Development and Field-Installation of a Mathematical Simulation Model in Support of Irrigation Canal Management

(Research Paper)

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Jean-Pierre Baume
Hilmy Sally
Pierre-Olivier Malaterre
and
Jacques Rey

INTERNATIONAL IRRIGATION MANAGEMENT INSTITUTE
and
CENTRE NATIONAL DU MACHINISME AGRICOLE, DU GÉNIE RURAL, DES EAUX ET DES FORÊTS

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Please direct inquiries and comments **to:**
Information **Office**
International Irrigation Management Institute
P.O. **Box** 2075
Colombo
Sri Lanka

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Contents

Symbols

A	=	cross section area of flow
B	=	channelsurface
C_F	=	discharge coefficient for free-flow undershot gate
C_G	=	reference discharge coefficient for gate
C_S	=	discharge coefficient for submerged undershot gate
g	=	acceleration of gravity
H	=	total head
h	=	height of water surface above datum
h_S	=	gate height for overflow
K	=	Strickler's coefficient
k	=	0 for lateral inflow, 1 for lateral outflow
k_F	=	flow reduction coefficient for submerged gate
L	=	length of weir crest
n	=	Manning's roughness coefficient
Q	=	volumetric rate of discharge
q	=	discharge per unit length
R	=	hydraulic radius
S_o	=	bed slope
S_f	=	friction slope
V_D	=	demandvolume
V_{EF}	=	effective volume
V_S	=	supply volume
V	=	meanvelocity
W	=	underflow gate or offtake opening
x	=	distance in the direction of flow
y	=	verticaldepthofflow
Z	=	elevation
Z_c	=	critical elevation
F	=	smallincrement
μ	=	discharge coefficient for pipe free flow
μ'	=	discharge coefficient for pipe submerged flow
μ_F	=	discharge coefficient for free flow weir
μ_S	=	discharge coefficient for submerged weir
$\sum$	=	summation symbol

Abbreviations

CEMAGREF	:	Centre National du Machinisme Agricole, du Génie Rural, des Eaux et des Forêts
FSD	:	Full Supply Depth
ID	:	Irrigation Department
IIMI	:	International Irrigation Management Institute
RBMC	:	Right **Bank** Main Canal
SIC	:	Simulation **of** Irrigation Canals
SIE	:	Senior Irrigation Engineer
EGA	:	Enhanced Graphics Adaptor

Foreword

THE MANAGEMENT OF a manually operated irrigation canal with a number of control structures presents a special set of challenges to the system manager, who is often confronted with the problem of identifying and implementing a coordinated operational strategy to meet water delivery targets in the absence of adequate and reliable information on how the system is functioning.

The development of a microcomputer-based mathematical flow simulation model of the Kirindi Oya Right Bank Main Canal is the first phase in IIMI's efforts to provide canal managers with an innovative decision-support tool to help them meet these challenges.

This research project was also the beginning of a fruitful, mutually rewarding and lasting relationship between the International Irrigation Management Institute (IIMI) and the Centre National du Machinisme Agricole, du Génie Rural, des Eaux et des Forêts (CEMAGREF). This research paper, co-authored by staff members of both institutions, is yet another outcome of this excellent, collaborative relationship.

Khalid Mohtadullah
Director for Research
International Irrigation Management Institute

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THE RESEARCH STUDY described in this publication was carried out jointly by the International Irrigation Management Institute (IIMI) and the Centre National du Machinisme Agriwle, du Génie Rural, des Eaux et des Forêts (CEMAGREF), France. We are grateful to the management of both institutions for having supported this collaborative venture which has proved to be of immense mutual benefit.

We are deeply indebted to the Irrigation Department, Sri Lanka for permitting the use of the Kirindi Oya Right Bank Main Canal as the field site for implementing this study. The wholehearted cooperation extended at all times by the Chief Resident Engineer, the Senior Irrigation Engineer and the other staff of the Kirindi Oya Irrigation and Settlement Project has been particularly gratifying. The close links that were forged between the research team and the Irrigation Department in the course of this study are being further strengthened in the next phase of activity where we move from software development to field-application of the simulation model as a practical management tool.

A Study Advisory Committee oversaw the progress of the research project throughout its lifetime. We wish to extend our sincere thanks to all its members who gave of their time and intellect to ensure that the project objectives were fulfilled. A special tribute is due to the indefatigable Committee Chairman, Mr. Remy Pochat, who so effectively guided the work of the Committee.

We would like to place on record the key role played by Daniel Berthery, who, during his tenure on IIMI's staff, was instrumental in getting this research study off the ground. We also wish to acknowledge the contribution of Frederic Certain, Project Leader from the CEMAGREF side in the initial stage, who was responsible for much of the early software development.

This document benefitted from the comments and suggestions made by a number of people at various stages of the project; we cannot obviously cite all of them; Dr. R. Sakthivadivel and Dr. W. Schuurmans acted as technical reviewers. Intensive editing by Nimal A. Fernando helped a lot in making this document easier to read.

We also wish to take this opportunity to express our deepest gratitude to the Government of France, which provided the major funding support for the implementation of this research study and for the production and dissemination of this document.

Jean-Pierre Baume
Hilmy Sally
Pierre-Olivier Malaterre
Jacques Rey

Executive Summary

IT IS ACKNOWLEDGED that suboptimal performance of irrigation systems may often be traced to deficiencies in managing the conveyance and distribution of water in the main system.

One of the handicaps faced by irrigation managers in preparing coherent overall operational plans is the ahsence of a decision-support tool capable of providing them with a holistic view of the system. Consequently, in many manually operated irrigation schemes, system management tends to be the **sum** total of a number of uncoordinated, individual interventions at the different control pints, resulting in operational losses and inefficient water distribution.

It **was** in this context that IIMI decided to embark upon a research project to seek ways ***of*** improving irrigation performance through the identification and implementation of effective and responsive main canal operations.

Technical, social and economic constraints limit the scope for carrying out such research through direct experimentation on real-life irrigation systems. Therefore, mathematical simulation, which allows repetitive testing of a wide variety of design and management alternatives without adverse impacts on the physical infrastructure or normal system operations, was adopted as the research methodology.

This paper describes the development and field-installation **of** a mathematical flow simulation model for the Kirindi Oya Right Bank Main Canal (RBMC) in Sri Lanka, the scheme selected for IIMI's first pilot study on this subject. The work, which constitutes Phase I of an IIMI research program intended to be of regional scope, was carried out in partnership with CEMAGREF, France and the Irrigation Department, Sri Lanka.

The RBMC simulation model is intended to serve not only as a research and training tool to study the hydraulic behavior of irrigation canals but also **as** a decision-support tool for managing a manually operated irrigation system, wherein lies its innovative feature. Therefore, in developing the model, special attention was paid to incorporate user-friendly input-output interfaces to facilitate its use by canal managers and nonspecialists in computer technology and numerical hydraulics.

The principal features and theoretical concepts underlying the development of the three independent software units which make **up the** core of the model are detailed in the paper. The three units respectively generate and verify the canal topography, carry out steady flow computations, and simulate canal operations under unsteady flow conditions.

The modular architecture adopted for the software will allow for future addition of other computational units **as** needed, such **as** automatic regulation modules, demand prediction modules, etc.

Extensive field measurements were carried out to evaluate the physical and hydraulic parameters needed to calibrate the model. The values obtained for most of these parameters show deviations from the values assumed at the design stage, highlighting the importance of regularly monitoring and updating these parameters in order that the model continues to accurately simulate the hydraulic behavior of the canal.

A limited set of model applications is also presented with a view to illustrating the capability of the model to address a range of typical canal design and management issues.

Comprehensive field-testing of the simulation model as a decision-support tool in canal operations is underway, under Phase II of the research project which commenced in February 1991. The results are expected to confirm the feasibility of using simulation models in support of the practical management of a manually operated irrigation system, and will be presented in a forthcoming publication.

CHAPTER 1

Introduction

BACKGROUND

IRRIGATED AGRICULTURE, THOUGH practiced only on about 15 percent of the world's total cultivated land, acwunts for more than 40 percent of the total world food production. Policymakers, planners and donors hence justifiably express wncern over the poor performance of irrigation systems, especially in the context of a growing population, increasingly scarce land and water resources, and fewer opportunities for investments in new irrigation development. One way of meeting the additional demand for food is by increasing cropping intensities. This necessarily implies more efficient use of available water and improved performance of existing irrigation systems.

Early efforts at improving irrigation system performance tended to focus on the tertiary level and the main system was generally assumed to be functioning according to design and delivering reliable, adequate and timely quantities of water. But, inefficient management of the conveyance and distribution of water in the main system often negates even the best efforts of farmer organizations and irrigation agencies to achieve equitable water supply below turnouts. Main system management is considered to hold the key to improving canal irrigation performance (Chambers 1988).

The use of traditional research methodologies involving field experimentation to investigate main system management practices is seldom possible in real-life irrigation systems. Farmers would be inconvenienced and crops could be adversely affected. Moreover, monitoring, analysis and evaluation of hydraulic phenomena, which vary rapidly in time and in space, based purely on physical observations are difficult. The experiments would also be difficult to replicate.

Mathematical flow simulation models offer a viable alternative to direct experimentation on the physical system. Any number of repetitive tests can be run to study system behavior under a variety of design and management scenarios without modifying the physical infrastructure of the canal or disrupting its normal operations. The potential impacts of any planned design and/or management change can be evaluated prior to actual implementation. Effective and responsive operational practices, compatible with the physical facilities and the management capacity of the agency can be identified.

It must, however, be emphasized that a simulation model cannot be properly developed and applied independent of field work. The model will only be as good as the input data used to describe the real system. Site-specific data gathering to build and calibrate the model and to ensure that all hydraulically significant canal features are accurately represented constitutes an integral part of modeling.

Mathematical flow simulation models have unfortunately tended to be the preserve of researchers, consultants and hydraulic specialists. The models themselves are often not easy to use by someone unfamiliar with their development. Furthermore, Gichnki **(1988)** reports that "the major effort in modeling has been concentrated in a few developed countries." Irrigation agency staff, in particular, have little or no opportunity to use such models to meet their operational needs. On the other hand, Wade and Chambers **(1980)** emphasize the need to devise appropriate methods to aid system managers in scheduling and distributing irrigation water. It is in this light that IIMI decided to embark upon a project aimed at developing a user-friendly mathematical flow simulation model to address main system management issues.

The specific objectives being pursued are:

1. To provide a state-of-the-art research and training tool to investigate the hydraulic behavior of the main canal, with particular emphasis on understanding the interactions between the design, management and performance of the canal.
2. To identify appropriate operational practices for the main canal, which are compatible with its physical and organizational infrastructures.
3. To implement, with the assistance of the irrigation agency, such operational practices and to assess their impact on the manageability and performance of the canal.
4. **To** demonstrate, through a rea-life application, the feasibility of using a mathematical flow simulation model **as** a decision-support tool in a manually operated canal system.

This paper describes the first phase of the research project carried out by IIMI and CEMAGREF which was focused on the development and application of the mathematical flow simulation model to the Kirindi Oya Right Bank Main Canal (RBMC) in southern Sri Lanka. The Irrigation Department, Sri Lanka collaborated with IIMI and CEMAGREF in implementing the project. Software development, production of a comprehensive set of manuals, calibration of the model, as well as applications to formulate suitable operational responses to some typical canal management problems have been completed. The second phase of the project, involving comprehensive field-testing **of** the simulation model **as** a decision-support tool in support of canal operations at Kirindi Oya, is currently underway.

THE FIELD SITE

Selection

A review of potential sites in Sri Lanka for the implementation of the research project led to the choice of the Kirindi Oya Right Bank Main Canal on the **basis** of its physical features and management problems (IIMI **1987).**

The criteria adopted in the selection included

1. The physical features of the site, which determine (a) the amount of data (topographical, hydraulic, hydrologic, etc.) necessary to represent the system and its environment with a reasonable degree of accuracy, and (b) the complexity of the intended model and the level of difficulty in interpreting the results.
2. The amount of data already available and the practical difficulties in identifying *a priori* important data and in organizing their collection.
3. The magnitude and nature of the problems faced in relation to water distribution.
4. The degree of interest and participation of the irrigation agency in data collection and in future use of the model.

The Physical Context

The Kirindi Oya Right Bank Main Canal (RBMC) is part of the Kirindi Oya Irrigation and Settlement Project in southern Sri Lanka (Figure 1). The principal objectives of the project are: (a) the augmentation of water supplies to the existing older irrigated area of around 4,500 hectares (ha); (b) provision of irrigation facilities, through the right bank and left bank main canals, to an additional command area of about 8,400 ha; and (c) the settlement of over 8,000 families on the newly developed lands.

The RBMC itself was intended to irrigate about 5,000 ha of land. The development of 3,650 ha (consisting of irrigation tracts 1, 2, 5, 6, and 7) has been completed to date. But only the 2,743 ha of tracts 1, 2, and 5 were irrigated when the scheme was commissioned in 1986; tracts 6 and 7 received imgation water for the first time in 1991.

The RBMC is an unlined earth canal, 32 kilometers long, with a design bed slope of 3 in 10,000 (30 cm per km) and is fed by the Lunuganwehera reservoir (198 million m^3 active storage capacity), from which the left bank main canal also takes off. The RBMC 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 simulation model being discussed covers only the first 25 km of the main canal, encompassing irrigation tracts 1, 2 and 5. A total of 33 distributary and field canals take off from the main canal over its 25-km length (Figure 2).

Figure 1. Kirindi Oya Irrigation and Settlement Project (as planned).

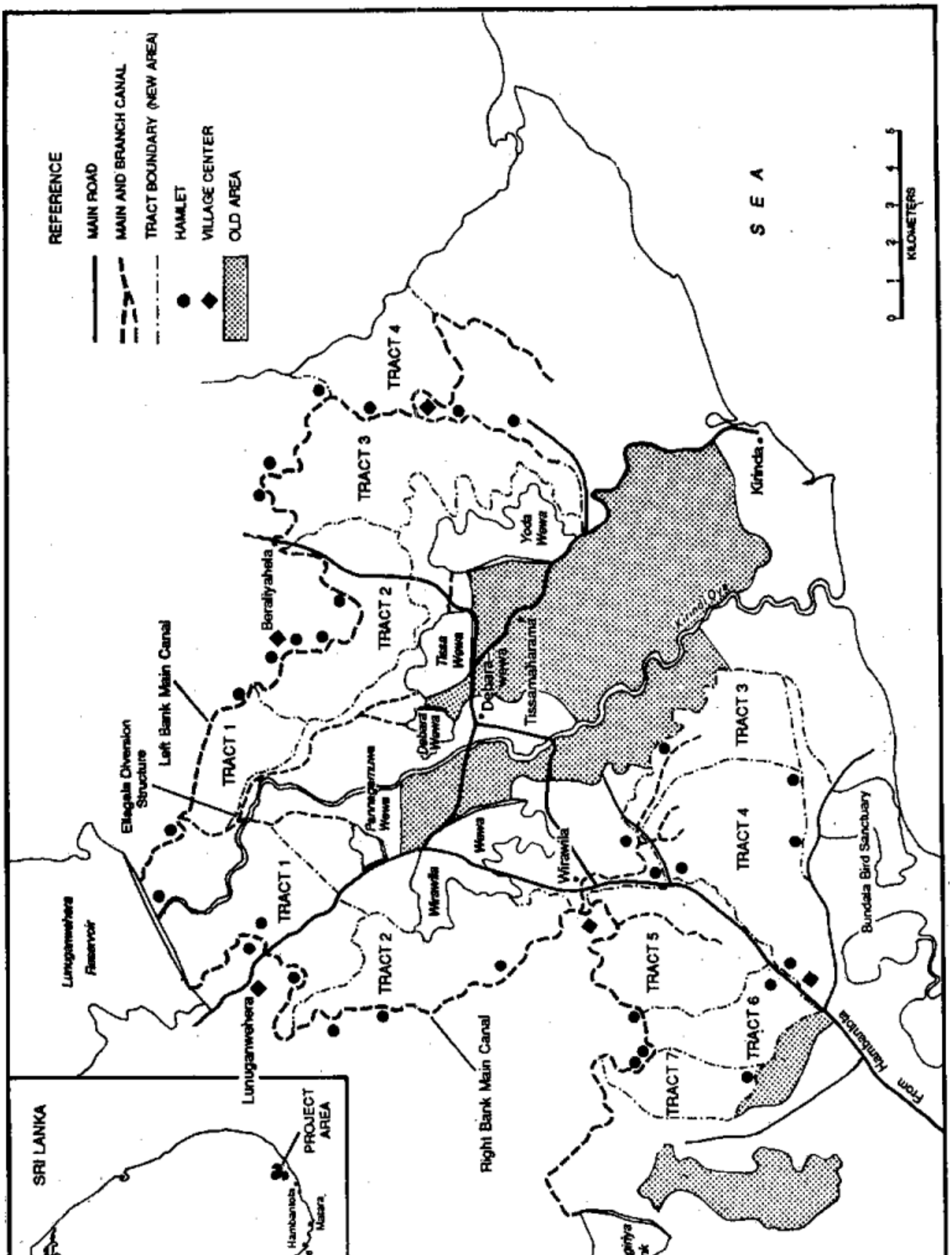
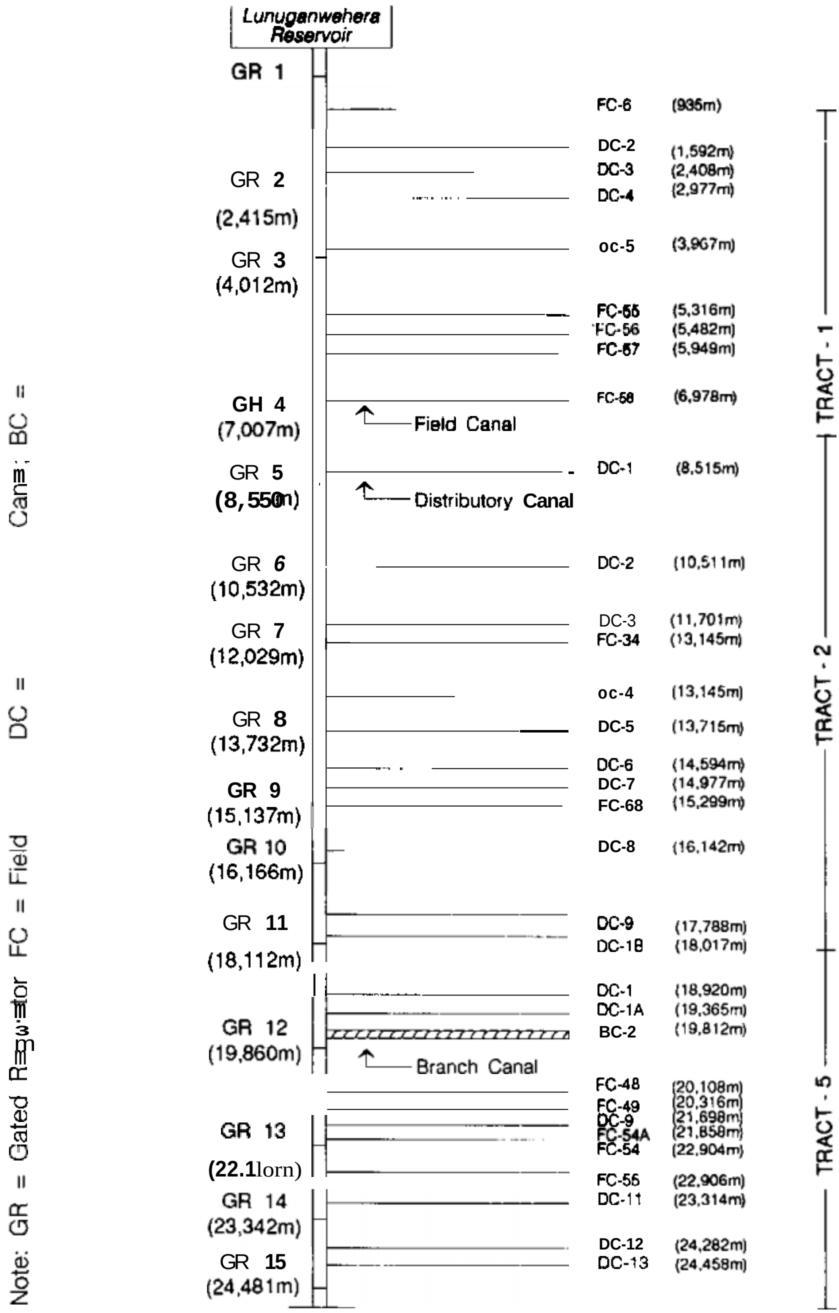
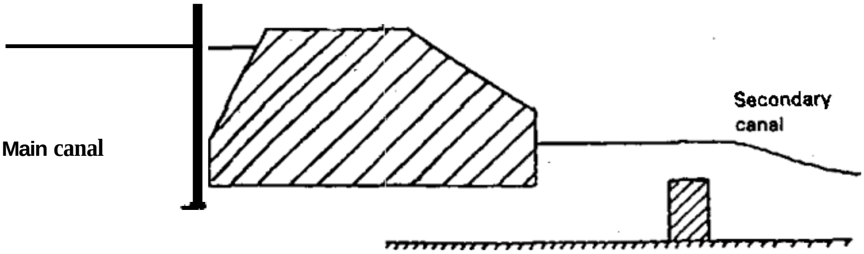


Figure 2. Issue tree diagram of the Kirindi Oya Right Bank Main Canal.



