Developing A Modeling Tool for Flow Profiling in Irrigation Distribution Networks

[1*]Y. Huang and [2]G. Fipps
[1]USDA-ARS, APTRU, Stoneville, MS 38776
[2]Texas A&M University, Department of Biological and Agricultural Engineering, College Station, TX 77843
(*Corresponding author – address: 141 Experiment Station Rd., Stoneville, MS 38776
E-mail: yanbo.huang@ars.usda.gov)

ABSTRACT

Efforts are underway to rehabilitate the irrigation districts, such as in the Rio Grande River 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 is to develop a modeling tool for modeling the water flow profile in irrigation distribution networks and addresses the issues of GIS (Geographic Information System) integration to aggregate spatial distributed information and GUI (Graphic User Interface) building for user-friendly access to enhance the tool. The goal of developing the modeling tool is to make the modeling process simple, fast, reliable and accurate. On the basis of theoretical study, the procedures and algorithms are developed and verified from a single channel to a branching network with higher complexity. 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, GIS, GUI

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. Models and software packages are commercially or research available for modeling and gate automation of irrigation channels, such as SOBEK (Delft Hydraulics, Delft, Netherlands), CanalCAD (Laboratoire d'Hydraulique de France, Grenoble, France; Parrish Engineering, Beaverton, Oregon, USA), Mike 11 (Danish Hydraulic Institute, Hørsholm, Denmark), SIC (Cemagref, Antony Cedex, France), HEC-RAS (IWR, US Army Corps of Engineers, Davis, California, USA), CanalMan (Utah State University, Logan, Utah, USA) etc. These software packages are costly, typically SOBEK and CanalCAD. They are for general use and difficult to be customized for applications under specific conditions. This study is to develop a modeling tool for modeling the water flow profile in irrigation distribution networks in the Rio Grande River Basin in Texas and other similar areas. The goal of developing the modeling tool is to make the modeling process simple, fast, reliable and accurate.

2. STUDY AREA

Irrigated lands in different areas have different characteristics. Our research 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 upsteam 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 both open channels and pipelines

In the Lower Rio Grande Valley in Texas, our major 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. Drainageways are shallow and have low gradients. Typically the canals and pipelines in the distribution networks have small hydraulic gradients with few relief pumps.

This study will focus on irrigation canals. The study and procedures for flow computation through pipeline systems have been well established (Jeppson, 1977; Larock et al., 2000; Walski et al., 2003; Watters, 1984; Watters, 1979). With the established procedures it is not hard to extend the procedures developed in this study to handle different type of irrigation systems altogether segment by segment: lined canal, unlined canal and pipeline.

3. THEORETICAL BACKGROUND

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 SUF is the fundamental flow type flow in open-channel hydraulics. Because unsteady uniform flow in practice is uncommon, “uniform flow (UF)” is usually used to refer only to steady uniform flow. The following equation is for computation and analysis of the UF (Chow, 1959):

$$Q = \frac{1.49}{n} AR^{2/3}\sqrt{S_o} \quad (1)$$

where Q is the discharge in cfs, n is the roughness factor of the channel called Manning coefficient, A is the channel cross-section area, R=A/P is the hydraulic radius and P is the wetted perimeter of the cross-section, and S_o is the channel bottom slope. With known S_o, n and normal depth y_n (ft), the equation gives the normal discharge Q (cfs). Inversely, when the discharge, the slope, and roughness are known, the equation can give the normal depth y_n because A and R are the function of y_n.

The SGVF can be computed and analyzed by observing the conservation of mass and energy (Chow, 1959). The following ordinary differential equation is with the assumption of small bottom-slope angle of prismatic channel (Chow, 1959):

$$\frac{dy}{dx} = \frac{S_o - S_f}{1 + \alpha d(V^2/2g)/dy} \tag{2}$$

where y is the water depth (ft), x is the length of the channel bottom (ft), α is the energy coefficient, V is the mean velocity of flow through the cross-section (fps), S_o is the channel bottom slope, and S_f is the energy line slope.

Further, the USGVF can be computed and analyzed using the Saint-Venant equations observing the conservation of mass and momentum (Chow, 1959). 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 these fundamental equations the solutions can be cascaded along a canal channel and the layout of a distribution network under different initial and boundary hydraulic conditions.

