CEMAGREF-IIMI International Workshop on
The Application of Mathematical Modelling for the Improvement of Irrigation Canal Operation
October 26-30 1992, Montpellier, FRANCE

THE UTILITY OF A SIMULATION MODEL FOR PAKISTAN CANAL SYSTEMS:
Application Examples from North West Frontier Province and Punjab

Zaigham Habib[1], Bagh Ali Shahid[2] and Muhammad Nawaz Bhutta[3]

ABSTRACT

A mathematical hydraulic model has been applied to simulate the hydraulic and operational conditions of two irrigation systems in Pakistan which have different design and operational objectives and management rules. The Chashma Right Bank Canal (CRBC) system in North West Frontier Province (NWFP), still under construction, is designed for crop based demand irrigation operations where large quantities of water will be supplied per unit service area to meet peak water requirements. The Lower Chenab Canal (LCC) system in Punjab is nearly 100 years old and was designed to supply a maximum service area with a modest quantity of water for low intensity of irrigated agriculture. The utility of a hydraulic model "SIC" is evaluated for two canals, one from each system and of very different scales. The CRBC main canal has a maximum design discharge of 137 m^3/s; by contrast, a small secondary channel in the LCC, Lagar Distributary, has a 1.08 m^3/s design discharge. Preliminary results confirm that in its present state, the SIC model is more suitable for operational decision support at the main system level and infrastructural environment of the CRBC Canal system than it is at the secondary canal level in the LCC system where physical control opportunities are markedly less.

INTRODUCTION

The Indus Basin Irrigation System of Pakistan is composed of 43 main canals which serve large distributary canals and those feed minor canals. The distributary and minor canals serve groups of watercourses. A continuous flow of water is delivered to the farmers through watercourses on a rotational basis. As the surface supplies are generally less than crop water requirements, the irrigation systems are designed for an equitable distribution of water. Recently two canals, Chashma Right Bank Canal and Lower Swat Canal are designed on a crop based irrigation approach.

The existing equity of water distribution conditions along the secondary canals of Gugera Branch of Lower Chenab Canal (LCC) in Punjab, Pakistan were evaluated by Bhutta (1990) and Bhutta and Vander Velde (1992). They found that existing water distribution along irrigation canals is not equitable. They concluded that practical procedures for the improvement of the system operation needed to be identified and tested.

---

1. Systems Analyst, International Irrigation Management Institute, 1-A/B Danepur Road, GOR-1 Pakistan.

2. Principal Irrigation Engineer, International Irrigation Management Institute, 1-A/B Danepur Road, GOR-1 Pakistan.

3. Director (Drainage), International Waterlogging and Salinity Research Institute, 128-N, Model Town Ext. Lahore, Pakistan.

The operation of irrigation systems to achieve the design and operational objectives has become a challenge for the irrigation engineers in the country. This paper describes an application of a mathematical hydraulic Model to Lagar Distributary and Chashma Right Bank Canal (CRBC) to identify improved operational and maintenance procedures.

A hydraulic model can be physically or mathematically based. A mathematical model is an idealization of a real canal system. It can be used to study the management scenarios of an existing or a proposed system. A mathematical model is based entirely on equations and numbers. The flow simulation can be done well by a mathematical model. Once the hydraulic model has been calibrated, different operational scenarios can be tested. An application of a computer based mathematical model "Simulation of Irrigation Canal (SIC)" is tested in this study.

SIC is a flow simulation model, capable of simulating a given canal for which the topography, parameters of the structures and boundary conditions are known. The detail of the model is given in CEMAGREF 1990. Steady state as well as unsteady state conditions can be simulated with this model. The utility of the model was tested for application to a main canal and a secondary canal in Pakistan.

OBJECTIVES

Irrigation systems in Pakistan are so big that if any management intervention for the improvement of their performance is tested in real field situations it will be too costly and also time consuming. These management interventions can be tested with the models at comparatively less cost and time. Most of the mathematical hydraulic models being used in the world are at the experiment and development stage. Many of those need further development to suit practical purposes. The utility of SIC model is tested for selected Pakistan irrigation canals. The main objectives of the study are:

i) to calibrate and verify SIC Model for selected Pakistan canals to properly simulate the hydraulic and operational conditions of the irrigation system;

ii) to identify modifications needed in the model for application to Pakistan Canals, if any;

iii) to apply the model for testing the impact of existing operational procedures on canal performance; and

iv) to apply the model for studying the effect of maintenance options on water distribution.

APPLICATION OF MODEL TO LAGAR DISTRIBUTARY

The model is calibrated for Lagar Distributary before its application to different operational and maintenance management scenarios. The field data required for this purpose are collected by IIMI Pakistan.

Lagar Distributary Description

The Lagar Distributary was selected for the study because it is a typical secondary canal of Pakistan. Most of the input data required for the model had been collected in earlier studies. Lagar Distributary offtakes from the right bank of the Upper Gugera Branch Canal of LCC at RD 118000 ('reduced distance' expressed in feet from the head of the canal). It is in the administrative division of Sheikhupura District, Punjab. The canal administration unit is Farooqabad Sub Division of Upper Gugera Division. Its total length is 18950 m. It has a design full supply discharge of 1.08 m^3/s. It delivers water to 24 outlets/turnouts and Jhinda Minor which serves a set of further 6 outlets. A schematic diagram of the distributary is given in Figure 1. These outlets serve a Culturable Commanded Area (CCA) of 6619 ha. The water allowance is 0.16 l/s per ha. The average authorized outlet discharge is 32 l/s.

Design bed width of the distributary varies from 3.96 m. in the head reach to 1.22 m. at the tail. The design full supply depth (FSD) ranges from 0.64 to 0.34 m. Design free board varies from 0.46 to 0.31 m. The tail one third of the distributary is brick lined. The discharge at the head of the distributary is regulated with the help of "*karries*" (wooden stop logs) at the intake structure. To adjust the discharge is a difficult job and therefore the regulation at this point is not active. All the outlets are fixed structures. Their discharge vary with the variation in the water level in the distributary canal.

