Developing a modeling tool for flow profiling in irrigation distribution networks

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Abstract: Efforts are underway to rehabilitate the irrigation districts, such as in the Rio Grande Basin in Texas. Water distribution network models are needed to help prioritize and analyze various rehabilitation options, as well as to scientifically quantify irrigation water demands, usages, and losses, and to help manage gate automation. However, commercially available software packages were limited in applications due to their high cost and operational difficulty. This study aims to develop a modeling tool for modeling the water flow profile in irrigation distribution networks. The goal of developing the modeling tool was to make the modeling process simple, fast, reliable and accurate. On the basis of methodological study, the modeling tool has been developed for branching canal networks with the assumption of steady gradually varied flow. The flow profile calculation of the tool was verified from a single channel with 1% root mean squared error compared to the benchmark calculation and a branching network with 5% to 12% relative errors compared to check point measurement along the network. The developed modeling tool will be able to play an important role in water quantification for planning, analysis and development for modernization of irrigation systems.

Keywords: irrigation distribution network, modeling tool, flow profiling

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1 Introduction

Irrigation distribution networks are used extensively for agricultural water supply. Irrigation districts deliver water to farms through the channels and pipelines. Efforts are underway to rehabilitate the irrigation districts. Quantitative evaluation tools are needed to help prioritize and analyze various rehabilitation options, as well as to scientifically quantify irrigation water demands, usages, and losses, and to help manage gate automation. There has been much research in developing computer models and software packages for water resources planning and management through the past three decades[1]. Models and software packages are commercially or research available for flow modeling and gate automation of irrigation channels. Examples are: SOBEK (Delft Hydraulics, Delft, Netherlands), an integrated 1D/2D modeling program for water management, design, planning and policy making in river, rural and urban systems (http://www.sobek.nl/prod/index.html); CanalCAD (Laboratoire d'Hydraulique de France, Grenoble, France; Parrish Engineering, Beaverton, Oregon, USA), a hydrodynamic simulator of both steady and unsteady flow in canal systems with manual or automatic gates (http://www.iihr.uiowa.edu/projects/

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canalcad/index.html); Mike 11 (Danish Hydraulic Institute, Hørsholm, Denmark), a versatile and modular engineering software tool for modeling conditions in rivers, lakes/reservoirs, irrigation canals and other inland water systems (http://www.dhisoftware.com/mike11); SIC (Cemagref, Antony Cedex, France), a simulation model for canal automation design (http://canari.montpellier.cemagref.fr/papers/sic30.pdf); HEC-RAS (IWR, US Army Corps of Engineers, Davis, California, USA), a software package that allows one-dimensional steady and unsteady flow calculations in natural channels (http://www.hec.usace.army.mil/software/hec-ras); and CanalMan (Utah State University, Logan, Utah, USA) a model that performs hydraulic simulations of unsteady flow in branching canal networks (http://www.engineering.usu.edu/bie/software/canalman.php). These models or software packages are for general use and either expensive, such as SOBEK and CanalCAD or are difficult to be customized for applications under specific conditions even free downloadable, such as HEC-RAS and CanalMan.

Models have been evaluated for irrigation systems. Wallender[2] has done model simulation for both a single furrow as well as on a field-wide basis. Model simulations were evaluated to determine the importance to irrigation performance of each spatially-varying model input. Esfandiari and Maheshwari[3] studied four furrow irrigation models, referred to as the Ross, Walker, Strelkoff and Elliott models for their prediction of advance and recession times and runoff, and for their computational time per simulation run and volume balance error under three field conditions in south-east Australia. Hidalgo et al.[4] developed a procedure for calibrating on-demand irrigation network models. This procedure compared a new objective function with two more commonly used objective functions. This procedure was applied to an on-demand irrigation network located in Tarazona de La Mancha (Albacete, Spain) where flow and pressure at hydrant level was measured. Islam et al.[5] presented a hydraulic simulation model developed for steady and unsteady flow simulation in irrigation canal network. The model uses the implicit four-point Preissmann scheme for discretization of the Saint-Venant equations and solves the resulting equations using the sparse matrix solution technique. The model is applicable for simulating flow in a series of linearly connected reaches, and branched as well as looped canal networks. In general, unsteady gradually varied flow (USGVF) can be described by the Saint-Venant equations[6]. These equations are simultaneous partial differential equations with a number of boundary conditions. However, in practice use of a unsteady canal model requires serious investments of time and personnel[7]. As a special case of USGFV, steady gradually varied flow (SGVF) can be described by a single ordinary differential equation[6], which is much more easily implemented than the Saint-Venant equations. In many cases the description of SGVF is very useful and effective and the USGVF could be simplified to cascaded SGVFs in solving problems in flow computation and analysis.

