mathematical models | Simulation models | irrigation canals

Sri Lanka
Kirindi Oya

CHAPTER 9

Application of Mathematical Models for Simulation of Canal Operations at Kirindi Oya, Sri Lanka: Preliminary Results

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INTRODUCTION

A NUMBER OF mathematical models are currently available for studying the hydraulics of rivers and canals. Models have also been developed for the automatic regulation of irrigation systems in real time. Compared to these models, the innovative feature of the mathematical flow simulation model of the Kirindi Oya Right Bank Main Canal (RBMC) is that it provides new opportunities for studying manually operated irrigation systems. In fact, the development and application of the RBMC flow simulation model constitutes the first stage of a research program intended to be of regional scope. The overall objective is to demonstrate the feasibility of using simulation models as decision-support tools for improved performance of manually operated irrigation systems.

Phase I of the Kirindi Oya RBMC simulation model project consisting of software development, field calibration of the model, and production of a comprehensive set of manuals was completed in 1990. IIMI's partners in implementing this project were CEMAGREF,[55] France and the Sri Lanka Irrigation Department (ID). Partial funding support was provided by the French Ministry of Foreign Affairs. This was supplemented by substantial contributions in terms of staff time from IIMI and CEMAGREF.

This paper presents the first results of using the Kirindi Oya RBMC model to formulate appropriate operational responses to some typical canal management problems, identified in consultation with the canal managers themselves. The material presented in the paper draws on unpublished internal reports as well as documents presented to the Study Advisory Committee, the Sri Lanka Consultative Committee, and IIMI technical staff seminars. The results should however be considered indicative at this stage in the sense that they have yet to be implemented and evaluated in the field. Comprehensive field-testing of the simulation model as a decision-support tool in

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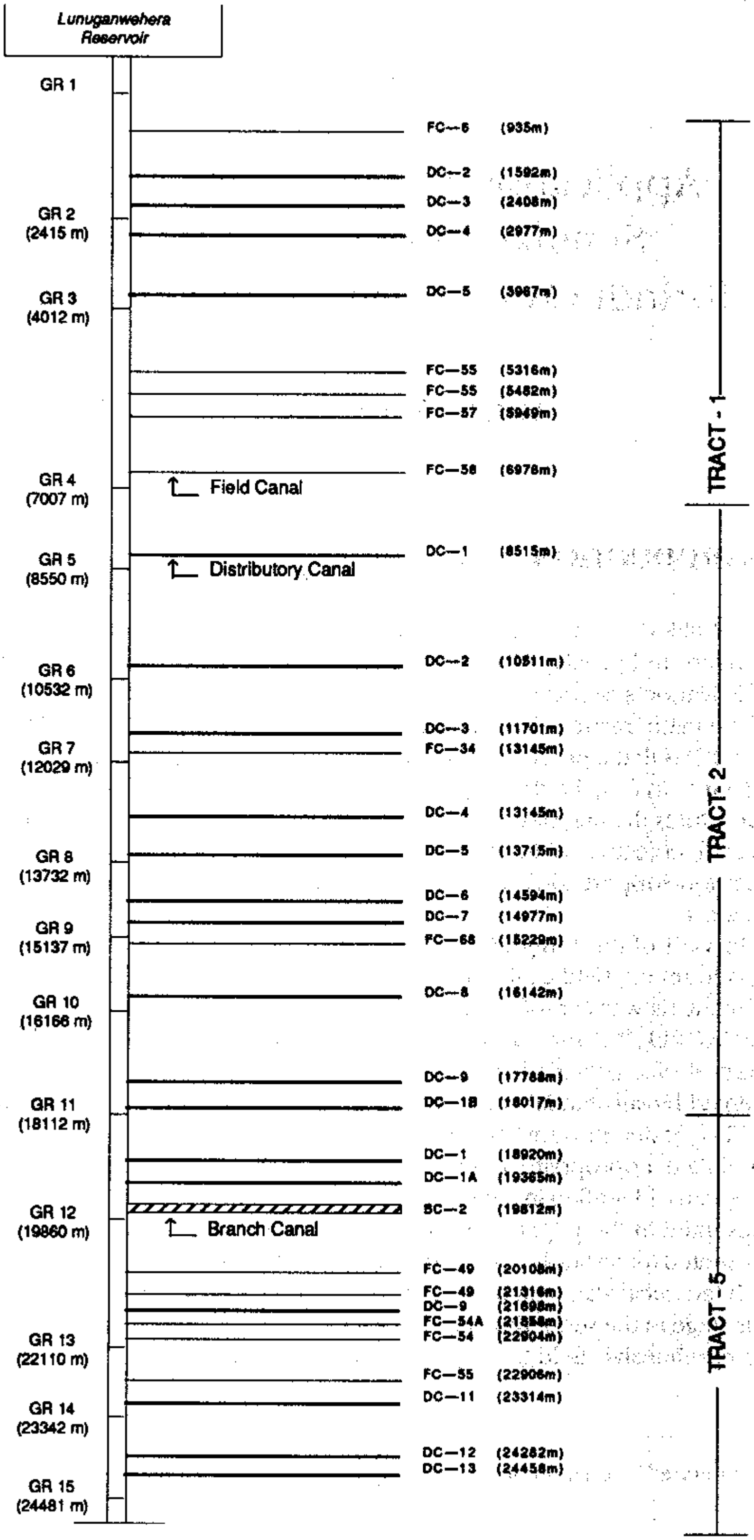
[55] Centre National du Machinisme Agricole, du Génie Rural, des Eaux et des Forêts.

Figure 9.1. Issue tree diagram of the Kirindi Oya Right Bank Main Canal.



Note: GR = Gated Regulator; FC = Field Canal; DC = Dirstributary Canal; BC = Branch Canal.

canal operations will be carried out under Phase II of the project which began in February 1991.

KIRINDI OYA RIGHT BANK MAIN CANAL (RBMC)

The Physical Context

The RBMC was developed as part of the Kirindi Oya Irrigation and Settlement Project in southern Sri Lanka (Figure 10.2 on p.209). The principal objectives of the project are: (a) the augmentation of water supplies to the existing irrigated areas covering around 4,500 hectares (ha), and (b) the settlement of over 8,000 families in the newly developed irrigated command area of around 8,400 ha. The RBMC itself was intended to irrigate about 5,000 ha of land. The development of about 3,650 ha (consisting of Tracts 1, 2, 5, 6, and 7) has been completed to date. But only the 2,743 ha of Tracts 1, 2 and 5 have ever been irrigated; Tracts 6 and 7 received irrigation water for the first time in 1991.

The RBMC takes off from the Lunuganwehera Reservoir (198 million m^3 active storage capacity). The canal is 32 kilometers (km) long and unlined with a design bed slope of 3 in 10,000 (30 cm per km). It was designed to carry a discharge of 13 m^3/s at its head but this value rarely exceeds 7 m^3/s under present operating conditions.

The RBMC model presently covers only the first 25 km of the main canal encompassing Tracts 1, 2 and 5. A total of 33 distributary and field channels take off directly from the main canal in this 25-km length (see issue tree in Figure 9.1). The offtakes are gated and of the undershot type. The downstream water level is controlled by weirs (either sharp-crested or broad-crested) that also serve as flow measuring devices (Figure 9.2). The broad-crested weirs are of fairly recent origin, having been constructed in place of the original sharp-crested weirs that were often found to be functioning under submerged flow conditions.

The RBMC is further characterized by the presence of 14 gated cross-regulators for water level control. The regulators are made up of a set of movable undershot gates (ranging in number from 5 at the head, to 2 at the tail) and a couple of lateral sidewalls (Figure 9.3).

The Operational Context

The managing agency in Kirindi Oya is the Sri Lanka Irrigation Department (ID). A succinct view of the organizational chart of the project is shown in Figure 9.4. Water management activities are under the responsibility of a Senior Irrigation Engineer (SIE). Main canal operations are centered on maintaining full supply depth (FSD), corresponding to the crest level of the sidewalls, at each of the cross regulators. Gate operators attempt to achieve the target levels by adjusting the openings of the regulator

Figure 9.2. Offtake.

