REPORT NO. R-23

Université de Montpellier
(USTL)
Montpellier, FRANCE

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ALTERNATIVE SCENARIOS FOR IMPROVED OPERATIONS AT THE MAIN CANAL LEVEL: a study of Fordwah Branch, Chishtian Subdivision using a mathematical flow simulation model

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IIMI/Cemagref Practical training for the 'DEA National d'Hydrologie'

Final report
September 1995

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FOREWORD

This report is the thesis or final report for the Master of Science program of Mr. Xavier Litrico. He completed the requirements for an M.S. degree in Agricultural Engineering (Genie Rural in French) from the Ecole Nationale due Genie Rural des Eaux et des Forêts (ENGREF) in Montpellier, France. He spent six months in Pakistan during 1995 to complete all of the necessary field work and analysis. This reproduction is identical with the document accepted by ENGREF in September 1995. This report is being resurrected for publication as a research report because of some rather unique large-scale field exercises in canal hydraulics that advanced our research capability.

We have a number of national and international students participating in the research program of the Pakistan National Program of the International Irrigation Management Institute. Their theses and dissertations are retained in our library for ready reference. Only a few of these documents are selected for publication in our research report series. The principal criteria for publishing is good quality research and a topic that would be of interest to many of our national partners.

This report is an output of a collaborative research program with CEMAGREF, the French national research organization for agriculture, water and forests. This research is an important part of the project "Managing Irrigation for Environmentally Sustainable Agriculture in Pakistan" funded by the Government of The Netherlands.

This research was supervised by Mr. Marcel Kuper of IIMI. In the meantime, he has also supervised two M.Sc students from the University of Technology, Delft. The combination of the three M.Sc studies are related. One was published as Report R-10.

Xavier Litrico began a Ph.D in December 1995 that is being funded by the French Government. His research is involved with canal hydraulics.

Gaylord V. Skogerboe, Director
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Abstract

Irrigation is a necessity in the Punjab, and water, in terms of timeliness, reliability and quantity is one of the limiting factors for an increase in agricultural production. The performance of the system is widely found poor in terms **of** delivering water surface resources according to the official water allocation and scheduling. Canals are almost never in steady state and fluctuations entering the system are amplified by numerous operations at cross regulators. This leads to a high number of breaches, and to an increase in the variability and unreliability of the supply at the secondary level.

This study aims at evaluating the **effect of operations at the main canal level on the water distribution to distributaries,** and propose **alternative rules of operations at a local level that can help the irrigation managers meeting their targets more effectively.** It focuses on the tail part of Fordwah Branch, located in the Chishtian Subdivision.

First a diagnosis of the system was done, to understand operational rules, and their impact on the hydraulic performance of the canal. The diagnosis was further refined with the help of a mathematical hydraulic model. This model is calibrated for the part of the canal in the Chishtian Subdivision. The limits for improved operations are derived from the first outputs of the calibrated model. The diagnosis helped to articulate the key elements to be addressed in alternative operational scenarios and to define the scope for improvement.

A regulation module was then used to simulate present manual operations at cross regulators and offtakes. This regulation module also enables to simulate alternative scenarios of operations at a local level.

Five typical operational situations irrigation staff faces were identified, and five scenarios are simulated for these situations. The results of the simulations show that :

- the discharge downstream of a regulator can be smoothened by eliminating the operations that are not hydraulically justified,

- it is possible to stabilize the levels and discharge in the canal by passing all the fluctuations in one distributary,

- the introduction of a communication system between gate operators can increase the performance of the system by giving a security margin in case of emergency, and tempering the fluctuations instead of amplifying them,

- the introduction of a hydraulic simulation model at the manager level and the implementation of a feed forward command would have a positive effect on the hydraulic state of the canal, and on the distribution to secondary canals. This is an option that seem quite unrealistic, at least for the coming years, as it also implies many changes in the data collection and evaluation processes.

Two field tests of one of these scenarios were performed, in collaboration with the irrigation staff with good results:

- in the first field test, the discharge was kept constant downstream of a cross regulator **by** operating an upstream offtake,

- in the second field test, the canal stayed at full supply and in steady flow during more than **36** hours. The gauge readers were given information on the future perturbations coming from upstream, which allowed them to operate much less than usual and not to amplify little fluctuations resulting in **a** steady state for Fordwah Branch.

Acknowledgments

This study would not have been possible without the help of IIMI field staff in Bahawalnagar. I appreciated their hospitality, enthusiasm and sympathy. I am especially grateful to Mushtaq Khan, field station leader, for his continuous support, Anwar Iqbal and Khalid Mahmood, field assistants, for their valuable help during my stays in the field.

The I&PD staff helped me in carrying out the field tests: Javaid Qureishi, XEN Fordwah, Muhammad Rashid, SDO Chishtian, Samad Asghar, SDO Headquarters, Pacha, Sub-engineer Takht Mahal section, and all the gauge readers in Chishtian Subdivision took a full part in this study. I thank them for their collaboration.

I would like to thank Marcel Kuper for being my supervisor, his advises were always helpful, in technical as well as in other fields. I also wish to thank him together with Pierre Strosser, for the hospitality and sympathy they provided during those **5** months in Pakistan.

I want to include all the IIMI-Pak staff in these thanks, for the good moments I passed here.

I am thankful to Pierre-Olivier Malaterre for being my supervisor in France, and to the Irrigation Division of Cemagref for giving me the opportunity to do this practical training in Pakistan.

I thank Amer Iqbal, Secretary, for formatting this document.

List of Abbreviations

CCA	: Culturable Command Area
Cemagref	: French research center for agricultural and environmental engineering
Disty	: Distributary (secondary canal)
D/S	: downstream
ENGREF	: Ecole Nationale du Génie Rural, des Eaux et des Forêts
FSD	: Full Supply Depth
GCA	: Gross Command Area
IIMI	: International Irrigation Management Institute
IMIS	: Irrigation Management Information System
I&PD	: Irrigation and Power Department
ISRIP	: International Sedimentation Research Institute in Pakistan
ITIS	: Information Techniques for Irrigation Systems
PID	: Proportional, Integrated, Derivative
RD	: Reduced Distance (= 1000 ft)
SDO	: Sub-Divisional Officer
SE	: Superintending Engineer
SFP	: Steady Flow Period
SIC	: Simulation of Irrigation Canals
U/S	: upstream
USTL	: Université des Sciences et Techniques du Languedoc
XEN	: Executive Engineer
WAPDA	: Water And Power Development Authority

Conversion of Units

1 acre = 0.4047 ha
1 cumec (cubic meter per second) = **35.31** cusecs
1 cusec (cubic foot per second) = **28.32** l/s
1 ft (foot) = **0.3048** m
1 square foot = 0.0929 m^2

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1. INTRODUCTION

1.1. Irrigation in Pakistan

Agriculture plays an important role in Pakistan's economy. It accounts for 26% of the GDP, provides 80% of the overall value of exports, and employs 54% of the labour force (Siddiqi, 1994). Since the climate is arid to semi-arid and annual evaporation far exceeds rainfall over much of the cultivated area, irrigation is a necessity, and 75% of the cultivated area is irrigated.

With a command area of over 16 Mha, the Indus Basin Irrigation System is one of the largest contiguous irrigated systems in the world. It encompasses the Indus River and its major tributaries, three major reservoirs, 19 barrages or headworks, 12 link canals, 46 canal commands, and over 107,000 watercourses. The total length of the canals is about 60,000 km (ibid.).

In this system, the water is diverted from main canals to branch canals, then to distributaries and watercourses. It was designed a century ago as a gravity flow, run-of-the-river system with an objective of extensive and equitable use of water : the water has to serve as large an area as possible, and to be distributed equitably to sustain as large a rural population as possible at low cost. It is a supply-based system, i.e. the water allocation and distribution is essentially controlled by the supply at the head of the canal.

Agricultural production €or three major crops (wheat, rice and sugar cane) has been stagnant for the past 20 years whereas the population growth remains at a high rate of about 3%.
Initially authorized €or 50-75%, cropping intensities have now generally gone up to more than 125%. Crop water requirements can no longer be fulfilled by canal water alone, and the farmers exert tremendous pressure on this supply. Ground water resources also have increasingly been used through the development of tubewells (Kuper et al., 1994).

1.2. Purpose of the study

Studies undergone in many irrigation schemes in the Punjab have shown that the performance of this system is poor in terms of delivering equitably surface water resources (Kuper and Kijne, **1992,** World Bank, **1993).** Moreover, water is not distributed according to the official water allocation and scheduling. The regulation of the canals is left to a great extent to local operators, who receive only occasional orders from managers. This local staff react to local variables, unaware of future perturbations and targets, and generally do not take into account the effects of their operations on the downstream portions of the canal. This 'local control' has been shown to have a negative impact on the fluctuations entering the system, amplifying them as the operations increase in number towards the tail of the system (Kuper et al., **1994).**

The present study aims at improving the water distribution to secondary canals, by proposing alternative rules of operations at the main canal level. These alternative scenarios are proposed within a given strategy of water management.
The underlying hypothesis **is** that an improvement at the main system level will have a positive effect on crop production.
The relation between operations at the main canal level and crop production is assessed with the help of studies at different levels:

- at the distributary level (secondary canal), an undergoing study focuses on the effect of maintenance on the water distribution. It will also **look** at the effect of fluctuations in the discharge at the head of a distributary in terms of equity of water distribution to tertiary outlets.

- at the watercourse level (tertiary canal), Barral **(1994)** developed a model to assess the canal water supply at the farm level. Given the tertiary channel inlet discharge and the water allocation system (which is a roster of turn), it calculates the volume of canal water delivered at the-farm inlet.

- at the farm level, models are developed to assess the effect of canal water supply on crop production (Strosser and Rieu, **1994).**

This study is a part of this global approach, which will link operations at the main canal level to crop production.

1.3. Objectives and approach of the study

Many studies undergone in the Fordwah Branch system have shown that the system performs poorly (Kuper and Kijne, 1992, Van Essen and Van der Feltz, 1992, Jacobs and Schoonderwaldt, 1992, Rivière, 1993). Moreover, the performance of Chishtian Subdivision was shown to be impeded by operations inside the Subdivision (Kuper et al., 1994). The question then stands whether it is possible to improve the performance of this Subdivision by intervening on the operational side.

1.3.1. Objectives

Definition of the problem: the performance of Fordwah Branch has been shown to leave scope for improvement, and to be influenced **by** operations inside the system.
The main objective of this study is to *identify the way in which alternative management in Chishtian SubDivision can improve water delivery to distributaries in the Fordwah Branch Canal system.*

This main objective implies several research questions:

1. What are the official and actual strategies for water management?
2. What are the actual operational rules, in case of routine management as well as in case of emergencies
3. What are the physical limits of the system and what is the scope for improvement in the present management
4. What criterion or indicator to use to quantify the improvement

1.3.2. Approach of the study

This study is limited at the main canal level, and aims at analyzing the operations of Fordwah Branch in Chishtian Subdivision, by studying it from an hydraulic point of view.

The choice was made to use a hydraulic model to simulate the hydraulics of this canal. This will provide a useful tool to test some situations. **As** the study focuses on operations inside the system, a regulation module will be used to simulate manual operations at hydraulic structures. This regulation module will have to be calibrated in order to be able to realistically represent actual operations.

A diagnosis of actual management will be needed to be able to propose new rules of operations, that will be simulated and evaluated with the computer tools.

A field test of a scenario will be the final step of the study

The study will therefore take the following steps:

- Diagnosis of the system, to determine the hydraulic and management constraints and dysfunctionings, from a qualitative point of view. This will give an understanding of the operational rules, necessary step to be able to propose and test alternative scenarios.

- Calibration of a hydraulic model to simulate the flow in the canal. This calibration is followed by a validation process in unsteady state, to check the reliability of the calibration of the model.

- Development and calibration of a module to simulate manual operations of cross structures and offtakes. Once these two calibrations are performed, present situation can be simulated, and new rules of operations can be tested.

- Development and simulation of alternative scenarios, taking into account different options at a local level, i.e. concerning the rules of regulation at control points. The global level, i.e. concerning the water scheduling in the system is supposed to be fixed.

- Field test of a selected scenario, in collaboration with the Irrigation Department.

2. RESEARCH LOCALE

2.1. General description

The command area of the Fordwah-Eastern Sadiqia area is located in the southeast of the Punjab, Pakistan. It is bounded by the Sutlej River in the northeast, by the border with India in the east and by the Cholistan desert in the southeast.

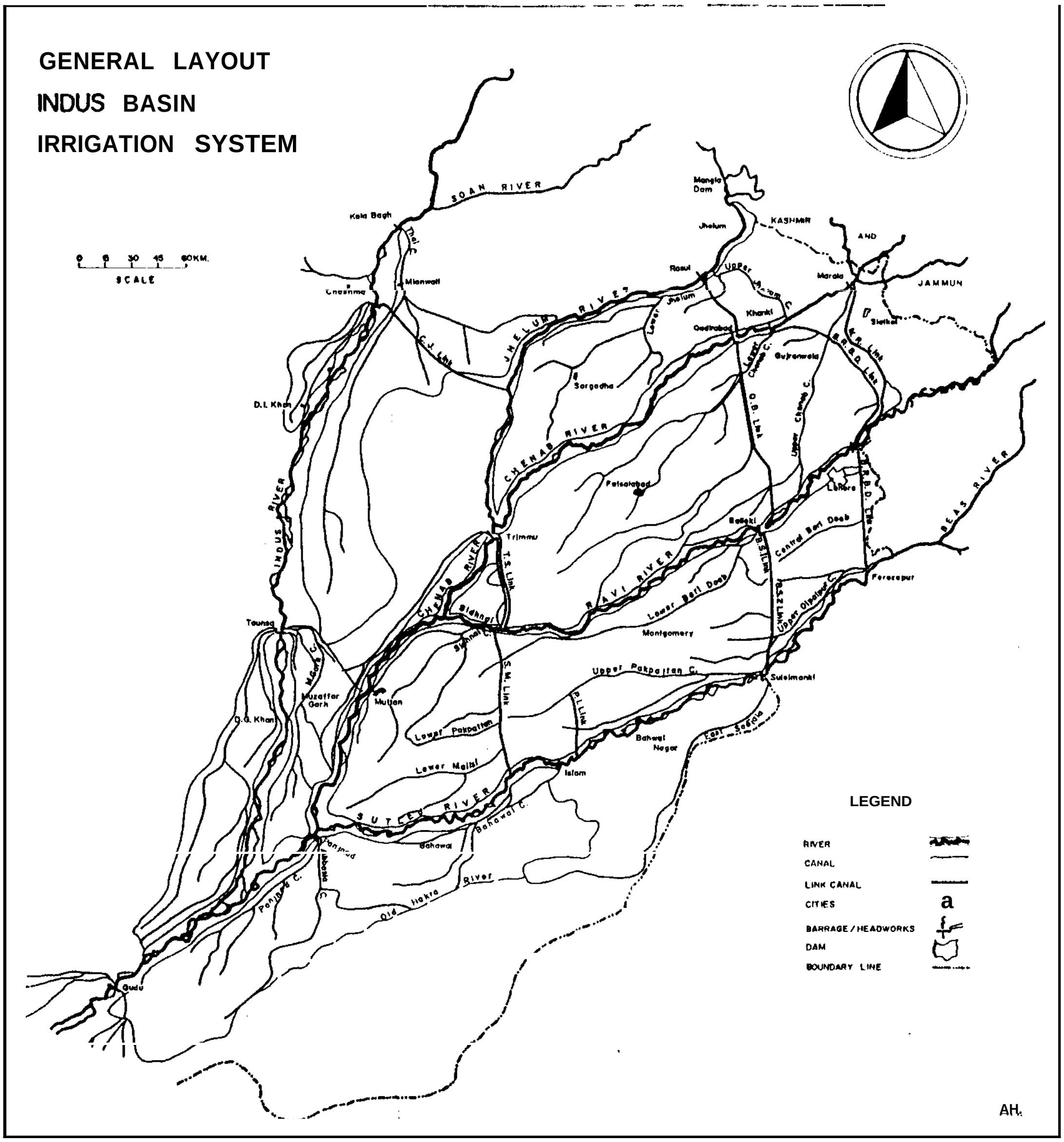


Figure 1: Map of the Indus basin irrigation system

Suleimanki Headworks is a large barrage on the Sutlej River, built in the 1920s by the British. The barrage has an average width of 600 meters at the head, and an average depth of 3 meters. Its capacity is about ***25.5*** millions of m^3. Three main canals offtake from this barrage, the Fordwah and Eastern Sadiqia Canals on the left bank, and Pakpattan Canal on the right bank. Fordwah Canal splits in two branches at RD 44.8 [1], Fordwah Branch and Macleod Ganj Branch.

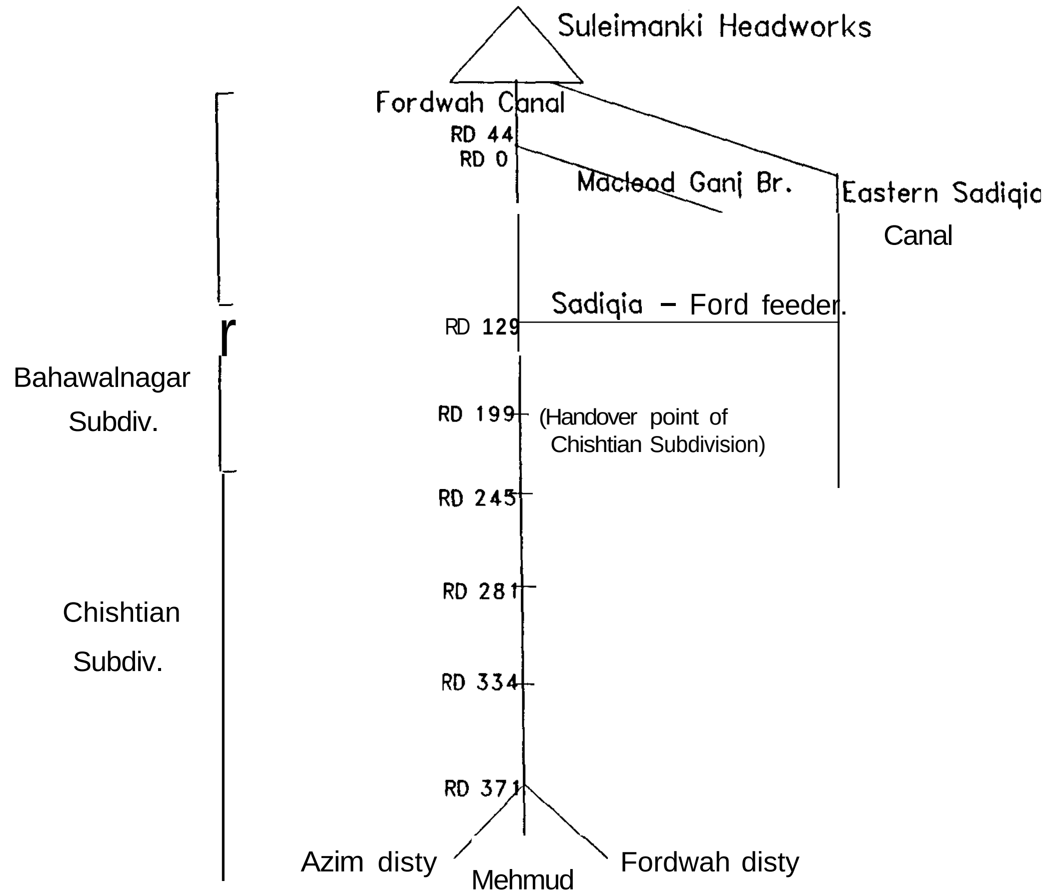


Figure 2: Physical scheme of Fordwah Division

Limits between sections on the main branch in Chishtian Subdivision, and also between Subdivisions in the Division are indicated in the figure.

[1] RD = Reduced Distance from the head **of** the canal, in 1,000 feet. 1 RD = **304.8** meters.

General features of the system (Kuper and Kijne, 1992):

The Fordwah Eastern Sadiqia area covers 301,000 ha, out of which 232,000 ha are culturable commendable. The climate is semi-arid with annual evaporation (2400 mm) far exceeding annual rainfall (260 mm). Most of the rain fall occurs during the monsoon period, between July and September. The highest temperatures occur during May and June (between 30 and 50 degrees Centigrade), and the evaporation rate is about 13 mm/day. The cropping pattern is cotton, rice and sugar cane in the Kharif season (summer flood season, from April to October) and wheat and fodder in the Rabi season (dry winter). This area is part of the Sutlej Valley Project undertaken in the 1920s and completed in 1932. Both Fordwah and Eastern Sadiqia canals receive their supply from link canals since partition, as the water from the Sutlej River is used by India. In Kharif the supplies stem mainly from the Chenab River, and in Rabi they come from the Mangla reservoir.

2.2. Organizational setup

There are different levels of management units in the Punjab Irrigation System. The Zone is the biggest unit, and a Chief Engineer is in charge of it. The Circle is the next unit, headed by a Superintending Engineer (SE). Then comes the Division, which is the basic irrigation unit, headed by an Executive Engineer (XEN). It is divided in Subdivisions, headed by Assistant Executive Engineers called Sub-Divisional Officers (SDO). The Subdivision itself is divided into different Sections, each of them headed by a Sub-Engineer.
The Fordwah Main Canal System is administrated by the Fordwah Division, itself divided into three Sub-Divisions:

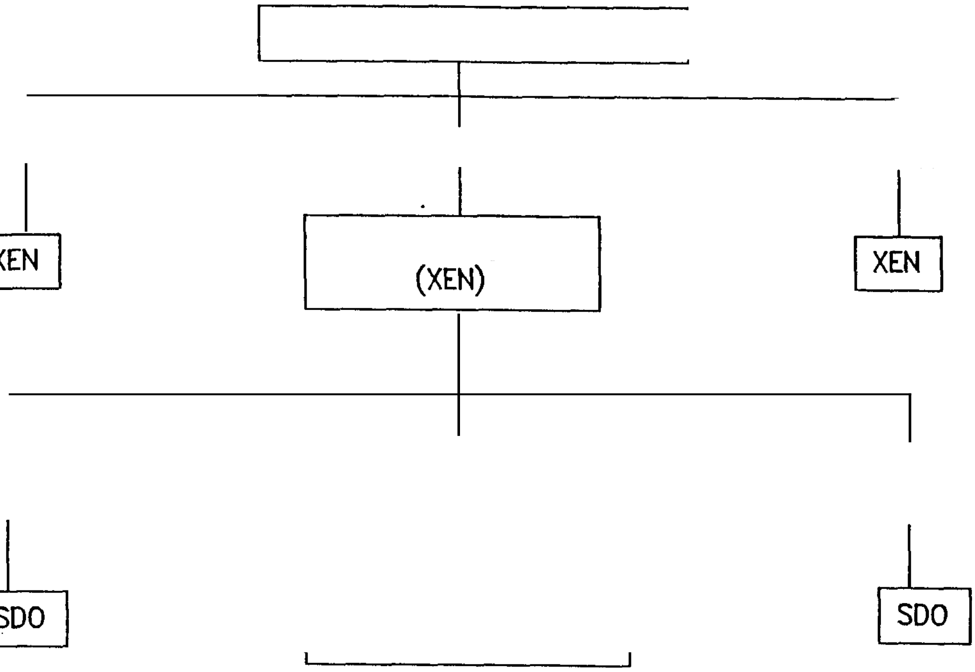
- Minchinabad Subdivision (Fordwah Canal RD 0 to 44.8, Fordwah Branch RD 0 to 129 and Macleod Ganj Branch)
- Bahawalnagar Subdivision (RD 129 to 245 of Fordwah Branch)
- Chishtian Subdivision (RD 245 to 371 of Fordwah Branch)

There is a fourth SDO who is in charge of the public tubewells in the Division.

Table 2.2.1: Command areas in Fordwah Division (ha).

Sub-divisions	**Minchinabad**		**Bahawalnagar**		**Chishtian**	
Command Area	**GCA**	**CCA**	**GCA**	**CCA**	**GCA**	**CCA**
Perennial	2900	2630	4539	4250	23477	20625
Non-perennial	86057	79392	48289	42726	49923	45598
Total	88957	82022	52828	46976	73400	66223

Note: GCA=Gross Command Area, CCA=Culturable Command Area



This study focuses on the Chishtian Subdivision, which is located at the tail of Fordwah Branch. This Subdivision is divided into five Sections.

Table 2.2.2 Sections of Chishtian Subdivision (source: Kuper and Kijne, 1992).