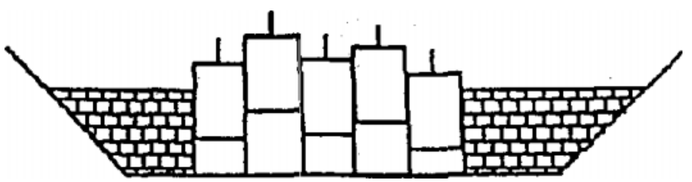
The offtakes are gated and are of the undershot type. The downstream water level is controlled by weirs (either sharp-crested or broad-crested) which also serve as flow measuring devices (Figure 3). The broad-crested weirs are of fairly recent origin, having been constructed to gradually take the place of the original sharp-crested weirs which were often found to be functioning under submerged-flow conditions.

Figure 3. An offtake cum flow measuring device.



Water level control in the RBMC is ensured by 14 gated cross-regulators, composed of a set of manually operated undershot gales (ranging in number from 5, at the head of the main canal, to 2, at the tail) and a pair of lateral sidewalls (Figure 4). The presence of these regulators theoretically provides the canal manager with a great degree of flexibility in managing the conveyance and primary distribution of water. In point of fact, the hydraulic interdependency between successive canal reaches and difficulties in coordinating the operations of the cross-regulators render the overall management of the canal complex.

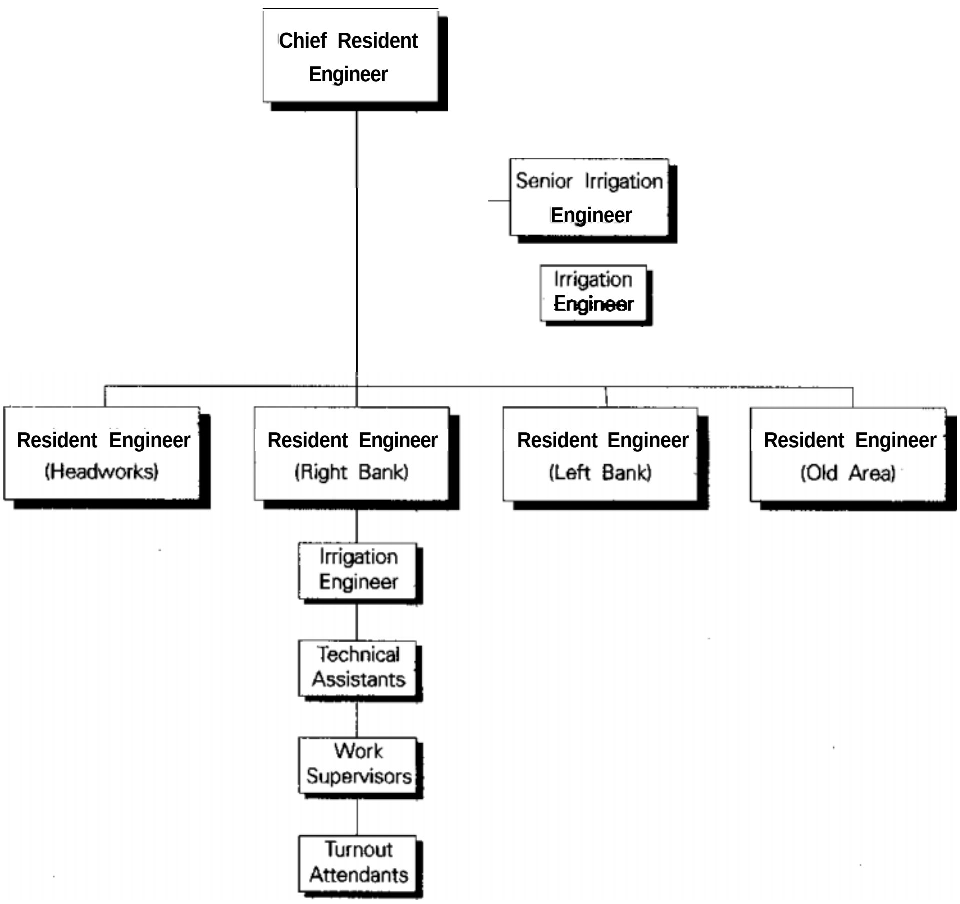
Figure 4. A gated cross-regulator.



The Operational Context

The managing agency in Kirindi Oya is the Irrigation Department (ID) of Sri Lanka. A succinct view of the organizational chart of the project is shown in Figure 5. Water management activities are coordinated by a Senior Irrigation Engineer (SIE). The operational objective of cross-regulator gate operations is to maintain full supply depth (FSD), corresponding to the crest levels of the regulator sidewalls, by adjusting the openings of the regulator gates. Gate operators usually have the added responsibility of operating a certain number of offtake gates (along the main canal as well as along neighboring distributary and field canals). The objective here is to deliver target discharges, assessed by means of the measuring devices located at the heads of these canals.

Figure 5. Organizational chart of the **Kirindi Oya** Irrigation **and** Settlement **Project.**



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 the canal water levels which, in turn, result in inequitable water distribution. Efficient and responsive canal operations assume even greater importance given the chronic water-short situation (which has prevented double-cropping from being practiced throughout the Kirindi Oya scheme) and the promotion of diversified cropping. Furthermore, the original plan to extend the irrigated command area to include tracts 3 and 4 has been abandoned.

In this context, seeking greater efficiency in water distribution through improved main canal operations is highly relevant.

Constraints to effective main canal operations have come to light in the course of this present research project as well as in other studies dealing with aspects such as:

* The impact of design on the management and performance of the main canal (IIMI 1989).
* Irrigation system management and crop diversification (IIMI 1990).
* Management decision-making processes (Nijman 1992).

However, the lack of a suitable decision-support tool makes it difficult for the engineers in charge of water management to prepare overall operational strategies involving the entire canal and all its control points. This situation is aggravated by inadequate information transfer procedures. Gate operators sometimes unknowingly carry out inappropriate local adjustments to control structures which result in instabilities in canal water levels throughout the system. Such uncoordinated canal operations give rise to substantial operational losses and water waste. The simulation model, supported by suitable communication facilities, can help avoid such potentially costly, trial-and-error interventions on the part of gate operators.

A SIMULATION MODEL

In implementing this research project, IIMI (collaborated closely with CEMAGREF, who provided expertise and experience in software development related to numerical open-channel hydraulics. Partial financial support was provided by the Government of France. Work began in early 1988 with an initial topographical survey of the Kirindi Oya RBMC.

The RBMC software package was built around numerical computational modules originally developed by CEMAGREF to run on mainframe computers. They had to be substantially modified to allow running on microcomputers. Conversational and user-friendly input and output interfaces were incorporated to make the model accessible to canal managers without special training in numerical hydraulics or computer technology and for easy interpretation of results.

The modular structure of the software allows the addition of other modules without extensive re-programming of the initial software package. For example, a hydrologic module (to provide inputs relating to water resources data, rainfall, intermediate tanks flows, etc.), a water demand module (which computes requirements at the farm level and progressively aggregates the demand up to the offtake located in the main canal), or an automatic regulation module (to test real-time operational procedures) may be added. A detailed description of the component units of the model, its field calibration and some applications will be presented in this paper.

CHAPTER 2

The Simulation Model

MAIN FEATURES

THE KIRINDI OYA RBMC software is a mathematical model which simulates the hydraulic behaviour of the Kirindi Oya Right Bank Main Canal under steady and unsteady flow conditions.

The model has been designed to serve not only as a training and decision-support tool for the canal managerbut also as a research tool for the improvement of the hydraulic functioning of the main canal.

The model is dedicated to the Kirindi Oya Right Bank Main Canal but the specialist may adapt it to any other non-branched, non-looped canal configuration.

The RBMC software is designed to run on an IBM PC-AT or PS/2 compatible microcomputer under MS/DOS operating system. It needs a mathematical coprocessor and an EGA screen. There must be at least 1MB of RAM and about 2 MB of memory is required on the hard disk for the three units. The software also has graphics output capability on HP compatible plotters (HPGL language) in A4 or A3 format.

The model is built around three main computer programs (TALWEG, FLUVIA and SIRENE) that respectively carry out topography generation, steady flow computation and unsteady flow computation. These units can be run either separately or in sequence.

Unit I generates the topography files used by the computation programs of units II and III. Access to this unit is restricted to advanced users familiar with hydraulic modeling. No user-friendly interface has thus been developed for this unit. It can be accessed through a hidden menu. It allows the advanced user to input and verify the data obtained from a topographical survey of the canal.

Unit II performs the steady flow computation and generates the water surface profiles for any given combination of offtake discharges and cross-regulator gate openings. These water surface profiles may be used as initial conditions for the unsteady flow computation in Unit III. Unit II also allows the determination of offtake gate openings and adjustable-regulator gate settings required to satisfy a given water distribution plan while simultaneously maintaining a set of target water levels in the main canal.

Unit III carries out the unsteady flow computation. It allows the user to test various scenarios of water demand schedules and operations at the head works and control structures. Starting from an initial steady flow regime, it will help the user to identify the best way to attain a new water distribution plan. The efficiency of the operational strategy may be evaluated via a set of water delivery indicators computed at the offtakes.

The RBMC model is an efficient tool that allows the canal manager as well as the researcher to quickly simulate a large number of hydraulic design and management confi rations of the canal. User-friendly interfaces have been developed so that people with a minimum knowledge in computer science and in hydraulics can run the model. The software is menu-driven (except for Unit I, which is reserved for specialists).

On-line help screens have also been developed to enable the user of the RBMC model to work more quickly without systematically needing to refer to the printed documentation.

The model generates a large volume of numerical results, especially Unit III, which carries out the unsteady flow computation. These results can be displayed in three different ways:

1. While the program is running, general information on the progress of the calculation is displayed.
2. After the calculation, specific programs allow the display of results as numerical arrays. These arrays are displayed on the screen or stored as ASCII files that may be printed later.
3. After the calculation, another set of special programs allows the display of results in graphical form. These graphics are displayed on the screen (EGA) or stored in files to be printed on a HP compatible plotter.

LIMITATIONS

The RBMC model has been specifically adapted to the Kirindi Oya Right Bank Main Canal and it is not able to simulate:

* Branched or looped networks.
* Supercritical flows (the water level is forced to the critical depth if a supercritical zone is detected).
* Dry-head flows.

A new model has been developed by CEMAGREF based on the experience of the RBMC model. This model, called SIC (Simulation of Irrigation Canals), is able to simulate branched networks (under steady and unsteady flow conditions) and looped networks (under steady flow conditions). Many other features of the RBMC model have been improved in SIC. This software is distributed by CEMAGREF, Montpellier, France.

THEORETICAL CONCEPTS

In this section, the theoretical concepts on which the model is based to take into account the canal topography as well as to carry out the hydraulic computations themselves are presented. The flow simulation in the model is based on one-dimensional hydraulic computations under steady and transient regimes. A comprehensive list of symbols and abbreviations being used is given on pages vi and vii.

Unit 1 – Topography Module

A main canal network is an open-channel water distribution system, which conveys water from a source (reservoir or river diversion) to various offtakes that deliver water to user groups via secondary and/or tertiary canals.

The hydraulic modeling of such a network needs to take into consideration the real canal topography: the main canal network topology and the geometric description of the main canal. Unit 1 is responsible for managing all the topographic components used by the model.

Hydraulic Network

The hydraulic network is divided into homogeneous reaches (in terms of discharge, i.e., with no local inflows or outflows) located between an upstream node and a downstream node. Links between reaches occur only at the nodes.

Choice of reaches: The choosing of reaches by the model user is subject to some constraints; constraints due to network topology, and constraints associated with points of inflows or outflows (which can occur only at model nodes).

The user may, however, divide any part of the canal into several reaches in order to take into account some particularity, even if such a division is not imposed by the constraints described above.

For instance, one can create a different reach for a lined canal zone (low roughness), and an unlined canal zone (high roughness). One can also create reaches for administrative or other reasons.

The division into reaches for the user's convenience does not influence the results of the hydraulic calculation. Generally, when one divides a reach artificially, the lowermost cross section of the upstream reach is the same as the uppermost cross section of the reach immediately downstream.

If different regulating or control devices exist across the canal, they can he integrated within a reach and do not need any special division.

Choice of branches: The choice of reaches is principally linked to the hydraulic constraints. In order to let the user visualize portions of the canal that he wants to treat together, he may group a number of reaches into a branch. A branch is therefore a group of reaches serially linked to one another (Figure 6).

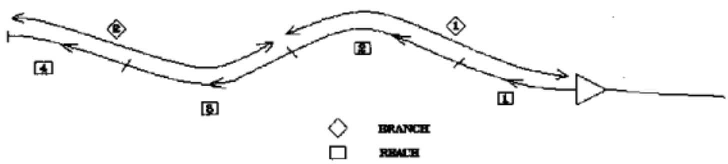
Classification ofreaches: In incorporating the network topology into the model, reaches are identified by their nodes. The position of a reach in the network is defined by the names of its upstream and downstream nodes. The direction of flow is defined at the same time. The network topology can he simply described as an oriented graph.

The reaches constitute the arcs of that graph delineated by the nodes, upstream and downstream. They are automatically numbered by the program according to the order in which they are input into the data file.

Subcritical flow being controlled by the downstream conditions, the calculation of a water surface profile proceeds upwards, commencing at the downstream end.

Therefore, a relationship between water surface elevation and discharge is needed as a downstream boundary condition to start the calculation.

Figure 6. Reaches and branches.



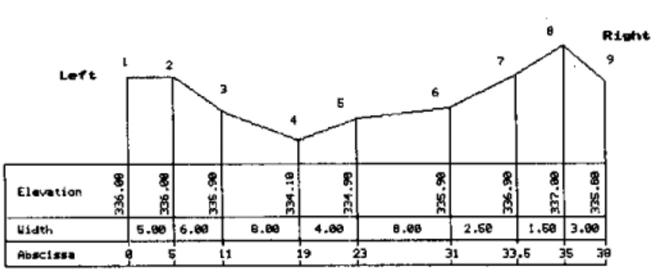
Cross Sections

The geometry of the reaches is the basic element of all the hydraulic calculations. The reach geometry is determined by the cross-section profiles characteristic of the shape and the volume of the canal. The elevations are indicated with reference to a unique datum in order to allow computation of the local slopes. The cross-section profiles are situated along the curvilinear canal abscissa (longitudinal abscissa).

Generally, the main sluice at the **dam** is chosen as the origin with the orientation in the direction of flow. All reaches being in series, only one longitudinal abscissa is required for the whole model.

The cross sections *can* be described and entered in three different ways: abscissa-elevations, width-elevations and parametric form (Figure 7). The type of description may vary from one section to another, within a given reach.

Figure 7. Cross sections.



Cross sections provided by the surveyor are usually abscissa-elevation. Each point is input in terms of its cross-wise abscissa and its elevatiop.

In the width-elevation description of a cross section, for each value of elevation, the width of the corresponding section is entered. This description is generally adopted when one does not have precise information on the section or if the section is symmetrical. If one enters the cross section in width-elevation couples, any asymmetry in the section is not taken into account. The wetted perimeter is computed assuming a symmetrical section. The only way to take into account an asymmetrical section is to enter it in terms of abscissa-elevation couples.

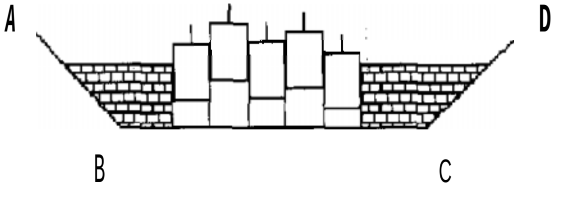
Sections of special geometrical shape can be input in parametric form (circle, culvert, power relationship, rectangle, trapezium or triangle).

Singular Sections

Cross sections containing cross structures are called singular sections. In these sections, the general hydraulic laws for computing water surface profiles are not applicable. These laws are replaced by the discharge formulas of the structures.

It is not necessary to describe the hydraulic devices when entering the canal geometry into the model, but the sections where they are located have to be indicated. The dimensions of the canal, and not those of the devices, should be entered at this stage. For instance, in Figure 8, the ABCD profile is entered.

Figure 8. A singular section.



A singularity is considered to occur at a single point in relation to the whole reach. One must thus enter two sections, supposed to be *at* the same abscissa, corresponding to the upstream and downstream sides of the singularity.

The upstream section represents the canal dimensions upstream of the device, while the downstream section represents the canal dimensions downstream of the device. If one inserts only one section in order to describe a singular section, the program automatically generates a supplementary section.

Computational Sections

Data sections may be unequally distributed along the canal. In fact, the model user should select sections which best represent the canal dimensions, the changes of slopes, and so on. Depending on the regularity of the canal, the spacing of the data sections may then he small or large.

For the hydraulic calculation, the spacing between computational sections should be such that a reliable estimation of the water sutface profile is possible. This spacing is chosen by the model user depending on his knowledge of the canal hydraulic behaviour (when the data sections are too far apart, the model interpolates supplementary computational sections in order to allow a better simulation of the water surface profile).

All the entered data sections are retained as computational sections. Irrespective of the manner in which a data section was defined, the program transforms it into width-elevation data[1] for storage and interpolation. Only groups of 4 characteristic values — elevation, width, wetted perimeter[2], and area — are finally retained.

Interpolation of computational sections: If the distance between two data sections is more than one computational (space) step, the program interpolates computational sections between these two data sections, in accordance with the step decided by the user.

In reality, the space step is adjusted in order to give a whole number of equal computational intervals between the two sections considered. The interpolation is performed at constant water depth.

Acomputational section could thus have up to double the number of points of the data sections. Therefore, one must adopt some criteria to eliminate certain points and to avoid storing too many points.

Any point of the Computational section which does not modify the section area by more than 5 percent is eliminated. Similarly, all points which do not modify the wetted perimeter by more than 10 percent are excluded. Therefore, whatever the water depth may be, a precision of 5 percent regarding the estimation of the area and 10 percent regarding the wetted perimeter can be expected.

---

1 At any cross section, the program **looks** for all the high and low points. It determines right and left hanks and eliminates points outside the bed. Using the high and **low** points, it divides the cross section into channels, and for each channel it effect. the transformation into the width-elevation format. Then, **for** each elevation, it adds the widths of all the channels.

2 If a section had been entered in **terms of** abscissa-elevation, the welted perimeter would take into account the section asymmetry.

The computational sections are completed vertically by a fictitious point located 100 meters above the canal bank elevation in order to allow calculation even if overtopping occurs.

The computational sections are numbered within each reach **so** that if the computational space step is modified in any given reach, the numbering of the sections within other reaches will remain unchanged.

Interpolation of singular sections: In the case of a singular section, it is necessary to have two computational sections at the same abscissa. **If** the model user entered two data sections at the same abscissa, both these sections are retained **as** computational sections. If only one section was entered, the downstream computational section is interpolated using the singular data section and the data section immediately downstream with a 1m step. The interpolated section is then placed at the same abscissa as the singular section. Therefore, one has to take care to enter two sections at the same abscissa, especially if the bed elevations upstream and downstream of the device are different or if the section dimensions are different.

Unit 2 – Steady Flow Module

Unit *2* computes the water surface profile in a canal under steady flow conditions. This water surface profile can be used as the initial condition for the unsteady flow computation in Unit **3.** Steady flow calculations also allow the testing of the influence of modifications to structures, canal maintenance, etc.

hi addition, a sub-module of the steady flow module computes the offtake gate openings to satisfy given target discharges. Another sub-module computes the cross-regulator gate openings to obtain a given targeted water surface elevation upstream of the regulator (e.g., Full Supply Depth). These sub-modules, therefore, allow computation of gate settings to satisfy a given demand-supply configuration of water flow.

It should be emphasized that in actual canal operations, the steady flow regime represents the objective to be attained, and that it is the unsteady flow model which will indicate how best to reach it in time.

Eqwtwn of Gradually Varied Flow in a Reach

The canal being divided into homogeneous zones (the reaches), the problem gets reduced to calculating the water surface profile under subcritical, steady flow conditions in a reach.

The classic hypotheses of unidimensional hydraulics in canals are considered to apply when:

* The flow direction is sufficiently rectilinear, so that the free surface could be considered to be horizontal in a cross section.
* The transversal velocities are negligible and the pressure distribution is hydrostatic.
* The friction forces are taken into account through the Manning-Strickler coefficients.

Tberefore, only monodimensional steady flow is studied, and only subcritical flow is considered.

Differential equation of **the water surface profile:** The equation of the water surface profile in a reach can be written as follows:

$$\frac{dH}{dx} = -S_f + k\frac{qQ}{gA^2} \qquad [1]$$

with:

$$S_f = \frac{n^2Q^2}{A^2R^{4/3}}$$

To solve this equation, an upstream boundary condition in terms of discharge and a downstream boundary condition in terms of water surface elevation are required.

In addition, the lateral inflow and the hydraulic roughness coefficient along the canal should be known. As the equation does not have an analytical solution in the general case, it is discretized in order to obtain a numerical solution. Knowing the upstream discharge and the downstream water elevation, the water surface profile is integrated step by step, starting from the downstream end.

Integrating equation [1] between *two* sections, i) and j) gives:

$$H_j - H_i - kq\frac{\Delta X_{ij}}{2g}\left(\frac{V_j}{A_j} + \frac{V_i}{A_i}\right) + \frac{S_{fi} + S_{fj}}{2}\Delta x_{ij} = 0 \qquad [2]$$

Equation [2] can be written as follows:

$$H_i(Z_i) = H_j + \Delta H(Z_i)$$

A subcritical solution exists if the curves $H_i(Z_i)$ and $H_j + \Delta H(Z_i)$ intersect
For this, it is necessary that:

$$6 = Hj + \Delta H(Z_{Ci}) - Hi(Z_{Ci}) > 0$$

Z_{Ci} is the critical elevation defined at i by $\frac{Q_i^2 B_i}{gA_i^3} = 1$

$6 > 0$: Subcritical solution.

$6 < 0$: Supercritical solution. One assumes systematically the critical depth. The water surface profile is therefore overestimated.

