δt . Together, they affect the computational time (CPU time) and the stability and accuracy of the solution. They are related through the Courant-Friedrichs-Lewy stability condition (Hirsch 1990):

$$CFL = \frac{(|u| + c)\Delta t}{\Delta x} \leq 1 \Rightarrow \Delta t \leq \frac{\Delta x}{|u| + c} \quad [11]$$

This expression involves the use of the dimensionless *CFL* number. In the model, the *CFL* number is assigned a constant value which, given a fixed mesh size allows for variable time step values. The restriction imposed by the CFL condition over the time step size in explicit schemes can be somewhat relaxed by means of a semi-implicit discretization of the friction term:

$$(S_f)_j = \left(\frac{|Q|n^2}{A^2 R^{4/3}} \right)_j^n [\alpha Q_j^n + (1 - \alpha)Q_j^{n+1}], \text{ with } 0 \leq \alpha \leq 1 \quad [12]$$

Where α is the degree of implicitness of the approach, and goes from 1 (explicit) to 0 (fully implicit), n is the Manning coefficient, and R is the hydraulic radius. The rest of the source terms are discretized by simple nodal evaluation at a known time level.

Boundary Conditions

The general finite difference procedure must be modified for boundary points, both external (upstream and downstream) and internal (in-line structures and junctions).

In the channel network considered herein, boundaries consist of the inlet point (upstream boundary) and the junctions (double and triple). Two types of numerical boundary conditions are therefore used and are described next.

Upstream Inflow

It is well known from the theory of characteristics that the flow regime determines the number of required physical boundary conditions. For subcritical flow, the information coming through the outgoing characteristic equation at the upstream boundary provides one equation, that is, one numerical boundary condition. Therefore, only one upstream boundary condition is necessary ($h=h(t)$, $Q=Q(t)$ or $Q = Q(h)$) to find the solution at that point. One form for the characteristic flow equations is:

$$\frac{DQ}{Dt} + b(h) \left(-\frac{Q}{A} \pm \sqrt{g \frac{A}{b(h)}} \right) \frac{Dh}{Dt} = gA(S_0 - S_f) + \left(\frac{3}{2} \frac{Q}{A} \mp \sqrt{g \frac{A}{b(h)}} \right) i \quad [13]$$

The + sign corresponds to the positive characteristic C_+ and the – sign to the C_- . The latter can be used at the upstream boundary.

The differential form in [13] can be easily approximated by an algebraic expression like

$$Q_1^{n+1} = C_1(Q_1^n, h_1^n, Q_2^n, h_2^n) h_1^{n+1} + C_2(Q_1^n, h_1^n, Q_2^n, h_2^n) \quad [14]$$

Using a fixed spatial grid, the method is based on a backward integration and linear interpolation known as Hartree method (García-Navarro and Savirón, 1993). An iterative procedure leads to the desired accuracy. The discretization [14] and the information supplied by the physical boundary condition form a system of two equations easy to solve for the two unknowns at the upstream end. In the cases presented here, the physical boundary condition is a constant inflow discharge Q_p ,

being the water depth h_p immediate. The value obtained is checked to avoid supercritical inflow.

Stream confluence or bifurcation

In the channel network of our model, there are junctions of two channels (double) and junctions of three channels (triple). In order to illustrate the procedure followed, let us consider a triple junction in which two streams join and form the third one (Fig. 2).

Three points P_1 , P_2 and P_3 are considered. They are located at the extremes of the joining channels so that P_1 and P_2 are the downstream ends for reaches 1 and 2, and P_3 is the upstream end for channel 3. Obtaining the solution at these points every time step implies to solve for six unknowns, three discharges Q_1 , Q_2 , Q_3 , and three water depths h_1 , h_2 , h_3 , hence requiring six equations to solve a junction of this type.