The objective of the study was to develop a modeling tool based on the description of SGVF for modeling the water flow profile in irrigation distribution networks in the Rio Grande Basin in Texas and other similar areas. The developed modeling tool will make the modeling process simple, fast, reliable and accurate.

2 Study area

Irrigated lands in different areas have different characteristics. This study will focus on the irrigated areas with the following characteristics:

- The waterways are shallow and have small hydraulic gradients. In other words, the channel bottom slope is small and the water flows mildly from upstream to downstream with gravity and sufficient head pressure;
- The distribution networks are dendritical, i.e. the routes of the networks are branched but not looped.
- The networks are open channels

In the Lower Rio Grande Valley in Texas (Figure 1), the project area, the elevations range from sea level in the east to about 200 m in the northwest, but are mainly less than 100 m. Much of the area is nearly level. Drainage ways are shallow and have low gradients. The canals and pipelines in the distribution networks have small hydraulic gradients with few relief pumps. The objects of this study will be irrigation canals.

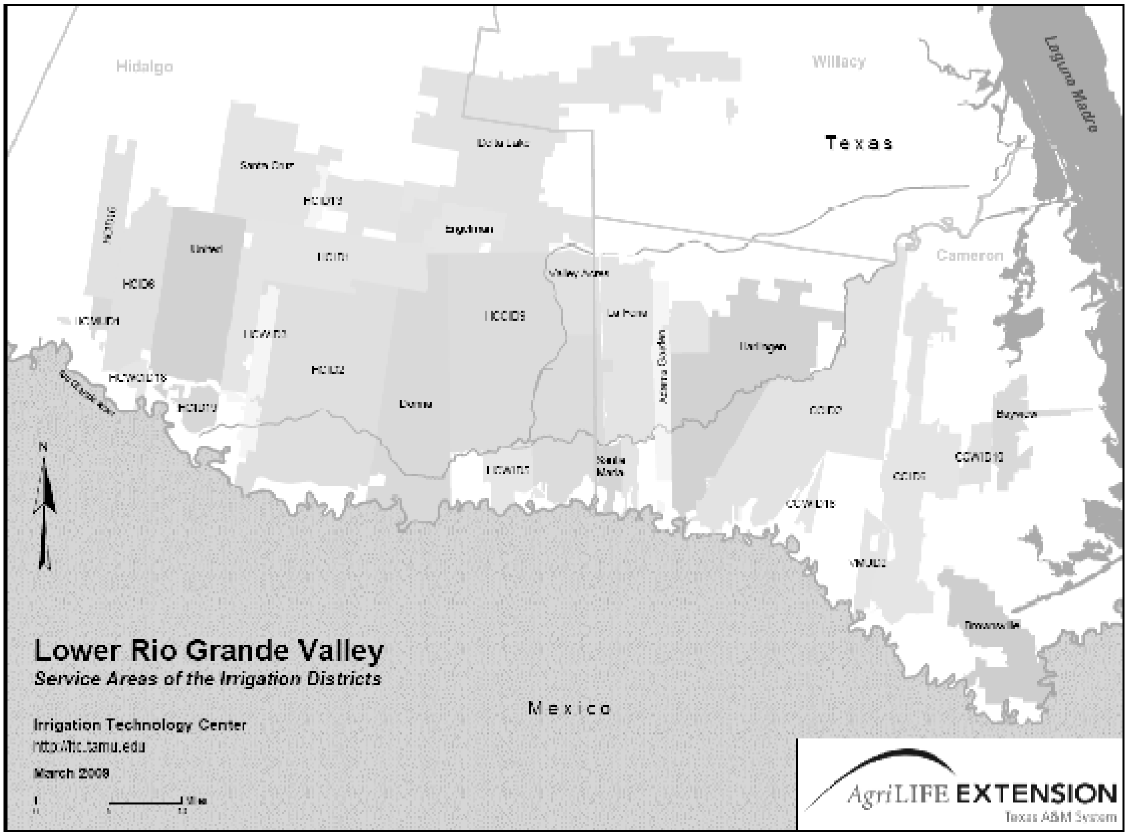

Figure 1 Service areas of the irrigation districts in Lower Rio Grande Valley of Texas

3 Computing methods

In the open-channels (canals) of irrigation networks, water flows are typically categorized as:

1) Steady uniform flow (SUF); 2) Steady gradually varied flow (SGVF); and 3) Unsteady gradually varied flow (USGVF).