If a solution does exist, one has to numerically solve an equation of the form $f(Z_i)=0$, for which Newton's Method is used.

Cross Structure Equations

When cross structures exist on the canal (singular section), the water surface profile equation cannot be used locally to calculate the water surface elevation upstream of the structure. The hydraulic laws of the different devices present in the section must be applied.

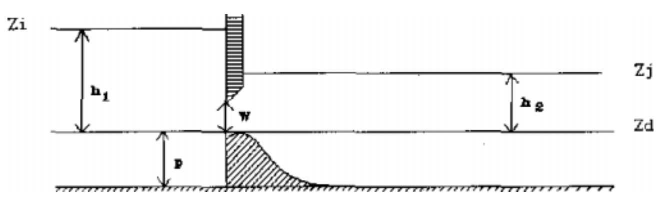
The modeling **of** these devices is a delicate task when developing open-channel mathematical models. The equations used to represent the hydraulic devices are numerous and do not cover all the possible operating conditions.

In particular, it is rather difficult to maintain the continuity between the different formulations as, for example, at the instant **of** transition between free flow conditions and submerged conditions, or between upen-channel conditions and pipe flow conditions.

What has been chosen here is a simple way of modeling the weir/orifice type of devices (high sill elevation) and a formulation derived from the previous case, giving better results for the weir/undershot gates (small sill elevation). The sill elevation is indicated as p in Figure 9.

More details of the cross structure modeling are given in Annex 1.

Figure 9. Cross structure description.



Equation at a singular section: The water surface elevation at a singular section is computed using the equations presented in Annex **1.** The flow at the section is equal to the sum of the discharges through each device (e.g., gate, weir).

$$\sum_{k=1}^{n} f_k\,(Z_i, Z_j) = Q \qquad [3]$$

n is the number of devices in the section and Q the flow at the section

$f_k\,(Z_i, Z_j)$ is the discharge law of the device number k, for instance, for a submerged weir:

$$f_k(Z_i,Z_j) = \mu L\sqrt{2g}(Z_i-Z_j)^{1/2}(Z_j-Z_d)$$

If the discharge and the downstream elevation Zj are known, the water surface elevation Z_i upstream of the device can be calculated.

This means that one has to solve an equation of the form $f(Z_i)=0$ (using Newton's Method).

Regulator: At each singular section, one particular gate can he chosen to play the role of a regulator.[1] The opening of this gate is unknown. The maximum possible opening and the target water elevation (e.g., Full Supply Depth) upstream of the gate are known. This results in an equation at the singular section similar to the previous one, but in this case, the unknown **is** no longer the upstream water surface elevation but the opening of the gate working **as** a regulator. One ends up with an equation of the following type:

$$Q-\sum_{k=1}^{n} f_k(Z_i,Z_j) = f_r(Z_i,Z_j,W) \qquad [4]$$

with:

$k = 1$ to n:	For gates with fixed openings.
W:	The regulator opening to he calculated.
Z_i:	Known value (targeted upstrcam water elevation).
$f_k(Z_i,Z_j)$:	The discharge going through the fixed gate number k for the target upstream water elevation Z_i and the downstream water elevation Zj. The equations considered are those described for the weirs and the gates.
$f_r(Z_i,Z_j,W)$.	The discharge going through the regulator type gate for an opening W and the target upstream water elevation.

The $f_k(Z_i,Z_j)$ are known values. Then equation [4] is reduced to $f_r(Z_i,Z_j,W)$ = constant. One, then, has to look for the zero of a function, hut this time, the unknown is W.

Offtake Equations

The lateral offtakes correspond to points of outflow. Therefore, they are obligatorily located at the upstream nodes of the reaches. Under steady flow conditions, one cannot compute the real offtake discharge corresponding to a given offtake gate opening, **as** this can he done only with a looped model. But, knowing the offtake target discharge, the program is able to calculate the corresponding offtake gate opening.

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1 This means that the opening of this gate is not fixed ***a priori.*** Instcad, the model will compute the opening required to maintain a target water level immediately upstrcam. The openings of all uther gates are fixed ***a priori.***

The offtakes are modeled according to the same hydraulic laws as for cross structures. The originality of the approach resides in the consideration of a possible influence of the offtake downstream conditions.

Offtake downstream conditions: In order to include the possibility of submerged flow conditions at the offtakes, three types of offtake downstream conditions (i.e., at the head of the secondary canal [see Figure 10]) can be modeled:

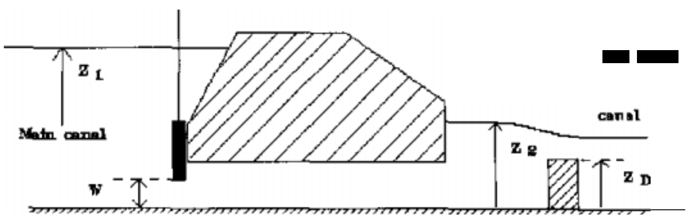
* A constant downstream water surface elevation.
* A downstream water surface elevation Z_2 that varies with the water surface elevation upstream of a free flow weir:

$$Q(Z_2) = \mu L\sqrt{2g}(Z_2-Z_D)^{3/2} \qquad [5]$$

* A downstream water surface elevation that follows a rating curve of the type:

$$Q(Z_2)=Q_0\left(\frac{Z_2-Z_D}{Z_0-Z_D}\right)^n \qquad [6]$$

Figure 10. Offtake description.



Equations: For the discharge, the equations described above are used for the undershot gates. If the offtake is circular, one has to calculate the width of the equivalent rectangular opening in order to be able to use the equations presented in Annex 1.

Then, an equation **of** the following type should he solved:

$$f_p(Z_1, Z_2, W) = Q_P$$

with:

Q_p :	Target offtake discharge.
Z_1 :	Upstream water surface elevation in the main canal obtained via the water surface profile computation.
Z_2 :	Downstream water surface elevation. This is either known, or its value depends on the offtake discharge Q_p and the chosen offtake downstream condition. If Z_2 is a function of Q_p, then, an equation **of** the following **form** is obtained:

$$f_p\left(Z_1, f_s^{-1}(Q_p), W\right) = Q_p \qquad [7]$$

with f_s being the rating curve corresponding to the chosen offtake downstream condition.

Therefore, in all cases, the problem is to find the zero of a function with W as the unknown. The bisection algorithm is used.

Unit 3 – Unsteady Flow Module

Unit **3** computes the water surface profile in the canal under unsteady flow conditions. The initial water surface profile is provided by Unit 2 (steady flow module). Unit **3** allows, for example, the study **of** the transition from one rotational schedule to another. In addition, it calculates the offtake discharges knowing the offtake openings.

But, unlike in Unit 2, it is not possible to automatically compute a regulator gate opening knowing the target upstream water level. It is necessary to incorporate special regulation modules in order to address this problem or similar problems involving water surface elevation or discharge targets.

Saint-Venant's Equations

The canal is divided into homogeneous **zones,** the reaches. In computing the unsteady flow water surface profile in a single reach, the same hypotheses as for Unit **2** are applicable. Furthermore, only smooth transient phenomena are considered. The propagation of a surge cannot he simulated.

Two equations are needed to describe unsteady flow in open channels: the continuity equation and the momentum equation.

The continuity equation which accounts for the conservation of the mass of the water is expressed as (Ven Te Chow **1988):**

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q \qquad [8]$$

The momentum equation or dynamic equation is expressed as:

$$\frac{\partial Q}{\partial t} + \frac{\partial Q^2/A}{\partial x} + gA\frac{\partial z}{\partial x} = -gAS_f + kqV \qquad [9]$$

The partial differential equations must be completed by initial and boundary conditions in order to be solved. The boundary conditions are the hydrographs at the upstream nodes of the reaches and a rating curve at the downstream node of the model (because subcritical flow conditions prevail). The initial condition is the water surface profile resulting from the steady flow computation (Unit 2).

Implicit Discretization

Saint Venant's equations have no known analytical solutions in real geometry. They are solved numerically by discretizing the equations: the partial derivatives are replaced by finite differences. Various schemes may be used to provide solutions to these equations. The discretization chosen in the RBMC model is a four-point implicit scheme known as Preissmann's scheme (Cunge et al. 1990).

This scheme is implicit because the values of the variables at the unknown time step also appear in the expression containing spatial partial derivatives.

The double sweep method is then used to solve the linear system obtained when discretizing the Saint Venant's equations. The singularities and the offtakes have to be introduced in the double sweep process. More details on this method are given in Annex 2.

Performance indicators

Some performance indicators have been incorporated with a view to evaluating the water delivery efficiency at the offtakes. They allow the integration of the information on water delivery, either at a single offtake or at all the offtakes. There are two kinds of indicators: volume indicators and time indicators.

Volume Indicators: The volume indicators refer to three kinds of volumes:

* The demand volume (V!), which is the target volume at the offtakes.
* The supply volume (V_S), which is the volume supplied at the offtakes.
* The effective volume (V_{EF}), which is the really usable part of the supply volume.

The definition of the effective volume depends on two coefficients, W and X (in percentage). Only the supply discharge close to the water demand is taken into account (see Figure 11).

In this figure the effective volume is shaded. It can be defined by:

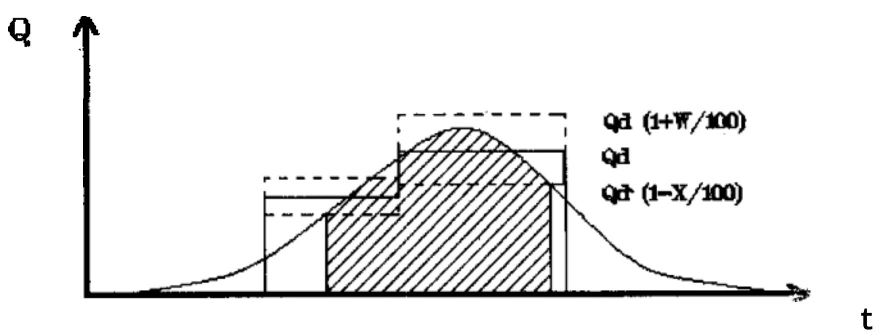
If $(1 - X/100).Q_D \leq Q_S \leq (1 + W/100).Q_D \Rightarrow Q_{EF} = Q_S$

If $Q_S < (1 - X/100).Q_D \Rightarrow Q_{EF} = b$
If $Q_S > (1 + W/100).Q_D \Rightarrow Q_{EF} = Q_D$

We define three volume indicators:
* Indicator $IND1 = V_S/V_D$
* Indicator $IND2 = V_{EF}/V_D$
* Indicator $IND3 = V_{EF}/V_S$

These indicators can be defined for a single offtake or for a set of offtakes.

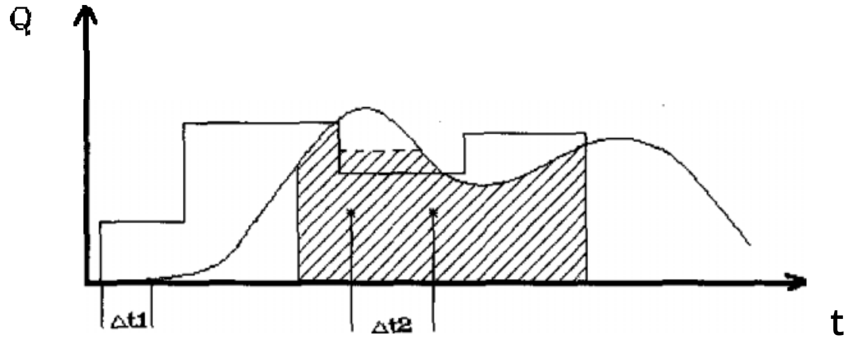
Figure 11. Definition of effective volume.



Time indicators: T_D is defined as the total period of time during which the demand discharge is non-zero and T_{EF} as the total period of time during which the effective discharge is non-zero. The time indicator $IND4 = T_{EF}/T_D$. It compares the duration of delivery of the effective volume with that of the demand volume. This indicator is dimensionless and can only he calculated for individual offtakes since it doesn't have any significance for all the offtakes taken together.

Two time lags, $\Delta T1$ and $\Delta T2$, can be detined. $\Delta T1$ is the time separating the start of the water demand and the start of the effective discharge. This time is positive if the effective discharge arrives after the demand discharge (cf. Figure 12). *AT2* is the time lag between the centres of gravity of the demand hydrograph and the effective delivery hydrograph.

Figure 12. Definition of time lag.



All these indicators are defined for each offtake. They can be calculated for any particular period of the simulation that the user wants to focus on.

MODEL INSTALLATION

The present model is limited to the first 25 km of the Kirindi Oya Right Bank Main Canal and the input data, both physical and hydraulic, are from this stretch of main canal. However, the model can easily accommodate eventual extensions by modification of its topography unit.

Application of the model requires site-specific data collection. These data are very important to accurately simulate the hydraulic behaviour of the canal. Furthermore, these data can be used to define the present performance of the system and the possible benefits of using the model.

Input Data Requirements

The model needs two categories of data: topographical and geometrical data and hydraulic data. The input data requirements for the steady and the unsteady state models are exactly the same:

Topographical and geometrical data: These were gathered in the course of a topographical survey, and included: (a) the locations and descriptions of all cross-regulators, offtakes and other singularities on the RBMC; (b) longitudinal profile of the canal bed; and (c) cross sections of the canal at appropriate intervals (100-meter intervals were used in the RBMC) to enable, as far as possible, the capture of all hydraulically significant features.

In order to have an accurate description of the canal geometry, it is important to:

* Have a precise topographical survey along the canal with reliable bench marks. These bench marks will be further used for the water level measurements.
* Choose the cross sections to be surveyed to correctly represent the water volumes (the cross sections inducing widenings or narrowings must be taken into account) and the changes of bed slope.

Therefore, it is not compulsory to choose equidistant cross sections but rather to adapt the distance between surveyed cross sections to the canal features.

Hydraulic ***dafa:*** The hydraulic information required includes: (a) roughness coefficients for the different reaches of the canal; (b) head-discharge relationships and discharge coefficients for the offtakes and regulators; and (c) seepage losses along the canal. Estimates of some of these parameters were obtained in the course of the measurement campaign which served to calibrate the model.

Calibration

A measurement campaign was carried out, over a 10- y period in April-May 1988, by a joint IIMI-CEMAGREF team (Sally et al. 1989) with the assistance of the Irrigation Department. In carrying out all these observations and measurements, a primary **concern** was to cause minimum disruption to normal irrigation activities in the RBMC project area.

The field measurement campaign was an essential step in the development and exploitation of the Kirindi Oya RBMC mathematical flow simulation model. In order that it yield reliable and useful results, the model should accurately reflect the physical and hydraulic features of the canal. The field measurements contribute to the matching of the model behaviour to actually observed situations. The staff of the irrigation agency would then he able to recognize the model **as** truly representing "their" canal.

Steady Flow Measurements

In this phase, an inventory of the status of the RBMC system under given steady **conditions** of canal water flow and gate settings was taken. The Irrigation Department had agreed not to alter the main canal discharge or the gate settings until the end of the calibration campaign. Marks were painted on the gate spindles so that it would be possible to ascertain at a glance if any of these gate settings had been altered.

Water surface profiles in the main canal were computed by measuring water levels at all ***offtakes,*** and upstream and downstream of each ***cross*** regulator (denoted GR2 to GR15) with respect to temporary bench marks (TBM) of known elevations established at these locations.

The discharges at different points in the RBMC as well as at some offtakes were estimated using an **OTT-C31** current meter. Gauging in the main canal was performed from 9 different bridges and at the heads of some canals taking off from the main canal.

The openings of **all** regulator and offtake gates were computed via observations of their respective spindle heights; the relations between gate openings and spindle heights had been established earlier for each gate.

Unsteady Flow Measurements

The main purpose of this operation was to monitor the propagation along the RBMC of a wave generated by a sudden additional discharge at the head of the canal. The magnitude of this additional discharge was determined by trial runs of the simulation model (both steady and unsteady states) on the IBM-PC/AT microcomputer of the Kirindi Oya Irrigation and Settlement Project. The initial conditions for the unsteady flow simulations corresponded to the state of the system (water surface elevations, discharges, etc.) observed during the steady flow measurement.

The choice of the magnitude and duration of the additional release to be made at the headworks was a compromise between: (a) considerations of safety which required that the flow should not be so great that the RBMC would overflow its banks at some point; and (b) the need to generate a wave that would not attenuate too soon, thereby making it difficult to monitor its arrival and progress, especially towards the tail of the canal. It was finally decided that an extra release of about 1.5 m^3/s over 3 hours would be suitable.

At 06:30H on 2 May 1988, this additional release was made at the Lunuganwehera Reservoir headworks. Water level variations were recorded every 10 minutes upstream and downstream of the side check wall of every cross-regulator. These tasks were performed by 15 students from a local school each of whom was equipped with a watch, ruler, and record hook.

The variations in water levels recorded by the automatic data loggers at the cross-regulators GR3 (4,012 m) and GR12 (19,860 m) in response to this additional release are illustrated in Figures 13 and 14. The plot indicates that the wave first arrived at the GR3 location around 07:00H (or half an hour after it was released at the headworks), and that the peak arrived at about 0940H.

Figure 13. Water levels at GR3 on 2 May 1988 (water was released from the dam at 06:30H).

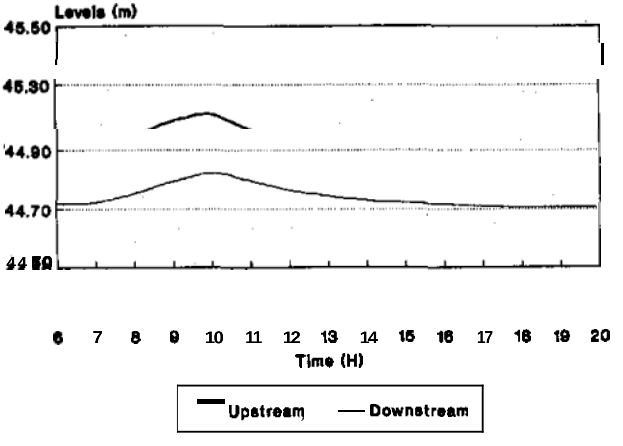
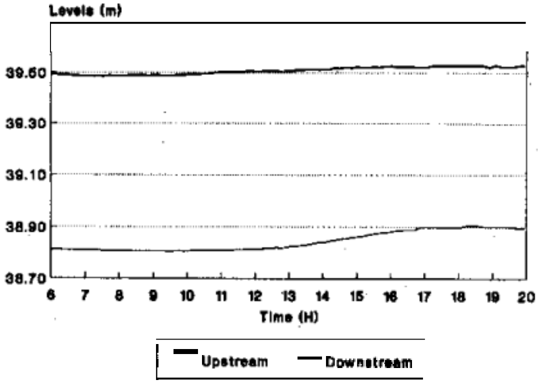


Figure 14. Water levels at GR12 on 2 May 1988 (water was released from the dam at 06:30H).



At the same time, the RBMC discharge was gauged from time to time at the first bridge, Brl (1,493 m, cf. Table 1). The gauging results were as follows:

Table 1. RBMC discharge measurements during unsteady flow calibration phase.

Time (H)	Discharge (litres/s)
Initial Value	4,607 (Steady flow value)
07:23	5.831
0745	5,765
09:15	6,159 (New steady flow regime)

The main sluice was returned to its original position at 09:40H. The supplementary discharge measured was 1.552 m^3/s.

The water level observations at the cross-regulators continued until the new steady flow regime was established at each location. This occurred progressively at each regulator, upstream to downstream.

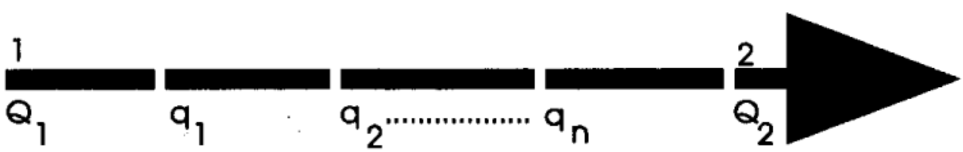
Interpretation of the observations was however rendered difficult by the cleaning of the protective grill at the upstream end of the siphon located at a distance of approximately 7 km. Weeds and other debris had been deposited against this grill overnight, causing an accumulation of water in the canal reaches upstream of this location. Their removal (which took place between 07:30H and 08:30H) provoked a sudden release of this stored water resulting in the propagation of another positive wave downstream of the siphon (and possibly a negative wave upstream of it).

The average velocity of the main wave propagation was around 3 km/h (1.9 miles/h), whereas the peak of the wave was propagated at a velocity of 1.8 km/h (1.1 miles/h). Such values are useful for design purposes and for estimation of response times.

Canal Losses

The discharge values obtained by current metering were also used to compute seepage and percolation losses in each gauged reach of the RBMC by an inflow-outflow method. It was assumed that the losses are uniformly distributed over the entire reach and the flow conditions are steady. Wherever the offtake has been gauged, the measured discharges were taken into account in the computation. Otherwise it was assumed that the targeted discharge is being delivered at the offtake.

Consider a typical portion of the canal, 1–2, with n offtakes:



$$Loss = \frac{(Q1 - q1 - q2 - \ldots - qn - Q2)}{x2 - x1}$$

Where:

Q1, Q2	= Discharge at upstream and downstream ends of reach 1–2
x1, x2	= Relative distances of upstream and downstream ends
q1, ... qn	= Discharge at offtakes 1, ... n

The losses were computed according to the above method for the different canal reaches bounded by the bridges where gauging was performed. The results for all reaches, except the last two, are given in Table 2. In reach Br8 – Br9, the sum of the outflows is greater than the inflow, possibly indicating that the target discharge in DC11 (distributary channel No. 11) is not achieved. On the other hand, unusually high losses seem to occur in reach Br9, which could mean that the actual discharge in DC12 exceeds the target value.