Model Calibration and Verification for Lagar Distributary

Lagar Distributary was described in model files such that its physical features were represented properly. The following features characterize the Lagar Distributary in the model files:

- at the head of the distributary, variations in discharge over time are imposed;
- fixed structure outlets with head discharge relationships are defined as gated structures with fixed opening;
- the downstream boundary conditions are defined as head-discharge tables at the end of the distributary; and
- three outlets at the tail (21TL, 22TF and 23TR) are defined as end of the canal in the model.

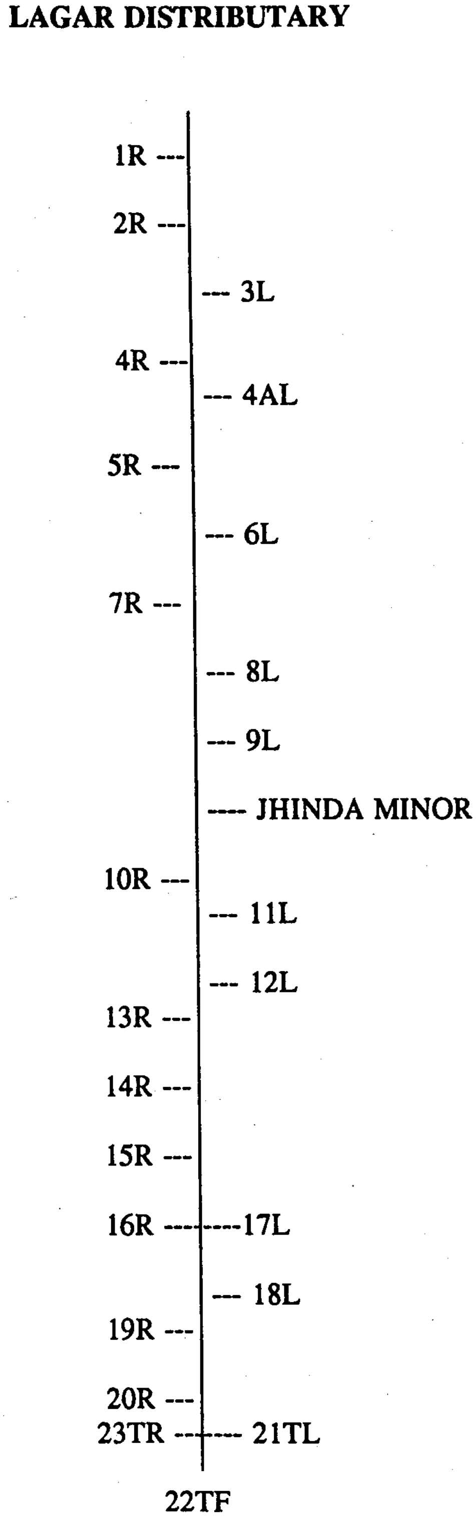
A field survey was conducted to collect the topographic data of the channel. The head-discharge relationships of the distributary and its offtakes were developed based on extensive field measurements of discharge and water levels. The coefficients of the head discharge relationships are entered as coefficients of the gated structures as input to the model. The seepage losses were computed from the differences between the measured discharge at the head of the distributary and the sum of measured discharges of outlets. The value of seepage rate is determined as 5.6 l/s/km for this distributary. The same value is used as input for the model. A Manning's roughness coefficient (n) of 0.025 is used for the unlined sections of the Lagar Distributary and for lined sections n is taken as 0.020.

The water level and the discharge of the distributary and its offtakes were measured in the field under steady state conditions for the calibration of the model. The SIC model is calibrated for Lagar Distributary using the unsteady state option of the model but the discharge at the head is kept constant for this simulation. This is done because the present version of the model can not do loop calculations under steady state conditions. In Lagar Distributary, like other Indus Basin distributaries, the offtakes are ungated. It is not necessary for gate openings to be computed, but more important would be the option to compute discharge into ungated offtakes in the steady flow module. There should be an option in the model for non-targeted discharges for steady state conditions.

The model generated outputs, water elevations and discharges, are compared with the measured values for the same head discharge of the distributary. The measured and predicted values of outlets discharges are compared in Figure 2. The predicted discharges are close to these measured for most

Figure 1

Schematic Diagram of Lagar Distributary



of the outlets. The difference between the measured and predicted values of the outlets discharges varied from 0.000 to 0.014 m³/s (0% to 25% of outlet discharge). Figure 3 shows the comparison between the measured and predicted values of water surface elevations along Lagar Distributary. The predicted elevations are quite close to measured elevation for the major length of canal. The difference between the measured and predicted elevations varied from 0 to 31 cm. The design upstream head of the outlets ranges from 21 cm to 64 cm. The difference in measured and predicted upstream head is more than 100% for a few outlets.

Figure 2 Comparison of Measured and Predicted Discharge of Outlets of Lagar Distributary