Section	Area of authority, Fordwah Branch (RD)	Area of authority, distributaries (RD)
Takht Mahal	245-281	Mohar, Daulat (0-63), 3L, 4L, Phogan
Chak Abdullah	281-334	Jagir, Masood, Shahar Farid (0-47), Soda
Chishtian	334-371	5L, Mehinud, Fordwah (0-64), Azim (0-52)
Khetngarh		Daulat (63-tail), Shahar Farid (47-tail)
Hasilpur		Fordwah (64-tail), Azim (52-tail)

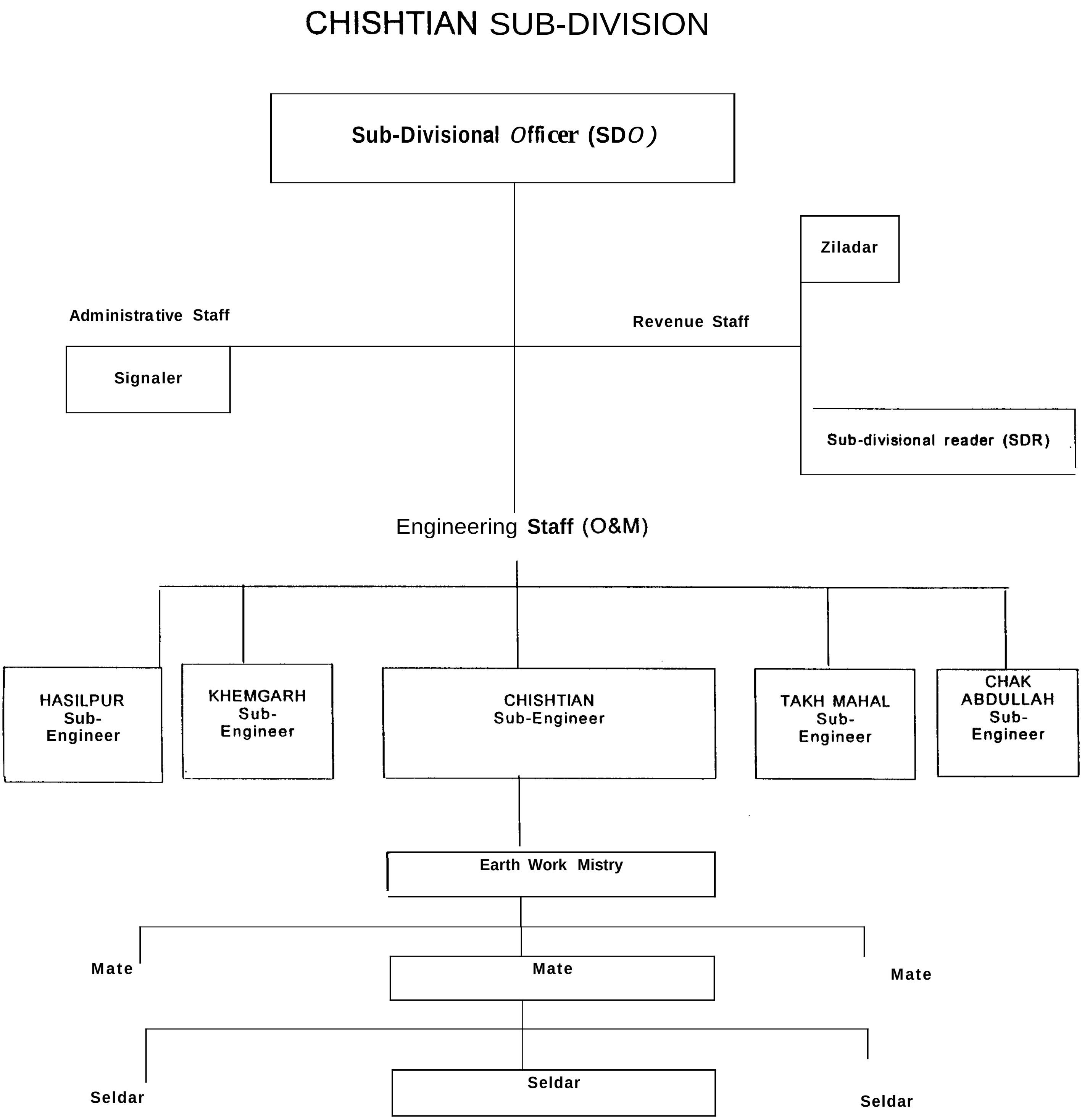


Figure 4: Organizational chart of the Chishtian Subdivision (source: Riviere, 1993)

3. Fordwah Branch in Chishtian Subdivision

Fordwah Branch has a total length of 123 km, 38.4 km of which are in the Chishtian Subdivision (from RD 245 to RD 371, tail of the main branch). The design discharge at RD 199, handover point of Chishtian Subdivision is 36.3 m^3/s or 1282 cusecs [2]. Its width is about 35 m at RD **199,** and **15** m at the tail. The average slope is 0.020%, or 1/5000. The water is delivered ***to*** the secondary network (distributaries) thtough 14 offtake structures (gates, culverts or open-flumes), and there are also 19 direct outlets supplying watercourses. The total CCA (Culturable Command Area) of Chishtian Subdivision is 67,597 ha. Out of the 14 distributaries, 9 are non-perennial, which means that they are entitled to water during the Kharif season only, and ***5*** are perennial, with supplies the year round. As the water available in Rabi season was not sufficient to fulfill water requirements, the designers decided to give water only to some areas during this season. The denominations of perennial and non-perennial were also given to areas in regards with their propention to waterlogging. The area prone to waterlogging were labelled non perennial, and would receive a maximum of three allocations in Rabi to save the wheat crop (Kuper and Kijne, **1992).**

The data given in the table below are updated for **93/94** season. The water allocation is calculated from actual values: it is the design discharge divided by the area of land to be irrigated, that is the CCA multiplied by the intensity factor (70% for non perennial channels, 80% for perennial channels). This gives the water allowance at the head of a distributary. To get the water allocation at the head of the water course, seepage losses have to be deducted from this value.

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[2] **1 cusec = 1 cubic foot/s = 0.028 m^3/s, or 35.31 cusecs = 1 m^3/s**

Table 2.3.1.: Characteristics of distributaries in Chishtian Subdivision

Name of distributary	RD	CCA (ha)	Status[3]	Design discharge (m^3/s)[4]	Water allocation (l/s/ha)
Daulat	245+600	13,255	NP	5.9	0.64
Mohar	245+600	1,706	NP	1.0	0.84
3L	245+600	1,166	NP	0.5	0.61
Phogan	267+700	949	NP	0.5	0.75
Khem Gahr	281+000	2,032	NP	0.85	0.60
4L	281+000	840	NP	0.45	0.77
Jagir	297+500	1,604	P	0.71	0.55
Shahar Farid	316+400	10,364	NP	4.3	0.59
Masood	316+400	3,004	P	1.0	0.42
Soda	334+000	3,935	NP	2.2	0.80
5L	348+800	357	P	0.11	0.39
Fordwah	371+600	14,847	P	4.5	0.39
Mehmud	371+600	813	P	0.2	0.31
Azim	371+600	12,191	NP	6.9	0.81

The total required discharge to feed all the distributaries at their design discharge is therefore:
Sum of Qdesign of disties + Qdo + Qseepage = 29.12 + 1.34 + 3.22 = 33.68 m^3/s with

Qdo = discharge through direct outlets (measured in the field),
Qseepage = seepage losses (estimated with inflow-outflow method).

The design discharge at this point (36.3 m^3/s) is much higher than this value, indicating a possible overestimation of the design value for an unknown reason.

The water levels are maintained all along the canal from RD 199 to the tail by means of 5 gated

[3] P = Perennial, NP = Non-Perennial

[4] The design discharges of 3L and Jagir have been changed (former values: 0.65 for 3L and 1.1 for Jagir).

The water levels are maintained all along the canal from RD 199 to the tail by means of 5 gated cross-structures (or regulators) and 2 weirs. Most of the distributaries, at least the most important ones offtake just upstream of one regulator. Only 3 of them, Phogan, Jagir and Soda are not under the direct control of a cross regulator.

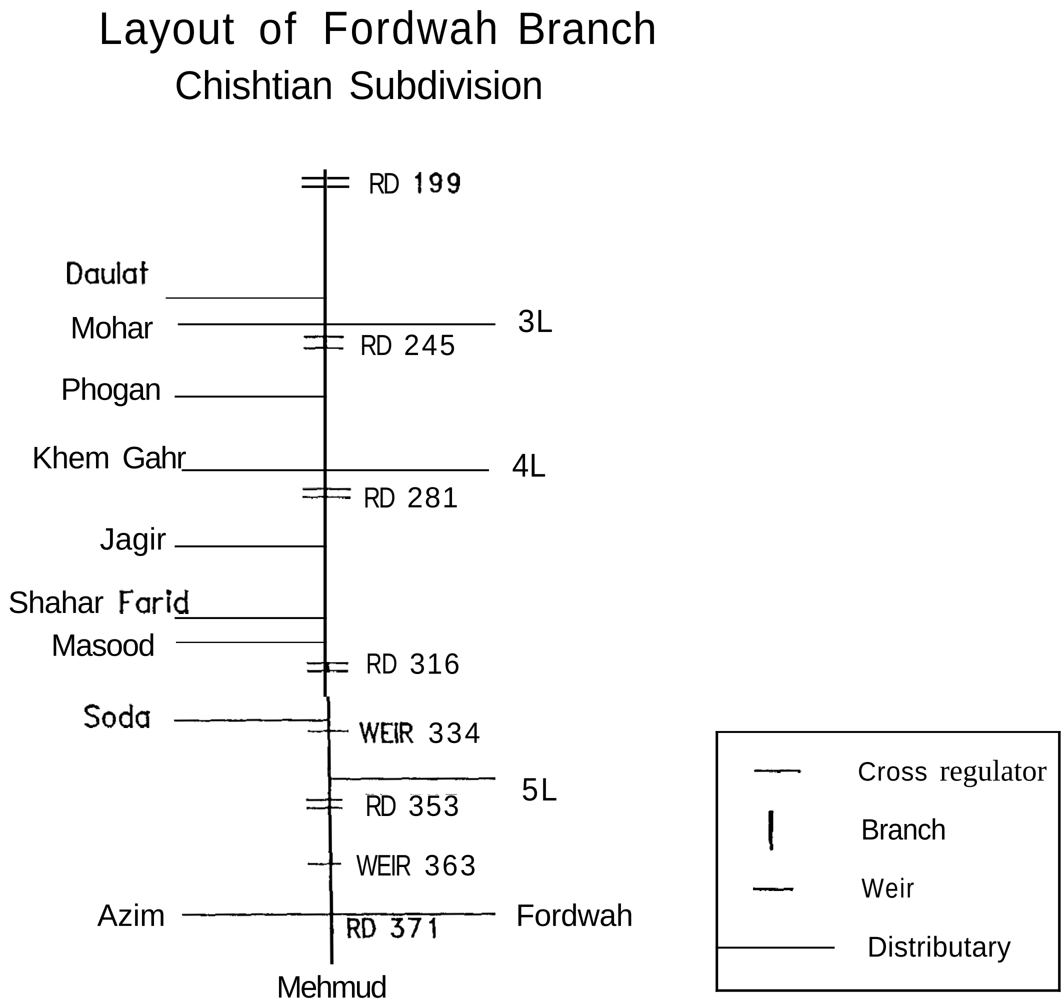


Figure 5: Layout of Fordwah Branch

3. RESEARCH METHODOLOGY

The methodology and the tools used in this study are detailed in this chapter.

3.1. Methodology

The use of a hydraulic model in improving manual operations of an irrigation canal is a rather recent approach, that had not been field tested. In the present study, the process was followed until the field test of a scenario, performed together with the manager.

Five steps were distinguished in the process:

I. Diagnosis[5] of the system:

1. Review of previous studies

A first diagnosis is necessary to define problems in the system that need to be addressed. This step was already done in our case, as many studies had already been done on the same system. Studies in Pakistan as well as in other countries (Sri Lanka, Indonesia) were used to define the objectives of the present study (see references).
It enabled us to delineate the boundaries for the study, and to define the approach of the study.

2. Monitoring and discussions with the managers

The monitoring provides quantitative as well as qualitative data. The quantitative data is given by physical monitoring, while the qualitative data comes from discussions with the managers.
The physical monitoring (discharge and water levels measurements, topography, dimensions of structures) enables the calibration of structures. It facilitates the further monitoring of the system, where discharges are computed from water levels. The intensive monitoring periods, where the levels are measured every hour should not be limited to day time. **A** lot of operations occur also at night, at least in the system studied. Such a monitoring period can last for **3** or **4** days, and it gives an interesting basis for discussion with the managers.
The discussions with the managers and with operational staff are very productive if they are based on a real situation, observed in the field. We used visual tools to restitute the results of the measures. They were found helpful to clearly identify and rank the factors having an effect on the system. **A** first definition of the strategy and operational rules can be derived from these discussions.

[5] We use the term diagnosis as the identification of strengths, weaknesses, opportunities and threats of a system.

3. Calibration of the hydraulic model

The calibrated model provides supplementary outputs for the diagnosis, as it quantifies the capacity of reaches, the maximum discharge, the values of Manning coefficients, the minimum free-board. It also gives the points with a risk of overtopping as a first output with basic simulations (chapters 6.1 and 6.2). The physical limits for operations were derived from these data.

4. Calibration of the regulation module

A regulation module was developed to simulate manual operations, and to be able to simulate a change in those operations. This regulation module was calibrated for a set of operations monitored in the field. The calibration gives an evaluation of manual operations at hydraulic structures, which is also used in the diagnosis (chapter 6.1). Once calibrated, this module can give an a priori evaluation of manual operations, and provides a way to simulate actual as well as alternative manual operations.

5. Evaluation of present management

The present management was then evaluated, at the main canal level: indicators were defined to evaluate the performance of water distribution to secondary canals. An indicator was defined from a study at a distributary level, and other indicators used in previous studies at the main canal level were also selected (see chapter 3.4). The evaluation showed that a scope for improvement was possible in the operations of structures. This diagnosis step gives the managing and physical constraints of the system, shows the weak points in the system and the possible scope for improvement (chapter 4). This diagnosis is also refined with outputs of the further advancements of the study, during the evaluations especially.

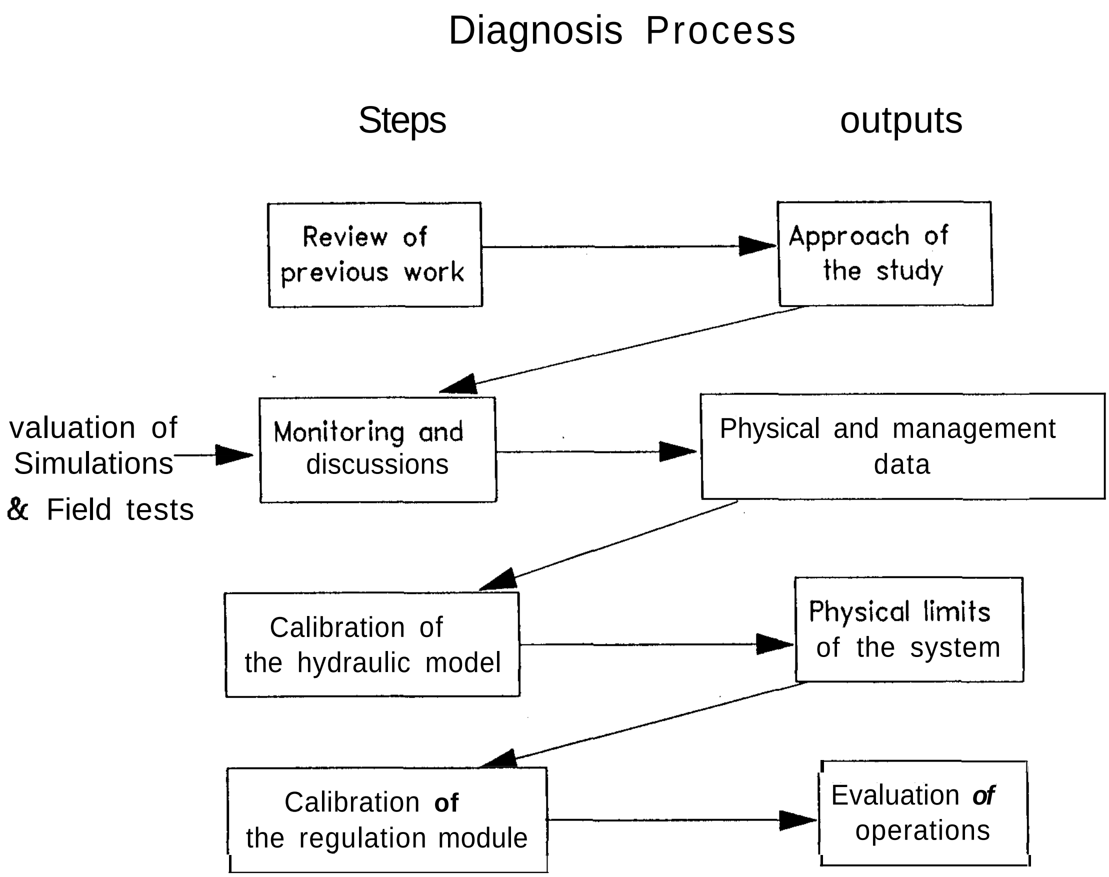


Figure 6: Schematization of the diagnosis step

II. Development of scenarios

1. Identify typical operational problems

This stems from the diagnosis of the system, and especially from the discussions with the managers.

2. Define typical situations representing these problems

As we are going to use a model to represent the functioning of the system, either typical situations can be simulated, or situations measured in the field can be used. These situations need to be defined precisely (inflow and strategic option) and will be used to simulate the different scenarios.

3. Propose and elaborate alternative scenarios

They should take into account the limits determined in the first step, and address the typical problems identified (chapter 5). These scenarios are to be discussed with the managers, especially if implementation is envisaged.

III. Simulation of alternative scenarios:

1. Simulate scenarios

Use the hydraulic model and the regulation module to simulate the defined scenarios on the different situations (chapter **6.3).**

2. Evaluate the scenarios

With the indicators defined in step 1.5. New rules can be tested if the first ones are not performing well enough. At this stage, results can be presented to the managers, to have a feedback from them, and implicate them in the next steps.

The process goes on until positive results are obtained

IV Field est of a] scenario:

1. **A** scenario is selected

Among the scenarios simulated in the third step. To select a scenario, the criteria to be used can be: performance in the simulations, interest of the managers, possibility of implementation.

2. Field test

It has to be done together with the manager, to provide a common basis for further discussions. Monitoring is done during the field test, to be able to evaluate it (chapter **6.4).**

3. Evaluation

It has also to be done together with the manager, to see if an implementation is possible, or what improvements need to be done.
The outputs of this field test can be taken into account for a possible refinement of the scenario.

V. Implementation:

This step depends on the manager, it is the implementation of the ested scenario for rou ine management The researchers can provide technical assistance during this phase.

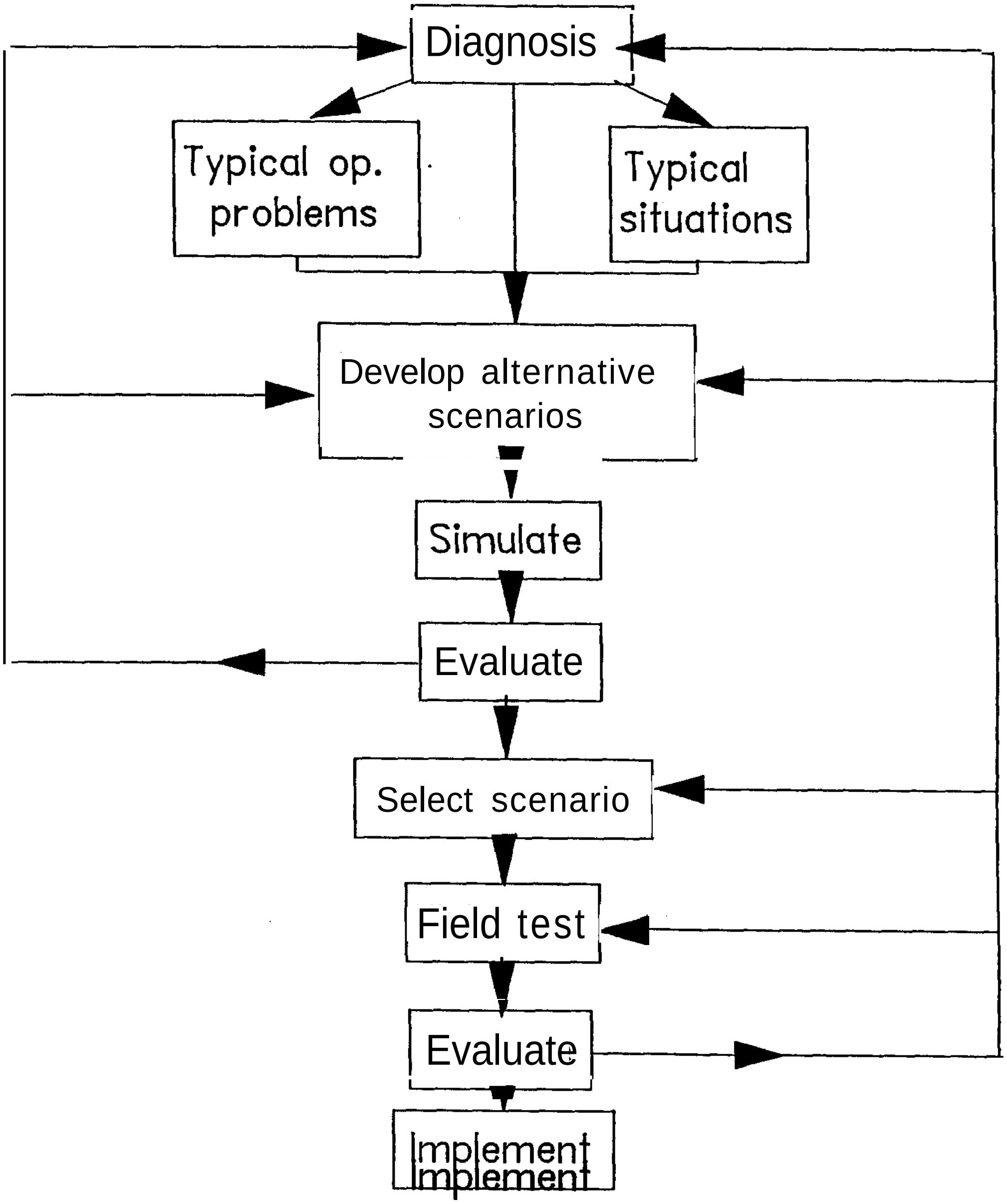


Figure 7: Schematization of the methodology

3.2. Description of **SIC** model and regulation module

3.2.1. SIC model

SIC, "Simulation of Irrigation Canals", is a mathematical flow simulation package developed by Cemagref that enables to model the hydraulics of irrigation canals, and simulate their functioning. This needs quite a lot of accurate data, because 'accuracy of results always depends on accuracy of data'.

The hypothesis used in SIC are the hypothesis **of** unidimensional hydraulics in canals:

- the flow direction is rectilinear, so that the water surface can be considered horizontal in a cross section,

- transversal velocities are negligible, and the distribution of pressure is hydrostatic.

Therefore, only unidimensional and subcritical flow will be simulated.

SIC is build around three main computer programs, TALWEG, FLUVIA and SIRENE, which generate topography, calculate steady flow and unsteady flow respectively.

Those three programs are part of three modules, namely Topography, Steady Flow and Unsteady Flow modules, respectively.
Description of the three modules (Cemagref, 1992):

- **Topography module** (Unit **I):** the topography and geometry of the canal are entered and processed for the calculations. The topography data are generated in ASCII format and saved as a .TAL file.

- **Steady Flow module** (Unit 11): the hydraulic data necessary for a steady flow computation can be entered and modified (.FLU file). Water levels, discharges and openings for offtakes and cross structures are computed using the Manning-Strickler formula for a given inflow. SIC uses special equations for gates and weirs, modifying the Cd according to flow conditions.

Equation of the water profile in a reach:

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

with:

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

where **A** = cross-section area of flow
g = acceleration of gravity
H = total head
k = 0 for lateral inflow, 1 for lateral outflow
q = discharge per unit length
Q = volumetric rate of discharge
Sf = friction slope
n = Manning's roughness coefficient
R = hydraulic radius

For **solving** 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. With these data, the water surface profile is integrated step by step starting from the downstream end.

- **Unsteady Flow module** (Unit III): the water levels and discharges are calculated using the Saint Venant equations for varying inflow and operations. Those data are stored in an ASCII .OUV file. The initial water surface profile is provided by Unit II (Steady Flow module). This module allows to test various scenarios of water demand schedules and operations at the head works **or** at control structures.
Barre de Saint Venant equations:
Continuity equation:

$$\frac{\partial A}{\partial t}+\frac{\partial Q}{\partial x}=q$$

Momentum equation:

$$\frac{\partial Q}{\partial t}+\frac{\partial Q^2/A}{\partial x}+g.A\frac{\partial z}{\partial x}=-g.AS_f+kqV$$

with **the** same notations **as** above, and
z = water elevation
V = mean velocity

These equations are solved numerically by discretizing them according to the Preissmann's scheme, and using a double sweep method.