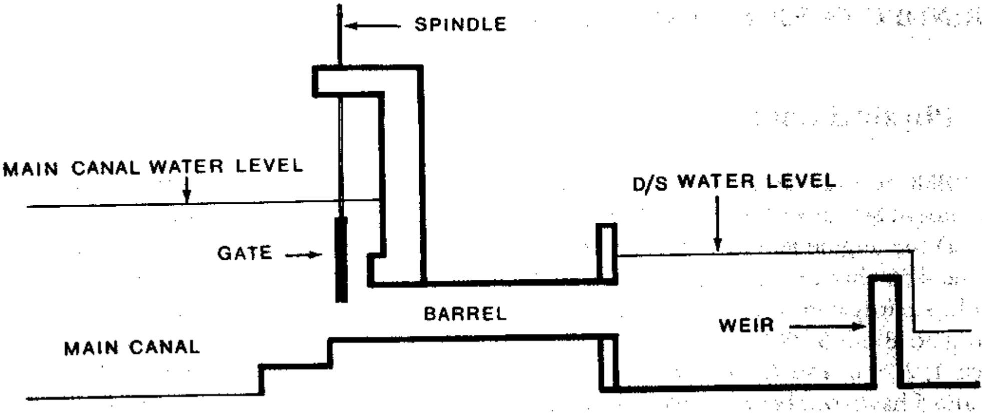


Figure 9.3. Cross-Regulator.

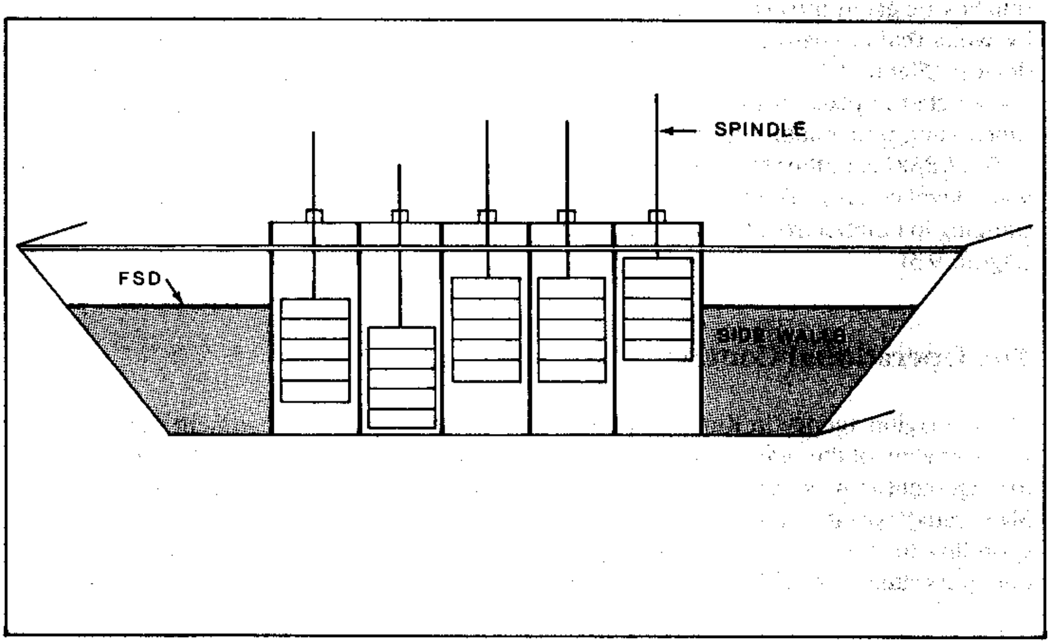
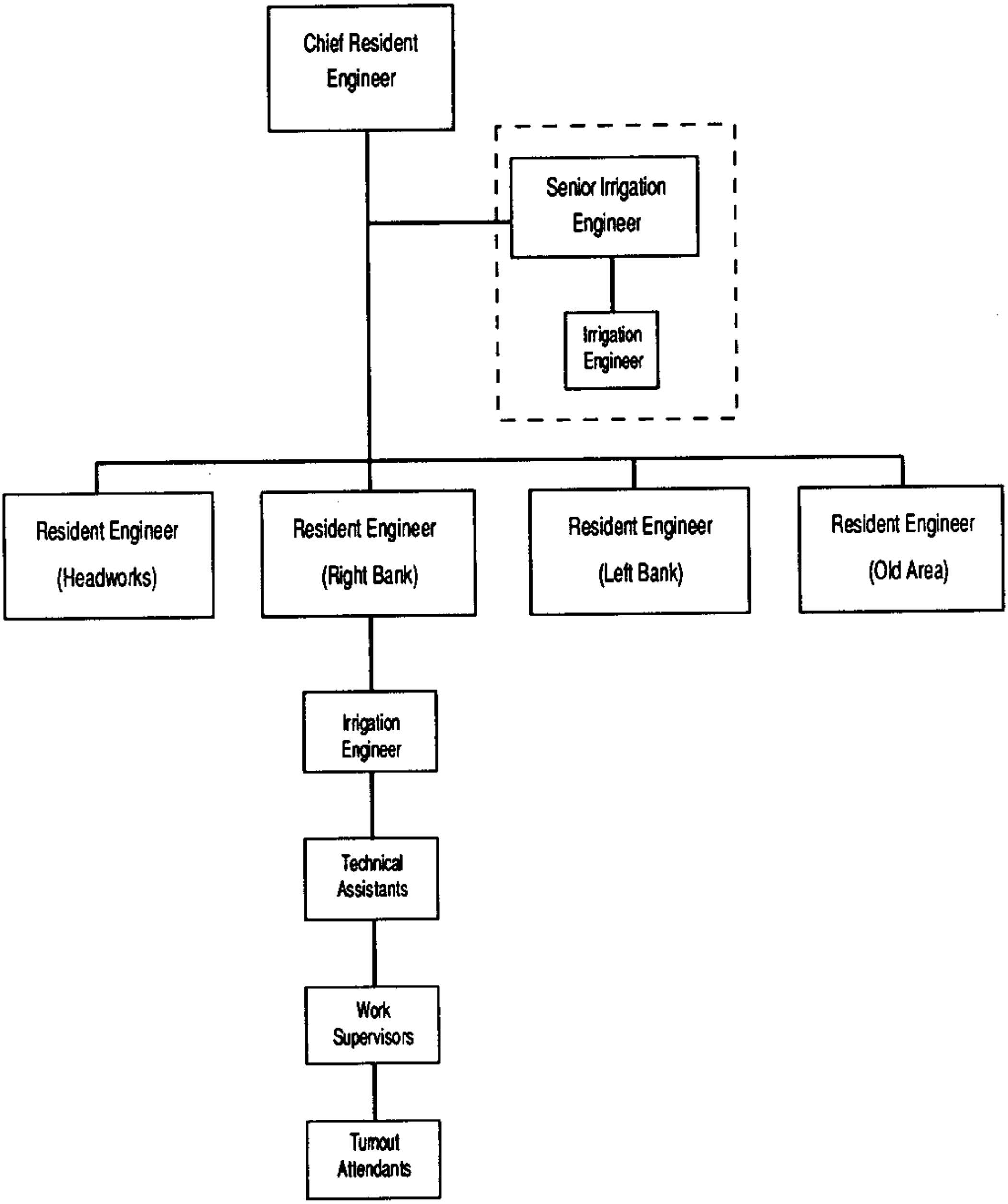


Figure 9.4. Organizational chart of the Kirindi Oya Irrigation and Settlement Project.



gates. A cross-regulator operator usually has the added responsibility of operating a certain number of offtake gates (along the main canal as well as along nearby secondary or tertiary canals). The objective here is to deliver target discharges, assessed by means of the measuring devices located at the heads of these canals.

Even though the operational objectives are fairly well-defined, the operational plans to actually attain these objectives are less-evident, especially at the cross-regulators. Ad hoc, uncoordinated interventions at the regulators give rise to instabilities in canal water levels which in turn could result in inequitable water distribution. This problem assumes even greater significance given the chronic water-short situation of the Kirindi Oya Scheme. Indeed, double-cropping throughout the scheme has not been achieved, and crop failures due to curtailment in irrigation supplies have occurred. Furthermore, the original plan to extend the command area to include Tracts 3 and 4 has been abandoned.