The SGVF can be computed and analyzed by observing the conservation of mass and energy with an ordinary differential equation[6]. Further, the USGVF can be computed and analyzed using the Saint-Venant equations observing the conservation of mass and momentum[6]. It can be derived mathematically that the SGVF is a special case of the USGVF. The Saint-Venant equations are partial differential, so the implementation of the computation is much more difficult. In practice the SGVF is very useful and effective in solving a lot of problems in flow computation and analysis. With the fundamental equation the solutions can be cascaded along a canal channel and the layout of a distribution network under different initial and boundary hydraulic conditions.

Non-uniform flow is the prevailing flow conditions in irrigation systems. For the area the irrigation channels are shallow and have small hydraulic gradients such as the Rio Grande Basin in Texas the SGVF is the dominate flow type unless some transient processes typically happened around gate structures would result in the USGVF flow condition. Therefore, the computation of the SGVF profiles in irrigation distribution networks is the technique needed in developing the modeling tool.

3.1 SGVF flow profile computation

The computation of the SGVF profiles basically solves the governing ordinary differential equation. The main objective of the computation was to determine the shape of the flow profile. Broadly three methods of the computation were classified as[6]: the graphical-integration method, the direct-integration method, and the step method.

The graphical-integration procedure is straightforward and easy to follow but it may become very laborious when applied to actual problems. Because the differential equation of the SGVF cannot be expressed explicitly in terms of y for all types of channel cross sections, a direct and exact integration of the equation is practically impossible; hence, so far this method has been developed either to solve the equation for a few special cases or to introduce assumptions that make the equation amenable to mathematical integration[6]. Basically a step method is to divide a channel into short reaches and carry the computation step by step from one end of the reach to the other. There are a great variety of step methods. Some appear superior to others in certain respects, but no one has been found to be the best in all applications.

This study gives a step method based on the need of flow profile computation for irrigation channels. This method divides a channel to small reaches. The length of the reaches cannot be too big because this may cause the iterative procedure to fail, and cannot be too small either because this should increase computational burden. With the divided reaches the computation starts from the downstream end of the channel for subcritical flow (from the upstream end for supercritical flow) by applying the Bernoulli equation to the reach:

$$y_u + a\,\frac{v_u^2}{2g} + S_o\Delta x = y_d + a\,\frac{v_d^2}{2g} + S_f\Delta x \qquad (1)$$

where v_u and v_d are the flow velocities at the upstream and downstream ends of the reach respectively; α is the velocity distribution coefficient which takes into account that in channel cross-section the distribution of velocity is not uniform; Δx is the length of the reach; S_o is the channel bottom slope.

The solution of the equation for subcritical flow will be water depth h and water level $z=y+\Delta z$ at the upstream end of the reach where Δz is the difference between the elevations at the upstream and downstream ends of the reach. Equation (1) can be reformed to solve the water depth at the upstream end of the reach:

$$y_u = y_d + a\,(\frac{v_d^2}{2g} - \frac{v_u^2}{2g}) - S_o\Delta x + S_f\Delta x \qquad (2)$$

and

$$S_f = \frac{n^2Q^2}{\bar{A}^2\bar{R}^{4/3}}$$
$$\bar{A} = \frac{A_u + A_d}{2} \qquad (3)$$
$$\bar{R} = \frac{R_u + R_d}{2}$$

where A_u and A_d are the channel cross section areas of the upstream and downstream ends respectively; R_u and R_d *are* the channel hydraulic radiuses of the upstream and downstream ends respectively.

With the solutions as the initial conditions the equation can be applied to the next reach and so on.

The computation at each reach is an iterative process. Given Q, n, S_o, and channel cross section parameter such as bottom width b and side slope s for a trapezoid cross section, at the beginning the upstream end water depth y_u *was* set to be the downstream end water depth y_d which was from the solution of the previous reach or the initial condition at the channel downstream end, i.e. $y_u=y_d$. With the initial y_u a new estimate of the unknown water depth using equations (2) and (3) was calculated as $\hat{y}_u$. Then, the initial water depth was compared with the estimated depth with $|\hat{y}_u - y_u| < e$ where e was a pre-set small number for stopping the iteration. If the stopping condition is met, the iteration will stop and $\hat{y}_u$ is the solution; otherwise set $y_u = \hat{y}_u$ and continue the iteration.

3.2 Branching network SGVF flow profiling

The algorithm above can be used to compute SGVF flow profiling in a canal channel or a distribution network by cascading the solutions step by step along the canal channel and the layout of a distribution network under different initial and boundary hydraulic conditions.