Below is schematized the structure of **SIC** model.

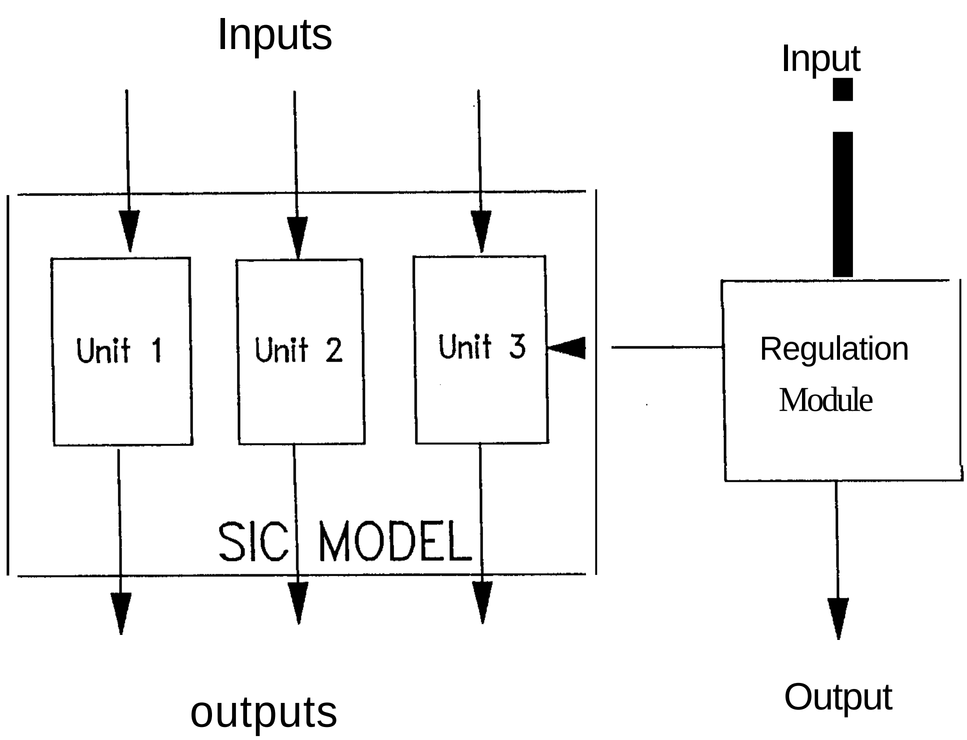


Figure 8: Diagram of SIC model

The data needed to model a canal with **SIC** are therefore:

Physical data:

- geometry of the canal (sample of cross-sections representing the canal)
- description and dimensions of structures (cross-structures, gated regulators, weirs, and offtakes), crest levels, width, height of gates, diameters,
- downstream boundary condition for offtakes and tail of the system.

Hydraulic data:

- inflow at the head
- operations at structures

SIC allows to display the results of the computations as graphs or as numerical results.

The **outputs** of **SIC** are:

Unit **I:** data on the geometry of cross sections.

Unit **11:** discharges, water levels, volumes, velocity, Froude number, gate or weir positions for offtakes, cross regulators settings.

Unit **III**: discharges, water levels, gate or weirs positions for offtakes, performance indicators, time lags.

Regulation module: time and amplitude of operations.

3.2.2. Regulation module

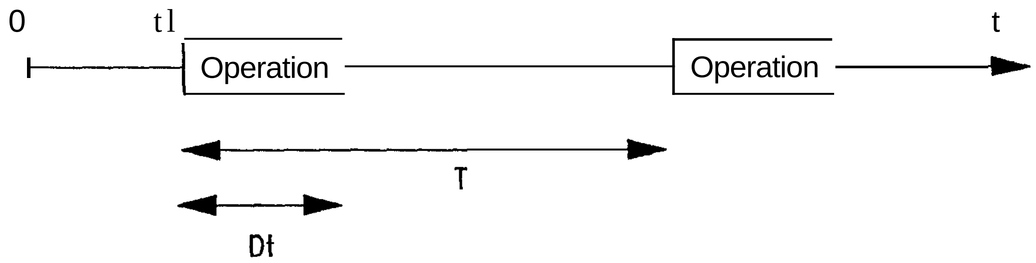
SIC package allows to regulate a modeled canal by operating the cross structures with the programming of a regulation module. This module computes the opening at each time step and imputes them to the regulators. This module is integrated in the Unsteady Flow module of **SIC** (Unit **III**).

The regulation module used for this study is derived from the one developed by Malaterre (1989). This module simulates manual practices with the **use** of various parameters to match the practices observed in the field.

Five parameters are used to simulate the time schedule:

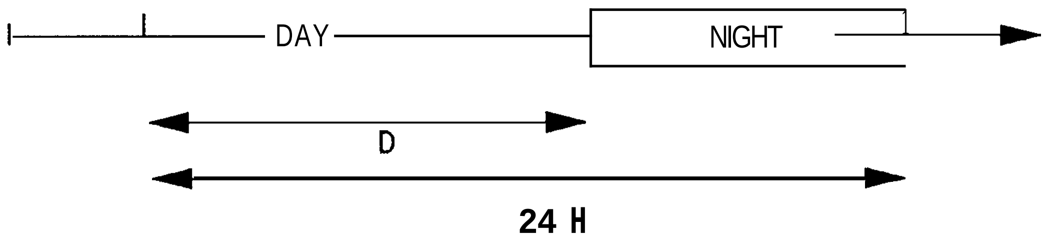
Three to define the timing of operations:

1. time of the first operation (tl)
2. time between two operations (T)
3. time of an operation (Dt)



Two to define the night period:

4. time of beginning of day work (t0)
5. duration of a day work (D)



Six parameters are used to set the boundaries for operations:

6. lower limit of intervention (low)
7. upper limit of intervention (up)
8. maximum amplitude for closing operation
9. minimum amplitude for closing operation
10. maximum amplitude for opening operation
11. minimum amplitude for opening operation

The limits of intervention are given in centimeters around the targeted upstream level. The module considers that the target is reached when the upstream level is within the range [FSD-low, FSD+up]. The maximum and minimum amplitude are limits set to operations, so that the computed changes of openings are not too small or too large.

The three last parameters are the amplification coefficients for opening and closing operations (it is a multiplicative coefficient applied to computed openings to get the observed openings), and the targeted upstream level (called FSD, Full Supply Depth):

10. amplification coefficient for opening operation
11. amplification coefficient for closing operation
12. targeted water surface elevation (FSD)

All these twelve parameters can have different values for each regulator.

This module simulates an operator who checks periodically the upstream level at his point, and opens or closes the gates according to some rules, only if the observed upstream level is not within the range defined around the target upstream level FSD.
The time coefficients are here to reproduce real timing of operations.

Rules for the cornputation of openings:

We used two methods in this study for the computation of the opening of a gate. **As** the operations of gauge readers at cross regulators were found very good in terms of stabilization of the upstream water level, the computation of openings for cross regulators in the regulation module called "Gateman" uses a hydraulic rule at a structure.

a. Evaluation of the operations at the regulators using a hydraulic formula

The method is derived from the one used by Malaterre **(1989).** It enables to predict the gate opening of a cross regulator, knowing the actual opening, actual upstream and downstream levels and the targeted upstream level.

Description of the method:

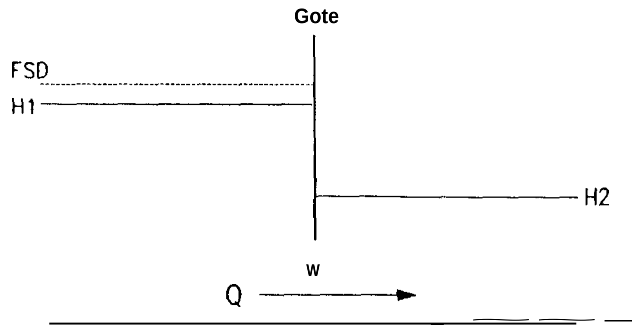


Figure 9: Cross regulator

Suppose that the upstream level (Hl) is lower than the targeted upstream level (FSD) when the operator observes the water levels. The operator will close the gate of the regulator to raise this upstream level to FSD.
By doing this, he will provoke an upstream and a downstream wave.

The assumptions are as follows[6]:

- the canal is in steady state
- the discharge through the structure is constant
- the downstream level at the structure is constant

[6] The assumptions made here are not always realistic, as in the Punjab, canals are rarely in steady state, with **a** quite important number **of** operations. The time needed for stabilizing the canal after an operation is almost never attained. Nonetheless, the results obtained while comparing predicted and real values are quite good, allowing **us** to validate this method.

We assume that the canal is in steady state, therefore, after a certain time, if no other operation is performed elsewhere in the canal, a new steady state will be reached, where the upstream level will be at **FSD,** with the downstream level remaining the same as before operation, the discharge being constant.

The regulation module "Gateman" uses SIC equations, that take into account all types of flow conditions (see Cemagref, **1992).** SIC equations give the discharge through a gated structure as a function of the opening, the upstream water level and the downstream water level.

If we write this equation in our case, we get:

Before operation:

$$Q = f(w, H_1, H_2)$$

After operation:

$$Q = f(w', FSD, H_2) = g_{FSD, H_2}(w')$$

with Q = discharge through the gate
w = opening before operation
w' = opening after operation

Then w' can be computed by determining the value:

$$w' = g^{-1}_{FSD, H_2}[f(w, H_1, H_2)]$$

This is done using a dichotomy method.
This w' is the theoretical value for the new opening, given the assumptions we mentioned above. This will be referred to below as the "computed opening".

<u>Example of application with the classical formula for a submerged cross regulator:</u>

This method was used by Malaterre for the operations in Kirindi Oya Right Bank Main Canal, Sri Lanka. It was considering only submerged cross regulators, as it was the case in this canal. It has the advantage to give an explicit formula for the computed opening.
The hypothesis however are more restrictive as the discharge coefficient is also supposed to stay constant, and free flow conditions are not considered.

If we write the classical hydraulic formula for a submerged gate before and after operation, we get:

Before operation:

$$Q=C_d\sqrt{2g}\,L\,w\sqrt{H_1-H_2}$$

After operation:

$$Q=C_d\sqrt{2g}\,L\,w'\sqrt{FSD-H_2}$$

with Q = discharge through the gate .
Cd = discharge coefficient of the gate
g = gravitational acceleration
L = width of the gate
w = opening before operation
w' = opening after operation

As Q and Cd are supposed to stay constant, we can compute w':

$$w'=w\sqrt{\frac{H_1-H_2}{FSD-H_2}}$$

We made the comparison between observed and computed openings for several operations at each regulator, using the gate operations ratio R, defined as:

R = w observed/w computed for an opening operation

R = w computed/w observed for a closing operation.

This ratio represents the ratio between real and required operations (according to the formula). Then, a ratio $R < 1$ will mean that the operator underestimates the amplitude of the operation, and a ratio $R > 1$ that he overestimates it, with reference to the computed opening.

b. Rule for the computation of the opening of an offtake:

To this existing module were added new rules of operation, and the possibility to operate an offtake just upstream of a regulator to regulate the discharge in the main branch.

This rule is based on a discharge balance.

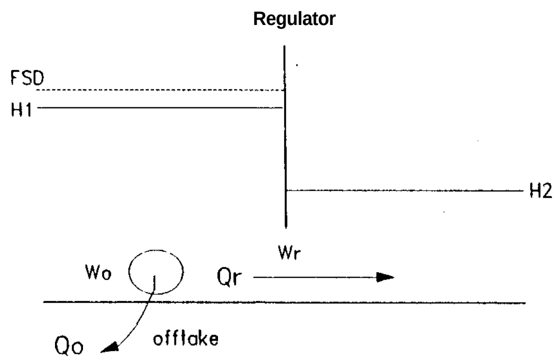


Figure 10: Cross regulator and offtake

The letters 'o' and 'r' refer to 'offtake' and 'regulator' respectively,

Suppose that the upstream level (H1) is lower than the targeted upstream level (FSD) when the operator observes the water levels. The operator will close the gate of the offtake to raise this upstream level to FSD.

We know the total discharge Qt arriving at the control point:

$$Qt = Qr + Qo$$

Let Qrt be the targeted discharge at the regulator (i.e. the discharge through the regulator when the upstream level is equal to FSD). If the upstream level is lower than FSD, this means that the discharge going through the offtake has to be reduced, to keep a full supply in the main branch.

Then, the new discharge that has to be passed through the offtake is:

$$\begin{aligned} Qo' &= Qt - Qrt \quad \text{if} > 0 \\ &= 0 \quad \text{otherwise.} \end{aligned}$$

We can then compute wo', the offtake opening that will allow this Qo' with the upstream level equal to FSD.

To summarize this method:

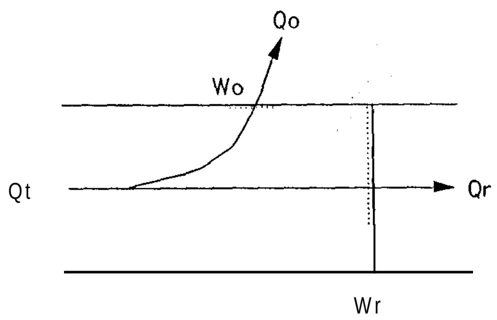


Figure 11: Before operation

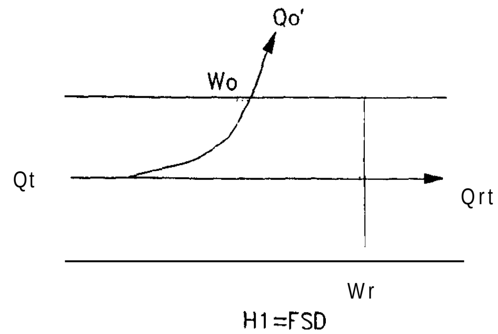


Figure 12: After operation

The operation is therefore determined by a change in the upstream level, but the new opening is computed according to the discharges in the regulator and the offtake.

The regulation module Gateman operates the offtakes according to some coefficients similar to those used for the operation of regulators:

1. Minimum amplitude for opening operation
2. Maximum amplitude for opening operation
3. Minimum amplitude for closing operation
4. Maximum amplitude for closing operation
5. Lower limit of intervention
6. Upper limit of intervention
7. Amplification coefficient for closing operation
8. Amplification coefficient for opening operation

Two coefficients have been introduced, to enable the change of priorities:

9. Opening to apply to the offtake to have the target discharge with H1 = FSD when there is a change in priority

10. Opening to apply to the regulator to have the target discharge with H1 = FSD

The timing of operations is assumed to be the same for operations for both a cross regulator as well as an offtake.

c. Rule implemented in case of a rotation:

We define a "control point" as a location where the level in the main canal can be regulated. It is therefore a location where there is a gated cross structure. **A** control point also encompasses the distributaries offtaking directly upstream of the cross regulator.
When priorities are introduced, it is done with reference to control points:
At one control point, the priority is either to

- an offtaking distributary at this point
- the main branch downstream.

* If a distributary at the control point is in first priority, then the cross regulator is operated, in order to maintain a constant upstream level. The gate of the distributary is not operated.

* If the main branch downstream is in first priority, then the gate of the distributary is operated, while the gates of the cross regulator are not moved. The discharge going through the regulator remains constant if the operator manages to keep the upstream level constant by operating the gate of the offtake.

* For a change in priority, we need to give the gate settings requited to feed the distributary or the branch downstream with the required discharge, assuming that the upstream level will stay around FSD. This has to be given in the .REG file (coefficients **9** and 10 for offtakes). This file also contains the timing of priorities at a control point (see annex C.2).

3.3. Setting up a hydraulic model

3.3.1. Field survey

As mentioned above, a hydraulic model requires a lot of data. This data was provided by a field survey, during which cross-sections of the canal, dimensions and location of structures (cross regulators, weirs and offtakes) were measured. (see **annex A** for dimensions of structures in Fordwah Branch, Chishtian Subdivision)

This data is called 'physical data', as it gives the physical description of the system. One must be aware that this data is subject to a lot of changes: siltation or scouring may modify the geometry of the canal, changes in crest elevations, widths of structures may also happen. This is why this data has to be updated periodically.

For this study, 97 cross-sections of Fordwah Branch have been taken, which gives in average one cross section every 600 m. These cross sections have been entered in the model, and all simulations have been done with these measures of actual geometry of the canal,

3.3.2. Field measurements

Once the physical data has been entered in the model, field measurements need to be done to enable calibration. The structures have first to be calibrated, and also the downstream conditions for distributaries.

Once the structures are calibrated, the discharge is derived from knowing the upstream water level, downstream water level (if the structure is submerged), and the gate opening. This is why the field measurements consist in monitoring water levels and gate openings.

The water levels and gate openings at each structure are measured with reference to a White Mark. The elevation of these White Marks are measured with reference to the crest of the given structure, and white marks readings allow to determine the upstream water level above crest, the downstream water level above crest, or the gate opening (see Figure 12).

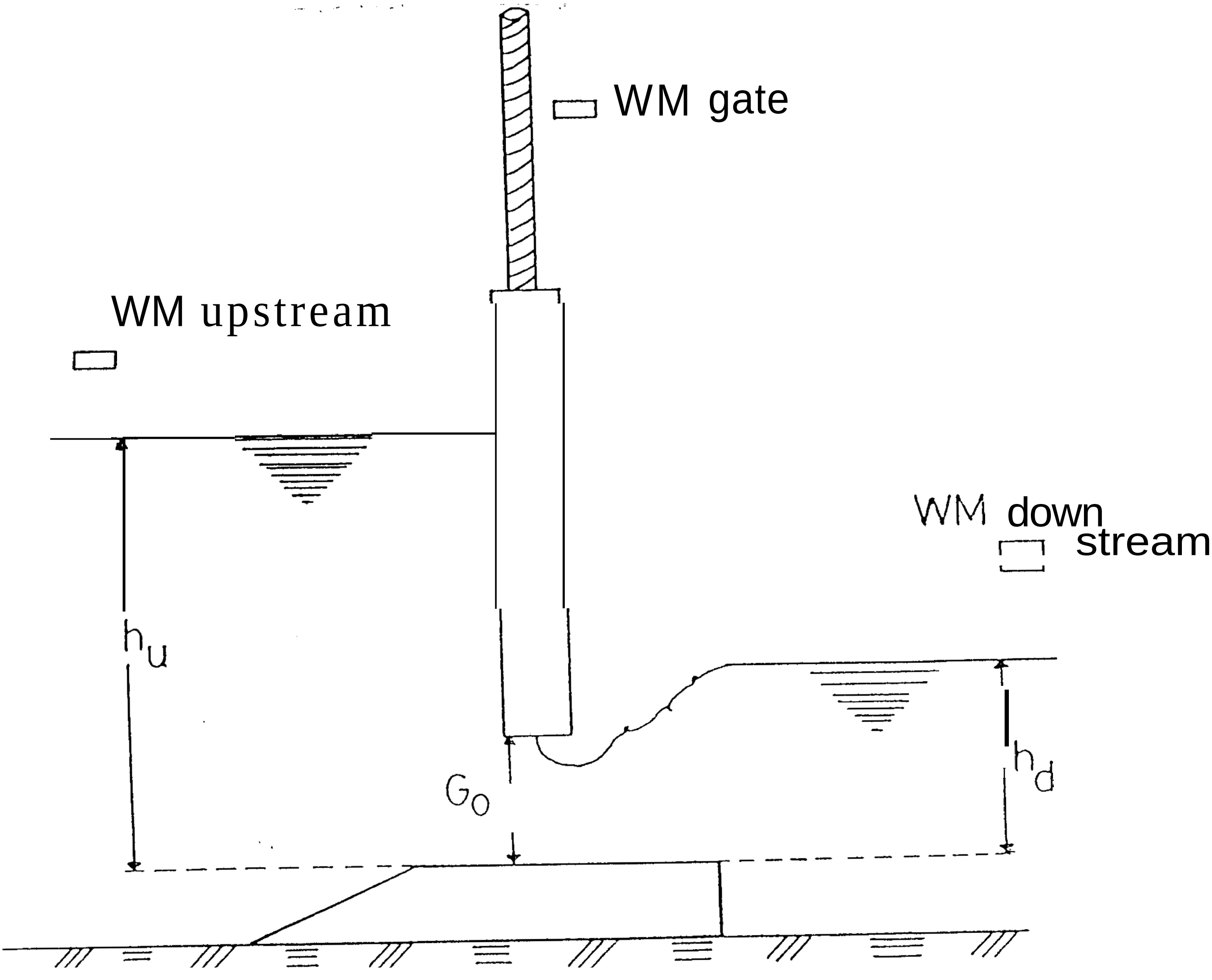


Figure 13: White Marks at a structure

3.4. Model calibration

3.4.1. SIC calibration

To calibrate the hydraulic model for a canal, we have to determine the following unknown variables :

1) Discharge coefficients for cross structures
2) Discharge coefficients for offtakes
3) Downstream boundary conditions for offtakes and for the tail of the system
4) Seepage losses
5) Manning-Strickler coefficients for reaches

Some of these coefficients are derived from field measurements, and others are adjusted by running the model so that simulated values and measured field values are within an acceptable range of accuracy.

As some work has already been done last year on this canal with the same package, the calibration work was not done all over again. For the new model, new cross sections have been entered, and the dimensions of the structures and offtakes have been fixed. We assumed that the discharge coefficients for offtakes computed for the last calibration were good as well as the downstream boundary conditions, and used the same values in the model. These values were also checked during a calibration training organized by IIMI and ISRIP between the 28th of May and the 6th of June (ref.). Therefore, only seepage losses, Manning-Strickler coefficients and discharge coefficients for cross structures were calibrated with the new set of measures.

The calibration of the model has been done following three steps:

- seepage losses calculation
- determination of Cd for cross structures
- calibration of Manning's roughness coefficient

The calculation of seepage losses and Manning-Strickler coefficient requires a steady flow period, during which the canal is in stable state.
A steady flow period (SFP) has to have a minimal duration, which is the time necessary for a wave to pass through the system considered. In other words, the duration of a SFP has to be greater than the response time of the system.

The timelags for each reach of the system studied have been determined with the use of the model calibrated last year. To do this, we simulate a step of 1 m^3/s in the discharge at the upstream end of a reach, and we compute the time of arrival of this wave downstream of this reach. The time lag was defined as the time when 50% of the wave (elevation) reaches the tail of the considered reach. These computed values are close to what was observed in the field (see simulations in unsteady state in the validation part). Those values were also checked after the calibration of the model,

Table 3.4.1: Timelags in Chishtian Subdivision, with a discharge of 25.45 m^3/s at RD 199.

Reaches (between 2 cross regulators)	Time lag computed in SIC
D 199 - D 245	4h00
D 245 - D 281	3h30
D 281 - D 316	3h30
D 316 - D 353	3h00
D 353 - D 371	3h00
TOTAL	**17h00**

Calibration process

Data collection:

The data used for the calibration of the model in steady state consisted of a set of water levels monitored hourly during 72 hours from 2 to 5 June 1995 at 9 different points along the canal. These nine points consisted of the six control points (D199, D245, D281, D316, D353, D371) plus 3 offtakes (Phogan, Jagir, Soda) located in the middle of the reaches 2, 3 and 4.

The model was cut into two parts, as no SFP was long enough to calibrate the whole canal. The separation was done at the regulator at RD 353, as it is the only free flow structure inside the subdivision. **As** the discharge coefficients (Cd) for submerged cross structures appeared to be very variable with the gate opening, it was difficult to have a good accuracy on the discharges at these points. It was not the case for gated offtakes, as the downstream condition had already been calibrated, allowing us to know the discharge with a good accuracy.
The calibration process was then carried out for the two separate models. In those two models as well as in the final model used, the direct outlets were defined as offtakes with an imposed discharge in SIC. The discharges through these outlets were measured during IIMI's calibration training, and as their description was not complete (no sill elevation), we assumed that their discharge was not varying much with time. The total discharge taken by these outlets amounts to 1.34 m^3/s.

Seepage calculation:

The seepage was first estimated by inflow-outflow method with the field values (discharges computed from water levels, openings and Cds). Then, the model was run and the levels were matched by adjusting the Manning-Strickler coefficients, and the Cds. The discharges at the tail of the two models were then used to see if the seepage was too high or too low. The accuracy on seepage computation by the instant seepage test is given in annex.