Table 2. Canal losses.

Bridge	Q (l/s)	Relative Distance (m)	Discharge (l/s)	Length (m)	Mean loss (l/s/m)
Brl	4,607	1,493	0	3,551	0.023
Br2	4,526	5,044	DC1: 110	3,762	0.031
Br3	4,301	8,806	DC2 368 DC3 108 FC34: 21	3,286	0.067
Br4	3,583	12,092	DC4: 59 DC5: 131 DC6: 292	2,693	0.059
Br5	2,529	14,785	DC7 120 FC68: 21 DC8: 57 DC9 104	3,059	0.036
Br6	2,529	17,844	DC1B: 62 DC1: 41 DC1A: 55	1,958	0.107
Br7	2,156	19,802	BC2: 1223 FC48: 15 FC49: 31	1,876	0.016
Br8	857	21,678	DC9 100 FC54A: 31 FC54: 10 FC55: 5 DC11: 264	2,542	Aberrant DC11; targeted discharge might not be achieved,
B19	501	24,220	DC12: 62 DC13: 190	310	Aberrant DC12; targeted discharge might be exceeded.

It will be observed that there is a wide range of variation in the losses obtained for the different reaches. This reflects the variation in construction of the different canal sections, depending on whether it is built entirely below the natural terrain ("cut"), entirely above the natural terrain ("fill"), or partly in "cut" and partly in "fill." The losses would be least in the first situation and greatest in the second.

The weighted mean value for RBMC losses, taking into account the values obtained between Brl to Br8, is 0.043 l/s/m (or 2.44 cusecs/mile).

This is equivalent to a loss of 1.13 m^3/s (or 40 cnsecs) over the 25 km of canal from the headworks to the cross-regulator GR15. This also corresponds to a loss of approximately 25 percent with respect to the discharge of 4.607 m^3/s measured at Bridge 1.

Although the loss values obtained at this stage were only approximate, they, nevertheless, give some indication of canal losses in a situation where hardly any information was available previously. The approximate nature of these results is due to the uncertain offtake discharges (ql, q2, ... etc., were not directly measured at all the offtakes). In the computation of losses, it was assumed that the ungauged offtakes were delivering flows equal to their respective targets. This is perhaps not always true, as evidenced by the unusual results obtained in reaches Br8–Br9 and Br9. More reliable estimates of canal losses would have been obtained if discharge measurements had been performed at all the offtakes, or if all the offtakes had been closed. It is advisable to carry out measurements under the latter conditions (i.e., flow only in the main canal with all offtakes closed).

Estimation of Roughness Coefficient

The roughness coefficient is an indicator of the resistance offered by the canal to the flow of water. It could display both spatial and temporal variations since the canal condition itself could vary at different points of the canal and could evolve over time (e.g., weed growth). The roughness coefficient (in the form of the Strickler coefficient) is an essential model parameter used in the computation of the friction gradient.

The standard Manning-Strickler equation for open channel flow can be represented as:

$$Q = K.A.R.^{2/3}.S_o^{1/2} \qquad [10]$$

S_o = 0.0003 (medium bed slope)

This equation is valid only for uniform flow, which does not usually prevail in main irrigation canals. This is due to the backwater effects caused by downstream regulating structures. In fact, in the Kirindi Oya RBMC, most regulators are located in the backwater curve of the regulator immediately downstream.

In the field, however, for the purpose of carrying out immediate unsteady flow simulations, the Manning-Stnckler equation was used to obtain friction coefficient values at the gauged sections. Only the result? obtained at the first four bridges (Brl to Br4) were conserved as the other bridges were obviously influenced by downstream regulators at the time of measurement. The results are indicated in Table 3.

Table 3. Estimation of roughness coefficients.

Location	Q(l/s)	A(m^2)	$R^{2/3}$	Strickler K	Manning n
Br1	4607	12.75	0.968	22.0	0.045
Br2	4526	11.28	0.913	25.4	0.039
Br3	4301	11.50	0.945	**22.9**	0.044
Br4	3583	10.91	0.913	20.8	0.048

The above values of roughness coefficients were only preliminary estimates used to plan the field measurements under unsteady flow conditions (see page 27).

The final values to he adopted in the simulation model were obtained with the completion of the calibration computations. This invulvcd adjusting the value of the roughness coefficient for the different reaches by a manual iterative method until there was reasonable agreement between the computed and observed water surface elevations, at the same time, ensuring that there was conservation of the volumes **of** water being conveyed in the different canal reaches.

The final Strickler roughness coefficients obtained are between 25 and 35. These are less than the value of 40 assumed at the design stage. This implies that the canal roughness is higher than what was originally assumed., leading to **a** proportional reduction in canal carrying capacity.

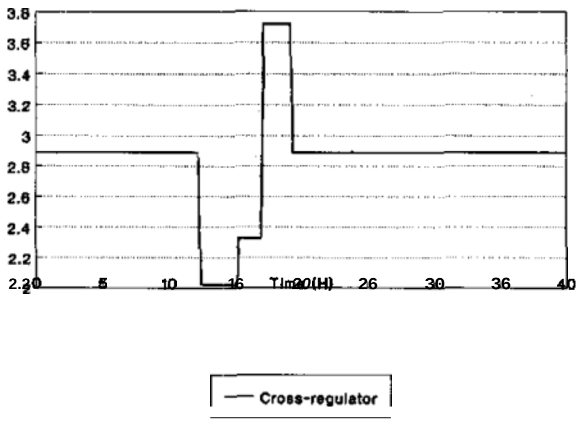
Calibration of Cross-Regulator

The calibration was carried out **at** cross-regulator GR3, where **IIMI** had installed automatic data-logging equipment to continuously monitor water levels in the main canal and in the nearby DC5 distributary canal. The object of the calibration was to determine an appropriate coefficient of discharge for the regulator gates. The value obtained will be considered to he representative **for all** the regulators.

GR3 is **a** 5-bay regulator. For **a** given combination of gate settings, measurements of water levels upstream and downstream of the cross-regulator were made with respect to the top of the side check walls (corresponding to the **Full** Supply Depth [FSD]). The spindle heights from which the relevant gate openings were derived were **also** noted. Adjustments made to gate settings were such that, as far as possible, the same opening was maintained at each of them, It was, however, not possihlc to make adjustments to one of the gates which remained blocked.

This particular experiment was performed on ***26*** April 1988 when the main canal discharge **was** 4.475 m^3/s. The different gate openings cffccted that day, expressed in terms of area **of** opening, **are** shown in Figure 15.

Figure 15. GR3: Gate openings, 00:00H on 26-04-88 to 16:00H on 27-04-88.



Since all offtakes upstream of this regulator were closed during the measurement campaign, the main canal discharge was not affected by changes to gate settings. Measurements corresponding to five different gate settings were made for this value of discharge.

Measurements were made only after allowing time for the upstream water level to regain stability following a gate adjustment. Although over two hours had elapsed between successive sets of gate adjustments, examination of the water levels recorded by the data logger upstream and downstream of the regulator **GR3** that day (Figure 16) indicates that complete stability had, in fact, not been attained at the end of each set of adjustments. **(This** also demonstrates the wealth of useful information that can be obtained from the continuous data-logger records.)

Figure 16. GR3: Levels upstream and downstream, 00:00H on 26-04-88 to 16:00H on 27-04-88.

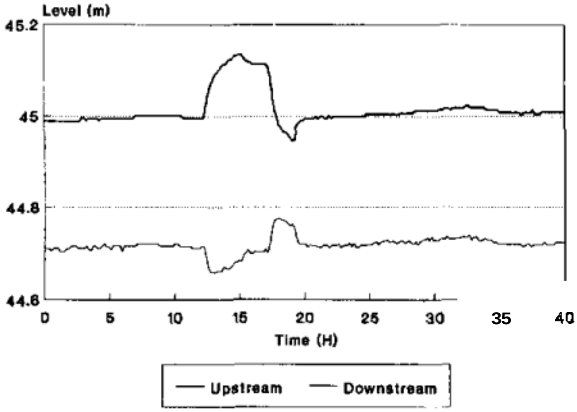


Figure 17. GR3: Total head and head over sidewalls, 00:00H on 26-04-88 to 16:00H on 27-04-88.

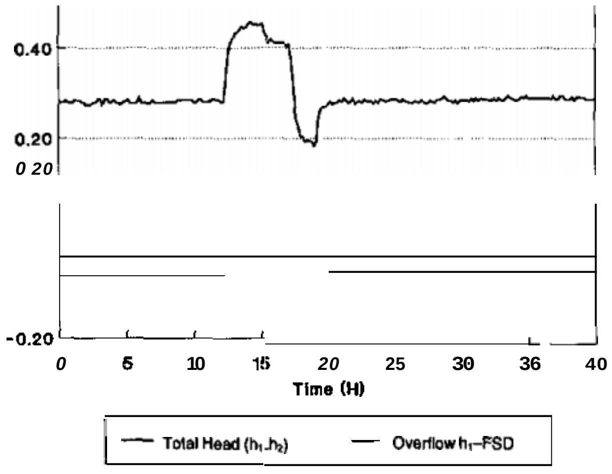
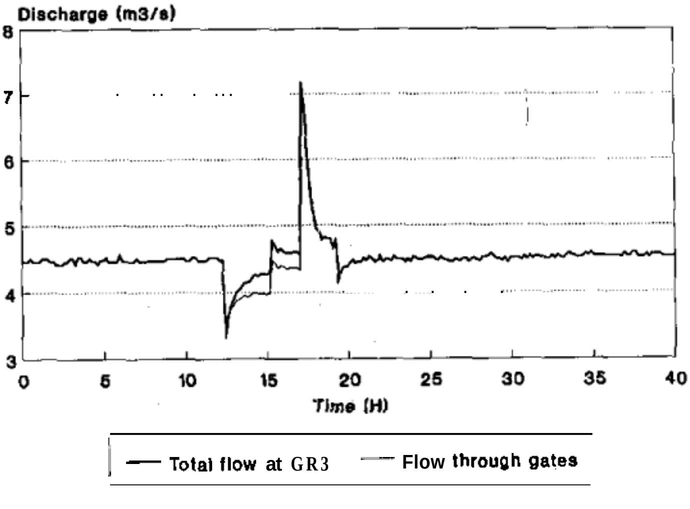


Figure 18. GR3: Flow, 00:00H on 26-04-88 to 16WH on 27-04-88.



The flow actually passing through the regulator gate openings was estimated by subtracting the flow over the side check walls and over any of the gates themselves (wherever applicable) from the main canal discharge value (Figures 17 and **18).** The classic equation for free flow over a weir was used with a discharge coefficient **of** 0.40 to compute these overflows:

$$Q_w = 0.40L\,(2g)^{1/2} h_1^{3/2} \quad [11]$$

where:

Q_w = Discharge over the weir (side check walls or gates themselves in this case)

The flow through the cross-regulator gates, denoted by Q_g, is then given by:

$$Q_g = Q_o - Q_w \quad [12]$$

where Q_o is the observed main canal discharge at the time of the experiment.

The following equation was used for the discharge through the cross-regulator gates:

$$Q_g = C_s.A.[2g\,(h_1 - h_2)]^{1/2} \quad [13]$$

where:

Q_g = Discharge through the regulator gates
A = Total area of flow through gates

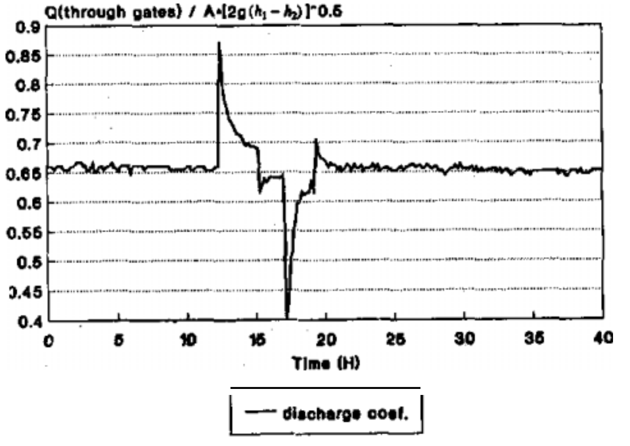
All quantities in equation [13] are known (or measured) except for the coefficient of discharge C_s, which can thus be calculated.

The values of C_s obtained at the end of each set of gate adjustments are as follows:

Test **1, C_s = 0.659;**
Test **2, C_s = 0.695;**
Test **3,** C_s = 0.648;
Test **4, C_s = 0.620;**
Test **5, C_s = 0.657;**

The range of different values obtained is perhaps due to the fact that fully stable conditions were not prevalent at the time of measurement. Figure **19** shows the different values of C_s obtained for the whole period from 0000H on **26** April to **16:00H** on **27** April. It would appear that the most persistent value of C_s for this period is around **0.66.**

Figure 19. GR3 of Kirindi Oya RBMC Coefficient of Discharge, 00:00H on 26-04-88 to 16:00H on 27-04-88.



Calibration of Offikes

If the offtakes are simulated by an imposed discharge, there is no need to calibrate the offtake. But to compute the offtake gate opening for a given discharge, or having input the gate opening law to compute the discharge through the SIRENE program, a good knowledge of the hydraulic coefficient of each offtake is needed. To calibrate the coefficient, the same procedure as for the cross-regulator is used. The water level upstream and downstream of the offtake for a given steady discharge on the distributary canal is recorded and the measurement of the gate opening used to compute the discharge coefficient. But, to accurately model the offtake, the downstream law is necessary in order to account for the influence of the distributary canal on the offtake submergence. If the downstream law is a stagedischarge relationship, the discharge and the downstream level for different discharges should be measured. This should be done for all the offtakes. But, for the discharge coefficient, if the offtakes are of standard dimensions, it is sufficient to calibrate one of each size.

Final Calibration Procedure

The final calibration procedure involves progressively adjusting the Manning-Strickler roughness wefficient until there is an acceptable fit between the observed and the model-generated steady-state water surface profiles. The simulation starts at the downstream end of the model and the results of the model are compared with the field-ohserved values. The roughness coefficient is changed until the computed depth reaches the observed one from tail end to head end. If it is a singular section, the discharge coefficient is adjusted to match the actual difference of water levels upstream and downstream of the cross regulator.

If the result is some unexpected coefficients, then the model topography or the computational step has to be verified to find out the reason for this distortion.

In the case **of** the **RBMC,** a good fit was obtained for the steady flow water profile **of** the downstream part of the canal. The results were not as good for the upstream part. Subsequent field investigations revealed that this was due to topographical problems (cf. Figures ***20*** and 21). The final ***set*** of Strickler coefficients ranged from 25 to 30 in the downstream part **of** the canal and from *25* to **35** in the upstream part.

Figure 20. Model calibration: Computed and measured water surface profiles.

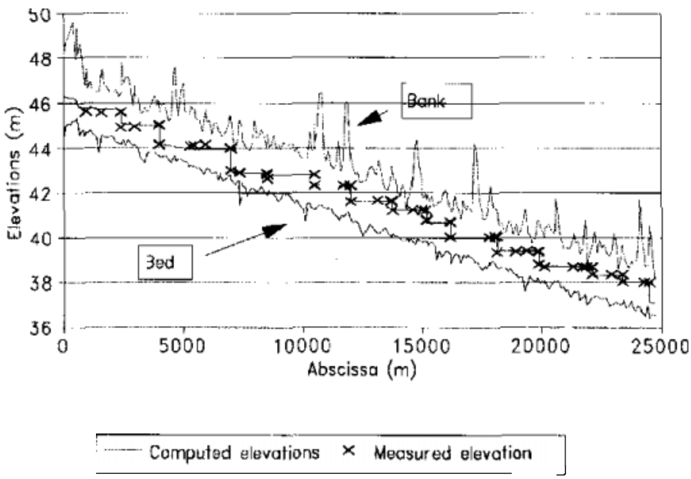
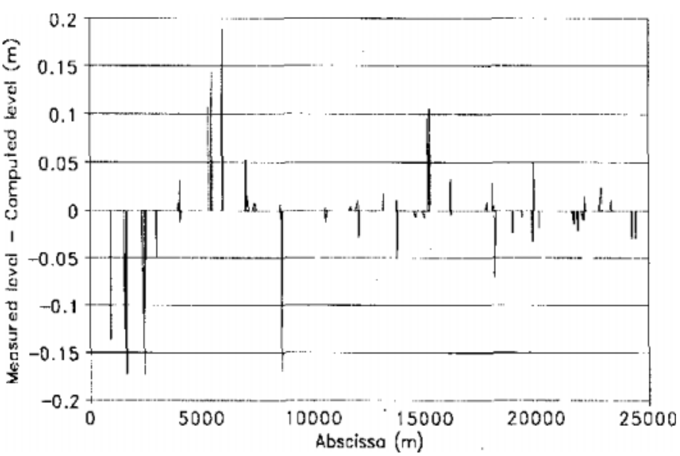


Figure 21. Model calibration: Differences between computed and measured water surface profiles.



CHAPTER 3

First Applications

THIS CHAPTER **PRESENTS** the first results of using the Kiriidi Oya RBMC model to formulate appropriate responses to some typical canal management problems, identified in consultation with the canal managers themselves. However, the results should be considered indicative at this stage, in the sense that they have yet to be evaluated in the field.

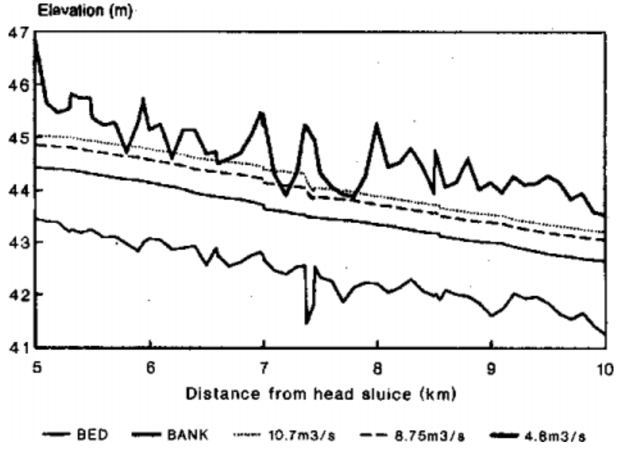
STUDY OF CANAL DESIGN AND HYDRAULIC PARAMETERS

Determining Maximum Carrying Capacity of the Main Canal

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

Figure 22 shows the water surface profiles between the 5-km and 10-km 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.

Figure 22. Water surface profiles.



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 focused on.

The absolute carrying capacity can also be computed for each reach of the main canal, with the cross-regulators fully opened and all the offtakes fully closed. The results are shown in Table 4.

Table 4. Absolute carrying capacities of reaches.

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	10.7
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

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

For example, the water requirement computations done by the Irrigation Department assume a peak water requirement of 2.7% l/s/ha. Under these conditions, a maximum discharge of 9.25 m^3/s can be released at the main sluice without causing overtopping of the canal banks 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 a water requirement of only 2 l/s/ha at the head of each main canal offtake had to be satisfied, the maximum possible main sluice discharge is found to be 8.75 m^3/s. After satisfying the discharge requirements at the offtakes and compensating for seepage losses, 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 (about 900 ha).

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). 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 were:

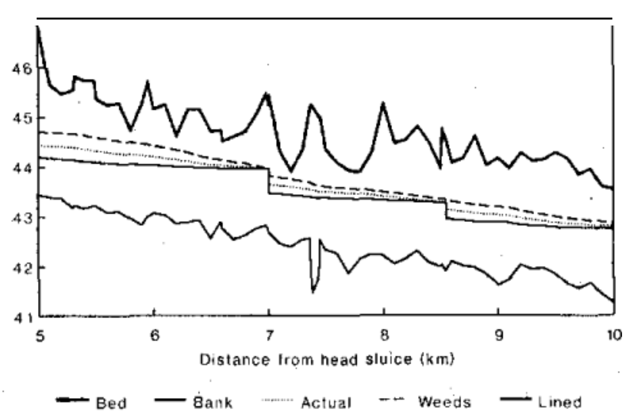
1. Tracts 1,2 and *5* under irrigation with a head sluice discharge of ***6*** m^3/s.
2. Cross-regulators set inadjustablemode to maintain FSD immediately upstream (this is the usual operating conditiou, irrespective of the physical condition **of** the canal).

Weed growth in the canal (increased roughness) was simulated by decreasing the Strickler coefficient value of each of the canal reaches 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 (the Irrigation Department estimates a Manning's coefficient **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 **23** shows the variation of water surface elevation in the 5km-10km reach **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.

Figure 23. Variation of water surface elevation.



The offtake discharges are the same for the three cases. The 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 to 10.15m^3/s for average cement mortar lining (an increase of 1.4 m^3/s with respect to the maximum permissible head sluice discharge of 8.75 m^3/s obtained for an offtake discharge scenario of 2 l/s/ha). A discharge of 4.25 m^3/s becomes available at the tail end.

For the weed-infested canal, the maximum permissible head sluice discharge falls to **as** low **as** 7.5 m^3/s.

The importance of canal maintenance and its impact on canal carrying capacity is thus demonstrated. The use of the simulation model can be extended to include scheduling of maintenance activities and identification of bottlenecks to canal carrying capacity.

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, takmg 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.

BETTER UNDERSTANDING OF CANAL BEHAVIOR

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 and studied with the help of the unsteady flow unit of the model.

Forexample, 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 instinctive operational response would be to increase the main sluice discharge by this amount.

If none of the intervening devices are operated (the openings of the cross-regulators and offtakes being maintained at their previous steady state values) 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 (Table 5). This is because the head-end offtakes 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.

Table 5. Impact of cross-regulator operation on the conveyance of water to the tail end of the RBMC.