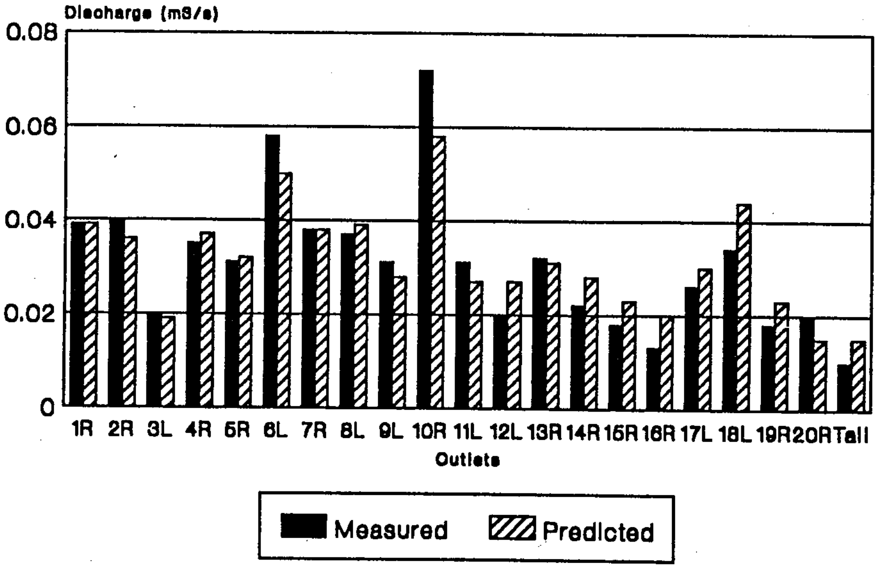
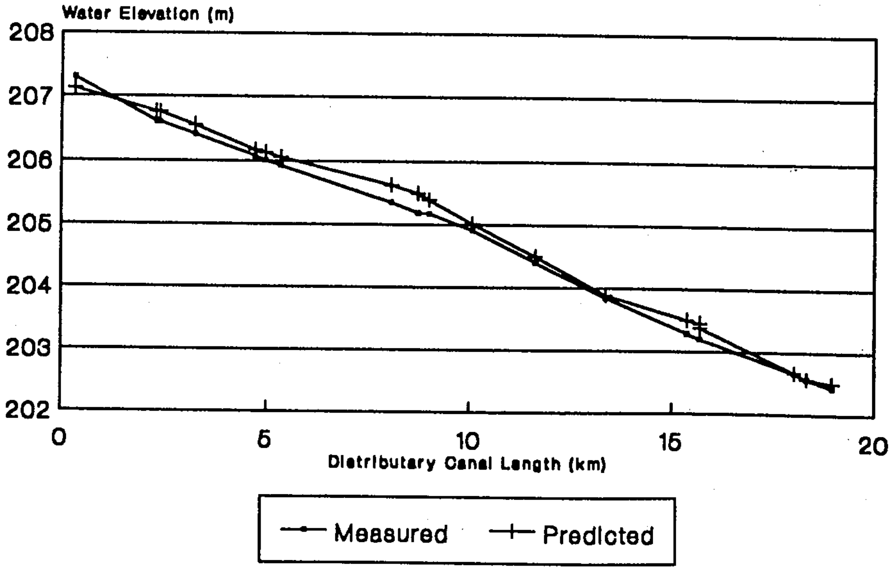


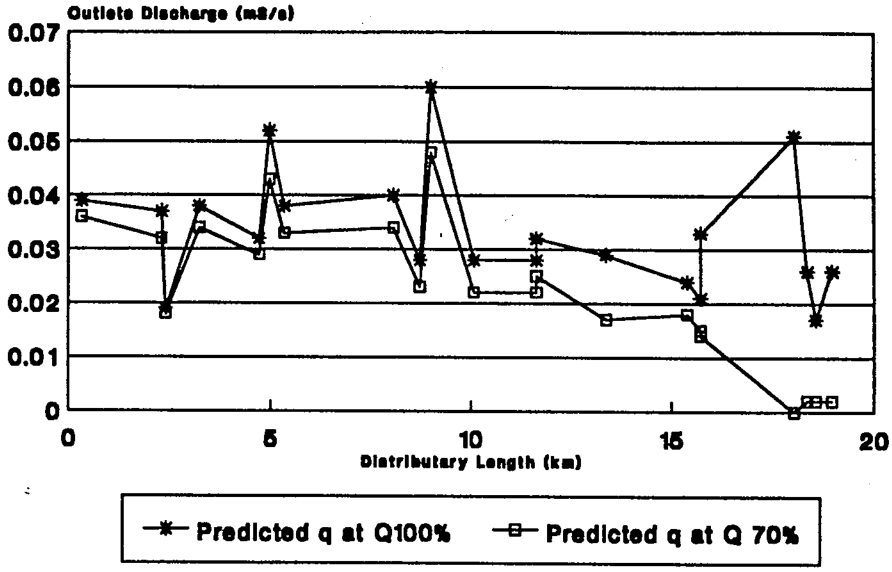
Figure 3 Comparison of Measured and Predicted Water Elevations along Lagar Distributary



Testing of Operational Parameters for Lagar Distributary

The range of discharge at the head of a distributary still accepted by the ID for "normal" distributary operations is between 100% and 70% of the design full supply discharges. The model is applied to predict the outlet discharges at the upper and lower end of the presently acceptable discharge range under the existing physical conditions of the distributary. This is done because the objective of operation for the distributary is equitable distribution of water. The predicted discharges for both discharge levels are shown in Figure 4. The figure reveals that the reduction in the discharges for tail outlets is remarkably more than the outlets at the head when the discharge at the head of the distributary is reduced from 100% to 70%. It is already understood that tail outlets are getting less than their share of water at 100% of distributary discharge. This indicates that the reduction in discharge from 100% to 70% at the head of a distributary, adversely effects the equity conditions along the distributary. Such findings suggest that a thorough review of, and possible change in, the long-standing acceptable range for normal distributary operations by the ID is needed. This is one example of operational scenario being tested by the model but more options could be tested such as rotational irrigation supplies between and along the distributaries and impact of additional/temporary outlets on distributary performance etc.

Figure 4 Impact of Dist'y Discharge Reduction on Water Distribution Along Lagar Distributary