In this context, there is a lot of scope for greater efficiency in water distribution through improved main canal operations.

Constraints to effective main canal operations have been identified in the course of this present research project as well as in other studies conducted in Kirindi Oya, covering aspects such as:

- the impact of design on the management and performance of the main canal (IIMI, 1989),
- irrigation systems management with a view to crop diversification (IIMI, 1990), and
- management decision-making processes (Nijman, 1991).

The lack of a suitable decision-support tool makes it difficult for the SIE in charge of water management to formulate appropriate system-wide operational strategies. This situation is exacerbated by inadequate information transfer procedures. Operators could therefore sometimes unknowingly carry out local operational interventions which are not only unnecessary but which may also create perturbations that give rise to instabilities in canal water levels throughout the system. Malaterre (1989) showed that too frequent gate adjustments at a single regulator to maintain the water level within ±2 cm of FSD could result in oscillations in the water surface profile with amplitudes as high as 19 cm in some locations.

The difficulty in formulating hydraulically optimum strategies for routine operations is compounded when confronted with exceptional operational situations such as, (a) reestablishing steady state in the canal following deviation from the target condition, (b) filling of the canal, and (c) responding to rainfall.

PRINCIPAL FEATURES OF THE RBMC SIMULATION MODEL

The RBMC software is designed to run on an IBM PC-AT or PS/2 compatible microcomputer under the MS-DOS operating environment. A mathematical coprocessor and EGA monitor are required. Graphics output capability on HP compatible plotters is also available.

The model is menu-driven and includes user-friendly interfaces to allow the canal manager as well as the researcher to quickly simulate a variety of hydraulic design and management configurations on the canal. The user also has access to an on-line help procedure while running the model.

The model consists of three software units that can be run either independently or sequentially:

1. A *Topography Unit* for the input and verification of the topographic data of the canal; it creates the topography files needed to run the computational programs of the steady and unsteady flow units.

2. A *Steady Flow Unit* that computes the canal water surface profile for any given combination of offtake discharges and cross-regulator gate openings; the user can also impose a target discharge at each offtake and reference levels at the cross-regulators and the model will compute gate openings along the canal; it also generates the initial hydraulic conditions for running the unsteady flow unit.

3. An *Unsteady Flow Unit* that simulates the transient flow conditions occurring in response to changes in the water supply or demand, or due to operations at the headworks and/or control structures; the user will be able to identify the best way to move from an initial water distribution plan to a new plan; the efficiency of different operational strategies can be evaluated through a set of performance indicators that integrate information on water delivery, either at a single offtake or at all the offtakes. Two types of indicators have been defined: (a) volume indicators, and (b) timeliness indicators.

Units 2 and 3 generate numerical as well as graphical outputs. The graphical outputs in particular can be used as training material for canal managers and irrigation professionals to study the hydraulic behavior of main canals.

MODEL CALIBRATION AND VERIFICATION

The reliability of model results largely depends on the quantity and quality of the input information used to describe the physical and hydraulic features of the real system. Calibrating the model to reflect the field conditions as accurately as possible is an essential prerequisite to carrying out meaningful applications of the model.

The physical information required includes (a) a longitudinal profile of the canal bed, (b) cross sections of the canal at suitable intervals to capture all hydraulically significant features, and (c) the locations and dimensions of all cross-regulators, offtakes and other singularities along the canal. The physical information was gathered in the course of a topographical survey.

The hydraulic information includes (a) roughness coefficients for the different reaches of the canal, (b) head-discharge relationships and discharge coefficients for the offtake and regulator gates, and (c) seepage losses along the canal. The values of these parameters were estimated in the course of a number of field measurement campaigns, the first of which is described in Sally et al. (1989).

Verification (by comparing field observations with model generated outputs of water levels and discharges) and subsequent updating of the hydraulic parameters (and, less frequently, the topographical information) as needed are an integral part of model maintenance. This ensures that the model continues to yield realistic results.

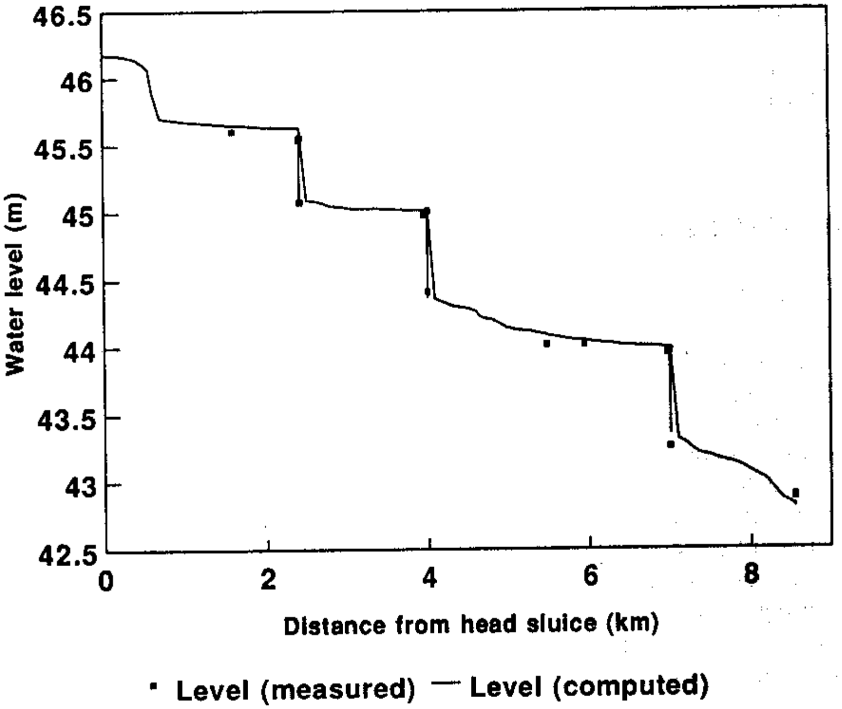
Field observations carried out during the *yala* (or dry) cultivation season in 1990 under both steady and unsteady flow conditions have highlighted the good predictive capability of the model with the present set of hydraulic parameters. (As an example, Figure 9.5 compares model output with field measurements on 29 June 1990.)

The values of these hydraulic parameters are:

Offtake (undershot) gate discharge coefficient = 0.60
Cross-regulator gate discharge coefficient = 0.48 - 0.60
Cross-regulator sidewall discharge coefficient = 0.40
Strickler coefficients = 25-35 (i.e., Mannings n = 0.04-0.029)
Seepage losses estimated at 26 l/s/km (average).

The model also has the flexibility to accommodate different values of roughness coefficients and seepage loss per reach, if necessary (e.g., different values associated with "cut" and "fill" sections of canal).

Figure 9.5. Comparison of model output and field measurements of water level in Tract 1 (29 June 1990).



MODEL APPLICATIONS AND FIRST RESULTS

Field experimentation on live irrigation systems to identify innovative operational practices is seldom feasible — people and crops could be adversely affected. In addition, understanding the hydraulic behavior of the entire canal, especially during unsteady flow conditions, is quite difficult, given the presence of a number of regulating devices and the strong interdependency between the reaches. Most of the time, canal managers are thus unable to formulate optimum operational strategies as they are not in a position to foresee the impact of their decisions.

This dramatic lack of knowledge about system behavior can be reduced by using the simulation model as a decision-support tool. The impact of any planned intervention can be assessed prior to implementation in the field. Inadequacies in the current management response can be detected and quantified. Proposals for improvement can in turn be studied with the support of the model. Those which are compatible with the existing operational environment (i.e., physical infrastructure, staff, communication facilities, etc.) may then be selected for field implementation and evaluation.

In this chapter, the results of applying the Kirindi Oya RBMC model to address some typical operational problems that reflect, to a large extent, the concerns expressed by the ID operations staff are presented. We shall conclude this chapter with some applications of the model for design verification, which were again carried out at the request of the Irrigation Department.

The interaction with ID field staff, the ultimate end users, in planning and carrying out these simulations has been extremely rewarding. The impact of the model from the training perspective should be emphasized. The ID staff truly appreciate the fact that the simulation model helps them to enhance their knowledge of the hydraulic behavior of the canal and now recognize the range of possibilities afforded by the model to identify effective and responsive main canal operational practices.