Determination of discharge coefficients for cross-structures (Cd) and calibration of Manning's roughness coefficients:

The calibration module of SIC was used to calibrate discharge and Manning coefficients. The discharge coefficients for the cross regulators at RD245, 281, and 316 were quite difficult to determine, because they appeared to be very sensitive to gate openings. The values obtained are not exactly the outputs of the calibration module, but are adjusted after a validation in unsteady state, when the gates are operated quite often.

Validation in unsteady state

The process of verification under unsteady state was used to slightly modify some coefficients after calibration in steady state: the objective was to get the best correspondence between simulated and real values during an unsteady flow simulation. This process is delicate, as a lot of coefficients have an influence on the hydraulic state of the canal, and some of them are changing with time (SIC adjusts the Cd of the gates with the submergence ratio). It is therefore very important to diagnose possible causes when a discrepancy is detected. Then, small changes in concerned coefficients can be tested by running the steady flow tnodule of SIC, comparing the results with field data, and then run a simulation in unsteady state. When a calibration in steady state has been performed, it is the reference point for the changes done in unsteady state. The changes are accepted in steady state when they do not cause a change in water level or in discharge more than the acceptable range defined. The range used for this calibration was of 5 cm for the levels, and 10% for the discharges going into the distributaries. The discharge in the canal was checked at the two free flow structures in the system, at RD 353 and RD 371.

Summary of the process of validation in unsteady state:

1) Run the model in unsteady state with the monitored openings

2) Compare the results with field values (levels, and discharges at free flow structures)

3) Diagnose the cause of the observed difference
4) Change the given coefficient in the .FLU file (Unit II), and run the Steady Flow module to check if the difference in levels and discharge is within the defined acceptable range of variation

5) Run again the Unsteady Flow module to see the effect of this change.

This trial-and-error process is performed until a satisfying level of accuracy is reached, Limitation:

The limitation of this method is that it needs a steady flow period preceding the unsteady flow period. If it is not the case, then the process of going back to the steady flow unit to check a change in unsteady flow is not possible.

As it is very difficult to have a good steady flow period in some canals where gates are operated very frequently, such a calibration has to be done in collaboration with the Irrigation Department, to try to limit the gate operations.

3.4.2. Regulation module calibration

The calibration of the regulation module was done in two steps:

- analysis of field data (taken during the **3** day monitoring period) together with the outputs of the interviews of gate operators, allowing to determine a range of values for coefficients
- calibration of coefficients by trial and error (simulations in unsteady state).

1. Interviewing the gate operators during a monitoring period is very important for someone interested in operations. During the **3** day period, this gave very valuable outputs in terms of the understanding of their actions, for specific operations. The analysis of the field data has to be done with the outputs of the interviews. This will give approximate values for the coefficients used in the regulation module.

2. The simulations in unsteady state are used to modify some coefficients that also take into account the errors of the model. The errors come from two different sources:

   - errors generated by SIC (see verification of the calibration)

   - errors stemming from assumptions made in the regulation module that are not verified.

Calibration process:

The calibration process is done for each regulator separately, imposing on the others the real operations measured in the field.
Therefore, the simulated operator will have to react to the same fluctuations as the real one (if the hydraulic model is accurately representing reality), and not to the actions of other simulated operators, which would multiply the inaccuracies.

1. First, the following values are fixed:

- beginning of day work,
- duration of day work,
- time of an operation,
- maximum and minimum amplitude for opening and closing operations,

The other coefficients are estimated. The gate operation ratios are computed as the average of ratios for the monitoring period. All these values are derived from field observations and interviews.

2. Then, the last coefficients to be adjusted are:

- the lower and upper limits of intervention,
- the target upstream level (FSD),
- the time between two operations.

This is done by trial and error by running the unsteady state module of SIC linked with the regulation module. The first step of the calibration process gives an idea of the average value of these coefficients. The upper and lower limits of intervention are derived from interviews of gauge readers. The time between two operations is a coefficient taking into account the error generated by the way the module computes the openings: this module looks at the upstream level at one moment to determine the opening, and does not take into account the speed of variation of the water level, or any other kind of information (see the description of the method and its hypothesis). This means that it is not able to react well to a brutal change in water level, except if it operates more often.

Procedure for the simulations:

The time between two operations, the limits of intervention , the amplification coefficients and the FSD are set to the values derived from the first step of the calibration process. Then a simulation is carried out, with the same inflow as during the monitoring period, and the same operations at offtakes. The following diagnosis grid is applied, and coefficients are adjusted following this grid. The simulation and diagnosis are carried out again after having changed the coefficients in the .REG file (ASCII file containing the coefficients for the regulation module). The process is repeated until a satisfactory level of accuracy is reached.

Diagnosis grid:

- if there is a time lag between an observed and simulated operation, it means that the regulation module did not act at the right moment:
    * if he operated the gate later than the real operation, and if the simulated water level is effectively dropping or rising at this moment, the time between two operations has to be decreased to enable the module to react in time to this change.
    * if the simulated operation occurs before the real operation, there might be a problem in the calibration of the hydraulic model (value of the Manning-Strickler coefficient,. ..).

* if the level is not changing at this moment, this might come from a problem in the calibration of the hydraulic model, but also from the effect of operations done before: if some operations have been eliminated because they were not justified, it will affect the local state of the canal (the upstream level will be lower, for example, if the regulator has not been closed), and this might also affect the time of response to a wave (in our case, a positive wave will first have to fill the storage capacity left by the low level, and the level will rise some time later, explaining the time-lag).

if the simulated operation is larger than the real operation (taking into account the global amplitude of operation in response to a wave), it might stem from two reasons:

* the **FSD** is too low (if the operation is an opening of the regulator), or too high (if it is a closing operation)

* the amplification coefficient for closing or opening operations (depending on the operation) is too high

- if the simulated operation is too small, take the symmetric reasons of the above case.

- if an operation in the field is not simulated by the module, it might stem from three reasons:

  * the operation is not justified hydraulically
  * the upper or lower limit of intervention is too big
  * the variation in upstream level simulated by the unsteady flow module of SIC does not correspond to reality. The calibration of the hydraulic model is therefore in cause.

- if an operation simulated by the module is not observed in the field data, it may stem from two reasons:

  * the upper or lower limit of intervention is too small
  * problem in the calibration of the hydraulic model.

Criteria used for the calibration of the regulation module:

Preference was given to a good accuracy in the upstream level, together with a good accuracy in the global amplitude of operations in response to a wave, rather than trying to simulate the exact openings. The time of the first operation in reaction to a wave was also taken as a criteria. This provides a way to calibrate the cross regulators. The calibration for offtakes operations was not possible, as the data did not show enough justified operations. The coefficients at an offtake were assumed to be the same as the ones used for the regulator, except the coefficients for gate operations, that were assumed to be equal to **1.**

3.5. Simulate alternative scenarios

3.5.1. Preliminary definitions

Here are defined three important terms for our study:

A ***policy*** is a course of action adopted by a government, person or institution.
A ***strategy*** is a plan of action to implement a policy with a given set of resources.
A ***tactics*** is the procedure followed in the implementation of a strategy.

In our case study, the **policy** is the official rules of distribution of surface water, or the water allocation. It is defined by the Irrigation Department, by the Chief Engineer, or the Superintending Engineer. The **strategy** is the way this policy is implemented, i.e. the scheduling of water distribution. This is defined by the Executive Engineer, and Subdivisional Engineers in a Division. And the **tactics** is the local rules followed in the implementation of the strategy, i.e. the operations at hydraulic structures, performed by gauge readers who operate gates.

Water management	Institutional level
Policy	Ministry, CE, **SE**
Strategy	XEN, SDOs
Tactics	Gauge readers

Fordwah Branch Canal in Chishtian Subdivision can be considered as a system with inputs (control action variables), outputs (controlled variables), and perturbations (non controllable inputs) with the denomination used by control engineers.

- The inputs are the operations at offtakes and cross regulators, and the inflow in the canal. In the case **of** Chishtian Subdivision, the inflow can be considered as a non-controllable input (see present situation in chapter **4).**

- The outputs are the discharges supplied to the distributaries, and the associated water levels.

The management of this system can be separated in two levels:

- a strategic management, performed by the managers (XEN, SDOs). This is the way managers choose to distribute water to the secondary network, given the inflow at the head. It is the role of the supervisor in control engineering.

- a tactical management, performed by the gauge readers. This is the way local staff operate to implement strategic orders. This is the role of the controllers in control engineering.

We will work on the operational side, i.e. at tactical level, assuming that *a realistic and clear* ***strategy*** *has been defined*. This strategy can be in accordance with the official policy or not.

Then, given this strategy (that will be clearly mentioned for each simulation), we will test different tactical scenarios, i.e. different means to try to achieve the objectives defined in the strategy.

A scenario will therefore be defined as a **tactical option** implemented in the field. In other terms, a scenario corresponds to a choice of a set of controllers on the canal.

Tactical scenarios:

- scenario 0, Real operations: this scenario can be simulated in SIC with the real inflow patterns, for which we know the corresponding openings.
- scenario 1, Improved actual operations: this scenario is simulated with the calibrated regulation module, where hydraulically unjustified operations are eliminated. It can be simulated for regulators, for offtakes or for both.
- scenario 2, "Buffer" option: this scenario is based on the use of the buffer capacity of reaches, to temper the fluctuations of discharge.
- scenario 3, Information between gate ouerators: this scenario simulates the implementation of a communication system between gate operators. It was simulated for the two operators at the tail of the canal.
- scenario 4, Feed forward control: this scenario supposes that the manager has an intimate knowledge of his system, and has an estimation of future perturbations. He can therefore predict the gate openings and give corresponding orders to the gate operators.
- scenario 5, Automated nates: the gates are operated by automatic devices, controlled locally (see chapter 5 for a more detailed description).

These scenarios will be tested for different inflows and different strategies.

To test a scenario, we need to choose:

- a strategic option,
- an inflow at the head of the canal.

This will be called a **situation** (the **inflow** and the **strategic option** are given).

The situations were chosen from the diagnosis of the management of the system and are presented in chapter 5.

3.5.3. Performance indicator

This study is based on the hypothesis that fluctuations in water distribution at the main canal level have an influence on crop production. This hypothesis can be assessed with other studies that are done at the distributary and watercourse levels. The objective of an irrigation system at the main canal level is often to deliver a constant discharge to the distributaries during a certain period of time (at least a few days). A study undergone at the distributary level (Hart, 1995) will give an indication of the effect of variability in the inflow at the head of a distributary on the water distribution to outlets. From this study, a range of acceptability in the variation of discharge at the head of the distributary can be determined: the lower value is the discharge for which water does not reach the tail of the distributary, and the upper value is the discharge for which overtopping occurs. These values have been determined for Fordwah distributary with the use of the SIC hydraulic model (85 and 110% of design discharge).

This range of discharge is very dependent on the physical state of each distributary, and the results obtained for Fordwah distributary may not be directly extrapolated to other distributaries. The design discharge is not always a good reference in some distributaries that have been affected by siltation, or whose outlets have been changed. It would be preferable to use the actual target discharge. As clear targets were not available for Chishtian Subdivision, we assumed that the range of discharge determined for Fordwah disty was a good approximation for these upper and lower values, taking the actual design discharge as a reference.
We therefore take the actual design discharge as a reference, and we define the lower and upper value as 85 and 110% of the design discharge. This gives the range of acceptability for variations in discharge for our study.

With this range determined for each distributary, it is possible to define **indicators of performance** at the main canal level, to quantify the performance of the water supply to each distributary, in terms of delivering a discharge within a defined range of acceptability. This indicator will also be used to evaluate the different scenarios simulated in **SIC.**

It is defined as the percentage of time during which the discharge at the head of a distributary is within the range of discharge previously defined.

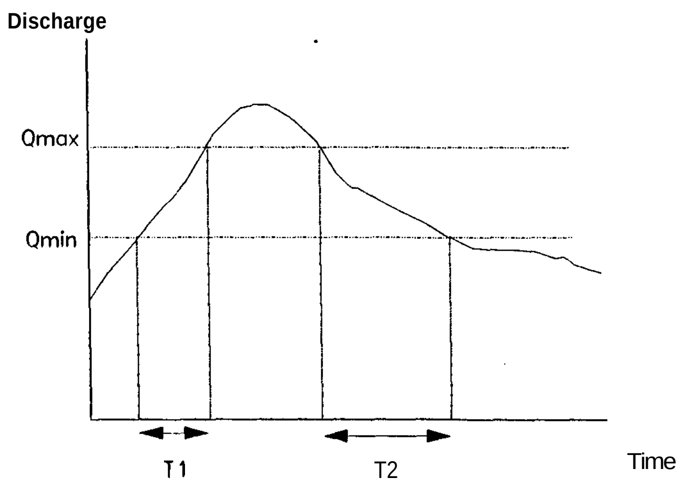


Figure 14: Example of the indicator used

In this example, the discharge at the head of a distributary is plotted over time; only the times T1 and T2 during which the discharge is within the acceptable range will be taken into account for the computation of the indicator.

Here, Qmax = 110%*Qdes and Qmin = 85%*Qdes.

This indicator was used to evaluate present situation and compare it with alternative scenarios.

To compare scenarios for a given situation, we also computed the difference between indicators; it gave the increase in percentage of time during which the discharge is within the defined range around Qdesign. **As** the effect is not the same for a small or a big distributary, the difference was also expressed in terms of volume, assuming the discharge was equal to the design discharge **for** each disty. This is called the ***'useful volume'*** delivered to a distributary, as the volume delivered when the discharge is below the lower limit does not feed all the distributary, and when the discharge is above the upper value, there is a risk of breaches.

To be able to evaluate and compare different scenarios for a given situation, we also defined an **aggregated indicator,** that gives the total amount of useful volume for all the distributaries. It is the sum of the useful volumes for all the distributaries in the Subdivision.

As the objective is also to minimize the duration **of** fluctuations in levels in the main canal, or to minimize the amplitude of fluctuations in discharge in distributaries, some important variables were taken into account in the evaluation of scenarios:

- number of operations
- fluctuations in U/S level at regulators
- discharge at important locations (at a disty in first priority, for example)
- time needed to reach the target .
- time of stabilization.

4. WATER MANAGEMENT IN CHISHTIAN SUBDIVISION

The management will be presented in three parts, as there are three different levels : how the system was designed to function, how it is officially managed, and finally how the official rules are implemented in the field.

4.1. Design concepts

4.1.1. Introduction

The basic general concept of irrigation in the Indus Basin is an equitable and extensive distribution of the available canal water; the flow is equitably' divided with relation to the area served (Siddiqi, **1994).** It is a supply-based system, which means that the water allocation and distribution are essentially controlled by the supply at the head of the system. Water is spread over as large an area as possible. For the Chishtian Subdivision, the design cropping intensity was **80%** for perennial canals, which means that **80%** of the CCA can have one harvest a year, or **40%** can have two harvests a year. Originally, **48%** should be cultivated during the winter season Rabi, and the remaining **32%** during the summer season Kharif. For non perennial canals, the intensities are **35%** each season (Ahmad et al., **1988).**

The water flowing in canals in the Punjab is heavily loaded in sediment, especially in Kharif season, when the water comes from the melting of snow in the mountains. This has led to problems of siltation, and the design of canals is focused at avoiding such problems.
Canals in the Punjab have been designed according to the concept of regime flow. **As** mentioned in the Manual of Irrigation Practices **(1961),** the 'regime flow' is defined as 'that state of a stream, flowing in self-borne alluvium, where there is neither silting nor scouring. Regime flow also postulates normal flow as a preliminary condition. '
This concept was first introduced by Kennedy in **1895,** under the term 'silt stable' flow. He developed an empirical relation between the critical velocity (non-silting non-scouring) and the depth of a canal, assuming that the silt carrying capacity of a channel was independent from the bed width. The problem of silt transport in irrigation canals in the Punjab was studied by many engineers/researchers, and Lacey's so called 'regime theory' was adopted by the Central Board of Irrigation in **1934** as the basis for designing silt stable alluvial channels (Bakker et al., **1986 p.9).** This 'theory' gives empirical relations to design a canal to avoid siltation or scouring. As the Sutlej Valley Project canals were constructed in the **1920s,** their design was based on the Kennedy-Lindley concepts (Wapda, p.***5).***
Logically, this implies that irrigation canals in Punjab are designed for steady state, as normal flow is a steady flow state. Nevertheless, it is specified in the manual that the discharge can vary between 70 and **110%** of the design, without causing much siltation or scouring.
Siddiqi also mentions the fact that the main channels (canals, branches) were designed to be self-

' **This is the official concept, which is not quite true, as some areas have different water allocations.**

Siddiqi also mentions the fact that the main channels (canals, branches) were designed to be self-regulating canals, to be managed with 'minimum human interference' (Siddiqi, **1994, p.40).** The maintenance work was to be done only at the distributary level, where there is no devices for manual control. We can therefore assume that the number of operations at the main system level (that is actually very high in Chishtian Subdivision) was designed to be minimum.

4.1.2. Design data on Fordwah Branch

The design drawings were compared with data from the survey carried out on Fordwah Branch in January **1995.**

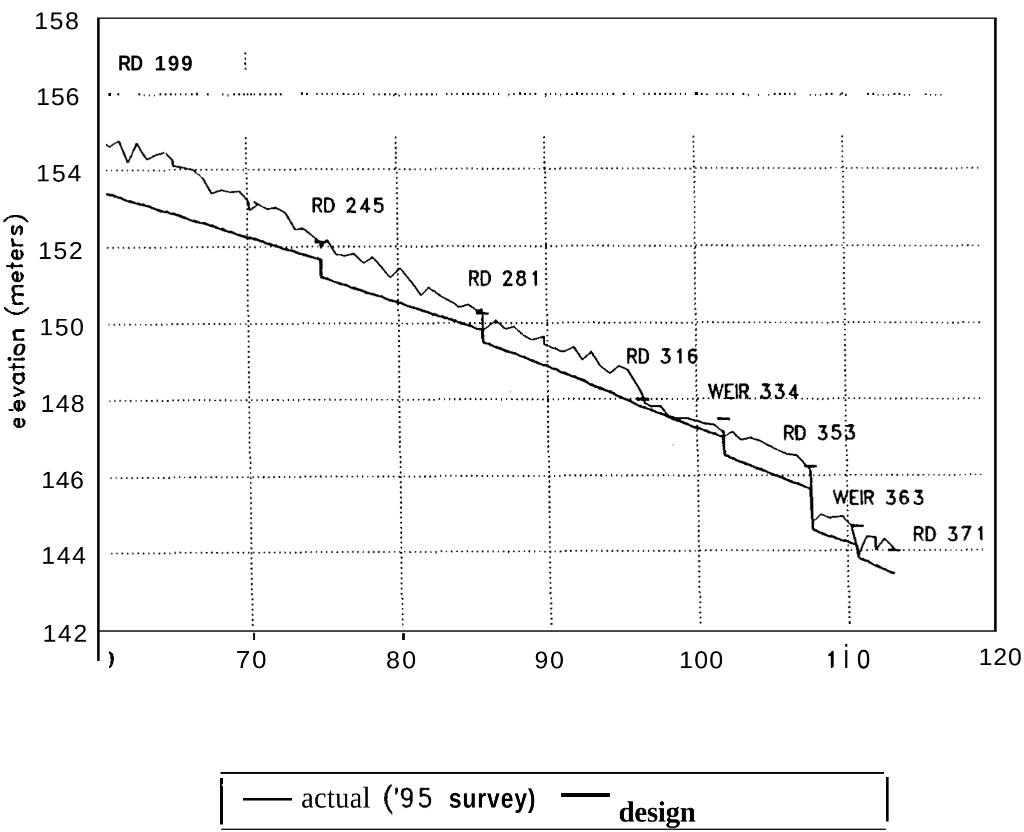


Figure 15: Longitudinal profile of Fordwah Branch

This graph shows clearly that there is siltation in the reaches of the canal. This siltation is partly due to operations, as during Rabi season, the canal is run at a low discharge (about **14** m^3/s, which is less than **40%** of the design discharge), therefore the velocity is lower, and siltation occurs. The canal has perennial and non perennial distributaries, which is one reason why it is

still run during Rabi even if the inflow is not sufficient to ensure a regime flow.
Also during Kharif, the average discharge is 11 m^3/s (about 400 cusecs) less than the design (Qdes = 36.3 m^3/s, or 1282 cusecs). **As** the water is much more loaded with silt at that season, siltation is likely to happen more at this time.

Capacity of reaches:

This siltation also has an influence on the operations in this system: the capacity of the canal has been reduced, **reducing also the margin of security for operations.**

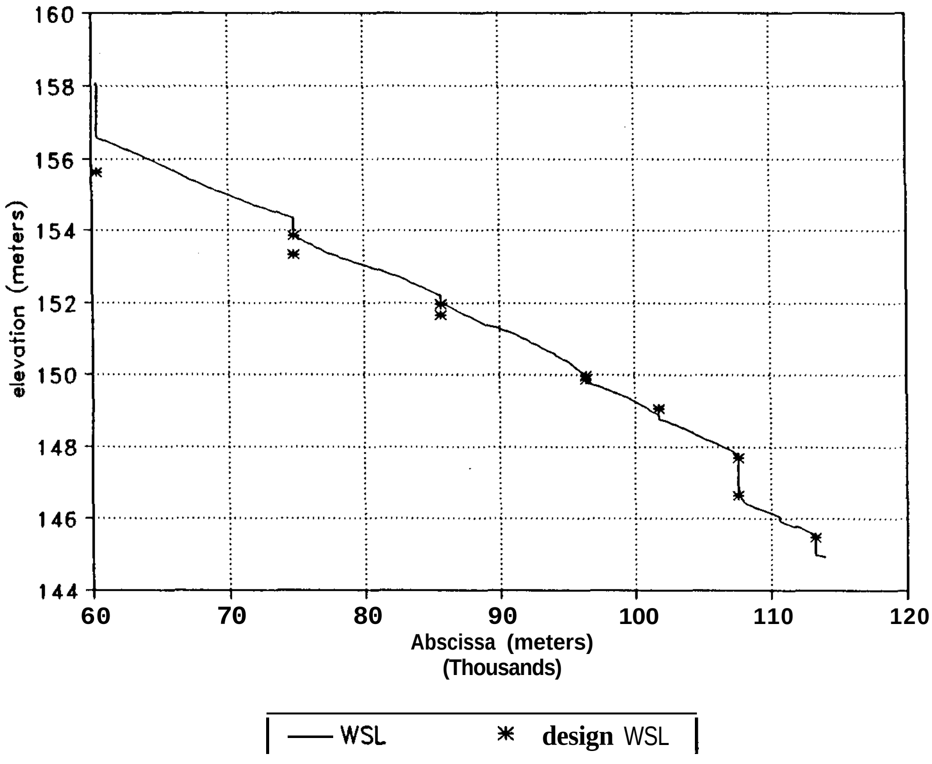


Figure 16: Water surface levels: comparison designlactual (Q=Qdesign)

The water levels are higher than the design water levels in the first reaches, that are the most heavily silted. This explains the loss of storage capacity in those reaches. In the fourth reach (RD 316 - 353), the water levels are lower than the design. This may come from an error in the design data that were available, or from an anticipation of the silting up of the bed in this reach. It seems to show that this reach was designed to have a big storage capacity.

The storage capacity of reaches at full supply level has been computed from the design drawings, and is given in the table below.

Reaches	Storage capacity	Free board
D199 - D245	916.6	76 cm
D245 - D281	444.9	76 cm
D281 - D316	431.0	76 cm
D316 - D353	370.2	76 cm
D353 - D371	182.5	76 cm

This table can be compared with the first outputs of the hydraulic model (chap. **6).** The capacity is much lower in the actual situation than in the design, especially in the first reach, which happens to be the most heavily silted. The free board is also reduced in actual situation, because of siltation, giving a smaller security margin for operations.

Cross-structures:

The submergence ratio for design conditions for the cross structures are given in the table below. **A** structure is considered as submerged when the submergence ratio, H2/H1 is higher than **0.67.**

Table 4.1.2.: Submergence ratio for design situation (at Full Supply Level) in Chishtian Subdivision.

Structures	Design H1 (ft)	Design H2 (ft)	Submergence Ratio	Actual conditions
RD 199	6.66	-1.33	-0.20	Free flow
RD 245	5.7	3.96	0.69	Submerged
RD 281	5.6	4.61	0.82	Submerged
RD 316	6.43	6.08	0.94	Submerged
WEIR 334	5.24	3.68	0.70	Submerged
RD 353	4.8	1.32	0.27	Free flow
WEIR 363	5.08	4.03	0.79	Submerged
Fordwah	4.4	3.59	0.81	Submerged
Azim	4.4	1.20	0.27	Free flow

It is interesting to see that the structures were not designed to function under free flow conditions at full supply level. Nonetheless, the falls that were present at the design could provide free **flow** conditions when the supply is **less** than the design. In present situation, the regulators at **RD** 245 and **RD** 281 are sometimes free flow, but **D** 316 is highly submerged.