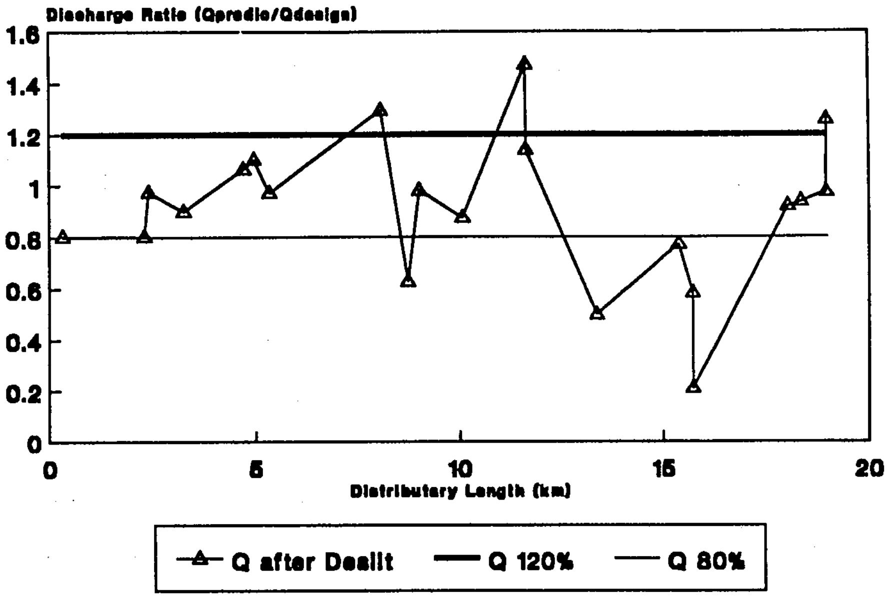
Testing of Desiltation as Management Option

The existing physical conditions of the Lagar Distributary are such that it has been silted up from head to tail. Therefore water levels in the head reach are relatively higher than the design. As the outlets are fixed structures and the size of opening can not be changed, consequently the outlets in the head reach draw more water than their share because of higher upstream heads. As a result the tail outlets of the distributary face a shortage of water supplies.

ID desilted Lagar Distributary from head to tail during the annual closure of 1992. The model topographic files were prepared for the post desiltation condition of the distributary. The model was run to predict outlets discharges and water surface elevations for the post desiltation condition. The discharge at the head of the distributary was kept to design value. The predicted discharges of outlets are shown in Figure 5. The figure also shows an acceptable range of the outlets discharges from

120% to 80% of their design. This variation of discharges from 120% to 80% is acceptable for field conditions. Skogerboe et al. 1987 also mentions about the difference in discharges under field conditions. Moreover, if the farmers are guarantied for 80% of their design supplies they may manage their agriculture satisfactorily. In the figure the predicted discharges of outlets are compared with the acceptable range.

Figure 5 Impact of Desiltation on
Outlet Discharges of Lagar Distributary



Although the discharge of many of the outlets fall in the acceptable range, but the discharge of outlets: 9L, 14R, 16R and 17R are still lower than the acceptable range. The main reasons are:

- The measured elevation of the crest of outlets 9L, 14R and 17L are lower than the bed of the distributary. This situation can not be defined in the existing version of the SIC model. Therefore a false height of the crest is entered in the input file of the model which is higher than the measured. The factious raising of crest elevations result as a reduction of outlets discharges.

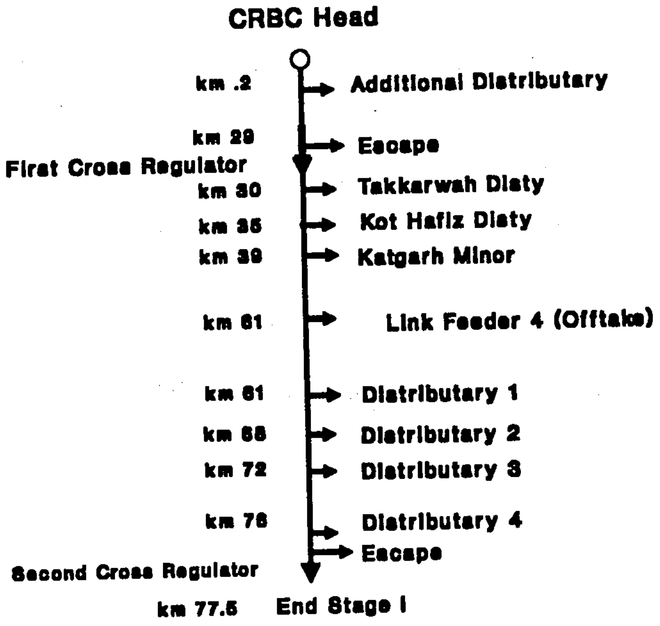
- The flow conditions of outlets 14R, 15R, 16R and 17L are submerged when the water level in the distributary is lower than a certain level in this reach. In case of submerged structures the discharge coefficient is a function of upstream and downstream elevations. It is difficult to incorporate such situation in the model when the downstream water level does not remain constant. Therefore the discharge of such structures are computed by taking the discharge coefficient values less than free flow coefficients to match the general conditions of outlets discharges. When the water level in the distributary raises from that level the flow conditions are changed to free flow. This situation happens occasionally. However, when the distributary is desilted the water level in the head reach is lowered and resulted an increase of level in this reach. This level may be the range of free flow conditions of the outlets. Therefore the predicted discharges may be lower than the design.

Figure 5 shows that there is a marked improvement in the discharges of tail outlets. The predicted discharges of the tail outlets are shown higher than measured. One reason for this could be that the

ACOP (WAPDA) started monitoring of canal in 1987 after the inception of stage I, when the water appeared in borrow pits on both sides of the canal.

The distributaries 1, 2, 3 and 4 at the end of the stage I have been selected as sample for IIMI studies (Figure 6). Distributary I has a pipe offtake while the three others have barrel offtakes and special downstream structures. Wooden-stop logs ("*Karries*") are placed horizontally in these structures and water flows over them. Therefore the height of crest can be changed easily whenever required. The observed change within a month for the sample distributaries was 0.3 to 1 m. These structures are used to feed the tertiary offtakes located upstream of this point that serve relatively high located command areas.

Figure 6 Schematic Diagram of CRBC Canal



Calibration of SIC Model for CRBC

The calibration of the model entails an accurate assessment of the parameters crucial in determining the flow dynamics. The accuracy of the prediction depends upon a large body of accurate field measurements and a good estimation of the boundary conditions. For CRBC a good amount of the reliable field data were available due to the continuous monitoring of the last four years. Additional information has been collected by the field team of IIMI.

The following primary data collected by IIMI and WAPDA were used for the calibration:

* canal profiles of four consecutive years taken by soni sounder.

* design and existing dimensions and sill levels of all regulating devices.