Branching irrigation distribution networks are dominated in the studied areas. This kind of networks typically consists of laterals, second-level laterals, and even third-level laterals along a main canal. The flow profile computation over a branching network starts by initializing discharge and water depth at the one end of the main canal. Then when the computation proceeds to a lateral, the computation needs to continue by initializing discharge and water depth at the one end of the lateral. When the computation proceeds to a second-level lateral, the computation needs to continue by initializing discharge and water depth at the one end of the second-level lateral. Keep on going like this until the farm turnouts are reached and the computations needs to recursively go back to the main canal. The same procedure follows when the second, third, … laterals are met. The computation will stop when it proceeds to the other end of the main canal. Figure 2 shows the flow chart of the procedure of subcritical SGVF profile computation over a branching irrigation network. This procedure can handle the branching irrigation networks in arbitrary layouts as long as they only have the first-level laterals. This procedure can be easily extended to the cases of arbitrary branching networks with second-level, third-level, and *n*-level laterals.

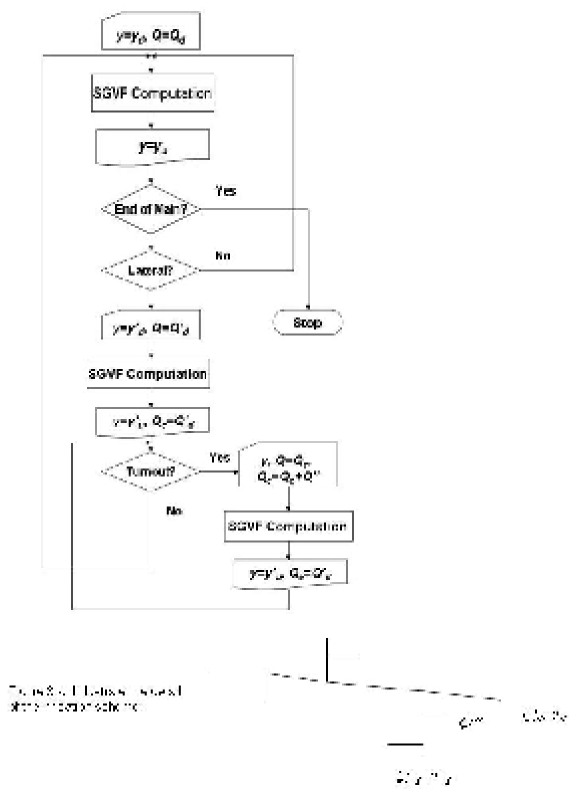

Figure 2 Flow chart of computation of subcritical SGVF profile over branch irrigation networks with first-level laterals

3.3 Gate calibration

Gate is the most popular structure for controlling water flow through irrigation channels. In general, four different flow regimes can occur at gate structures. Each of the four regimes has a standard equation to characterize the flow through the gate structure[9,10]:

1) Free orifice (FO): it is free gated flow

$$Q = C_{fo} L G_o \sqrt{2g(y_u - 0.5G_o)} \quad (4)$$

where L is the gate size; G_o is the gate opening; y_u is the water depth upstream of the gate structure; C_{fo} is the discharge coefficient of the FO flow.

2) Submerged orifice (SO): it is submerged gated flow

$$Q = C_{so} L y_d \sqrt{2g(y_u - y_d)} \quad (5)$$

where y_d is the water depth downstream of the gate structure; C_{so} is the discharge coefficient of the SO flow.

3) Free non-orifice (FN): it is free weir flow

$$Q = C_{nf} L \sqrt{y_u} \quad (6)$$

where C_{nf} is the discharge coefficient of the FN flow.

4) Submerged non-orifice (SN): it is submerged weir flow

$$Q = C_{sn} L y_d \sqrt{2g(y_u - y_d)} \quad (7)$$

where C_{sn} is the discharge coefficient of the SN flow.

In practice, although water flow can transit from one regime to the other, many canal gate structures and channel constrictions such as flumes operate mostly under a single flow regime.

To use any one of the equations (4), (5), (6), and (7) to characterize the water flow though a specific gate structure, the corresponding discharge coefficient, C_{fo}, C_{so}, C_{fn}, or C_{sn}, needs to be determined (calibrated). The calibration procedure is as follows[9]:

1) Conduct field survey around the concerned gate structure: gate dimensions and gate upstream and downstream channel hydraulic characteristics.

2) Determine the flow regime by experience or by some computation about water flow through the gate structure.

3) Find out the standard equation of a specific gate structure for the determined flow regime: equation (4), (5), (6), or (7).

4) Rearrange the equations (4), (6), and (7) in the following general form:

$$Qp(y_u, y_d, G_o, L) = Cq(y_u, y_d, G_o) \quad (8)$$

where C is C_{fo}, C_{fn}, or C_{sn}.