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 (Huang, 2004):

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

$$Q = C_{fo} L G_o \sqrt{2g(y_u - 0.5G_o)} \tag{3}$$

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

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

$$Q = C_{so} L y_d \sqrt{2g(y_u - y_d)} \tag{4}$$

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

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

$$Q = C_{nf} L \sqrt{y_u} \tag{5}$$

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

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

$$Q = C_{sn} L y_d \sqrt{2g(y_u - y_d)} \qquad (6)$$

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

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

4. AVAILABLE MODELS AND SOFTWARE PACKAGES

There has been much research in developing computer models and software packages for water resources planning and management through the past three decades (Wurbs, 1994). However, only a few of them are available in the market or engineering research for flow modeling in irrigation channels. Examples are: SOBEK, 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, a hydrodynamic simulator of both steady and unsteady flow in canal systems with manual or automatic gates (http://www.iihr.uiowa.edu/projects/canalcad/index.html); Mike 11, 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, a simulation model for canal automation design (http://canari.montpellier.cemagref.fr/papers/sic30.pdf); HEC-RAS, 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, 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.

5. COMPUTING METHODS

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 we need in developing the modeling tool.

5.1 SGVF Flow Profile Computation

The computation of the SGVF profiles basically solves the governing equation like equation (2). The main objective of the computation is to determine the shape of the flow profile. Chow (1959) broadly classified three methods of the computation: 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 (Chow, 1959). 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:

$$y_u + \alpha\frac{v_u^2}{2g} + S_o\Delta x = y_d + \alpha\frac{v_d^2}{2g} + S_f\Delta x \qquad (7)$$

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 account that in channel cross-section the distribution of velocity is not uniform, Δx is the length of the reach, and 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 (7) can be reformed to solve the water depth at the upstream end of the reach:

$$y_u = y_d + \alpha\left(\frac{v_d^2}{2g} - \frac{v_u^2}{2g}\right) - S_o\Delta x + S_f\Delta x \qquad (8)$$

and

$$\begin{aligned} S_f &= \frac{n^2Q^2}{\overline{A}^2\,\overline{R}^{4/3}} \\ \overline{A} &= \frac{A_u + A_d}{2} \\ \overline{R} &= \frac{R_u + R_d}{2} \end{aligned} \qquad (9)$$

where A_u and A_d is the channel cross section areas of the upstream and downstream ends respectively, and R_u and R_d is 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 is set to be the downstream end water depth y_d which is 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 (8) and (9) is calculated as $\hat{y}_u$. Then, compare the initial water depth and the estimated depth with $|\hat{y}_u - y_u| < \varepsilon$ where ε is 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.

5.2 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 area we studied. 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 1 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, … laterals.