- stage-discharge measurements for ten locations of the main canal .discharge measurements of all offtakes taken once in a month since 1990.

- seepage losses.

- daily water levels and gate openings for four sample distributaries and cross regulators. IIMI has installed the automatic data loggers for these locations.

- Water levels in the main canal.

- data sets taken by IIMI staff including discharge measurements, gate openings, up and down water levels, flow conditions and the downstream operations (height of *"Karries"*).

For the steady state, the model is calibrated with imposed discharges of the offtakes and the actual gate openings of the cross-regulators. The Manning coefficient for different reaches varies from 0.018 to 0.035. Seepage and evaporation losses vary from 10 l/s/km to 65 l/s/km. Seepage losses increase rapidly with increase in discharge so a good estimate of these losses at all flow rates is required for the calibration.

The calibration of the first regulator was carried out to determine the discharge coefficient 'K' for the equation:

$$Q = K\ 2g^{0.5}\ L(Ha^{1.50} - (Ha\text{-}W)^{1.5})$$

where

K = coefficient;
Ha= upstream water head (m);
L = length of gate (m);
W = opening of gate (m).

The measured stage-discharge sets indicated a small shift in the computed value of 'K' with a change of gate openings (WAPDA staff also mentioned a discrepancy between measured and calculated flow passing through this regulator). The analysis of data and model output show that the actual gate opening is slightly higher than what has been recorded. A correction factor of 0.06 m is introduced for all gates of the regulator to incorporate the effects of the design imperfections and/or side leakage, a technique referred by Skogerboe et. al (1987).

For the second regulator, which is a complex structure, the same equation is found satisfactory with discharge coefficient of 0.60. The syphon itself has not been modelled at this stage of the study.

Figure 7 shows the predicted and measured water surface elevations for 41, 67.5 and 107 m^3/s head releases. These are respectively the lowest, the average and the highest crop water demands for the whole command area of CRBC. The model is first calibrated for 67.5 m^3/s and then tested for the other two sets of head-discharge measurements. The match is best for the 67.5 m^3/s. The difference between predicted and measured elevations is maximum at 107 m^3/s (5 cm to 47 cm) specially at the upstream of the 1st regulator. One reason for the higher prediction is the error factor of the gates; another reason would be a higher seepage from the banks than estimated. The seepage behavior of the main canal needs to be studied in detail.

Even after the calibration, a correct and timely assessment of the changes in the boundary conditions is important which requires a continuous and close coordination with the field staff responsible for the operation and management. The calibration for the specific sets of boundary conditions is imp for CRBC due to the transient hydraulic conditions of the system.

The guidelines provided in the SIC manual to define the structures like syphon are not sufficient.

Application of the Model and Discussion of Results

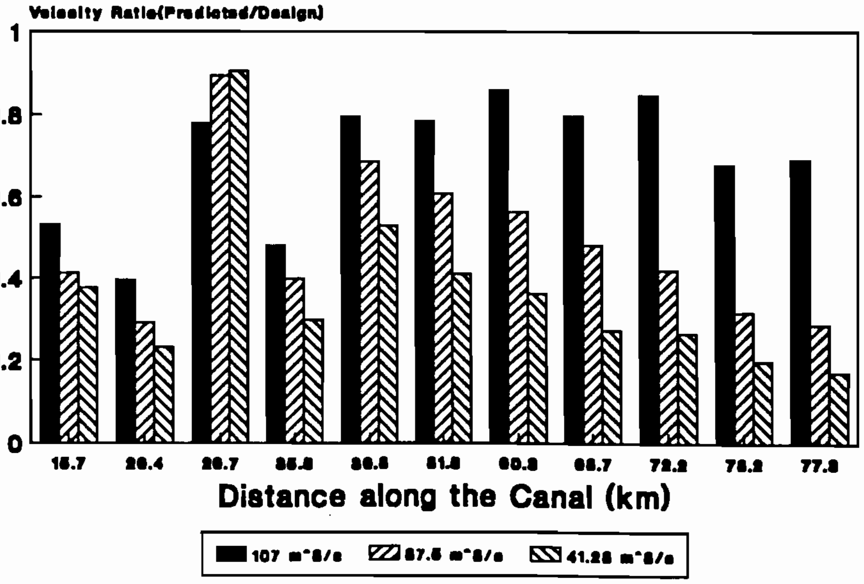
Hydraulic Performance of CRBC stage I

A proper hydraulic functioning of stage I is critical for the performance of the whole CRBC system. The managers still have to decide about the feasible range of the flow rates and the velocities. The model is used to study the hydraulic performance and water distribution capacity of the system for different head releases, from 40 to 113 m^3/s. The results are compared with the field data and the predicted trends are verified wherever possible.

At 40 m^3/s discharge at the head of two distributaries of stage I cannot take their proportionate share if the share of the stage II & III is passed on to the stage II. At 113 m^3/s the predicted free board is less than 0.6 meters at 32-36 km of canal reach. It has been verified by the field observation. Recommended free board for 138 m^3/s is about 1 m.

The variation of velocity in different reaches provide useful information about the effects of existing practices on the system behavior. The design velocity for the maximum flow of 138 m^3/s is a reference velocity to estimate the range of operational velocities. The ratio of predicted and design velocity for three discharge levels are shown in Figure 8. At 107 m^3/s (80% of design) the highest and lowest velocity ratios at different location are 0.9 and 0.55 m/s respectively. At 67.5 m^3/s (50% of the design) highest and lowest velocity ratios are 0.65 and 0.30 m/s. At 41.23 m^3/s (38% of design) the maximum and minimum velocity ratios are 0.59 and 0.15 m/s. For 107 m^3/s the lowest ratio occurs before the 1st regulator while for 67 & 41 m^3/s the lowest ratio is before the 2nd regulator.