Following Abbot (1992) and Chow (1959), a simple method based on the discretization of the mass conservation equation at every junction boundary point has been used. In the discretization of the mass equation:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = -i \quad [15]$$

the spatial derivative is expressed differently depending on the character (upstream/downstream end) of the junction node (Figs. 3 and 4). For an upstream node like P_3 a forward difference is used :

$$\frac{\partial Q}{\partial x} = \frac{Q_s - Q_M}{\Delta x} \quad [16]$$

and backward for a downstream node like P_2 :

$$\frac{\partial Q}{\partial x} = \frac{Q_M - Q_S}{\Delta x} \quad [17]$$

where M is the index of the junction node of the reach and S indicates the immediate neighbor in that reach. Using [16] or [17] in [15]:

$$b(h) \frac{h_M^{n+1} - h_M^n}{\Delta t} + \frac{Q_S^{n+1} - Q_M^{n+1}}{\Delta x} = -i \quad [18]$$

Three independent equations are obtained, one for every reach meeting at this junction.

These are the numerical boundary conditions in our model. Three physical boundary conditions are then applied. One of them is the global mass continuity equation at the junction which, in order to take into account the possible flow signs, can be written as:

$$\beta_1 Q_{M1}^{n+1} + \beta_2 Q_{M2}^{n+1} + \beta_3 Q_{M3}^{n+1} = 0 \quad [19]$$

where subindexes $1, 2, 3$ run now over the three junction nodes at the end of the three meeting reaches. As for the sign convention, $\beta_i = 1$ when the end of reach i is downstream and $\beta_i = -1$ if it is upstream.

Two more junction boundary conditions must be imposed. It is very frequent the use of a common junction water depth h :

$$h_{M1}^{n+1} = h_{M2}^{n+1} = h_{M3}^{n+1} = h \quad [20]$$

Different options include the use of a common energy amount, some energy loss rule or the specification of a momentum balance. All these possibilities are however much more time consuming and have not been proved superior.

It is not difficult to find the expression of the junction depth h and that of the junction discharges. From [18] at an upstream point for instance:

$$Q_M^{n+1} = b \frac{\Delta x}{\Delta t} (h_M^{n+1} - h_M^n) + Q_S^{n+1} \quad [21]$$

or:

$$Q_M^{n+1} = C_A h_M^{n+1} + C_B \quad [22]$$

with:

$$C_A = b(h) \frac{\Delta x}{\Delta t} \quad C_B = Q_S^{n+1} - b(h) \frac{\Delta x}{\Delta t} h_M^n \quad [23]$$

Using [19] for two situations like the ones plotted on Fig. 4:

$$Q_{M1} \pm Q_{M2} - Q_{M3} = 0 \quad [24]$$

the + sign corresponds to the confluence and the – sign to the bifurcation. Substituting every Q_{Mi} according to the character of the Mi end point, and imposing [20], the common junction depth at the new time is:

$$h = h_M^{n+1} = h_M^n + \frac{\Delta t}{\Delta x} (Q_{S1} - Q_{S3} \pm Q_{S2}) \quad [25]$$

Once this is known, the three discharges Q_{M1} , Q_{M2} y Q_{M3} can be directly calculated.

MODEL VALIDATION

Summary of the main features of the case study

The second numerical experiment reported in the companion paper was used for validation purposes and is briefly recalled here for completeness. The experimental field was rectangular, and had a total of 40 interconnected furrows, each of them being 50 m long. The furrow spacing was 1.3 m, and therefore the second field dimension was 52 m. The level furrow setup was irrigated for 31 min with a total discharge of 52.6 L s⁻¹.

The following infiltration equations were derived for the experimental conditions:

$$Z = 0.00607 Q^{0.4} \tau^{0.515} \quad [26]$$

$$Z = 0.0213 WP \tau^{0.515} \quad [27]$$

in which Z is the furrow infiltration ($\text{m}^3 \text{m}^{-1}$), WP is the wetted perimeter, Q is the furrow discharge (L s^{-1}), and τ is the opportunity time (min). These equations apply to the irrigation furrows, but not to the distribution channels, where infiltration was prevented by a plastic film.