Equation (4) can be assumed in the form:

$$\log_{10}(C_{so})=a+b*\log_{10}(y_d/G_o) \quad (9)$$

where a and b are regression coefficients.

I. Based on the n sequential measurements of (Q_i, y_u^i, y_d^i, G_o^i) (i=1,2,…,n), calculate ($q_i(y_u^i, y_d^i, G_o^i)$, $Q_i p_i(y_u^i, y_d^i, G_o^i, L)$) for equation (8) or ($\log_{10}(y_d^i/G_o^i)$, $\log_{10}(C_{so}^i)$) for equation (9) (i=1,2,…,n).

II. Based on the calculation, the regression equation is formulated as:

$$p=\hat{C}q/Q \quad (10)$$

for FO, FN, or SN flow where $\hat{C}$ is the estimated value of C, or

$$\log_{10}(C_{so})=\hat{a}+\hat{b}*\log_{10}(y_d/G_o) \quad (11)$$

where $\hat{a}$ and $\hat{b}$ are the estimated values of a and b respectively.

III. The performance of the calibration can be evaluated by calculating the standard deviations of residuals.

4 Modeling tool prototyping

Using the method above the modeling tool was programmed and developed. In order to make the modeling process simple, fast, and accurate, three modules have been developed:

1) SGVF computation for a single canal channel

2) SGVF computation for branching canal networks

3) Flow computation through control sections

The third module is for computing the flow through gates, weirs, and flumes. The discharge and depth relationships were calibrated and saved for model implementation.

C++ programming language was chosen for prototyping the modeling tool. The programs were designed and developed using the principles of OOP (Object-Oriented Programming).

5 Model validation

The water flow profiles in a single irrigation canal channel and a branching network irrigation scheme were computed for model validation. As the benchmark, the data of the single irrigation canal channel were taken from Chow (1959)[6]. The computing results were compared with Chow’s computation to validate the computation of this study.

The irrigation scheme is a real-world irrigation branching network (Figure 3), which spans about 1700 m and is located at an irrigation district Jamaica around an area that has similar characteristics as the Lower Rio Grande Valley of Texas. The data were measured and collected from the field survey and flow measurement. The computing results were verified with check point values along the irrigation system. In the scheme, the main canal goes through the points 1, 2, 3, 4, 5, 6, and 7. In the main canal at the upstream end is a sharp-crested weir (HS1). A siphon wall is in the middle (HS2). At the downstream end is another sharp-crested weir (HS5). Two laterals are from the main canal through two sluice gates: HS3 and HS4 respectively. HS3 was fully shut down during field survey and measurement. HS4 was open to allow water flow to go through the points 5a, 5b, 5c, 5d, 5e, 5f, 5g and 5h. HS 6 and HS7 are two sluice gates to the farm turnouts at 5f and 5h respectively.

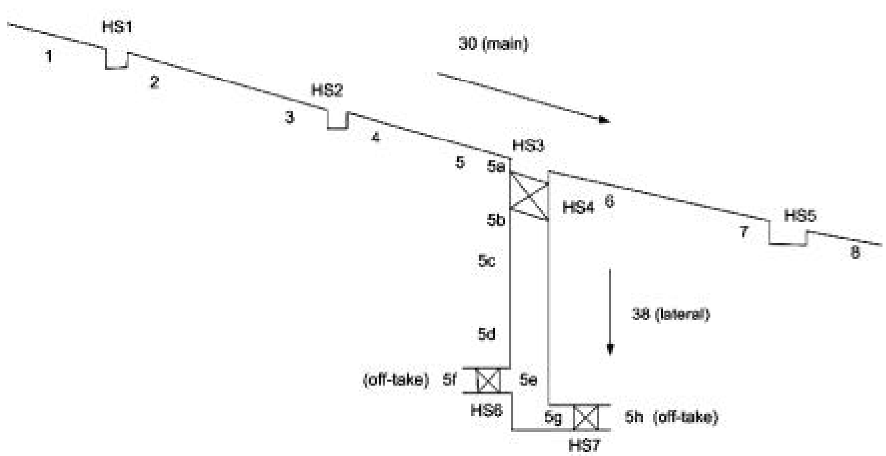

Figure 3 A branching network irrigation scheme

Before computing the water level profile, these sluice gates need to be calibrated. With the data collected during the field survey and flow measurement the results of the calibrations are shown in Table 1.

Table 1 Gate calibration in the branching network irrigation scheme.