Figure 8 Comparison of the Predicted and Design Velocity along CRBC for different Flows



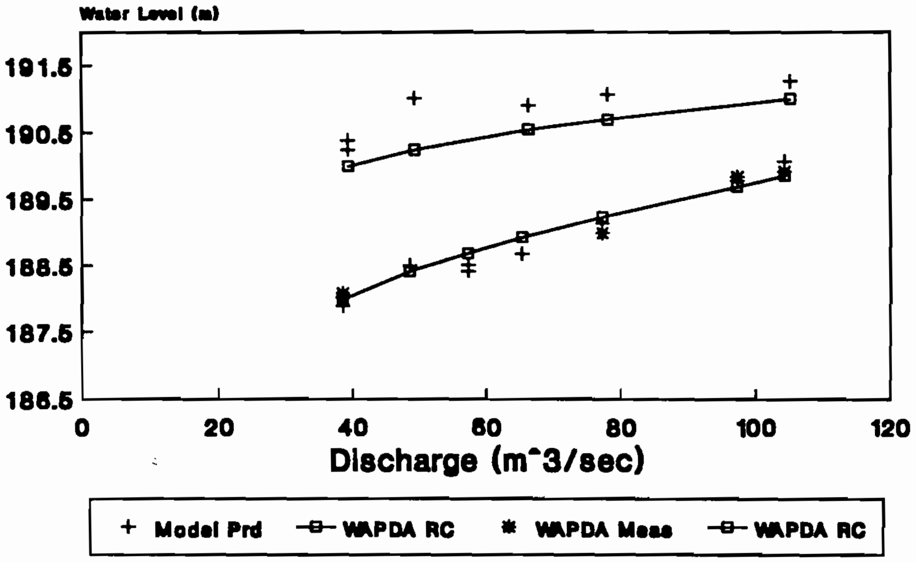
The velocity variations along the canal are related to the back water effects, particularly up-stream of the cross regulator. Maintaining the design water elevation at the tail of stage I results in low velocities and flatter slopes. The water storage in the main canal is a common phenomena for the demand based regulated structures. But a serious implication of this practice for CRBC is the silt accumulation process which has already started in these reaches.

The simulation results are compiled in the form of stage(depth)-discharge sets for the different reaches of the canal (stage-discharge is a commonly used indicator in the Indus Basin to judge the working of the main canals). The discharge range used for the simulation was physically monitored during two years of surveys.

The measured values of losses are used for each set.

Figure 9 shows a comparison between the predicted and measured stage-discharge relations at upstream and downstream of the first cross regulator. The predicted values are mostly in good agreement with the measured data. Upstream of the first regulator a consistent difference of water levels is shown in Figure 9. This shift indicated a missing constant. The correction factor mentioned above has properly accounted for this shift.

Figure 9 Predicted and Measured
Stage-Discharge Up and Down-stream
of First Regulator



These results show that the canal can safely carry the range of discharge to satisfy the crop demands. The managers can use the SIC model to accurately predict the behavior of canal (CRBC) and its components. It can be very helpful to achieve the objectives of the present monitoring irrigation project by simulating a variety of situations which are difficult to acquire in the field and provides a better insight of the situation. The amount of actual field measurements can be considerably reduced while the observation of important field indicators can be ascertained during a well organized field campaign.

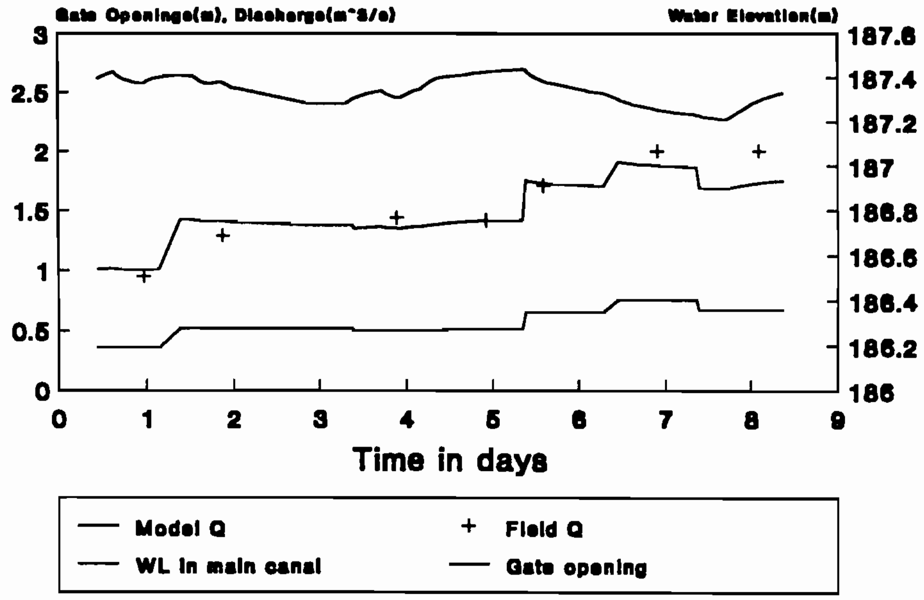
Unsteady Flow Conditions and the water diverted to the Offtakes

The appropriate distribution of water under all flow conditions is one of the major demands of the CRBC project. A good estimation of the volume diverted to the offtakes is necessary for the accurate simulation of the unsteady conditions. The computed distribution pattern of one week for sample distributaries is compared with the direct discharge measurements available almost for every day.

Distributary 1 always maintained the free flow conditions at the intakes while distributary 4 remains submerged. The flow conditions of other two distributaries vary from free to submerged flow depending upon the functioning of the downstream *"Karries"* structure. The height of the *"Karries"* frequently changed to feed the tertiary offtake before this structure. A statistical rating of the data shows that the downstream conditions of these offtakes can not be represented by a rating curve. A 'weir type downstream' option is used, the sill height is taken equivalent to the average height of the *"Karries"*. For distributary 1 an equivalent rectangular structure is defined for the gated pipe.

The predicted discharges are very close to the measured for distributary 4. A variation of 5 to 20% is observed for the other three. Figures 10 and 11 show the gate openings, water elevation in the main canal and the predicted and measured discharges for distributary 3 and 4. The gate opening is the most important parameter and determines the pattern, a small effect of the turbulence in the main canal can also be seen. The downstream weir incorporates a head loss but the downstream levels are not available quantitatively.