Spatial intervals Δx of 0.13 m and 0.50 m were chosen for the distribution channels and the irrigation furrows, respectively. Values of $CFL = 0.85$ and $\alpha = 0$ were used. These values led to stable calculations in all cases.

Infiltration was modeled using four different approaches. In the first case, infiltration was not adjusted for discharge or variable wetted perimeter effects. The infiltration equation used corresponds to the one developed for the average discharge conditions within the field, $52.6 \text{ L s}^{-1} / 40 \text{ furrows} = 1.32 \text{ L s}^{-1}$ (it will be referred to as *uniform*). In the second case, infiltration in a furrow was computed using the furrow inlet discharge as a variable; the infiltration equation varies among furrows and in time for each furrow (Q_{inlet}). The third case corresponds to using at each node the nodal value of discharge, therefore, a different infiltration equation is used at each node and time step (Q_{nodal}). Finally, the fourth case corresponds to using a different equation for each furrow, in which the inflow value in Eq. [26] is the average discharge along the furrow ($Q_{average}$). Since discharge can flow in both senses, all the values of discharge discussed in this paragraph are taken as absolute values.

Simulation results were compared with the field measured advance and flow depth and discharge measured at the inlet of furrow # 1. The measured depth reached a constant value after a certain time (much shorter than the cutoff time) that will here forth be denoted as reference depth = 0.140 m, and the measured discharge will be called reference discharge = 5.13 L s^{-1} . The first simulation used a uniform Manning n for the entire field. Different values of n were explored, and best results were obtained with $n = 0.020$ and $n = 0.025$ (Table 1). Figure 5 presents a plot of the observed advance compared to case n20n (obtained for uniform $n = 0.020$ and nodal discharge). In this case the reference depth was well predicted (0.141 m), but the advance curve and particularly the reference discharge were not. Throughout the eight cases presented in Table 1, the best advance predictions were obtained using nodal discharge to compute infiltration, and less accurate predictions were obtained using the other three infiltration calculation approaches. As an example, the advance curve for case n20u (uniform $n = 0.020$ and uniform discharge) is presented in Fig. 5. The proposed value for Manning n results unacceptable, since the experimentally determined values (Table 1 of the companion paper) were 0.05 in three cases and 0.04 in one case. In view of the results, the nodal value of discharge was used in subsequent simulations.

The following validation attempt was based on the use of non-uniform Manning n . This option is fully justified by the experimental conditions. Since the distribution channels were covered by a plastic film a new simulation test was run using different n values for the furrows and distribution channels. The Manning n of the film was estimated as 0.015 - 0.020, while the Manning n of the irrigation furrows was set at the experimentally determined values of 0.04 and 0.05. The results are presented in Table 2. The simulated reference depth is relatively close to the target, particularly for case

n15/50n (with Manning n values of 0.015 in the channels and 0.050 in the furrows). A significant improvement was obtained in predicting the reference discharge. The simulated values were much lower than in the previous simulation cases (based on a uniform Manning n value) and showed a relevant agreement with the observed value (in case n15/50 the simulated value was 4.77 L s^{-1} , while the observed value was 5.13 L s^{-1}). Case n15/50 also resulted in the best fit to the advance equation (Figure 5), and was considered as the final validation data set. The simulated time of advance was 28.0 min, slightly below the observed value of 31 min. As previously discussed, small differences in soil surface elevation near the end of the advance phase can have profound effects on final advance time predictions.

Figure 6 presents the measured and simulated flow depth and discharge hydrographs at the inlet of furrow #1. Simulated values correspond to the n15/50 test. While the observed flow depth appears to stabilize quickly, the simulated depths continue to increase for a longer period. This difference may be attributed to local flow effects created by undulations of the soil surface or by the proximity to the bifurcation node. The simulated discharge hydrograph is similar to the observed one, although the initial peak value is strongly overestimated.