Gate	Flow regime	Gate status	Gate flow equation
HS3	NA	Closed	NA
HS4	Free Weir Flow	Open	$Q = C_{nf} L y_u^{1.5}$ $C_{nfw} = C_{nf} / \sqrt{2g}$ $\hat{C}_{nfw}$ =0.0469
HS6	Submerged Orifice Flow	Open	$Q = C_{so} L y_d \sqrt{2g(y_u - y_d)}$ C_{so}=0.3148 $(y_d/G_o)^{-0.2917}$
HS7	Submerged Orifice Flow	Open	$Q = C_{so} L h_d \sqrt{2g(y_u - y_d)}$ C_{so}=0.609677$(y_d/G_o)^{-2.2873}$

With all of the data collected in field survey and flow measurement, and gate calibration equations, the modeling tool computed the water level profile over the branching network irrigation scheme automatically. A group of measured data was used to initialize model computation (from main and lateral downstream ends) and to verify the computation results at some check point through the network scheme. This group of data is listed Table 2.

Table 2 A group of measured data for model computation initialization and verification.

	Data	Usage
Head on HS5	0.22 m	Initial condition
Gate Opening at HS4	0.30 m	Boundary condition
Discharge over HS5	0.35 cms*	Initial condition
Gate Opening at HS6	0.37 m	Boundary condition
Discharge through HS6	0.01 cms	Initial condition
Gate Opening at HS7	0.58 m	Boundary condition
Discharge through HS7	0.03 cms	Initial condition
Depth upstream of HS7	0.255 m	Initial condition
Depth upstream to point 5	0.67 m	Verification
Head on HS1	0.21 m	Verification

Note: *cms – cubic meter per second.

6 Results and discussion

6.1 Single irrigation canal channel

Chow (1959) [6] gave an example of computing the subcritical water level profile in a trapezoid channel. This profile was created by a dam which backs up the water to a depth of 1.53 m immediately behind the dam. This channel carries a discharge of Q=11.33 cms with b= 6.10 m (channel bottom width), s=2 (channel side slope), S_o=0.0016, and n=0.025. The length of the profile is about 732 m.

Chow used two methods for the computation: the graphical-integration method and the direct step method. The computation of this study was compared with Chow's direct step computation. Table 3 shows this comparison. RMSE (Root Mean Squared Error) of this computation with Chow's computation is 0.011219 (1%), which indicates that the computation of this study is very close to what Chow computed. Figure 4 is the plot of the comparison of the computed water levels.

Table 3 Computed water level profiles of the trapezoid channel (1.53 m water depth behind the dam, Q=11.33 cms, b=6.10 m, s=2, S_o=0.0016, n=0.025, and length of the profile is 732 m)

x – distance to the channel downstream end /m	y – computed water level profile in this study /m	y' – computed water level profile by Chow (1959)[6] /m	$[(y-y')/y']^2$
0.00	1.52	1.52	0
47.09	1.54	1.53	3.92E-06
97.22	1.56	1.55	3.45E-05
148.86	1.59	1.57	9.32E-05
206.58	1.63	1.61	0.000129
270.38	1.67	1.65	0.000218
347.85	1.74	1.71	0.000256
396.46	1.79	1.76	0.000242
455.70	1.85	1.82	0.000225
493.68	1.90	1.87	0.000214
539.25	1.95	1.93	0.000159
575.70	2.00	1.97	0.000151
622.79	2.06	2.04	7.97E-05
663.80	2.12	2.10	5.22E-05
721.53	2.20	2.19	3.09E-05
		RMSE	0.011219(1%)

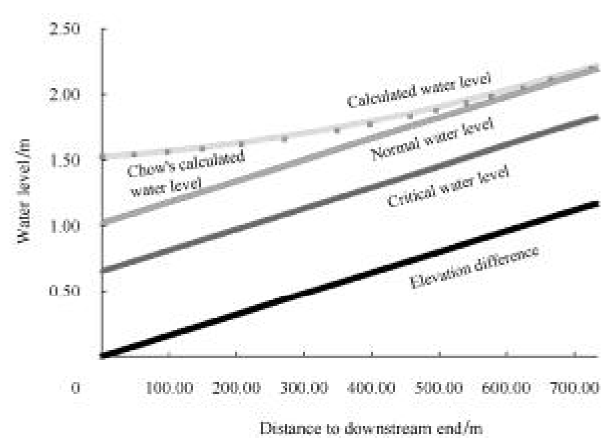

Figure 4 Plot of computed water level profiles of the trapezoid channel

6.2 Branching network irrigation scheme

All of the computing results are summarized in Table 4 compared with the measured data at check points. The following figures illustrate the numbers in the table.