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

If the devices are to be operated to accommodate the increased main sluice discharge so **that the magnitude and duration of 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.

The times at which the cross-regulators should be operated are shown in Table 6. There was no need (with one exception) to operate the offtake gates because operation of the regulators alone, which modified the main canal water levels, was sufficient to maintain discharges through the offtakes at their desired values.

All operations can be completed in 5 hours and hence within a normal working day. Furthermore, by executing these operations, the variation in offtake discharge during this period is kept within reasonable limits, as evidenced by the value of the performance indicator, *IND1* (defined as the ratio between the volume delivered and the target volume) for the offtakes which remains between 0.95 and 1.12.

Tat Times at which cross-regulators should be operated.

Cross-regulator	Distance from head sluice (m)	Time of operation (after head sluice) (hours:minutes)
GR2	2,415	00:30
GR3	4,012	01:00
GR4	7,007	01:50
GR5	8,550	0200
GR6	10,532	02:20
GR7	12,029	02:40
GR8	13,732	0250
GRY	15,137	03:00
GR10	16,166	03:10
GR11	18,112	03:30
GR12	19,860	0400
GR13	22,110	04:30
GR14	23,342	0450
GR15	24,481	04:50

Evaluating Impact of Interventions at Nearby Gates on Offtake Discharge

Thii problem will be briefly illustrated using the distributary canal DC5 and cross regulator GR3 of Tract 1.For example, what would be the impact on the discharge in DC5 of a sudden gate opening at regulator GR3, located immediately downstream?

Suppose that the main canal discharge at GR3 is 2 m^3/s and that one of the gates of regulator GR3 is fully opened at 06:00H, the main canal level immediately begins to fall and the discharge in DC5 shows a corresponding decrease from the initial steady state value of 142l/s. Three hours later (at 09:00H), the discharge is zero (Figures 24 and 25).

The shortfall of discharge with respect to the steady state value of 142l/s in distributary canal DC5 will persist as long as remedial action (i.e., reducing the gate opening at regulator GR3) is not taken.

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

Figure 24. Variations in main canal water level upstream of regulator GR3.

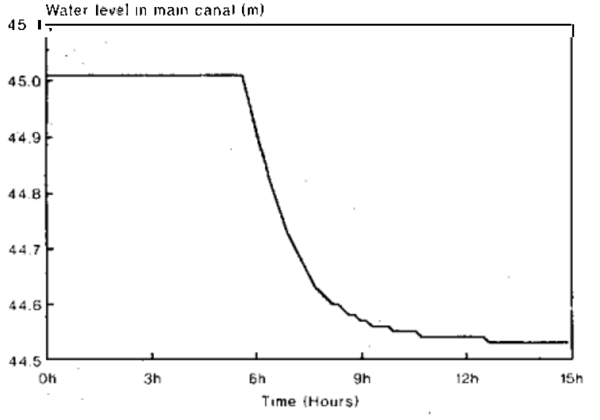
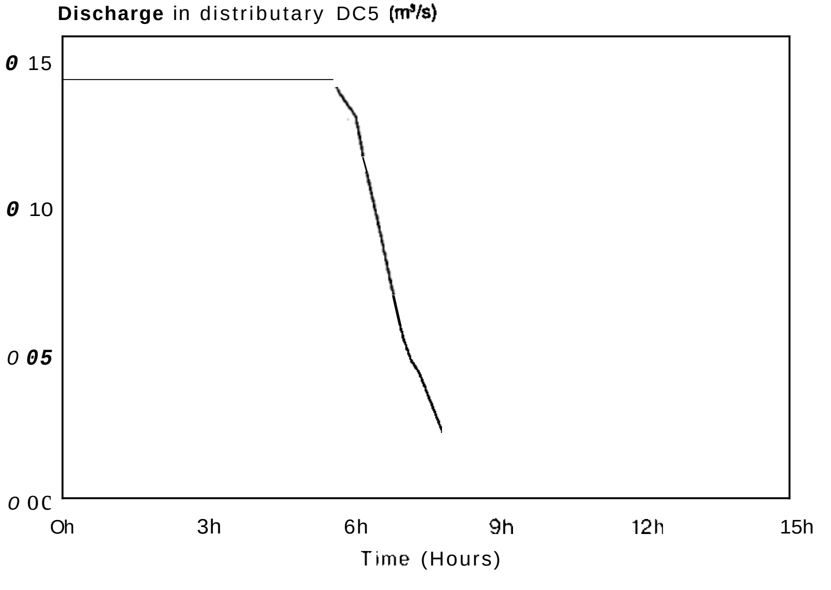


Figure 25. Variations in discharge of distributary canal DC5 following gate opening at regulator GR3.



TOWARDS AN OPERATIONAL USE

One possible application of the RBMC Mathematical Flow Simulation Model is to simulate a wide range of canal operational practices to analyze prospects for improving these operational practices (Malaterre 1989 and Rey 1990). These simulations can be performed as the architecture of the model has been designed to allow FORTRAN programming *so* that any given operational rule can be written into it in order to test it. These rules are stored in a small independent source file that can be compiled and linked to the Unsteady Flow Simulation Model. This linked module will produce, at any computation time step, any desired information along the canal (levels, discharges, openings) and compute, according to the rules to be tested, the new gate openings at the cross-regulators and (if desired) the new main sluice discharge.

But, the first step in undertaking such studies is to choose the rules to be tested. These rules can be very different from each other; for example, rules for a daily manual operation using some elements of information on the canal, or rules for a real time regulation computing the new gate openings of the cross-regulators at every time step, using information of past canal hydraulic states all over the system and predictions of the future offtake targets.

The term "improvement" means that some criteria are used to evaluate the tested operational practices. These criteria are also linked to the selected scenario. In some cases, it can be the total volume of water flowing out at the tail end the canal, or the duration before stabilization from one steady state to another steady state, or the amplitude of the water surface fluctuations at the offtakes (which influences the water delivery), or the difference between the water supplied at the offtakes and the corresponding targets (decided by the canal managers), etc.

The methodological approach illustrated below is based on the following steps:

1. Try to understand the present Operational practices.
2. Write them in the ad-hoc module of the model.
3. Test and evaluate the present rules being used for managing typical phases of canal functioning.
4. Propose, test and evaluate alternative operational practices.

Analysis of the Present Manual Canal Operations

A. Present practices

The first step is to understand the operational practices being used for the management of the canal. IIMI analyzed past reports and studies and collected data through data loggers during 1988.

Monitoring done by IIMI during the yala (or dry) season in 1988 gave an idea of the number and frequency of operations as well as the magnitude of oscillations of the water surface in the main canal. Complete information was gathered during other field visits in subsequent seasons. IIMI specially focused on the method used to evaluate the magnitude of the operation at the regulators in order to be able to simulate it on the mathematical model. This information was gathered from discussions with the operators and direct measurements at the regulators.

The analysis of this information is quite simple as far as timeliness, frequencies and duration of operations are concerned. The same remark holds good for the determination of the threshold of intervention. 'But, it is much more difficult to understand how the gate operators evaluate the magnitude of the operation. The method they use is mainly the outcome of three years of management of the same regulator. Indeed, most of the operators have been in charge of the same offtakes and regulator since the beginning of the project. Therefore, when operating a given regulator, they refer not only to the upstream level but also to some intuitive knowledge they have of the flow at that time and location.

As far as the timeliness is concerned, the basic rules are:

* An operator visits his regulator every 3 or 4 hours.
* If the water level upstream of this regulator is within 1 or 2 cm of FSD, he just waits 10 or 20 minutes and leaves. He can spend this time checking the level at a nearby offtake.
* When he has to operate, if the magnitude is sufficient, he will move 2, 3 or 4 gates.
* Then he will check the effect of his operation for about 1-1.5 hours. If necessary, he can make a correction to his first operation (if too strong or too weak, or in case of discharge modification).

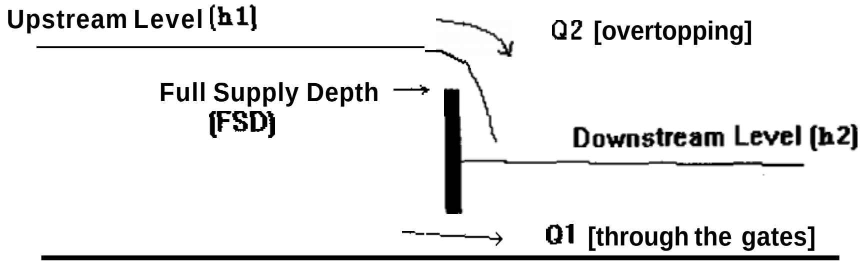
Field observations reveal that all operations are performed only during the day time (from 07:00H to 17:00H).

But the operational practice most difficult to model is the detennination (by each operator) of the new gate openings to be applied at a cross-regulator. What intuitive algorithm does he use to propose a new set of gate openings based on the available information on the hydraulic conditions at the regulator (upstream and downstream levels, present openings, knowledge of the ongoing transition, etc.)?

Given the good quality of the operations at the first regulators, the observed operations can be compared with the values obtained through a simple hydraulics calculation:

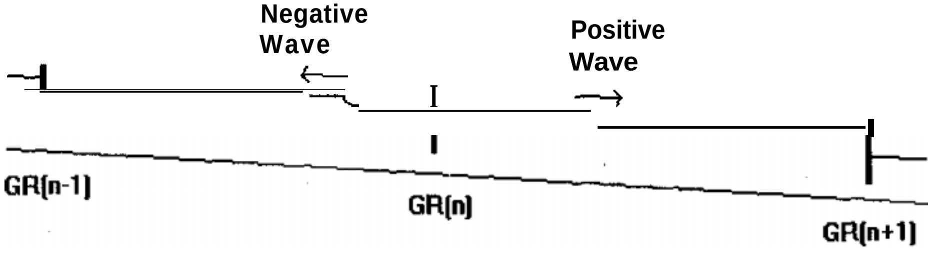
Consider the regulator GR(n) (Figure 26):

Figure 26.



When visiting the regulator, if the upstream level exceeds the FSD by more than 1 cm or 2 cm, the operator will open some gates to adjust this level. This operation will generate a positive wave downstream and a negative wave upstream (Figure 27).

Figure 27.



After a while (the period could be quite long — up to 10 hours), these waves should disappear (if no operation is performed elsewhere along the canal) and a new steady flow regime will be reached. If, at the next regulator GR(n+1), a proper operation has been made, the level will be close to FSD.

Therefore, taking into account the fact that the slopes are very low and assuming that the discharge is not very different, it can be assumed that after stabilization, the level downstream of GR(n) is about the same as before the operation.

With these hypotheses, the new opening that allows the attainment of FSD upstream of GR(n) can be computed. In the above sketch the discharge $Q1 + Q2$ has to pass entirely through the gates. Therefore using a ' mark for the new values:

$$Q'2 = 0$$

and

$$Q'1 = Q1+Q2 = Cg.A.\sqrt{2g.}\ \sqrt{(h1-h2)} + \mu_F L.\sqrt{2g.}(h1-FSD)^{1.5}$$
$$= Cg.A'.\sqrt{2g.}\ \sqrt{(FSD-h2)}$$

where:

A	is *the total gate opening area before operation.*
A'	is *the total gate opening area after operation.*
Cg	is *the discharge coefficient through the gates.*
μ_F	is *the discharge coefficient over the sidewalls.*
L	is *the width of the sidewalls.*

This formula is written in the case of overtopping over the sidewalls. If the upstream level is lower than FSD, there is no overtopping and $Q2 = 0$. But the same formula can be used with L = 0.

From this expression, the new total opening area A' and, therefore the new gate openings can be computed.

$$A' = F(A, h1, h2) \qquad [14]$$

As written above, this value supposes the validity of some hypotheses, but it allows the comparison of different operations. To a certain extent, it is a method giving a dimensionless value of operation (R coefficient presented hereafter).

For each operation monitored during the field visit, it is considered that the original state of the regulator corresponds to a total opening area A. After the operation, let the new opening area be $A1'$. This observed opening can be compared to $A2'$, given by the formula [14], by defining:

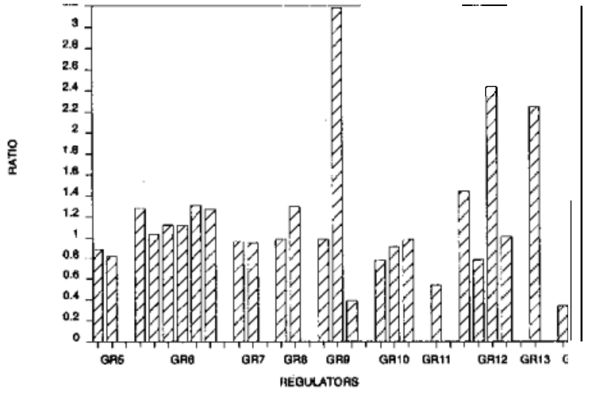
$R = A1'/A2'$ for an opening operation
$R = A2'/A1'$ for a closing operation

Therefore, $R > 1$ means that the operator has a tendency to overestimate the magnitude of operation compared to the formula. On the contrary, $R < 1$ means that he has a tendency to underestimate the magnitude of his operation.

In the following Figure 28, the values of this R coefficient for 26 recorded operations on 10 regulators are given. This is not enough to permit a complete study of the operational practices. But, nevertheless, some interesting elements come to light:

- * The coefficients are reasonably stable for the regulators **GR5, GR6, GR7,** GR8 and GR10 and are close to **1.**
- * For these regulators, the R coefficient is either always above **1** or always below **1.** This indicates that each operator has his own method of estimating the operation to be performed. But for a given operator, this method is consistent.
- * The *R* coefficients are much less stable for GR9, and GR11 to **GR14.**

Figure 28. Ratio between magnitudes of real and expected openings



An explanation for these important fluctuations may be:

a) The flow conditions in the lower reaches of the main canal were very unsteady and disturbed. Therefore, one hypothesis made to write the above formula is not valid and steady conditions could not be reached easily.

b) In this portion of the canal, the flow is often unsteady and therefore it is very difficult to establish consistent operational rules at the regulators.

c) In the upper reaches of the canal, the discharge is sufficiently high so that the water depth is close to the uniform depth. On the other hand, in the lower reaches of the canal, low flow results in a nearly horizontal water surface. In this case, the water level upstream of a regulator could have a significant influence on the preceding regulator. In other words, the regulators in the lower reaches of the canal are strongly connected (hydraulically) and influenced by each other.

In any case, the above figure confirms the field observation that the perturbations are very frequent towards the tail of the system.

This study demonstrates the general operational principles of the regulators. It underlines the different conditions prevailing at the head and at the tail end of the system. But, so far, it has been difficult to assert whether the pertnrbations at the tail end of the canal are inherent to the system or are mainly the consequences of suboptimal operational practices.

Field experimentation is necessary to evaluate different operational practices. But, it is not easy to do it because of the size of the system and the difficulties in checking the initial state of the canal, evaluating the external perturbations, etc. Moreover, it would need many instruments, operators and gaugings.

Simulation through a mathematical model of the canal is a good substitute to field experiments. With such a model, the initial state of the system, the type of perturbations, and the operations at the regulators can he defined and kept the same for the various tests so that the impact of a single variable parameter can be assessed.

Simulating the present operational practices.

The simulated operations of the cross-regulators must be as close as possible to the ones presently performed by the operators. They cannot be identical. On the other hand, because of manual operations, the real operations cannot be easily duplicated, even for exactly the same scenario. The important principle is to simulate the average timeliness, criteria and magnitude of operation.

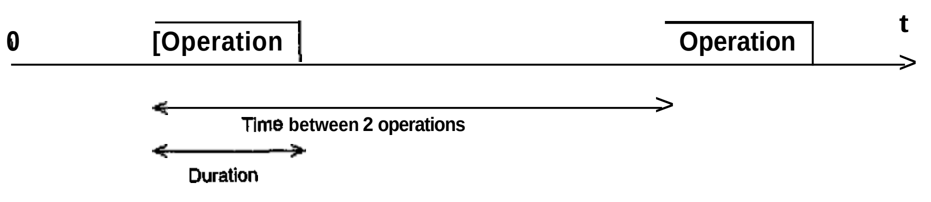
The study and development of this module simulating the present management practices of the regulators were time-consuming. This module had to be modified many times in order to match the complexity of the methods used by the operators in real life.

For each cross-regulator, the following parameters have been considered:

* Time between two operations.
* Duration of an operation.

They define the timeliness of the operations (Figure 29).

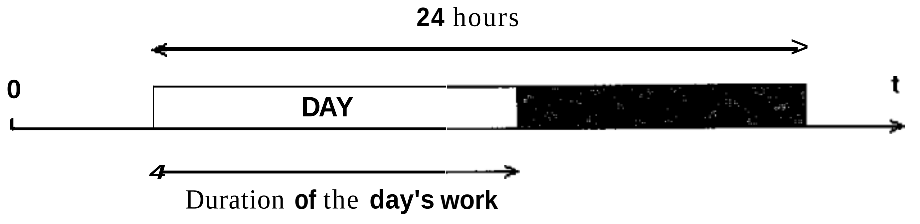
Figure 29.



In order to simulate the night time when operators usually do not work, the following parameters were used (Figure 30):

* The time at which work begins.
* The duration of the day's work.

Figure *30.*



As far as the operation itself is concerned, other parameters are defined:

* Lower threshold (FSD - ε).
* Upper threshold (FSD + ε).
* Maximum opening or closing during one operation.
* Minimum opening or closing during one operation.
* General coefficient of amplification of the magnitude of operations

The above parameters can be easily modified since they are read (by the unsteady flow model) in a text file. The choice of these parameters is the result of the field observations and successive developments of the module.

Some other additional rules have also been introduced into the module:

* If the level upstream of the regulator is within the authorized range, the operator checks it for 20 minutes and leaves if it is still alright.
* In the evening, an operator finishes his last operation before leaving even if the working day is over.
* If needed, an operator can make one correction to his first operation (if the water level is outside the range).
* In all of the following simulations, the gates are moved simultaneously and by the same amount. This is done to simplify the module but can be changed if necessary. Therefore, the opening indicated on the simulation figures is the same for all the gates of a given regulator.

Improving Present Operational Practices

Example 1: Increase of main sluice discharge.

PRESENT OPERATION& PRACTICES.

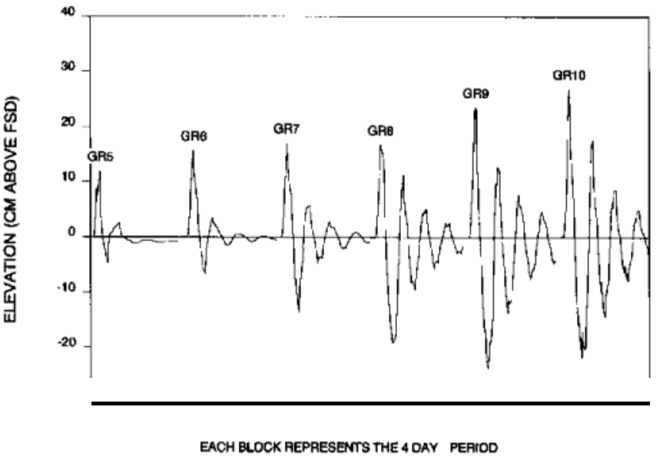
During a particular field visit, the main sluice discharge was increased from 130 cusecs (3.68 m^3/s) to 170cusecs (4.80 m^3/s). Two days after this operation, the canal conditions were still not stable. It is likely that it took one or *two* extra days to reach a new steady state. The scenario simulated in this section is based on this real situation observed in the field.

The main sluice discharge in the initial steady state of the canal is 130 **cusecs (3.68 m^3/s).** The discharges at the offtakes are the same as during the field visit. The water level is at FSD upstream of each regulator.

The parameters selected for the regulation module are:

* 1 operation every 4 hours.
* The operations are performed day and night.
* The operator stays 1 hour and 20 minutes for each operation.
* The thresholds of operation are -2 cm and +2 cm.

Figure 31. Water surface elevation upstream of the regulators, GR5 to GR10.



The result of the simulation shows that a new steady state is reached after **4** days of operation (cf. Figures **3 1 to 34). In** the upper reaches of the canal, this state is reached quickly (in 1 to 2 days) and perturbations are low (10 to 20 cm). But in the lower reaches, the magnitude of the perturbations goes up to **+40** or **-60 cm** and the time required for stabilization is up to **4** days. These results are close to the field observations.

Figure 32. Water surface elevation upstream of the regulators, GR11 to GR15.