The similarity between the observed and simulated two-dimensional advance pattern is remarkable, as shown in Fig. 7 for irrigation times 6, 14 and 23 min. The observed advancing front shows a fingering effect which can be readily attributed to the soil surface undulations: flow advances fast towards low spots and takes time to overflow local high spots. The measured standard deviation of soil surface elevation (14 mm) is high enough to create this effect. Both data sets reproduce the shift to the right of the

center of symmetry of the advancing pattern, and how this lack of symmetry is reduced in time, due to the equilibration of the discharges flowing through both distribution channels.

Time evolution of surface water and infiltrated water in level furrow irrigation

To follow how water flows through the network in level furrow irrigation, results of the surface and infiltrated water distribution were selected for $t = 6$ min. and $t = 23$ min. Contour maps of simulated flow depth (m) and infiltrated depth ($\text{m}^3 \text{m}^{-1}$) are presented in Fig.7. The furrows are numbered along the x axis, while the y axis runs along the furrow length. The inlet discharge was applied at the upstream end (length = 0) of furrow # 1. Hence, this point corresponds to a maximum flow and infiltrated depth. The infiltrated depth reaches a peak value close to $0.07 \text{ m}^3/\text{m}$ at the end of the advance phase. Minimum values of infiltration are found between the furrow extremes, where the opportunity time is lower.

Discharge distribution at the cutoff time

Figure 9 shows a three dimensional plot of the absolute values of discharge in the field at cutoff time. For furrows 1 to 13, water flows towards the left of the figure, while from furrows 14 to 40 flow is from both extremes and one of the nodal values of discharge is zero. The discharge inflowing furrows 1 to 40 through their upstream end (length = 0) adds up to the total irrigation discharge. Furrows 1 to 14 contribute to the discharge conveyed by the downstream distribution channel. The summation of the discharges in all 40 furrows at length = 50 m is zero (total inflow in 1 to 14 equals total

outflow in 15 to 40), since this channel does not have a net flow input. Discharge increases in both channels towards the end of the field (furrows 30 to 40), since a reflection wave produced after the channel flow hit furrow 40 is propagating backwards.

MODEL APPLICATION

Influence of the total inflow rate

Table 3 presents the results of simulating the irrigation of the experimental field using discharges ranging from 25 to 53 L s⁻¹. The average infiltrated depth (column 5) increased with the irrigation discharge from 26.1 mm to 33.9 mm. This is largely due to the direct relationship between infiltration and wetted perimeter (or discharge). As expected, an increase in the irrigation discharge also resulted in an increase in distribution uniformity, although doubling the discharge led to a modest increase in uniformity (six points). A decrease in the experimental discharge would have resulted in a lighter irrigation, at the expense of increasing the irrigation time.

In the companion paper, a level basin simulation of the experimental field irrigated with the measured discharge (53 L s⁻¹) produced an estimation of the time of advance of 185 min, with an average infiltrated depth of 223 mm. A level basin simulation using a discharge of 100 L s⁻¹ resulted in a time of advance of 61 min, and an average infiltrated depth of 0.141 m. In this particular case, if the experimental field was to be level basin irrigated, an increase in the irrigation discharge would be recommended in order to reduce the infiltrated depth to moderate values.

In the experimental conditions level furrow irrigation can be successfully operated at low discharges (below 50 L s^{-1}), while level basin irrigation requires a large inflow discharge (over 100 L s^{-1}). A simulation approach based on field infiltration experiments could be applied to each particular case in order to investigate optimum conditions for level furrow irrigation.