The design drawing also gives the natural surface level **(NSF)** compared to the bed, **bank and** design water surface levels of the canal. **A** canal is said to be in 'cut' (respectively in 'fill') when the natural surface level is on average above (respectively below) the water surface level. Fordwah Branch Canal is partly in 'cut', partly in 'fill' from **RD** 199 to **RD** 334, and then in 'fill' down to the tail **(RD** 371). The seepage in this canal is therefore likely to be higher in the tail part (Wapda, p.46). The tail is also more sensitive to possible breaches, as they could be much more important than in the part in 'cut'.

Cross-sections:

A look at some actual cross sections compared to the designed ones shows the change that has occurred in the geometry of the canal. The canal has widened its bed in some portions, the banks have been damaged at numerous points, because of cattle crossing the canal. This is the case for the cross sections at **RD** 246 and 281. In general, the banks are higher in actual situation, except at **RD** 334 and 363 for the left bank. This is the result of the rise of water levels compared to the design: instead of removing the silt, the banks are raised to keep a sufficient free board. Siltation is clearly visible in many sections, **RD** 200, 246, 281 and 363. Scouring seems to have occurred in the section at **RD** 316, which might explain why the water levels are lower than the design in the reach downstream this point.

We must therefore be aware that we are dealing with a canal that is very different from the designed canal. With this geometry, it may not be possible to distribute water to the distributaries according to the design discharges. If the design discharges are not realistic targets for the water distribution, the prospect of improving actual situation should be based on another reference than the design. This is why we propose to base the reference on field values, when they are available.

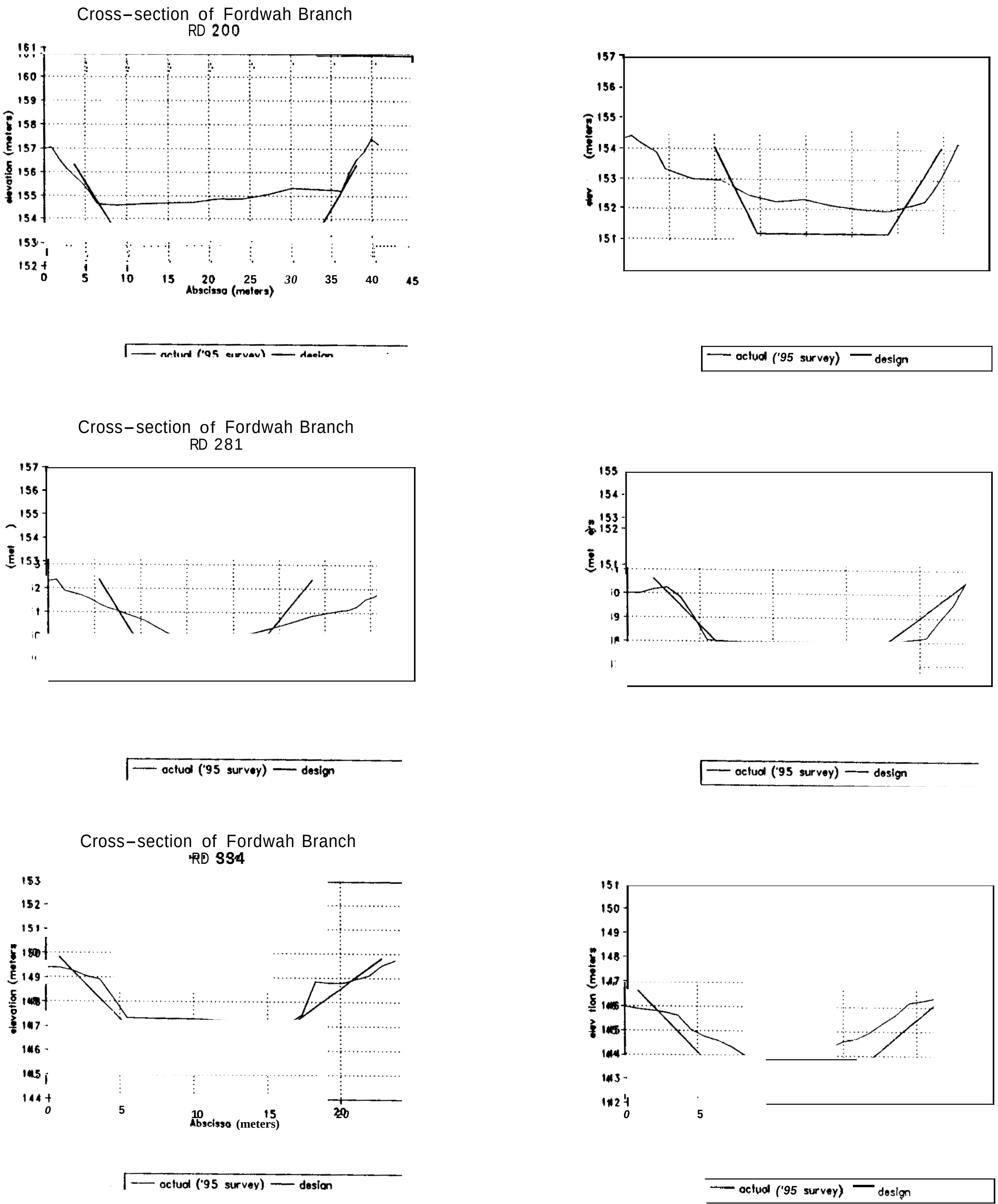


Figure 17 : Cross sections of Fordwah Branch

4.2. Standard operational procedures

Below are displayed the official procedures as they are described by the managers, which are different from the design procedures, and also from what is actually implemented in the field (see 'present situation'). These official procedures are taken from written data and oral interviews.

4.2.1. Water allocation

The allocation of water to the different Divisions, Subdivisions and distributaries is decided by the SE, XEN, and SDOs on the basis of indents. They can reduce or increase the water supply in main canals and distributaries according to the perceived demand in the command areas. The indent is a request made by SDOs, for the desired quantity of water in their subdivisions. Each SDO estimates the indent at the handover point of his Subdivision by summing the indents of distributaries, adding an estimation for direct outlets and seepage losses. The indent is first determined by the SDO at the tail of the Division, who passes it to the SDO upstream of his Subdivision. This process goes from the tail to the head of the system, with SDOs successively combining their indent with the one they receive from downstream.

In the Fordwah Division, the communication is made through telephone, and the final indent arrives to the XEN Fordwah, who is the indenting officer of Fordwah Division. This officer submits the combined indent for Fordwah Division to the XEN in charge of Suleimanki Headworks, who then tries to match the indents received from the three canals that take off from Suleimanki with the water allowances indicated by the Director Regulation, based in Lahore.

The indent point fixed in the Chishtian subdivision is at RD 199, handover point of the Subdivision.

4.2.2. Rotations

The discharge at RD 199 during Kharif 1994 was most of the time lower than the design discharge.

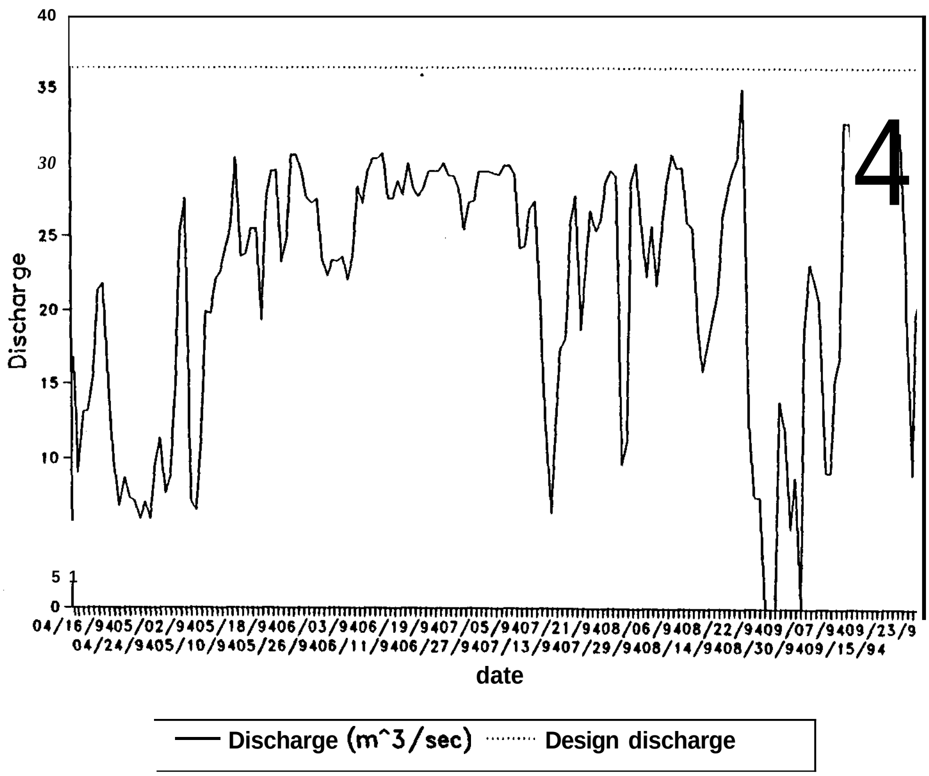


Figure 18: Daily discharge at RD 199, Kharif 1994, IIMI measurements

The supply comes mainly from the melting of snow in Kharif, and the storage is not sufficient to sustain the supply when needed. Furthermore, cropping intensities have increased dramatically since the design of the system; as the cultivated area has increased, the crop demand is much higher than the supply, which is itself lower than what was planned at the design.
The system is therefore water short, and the water can not be given to all channels as required at the same time. Two official rotations have been implemented:

- a 10-day rotation between the three divisions depending on Head Suleimanki,
- a 10-day rotation in the Fordwah Division for Kharif.

In this last rotation, first and second preferences are given to two Subdivisions of Fordwah Branch (Bahawalnagar and Chishtian). When a Subdivision is in second preference, priorities are given inside this Subdivision, between three groups of distributaries (see Warabandi Program for Kharif season 1995, Fordwah Division in annex A).

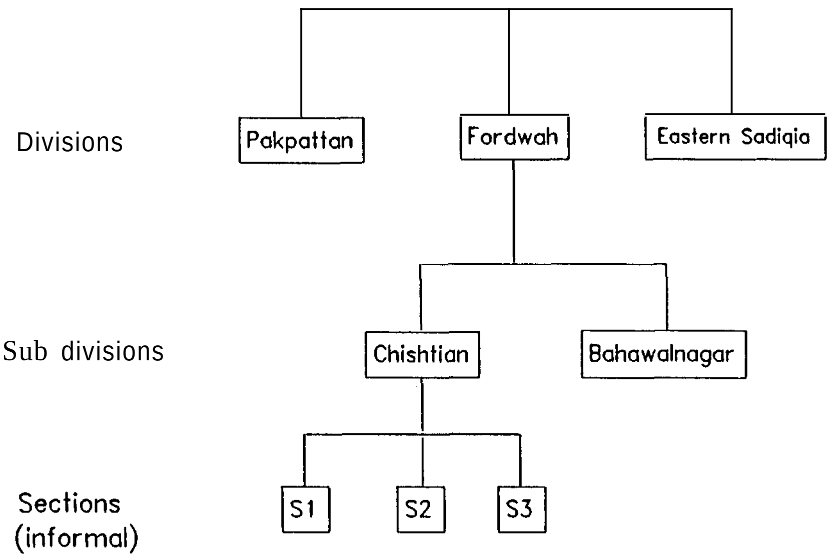


Figure 19: Rotations inside the system

Theoretically, a Subdivision in first preference should not suffer from water shortage, and should be able to supply water as requested to **all** its distributaries.

The water allocated to a distributary is supplied continuously to all watercourses. The water supplied to a watercourse is taken in full by a farmer for a certain period of time based on the amount of land he owns or cultivates (on average half an hour per acre). After completion of his turn, the water is then taken by the next farmer, following the order of the ***pacca*** (official) ***wurubandi.*** The warabandi is based on a 7-day rotation among the farmers of a watercourse. It is therefore very important to ensure at least **8** days of full supply to one disty that is in first preference*. The time separating the different rotations takes into account the time lag necessary for a wave to attain the head of the sub-system considered.

8 The eighth day is considered necessary to stabilize the flow.

4.2.3. Duties of the actors

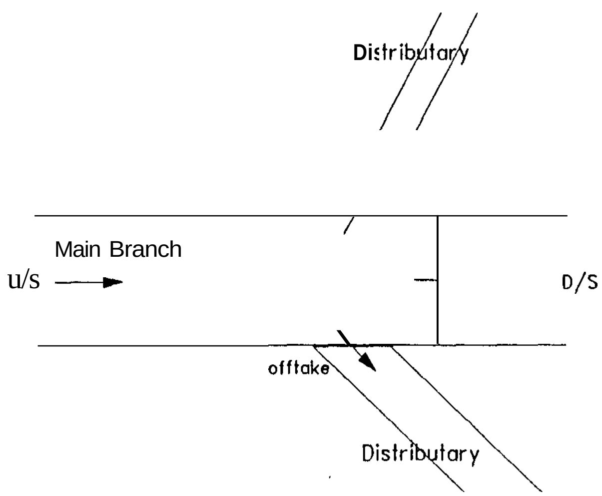
Rivière (1993) described the actors involved in the management of this system:

SubDivisional Officer: inside his subdivision, the SDO is responsible for operation and maintenance as well as for the assessment of water charges. His role is of crucial importance, as he has the authority to implement management decisions in the Subdivision.

Signaler: the signaler is responsible for communication within the system and with the higher levels (mainly the Division level). He has to collect the data from the field, to organize them so that the SDO can use them to make decisions.

Sub-engineers: they are the technical assistants of the SDO. One Sub-engineer is responsible for one Section, and has to implement SDO's instructions concerning operations and maintenance.

Gauge readers: the gauge readers are responsible for operations and monitoring at a control point. They are supposed to note twice a day the readings (water levels, and corresponding discharges), and to give these data to the Signaler. They have to implement the orders given by the SDO or Sub-engineers, by moving the gates or karrees under their control.



The flow diverted to distributaries is regulated by operating the cross regulator to maintain a constant water level upstream of the regulator, so that distributaries located just upstream receive a constant discharge.

In most other canal systems in the Punjab, water **is** distributed usually through proportional distribution, using weirs, and not through gates. **As** this system combines perennial and non-perennial canals, gates are necessary to differentiate the supply in Rabi.

<u>Remark:</u> the people responsible **of** operations at a control point are called 'gauge-readers' by the Irrigation Department. This denomination is significant, as it means that their main duty **is** to ***read gauges,*** not to operate gates (what they happen to do much more often).

4.3. Present situation

In this part the present situation of water management in the Chishtian Subdivision will be presented. The data presented come from meetings and discussions with Departmental staff and from observations in the field.

4.3.1. The actors and their relations

The actors of the system are basically (Rey, 1993b):

- People in charge of the decision-making process (**XEN,** SDO, Signaler)
- People in charge of the local implementation process and data collection (Sub-engineers, gauge readers)
- Water users (farmers)

The decisions taken by the **XEN, SDO** and Signaler concern the water scheduling in the system (rotational program, or decision to allocate water to such or such distributaries for such amount of time). The decisions taken by Sub-engineers or gauge readers concern the water distribution (what discharge to pass in a disty, operations of gates).
The farmers have an influence on the water distribution mainly, as they can come to the gauge reader controlling his distributary and request for more or less water. They also interfere at the water scheduling level, and even at the water allocation level, when they request **for** the inclusion of land in the CCA.

4.3.2. Water management in the Chishtian Subdivision

As cropping intensities are far above design, the demand of water has increased and a tremendous pressure has been put on the canal water supply. The canal water supply can no longer fulfill crops requirements, and managers have chosen to distribute the shortage of water with rotations at different levels.

4.3.2.1. Present rotational schedule

When Chishtian Subdivision is in first preference, if it happens to receive less water than the indent, an informal rotation is implemented, left to the discretion of the SDO. Orders are then given by any available means (messages, written or not) to the gauge readers, which prevail on the official rotation. These orders are very brief, and only concern one control point.
Small distributaries are generally not taken into account in this rotation, explaining their better performance as compared to bigger distributaries (Rivière, 1993). The implementation of this rotation, as it is not very clearly done, leaves a great margin of operation to the gate operators. In any case, they always prefer to satisfy the demand of 'their' distributaries. This situation leads to an increasing inequity from the head to the tail of the system.

4.3.2.2. Availability of flow in Chishtian Subdivision

One of the important constraints of this system is the inflow at RD 199. The figure below is based on IIMI daily measures.

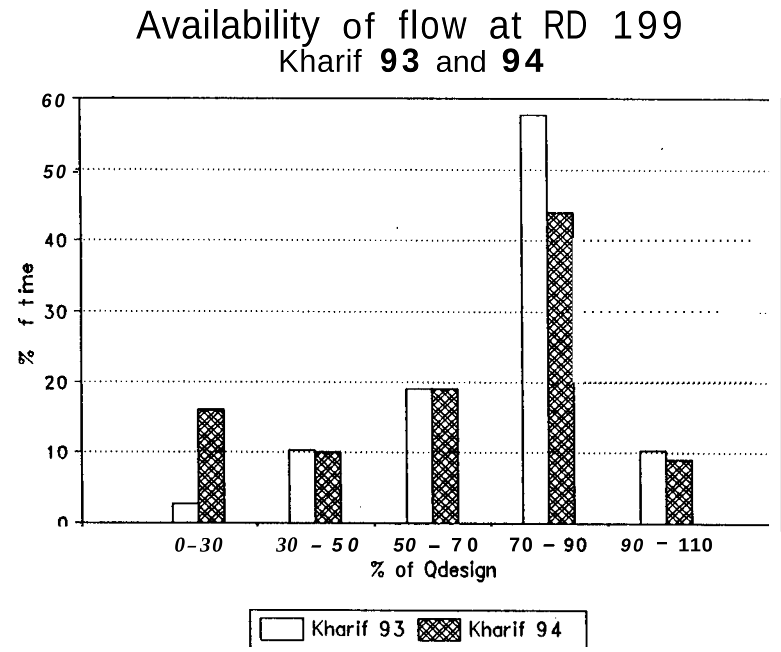


Figure 21: Availability of flow at RD 199

During 55 to 65 % of the time, the discharge at RD 199 is between 70 and 110 % of the design discharge. It means that the discharge at this point is also ***lower than*** 70%***of the design during 35 to 45% of the time.*** This constraint has a big impact on the state of the canal, in terms of siltation **as** well as in terms of operations and water distribution.

The average discharge at RD 199 during Kharif 1993 was 23.8 m^3/s, and 22.2 m^3/s during Kharif 1994, versus 33.6 m^3/s that is necessary to feed all the distributaries.

The performance of the Subdivision can also be evaluated with the indicator defined earlier, to assess the variability in discharge to distributaries. This is a way to evaluate the implementation of the rotational schedule.

This indicator gives the percentage of time during which the discharge at the head of distributaries was within the defined range of [85%, 110%] of the design discharge.

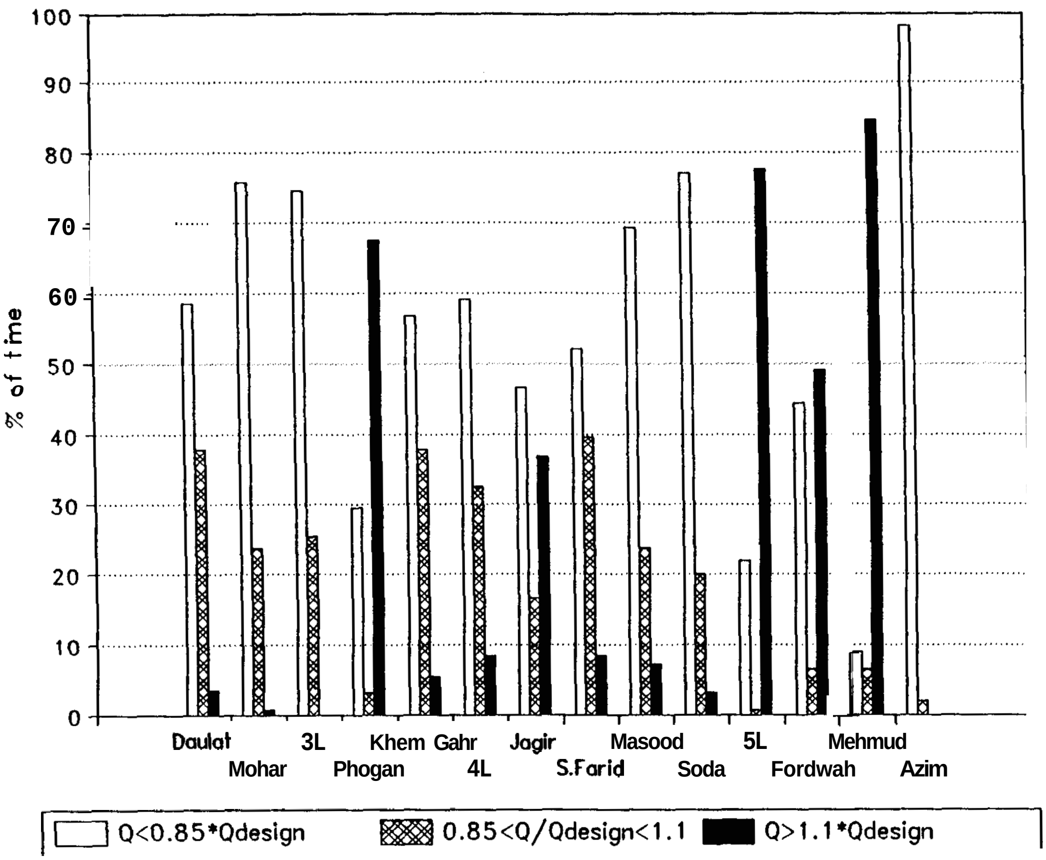


Figure 22: Performance of Chishtian Subdivision through an indicator of variability (Kharif 1994, IIMI measurements)

While reading this figure, one should keep in mind that the desired performance is not 100% for each disty. Because there is a rotation implemented, the theoretical percentage of time during which the discharge should be within the range [85%,110%] of the design discharge is 78%. During all the season, the Subdivision could have a shortage at RD 199 one day out of three[9] (when Chishtian Subdivision is in third preference), and during **this** shortage, 2 groups out of three could also suffer. Therefore, the discharge at RD 199 should be within the defined range of discharge 67% of the time, and this figure should be of 78% for the disties inside the

[9] The official rotation for Kharif 1994 was scheduled between the three Subdivisions.

Subdivision. It is clear that the performance of the Subdivision needs improvement, as no distributary has an indicator superior to **40%.**

Some distributaries are taking more than their share (mostly Mehmud, 5L, Phogan, Fordwah and Jagir), which means that the water is not equitably distributed inside the Subdivision.
Most of the Subdivision is suffering from a shortage of water during more than 50% of the season. The perennial canals appear to receive more water than non-perennial, Masood being the only perennial canal to have an important shortage during Kharif. This can be explained by the fact that the bed level **of** this distributary is much higher than the bed level of the main branch; it **is** therefore more sensitive to shortages, as the upstream level at that point has to be raised in order to feed this disty.

Phogan, which is non-perennial is one of the most favoured distributaries: this is due to the fact that the crest level of this open-flume has been lowered. The design discharge of this canal was not changed, explaining this high surplus. Soda, another open-flume not under the direct control of a cross regulator, is in shortage during more than 70% of the season. Two facts can explain this:

- the sill elevation of these open flumes are sometimes raised by putting karrees (stock wood) on the top **of** it, therefore lowering the discharge supplied,
- the design discharge **of** this distributary may be too high.

Remark: for this indicator, the reference discharge was taken as the design discharge. Ideally, the reference should be the demand of water, but as we are in a **supply based system** that needs to cope with a shortage of water, it would have been better to take the indent, or the targeted discharge supply to disties. This was not done, because there are no agreed targets for most of the disties. The actual targets are the indents, but are not realistic, and are almost never satisfied, as they are quite far above the water received. Targets can also be determined by interviewing the gauge readers.

4.3.2.3. Operation of the regulators

The regulators are operated by gauge readers, who live nearby their control point. They are usually experienced, and as the means of communication between them and the SDO or the signaler are very poor, they assume a great responsibility in the day to day management.
Their basic aim is to maintain the upstream level of the regulator at a constant point, the 'pond'. By doing this, they ensure a more or less constant discharge to the distributaries offtaking just upstream of their cross regulator. They also have to take into account the rotational orders they receive from the **SDO.** These orders are usually given in terms of priority and discharge for one channel: the channel in first priority will be given a full supply, and all the fluctuations will be passed to the channel in last priority. They sometimes receive instructions to pass a certain discharge downstream, but this happens very rarely. These orders are given at each control point, and when the priority is for the main canal, the operators do not know which distributary downstream is in first priority.