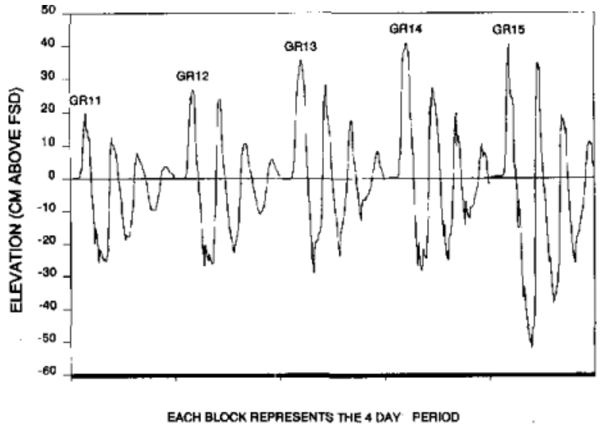
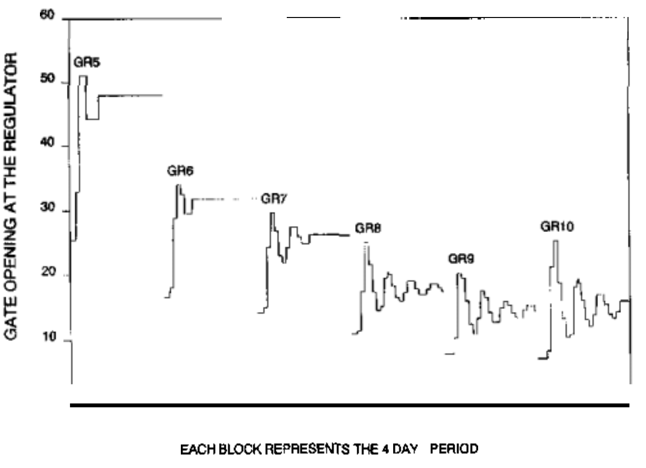


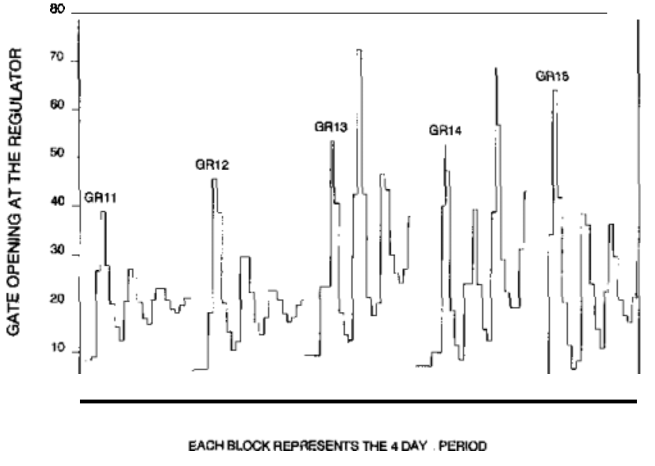
Figure 33. Operations of the regulators during the four days, GR6 to GR10.



The number of operations is 7 at the first regulator (GR5) and more than 20 further downstream (after GR8). For each regulator, it was observed that the gate openings fluctuate around the final values. The mutual disturbances generated by successive regulators are very important as it is difficult to properly operate in such a situation. This is the main reason why it takes such a long time to stabilize the canal.

Therefore, it can be concluded that the present operational practices do allow stabilization of the canal. But, new steady flow conditions can be reached only after several days and many operations. This is due to the fact that when an operator modifies the gate openings of his regulator he can use only local information (water levels at his regulator). But, this information is not reliable because it can be influenced by the operations at the other regulators. Therefore, many trials have to be performed. This problem is quite thorny; for example, if the main sluice discharge is changed every week a steady state will be rarely achieved resulting in frequent fluctuations of the discharges at the offtakes.

Figure 34. Operations of the regulators during the **four** days, **GR11 to GR15.**



PROPOSED IMPROVEMENTS

In this section, the scenario simulated through the model is the same as in the previous section but the operational practices are different. The improvements are mainly based on new information given to the operators. This information is the result of simulations through the mathematical model.

It was observed in the previous section that the gate openings were fluctuating around the final value before reaching it. This value *can* he computed by Unit 2 of the mathematical flow simulation model (steady flow). The improvement tested in this section is based on the direct setting of these new openings at the regulators. The problem to be solved is to know when to do so and to test the importance of the timeliness of such operations.

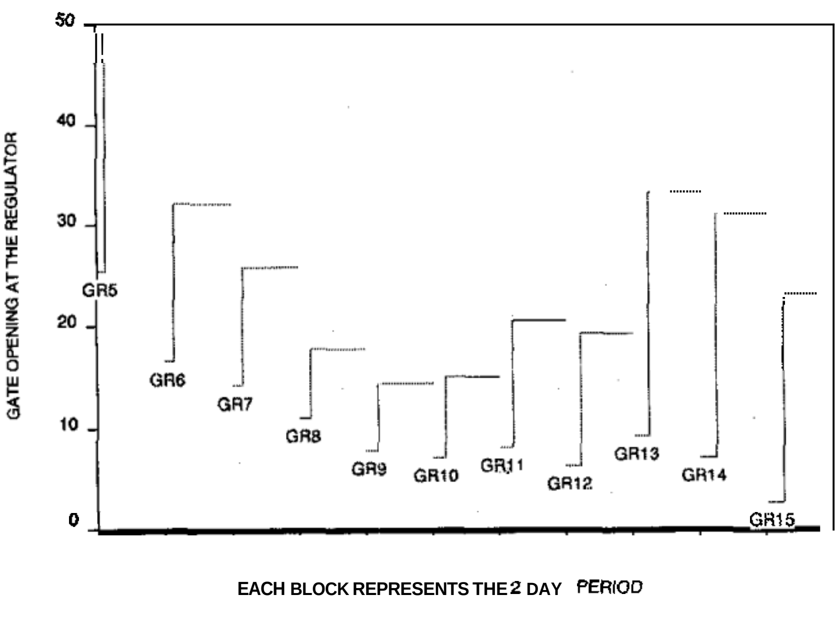
The first method tested was to evaluate the time lags between the main sluice and the regulators. The time lag is defined as the time between the release of the extra discharge at the main sluice and the time when half of the wave has reached the selected regulator. In some studies, the time lag definition takes into account the beginning or the maximum of the wave. But the accuracy of these methods is not very good. To evaluate the time lags, the wave propagation was simulated through the model with no operation at the regulators. The values obtained are given in Table 7.

Table 7. The time lag between the main sluice and the regulator (in hours: minutes).

GR2 : 1:20	GR 9 : 9:40
GR3 : 2:10	GR10: 1040
GR4 : 4:20	GR11: 11:50
GR5 : 4:40	GR12: 13:10
GR6 : 6:00	GR13: 1450
GR7 : 7:10	GR14: 15:40
GR8 : 8:40	GR15: 1630

These values match, very well, the time lags observed during field operations. These time lags were then introduced into the regulation module. The operator assignments were simulated to set the gate openings to the final values (computed by Unit 2; cf. Figure 35) at the time of extra discharge + time lag.

Figure 35. Operations of the regulators during the two days



But, as soon as the regulators are operated, the waves are accelerated and the time lags become much shorter. The decreases in time lags in this scenario are given in Table 8.

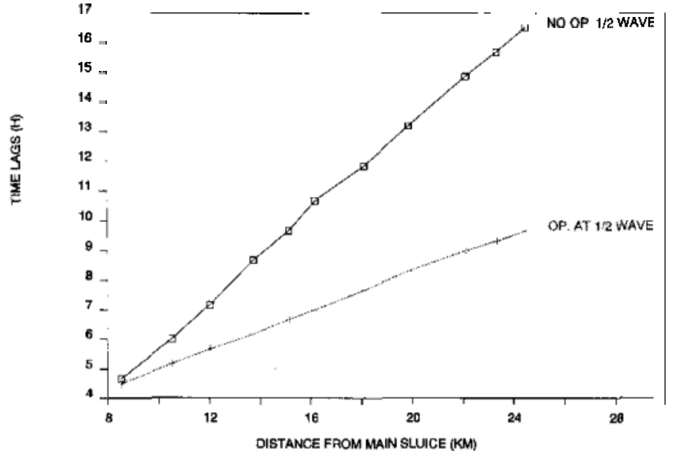
Table 8. Decreases *in* time lags (in hours: minutes).

GR2: - 0	GR 9 - 300
GR3: - 0	GR10: - 3:50
GR4: - 0	GR11: - 4:10
GR5: - 0:10	GR12: - 4:50
GR6: - 0:50	GR13: - 5:50
GR7: - 1:30	GR14 - 6:20
GR8: - 2:30	GR15: - 650

The time lags with and without operations at the regulators are given in Figure 36 for GR5 to GR15. For GR2, GR3 and GR4, the time lags are the same because these three regulators are not operated (as observed in the field).

It is not possible to use this method in practical operations because prior knowledge of the time lags is needed, but these time l a 5 themselves depend on the operations.

Figure 36. Time lags between main sluice and regulators, GR5 to GR15.



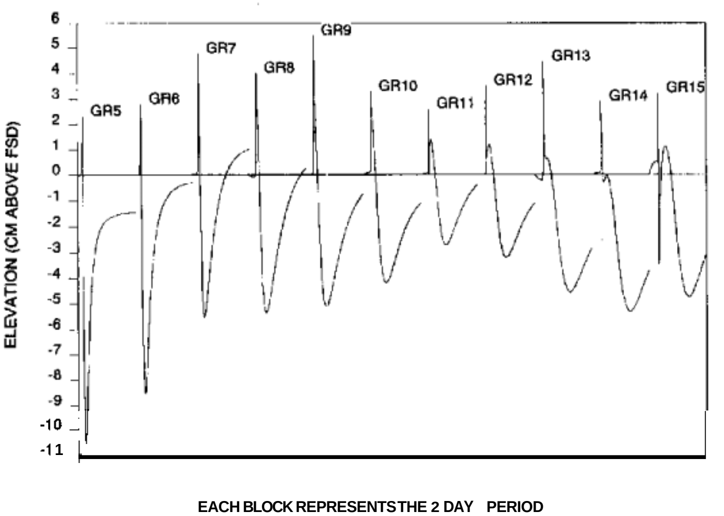
During the same simulation, it was observed that even if the time lags were changed downstream of an operated regulator, the maximum elevation of the wave was not modified. It is, therefore, possible to operate a regulator at any given moment of the wave's arrival if its elevation is observed. To do *so*, the first step is to simulate the wave propagation (with no operation at the regulators) and to observe the maximum elevation H of the wave at each regulator. Then the operators can be instructed to open the gates at the computed value when the wave elevation h is at a certain value (between 0 and H).

This method was simulated for:

h = 0 + 2 cm
h = 114 H
h = 112 H
h = 314 H
h = H - 2 cm

For an intervention at h = 0 + 2 *cm*, the water level decreases to **–10** *cm* and the time for total stabilization is more than **48** hours (cf. Figure **37).**

Figure 37. Water surface elevation upstream of the regulators for intervention **at** h = *0* + **2 cm.**



For **an** intervention at h = **1/4 H,** the magnitude of perturbation ranges **from** – 6 to **+11** ***cm.*** The time for total stabilization is around **48** hours (slightly more for **GR14**; cf. Figure **38).**

For an intervention at **h = 1/2 H,** the **time required for total stabilization** (water level within the 2-cm threshold at each regulator) is 26 hours. In fact, this time is much shorter for the first regulators in the upper reaches of the canal. The maximum magnitude of perturbations is **17** cm at **GR12** (cf. Figure **39).**

For an intervention at h = 3/4 **H,** the water levels upstream of the regulators **go** up to **24** *cm* above FSD (at **GR14).** The time for total stabilization is around **36** hours (cf. Figure **40).**

Figure 38. Water surface elevation upstream of the regulators for intervention at h = 1/4 H cm.

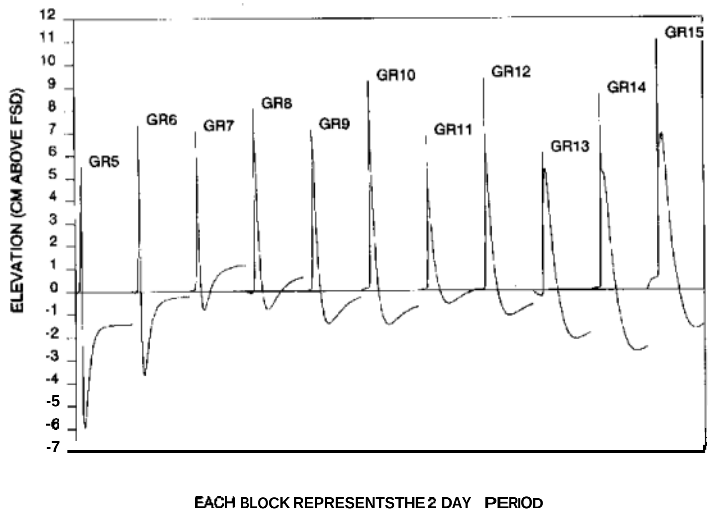


Figure 39. Water surface elevation upstream of the regulators for intervention at h = 1/2 H cm.

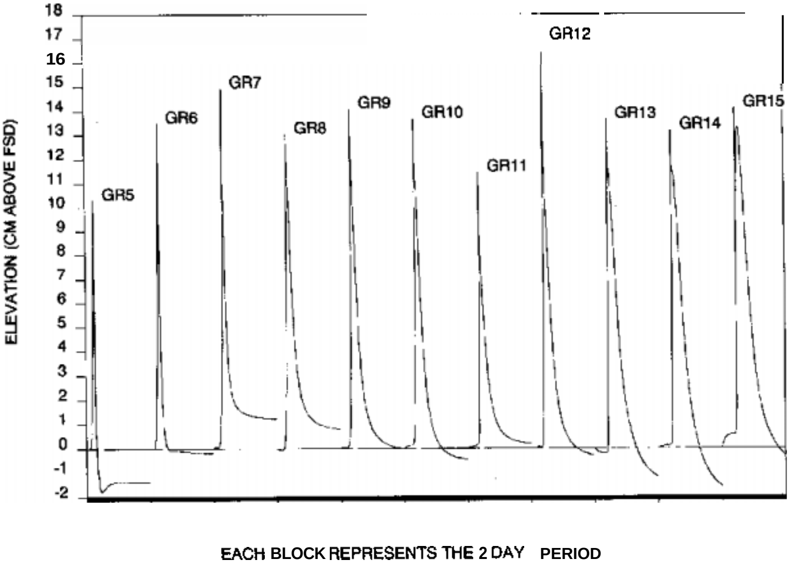
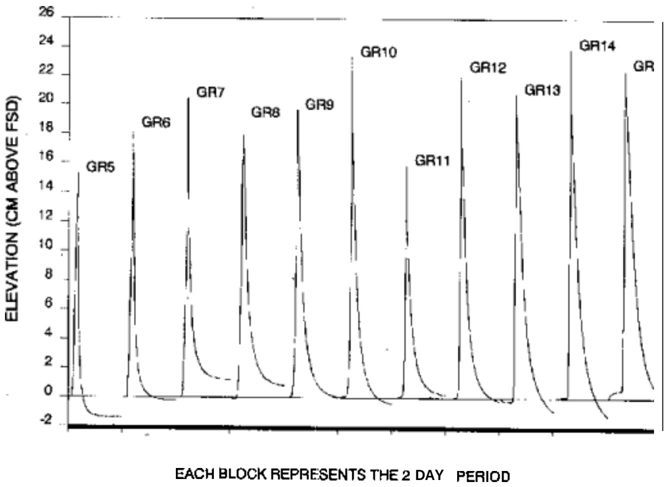
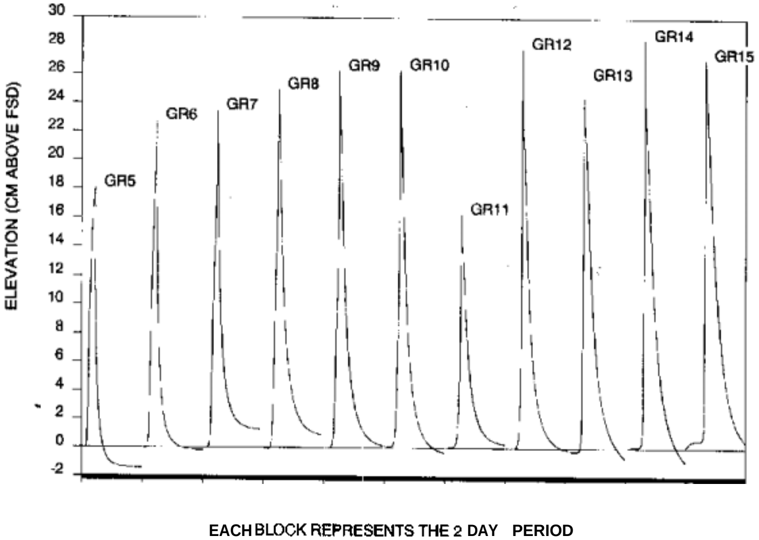


Figure 40. Water surface elevation upstream of the regulators for intervention at h = 3/4 H cm.



Finally, for an intervention at H – 2 cm, the water levels upstream of the regulators go up to 28 cm above FSD (at GR14). The time for total stabilization is around 40 hours (cf. Figure 41).

Figure 41. Water surface elevation upstream of the regulators for intervention at h = H -2 cm.

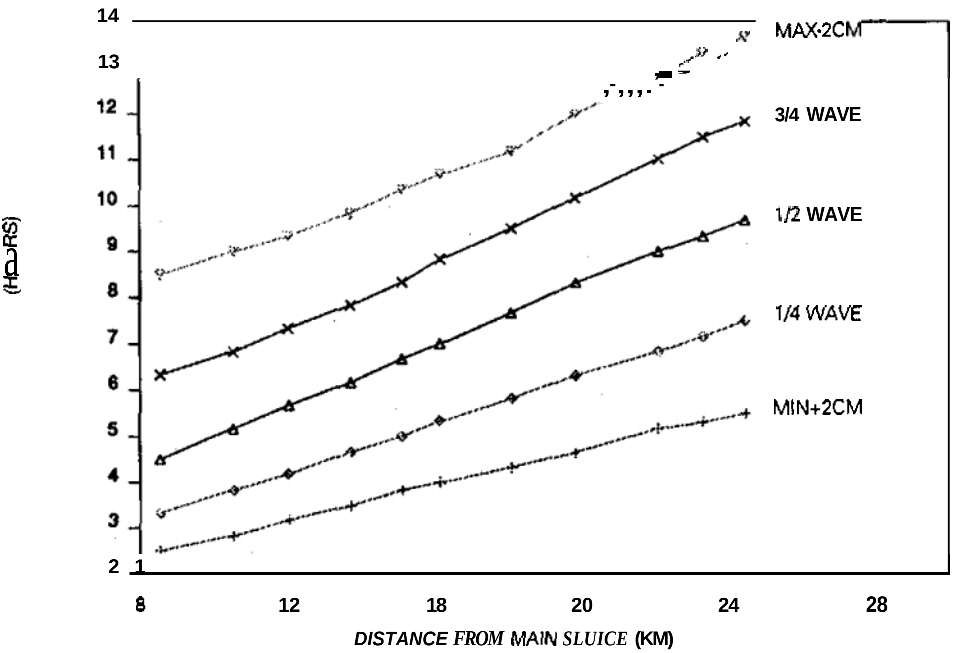


Therefore, a judicious time for operating the regulator is when about half the wave has arrived at the regulator. To be able to do so, the maximum elevation of the wave should be known. Using the mathematical flow simulation model this value can be estimated for each regulator.

Then the operator’s task is to set the openings computed by Unit 2 when half the maximum elevation is reached. But, to help the operator, it could be useful to tell him the approximate time of intervention. This time can also be estimated by the mathematical flow simulation model.

The times of operation for the different elevations of the wave are shown in Figure **42.**

Figure **42.** Times of operation of the regulators **GR5** to **GR15.**



In this figure, the reference time 0 corresponds to the time at which the extra release occurs. It can be observed that the duration between **1/4** and **3/4** of the wave is around 3 to **4** hours. This means that a precision of about 1 hour should he sufficient for the operations at the regulator. For example, if the main sluice discharge is increased at 7:00 a.m., with these operations, the wave will arrive (1/2 of its maximum elevation) at **GR10** at **7:00** + 6:50 = 1350. It can be assumed that, if the operators can set the new gate openings between **13:20** and 14:20, it will be done after **1/4** H and before 3/4 H.

For this scenario, the operator’s tasks (for a main sluice release at 7:00 a.m.) are shown in Table 9.

Table 9. Operator's tasks for a main sluice release at 7:00 a.m.

Regulator	Time of operation	Water elevation (1/2 H)	Magnitude of operation
GR5	11:30	+ 10 cm	+ 23.0 cm
GR6	1210	+ 13 cm	+ 15.5 cm
GR7	1240	+ 14 cm	+ 11.5 cm
GR8	1310	+ 13 cm	+ 6.7 cm
GR9	1340	+ 14 cm	+ 6.6 cm
GR10	1400	+ 14 cm	+ 8.0 cm
GR11	1440	+ 11 cm	+ 12.4 cm
GR12	15:20	+ 17 cm	+ 13.0 cm
GR13	1600	+ 14 cm	+ 24.0 cm
GR14	1620	+ 13 cm	+ 24.0 cm
GR15	16:40	+ 13 cm	+ 20.5 cm

CONCLUSION

The proposed, improved operational practices for a transition between two steady states allow a new steady state to be reached much faster than that with the present practices. The methodology is:

1. Simulate the initial state through Unit 2 of the simulation model.
2. Simulate the final state through Unit 2.
3. Compute the modification of the gate openings for each regulator.
4. Use Unit 3 to evaluate the maximum wave elevation above the sidewalls, with no operations at the regulators.
5. Simulate the wave propagation with operations when 1/2 of the wave has reached the regulator (h = 1/2 H).
6. Get the times of operations from the output regulation file of the above simulation.
7. Give the instructions to the operators in terms of time and magnitude of operations.

According to the model, this method allows the stabilization of the canal in less than one day. For example, if the extra discharge is released at 07:00 a.m. then all the operations will be performed before the evening and the canal should be stabilized the next morning. It would be very interesting to test it in the field and to observe the quality of the model predictions. But the accuracy of the method depends on the calibration of the model. Maximum precautions should be taken to minimize the calibration influences and it is better to compute the modifications of the gate openings (points 1, 2 and 3) rather than to give directly the new openings in absolute values. The criteria for operating include both time lags and wave elevations.

It should be noted that it would be good to allow the operators to check the upstream water levels (the next day, for example) and to modify the gate openings by a few centimeters only, if required.

Example 2: Correction of slow drifts at a regulator

This scenario is based on the following observation: during long periods of non-operation (for example, during nights), slow drifts of the levels upstream of the regulators may occur. These drifts can be as low as a few centimeters per day. The idea is, therefore, to test if it is possible and sensible to try to correct these small differences without generating extra perturbations, and if so, how ?