Influence of the base width of the distribution channel

Increasing the base width of the distribution channel (Table 3) from the experimental value of 0.33 m to 1.32 m led to a relevant decrease in the advance time (from 28.0 min to 19.4 min), and consequently in the average irrigation depth (from 33.9 mm to 23.5 mm). The effect on the distribution uniformity was not clear, since the differences were small (two points). Although an optimum uniformity value could be found in this particular case between base widths of 0.66 m and 0.99 m, the differences were considered non significant. The decrease in the average infiltrated depth is very important: a large distribution channel helps to obtain light irrigations. Since this is a common goal in surface irrigation systems, a design hint for level furrow irrigations is to ensure a large conveyance capacity at the distribution channels. This idea was already present at the manual by Erie and Dedrick (1979), where the authors also claimed that large distribution channels would help control erosion.

Influence of the soil characteristics

In order to characterize the influence of the soil properties (infiltration and roughness) on level furrow irrigation performance, three infiltration families (0.2, 0.5 and 1.0) and

two uniform values of Manning n ($n_1 = 0.04$, and $n_2 = 0.08$) were simulated in crossed combinations for a different case study (Table 4). The simulated field dimensions were 60 by 90 m (60 furrows, 60 m long and with a furrow spacing of 1.5 m). The furrow base was 0.13 m, and the side slope was 0.6. The distribution channel width was 0.52 m, and its side slope was 0.6. The infiltration equations were of the Kostiakov Lewis type:

$$Z = k\tau + f_0\tau \quad [28]$$

Where f_0 is the long-term infiltration rate ($\text{m}^3\text{m}^{-1}\text{min}^{-1}$). The Kostiakov-Lewis coefficients of the Soil Conservation Service infiltration families were obtained by Gharbi (1984) using regression techniques, and are presented in Table 4. In this exploratory analysis of the effect of soil infiltration on level furrow irrigation performance, the infiltration equation did not include a dependence on wetted perimeter or inflow discharge. The 0.2, 0.5 and 1.0 infiltration families can be roughly associated to the clay loam, silty loam and sandy loam soil textures, respectively.

The results presented in Table 4 include three parameters obtained from the simulation results: the time of advance, the time of recession and the average infiltrated depth.

For a given infiltration family all three parameters increase as the roughness coefficient increases. Similarly, for constant values of Manning n , the three parameters increase with increasing soil infiltration. Irrigation performance will therefore benefit from smoothly tilled soils free from vegetal growth. Light irrigations can be performed in soils with moderate to low infiltration, although the level furrow system did not produce excessive infiltration even in sandy loam soils.

SUMMARY AND CONCLUSIONS

The numerical model developed and presented in this work is able to simulate level-furrow irrigation flow in a variety of case studies. The numerical scheme used, second order explicit MacCormack, shows suitable for all the wet points, even those at the water front position which are problematic from the numerical point of view. Numerical stability was achieved by means of time steps compatible with a *CFL* value of 0.85. The friction term was discretized in a semi-implicit way in order to help to control stability at zones of low depth.

The experiments performed in the companion paper provided a case for model validation. The simulated results produced adequate agreement with the observed advance curve. The final flow depth and discharge at the upstream side of the first furrow were properly simulated. The time evolution of the location of the advancing front at the furrows was properly simulated by the model, although the observed advance showed a fingering effect that was attributed primarily to local differences in soil surface elevation.

The analysis of a number of case studies revealed that the conveyance capacity of the distribution channels must be high in order to be able to apply light irrigations. Due to the effect of discharge on furrow wetted perimeter and ultimately on infiltration, large discharges can result in large average infiltrated depths. Therefore, relatively small discharges may lead to optimum level furrow performance. Finally, although this irrigation system benefits from smoothly tilled, fine textured soils, irrigation performance on rough, sandy soils can be acceptable.

The proposed simulation model can be a valuable tool in the design and management of level furrow systems. Particularly, this model can be used in combination with level basin irrigation models to compare irrigation performance under both irrigation systems. This is particularly interesting since a farmer can switch from level basin to level furrow irrigation by performing a tillage operation. In a context of growing concerns about agricultural water use and the decrease in rural population, the level basin and level furrow irrigation systems stand as interesting options. In fact, they were long ago classified as methods for conserving water and labor (Erie and Dedrick, 1979).