Determining Offtake and Regulator Gate Settings to achieve Water Distribution Plans

This is a direct but very powerful application of the steady flow unit of the model. It addresses the concern expressed earlier about the lack of explicit operational plans to achieve target water levels and discharges.

Canal operation parameters (water demand at offtakes, main sluice discharge, cross-regulator gate openings and/or target water levels, etc.) are set and the steady state situation is simulated. Device openings, discharges, water surface elevations and volumes are generated. The simulation results maybe visualized in numerical or graphical format. In addition, a summary of key results of operational interest is also produced in the form of an Operational Printout. This printout lists the offtake and regulator gate settings (in terms of gate openings and corresponding spindle heights) required to satisfy the simulated water distribution plan. It is therefore meant to serve as an operational guide for the canal manager.

Evaluating Impact of Interventions at Nearby Gates on Offtake Discharge

This problem will be illustrated using the distributary canal DC5 and cross-regulator GR3 of Tract 1 (see issue tree diagram of figure 9.1 for their relative locations). An example of analysis is given below: (Question/Answer)

Q. What would be the impact on the discharge in DC5 of a sudden gate opening at regulator GR3, located immediately downstream?

A. One of the gates of regulator GR3 is fully opened at 0600. The main canal level immediately begins to fall and the discharge in DC5 shows a corresponding decrease from the initial steady state value of 142 l/s. Three hours later (at 0900) the discharge is zero (Figures 9.6 and 9.7):

This simple simulation is indicative of the type of investigation that can easily be performed using the unsteady flow unit. It demonstrates the consequences of carrying out localized interventions without consideration for possible consequences at neighboring locations.

Figure 9.6. Impact of opening of neighboring cross-regulator on main canal water level near DC5.

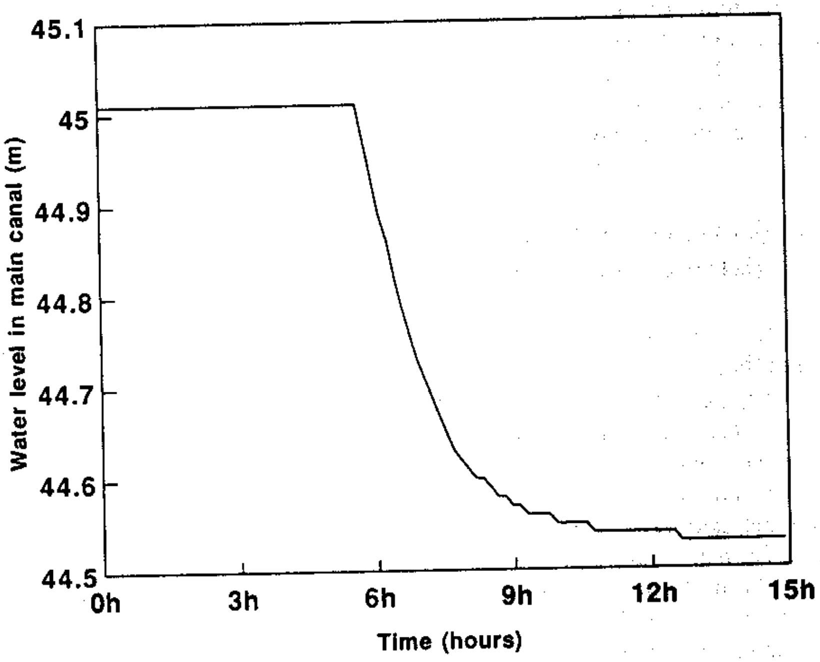
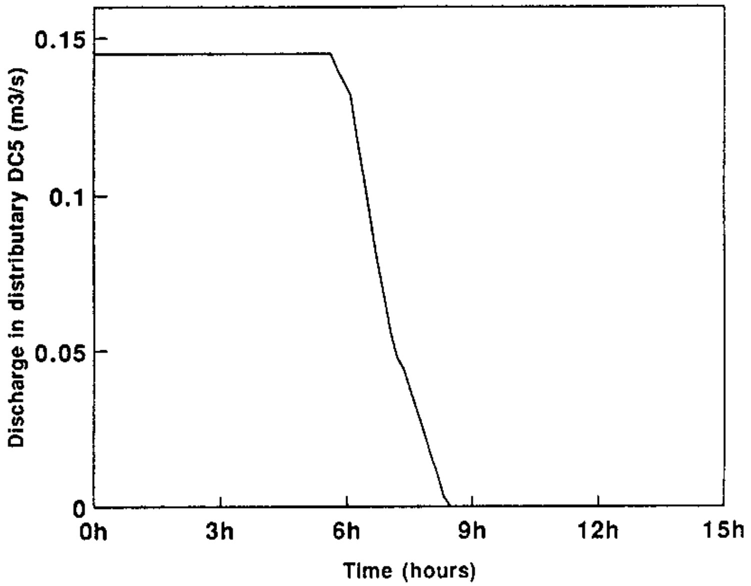


Figure 9.7. Impact of opening of neighboring cross-regulator on discharge in distributary canal DC5.



Determining Canal Filling Procedures

Canal filling is a critical operational problem that has to be faced at the beginning of the cultivation season as well as when water is reissued after a period of main canal closure (e.g., following substantial rainfall). The objective is to minimize the time taken to stabilize the main canal water level (i.e., rapidly achieve a steady state) and deliver the target discharges at all the offtakes, whilst simultaneously ensuring that the canal banks do not overtop.

The problem is to identify, both in terms of magnitude and time, suitable procedures for operating the main sluice, cross-regulators and offtakes to satisfy the above requirements. These procedures can be identified by running the steady and unsteady flow units of the model in sequence.

The field implementation essentially consists of opening the cross-regulator gates to their target values, reach by reach, once FSD is attained. The offtakes can also be opened to their target values at the same time.

The validity of a model-generated canal filling procedure was verified in the field in Tract 1 on 7-8 August 1990 (Rey 1990).

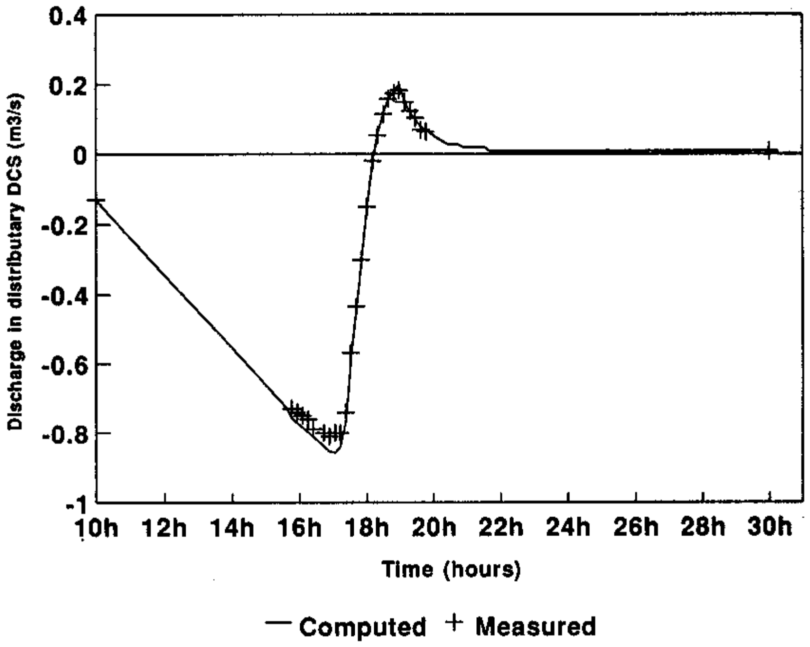
The offtakes and main sluice remained closed on 6 August 1990. At 1000 on 7 August cross-regulators GR2 and GR3 were opened so that the first two reaches of the canal were drained. In the afternoon, the main canal water level was very low. The levels upstream of the two regulators were:

GR2: (FSD-124) cm at 1520 H
GR3: (FSD-85) cm at 1730 H

The ID had scheduled an increase of the main sluice discharge to 2.8 m³/s (100 cusecs) in order to supply water to the older Badagariya system at the tail end of the RBMC. This opportunity was taken to observe the filling of the first 2 reaches of the canal and to field-test the simulation model results (magnitude of gate openings and time of intervention).