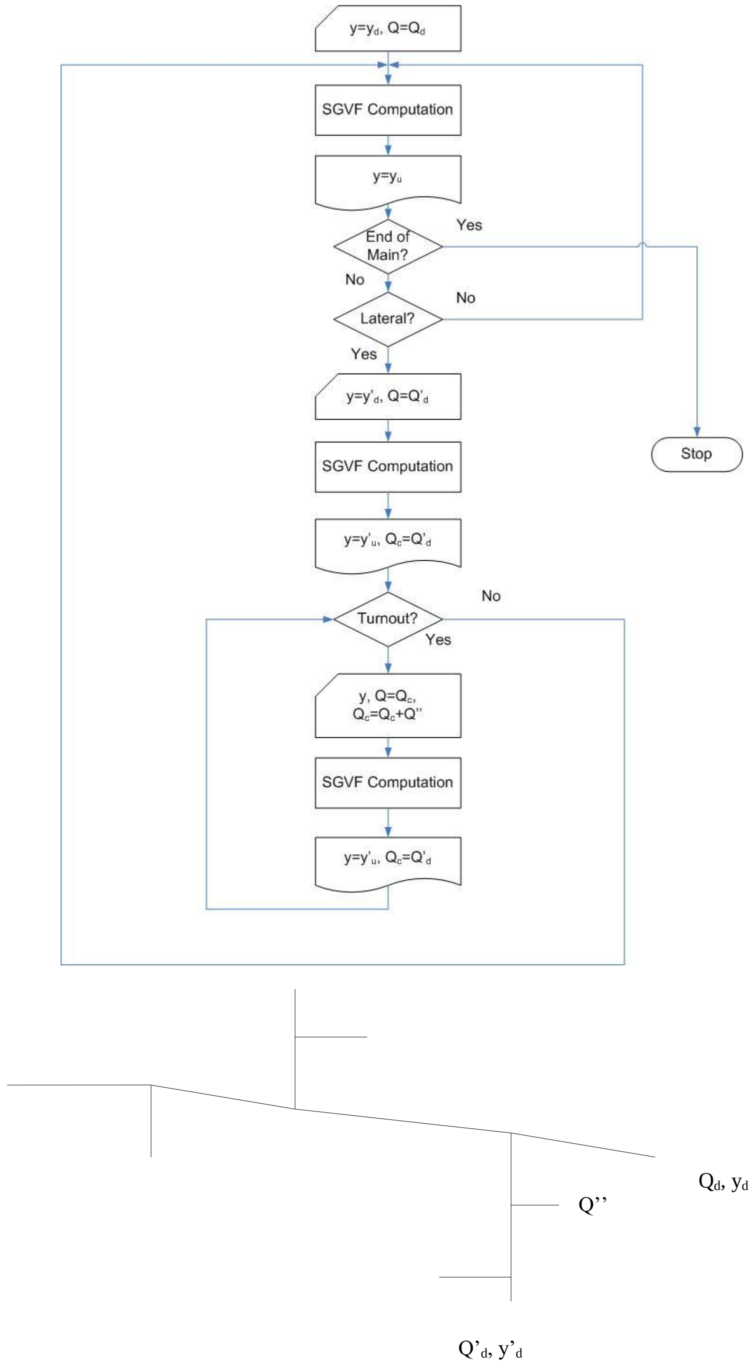

Figure 1: Flow chart of computation of subcritical SGVF profile over branch irrigation networks with first-level laterals.

5.3 Gate Calibration

To use any one of the equations (3), (4), (5), and (6) 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 (Huang, 2004):

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 (3), (4), (5), or (6).
4. Rearrange the equations (3), (5), and (6) in the following general form:

$$Qp(y_u, y_d, G_o, L) = Cq(y_u, y_d, G_o) \qquad (10)$$

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

   Equation (4) can be assumed in the form:

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

   where a and b are regression coefficients.
5. Based on the n sequential measurements of $(Q_i, y_u^i, y_d^i, G_o^i)$ (i=1,2,…,n), calculate $(q_i(y_u^i, y_d^i, G_o^i), Q_ip_i(y_u^i, y_d^i, G_o^i, L))$ for equation (10) or $(\log_{10}(y_d^i/G_o^i), \log_{10}(C_{so}^i))$ for equation (11) (i=1,2,…,n).
6. Based on the calculation, the regression equation is built as:

$$p = \hat{C}q/Q \qquad (12)$$

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

$$\log_{10}(C_{so})=\hat{a} + \hat{b}*\log_{10}(y_d/G_o) \qquad (13)$$

   where $\hat{a}$ and $\hat{b}$ are the estimated values of a and b respectively.
7. The performance of the calibration can be evaluated by calculating the standard deviations of residuals.

6. 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).

6.1 GIS Integration

Irrigation distribution networks are spatially distributed. It is beneficial to integrate GIS with a model of irrigation distribution network because in this way during the computation the model can use the spatial distributed hydraulic parameter data of the canal channels in the network from GIS and the GIS can present the results of the model computation for analysis from spatial distribution point of view.

GIS is specialized for spatial data management, visualization and analysis. For modeling, GIS can manage hydraulic data and even topographical information of irrigation systems, as well as the model output results. The integration of GIS data sources can be done in a number of ways. The most straightforward way is to let the models access to GIS data sources directly. In ArcView (Redlands, CA), attributes describing map features are stored in dBase tables. C++ model programs can input data from and output results to ArcView directly either by translation (Import/Export) between dBase tables and text files or through data source connectivity such as ODBC (Open DataBase Connectivity).