APPENDIX: REFERENCES

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APPENDIX: NOTATION

α	Implicitness coefficient for the friction term in the numerical scheme;
β	Directional coefficient at junctions;
δ	Spatial derivative;
τ	Opportunity time for infiltration;
E	Numerical error
A	Cross sectional wetted area;
a	Infiltration parameter in the Kostiakov-Lewis equation;
b	Cross sectional free surface width;
c	Small surface wave celerity;
CFL	Courant-Friedrichs-Lewy stability parameter;
$\mathbf{F}$	Flux vector;
Fr	Froude number;
g	Acceleration of gravity;
$\mathbf{H}$	Source term vector;
h	Water depth;
h_c	Critical depth;
h_s	Subcritical depth;
I_1	Hydrostatic pressure integral at the wetted cross section;
I_2	Hydrostatic pressure integral at the solid walls;
i	Infiltration rate;
j	Subindex for the computational nodes;
k	Infiltration coefficient in the Kostiakov-Lewis equation;

M	Subindex of auxiliary point for the analysis of bifucations;
N	Number of computational nodes in a furrow;
n	Manning's roughness coefficient;
Q	Discharge;
R	Hydraulic radius;
s	Subindex of auxiliary point for the analysis of bifucations;
S_0	Bed slope;
S_f	Friction slope in the main flow direction;
t	Time;
U	Vector of conserved variables;
u	Flow velocity;
UD	Uniformity of distribution;
V	Volume of water;
x	Spatial coordinate;
Z	Infiltrated dose;
$\bar{Z}$	Average infiltrated dose;
$\bar{Z}_{25}$	Average infiltrated depth in the 25% of the field area receiving less water

APPENDIX: TABLES

Table 1. *Advance and reference depth and discharge results for different simulation conditions based on the use of uniform but different values of Manning n , and four implementations of the furrow infiltration equation.*

Table 2. *Advance and reference depth and discharge results for different simulation conditions based on the use of four non-uniform sets of values of Manning n . Nodal discharges were used in all cases for infiltration estimation.*

Table 3. *Influence of the irrigation discharge and the base width of the distribution channel on level furrow irrigation performance.*

Table 4. *Influence of the soil infiltration properties and the Manning n on level furrow irrigation performance.*

Table 1. *Advance and reference depth and discharge results for different simulation conditions based on the use of uniform but different values of Manning n, and four implementations of the furrow infiltration equation.*

Case	Manning n	Infiltration equation	Advance (%) at different times (min)						Reference	
			1	6	14	19	23	30	Depth (m)	Discharge (L s ⁻¹)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
n20u	0.020	Uniform	7.60	31.13	52.40	63.77	72.11	85.34	0.141	8.93
n20i		Q inlet	6.57	28.48	49.80	61.57	70.15	84.17	0.139	9.38
n20n		Q nodal	7.50	32.99	56.91	70.64	80.88	95.93	0.141	9.30
n20a		Q average	7.99	34.95	65.34	84.26	96.72	100.00	0.144	9.27
n25u	0.025	Uniform	7.06	30.10	50.15	60.88	68.63	81.32	0.152	8.72
n25i		Q inlet	6.23	27.50	47.79	56.68	66.96	80.88	0.149	9.12
n25n		Q nodal	6.96	31.86	54.56	67.16	76.72	91.81	0.151	9.06
n25a		Q average	7.40	33.48	60.69	77.70	90.59	100.00	0.153	9.05
Experimental			5.85	31.25	70.90	84.15	94.05	99.60	0.140	5.13

Table 2. *Advance and reference depth and discharge results for different simulation conditions based on the use of four non-uniform sets of values of Manning n. Nodal discharges were used in all cases for infiltration estimation.*