Rating tables:

The tables used by gauge readers to determine the discharge in the channels are based on a **Q(H2)** relationship. The formula used is $Q=K*D^{5/3}$, in which **K** is a constant and D is the water depth in feet. The discharge is then expressed in cusecs. The downstream gauge, whose 0 level is at the design bed level is read by the gauge reader, who then gets the corresponding discharge from the rating table. This method has the advantage of being simple of use, but necessitates frequent updating, as canals in the Punjab are subject to siltation or scouring, given the substantial amount of silt they carry. At some points where changes in the bed level are too frequent[10], rating curves are based on structure formula; the discharge **is** given according to the gate openings, when the upstream level is near the pond (for structures under free flow conditions). This is the case for two distributaries:

- Shahar Farid distributary at RD **316,**
- Azim distributary at RD **371.**

Remark: The **Q(H2)** relationship used by the I&PD corresponds to a uniform flow in a rectangular canal where $D << B$, D being the water depth, and **B** the width **of** the canal. If we write the Manning-Strickler formula for such a canal, we get:

$$Q = KSR^{2/3}(i)^{1/2}$$

with **K** the Strickler coefficient, S the cross section wetted area, **R** the hydraulic radius, and **i** the friction slope.

For a rectangular canal, $S = BD$, and $P = B+2D$ (wetted perimeter)

Then $R = S/P = D$ if $D << B$,

And $Q = aD^{5/3}$

with a constant.

---

[10] **This is usually due to local conditions, e.g. highly varying velocities, or a bed level much lower than the main branch.**

Priorities:

At one control point, a gauge reader usually operates two or three structures, one cross regulator, and one or two gated distributaries:

- when the main canal is in first priority, he operates the gates of the distributaries to maintain a constant discharge downstream of the cross regulator. This is checked by looking at the downstream gauge **of** the main branch, a rating table giving the relation $Q=f(H2)$.

- when a distributary is in first priority, he operates the gates of the cross regulator to maintain the upstream level at the pond, assuming that this will allow a constant discharge to the disty. All the fluctuations are therefore passed downstream in the main branch.

An important feature of this local management is that it is mainly performed to avoid breaches. As an answer to the question 'what is your main objective?', gauge readers told that they had to 'save the main branch' upstream of their control point, i.e. to prevent bank breaches.

4.4. Diagnosis

The management of Fordwah Branch in Chishtian Subdivision will be detailed in two steps: first the physical constraints, and then the management in itself will be analyzed, to give a final diagnosis of the system.

4.4.1. Physical constraints

Inflow:

- There are a lot of fluctuations at the head of the canal, due to local operations (at RD 199 or upstream), or to the inflow at the headworks (at Suleimanki barrage).

- In Kharif season, the inflow at RD 199 is less than 90% of the design discharge during 80% of the time. This means that there will always be at least one big distributary which will suffer from a shortage of water in the Subdivision.

- This low inflow also has an effect on the shape of the canal in terms of siltation (as the regime theory specifies that the discharge has to be higher than 70% of the design discharge to avoid silt deposition). In the case of Fordwah Branch in Chishtian Subdivision, the discharge at RD 199 was lower than 70% of the design during more than 50% of the time of the Kharif 1994 season.

Physical shape of the canal:

The comparison between actual and design situation showed clearly the changes that have occurred in this system. As a consequence, the capacity of the reaches has been reduced, and the water levels have increased, lowering the free-board (which gives a security margin for operations).

The size of the system is a predominant feature:

- The time lags between the headworks at Head Suleimanki and the tail of the system is more than 1 day. This is a strong constraint in case of emergencies, when the discharge at the head has to be reduced.

There is no escape at the tail of the system, therefore no security in case of emergency. This is an important constraint, as the tail is already very sensitive to fluctuations, due to the inflow or to operations inside the system; an escape at this point would give an important margin of security, and may improve the situation of Azim disty, which is used as an escape, and has been damaged by cuts[11] and breaches.

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[11] A cut is a deliberate damage to the bank of a canal. It is generally made on a distributary, to steal some water.

4.4.2. Management of the system

The following graph summarizes the management of the main canal, with the different levels of management, and the external influences. It is inspired by Rey (1993b) but adapted to the situation observed in the field for Chishtian Subdivision. The different parts of this scheme will be analyzed below, for physical as well as management problems.

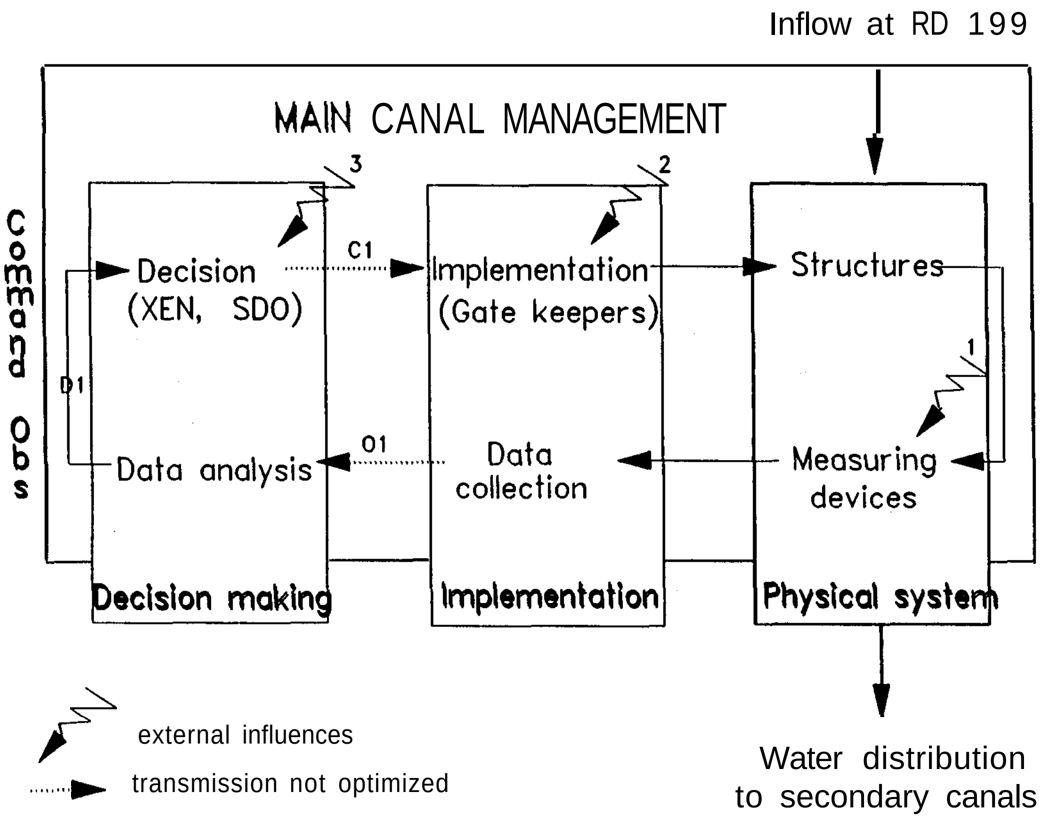


Figure 23: Schematization of the main canal management

Key distinguishes three main processes for the management:

- Command
- Observation
- Evaluation

In this graph are only mentioned two processes (command and observation), as there is practically no evaluation process.

The main canal management is also separated in three components:

- the physical system
- the implementation (tactical level)
- the decision making (strategic level)

The communication between these components is not optimized:

- the arrow C1 represents the orders given by the managers to the local staff; these orders are usually very specific, and do not concern routine management. They are given by means of little pieces **of** paper, or sometimes by oral transmission. It is therefore very difficult to keep track of these orders, to know the causes of changes in the canal management.

- the arrow 01 represents the transmission of data from the field to the manager; as the communication system is not functioning properly, this transmission takes some time, and this has a large effect in case of an emergency. If a breach occurs at the tail part of the system, the information takes time to reach the manager, and when the decision is taken to reduce the supply, the damage can already be very important.

- the arrow D1 represents the process of feed-back of data from data analysis to the decision making. This is could also be improved, as the decision taken by the managers are rarely based on field data.

Some operations of the observation process are not optimized:

- the measuring devices (gauges) need maintenance
- the data collection is not always accurate, as some data are reported according to the demand of manager, and not to the measure in the field

- the data analysis needs improvement.

The evaluation process is not performed at all by the Irrigation Department; a kind of direct evaluation comes from the farmers who interact with the Irrigation Department. An informal criteria used for the evaluation is the number of complaints coming from the farmers.

Decision making:

There are no agreed targets for the water distribution. The indent should be the target discharge to a distributary, but this indent is rarely defined in the field, and most of the time the gauge reader notes the actual discharge as the indent. For smaller distributaries, the indent is fixed at the design discharge, because they are not incorporated in the rotation. In case of emergencies (breaches, or rains), which appear to happen very often, the main strategy is to 'save the main branch', then the perennial canals. A breach can occur in a distributary without causing big consequences for the manager, whereas if a breach occurs in the main branch, the SDO is immediately suspended, without inquiry. The responsibility is left to the local staff, who are responsible for the situation at each control point. The gate operations are not monitored, and actual flows are not well recorded, because of outdated rating tables. In general, the management appears to be leave much responsibility to the local staff; this leads to an increased inequity in case of water shortage: as gauge readers tend to satisfy the demand at their point, the upstream distributaries are favoured compared to distributaries located at the tail.

Implementation:

The gate operators perform a local control, with very scarce information about changes in the discharge or operations upstream or downstream of their point. They are given a great independence, and receive very few operational orders. The local control performed by gate operators aims at preventing breaches, when this should be the duty of the strategic control, at a higher level. The system is very often faced to emergencies in the Kharif season, mostly because of heavy rains.

There are ***external influences*** that affect the performance of the management. These interferences usually are the fact of farmers wanting more water for their crop. They can act at the main system level, or at the distributary level (impacting indirectly on the main system).
Numbers in brackets refer to the arrows in the graph.
At the main system level:

- alter measuring devices (gauges) [1].
- alter implementation of orders, by operating gates [2].
- request for direct outlets from the main branch [3].

At the distributary level:

- alter measuring devices (gauges) [1].
- steal of water by cutting banks or installing illegal pipes.
- request for a change in the size of outlets [3].
- request for including more land in CCA [3].

Management diagnosis:

As a consequence of all this, the strategy of water distribution in this system is not clear. We will try to clarify it by distinguishing policy, strategy and tactics as it has been defined in the methodology.

The **policy** in the Punjab Irrigation system is an equitable distribution of the water available. The official **strategy** is the official rotation at the Division level. We have seen that the actual strategy is not in accordance with the official one. In fact, this actual strategy is the result of actions of different ***actors,*** that have different ***interests.***

* The decisions taken by the managers (XEN, SDO, Sub-engineers) are made under different constraints:

  - technical constraints,
  - socio-political constraints.

The areas where some influential farmers live will be of critical interest in the decisions made at this level. **As** a matter of fact, it is clear from the available data that some distributaries are favoured, whereas some are much more suffering. Daulat and Fordwah, for example seem to be two "critical subsystems" in that sense.

* The actions performed by gauge readers are also influenced by theirs own targets, i.e. feed their distributaries. **As** orders by managers are rather scarce, these actions are most of the time predominant to other strategies.

* In case of emergencies, the priority is to save the main branch, then almost all the responsibility is delegated to gauge readers. They are allowed to provoke a breach in a distributary **if** necessary, but a breach in the main canal has to be avoided at all costs.

The **tactics** is the way gauge readers implement the strategy. **As** this strategy is not clear, most of the time gauge readers act according to their own "strategy", and as it is linked with local interests, the result at a global level is an inequity in water distribution.

4.5. Conclusions

The management of this system was analyzed in three steps:

- first the design situation was studied,
- then the standard operational practices were described,
- finally, the present situation was presented and compared to the previous parts.

As the targets of the manager are not fixed, it is difficult to evaluate the performance of the system with clear criteria. We proposed an indicator to assess the performance of the water delivery to distributaries, in an attempt to link operations at the main system level to what is happening downstream, at the distributary level.

The management of the canal was analyzed with a simple diagnosis grid, separating the different levels of management, as well as the different processes in the management.

This diagnosis has pointed out weak points that have to be addressed in order to enhance the performance of the system. From this diagnosis, a few recommendations can be expressed for the strategic level of management:

1. Clarifv the targets: to implement a strategy, the manager needs to have clear and realistic objectives for water distribution. Once these objectives are defined, an evaluation of the management can be done.

2. Integrate an evaluation process in the management, once the targets are defined.

3. Communication system: the transmission of information is very poor in actual management. To implement a strategy, information means between the manager and the gauge readers are necessary. Gauge readers also request communication between themselves, so that they can be aware of operations performed at points located upstream, and prepare themselves for the required reaction.

4. Integrate the small disties in the rotation: small distributaries appeared to be favoured compared to bigger ones, because they were not included in the rotation. The implementation of the official rotational plan to all disties could improve the equity in the Subdivision, but might increase the complexity of the system.

The management of this system leaves scope for improvement, in many aspects; we will focus on the **operations inside the system,** using a hydraulic model to simulate new rules of operation.

5. ALTERNATIVE SCENARIOS

5.1. Manual operations

5.1.1. Review of previous work

Malaterre (1989), and Rey (1990) have already used the same **SIC** package in the prospect of improving manual operations of the Kirindi Oya Right Bank Main Canal in Sri Lanka. Malaterre showed that it was possible to stabilize the levels in the canal in a short time after a release of discharge at the head, if gate operators have some information concerning the wave. They have to open the gates at mid-wave so that the amplitude of the wave and the time of perturbations are minimized. He defined two situations, and simulated actual and improved operations with the regulation module.

Rey proposed to use this model in an operational way, in collaboration with the manager; he proposed to define with the manager typical managing phases which are to be studied with the simulation model, to find operational rules improving actual situation. He simulated some scenarios with the same package, using the same regulation module as Malaterre.

Kuper, Habib and Malaterre (1994) have also used **SIC** package to study operations in Chishtian Subdivision, in an a posteriori approach. They used an 11 day monitoring data to show possibility in improving local operations, and proposed two scenarios:

- one called 'improved localized control', aiming at reducing the number of operations at a local level to minimize the fluctuations,

- one called 'feed-forward control', where the manager is supposed to have an intimate knowledge of his system, and an estimation of future perturbations; he can then give orders to gauge readers for operations. The number of operations is reduced to one operation a day, and the effect was found beneficial for reducing perturbations.

As far as we know those three studies did not lead to a field test of scenarios. This is what we aim to do, on the basis of an accurate calibration of the model.

5.1.2. Presentation of scenarios

As defined in the methodology, a scenario can be tested when the two elements are chosen

- a strategic option
- an inflow at the head of the canal

The situations were chosen with regards to the operational constraints faced by the manager.

It came out of the analysis of operations that some are not justified hydraulically speaking. In the first situation (a), two scenarios are tested to compare the effect of local operations on a real inflow.

In the second situation (b), the same inflow is used, but the strategy is changed. Two scenarios are also tested to show the possible improvement by using the storage capacity of some reaches. Fluctuations that come from upstream are generally amplified by operations in the Subdivision. In the third situation (c) we compare the effect of actions at a regulator (RD 245) and at an offtake (Daulat) for a negative step of discharge. Is it possible to stabilize the flow in the main branch by operating an offtake instead of a cross regulator'?

At present, the rotational plan is rarely implemented in the field, and when it is, there is no planned action to deal with fluctuations created by a change in the priority order. The fourth situation (d) is a shift of rotational order from Daulat (distributary at the head of the Subdivision) to Azim (distributary at the tail. This is simulated with a stable inflow, as situations b and c will show how to deal with fluctuations coming from upstream..

The last situation *(e)*deals with a local maintenance having an effect on operations.

The scenarios and situations are summarized in the table below.

Table 5.1.1.: Simulated scenarios

No	Scenarios	Situations				
		a	b	c	d	e
0	Real operations	*				
1	Improved operations	*	*	*	*	
2	"Buffer"				*	
3	Information at the tail				*	
4	Feed forward		*		*	
5	Automated gates					

Situations:

Situation a: real inflow pattern,

This situation represents what was monitored in the field from 3rd to 5th of June. We compare real operations (scenario 0) with improved operations at regulators (scenario 1) simulated by the regulation module. As some operations were found to be unjustified, they will be eliminated (not simulated by Gateman). The comparison will be done in terms of fluctuations in discharge in the main canal and in the distributaries.

Situation b: increase of discharge at the head

This situation supposes that the Subdivision suffers from a shortage of water, and receives a surplus from the upstream subdivision.
We assume there is no change of rotation during this time:

- Azim and Fordwah are in first priority
- Daulat is in second priority
- Shahar Farid is in last priority

The objectives of the manager are supposed to be as follow:

- to feed Azim with the surplus of water
- to reach the target state as quickly as possible.

The target state is the state of the canal where the disties in first priority are supplied within the required range of discharge, and where the levels upstream of the regulators are stabilized around FSD. The real inflow pattern was prolonged with a stable discharge during **24** hours, to see the effect of the real inflow on the whole canal.

Two scenarios will be tested on this situation:

- improved operations (scenario 1)
- feed forward control (scenario **4).**

Situation c: Negative step of discharge at the head,

This situation supposes that the inflow at the head of the Subdivision drops abruptly of **2m^3/s** (70 cusecs). We assume that the preference order is the same **as** for situation (a) (tail distributaries Azim and Fordwah in first preference, Daulat in second preference and Shahar Farid in last preference).

We test two situations, one where Daulat distributary is used to take the fluctuation (as Shahar Farid distributary is already closed), so that the discharge downstream the main branch remains constant, and the other one, where the operator at RD **245** continues to try to reach his local targets, i.e. to feed his distributaries.

Situation d: Stable inflow, rotational priority shifts from Daulat to Azim

For this situation, the inflow at the head of the canal is supposed to be constant. We simulate a change in the preference order: Azim, which was in last priority shifts in first priority, and Daulat shifts from first to last priority.
Daulat is closed at the beginning of the simulation, and the objective is therefore to **pass** a wave of about 5 m^3/s from Daulat to Azim, as quickly as possible, without disturbing too much the distributaries in between.

Different tactical scenarios will be tested on this situation:

- Improved operations (1)
- Buffer capacity (2)
- Information at the tail (linked operations) (3)
- Feed forward control **(4)**

The comparison will be done in terms of discharge at Azim disty, fluctuations in water levels upstream of regulators, time of stabilization, time to reach the target.

Situation e: Stable inflow. local maintenance performed at RD **363.**

This situation aims at giving more flexibility in operations for the gauge reader at the tail of the system (RD **371),** by raising the left bank at RD **363.** This point just downstream of the weir at RD **363** has appeared as the weakest in the simulations for the maximum storage capacity. **A** local maintenance operation could strengthen this point, and therefore give more flexibility to the operator at the tail (RD **371).** The improvement will be expressed in terms of increase in storage capacity for the concerned reach, and increase in possible fluctuation in upstream water level at RD **371.**

5.2. Automated gates

A fifth scenario was tested where the gates are moved by automatic devices; these automated gates are operated following a Proportional Integral (PI) control algorithm.

5.2.1. Automation of irrigation canals

The automatic control of irrigation canals is **a** modern approach for the regulation of irrigation systems. Different **possible** options are proposed by researchers or policy makers (Plusquellec 1994, Malaterre 1994). The objectives of these methods are to improve the efficiency of canal systems.

The advantages of automation compared to manual operations can be listed as follows (from Plusquellec, 1988):

- efficient use of water resources
- high quality of service
- low cost of operation
- minimum manpower

The drawbacks are:

- **a** higher cost of installation
- need of regular maintenance
- fragility to vandalism
- loss of flexibility
- need **of** power facilities (usually)

The choice of an automatic control method depends on:

* hydraulic constraints (bank levels, available resource at head, storage volumes on-line and at the tail, siltation, water allocation policy: on demand, rotation, etc.)

* technological constraints (availability, vandalism, maintenance)

* socio-economical constraints (willingness, cost).

We will present an example of automatic control applied to Fordwah Branch after a presentation of general notions of automatic control.

5.2.2. Introduction to automatic control

a. General notions (Malaterre et al., 1995)

The automatic control considers a **system,** on which one applies **commands** (U) in order to control **output variables** (Y). The commands U are also called control action variables.

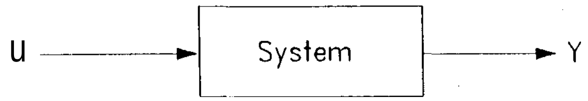


Figure 24: System, command and output

Control engineers distinguish two types of control:

- **Feedback control:** the command is a function of the difference between the measured output Y and the targeted output Yt.

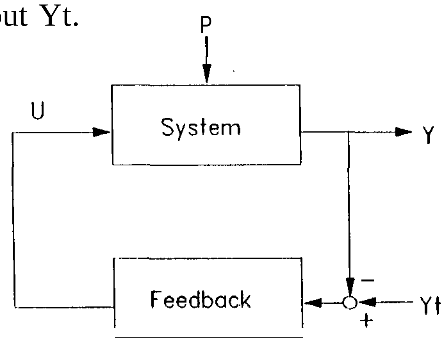


Figure 25: Feedback command

Perturbations are indirectly taken into account through their effect on the output Y.

- **Feedforward control:** the command **U** is computed from the knowledge of the dynamic of the system, of the targeted output Yt, and of an estimation of perturbations P^.
A feedforward command can improve the performance of a feed back, especially for systems with important time-lags.

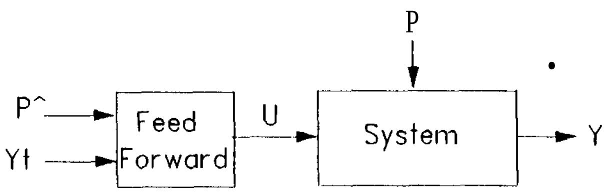


Figure 26: Feedforward command

A feedforward command is usually not sufficient, as errors stemming from the **model** and unknown perturbations are not taken into account. The association Feedforward with Feedback is often used, because the feed back corrects the errors of the feed forward.

The notion of Transfer Function (TF) is introduced for a system or a process. They can be found in Dieulesaint (1990). **A** transfer function is a function that represents the dynamic behaviour of the system studied: **it** gives the response Y of the system when the command **U** is known[12]

b. PID control

PID goes for Proportional, Integral, Derivative. It is the most well-known technic developed by control engineers, which is widely used in industry, because of its simplicity and efficiency in most of the cases.

Transfer function of a PID in continuous variables:

$$TF(p) = K\left(1 + \frac{1}{T_i p} + T_d p\right)$$

with p = Laplace variable
K = proportional gain
Ti = integral action time
Td = derivative action time.

[12] **As this is not the direct subject** of our **study, we will not develop the definitions** of **Transfer Function, Laplace Transform,** or **Z Transform.**

For a **PID** with discrete variables, we get:

$$u(k) = P(k) + I(k) + D(k)$$

where k represents tk = k.Dt, Dt being the sampling time.

The functions P, I and D are given by:

$$P(k) = K(y^*(k) - y(k))$$

I(k) is given with the Tustin approximation:

$$I(k+1) = I(k) + \frac{K\Delta t}{T_i} \cdot \frac{e(k+1) + e(k)}{2}$$

and

$$D(k) = -\frac{KT_d}{\Delta t}(y(k) - y(k-1))$$

with e(k) = y*(k)-y(k)
y* = target output variable.

c. Calibration of a **PID** controller

The calibration of a PID controller is the determination of the value of the coefficients K, Ti and Td.There are several methods for the calibration of a PID controller. We used the Ziegler-Nichols 'ultimate sensitivity' method, because of its simplicity: it enables to determine the coefficients without being obliged to study the transfer function of the process.

The system is looped with a controller only proportional, with a gain K. By progressively increasing this gain, we get a limit value Ku for which the system is at its limit of stability, periodically oscillating. The period of these oscillations is called Tu.

Then, the Ziegler-Nichols method gives the following values for the coefficients of the PID controller:

	K	Ti	Td
P	0.5 Ku		
PI	0.45 Ku	0.83 Tu	
PID	0.6 Ku	0.5 Tu	0.125 Tu

The sampling period of the command of the process should also verify:

$$0.01\,Tu < Dt < 0.05\,Tu$$

5.2.3. Application to Fordwah Branch

The objective is **to** develop a simple automatic regulation of Fordwah Branch canal, with little constraints and modifications compared to the present situation. We supposed that one constraint **of** actual management could be erased: the inflow at the head RD 199 can be adjusted to the demand. It can be seen as a situation where the Chishtian Subdivision is in first priority, and the indent is automatically supplied. We chose a downstream control method, to provide 'on demand' distribution. This would increase the equity in the system, as the tail would not be disfavoured compared to the head. In order to keep the actual physical shape of the canal, a local downstream control **was** rejected, as in this case banks need to be levelled.
The method is therefore a ***local distant downstream control.*** The water level at the downstream end of a pool **is** controlled by the gate opening of the cross structure located upstream of this pool.