6.2 GUI Construction

A user-friendly interface is important for the success of the modeling tool. GUI technology provides an effective method to construct such an interface between users and the modeling tool. In general, there are two approaches to construction of GUI. One is to construct a standalone GUI. Through a standalone GUI users can access to the modeling tool in an individual computer. Internet technology provides another approach to construction of GUI, Web-based GUI. Through a Web-based GUI users can access to the modeling tool in any client computers or terminals of the Web server. A modeling tool with Web-based GUI is powerful for geographically distributed users' access at the same time.

7. RESULTS AND DISCUSSION

The water flow profiles in a single irrigation canal channel and an irrigation scheme were computed. The data of the irrigation canal channel was from Chow (1959). The computing results were compared with Chow's computation. The irrigation scheme is a real-world irrigation branching network. 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.

7.1 Single Irrigation Canal Channel

Chow (1959) gave an example of computing the subcritical water level profile in a trapezoid channel. This profile is created by a dam which backs up the water to a depth of 5 ft immediately behind the dam. This channel carries a discharge of Q=400 cfs (11.33 cms) with b=20 ft (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 2,400 ft (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 1 shows this comparison. MSE (Mean Squared Error) of this computation with Chow's computation is 0.004427 ft^2 (0.000411 m^2) and the square root of the MSE (SRMSE) is 0.066533 ft (0.020279 m). Figure 2 is the plot of the comparison of the computed water levels.

Table 1. Computed water level profiles of the trapezoid channel.

x – distance to the channel downstream end (ft)	**y – computed water level profile in this study (ft)**	**y' – computed water level profile by Chow (1959) (ft)**	**(y-y')^2**
0	5	5	0
155	5.06	5.05	1E-04
320	5.14	5.11	0.0009
490	5.23	5.18	0.0025
680	5.35	5.29	0.0036
890	5.5	5.42	0.0064
1145	5.72	5.63	0.0081
1305	5.88	5.79	0.0081
1500	6.09	6	0.0081
1625	6.24	6.15	0.0081
1775	6.42	6.34	0.0064
1895	6.58	6.5	0.0064
2050	6.78	6.72	0.0036
2185	6.97	6.92	0.0025
2375	7.24	7.2	0.0016
		MSE	0.004427
		SRMSE	0.066533