Table 4 Modeling tool computation result summary

Location	Initial and computed discharge /cm • s^{-1}	Computed water depth /m	Measured water depth /m	Comment
Main downstream	0.35		0.22 m	Initial conditioned
Channel 6-7 (244 m)	0.35	0.49 m		
Channel 5e-5g (15 m)	0.03	0.23 m	0.26 m	Initial conditioned and the relative error between computed and measured water depth is 11.5%
Channel 5b-5e (122 m)	0.01	0.37 m		Initial condition
Channel 4-5 (244 m)	0.4	0.67 m	0.67 m	Based on the computation, the 0.67 m depth happened at about 6.1 m upstream point 5
Channel 2-3 (1,219 m)	0.4	1.07 m		
Main upstream Head on HS1	0.4	0.2 m	0.21 m	The relative error between computed and measured water depth is 5%

Figure 5 shows the computed water level profile in the downstream channel of the main canal (6-7). The channel length is about 244 m. The computed water level is very close to the normal depth at the distance of 3.05 m. So, if the distance is greater than 3.05 m, the flow can be considered uniform at the depth of 0.49 m.

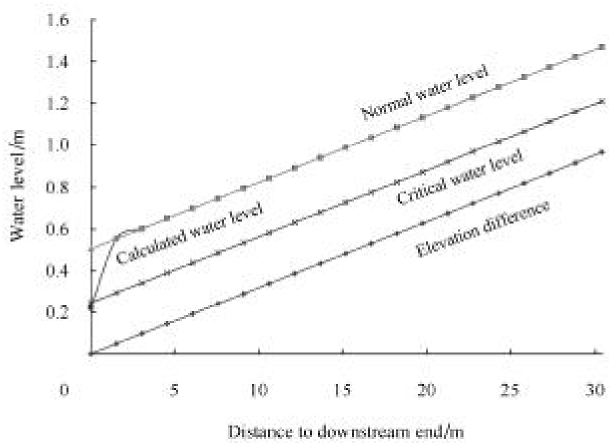

Figure 5 Computed water level profile of the main canal channel 6-7

Figure 6 shows the computed water level profile in the downstream channel of the lateral (5e-5g). The channel length is also about 15 m. The graphic indicates that the water level is going to but never reaches the normal depth. So, the flow in this channel is considered as pure gradually varied. The average water depth is about 0.23 m, which is 0.03 m away from the measured depth of 0.26 m (relative error is 12%).

Figure 7 shows the computed water level profile in the upstream channel of the lateral (5b-5e). The channel length is about 122 m. The computed water level is very close to the normal depth at the distance of 91 m. So, if the distance is greater than 91 m, the flow can be considered uniform at the depth of 0.37 m.

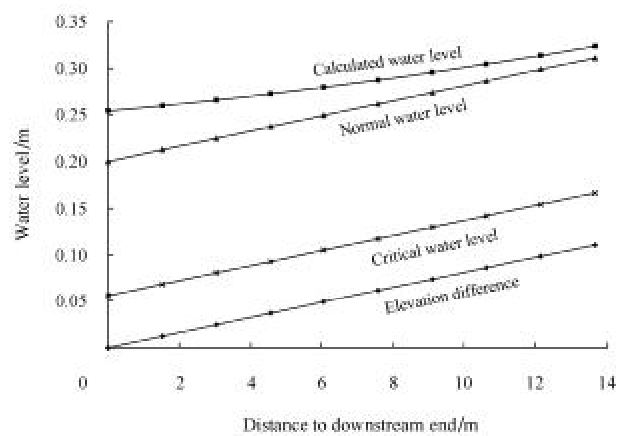

Figure 6 Computed water level profile in the downstream channel of the lateral 5e-5g.

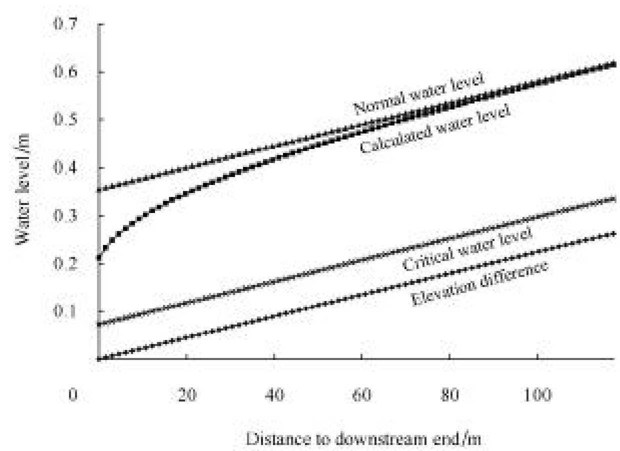

Figure 7 Computed water level profile in the upstream channel of the lateral 5b-5e

Figure 8 shows the computed water level profile in the intermediate channel of the main (4-5). The channel length is also about 244 m. The result indicates that the computed water level is going to but never arrives at the normal depth over the channel. So, the flow is considered as pure gradually varied. Using the depth curve, it can be derived that the 0.67 m water depth happened about 6.1 m upstream point 5, which matches the field measurement point.