PRESENT OPERATIONAL PRACTICES

First, the present operational practices have to be simulated through the model in order that they may be evaluated. These simulations are carried out using the module described above.

The initial state for this simulation is a canal where the water Level is at FSD at each regulator except at **GR8.** At this regulator, the upstream level chosen for the simulation is **5** cm above FSD (it may be supposed that it was the drift during the previous days). The regulators **GR2, GR3** and **GR4** are not operated as is the case in the field. The main hypotheses made for this simulation are:

* The operator checks his regulator every **4** hours.
* If the level is within the authorized thresholds, he just stays for 20 minutes and leaves.
* If not, he operates the regulator and stays **1** hour and **20** minutes to check his operation.
* The work day is from 07:00 a.m. to 05:00 p.m.
* The operation thresholds are –2cm and +2cm.

It is observed (cf. Figures **43** to SO) that:

* New steady conditions are attained, approximately, at the regulators **GR2** to **GR12** after about **15** operations and **48** hours of simulation.
* Further downstream, the magnitude of fluctuations is much greater (7 cm at **GR9, 5** cm at **GR10, 10** cm at **GR12,** 19 *cm* at **GR13,** etc.).
* The fluctuations are very acute for **GR13, GR14** and **GR15.**

Figure 43. Water surface elevation upstream of GR5 and GR6.

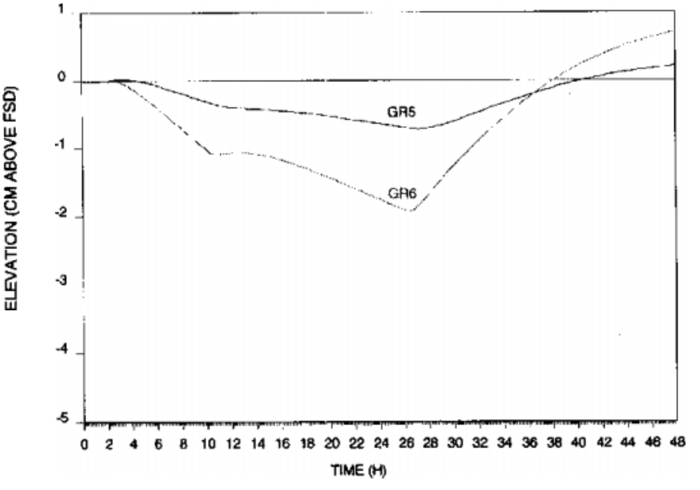


Figure 44. Water surface elevation upstream of GR7 and gate opening.

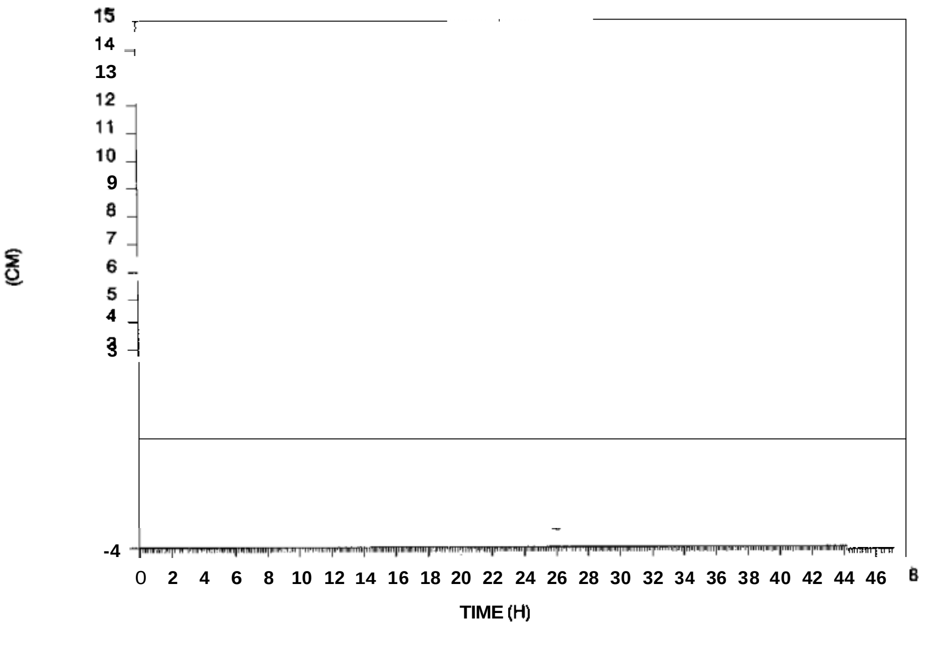


Figure 45. Water surface elevation upstream of GR8 and gate opening.

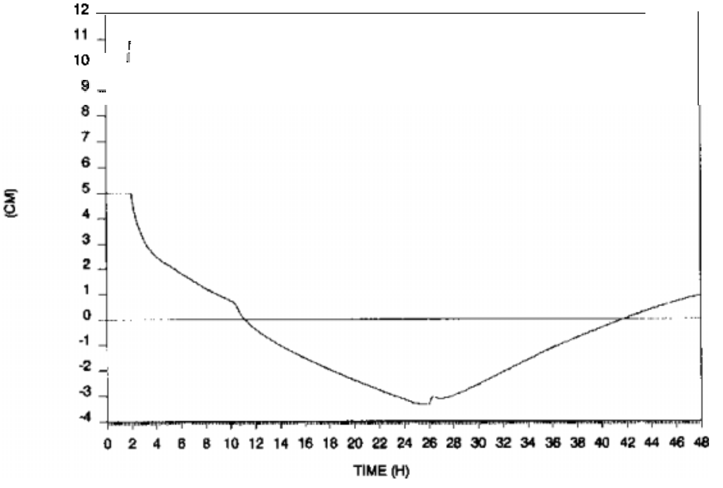


Figure 46. Water surface elevation upstream of GR9 and gate opening.

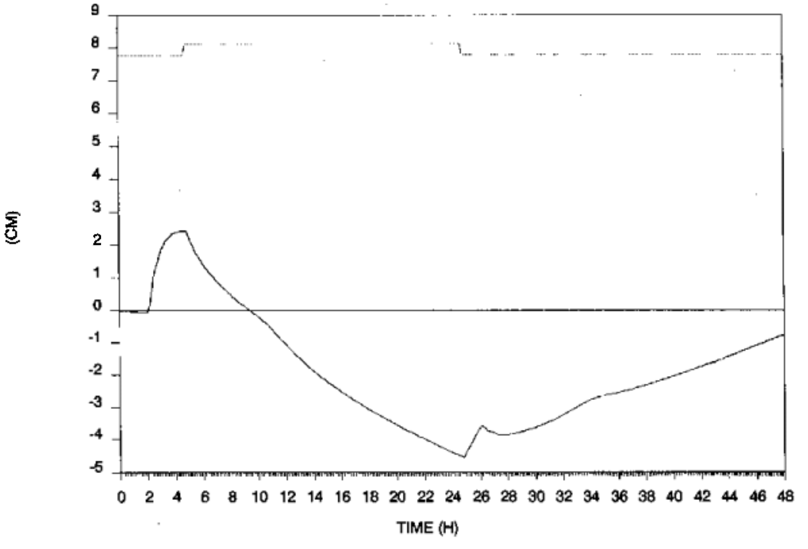


Figure 47. Water surface elevation upstream of GR10 and gate opening.

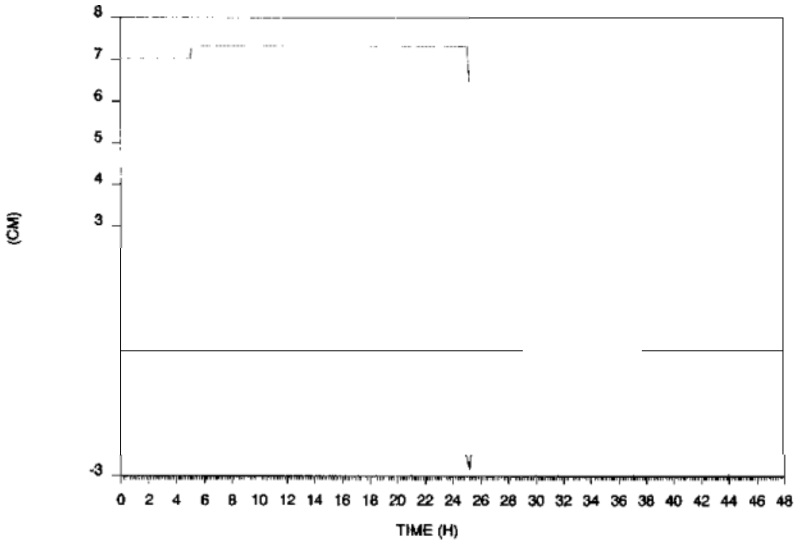


Figure 48. Water surface elevation upstream of GR11 and gate opening.

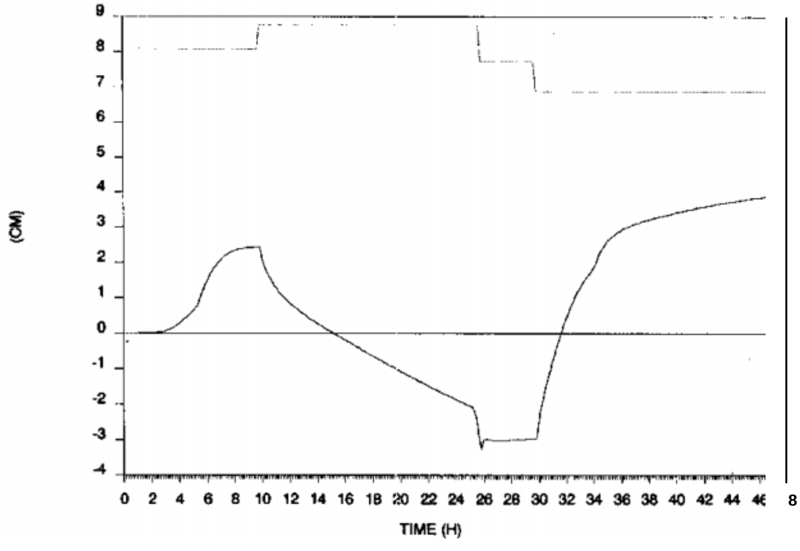


Figure 49. Water surface elevation upstream of GR12 and gate opening.

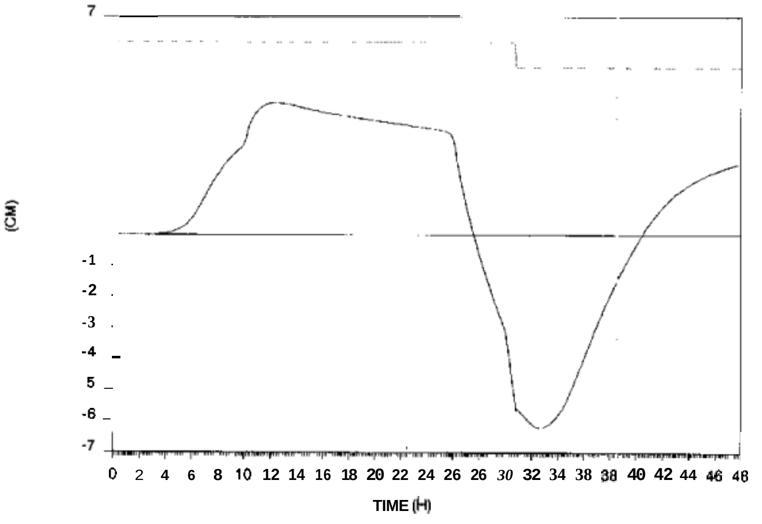
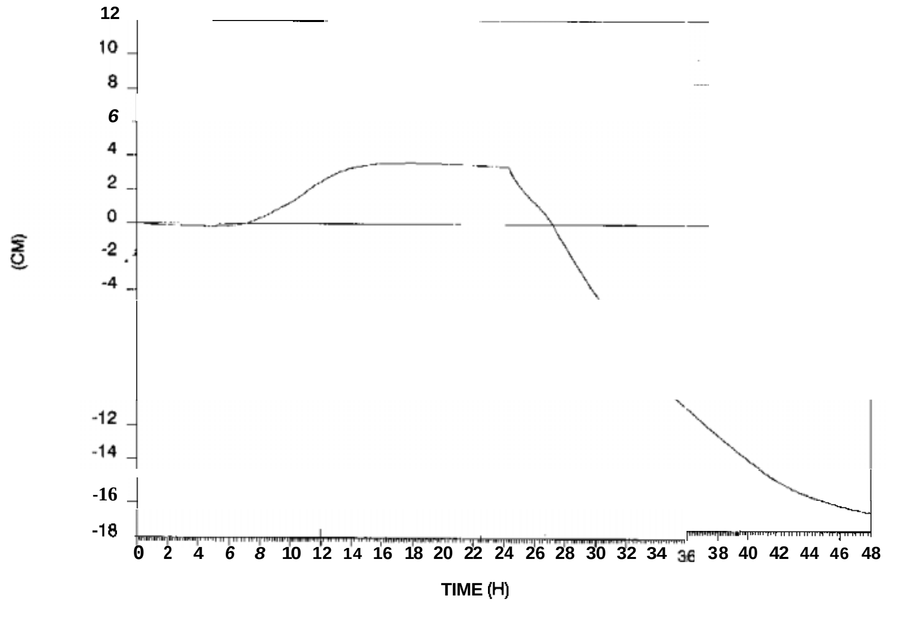


Figure 50. Water surface elevation upstream of GR13 and gate opening.



In the following section, the improvements introduced when other operators are informed **of** the operations performed at *GR8* are examined.

PROPOSED IMPROVEMENTS

Further simulations were carried out introducing a single modification with reference to the simulation illustrating the present operational practices. The modification is:

Informing the operators downstream of GR8 that they will observe a transient wave and that they should not operate. The operator of the regulator **GR7** (just upstream of GR8) can he informed that he should operate soon. The other operators further upstream should not observe any perturbation and therefore should not operate. All the other parameters of the regulation module are exactly the same.

The following results were obtained:

At the regulator **GR8,** one operation is enough to stabilize the upstream level around FSD (at less than ***2*** cm) after about **3** hours.

One operation at GR7 is also enough to settle levels within the threshold limits at GR5, GR6 and **GR7.**

Downstream of GR8, these levels reach FSD again after the wave transition. The threshold is overstepped only at GR9 and GR10. The maximum range is +2.5 cm above FSD at GR9 and the 2-cm threshold is exceeded during a period of ahout 6 hours. At regulator GR10, the level exceeds the authorized limit during a **period of** ahout **4** hours before stabilization at FSD.

In any case, these ranges are very limited compared to the ones in the previous section for the same scenario and the total number of operations on the **14** regulators is only ***2.***

CONCLUSION

During steady flow conditions, with **no** change of the main sluice discharge, slow drifts of the water surface elevations upstream **of** the regulators along the canal may be observed. If one of these levels exceeds an acceptable limit imposed by the canal design (overtopping problem, excess or shortage discharge at one offtake, etc.) correction is possible.

If the operators try to correct any perturbation without the information on the other operations along the canal, they risk amplifying these perturbations instead of stabilizing the canal flow. Therefore, the number of operations has to be limited and communication among the operators has to be improved.

When making a correction, the operators downstream have to be informed that they will observe a transient wave but should not make any gate adjustments.

On the other hand, the operator of the upstream regulator has to be informed that he will have to operate his regulator soon. If his regulator is not exactly at FSD and if he also wants to make a correction be can do ***so;*** but, in this case, he has to inform the upstream operator, and ***so*** on.

This scenario is based on a single correction of the water level at only one regulator but the method can be extended to 2 or **3** corrections. One operation at GR(n) implies no other change downstream but one operation to the upstream regulator GR(n-*1*). This is due to the fact that in the case of the Kirindi Oya canal, submerged flow usually occurs at each regulator which is in the area of the backwater effect of the next one. Therefore, if several corrections have to be made, the start should be from the regulator further downstream For example, if two drifts, one at GR6 and one at GR7, are to be corrected then there are two possibilities:

1. * Operate GR6 first.
   * Make the required correction at GR5 to reach FSD.
   * Wait for stabilization.
   * Then operate GR7.
   * Make the required correction at GR6 to reach FSD.

2. * Operate GR7 first.
   * Make the required correction at GR6 to reach FSD.
   * Make the required correction at GR5 to reach FSD.

It is observed that the second procedure requires less effort (3 operations instead of 5).

CHAPTER 4

Conclusions

THIS PAPER HAS focused on the conception and field installation of a mathematical flow simulation model for the Kirindi Oya Right Bank Main Canal. Theoretical aspects related to the development of the model software as well as practical procedures to calibrate the model in order that it accurately reflects the field conditions have been detailed.

The Kirindi Oya RBMC simulation model represents an effective research tool to explore the interactions between canal design and management with a view to achieving improvements in irrigation performance.

The model also provides the system manager with a decision-support tool which allows him to formulate effective and responsive canal-operation strategies, even under dynamic transient conditions. The holistic view of the hydraulic functioning of the canal which the model offers to the manager provides him with opportunities to enhance his understanding of the behavior of his system.

This should, in turn, facilitate dialogue with farmers, perhaps leading to a more productive role for them in system management. In this context, the use of the model as an innovative training tool should be encouraged.

The paper has also highlighted some of the capabilities of the model to address a range of main canal design, maintenance and operational issues. Further extensive applications to respond to both routine management situations (such as achieving a given water distribution plan) and exceptional events (such as the occurrence of rainfall) are being field-tested as part of the next phase of this research project aimed at using the simulation model in support of practical system management. Particular emphasis will he given to examining the organizational implications associated with adopting this innovative management tool in the context of a manually operated irrigation system.

This pilot experience in practical use of the model as a decision-support tool will provide useful insight into the scope for computer assisted management in manually operated irrigation systems and will probably determine prospects for further applications in other sites.

A generalized simulation model derived from the Kirindi Oya RBMC software has been developed by CEMAGREF. The application of this model, called SIC (Simulation of Irrigation Canals), to similar sites is expected to be facilitated by several additional developments such as an user-friendly interface for the Topography Unit and an automatic calibration module.

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Annex 1

Modeling Cross Structures

A distinction has been made between devices with a high sill elevation (called hereafter Weir/Orifice) and devices with a low sill elevation (called hereafter Weir/Undershot Gate).

WEIR/ORIFICE (HIGH SILL ELEVATION)

Weir – Free Flow

$$Q = \mu_F L\sqrt{2g}\,h_1^{3/2} \qquad [1]$$

Classical equation for the free flow weir $\mu_F = 0.4$, (Ref LENCASTRE, A (1986)).

Weir – **Submerged**

$$Q = \mu_s L\sqrt{2g}\,(h_1 - h_2)^{1/2} h_2 \qquad [2]$$

Classical formulation for the submerged weir.
The free flow/submerged transition takes place for:

$$h_2 = \frac{2}{3} h_1.$$

Thus,

$$\mu_S = \frac{3\sqrt{3}}{2}\mu_F \quad \text{for} \quad \mu_F = 0.4 => \mu_s = 1.04$$

The equivalent free flow coefficient can be calculated:

$$\mu_F = \frac{Q}{L\sqrt{2g}\;h_1^{3/2}}$$

It indicates the degree of submergence of the weir by comparing it to the introduced free flow coefficient. In effect, the reference coefficient of the device considered is that corresponding to the free flow weir.

Orifice — Free Flow

An equation of the following type is applied:

$$Q = \mu L\sqrt{2g}\,(h_1^{3/2} - (h_1 - W)^{3/2}) \qquad [3]$$

This formulation is applicable to large width rectangular orifices. The continuity towards the open-channel flow is assured when:

$$\frac{h_1}{W} = 1 \quad \text{One then has: } \mu = \mu_F$$

Orifice — Submerged

Two formulations exist, according to whether the flow is partially submerged or completely submerged.

Partially submerged flow:

$$Q = \mu_F L\sqrt{2g}\,[\frac{3\sqrt{3}}{2}((h_1 - h_2)^{1/2} h_2 - (h_1 - W)^{3/2})]$$

This applies for $\frac{h_1}{W} > 1$ and $h_2 > \frac{2}{3} h_1$

Totally submerged flow:

$$Q = \mu' L\sqrt{2g}\,(h_1 - h_2)^{1/2}[h_2 - (h_2 - W)]$$

$$\Rightarrow \qquad Q = \mu' L\sqrt{2g}\,(h_1 - h_2)^{1/2} W$$

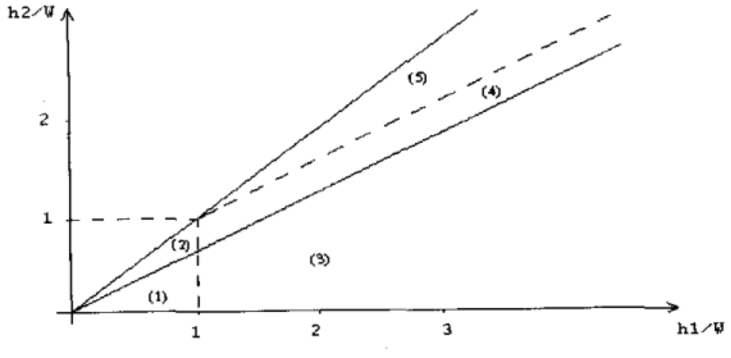
This is the classic equation of the submerged orifice,

with $\frac{h_1}{W} > 1$ and $h_2 > \frac{2}{3}h_1 + \frac{W}{3}$ *i.e.* $\mu' = \mu_S$

The operation of the weir/orifice device is represented in Figure AI.1. Whatever the condition of pipe flow, one calculates an equivalent free flow coefficient, corresponding to the free flow orifice:

$$C_F = \frac{Q}{L\sqrt{2g}\,W(h_1 - 0.5W)^{1/2}}$$

Figure A1.1.