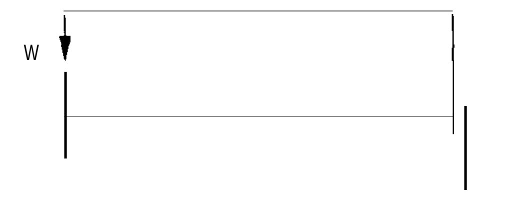


Figure 27: Distant downstream control

The method chosen can be described according to the terminology used by Malaterre (1994):

a. Considered variables:

- controlled variables: Ydn (water level at the downstream end of a pool), compatible with sloping banks.
- measured variables: Ydn, to limit number of sensors and communication lines.
- control action variable: w (gate openings of a cross regulator), to keep present gated devices and reduce the complexity of the controller. The drawback is that it increases coupling and non-linear effects.

b. Logic of control

- Type: feedback control, for perturbation rejection. We chose not to include a feedforward, to reduce the complexity of the controller, and to allow a local implementation instead of a centralized one.

- Direction: downstream control, for on demand distribution. One drawback is that it can increase siltation due to low flow velocities.

c. Design method

- Main technique: PI controller, the simplest method developed by control engineers. It gives a good performance on second order process with small time delay.

- Additional component: none (we could test with minimum gate movement and limits of intervention)

d. Field implementation

- Architecture: local distant
- Device: electro-mechanical gates

The scenario is therefore a regulation with 6 controllers in series:

Qhead	= PI(Zup RD199)
wRD199	= PI(Zup RD245)
wRD245	= PI(Zup RD281)
wRD281	= PI(Zup RD316)
wRD316	= PI(Zup RD353)
wRD353	= PI(Zup RD371)

(Zup RDxxx is the water level upstream of the cross regulator located at RDxxx).

The controllers are calibrated with Ziegler-Nichols 'ultimate sensitivity' technique. The calibration is done for each local controller, from upstream to downstream. The perturbation used to destabilize the system is an operation at Azim disty (closing of 20 cm).

The obtained coefficients are then reduced to reduce instabilities, from downstream to upstream. This latest step is required due to important coupling and non-linear effects.

Results of the calibration and of the test on a situation are given in chapter 6

6. RESULTS

6.1. Calibration and validation of the models

6.1.1. SIC model

a. Calibration

The calibration was performed with a maximum error of 3 cm in water levels at the important control points, (water levels upstream of regulators).
The results of the calibration are given in the table below, and are taken for two different steady flow periods for two parts of the system.

Seepage calculated from field values:

Reaches 1-4: SFP1, 2-3 June 1995

Qin	= 25.45
Qout	= 22.28
seepage	= 3.17 m^3/s, or 67 l/s/km

Reach 5: SFP2, 3-4 June 1995

Qin	= 11.3
Qout	= 10.59
seepage	= 0.31 m^3/s, or 63 l/s/km

Total: 3.48 m^3/s

Table **6.1.1**: Results of **SIC** calibration

Measuring points	**Measured in the field**			**Simulated in SIC**		
	Water elevations		**Q computed**	**Water elevations**		**Q**
	U/S	**D/S**		**U/S**	**D/S**	
D199	157.62	156.24	25.45	157.82	156.5	25.45
Daulat			2.96			3.24
Mohar			0.81			0.89
3L			0.34			0.39
D245	154.18	153.53		154.18	153.53	20.06
Phogan	152.49		0.45	152.49		0.49
Khein Gahr			1.2			1.15
4L			0.37			0.36
D281	151.86	151.61		151.87	151.62	17.06
Jagir	150.88		0.9	150.86		0.836
S. Farid			0.01			0.08
Masood			1.02			0.955
D316	149.93	149.59		149.94	149.60	13.98
Soda			1.92			1.64
Weir 334	148.65	148.55		148.69	148.59	
D353	147.45	146.33	11.3	147.45	146.37	11.04
Fordwah			5.19			5.29
Mehmud			0.65			0.72
Azim	145.44	144.41	4.65	145.47	144.36	4.31

The total seepage calculated is 3.2 m^3/s, or 60 **l/s/km**, or 12.5 % of the inflow.

In Pakistan, seepage is usually expressed in terms **of** cusecs per million square feet: the total wetted area is calculated using SIC's results (see annex D). The result is 12.4 million square feet (msf). The seepage is therefore 9.3 cusecs per msf.

The table below gives the adjusted coefficients used for this calibration. (See annex for the coefficients used for the offtakes)

Table **6.1.2:** Discharge and Manning coefficients used in **SIC**

Reaches	Manning coefficient	Structures	Cd
		D 199	0.56
Reach 1	0.019	D 245	0.51
Reach 2	0.019-0.023 [13]	D 281	0.60
Reach 3	0.02-0.021	D 316	0.58
Reach 4	0.024-0.023	D 353	0.63
Reach 5	0.02	Fordwah	0.56
		Azim	0.53

Remarks:

Some points showed a difference between measured and simulated values: the upstream water level at RD **199,** the downstream water level at RD **353,** and the downstream water level at RD **371.** This is due to a change in the geometry of the model of the canal:

- as **SIC** does not allow supercritical flow in unsteady state, two sections had to be deleted downstream of RD **353,** because there was a local drop in the bed elevation,

- the sections upstream and downstream of RD **199,** and downstream of RD **371** come from an old survey.

Nonetheless, these points are not important hydraulically speaking, as these three structures are free flow: the downstream level does not influence the upstream level, and RD 199 **is** the head of the system, therefore an error in the upstream level at this point does not influence what happens downstream (sub critical flow).
Other topographic modification have been made:

- as **SIC** does not allow the sill elevation of a cross structure to be lower than the bed elevation, at RD **245,** the bed elevation was artificially lowered to be able to

[13] **Two values are given when there is a measuring point inside the reach. The first value stands for the upstream part** of **the reach, the second for the downstream part.**

b. Verification of the calibration

The inflow during the monitoring period was quite stable during the first 40 hours (around 25.2 m^3/s), and then increased with some variations to 26.9 m^3/s. This inflow was very interesting for our study, as it enabled **us** to perform a good calibration in steady state, an then to simulate the canal under unsteady state, beginning from the water line obtained with the steady state module of **SIC.**

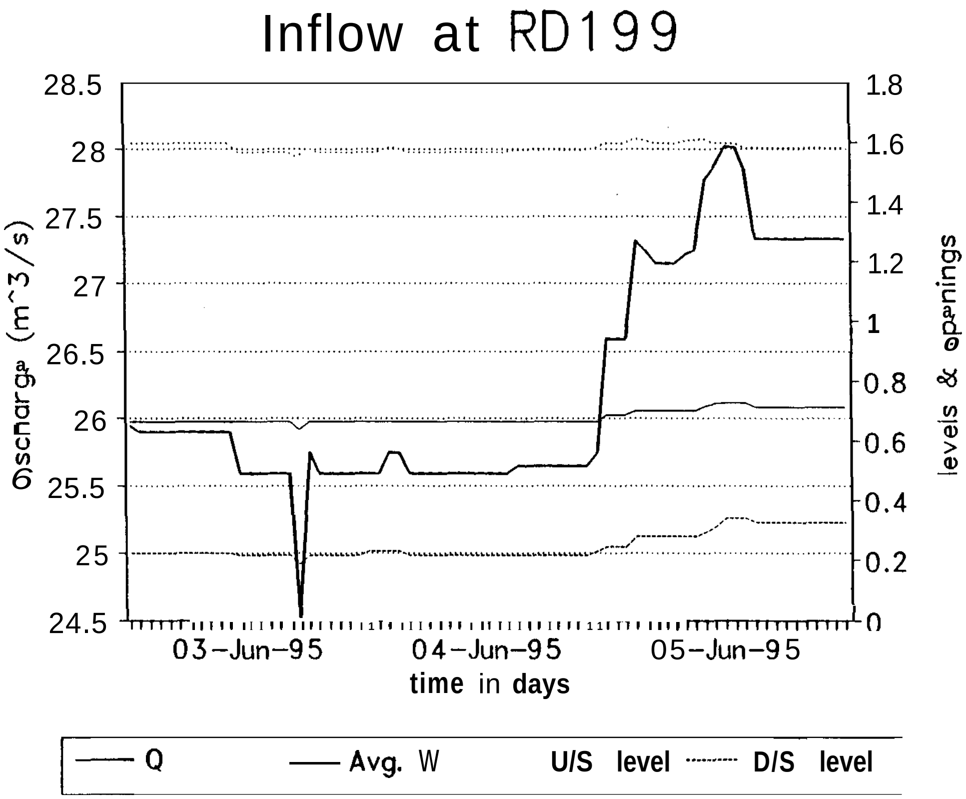


Figure 28: Inflow at RD 199 during the 72 hour period

The simulation was done with monitored operations at structures. The results represent therefore the simulation of actual situation with real operations. The comparison between observed in the field and simulated levels has been done at each measuring point. The maximum deviations from the observed water levels are described in the table below, along with the time when this deviation becomes greater than 6 cm.

Table **6.1.3:** Maximum deviation in U/S levels (simulated/observed values) and in percentage of the discharge in the canal

Measuring points	**Maximum deviations in U/S levels**	**Time for which Hf-Hsic > 6 cm**	**Max deviation in discharge** (%)
D **245**	**-7** cm +0 cm	**44h**	-
Phogan	-3 cm +0 cm	-	-
D **281**	**-4** cm +3 cm	-	-
Jagir	**-4** cm +1 cm	-	-
D **316**	**-6** cm +5 cm	39 h	-
Soda	-1 cm +6 cm	**35 h**	-
D **353**	**-14** cm +5 cm	47 h	**[-6%,+8%]**
D 371	-0 cm +12 cm	9 h	**[-16%,-4%]** (one point **46%** [14])

The simulated water levels are all within a range of 12 cm around the observed levels, but it is important to note that the deviation becomes greater than 6 cm only after quite a long time (36h), except for the tail end of the system. The fact that the tail is not showing as good results **as** the rest of the canal is mainly due to two reasons:

- the errors generated by the model propagate along the simulated system, and are more visible at the end. **As** the width of the canal is smaller at the tail, an error of DQ on the discharge will have a more important effect in terms of difference in water level at the tail than at the head.
- this tail part of the canal was not in steady state at the beginning of the simulation, it was calibrated apart from the upstream reaches, with another SFP. Therefore, the initial simulated water line was not similar to the real one; this explains the difference in water levels at the beginning of the simulations.

[14] This aberrant value occurs for a closing operation of 27 cm that brought the opening to 4cm, causing a very low discharge.

Remark: during a calibration training organized by IIMI along this canal, it was observed that the Cd for structures was very variable with the gate opening. The fact that the water levels simulated in SIC are deviating from observed values when gates are operated seems to show that **SIC** equations for some structures do not represent very accurately reality. There is still some research that has to be done in this field, and a PhD thesis is undertaken on this subject in Cemagref Montpellier, Irrigation Division.

During this 3 day monitoring period, the gate operators were interviewed about their actions when they did not seem to stem from a hydraulic change in the canal. The clearest action of this type was observed at RD 316, where the gate operator opened the gate of Shahar Farid disty at 10:30 the 4th of June; this disty was closed before, because it was in last priority, but the gate operator received an order from the Chakh Abdullah Sub-engineer to open this disty in order to give water to an influential farmer whose turn was on this day. The shortage of water has a drastic effect at RD 371, where Azim distributary is almost closed (opening = **4** cm) to maintain the upstream level constant in order to feed Fordwah distributary.

Graphs showing the results of the calibration (comparison between observed and simulated levels at each monitoring point) are given in annex B.

6.1.2. Regulation module Gateman

a. Analysis of operations

The gate operation ratio defined in the methodology was computed for all the operations at regulators during the monitoring period.

The analysis of the results is done for two points in the canal: one regulator located at the upstream end of the canal, RD 245, and Azim distributary, situated at the tail of the system. This distributary is operated as a tail regulator (or as an escape) to maintain a full supply in Fordwah disty.

The operator at RD 245 seems to act accordingly to the rule computed. The gate operations ratio is very close to 1 and is very consistent with time: the closing operations are performed with an average ratio of 0.99 and the opening operations with an average ratio of 1.08. During the monitoring period, he performed an average of 4 operations a day, mostly to respond to fluctuations coming from upstream.
Nonetheless, there are some operations that were not justified hydraulically:

- the 3rd of June, at 10:30 (operation a), the gate operator closed the cross regulator to raise the upstream level in order to feed **3L,** a little distributary offtaking just upstream of this regulator, on the request of a farmer. This distributary has an open flume offtake, therefore quite sensitive to changes in upstream level. This explains why he did not have to make a drastic change in the gate settings.

- at 21:30 the same day, he opened the cross regulator to keep a "safety margin" (operation b): he lowers the upstream level during the night, to prevent the consequences of a possible wave coming from upstream while he is sleeping. The gates of the regulator are then closed again in the morning (at 4:30, operation c), to raise the upstream level at its target.

- he does the same thing the next day: he opened the regulator at 22:15 (operation d), and closed it the next morning at 4:40 (operation e).

Those operations are not justified hydraulically, and therefore could be eliminated. The number of operations at this point could therefore be reduced from 11 to 6 for the three day period.

At Azim disty, the situation is radically different: this distributary is at the tail end of Fordwah Branch, and receives all the fluctuations provoked by the operations of regulators upstream.
The task of the gate operator is therefore much more difficult, as he has to react to bigger fluctuations, with no information about their duration or amplitude. This situation is very clear the 4th of June: at 10:30, a negative wave is provoked at RD316 by the opening of Shahar Farid distributary, which was closed, because in last preference. At 15:20, the gates of Azim are

closed of 27 cm, bringing the opening to only 4 cm. The reaction of the operator while closing the gate is quite drastic, but this way, he manages to keep the upstream level rather constant in order to give full supply to Fordwah distributary. **As** the closing operation was too strong, the upstream level rose too much, and he then had to open again Azim gates. Apart from these two operations linked to an emergency, the ratio for other operations is within an acceptable range, even if the operations always seem to be overestimated. The explanation for these overestimated operations is that the operator at this point always keeps a **safety factor** to prevent any further emergency. Situated at the tail end of the system, he is in charge of the last control point in Fordwah Branch, but he has no other option when a fluctuation comes than to pass it to Azim distributary because there is no escape. His instructions are to save the main branch (upstream of his control point) and Fordwah disty from breaches. During his operations, he therefore uses Azim as an escape, which explains the poor performance of this disty in terms of availability of flow.

The same analysis carried out for other regulators gave also similar results:

D281: The first closing operation on the 3rd of June at 12:50 (operation a) was a mistake, which was corrected by an opening operation one hour later (operation b). The regulator was also opened the 5th of June with a too large operation than required (operation c). Apart from these three operations, the ratio of amplification for operations is quite close to 1, showing a good correlation with computed values.

D316: The closing operation of the cross regulator on the 4th of June at 10:30 is the one following an order to open Shahar Farid distributary. The other operations are very good according to the amplification ratio, and are responses to changes in upstream level.

D353: The two closing operations on the 4th of June separated by one hour (at 12:10 and 13:15) are responses to the wave coming from RD 316. The first closing operation proved to be not sufficient, that is why the gate operator had to act more abruptly for the second operation.

Ratios between magnitude of observed and computed operations

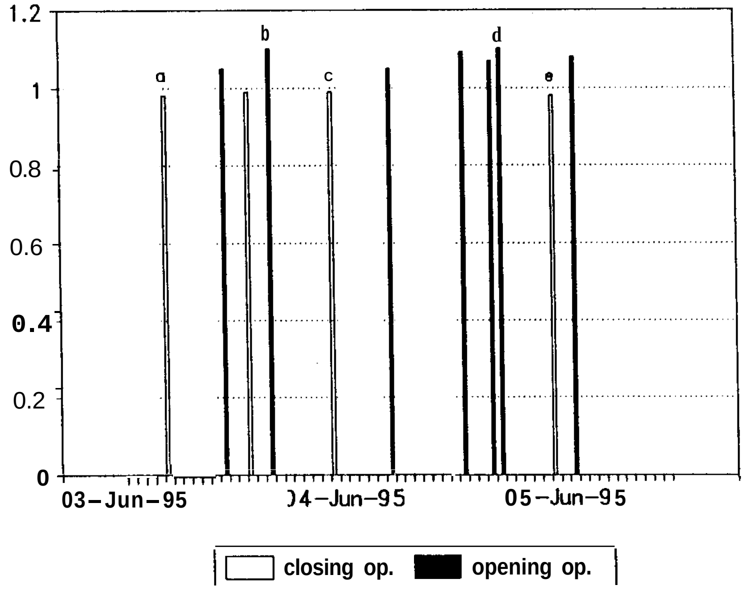


Figure 29: Cross regulator at RD 245

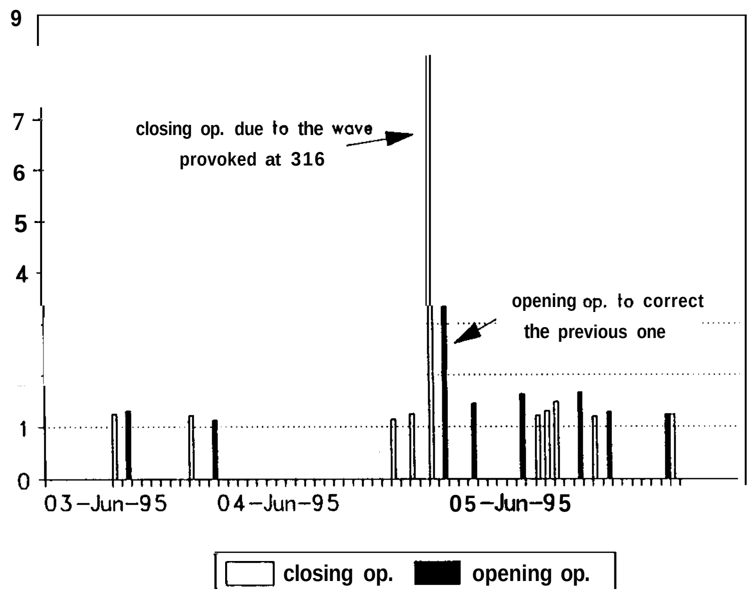


Figure 30: Azim disty at RD 371

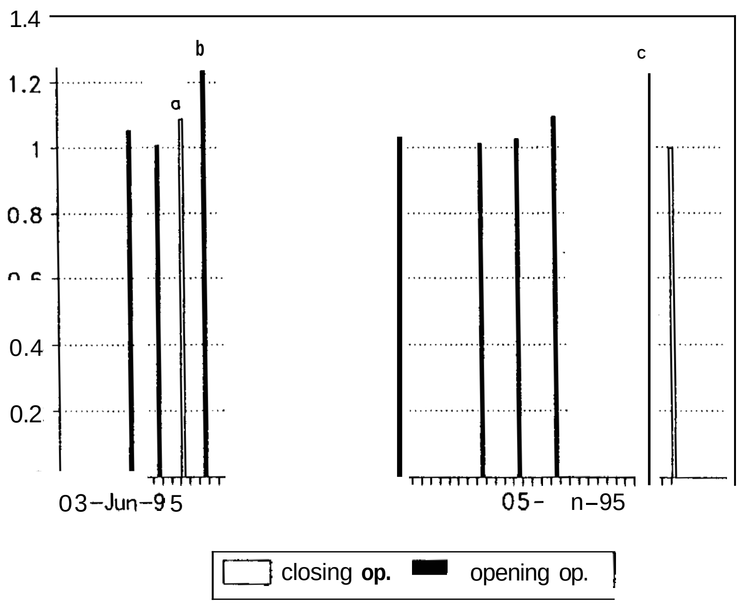


Figure 31: Cross regulator at RD 281

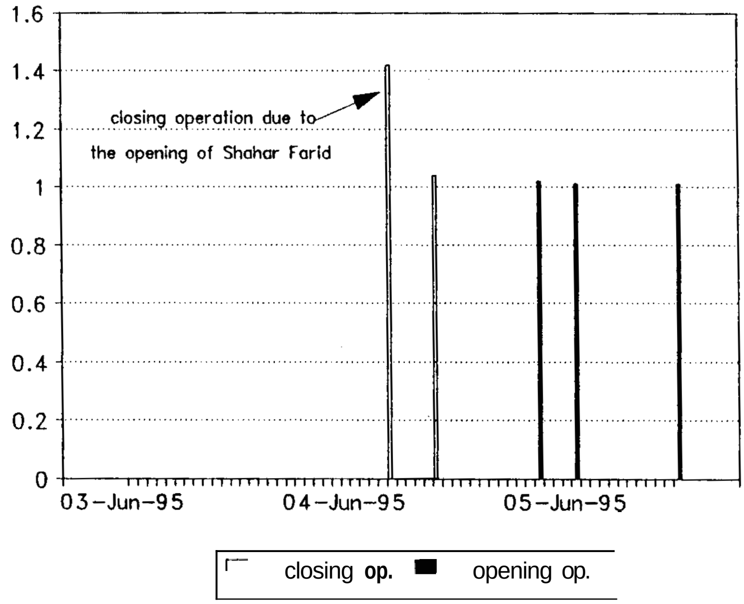


Figure 32: Cross regulator at RD 316

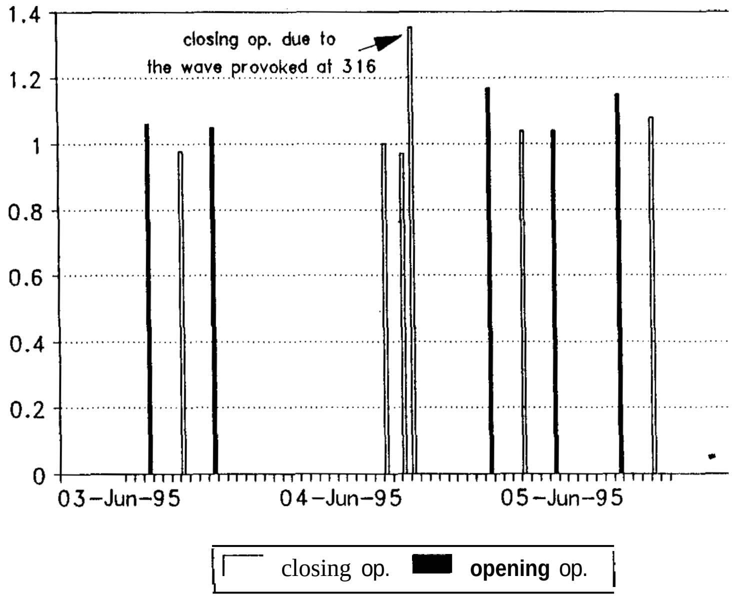


Figure 33: Cross regulator at RD 353

b. Results of the regulation module calibration

The objective of this regulation module is not to reproduce perfectly manual operations. As the hydraulic model was shown to have a rather good accuracy for relatively stable flow, this regulation module (with the assumptions made for its programming) is only valid for "normal" operations. The criteria chosen to evaluate the calibration are the time of the first operation and the global amplitude of operation in response to a wave.

Outputs of the interviews:

- the gauge readers in Chishtian Subdivision work all day long. As an operator is usually helped by a regulation baildar, the gates are operated day and night. The duration of the day work was therefore set to 24 hours.

- the maximum and minimum amplitudes of operation were found rather constant along the canal at 25 cm and 1 cm, except at the tail, where a closing operation of 27 cm was observed.

Table 6.1.4: Comparison between field values and values simulated by the regulation module.

Regulators	Field values		Simulated values	
	Amplitude	Time	Amplitude	Time
D245	12.5 cm	43 h	12 cm	46 h
D281	29 cm	46 h	36 cm	47 h
D316	28 cm	35 h [15]	33 cm	35 h
D353	22 cm	37 h	23 cm	37 h
D371	36 cm	38 h	28 cm	38 h

The effect of the opening of Shahar Farid at RD 316 has provoked a very strong reaction at Azim distributary at RD 371.

The calibration process at this point showed that:

- the amplification ratio had to be more than 2 to tackle the large and rapid fluctuations in U/S level.

[15] This wave is the one provoked by the opening of Shahar Farid disty, the 4th of June, at 10:30

- the level was stabilized by operating every 20 min, and the minimum opening of Azim was 16 cm. For the observed values, the operator closed Azim up to 4 cm, and then had to open it again, in two operations, to reach 18 cm. The variations in upstream levels are similar in the two cases, but the fluctuation passed through Azim is far less important in the simulated case.