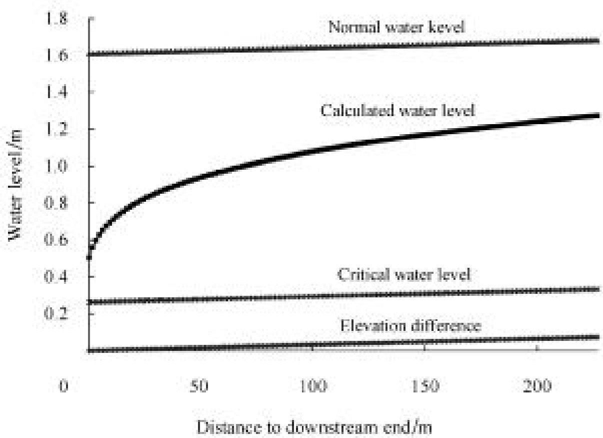

Figure 8 Computed water level profile in the intermediate channel of the main 4-5

Figure 9 shows the computed water profile in the upstream channel of the main (2-3). The channel length is about 1219 m. The water level is close to the normal depth at the distance of 244 m from the downstream end of the channel. So, if the distance is greater than 244 m, the flow can be considered uniform at the depth of 1.07 m.

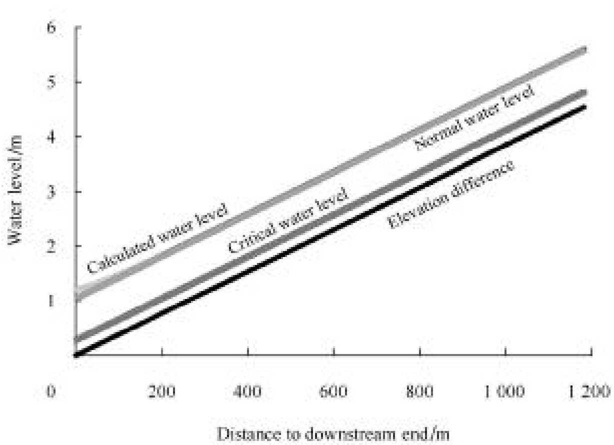

Figure 9 Computed water profile in the upstream channel of the main 2-3

Finally, the computation produced 0.2 m water head on the sharp-crested weir HS1, which is close to the measured depth of 0.21 m with the absolute error of 0.01 m (relative error is 5%).

7 Conclusions

This study has developed a tool for modeling the water flow profile in irrigation distribution networks. The modeling tool assumes SGVF flow in the branching canal networks. The tool starts the computation by initializing discharge and water depth at the end of the main canal and the laterals. It handles the branching networks in arbitrary layouts with first-level laterals. The method can be extended to the cases of arbitrary branching networks with second-level, third-level, and ***n***-level laterals. The modeling tool was evaluated in water flow profiling for a single irrigation canal channel and an irrigation branching canal network. The calculations of the modeling tool had a 1% RMSE compared to the benchmark calculation of a single channel flow and 5% to 12% relative errors compared to check point measurement along a branching canal network. The implementation and results of the modeling tool indicated a strong capability in handling the modeling task in different conditions such that the modeling process with the tool becomes simple, fast, reliable and accurate with much less cost and least configurations compared to commercially available models and software packages. The outcome of this study will be able to play an important role in water quantification for planning, analysis and development for modernization of irrigation systems for irrigation districts in the Lower Rio Grande Valley of Texas and any other similar areas.

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IJABE has been covered by Inspec

The International Journal of Agricultural & Biological Engineering (IJABE, http://www.ijabe.org) launched in 2008 has been accepted for coverage within the Inspec Database, after being successfully covered by Chemical Abstracts(CA), CAB Abstracts databases, and the CAB Full Text Repository.

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(By Wang Yingkuan)

Appendix: Email notice from Team Leader of INSPEC Acquisitions

Dear Dr Yingkuan,

Many thanks for your email, and apologies for the delay in replying.

I am pleased to confirm the International Journal of Agricultural & Biological Engineering has been accepted for coverage within the Inspec Database as from the issues that you have sent.

Please can you send issues as and when they are published to myself at the address mentioned below.

Kind regards,

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