Case	Manning n		Advance (%) at different times (min)						Reference	
	Canals	Furrows	1	6	14	19	23	30	Depth (m)	Discharge (L s ⁻¹)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
n15/40n	0.015	0.040	7.16	38.19	69.51	83.04	91.76	100.00	0.136	5.47
n15/50n	0.015	0.050	6.96	38.24	71.76	84.85	93.28	100.00	0.138	4.77
n20/40n	0.020	0.040	6.76	35.64	63.43	76.76	85.93	98.09	0.146	6.20
n20/50n	0.020	0.050	6.57	35.49	65.98	78.87	87.60	98.68	0.148	5.43
Experimental			5.85	31.25	70.90	84.15	94.05	99.60	0.140	5.13

Table 3. *Influence of the irrigation discharge and the base width of the distribution channel on level furrow irrigation performance.*

Design Parameter		Irrigation times (min)		$\bar{Z}_{25}$	$\bar{Z}$	UD
		Advance	Recession	(mm)	(mm)	(%)
(1)		(2)	(3)	(4)	(5)	(6)
Irrigation Discharge (L s ⁻¹)	25	45.6	73.3	18.7	26.1	71.68
	35	36.3	68.3	21.7	29.1	74.60
	45	30.9	66.9	24.5	31.8	76.81
	53	28.0	66.9	26.5	33.9	78.00
Base width of the distribution channel (m)	0.33	28.0	66.9	26.5	33.9	78.00
	0.66	21.3	44.8	19.6	25.8	75.89
	0.99	19.5	39.5	18.0	23.7	75.97
	1.32	19.4	39.4	18.2	23.5	77.45

Table 4. *Influence of the soil infiltration properties and the Manning n on level furrow irrigation performance.*

Manning n	Infiltration family	k $\text{m}^3\text{m}^{-1}\text{min}^{-a}$	a -	f_0 $\text{m}^3\text{m}^{-1}\text{min}^{-1}$	Time of Advance (min)	Time of Recession (min)	$\bar{Z}$ (mm)
(1)	(2)	(3)	(4)	(5)	(8)	(9)	(7)
0.04	0.2	0.00346	0.388	0.000057	48.5	100.4	32.3
0.04	0.5	0.00320	0.504	0.000117	58.9	189.7	39.3
0.04	1.0	0.00332	0.598	0.000212	77.5	229.4	51.6
0.08	0.2	0.00346	0.388	0.000057	68.0	184.0	45.3
0.08	0.5	0.00320	0.504	0.000117	81.6	224.8	54.4
0.08	1.0	0.00332	0.598	0.000212	104.1	295.0	69.4

APPENDIX: FIGURES

Figure 1. *Level furrow network sketch showing the irrigation furrows, the distribution channels and the entrance of the irrigation water.*

Figure 2. *Scheme applied at a triple junction.*

Figure 3. *Discretization of: a) Confluence; and b) Bifurcation.*

Figure 4. *Discretization of the spatial derivative: a) Downstream; and b) Upstream.*

Figure 5. *Advance diagram for the level furrow experiment (dots) and for the simulated level furrow cases $n_{15/50n}$ (Manning n 0.015 at the distribution channels and 0.05 at the irrigation furrows; nodal discharge), n_{20n} (uniform Manning n of 0.02 and nodal discharge) and n_{20u} (uniform Manning n of 0.02 and uniform discharge).*

Figure 6. *Time evolution of measured and simulated flow depth and discharge at the inlet of furrow #1.*

Figure 7. *Observed vs. simulated location of the advancing front at times 6, 14 and 23 min.*

Figure 8. *Three dimensional plot of simulated flow depth and infiltrated depth in the level furrow setup at times 6 and 23 min.*

Figure 9. *Three dimensional plot of the absolute value of nodal discharge at the time of cutoff (31 min).*

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