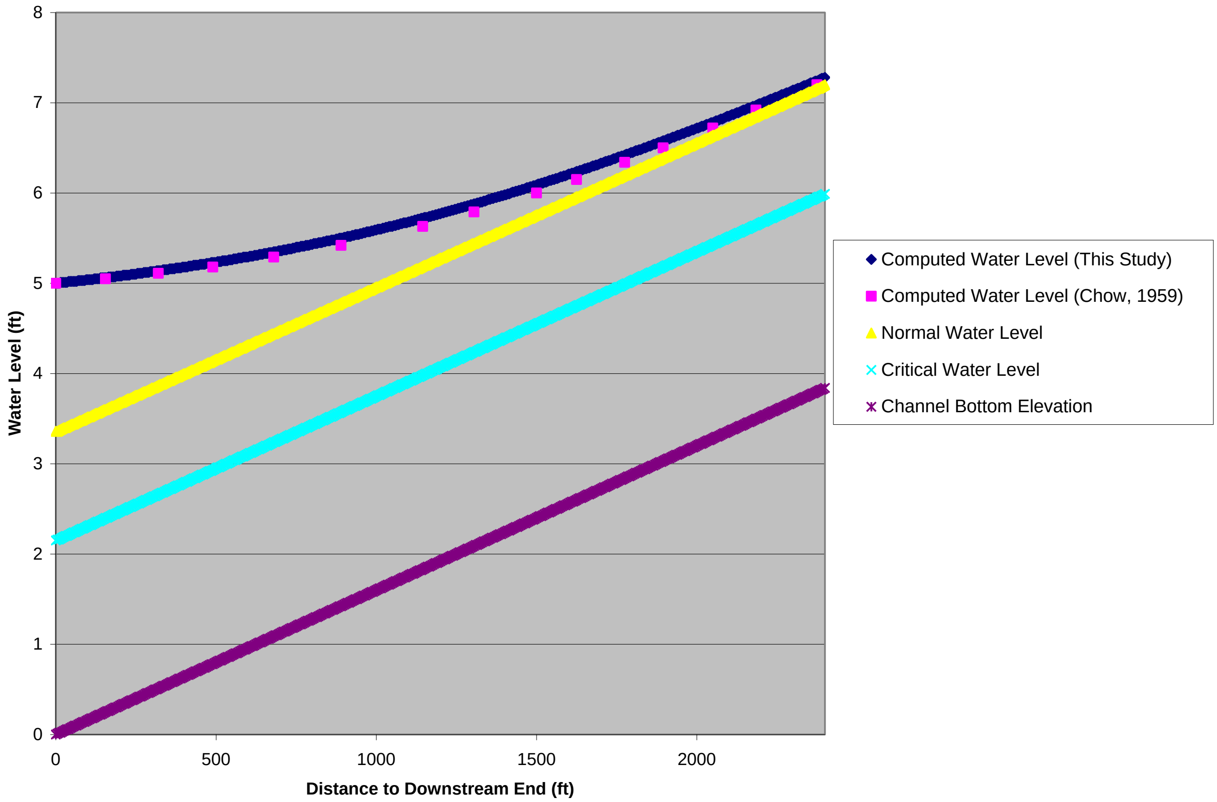

Figure 2: Plot of computed water level profiles of the trapezoid channel.

7.2 Branching Network Irrigation Scheme

A branching network irrigation scheme in practice is shown in Figure 3. 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.

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 2.

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 3.

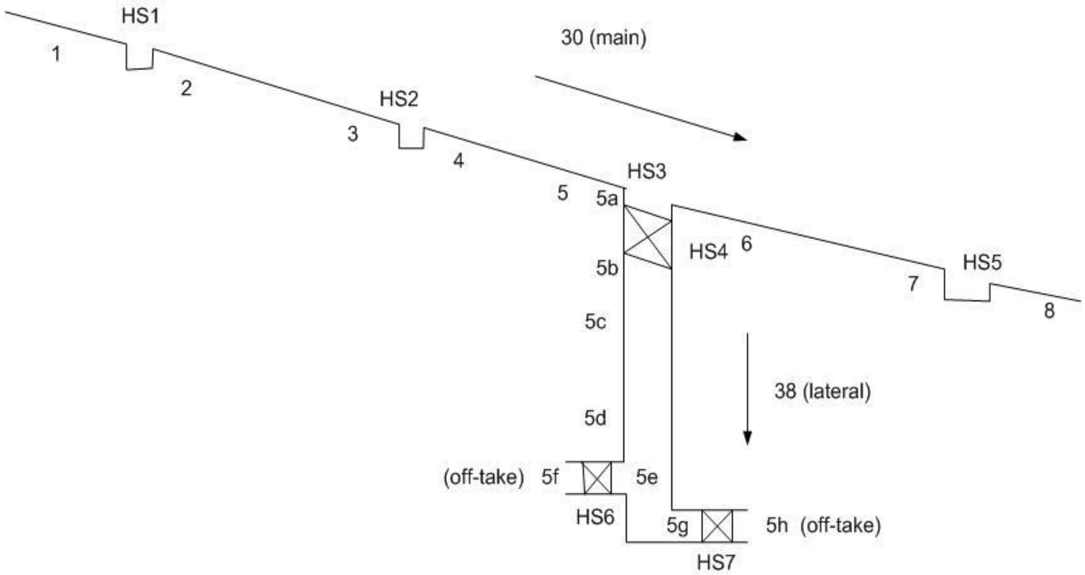

Figure 3: A branching network irrigation scheme.

Table 2. Gate calibration in the branching network irrigation scheme.

Gate	Flow Regime	Gate Status	Gate Flow Equation	Discharge Coefficient
HS3	NA	Closed	NA	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	$Q = C_{so} L h_d \sqrt{2g(y_u - y_d)}$	$C_{so}=0.609677(y_d/G_o)^{-2.2873}$

Table 3. A group of measured data for model computation initialization and verification.