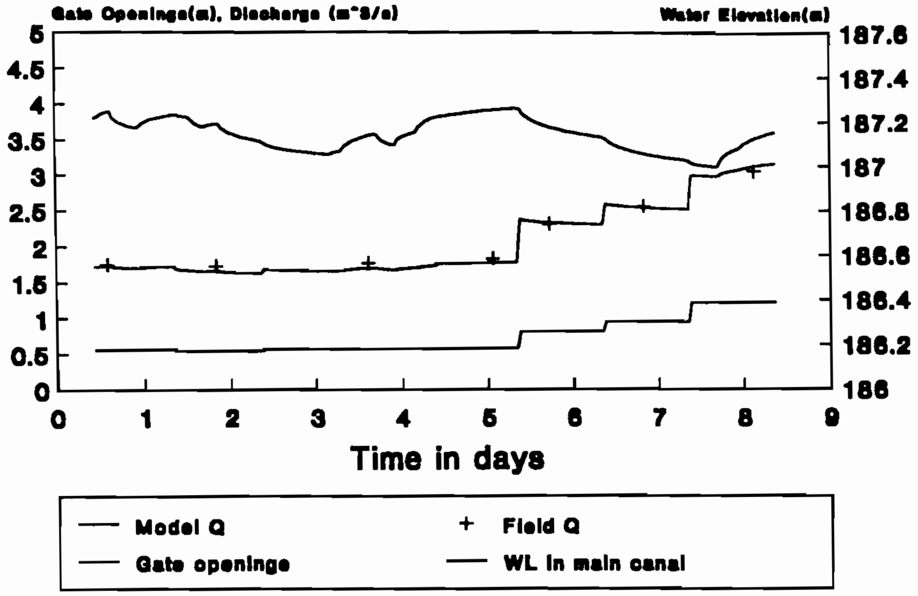
Figure 10 Unsteady Flow Simulation of CRBC at Distributary-3 Regulator



These results indicate that proper downstream conditions are required for a good estimation of the discharge of these offtakes. The results are better for distributary 4 because the downstream water level is almost constant under the current operations.

The accuracy of the water delivered to the offtakes depends upon the exact grasp of the structure behavior and the downstream conditions. The difference between the measured and estimated values is within the acceptable limits but some improvements in the model are required to properly estimate the behavior of the structures.

Figure 11 Unsteady Flow Simulation of CRBC at Distributary-4 Regulator



Hydraulic Evaluation of the options to handle a design constraint

To develop an optimum set of operations for CRBC when the actual demand is only 28 percent of the maximum demand is a big challenge for the operational engineers. The following targets must be achieved:

i) to provide a working head sufficient for all offtakes;
ii) to avoid siltation in the main canal; and
iii) to minimize extra supplies.

A solution for this problem is still to be established and tested in the field. Two potential methods to tackle this problem are probed in this section.

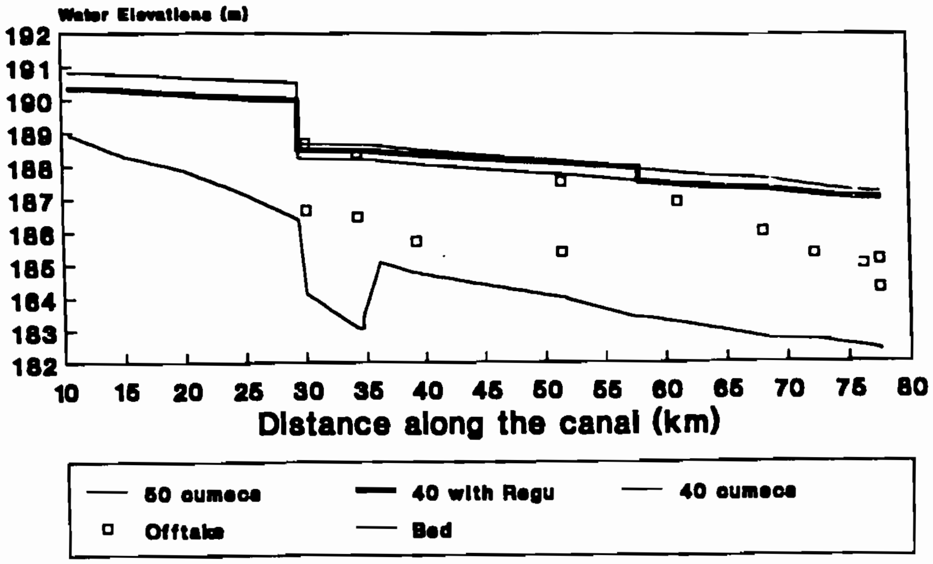
a) Insertion of another cross-regulator to raise the water surface levels in the middle reach (design modification).

b) Use of escape as an active control and to run the canal with 25% extra discharge (management option).

The simulation results indicate that the water surface levels can be achieved in both ways. But the introduction of a cross-regulator (22 kilometer down the first regulator) will considerably decrease the already low velocities between both regulators. While 50 m^3/s inflow provides relatively higher velocities in the main canal for the same water elevations at the tail of stage 1. In Figure 12 water elevations and the sill levels of the outlets are shown. The same gate openings for all structures and the same boundary conditions are used in three sets. The figure shows that at 40 m^3/s it is difficult to provide the required working head to all outlets.