The main sluice was opened at 1520H and the regulators GR2 and GR3 closed again at 1600H and 1730H, respectively. The first reach (DAM-GR2) was filled in two and a half hours. The GR2 gates were then opened to the target values (expected to ensure FSD) after voluntarily allowing a short phase of flow over the sidewalls. FSD was achieved after sometime and a steady state was established. A similar operation was performed at GR3 and a steady state corresponding to FSD was achieved at this location as well. The filling phase of reach GR2-GR3 is presented in Fig. 9.8.

Figure 9.8. Filling phase of the reach (GR2-GR3).



The following hypothetical example is now used to demonstrate the influence of the head sluice discharge scenario on the speed of canal filling:

Tracts 1, 2 and 5 are all under irrigation. The planned head sluice release is 6 m³/s. The cross-regulators are progressively adjusted to maintain FSD. It takes over 20 hours for the discharge in the last offtake (DC13 in Tract 5) to attain its target value.

We shall now examine how water can be conveyed to the tail end at a faster rate by temporarily maintaining the head sluice discharge at a higher value than the target requirement for a certain period of time.

Figure 9.9. Water level above FSD at GR15 for different head sluice releases.

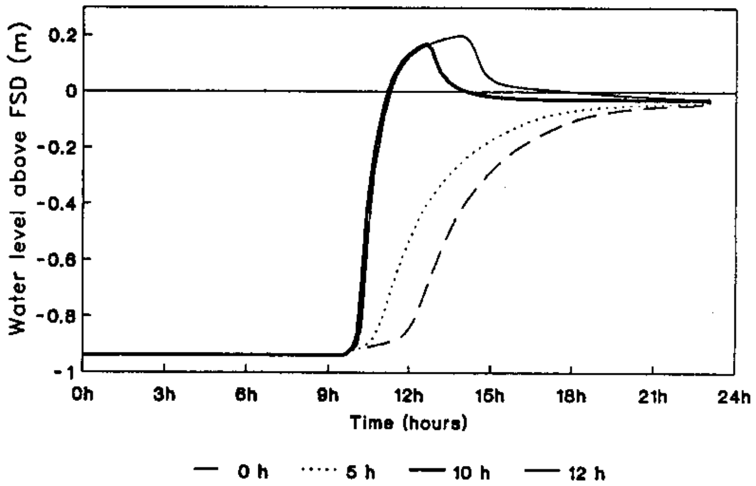
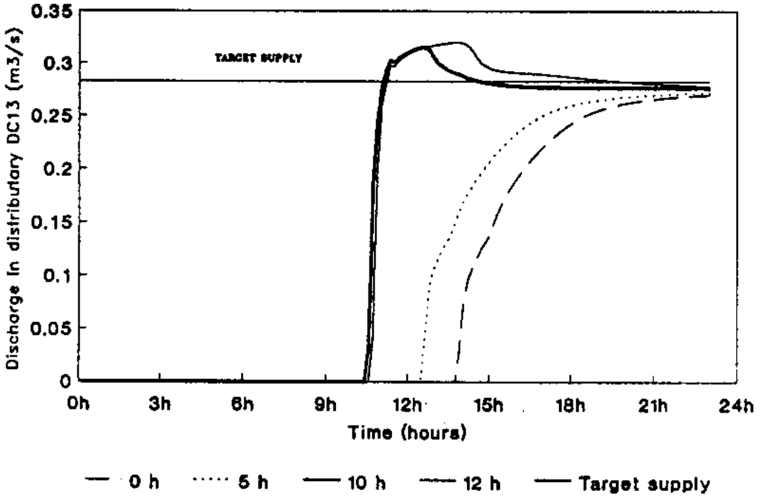


Figure 9.10. Discharge in DC13 (Tract 5) for different head sluice releases.



If the discharge is initially maintained at 8 m^3/s for a period of 10 hours, the tail-end offtake would begin to receive water 11 hours after the main sluice is opened. In addition to the reduction in the time lag for water conveyance, this approach has the added advantage that all major gate operations could be completed within the daylight hours of one day. The temporary oversupply of water can be used to fill the secondary and tertiary canals faster.

Figures 9.9 and 9.10 show the arrival time of the waves when the main sluice discharge is maintained at 8 m^3/s for 0, 5, 10 and 12 hours, expressed in terms of (a) main canal water level at cross-regulator GR15, located immediately downstream of distributary DC13, and (b) discharge in DC13 itself.

Using the Model as a Diagnostic Tool to Evaluate Water Delivery Efficiency at Offtakes

In the case of the offtakes, the operational target of the turnout attendants is to maintain a given head over the control weir located downstream of the offtake. Its value, read off a rating curve, corresponds to the target discharge for that offtake during the period in question. The required value is supposed to be achieved by trial and error adjustments to the offtake gate opening.

A number of studies (e.g., IIMI 1989 and Malaterre 1989) have highlighted the tendency for water deliveries at the offtakes to deviate from the theoretical water supply schedule in favor of an almost "on-demand" situation of satisfying farmers' requests in order that there are "no complaints." This gives rise to a basic dilemma where on the one hand, water resource constraints would seem to dictate that strict enforcement of the water delivery schedule is needed and on the other, this may lead to difficulties in the context where farmers' participation in system management is being actively sought. In hydraulic terms, ad-hoc interventions on the regulating devices could generate local closed loops that disrupt the entire functioning of the canal and which may sometimes even endanger its physical integrity.

Some of the reasons for deviations from the water supply schedule are:

- Incorrect assessment of the discharge by the operator due to inaccurate rating curve at the weir, or submerged flow conditions.
- Unexpected water-level variation in the main canal without corresponding correction of the offtake gate openings.
- Voluntary deviation from the schedule, perhaps due to:

  - a response to farmers' requests
  - a specific temporary need such as speeding up the filling of a secondary canal.

The canal manager presently lacks adequate means for diagnosing and analyzing the cause of observed deviations from the water supply schedule at the offtakes and for identifying appropriate corrective action. Diagnosis is difficult to perform without precise knowledge of the target state of the system in terms of gate openings. Overall, the lack of reference data on the settings of the devices makes it difficult for the manager to detect anomalies in operation. In this section we shall demonstrate the use of the RBMC simulation model for these purposes (see Rey, 1990 for a detailed presentation).

The water delivery hydrograph at any offtake can be generated using field observations and the simulation model as follows:

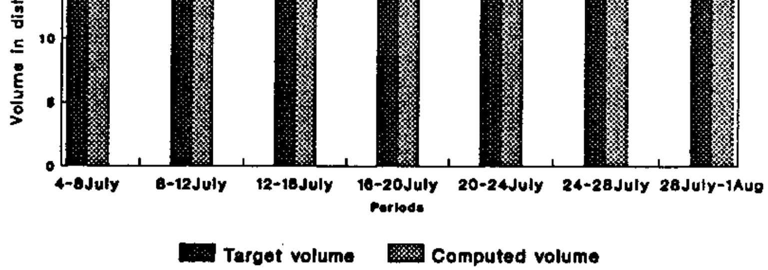
- Recording of offtake and regulator spindle heights (magnitude and time) whenever adjustments are made.
- Regular steady state measurements and verification.
- Unsteady flow simulations.

The simulations, based on observations made in Tract 1 during the month of July 1990, illustrate the use of the model as a diagnostic tool to estimate water delivery efficiency at offtakes. Distributary channels DC3 and DC5 will be used as examples.

Figure 9.11 clearly indicates the degree of oversupply to DC3 with respect to the theoretical target, computed in terms of cumulative volumes over 4-day rotational periods.

Figures 9.12 and 9.13 show some of the results of the routine field monitoring and simulations carried out on the offtakes in Tract 1 during July 1990.

Figure 9.11. Comparison of target and computed volumes over 4-day rotational periods (between 4 July-1 August 1990) in DC3 Tract 1.