	Data	Usage
Head on HS5	0.73 ft (0.22 m)	Initial condition
Gate Opening at HS4	0.98 ft (0.30 m)	Boundary condition
Discharge over HS5	12.5 cfs (0.35 cms)	Initial condition
Gate Opening at HS6	1.23 ft (0.37 m)	Boundary condition
Discharge through HS6	0.5 cfs (0.01 cms)	Initial condition
Gate Opening at HS7	1.91 ft (0.58 m)	Boundary condition
Discharge through HS7	0.99 cfs (0.03 cms)	Initial condition
Depth upstream of HS7	0.8366 ft (0.255 m)	Initial condition
Depth upstream to point 5	2.19 ft (0.67 m)	Verification
Head on HS1	0.70 ft (0.21 m)	Verification

Figure 4 shows the computed water level profile in the downstream channel of the main canal (6-7). The channel length is about 800 ft (244 m). The computed water level is very close to the normal depth at the distance of 10 ft (3.05 m). So, if the distance is greater than 10 ft (3.05 m), the flow can be considered uniform.

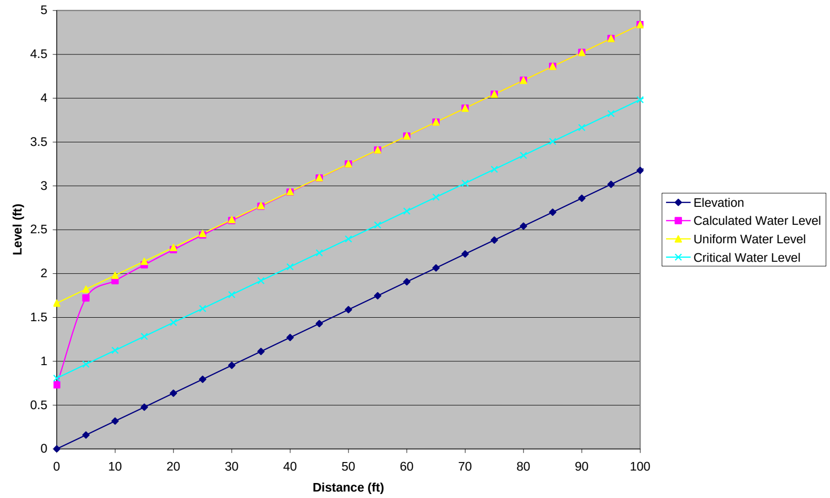

Figure 4: Computed water level profile of the main canal channel 6-7.

Figure 5 shows the computed water level profile in the downstream channel of the lateral (5e-5g). The channel length is also about 50 ft (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.

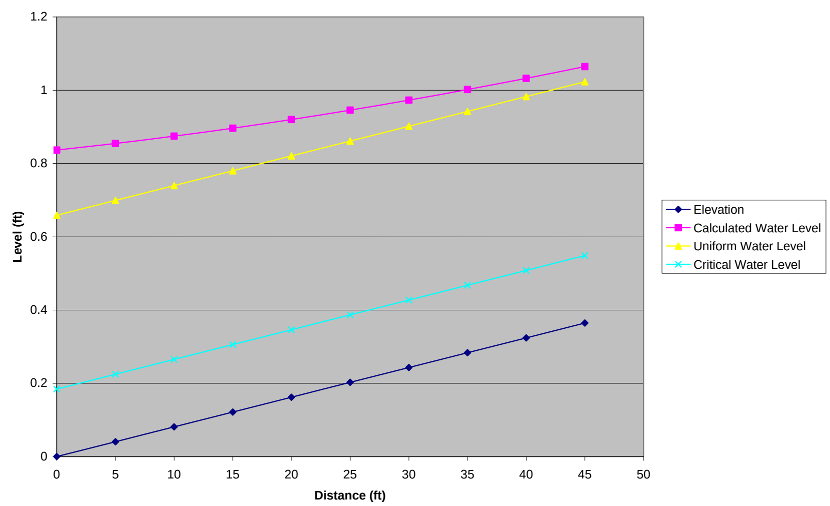

Figure 5: Computed water level profile in the downstream channel of the lateral 5e-5g.