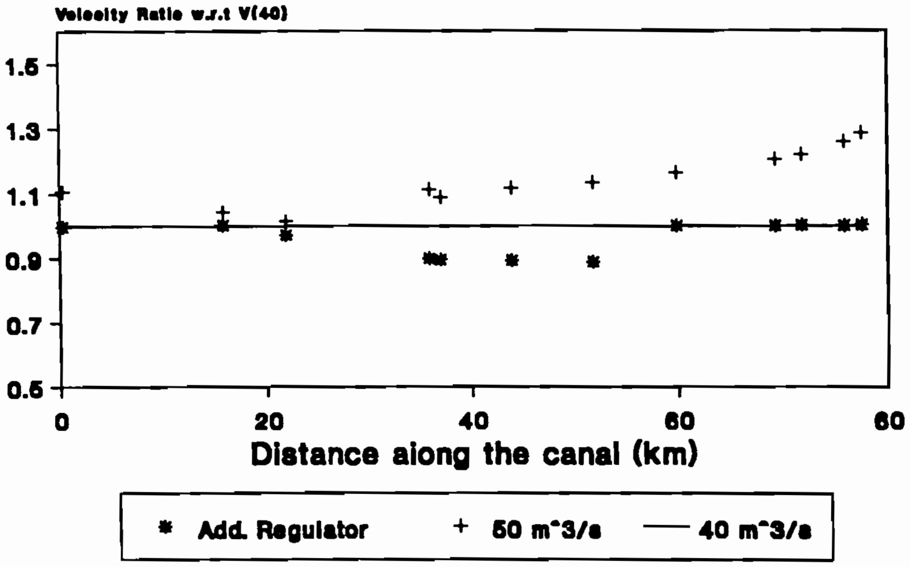
Figure 13 shows the ratios of the computed velocities with respect to the original velocity at 40 m^3/s. The range of the operational velocity of the stage I has not been yet computed and this study is not

Figure 12 Water Elevations with Additional Cross Regulator at 57 Km from Head



exhaustive, but the WSL and velocities shown in these figures clearly indicate that the 25% more discharge is a hydraulically favored option.

Figure 13 Impact of Cross Regulator and 25% extra Discharge on Velocity along CRBC



A good estimation of the parameters is essential before the field test. For the low demand period, the amount of extra discharge and a schedule of the operations to achieve the targeted objectives can be computed with the help of the model.

The management options can provide solutions for the design and operational constraints of CRBC. The suggested solutions of a problem can be evaluated before implementation using a flow simulation model.

This is the time for the operational staff of CRBC to prepare and practice well defined operational rules for the main canal and its offtakes. The existing practice supplies for the stage I is affecting the cropping pattern and practices. The surveys of last two cropping seasons show that the evolving cropping pattern in the area is different from the recommended one.

Before the full operation of the stage II, a reasonable estimation of the crop water requirements of both stages and the implementation of appropriate ten-daily schedule of the operations will save the manager from the uncertainties related to the water management and the cropping practices. The model can help the manager to evaluate and study many alternate options prior to implementation.

Preliminary results have been discussed with WAPDA Professional. The potential of the model to simulate the hydraulic conditions of CRBC was appreciated. A positive response by WAPDA was the installation of gauges at different locations to record the gate opening. A good coordination is expected (and required) in the future.

LIMITATION OF THE SIC MODEL FOR PAKISTAN CANALS

The following limitation were encountered during the application of SIC model to Lagar distributary and CRBC canal:

i) The offtake structures included in the interface do not accurately represent the generic Pakistan conditions. ✓ end 92

ii) If the sill elevation of an offtake is lower than the main canal bed level, the model does not accept the data. ✓ ok

iii) The present version of the model cannot do loop calculations under steady state conditions and therefore outlets discharges can not be computed. ✓ ok

iv) The program cannot accommodate more than one rating curve for an offtake. In case of Lagar Distributary the flow conditions of 7 offtakes change from non-modular to modular depending on the water level in the canal and vice versa. –

v) Input data for sill elevation is rounded to 2 decimal places. It would be better if it can accommodate up to 3 decimals. ? end 92

vi) The present version of the model does not allow for user's requested output in tabular form. Also it does not allow the printing of the graphics with more flexibility for user to select the titles, choose graph type and size etc. –

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

i) The model results indicate that the SIC has a potential as a decision support tool for the management of irrigation canals in Pakistan.

ii) The scenarios tested for Lagar Distributary show that the present version of the SIC Model needs more modifications for the application to secondary canals of Indus Basin. But the model can accurately predict the behavior of the main canal like CRBC, specially for the steady state conditions. It is a useful tool for monitoring the newly commissioned system by simulating a variety of flow conditions which are difficult to attain in the field.

iii) The difficulties faced in defining the offtakes structures of Lagar Distributary necessitate improvement in SIC Model to make it more generic for the secondary canals.

iv) The design of irrigation canals can be tested on the model before the execution of the projects with reasonably accuracy.

v) The model results suggest that a thorough review of, and possible change in, the long-standing acceptable range for normal operations by the ID is needed. The existing design and management of the system demand a decision support tool to be applied at different levels.

Recommendations

Following are the recommendations:

i) The types of outlets used in Pakistan irrigation systems should be included in the interface of SIC model to make it more generic for these systems.

ii) The model may be modified so that it should be able to do loop calculations under steady state conditions.

iii) The model needs modification for user's requested output in tabular form. It may also provide the facility of graphic printing with more flexibility for user to select the titles, choose graph type and size etc.

iv) For further study, it is suggested that the impact of allowing temporary/additional outlets along the Lagar distributary on its performance may also be tested with the model.

v) A close coordination between IIMI and system managers of CRBC is indispensable for productive output of the study.

REFERENCES

Bhutta 1990. *Effect of Varying Discharges on Equity of Water Distribution in Irrigation System,* Ph.D. Dissertation, Centre of Excellence in Water Resources Engineering, University of Engineering and Technology, Lahore, Pakistan, pp 224.

Bhutta M.N. and Vander Velde E.J 1992. *Equity of Water Distribution along secondary canals, Punjab Pakistan.* Forthcoming in Journal of Irrigation and Drainage Systems, the Netherlands.

CEMAGREF 1990. *Simulation of Irrigation Canals,* Theoretical Concepts, France, pp 50.

Skogerboe G.V., Shafique M.S. and Gandarllias C. 1987. *Field Calibration of Irrigation Structures for Discharge Measurements,* International Irrigation Center, Department of Agricultural and Irrigation Engineering, USU Logan, Utah, USA, pp 88.

ACOP, WAPDA, 1987-92 *Annual Monitoring Report of CRBC.*