The simulation model therefore proves to be useful not only to identify effective operational practices, but also for performance evaluation. Performance indicators such as Delivery Performance Ratio (DPR) and Relative Water Supply (RWS) can be computed, at one or more offtakes and over any period of time.

It was indicated earlier that sometimes there are voluntary deviations from the theoretical delivery target. Such situations can also be analyzed using the model. The discharge computed immediately after an operator's intervention is considered to represent the operators' discharge target, which could be different from the theoretical target.

Figure 9.12. Discharges in DC5, Tract 1 (28 July-1 August 1990).

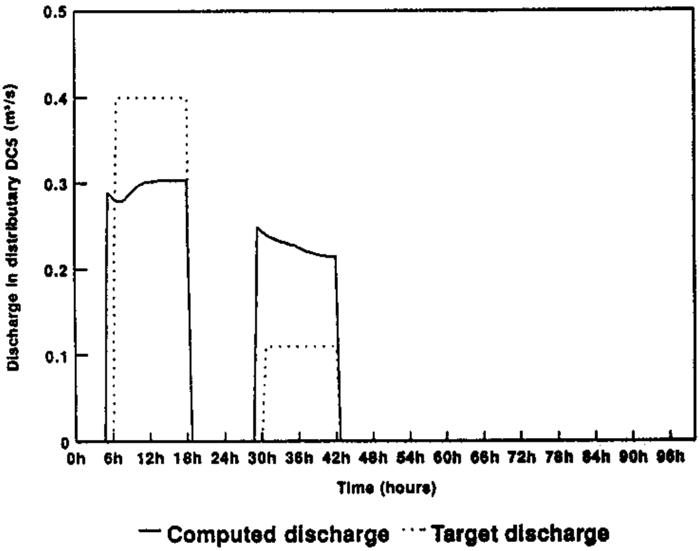
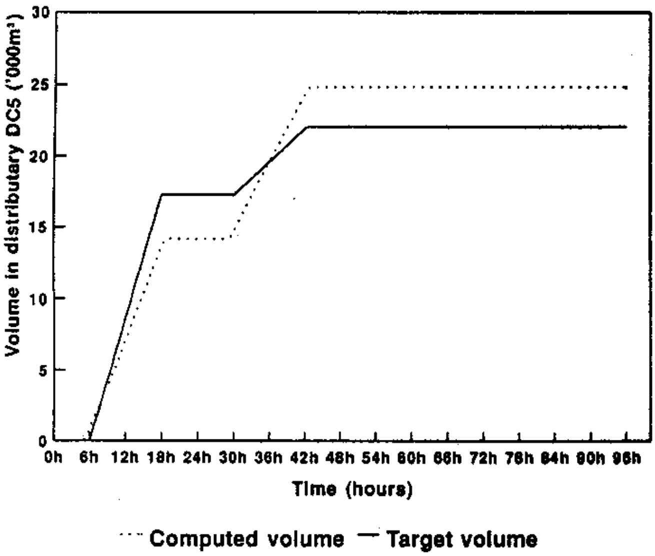
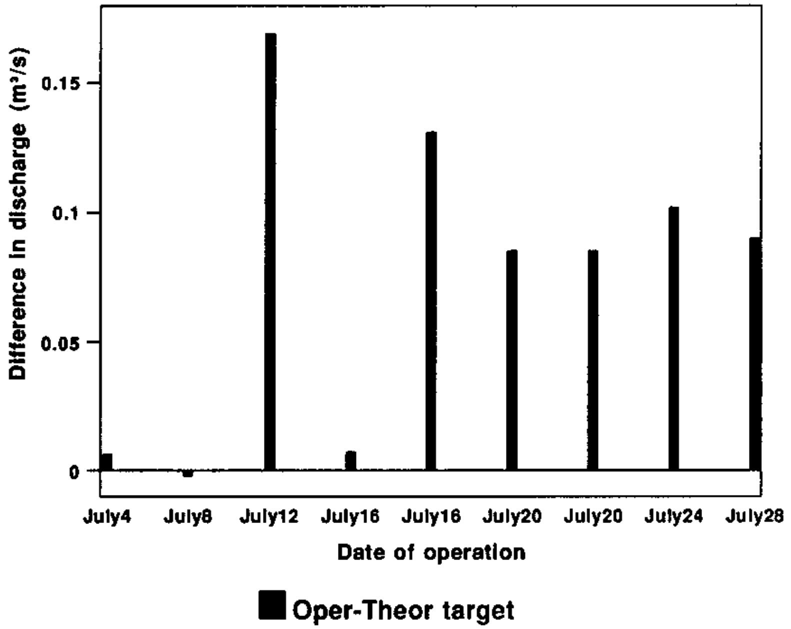


Figure 9.13. Volumes in DC5, Tract 1(28 July-1 August 1990).



Comparing the values of these two targets, as in Figure 9.14 will bring to light the existence of voluntary deviations. Figure 9.14 indicates that the operator's interventions on 4 July, 8 July, and 16 July resulted in his achieving the theoretical target discharge. The oversupply observed on the other days could be because the operator is responding to farmers' requests for water, which are usually more than the scheduled discharge. Investigations to identify the underlying causes could include verification of the weir rating curve, reconsideration of the water requirement computation assumptions, etc.

Figure 9.14. Deviation from theoretical target by gate operators at DC3 Tract 1 (4 July-1 August 1990).



Responding to Rainfall

Given the critically water-short context of the Kirindi Oya Project, ID staff often expresses the view that one way of economizing water use is through better use of rainfall. Rainfall in the project area exhibits considerable spatial variability. A precondition for implementing a strategy to respond to rainfall is information transfer (on the time and quantity of rainfall) from the field to the SIE in the water management unit. Effective communication channels for conveying operational instructions, almost in real-time, from the SIE to the field are also required.

The rainfall response problem can be analyzed in two stages:

1. When rainfall occurs in a particular area, turnout attendants normally close the offtake gates. If there is no rainfall in other areas, what actions should be taken to ensure that these areas continue to receive adequate water supply? The present management response is to adjust some gates, by trial and error.
2. What should be the response when heavy rainfall occurs throughout the project area? Presently, the water issues at the offtakes and main sluice are reduced in proportion to the amount of rainfall received. If the rainfall exceeds 25 mm the main sluice discharge is reduced to about 600 l/s (which corresponds to the empirical minimum discharge required to compensate for seepage losses along the canal), and regulators are closed, the ID objective being to maintain the levels in the main canal.

The problem can thus be summarized as shut-down (partial or complete) of the canals, and start-up (canal filling). As the canal filling phase has already been dealt with, only the shut-down phase is considered here.

If the turnout attendants proceed to close offtake gates in an uncoordinated manner, the accompanying increase in main canal discharge could create dangerous oversupply or overtopping in downstream reaches of the canal. If informed of the perturbation, the SIE would be able to determine appropriate operational responses, using the model, and transmit gate adjustment instructions to the field staff.

The following example will be used to illustrate the use of the model to determine operational procedures (provided of course that effective information transfer takes place). We shall assume that only Tracts 2 and 5 are being irrigated; Tract 1 remains uncultivated.

The gates of Tract 2 are closed at 0800 on a particular day, following heavy rainfall. The resulting increase in main canal level appeared to be within an acceptable range so that cross-regulator adjustments could be avoided in this particular case. The remaining question was then: What actions should be taken to continue to achieve target discharges and volumes in Tract 5 and to divert extra flow to the tail end (downstream storage facility)? Three possible responses were formulated. They were evaluated at offtake BC2 in tract 5 in terms of the deviations of volumes delivered with respect to the targets, and of the management effort for gate operations:

Scenario 1: Using the gate-opening computation mode of the unsteady flow unit of the model, the optimum gate adjustments to continue to maintain constant offtake discharges are obtained. The objective is achieved (within ±10%) with 7 operations at BC2 (from 0900H to 1500H).

Scenario 2: The set of offtakes in the first half of Tract 5 are adjusted in a single operation at 12 noon. The rest of the offtakes are adjusted in one operation as well, at 1300.

Scenario 3: No operations are performed in Tract 5.

Figure 9.15. Scenario 1: Seven gate operations at BC2.