Figure 6 shows the computed water level profile in the upstream channel of the lateral (5b-5e). The channel length is about 400 ft (122 m). The computed water level is very close to the normal depth at the distance of 300 ft (91 m). So, if the distance is greater than 300 ft (91 m), the flow can be considered uniform.

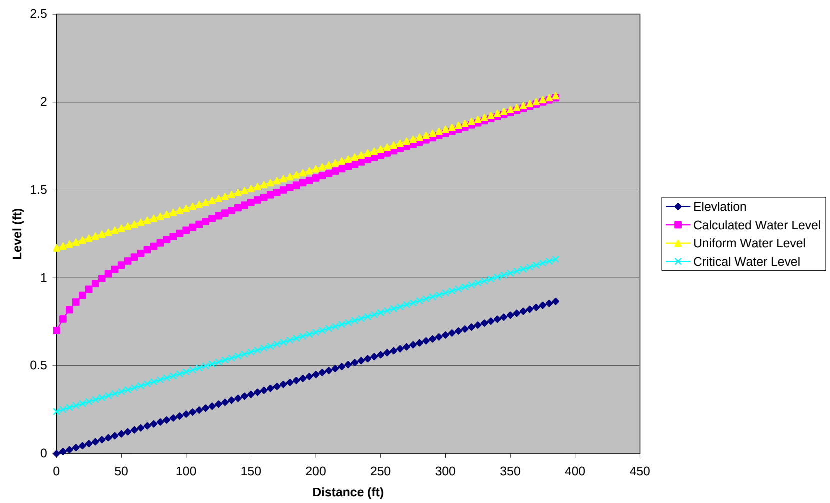

Figure 6: Computed water level profile in the upstream channel of the lateral 5b-5e.

Figure 7 shows the computed water level profile in the intermediate channel of the main (4-5). The channel length is also about 800 ft (244 m). The result indicates that the computed water level is going to but never close to the normal depth over the channel. So, the flow is considered as pure gradually varied. It is noted that a 2.19 ft (0.67 m) water depth could happen about 20 ft (6.1 m) upstream point 5.

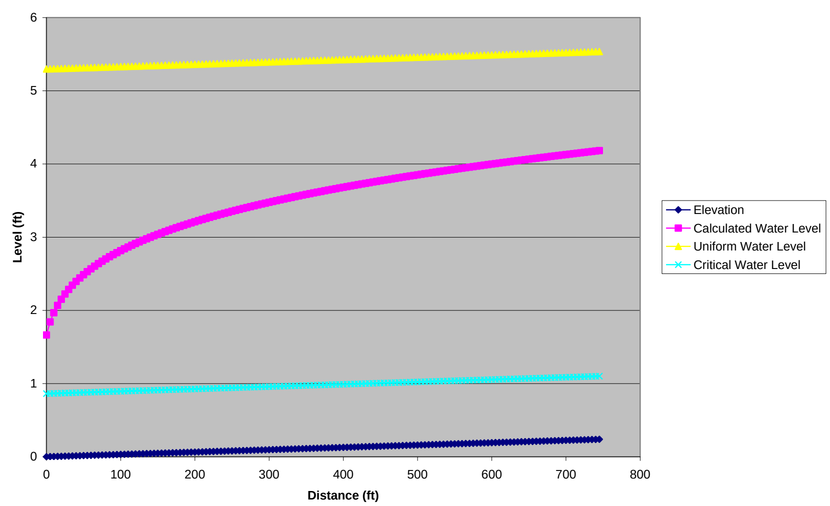

Figure 7: Computed water level profile in the intermediate channel of the main 4-5.

Figure 8 shows the computed water profile in the upstream channel of the main (2-3). The channel length is about 4,000 ft (1,219 m). The water level is close to the normal depth at the distance of 800 ft (244 m) from the downstream end of the channel. So, if the distance is greater than 800 ft (244 m), the flow can be considered uniform.