1 : Weir — Free flow
2 : Weir — Submerged
3 : Orifice — Free flow
4 : Orifice — Partially submerged
5 : Orifice — Totallysubmerged

WEIR/UNDERSHOT *GATE* (LOW SILL ELEVATION)

Weir — Free Flow

$$Q = \mu_F L\sqrt{2g}\, h_1^{3/2}$$

Weir — Submerged

$$Q = k_F \mu_F L\sqrt{2g}\, h_1^{3/2}$$

with k_F = coefficient of reduction for submerged **flow.**

The flow reduction coefficient is a function **of** $\frac{h_2}{h_1}$ and of the value a of this ratio at the instant of the free flow/submerged transition. The submerged conditions are obtained when $\frac{h_2}{h_1} > a$. The law **of** variation of the k_F coefficient has been derived from experimental results.

Let $x=\sqrt{1-\frac{h_2}{h_1}}$

If $x > 0.2 \Rightarrow k_F = 1-(1-\frac{x}{\sqrt{1-\alpha}})^{\beta}$

If $x \leq 0.2 \Rightarrow k_F = 5x(1-(1-\frac{0.2}{\sqrt{1-\alpha}})^{\beta})$

with $\beta = -2\alpha+2.6$

One calculates an equivalent coefficient for free flow conditions as before.

Undershot Gate – Free Flow

$$Q = L\sqrt{2g}(\mu.h_1^{3/2} - \mu_1.(h_1-W)^{3/2}) \quad [7]$$

It has been established experimentally th the undershot gate scharge coefficient increases with $\frac{h_1}{W}$. A law of variation of μ of the following form is adopted.

$$\mu = \mu_o - \frac{0.08}{h_1/W} \text{ with } \mu_o \approx 0.4$$

$$\text{Hence, } \mu_1 = \mu_o - \frac{0.08}{\frac{h_1}{W}-1}$$

In order to ensure the continuity with the open channel free flow conditions for $\frac{h_1}{W}=1$ it must have: $\mu_F = \mu_o - 0.08$.

Hence, $\mu_F = 0.32$ for $\mu_o = 0.4$

Undershot Gate – Submerged

Partially submerged flow:

$$Q = L\sqrt{2g}[k_F.\mu.h_1^{3/2} - \mu_1(h_1-W)^{3/2}] \quad [8]$$

k_F being the same as for open-channel flow.

The following free flow/submerged transition law has been derived on the basis of experimental results:

$$\alpha = 1 - 0.14\frac{h_2}{W}$$

$$0.4 \leq \alpha \leq 0.75$$

In order to ensure continuity with the open-channel flow conditions, the free flow/submerged transition under open-channel conditions has to be realized for $a = 0.75$ instead of $\frac{2}{3}$ in the weir/orifice formulation.

Totally submerged Bow:

$$Q = L\sqrt{2g}\left(k_F.\mu.h_1^{3/2} - k_{F1}.\mu_1.(h_1 - W)^{3/2}\right) \qquad [9]$$

The k_{F1} equation is the same as the one for k_F where h_2 is replaced by $h_2\text{-}W$ (and hi by $h_1\text{-}W$) for the calculation of thex coefficient (and therefore for the calculation of k_{F1}).

The transition to totally submerged flow occurs for:

$$h_2 > \alpha_1.h_1 + (1-\alpha_1).W$$

with:

$$\alpha_1 = 1 - 0.14\frac{h2-W}{W}$$

$$(\alpha_1 = \alpha(h_2 - W))$$

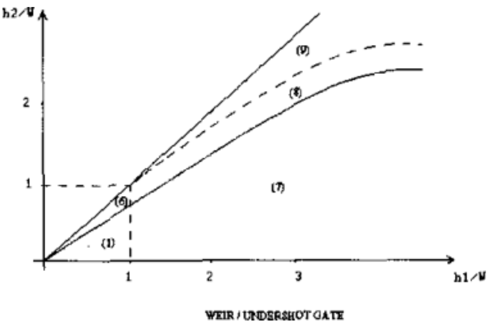
The functioning of the weir/undershot gate device is represented in Figure A1.2. Whatever the conditions of the pipe flow, one calculates an equivalent free flow discharge coefficient, corresponding to the classical equation for the free flow undershot gate.

$$C_F = \frac{Q}{L\sqrt{2g}\,W\sqrt{h_1}}$$

The reference coefficient introduced for the device is the classic C_G coefficient of the free flow undershot gate. It is then transformed to $\mu_o = \frac{2}{3}C_G$.

It is possible to get $C_F \neq C_G$, even under free flow conditions, since the discharge coefficient increases with the $\frac{hi}{W}$ ratio.

Figure A1.2.



1: Weir —Free flow
6 Weir — Submerged
7 Undershot gate — Freeflow
8: Undershot gate — Partially submerged
9 Undershot gate —Totally submerged

OVERFLOW

One takes into account the fact that the undershot gate has a certain height and if the water level rises upstream of the gate, water can flow over the gate. The flow overtopping the gate is then added to the flow resulting from the previous pipe flow computations. The overflow Q_S is expressed as follows, under free flow conditions:

$$Q_S = 0.4L\sqrt{2g}.(h_1 - W - h_s)^{3/2} \qquad [10]$$

h_s being the gate height

The weir is thus considered as having a discharge coefficient of 0.4 decided *a priori*. One uses the equivalent formula in the case of submerged overflow conditions:

$$Q_S = \mu' L\sqrt{2g}\ (h_1 - h_2)^{1/2}.(h_2 - W - h_s) \qquad [11]$$

with: $\mu' = \frac{3\sqrt{3}}{2}\mu = 1.04$

Annex 2

Modeling Unsteady Flow Computation

DOUBLE SWEEP METHOD

The Saint Venant's equations are transformed (through the Preissmann scheme) into a set of linear simultaneous equations connecting two sections i) and j):

$$A_{11}.\ \Delta Q_i + A_{12}.\ \Delta Z_i = B_{11}.\ \Delta Q_j + B_{12}.\ \Delta Z_j + B_{13} \qquad [12]$$
$$A_{21}.\ \Delta Q_i + A_{22}.\ \Delta Z_i = B_{21}.\ \Delta Q_j + B_{22}.\ \Delta Z_j + B_{23} \qquad [13]$$

Consider a reach having n computational cross sections. The system of equations to be solved is:

* Saint Venant's equations at every interval located **between** two computational cross sections, at every time instant t.
* Upstream and downstream boundary conditions.

Discretization transforms the reach into a series of n computational cross sections connected to each other by the two linear equations [12] and [13]. One then has $2(n-1)$ linear equations in $\boldsymbol{Q}$ and $\boldsymbol{Z}$. The two missing equations for the system resolution are provided by the upstream and downstream boundary conditions, that are linearized at each time step.

Upstream boundary condition:

$$R_1.\ \Delta Q_1 + S_1.\Delta Z_1 = T_1$$

In the case of a $Q(t)$ relation, one has:

$$R_1 = 1,\ \ S_1 = 0 \text{ and } T_1 = Q(t+\Delta t) - Q(t)$$

Downstream boundary condition:

$$R'_n.\ \Delta Q_n + S'_n.\ \Delta Z_n = T'_n \text{ with:}$$
$$\frac{\text{[illegible]}}{\Delta Z} = a, \quad R'_n = 1,\ \ S'_n = -\alpha,\ \ T'_n = 0$$

A linear system with 2.n equations has to be solved. Instead of inversing the system matrix, the double sweep method is employed.

If the equations [12] and [13] are written under the following form :

$$\Delta Q_i = A.\Delta Q_j + B.\Delta Z_j + C \qquad [14]$$
$$\Delta Z_i = D.\Delta Q_j + E.\Delta Z_j + F$$

A band matrix with only one diagonal on the lower triangular side is obtained
The first upstream-downstream sweep gives an upper triangular matrix:
For two consecutive sections i and j an impedance relation is combined:

$$R_i.\Delta Q_i + S_i.\Delta Z_i = T_i$$

with the equation [14]

$$R_i . (A_i . \Delta Q_j + B_i . \Delta Z_j + C_i) + S_i . (D_i . \Delta Q_j + E_i . \Delta Z_j + F_i) = T_i$$
$$R'_j \cdot \Delta Q_j + S'_j \cdot \Delta Z_j = T'_j$$

with

$$R'_j = R_i . A_i + S_i . D_i$$
$$S'_j = R_i . B_i + S_i . E_i$$
$$T'_j = T_i - R_i . C_i - S_i . F_i$$

The coefficients R'_j, S'_j and T'_j **are** set by normalizing, in order to avoid the propagation of numerical error.

$$R_j = \frac{R'_j}{\sqrt{R'^2_j + S'^2_j}}$$

One then has the new upstream impedance relation for section j:

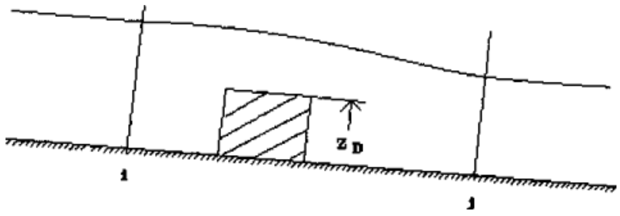
$$R_j . \Delta Q_j + S_j . \Delta Z_j = T_j$$

The second sweep allows the calculation of the Q and Z in each computational cross section by the way of the equations [15].

$$\Delta Z_i = D_i . \Delta Q_j + E_i . \Delta Z_j + F_i$$
$$\Delta Q_i = \frac{T_i - S_i . \Delta Z_i}{R_i} \qquad [15]$$

INTRODUCTION OF SINGULARITIES

Figure A2.1.



The problem to be solved in the case of a singularity is:

$$\begin{cases} R_i.\Delta Q_i + S_i.\Delta Z_i = Ti \\ \Delta Q_i = \Delta Q_j \\ Q_i(t) = f[Z_i(t)\ , Z_j(t)\ , W(t)] \end{cases} \qquad [16]$$

It is necessary to transmit the impedance relation:

$$R_j.\Delta Q_j + S_j.\Delta Z_j = T_j$$

to the downstream cross section of the singularity.

It is assumed that the device is moveable, and that va tion la $W(t)$ is knc 'n a 'ior

The device equation can be written at the instant t t $(n+1)dt$:

Then:
$$Q_i^{n+1} = f(Z_i^{n+1}\ , Z_j^{n+1}\ , W^{n+1})$$
$$\Delta Qi = f(Z_i^n + \Delta Z_i\ , Z_j^n + \Delta Z_j\ , W^{n+1}) - Q_i^n$$

An expression of the non-linear impedance relation is obtained in the following form:

$$\Delta Q_j = f(Z_i^n + (\frac{T_i}{S_i} - \frac{R_i}{S_i}\Delta Q_j), Z_j^n + \Delta Z_j, W^{n+1}) - Q_i^n \qquad [17]$$

Then the best possible linear approximation to this expression must be found. The tangential approximation equation of the device variation law can be written as:

$$Q_i^n + \Delta Q_i = f\ (Z_i^n + \Delta Z_i, Z_j^n + \Delta Z_j, W^{n+1})$$
$$= f(Z_i^n, Z_j^n, W^{n+1}) + \frac{\partial f}{\partial Z_j}(Z_i^n, Z_j^n, W^{n+1})\Delta Z_i + \frac{\partial f}{\partial Z_j}(Z_i^n, Z_j^n, W^{n+1})\Delta Z_j$$

One then gets:

$$R_j.\Delta Q_j + S_j.\Delta Z_j = T_j \qquad [18]$$

with

$$\begin{cases} R_j = S_i + R_i\dfrac{\partial f(\,)}{\partial Z_i} \\ S_j = -S_i\dfrac{\partial f(\,)}{\partial Z_j} \\ T_j = T_i\dfrac{\partial f(\,)}{\partial Z_i} + S_i(f(\,) - Q_i^n) \end{cases}$$

This method cannot avoid the tangential approximation error of the device variation law, but counterbalances it later in the following time step.

This means that "a correction wave" is included in the expression of the T_j coefficient in the form of an additive term, $S_i(f(\,)-Q_i^n)$ Nevertheless, errors due to the tangential approximation can be significant in the case of rapid variations of flow conditions (device operations, free flow to submerged flow transition, etc.).

It is, therefore, necessary to have an estimation, as precise as possible, of the evolution of the two variable Z_i and Z_j during the time Δt. The best linear approximation to the hydraulic law of the device can then be used. At each singular section, the three equations [17] are available.

A fourth equation is then needed in order to solve the system. This equation will take the form of a hypothesis for Z_j.

The hypothesis does not really attempt to get close to the missing $R'S'T'$ equation but rather its effects on the evolution of the Z_j value.

Assume that:

$$\Delta Z_j = k.\Delta Q_j \qquad [19]$$

with the value of k determined during the previous step.

The following procedure for the computation of $R_j\ S_j\ T_j$ is adopted

1) Hypothesis on Z_j ([19]) + [18] = expected ΔZ_j^* value

$$\begin{cases}\Delta Z_j = k.\Delta Q_j \\ R_j.\Delta Q_j + S_j.\Delta Z_j = T_j\end{cases}$$

=> expected ΔQ_j^* and ΔZ_j^* values

2) Computation of two expected values ΔQ_i^* and ΔZ_i^*

=> $\Delta Q_i^* = \Delta Q_j^*$

and expected ΔZ_i^* value from the $R_i S_i T_i$ impedance relation.

It is assumed that the real values ΔQ_i, ΔZ_i and ΔZ_j will he close to ΔQ_i^*, ΔZ_i^* and ΔZ_j^*.

This results in:

$$\begin{cases}dq_i = \Delta Q_i - \Delta Q_i^* \\ dz_i = \Delta Z_i - \Delta Z_i^* \\ dz_j = \Delta Z_j - \Delta Z_j^*\end{cases}$$

Then:

$$\begin{aligned}Q_i^{n+1} &= Q_i^n + \Delta Q_i = f(Z_i^n + \Delta Z_i\,,\, Z_j^n + \Delta Z_j\,,\, W^{n+1}) \\ &= f(Z_i^n + \Delta Z_i^* + dz_i\,,\, Z_j^n + \Delta Z_j * + dz_j\,,\, W^{n+1})\end{aligned}$$

with low values for dz_i and dz_j.

If $$f(*) = f(Z_i^n + \Delta Z_i^*\,,\, Z_j^n + \Delta Z_j^*\,,\, W^{n+1})$$

Then:

$$Q_i^n + \Delta Q_i = f(*) + \frac{\partial f(*)}{\partial Z_i} dz_i + \frac{\partial f(*)}{\partial Z_j} dz_j$$

$$\Rightarrow \Delta Q_i = f(*) - Q_i^n + \frac{\partial f(*)}{\partial Z_i}(\Delta Z_i - \Delta Z_i *) + \frac{\partial f(*)}{\partial Z_j}(\Delta Z_j - \Delta Z_j^*)$$

$$= f(*) - Q_i^n - \left(\frac{\partial f(*)}{\partial Z_i}\Delta Z_i * + \frac{\partial f(*)}{\partial Z_j}\Delta Z_j *\right) + \frac{\partial f(*)}{\partial Z_i}\Delta Z_i + \frac{\partial f(*)}{\partial Z_j}\Delta Z_j$$

By adding:

$$\begin{cases} R_i \, . \Delta Q_i + S_i \, . \Delta Z_i = T_i \\ \Delta Q_i = \Delta Q_j \end{cases}$$

The result is:

$$\begin{cases} R_j = S_i + R_i \dfrac{\partial f(*)}{\partial Z_i} \\ S_j = - S_i \dfrac{\partial f(*)}{\partial Z_j} \\ T_j = T_i \dfrac{\partial f(*)}{\partial Z_i} + S_i D \end{cases} \qquad [20]$$

with:

$$D = f(*) - Q_i^n - \left(\frac{\partial f(*)}{\partial Z_i}\Delta Z_i^* + \frac{\partial f(*)}{\partial Z_j}\Delta Z_j^*\right)$$

and

$$f(*) = f(Z_i^n + \Delta Z_i^* \, , Z_j^n + \Delta Z_j^* \, , W^{n+1})$$

How is equation [19] determined? Make the hypothesis that Z_j vanes between n and $n+1$ in the same way as it does between n-1 and n, with respect to variation of flow.

Then:

$$K = \frac{Z_j^n - Z_j^{n-1}}{Q_j^n - Q_j^{n-1}}$$

set $K = 0$

if $| Q_j^n - Q_j^{n-1} | < 0.01$

or if the slope of the downstream is of the same sign as the upstream $R'S'T'$ equation.

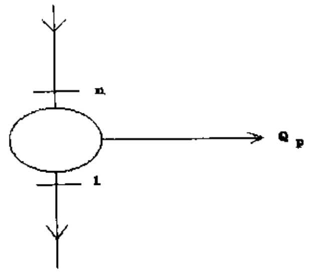
INTRODUCTION OF OFFTAKES

There are two different ways to compute the upstream impedance:

Offtakes of the form $Qp(t)$

Consider an offtake which is only described in the form of a time varying outflow relationship, $Qp(t)$ (Figure A2.2).

Figure. A2.2.



First sweep:

$$\begin{cases} R_n \,.\, \Delta Q_n + S_n \,.\, \Delta Z_n = T_n & \text{with } R_p = 1 \text{ , } S_p = 0 \\ R_p \,.\, \Delta Q_p + S_p \,.\, \Delta Z_n = T_p & \text{and } T_p = Q_p^{n+1} - Q_p^n \\ \Delta Z_n = \Delta Z_1 & \\ Q_n^n + \Delta Q_n = Q_1^n + \Delta Q_1 + Q_p^n + \Delta Q_p & \end{cases} \qquad [21]$$

$$\Delta Q_n = \Delta Q_1 + \Delta Q_p + Q_1^n + Q_p^n - Q_n^n$$

$$\frac{T_n}{R_n} - \frac{S_n}{R_n}\Delta Z_n = \Delta Q_1 + \frac{T_p}{R_p} - \frac{S_p}{R_p}\Delta Z_n + Q_1^n + Q_p^n - Q_n^n$$

$$\Delta Q_1 + \left(\frac{S_n}{R_n} - \frac{S_p}{R_p}\right) \Delta Z_1 = Q_n^n - Q_1^n - Q_p^n + \frac{T_n}{R_n} - \frac{T_p}{R_p}$$

Then:

$$\begin{cases} R_1 = 1 \\ S_1 = \dfrac{S_n}{R_n} - \dfrac{S_p}{R_p} \\ T_1 = Q_n^n - Q_1^n - Q_p^n \dfrac{T_n}{R_n} - \dfrac{T_p}{R_p} \end{cases}$$

One can go down the first sweep in this manner. The second sweep does not create any particular problems and one calculates $W(t)$ in order to satisfy the offtake discharge for the given water surface elevation at the node.

Offtakes of the form W(t)

Consider an offtake described in the form of a time-varying opening relationship, $W(t)$.
First sweep:
The problem is the same as in the previous case, and the system [21] results, in which R_p, S_p and T_p are written differently because $Q_p = f(Z_n, Z_v, W)$ (Z_v is the water elevation downstream of the offtake).

Calculation of Rp, Sp, Tp:

$$\begin{aligned} Q_p^{n+1} &= Q_p^n + \Delta Q_p = f(Z_n^{n+1}, Z_v^{n+1}, W^{n+1}) \\ &= f(Z_n^n, Z_v^n, W^{n+1}) + \frac{\partial f}{\partial Z_n}(Z_n^n, Z_v^n, W^{n+1})\,\Delta Z_n \\ &+ \frac{\partial f}{\partial Z_v}(Z_n^n, Z_v^n, W^{n+1})\,\Delta Z_v \end{aligned}$$

If one sets $Z_v = h(Q_p)$, then:

$$\begin{aligned} Q_p^{n+1} &= f(Z_n^n, Z_v^n, W^{n+1}) + \frac{\partial f}{\partial Z_n}(Z_n^n, Z_v^n, W^{n+1})\,\Delta Z_n \\ &+ \frac{\partial f(Z_n^n, Z_v^n, W^{n+1})}{\partial Z_v} \cdot \frac{\partial h(Q_p^n)}{\partial Q_p}\,\Delta Q_p \\ &\Rightarrow \Delta Q_p.\left(1 - \frac{\partial f(Z_n^n, Z_v^n, w^{n+1})}{\partial Z_v} \cdot \frac{\partial h(Q_p^n)}{\partial Q_p}\right) \\ &= f(Z_n^n, Z_v^n, w^{n+1}) - Q_p^n + \frac{\partial f}{\partial Z_n}(Z_n^n, Z_v^n, W^{n+1})\,\Delta Z_n \end{aligned}$$

Then:

$$\begin{cases} R_p = 1 - \dfrac{\partial f(Z_n^n, Z_v^n, W^{n+1})}{\partial Z_v} \cdot \dfrac{\partial h(Q_p^n)}{\partial Q_p} \\ S_p = -\dfrac{\partial f}{\partial Z_n}(Z_n^n, Z_v^n, W^{n+1}) \\ T_p = f(Z_n^n, Z_v^n, W^{n+1}) - Q_p^n \end{cases}$$

Description of the relationship:

$$Q_p = h^{-1}(Z_v, Z_D) \text{ and } Z_v = h(Q_p)$$

The Q_p law is the same as that for the cross structures. If the offtakes are circular, one calculates the width of the equivalent rectangle having the same opening.
According to the type of the offtake downstream condition chosen, one can calculate

$$\frac{\partial h}{\partial Q_p}$$

One can go down the first sweep in this manner. The second sweep doesn't create any problem. Knowing the water surface elevation at the node and the offtake opening, the discharge through the offtake can be computed.