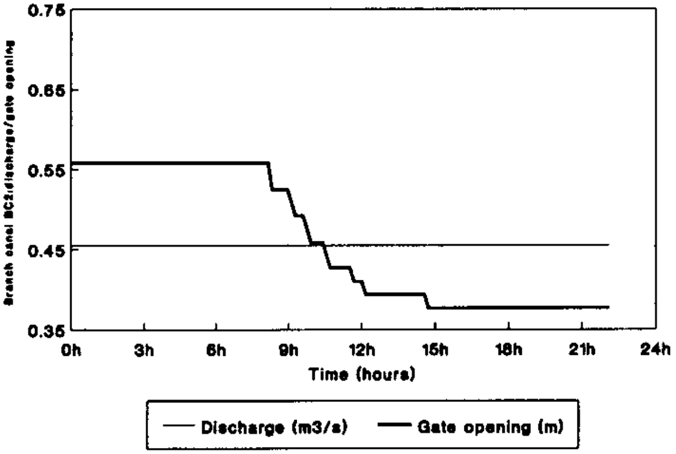


Figure 9.16. Scenario 2: One gate operation at BC2.

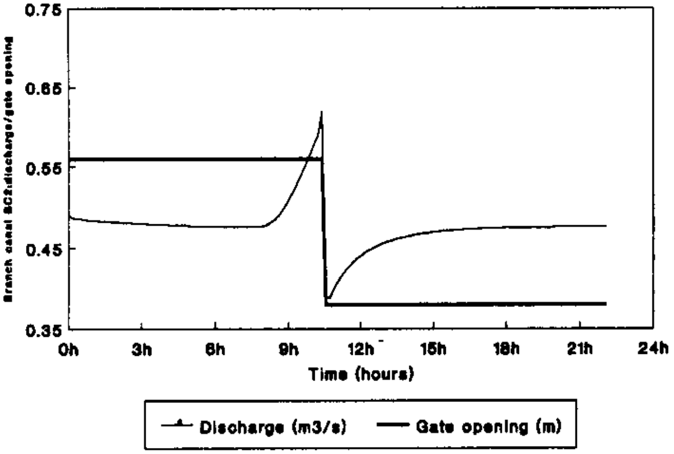
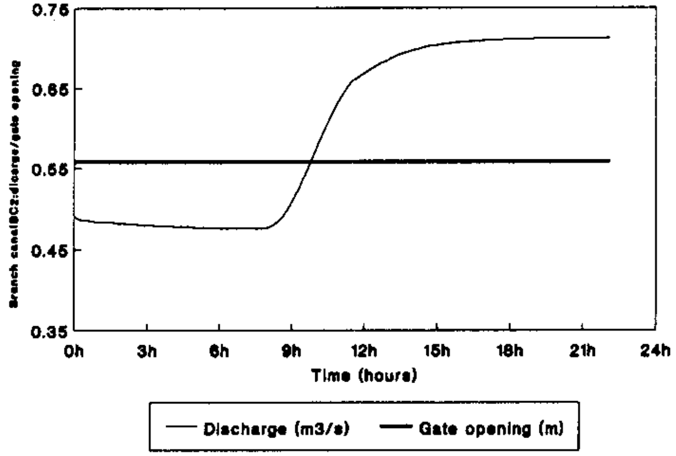


Figure 9.17. Scenario 3: No gate operations at BC2.



The results, shown in Figures 9.15, 9.16 and 9.17 indicate the following:

- If no actions are performed, more than 20,000 m^3 of water in excess of the target volume are diverted into BC2.
- Although operational scenario 1 is theoretically the most effective, it is more demanding in terms of management effort. The approximate response of scenario 2, requiring less effort, is also quite acceptable in terms of volumes. The SIE could choose to implement whichever scenario is compatible with the staffing and communication facilities at his disposal.

Effecting the Transition from One Steady State to Another

In a transient phase of functioning, there is a continuous process of evolution of the hydraulic parameters (e.g., water levels, discharges) till such time as a final steady state compatible with the imposed external conditions (e.g., main sluice discharge, gate openings) is obtained.

The management tasks in this context are then: (a) to achieve the expected final target state, and (b) to minimize the duration of the transient phase. Suitable dynamic strategies that enable the canal manager to fulfill the above tasks can be identified with the help of the unsteady flow unit of the model.

For example, consider the situation observed during one of IIMI's calibration campaigns when the main sluice discharge was 4.798 m^3/s (170 cusecs) and where only Tracts 2 and 5 were being supplied with water. Suppose that it is now required to convey an additional discharge of 1.118 m^3/s (40 cusecs) beyond Tract 5 in order to supply the small storage reservoir at the end of the RBMC.

The obvious operational intervention would be to increase the main sluice discharge by this amount.

If none of the intervening devices are operated in response to this change in main sluice discharge, only 143 l/s will arrive at the end of Tract 5 instead of the desired 1.118 m^3/s (see Table 9.1). This occurs due to the fact that the offtakes at the head are able to take more than their target discharges, thus deriving the most benefit from the increased head in the main canal. On the other hand, if appropriate adjustments were progressively made at the cross-regulators, the desired increase in discharge at the tail end of the main canal can be achieved (see last line of Table 9.1).

If one were to operate the devices to accommodate the increased main sluice discharge so that fluctuations in main canal water levels as well as in offtake discharges are minimized, the question then is to determine the time and amplitude of these operations. This information can be obtained by running the steady and unsteady flow units of the simulation model.

All operations can be completed in 5 hours and hence within a normal working day (Table 9.2).

There was no need (with one exception) to operate the offtake gates because operation of the regulators alone was sufficient to restore the main canal water level

Table 9.1. Discharges when intervening devices are not operated.

	Head	Tail	Increase at head	Increase at tail
Initial discharge (m^3/s)	4.798	0.082	-	-
Discharge after increase at head (no operations of gates) (m^3/s)	5.916	0.225	1.118	0.143
Discharge after increase at head (with operation of regulators only)	5.916	1.189	1.118	1.107

Table 9.2. The times at which the cross-regulators should be operated.

Cross Regulator	Distance from head sluice (m)	Time of operation (after head sluice) hh:mm
GR2	2,415	00:30
GR3	4,012	01:00
GR4	7,007	01:50
GR5	8,550	02:00
GR6	10,532	02:20
GR7	12,029	02:40
GR8	13,732	02:50
GR9	15,137	03:00
GR10	16,166	03:10
GR11	18,112	03:30
GR12	19.860	04:00
GR13	22,110	04:30
GR14	23,342	04:50
GR15	24,481	04:50

to the desired value to maintain discharges through the offtakes. During the period of transition, the value of one of the performance indicators (defined as the ratio between the volume delivered and the target volume) for the offtakes lies between 0.95 and 1.12, indicating that the variation in offtake discharge during this phase is negligible.

Determining Maximum Carrying Capacity of the Main Canal

The maximum carrying capacity problem is a straightforward application of the steady flow model which consists of estimating the maximum possible flow that can be conveyed in the main canal without overtopping the banks anywhere.

Figure 9.18 shows the water surface profiles between the 5- and 10- kilometer points of the canal for different values of main sluice discharges (the cross-regulators being fully opened and the offtakes closed). The overtopping that occurs around the 7-km point for higher values of discharge is clearly visible.

Once the points of the canal banks likely to overtop have been located, the amount of earthwork filling required to prevent this happening can also be estimated. Actual field verification of topography, etc., will be necessary. But the usefulness of the model is that it clearly pinpoints the likely weak sections where further field investigations should be focussed.

The absolute carrying capacity can also be computed for each different canal reach. The results obtained with the cross-regulators fully opened and all the offtakes fully closed are shown in Table 9.3.

The main canal capacity under different operational assumptions can also be studied using the steady flow unit of the RBMC model.

For example, ID water requirement computations assume peak water requirements of 2.72 l/s/ha. Under these conditions, a maximum discharge of 9.25 m^3/s can be released at the main sluice without causing the canal banks to overtop anywhere. But the water available at the tail end is now reduced to 1.46 m^3/s, which is insufficient to meet the peak irrigation requirements of the approximately 900 ha in the newly developed Tracts 6 and 7.