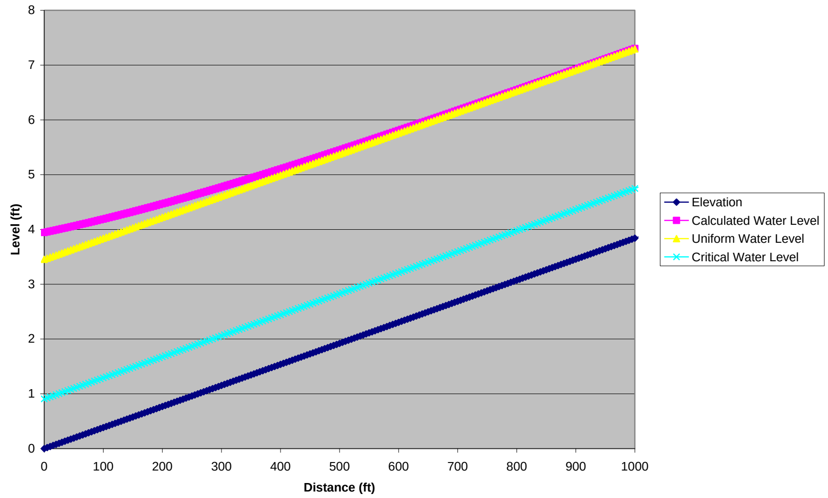

Figure 8: Computed water profile in the upstream channel of the main 2-3.

The computation finally produced 0.65 ft (0.2 m) water head on the sharp-crested weir HS1. All of the computing results are summarized in Table 4 compared with the measured data at check points.

Table 4. Modeling tool computation result summary

Location	**Initial and Computed Discharge (cfs)**	**Computed Water Depth (ft)**	**Measured Water Depth (ft)**	**Comment**
Main downstream	12.5 (0.35 cms)	NA	0.73 (0.22 m)	Initial condition
Channel 6-7	12.5 (0.35 cms)	Figure 4	NA	
Channel 5e-5g	0.99 (0.03 cms)	Figure 5	0.8366 (0.255 m)	Initial condition
Channel 5b-5e	0.5 (0.01 cms)	Figure 6	NA	Initial condition
Channel 4-5	13.97 (0.4 cms)	2.19 (0.67 m) (Figure 7)	2.19 (0.67 m)	Based on the computation, the 2.19 ft (0.67 m) depth happened at about 20 ft (6.1 m) upstream point 5
Channel 2-3	13.99 (0.4 cms)	Figure 8	NA	
Main upstream Head on HS1	13.99 (0.4 cms)	0.65 (0.2 m)	0.70 (0.21 m)	Only differ at 0.05 ft (0.01 m)

8. CONCLUSIONS

This study has developed a modeling tool for modeling the water flow profile in irrigation distribution networks. This modeling tool makes the modeling process simple, fast, reliable and accurate with much less cost compared to commercially available models and software packages. The results 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.

9. ACKNOWLEDGEMENTS

The authors wish to thank Mr. Milton Henry, Director of Engineering & Technical Services, Jamaican National Irrigation Commission for his work in field survey and flow measurement. This work was partially funded with support from the Rio Grande Basin Initiative supported by USDA-Cooperative State Research, Education, and Extension Service and administered by the Texas Water Resources Institute.

10. REFERENCES

V. T. Chow. 1959. *Open-Channel Hydraulics.* McGraw-Hill Book Company, New York.

Y. Huang. 2004. Water measurement for gate calibration. Biological and Agricultural Department, Texas A&M University.

R. W. Jeppson. 1977. *Analysis of Flow in Pipe Network.* Ann Arbor Science, Ann Arbor, Michigan.

B. E. Larock., R. W. Jeppson, and G. Z. Watters. 2000. *Hydraulics of Pipeline Systems.* CRC Press, Boca Raton, Florida.

T. M. Walski, D. V. Chase, D. A. Savic, W. Grayman, S. Beckwith, and E. Koelle. 2003. *Advanced Water Distribution Modeling and Management.* Haestad Press. Waterbury, CT.

G. Z. Watters. 1984. *Analysis and Control of Unsteady Flow in Pipelines.* ButterWorths Publishers, Boston, Massachusetts.

G. Z. Watters. 1979. *Modern Analysis and Control of Unsteady Flow in Pipelines.* Ann Arbor Science, Ann Arbor, Michigan.

R. A. Wurbs. 1994. Computer models for water resources planning and management. Water Resources Support Center, Institute for Water Resources, US Army Corps of Engineers.