On the other hand, if water requirements were only 2 l/s/ha at the head of each main canal offtake, all of which are assumed to be opened, the maximum possible main sluice discharge is found to be 8.75 m^3/s. After satisfying the discharge requirements at the offtakes, about 2.85 m^3/s is available at the tail end of the canal (GR15 location). This quantity is more than adequate to meet the water requirements of Tracts 6 and 7.

Figure 9.18. Water surface profiles between the 5-km and 10-km points of the main canal for different main sluice releases.

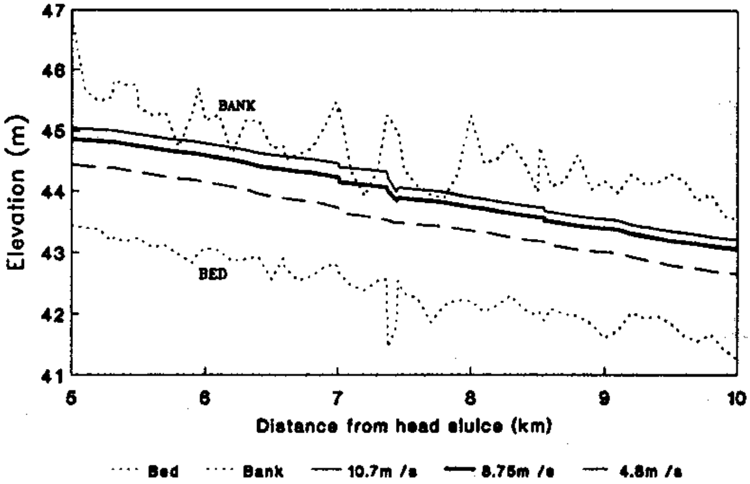


Table 9.3. Results when cross-regulators are fully opened and offtakes fully closed.

Reach	Distance (m)	Maximum discharge (m^3/s)
HS - GR2	0 - 2,415	> 11.6
GR2 - GR3	2,415 - 4,012	10.8
GR3 - GR4	4,012 - 7,007	9.3
GR4 - GR5	7,007 - 8,550	6.9
GR5 - GR6	8,550 - 10,532	> 11.4
GR6 - GR7	10,532 - 12,029	> 11.3
GR7 - GR8	12,029 - 13,732	> 11.3
GR8 - GR9	13,732 - 15,137	5.9
GR9 - GR10	15,137 - 16,166	5.9
GR10 - GR11	16,166 - 18,112	8.1
GR11 - GR12	18,112 - 19,860	6.1
GR12 - GR13	19,860 - 22,110	3.8
GR13 - GR14	22,110 - 23,342	4.9
GR14 - GR15	23,342 - 24,481	6.1

This set of simple simulations brings to light some of the design-management implications of attempting to satisfy the peak water requirements of the entire canal command at the same time (even if water resources in the reservoir permitted such an attempt). Staggered supply of irrigation water seems to be necessary. Different stagger options can also be evaluated using the model.

Impact of Canal Lining and Weed Growth on Carrying Capacity

The operating conditions used are:

1. Tracts 1,2 and 5 under irrigation with head sluice discharge of 6 m^3/s.
2. Cross-regulators set in adjustable mode to maintain FSD immediately upstream (this is the usual operating condition, irrespective of the physical condition of the canal).

Weed growth in the canal (increased roughness) was simulated by decreasing the Strickler Coefficient values by 10 (subject to a minimum value of 25) with respect to their calibrated values.

An increase in the Strickler Coefficient to 50 at every section was used to simulate a lined canal (ID estimates Manning's n of 0.018 for cement mortar lining). However, the canal cross sections were not altered in any way; in actual practice, a lined canal would have a uniform cross section. Seepage losses were also not altered, though this too would be reduced in the case of a lined canal. The results are nevertheless indicative of what would take place if the canal was lined.

Figure 9.19 shows the variation of water surface elevation between the 5-km and 10-km points of the canal under the same set of hydraulic conditions for 3 cases: 1) the canal in its present state, 2) a weed infested canal, and 3) a 'lined' canal.

The offtake discharges are the same for the 3 cases. Actual regulator gate openings maybe different but the water level in the main canal is maintained at FSD, wherever possible.

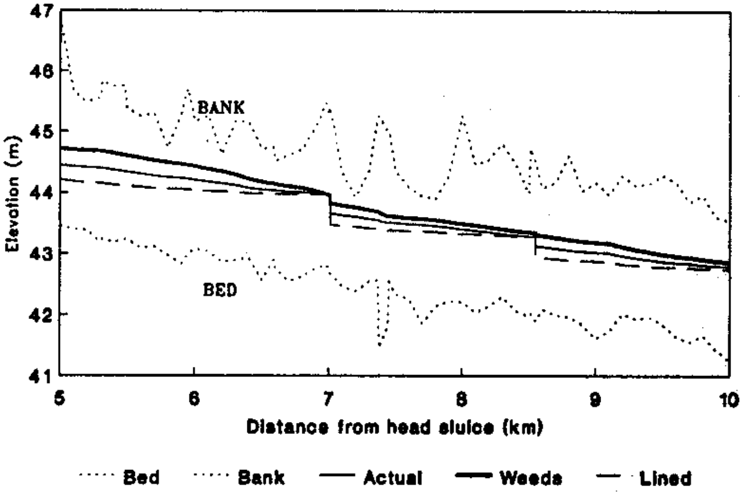
As expected, the highest water surface elevation is obtained when there is excessive weed growth. At two regulator locations, it is no longer possible to maintain the canal water level at FSD; this implies that if there is a lot of weed growth the canal banks could be overtopped even at relatively low discharges. The degree of weed growth cannot be expressed accurately in terms of a corresponding value of the roughness coefficient alone. The results are however of pedagogical interest and can also be used to orient further investigations.

The canal capacity increases by about 1.4 m³/s for average cement mortar lining (assuming an offtake discharge scenario of 2 l/s/ha). That is, the maximum permissible head sluice discharge is now 10.15 m³/s with a discharge of 4.25 m³/s becoming available at the tail end.

For the weed infested canal, the maximum permissible head sluice discharge falls to as low as 7.5 m³/s.

The importance of canal maintenance and its impact on canal carrying capacity are thus demonstrated.

Figure 9.19. Water surface profiles between the 5-km and 10-km points of the main canal for different roughness coefficients.



The potential benefits of lining the canal, at least in terms of increased carrying capacity are also shown. However, the actual benefits of canal lining would have to be assessed on an economic basis, taking into account factors such as smaller canal cross sections, less excavation, increased hydraulic radius and increased capacity, added cost of lining, different maintenance needs, etc.

CONCLUSIONS

The RBMC simulation model provides the system manager with a holistic view of the hydraulic functioning of the canal, especially useful under dynamic transient conditions. This in turn enables him to formulate coordinated canal operations strategies to respond to a variety of management situations, both routine and exceptional. Enhanced understanding of system behavior should also facilitate dialogue with farmers, eventually leading to a more productive role for them in water distribution activities.

From the point of view of IIMI's research, the Kirindi Oya RBMC simulation model provides an effective methodological approach to analyze problems related to main canal design and management. In addition, the model also serves as an innovative training tool.

This paper has highlighted some examples of the capabilities of the model to address the question of effective and responsive main canal operations. Specific applications covering routine management tasks (such as achieving a given water distribution plan) as well as exceptional events (such as canal filling and responding to rainfall) have been presented. Procedures for employing the model as a diagnostic and performance assessment tool have been outlined. Some issues pertaining to design and maintenance have also been addressed.

Extensive field-testing of the use of the model in support of practical system management is planned in the next phase of activity. IIMI's collaborator in this venture, the Sri Lanka Irrigation Department, has already manifested much interest in the potential uses of this innovative approach.

This experience in the practical use of the model will provide useful insight on the scope for computer-assisted management in manually operated irrigation systems and will probably determine prospects for extension of the methodology to other sites.

This paper has concentrated on the physical aspects of model use. However, it is recognized that there are a number of organizational implications related to the practical application of the simulation model as a management tool. Considerable improvement of the internal communication network and information transfer procedures, as well as attitudinal changes on the part of field staff will almost certainly be required if effective use is to be made of the range of possibilities afforded by the model. These aspects have not been discussed here but will constitute an integral part of the next phase of activity.

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