One by-product of this calibration is therefore that a perfect operator without any information on the future perturbations could stabilize quite well the canal without amplifying too much the fluctuations he receives from upstream; to do this, he has to operate quite often but avoid doing any abrupt action.

Table 6.1.5: Values of coefficients used for the calibrated regulation module.

Regulators	**Amplification coefficient**		**Time between 2 op.**	**Limits of intervention**	**FSD**
	opening	**closing**			
D245	1.08	0.99	1 h	+/- 2 cm	154.17
D281	1.09	1.05	2 h	+/- 2 cm	151.87
D316	1.01	1.05	20 min	+/- 2.5 cm	149.94
D353	1.09	1.07	20 min	+/- 2 cm	147.43
D371	1.68	2.03	20 min	+/- 2 cm	145.45

Remarks:

Reducing the limits of intervention (less than +\-2cm) was found to be of little influence. This stems from the way the regulation module computes the openings: a difference of x cm in upstream level will give a change of about 0.5*x/(FSD-H2) by linearisation of the equation giving the new opening. Therefore, changes in upstream level that were not detected with larger limits of intervention will not cause big changes in openings (some changes will even not be applied if they are smaller than the minimum amplitude of operation).

Changes in the time between two operations and the FSD were much more effective.

Graphs with the results of the calibration are given in Annex B.

Conclusions on the calibration

The hydraulic model SIC for Fordwah Branch was calibrated with a good accuracy in steady state, allowing a validation in unsteady state, where the observed operations were entered in the model to check its calibration over a period of 66 hours. This validation has shown that the levels all along the canal were not deviating too much from the observed values until about 48 hours, if we accept a range of variation of 8 cm. Apart from one point due to the influence of local operations, the discharge at the two control points is within a range of 12 % of the value derived from field observations.

With this model, we are able to simulate the hydraulic behaviour of Fordwah Branch canal in Chishtian Subdivision.

The regulation module calibration also gave good results, showing that the simulation of manual operations can be done with this module Gateman.
After the simulation of actual operations for a given inflow, we are now able to simulate operations on this canal for different inflows, and different rules of operation.

6.2. First outputs of the calibrated model

With this calibrated model, it is now possible to simulate the hydraulics of Fordwah Branch canal in Chishtian Subdivision. The first results are presented below.

6.2.1. Maximum discharge

This calculation is done using the Steady flow module of SIC.
The **theoretical maximum admissible discharge** at the head is given by the sum of maximum discharges to offtakes + Qoutlets + Qseepage. The maximum discharges to offtakes are taken as 1.1*Qdesign. This theoretical maximum admissible discharge for Chishtian Subdivision is **36.7 m^3/s** (1295 cusecs), if we take as a reference the design discharges given in chap 2.

This gives the limits for our calculations.

Process:

- gated offtakes are put in opening computation, with Qtarget=1.1*Qdesign,
- cross regulators are put in adjustable gate,
- the discharge at the head (RD 199) is increased until a model disfunction is observed (overtopping in the main branch, target upstream level not achieved at cross regulator, or target discharge not achieved at offtake).

Results:

Two maximum discharges were computed for Chishtian Subdivision, by doing two simulations with the steady flow module of SIC.

The first one is based on the following assumptions:

- the discharges to gated offtakes shall not be more than 1.1*Qdesign
- the upstream level at regulators shall not rise more than 10cm above actual pond.

This maximum discharge is therefore the **maximum discharge with actual local regulation**. The result is **33 m^3/s** (1165 cusecs).

The second one is the maximum possible discharge using full free board, and keeping the discharges to offtakes at a maximum of 1.1*Qdesign. It is therefore the **maximum admissible discharge without overtopping in the canal or in the distributaries**.
The result is **36.7 m^3/s** (1295 cusecs).
This simulation also gives the maximum storage capacity for reaches in the subdivision.

Table 6.2.1: Maximum discharges in Chishtian Subdivision

Offtakes	Qtarget (=1.1*Qdesign)	Simulation 1 (inflow=33 m^3/s)	Simulation 2 (inflow=36.7 m^3/s)
Daulat	6.49	5.10	6.49
Mohar	1.1	1.1	1.1
3L (*)	0.55	0.38	0.44
Phogan (*)	0.55	0.77	0.91
Khem Gahr	0.94	0.94	0.94
4L (*)	0.55	0.57	0.67
Jagir	0.80	0.80	0.80
Shahar Farid	4.73	4.73	4.73
Masood	1.1	1.1	1.1
Soda (*)	2.42	1.76	2.07
5L (*)	0.11	0.31	0.41
Fordwah	4.95	4.95	4.95
Mehmud	0.22	0.22	0.22
Azim	7.59	5.66	7.26

Note: the open flumes are signaled with a (*).

The following assumptions were made:

- the discharge going into the direct outlets is constant, equal to 1.34 m^3/s (47 cusecs)
- the seepage losses are constant, equal to 3.28 m^3/s (116 cusecs).

Remarks: These rather simple simulations give interesting results. The discharge going into 3L is never more than 80 % of the design discharge, even when the discharge at the head is maximum. This tends to point out a problem in the actual physical state of this distributary. The same remark applies to Soda disty. On the other hand, 5L disty takes always more than its share.

In both simulations, we observed that the discharges going into non gated offtakes (open flumes) were usually above the limit of 1.1*Qdesign. But this observation does not invalidate our calculations, as this is also the case in current situation (even when the discharge at the head is lower than 70% of Qdesign).

6.2.2. Storage capacity

The maximum volumes and the minimum free boards in reaches were also calculated for the two discharges calculated above, as well as for the situation observed in the field. The minimum free board is a critical limit for operations, and not an operational limit.

Table 6.2.2: Volumes in reaches (thousands of m^3)

Reaches	Actual situation Q = 25.45 m^3/s	Simulation 1 Q = 33 m^3/s	Simulation 2 Q = 36.7 m^3/s
D199 - D245	552	634.5	686.6
D245 - D281	321.2	381.5	408.7
D281 - D316	273.9	323.5	345.6
D316 - D353	226.7	232.7	262.6
D353 - D371	105.2	106.4	117.8

Table 6.2.3: Minimum critical free boards in reaches

Reaches	Actual situation Q = 25.45 m^3/s	Simulation 1 Q = 33 m^3/s	Simulation 2 Q = 36.7 m^3/s
D199 - D245	26 cm	23 cm	9 cm
D245 - D281	42 cm	22 cm	8 cm
D281 - D316	30 cm	22 cm	9 cm
D316 - D353	39 cm	39 cm	5 cm
D353 - D371	17 cm	17 cm	5 cm

Table 6.2.4: Increase in storage from actual situation to simulation 2.

Reaches	Increase in storage
D199 - D245	25.4 %
D245 - D281	27.2 %
D281 - D316	26.2 %
D316 - D353	15.8 %
D353 - D371	12.0 %

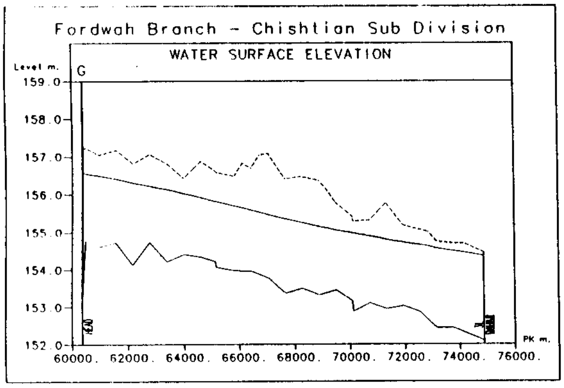
The risks of overtopping have been pointed out in different parts of the reaches (see Figure 34):

Reach 1: just upstream of D245
Reach 2: at Phogan disty
Reach 3: at Jagir disty
Reach 4: just upstream of D353
Reach 5: downstream the weir at RD 363

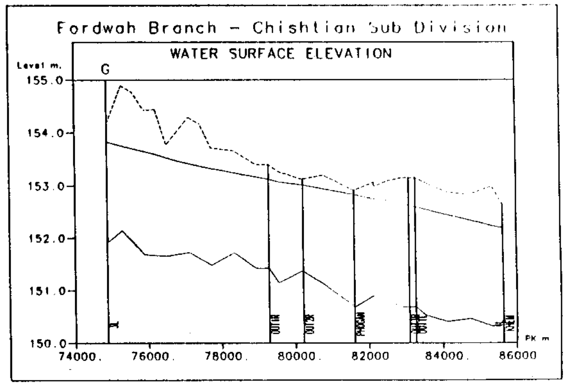
The reaches with the bigger possible increase of storage in % (reaches 2 and 3) are the reaches where the risk of overtopping is not located at a point just upstream of a regulator. The gauge readers have therefore no visual control for their operations. This might explain why the operators at these regulators keep a low pond, as they cannot check the critical points directly. On the contrary, the first and fourth reaches show a risk of overtopping only located just upstream of the regulators. The level is easily controlled at these points, allowing a storage without risks of overtopping elsewhere in the reach. Those two reaches are therefore the ones the more likely to be used for storage, as it could be done without risks of overtopping.

This is an important output of this study, as these two reaches can be used in case of an emergency:

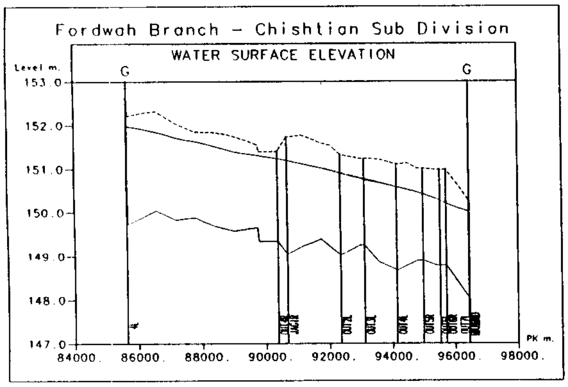
- the reach 1 (between RD 199 and RD 245), used as a "buffer" to temper the fluctuations coming from RD 199, could improve the situation in all the subdivision.
- a temporary increase of storage in the reach 4 (between D 316 and D 353) could give some time to the operator at RD 371, allowing him to decrease the upstream level at his point to be able to receive the coming wave without any problem of overtopping in the main branch, or of a sudden increase of discharge in Azim distributary.



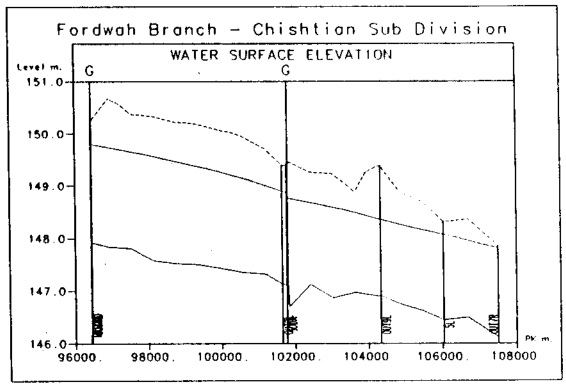
Reach 1 (D199 to D245)



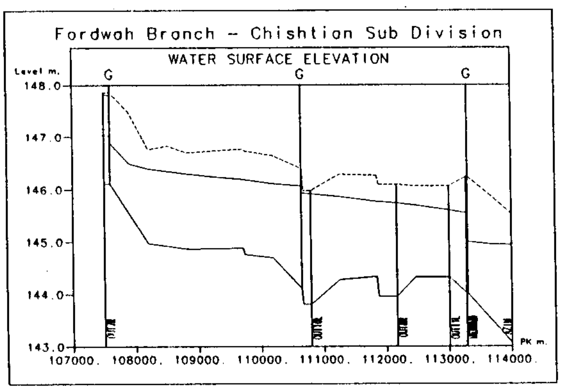
Reach 2 (D245 to D281)



Reach 3 (D281 to D316)



Reach 4 (D316 to D353)



Reach 5 (D353 to D371)

Figure 34: Minimum critical free-board for reaches & points with a risk of overtopping

6.3. Simulation of scenarios

The simulations with manual operations are first presented, and then the scenario 'automated gates', as it is quite far from actual possibilities for implementation.

6.3.1. Situation a: effect of local operations on a real inflow

For the real inflow pattern monitored in the field during 72 hours, we compare real operations with operations simulated by the regulation module Gateman, where unjustified operations at regulators are eliminated, keeping the real operations at offtakes.

As a result, the discharge downstream the regulator at RD 245 is 'smoothened', and as the number of operations is reduced, the fluctuations are not amplified. This is the case at each cross regulator, giving a much stable discharge downstream. The effect of this can be seen at the tail RD 371 (Figure 38), where the discharge passed in Azim disty is much more stable, even if it suffers from the negative wave provoked by the opening of Shahar Farid disty at RD 316.

Table 6.3.1. Number of operations at cross regulators for the scenarios 0 and 1 (situation a).

Regulators	**Scenario 0: Real operations**	**Scenario 1: Improved operations**
RD 245	11	7
RD 281	11	4
RD 316	5	11[16]
RD 353	11	12
RD 371	18	14

In the real situation, the operations are more numerous at the tail, as the gate operator has to respond to fluctuations created by unjustified operations upstream, especially the negative wave created at RD 316. The beneficial effect of improved operations is visible in the upstream part of the canal, not affected by this operation at Shahar Farid disty. The number of operations at RD 245 are reduced from 11 to 7, and from 11 to 4 at RD 281.

[16] The operations are sometimes more numerous with the use of Gateman; this is due to the fact that it only reacts to a hydraulic change, and cannot anticipate any action. This is particularly clear at RD 316, where the gauge reader has closed the cross regulator when he opened Shahar Farid, to keep the upstream level at the pond level. This is not possible for Gateman, who needs therefore more operations to stabilize the upstream level.

The discharge at Daulat disty corresponding to the real and improved operations of the cross regulator at RD 245 is shown on the Figure 36. Between the 12th and 22nd hours after the beginning of the monitoring period, a decrease in the discharge passed downstream corresponds to an increase in the discharge to offtakes. The fluctuations provoked by unjustified operations of the regulator also have an effect on the discharge to offtaking distributaries. These operations are eliminated with Gateman, and the result is a more constant discharge to the distributaries. The operations at the regulator were also aimed at delivering a constant discharge to 3L distributary, a small disty with an open flume offtake. As it is quite sensitive to changes in upstream level, the gauge reader operates the gates of the regulator to maintain the pond even when the deviation from the targeted upstream level is less than 1.5 cm. By suppressing these changes, the discharge to 3L is only 5% lower, and the discharge downstream the main branch is much more constant. As a matter of fact, the rotational plan stipulated that Daulat and the two other disties under the control of the regulator at RD 245 were in last preference. This little change in the discharge to 3L should not have provoked new gates settings at the regulator.

The positive effect of improved operations is also visible at RD 281, and at the tail RD 371 (Figures 37 and 38). The discharge downstream the regulator at RD 281 is smoothened, and the highly variable discharge given to Azim disty is also stabilized.

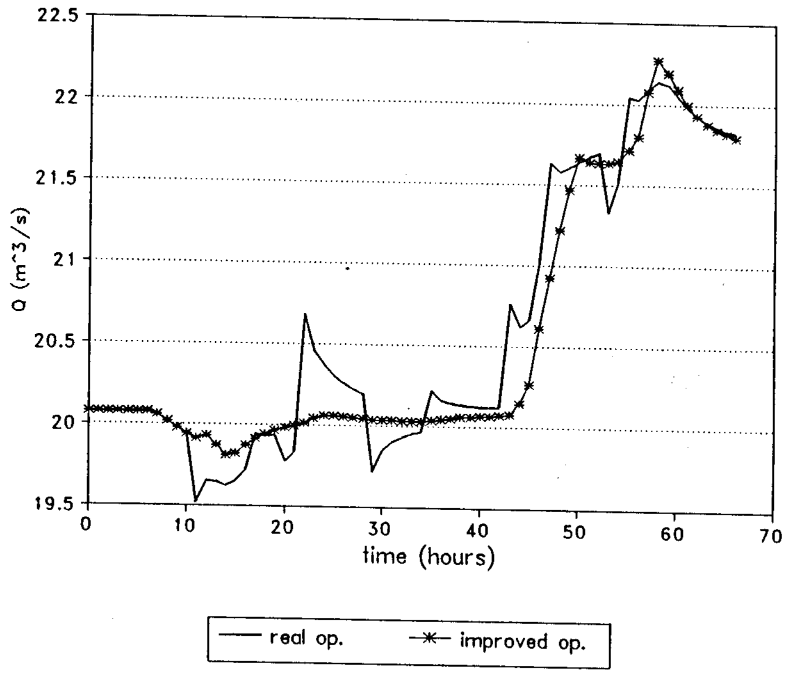


Figure 35: Discharge D/S RD 245

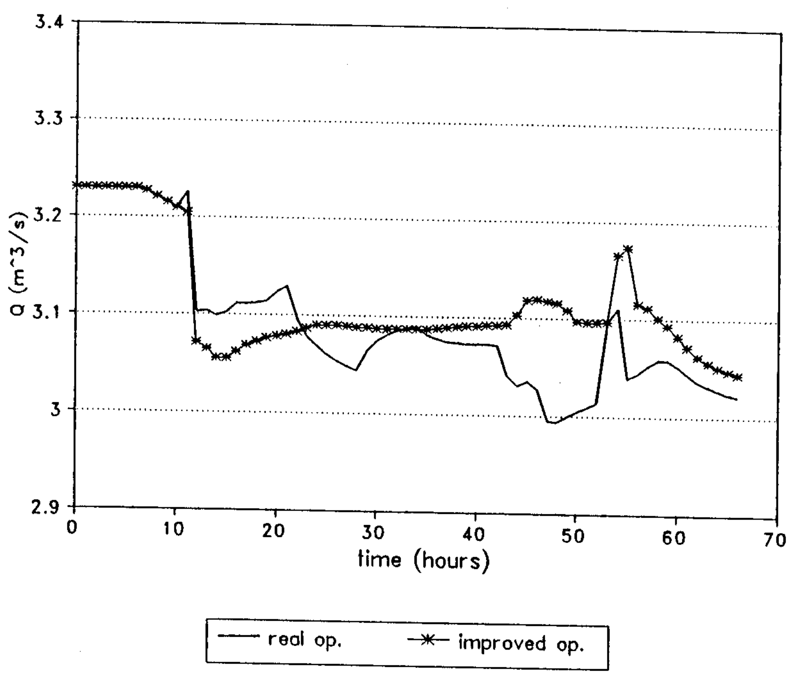


Figure 36: Discharge at Daulat disty

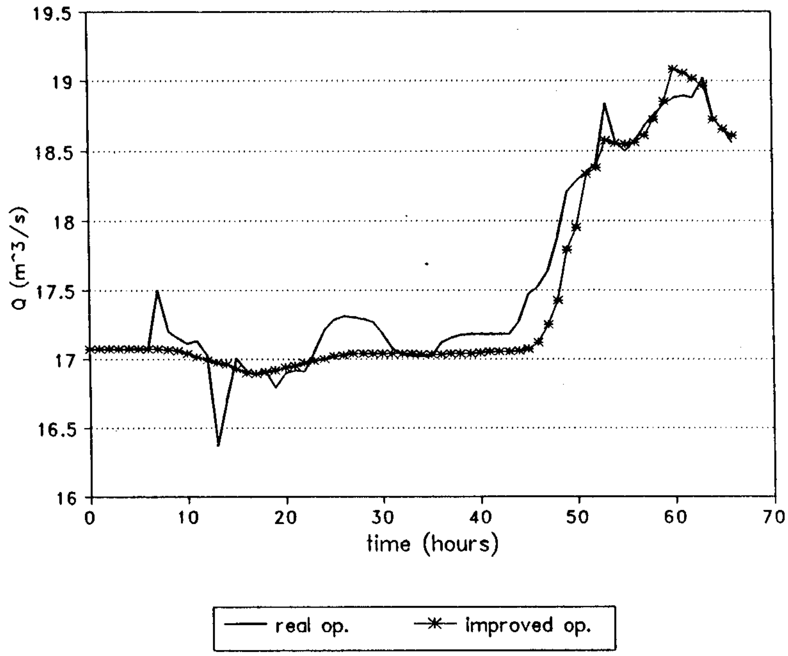


Figure 37: Discharge D/S RD 281

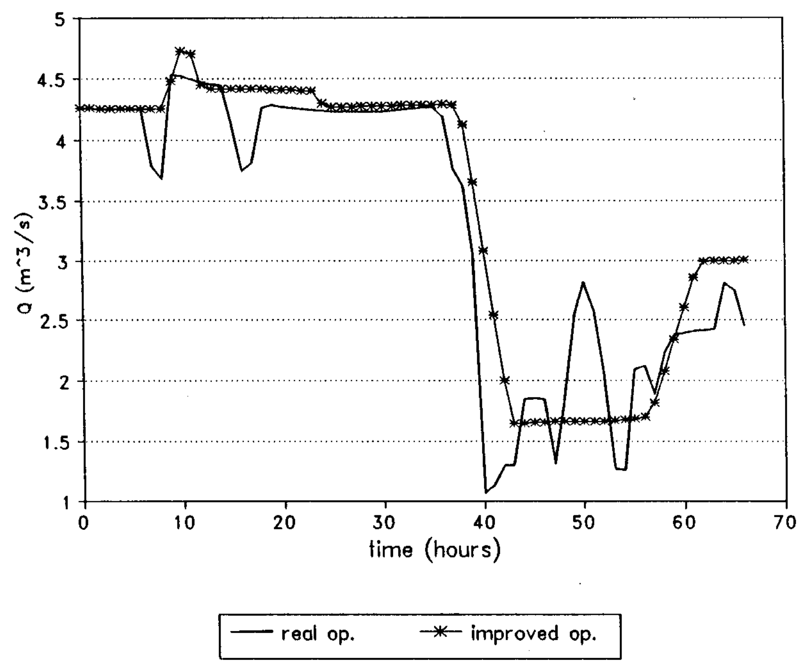


Figure 38: Discharge at Azim disty

The effect on the water distribution to offtakes can be assessed with the indicator defined in the methodology. The difference between the indicator for scenario 0 and 1 is displayed in terms of percentage of time. The other figure gives the same difference, but weighted with the design discharge of each distributary, to see the improvement in terms of increase in 'useful' volume (useful is meant by reference to the choice of the indicator defined in the methodology).

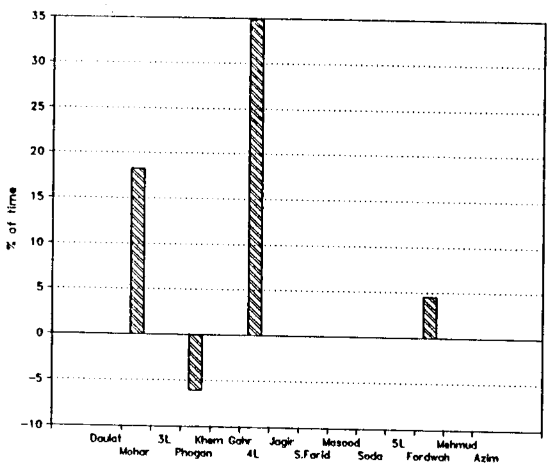


Figure 39: Difference between scenarios 0 and 1, % of time

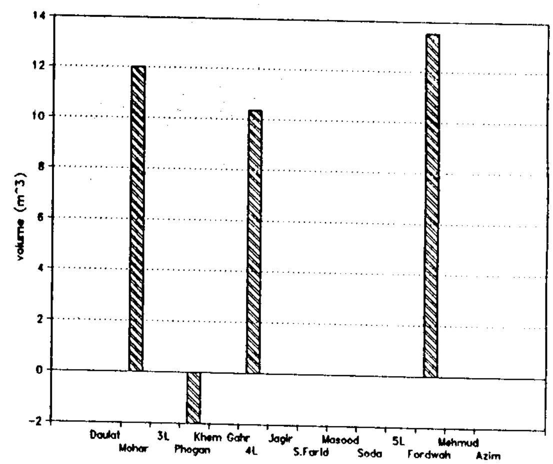


Figure 40: Increase in volume between scenarios 0 and 1

The effect of improved operations on offtaking distributaries is clear at the tail of the system, as the discharge to Fordwah disty is within the defined range of [85%, 110%] of the design discharge during 4 hours (1 for real operations), for a total duration of simulation of 66 hours. The increase in percentage of time is therefore of 4.5%. It represents an increase of 13.5 m^3 in terms of useful volume delivered to this distributary.

6.3.2. Situation b: increase of discharge at the head

With a real inflow prolonged of 24 hours, we compare improved operations and feed forward control. The discharge at the tail, that was supposed to benefit from the increase of discharge coming from upstream, is used to compare the two situations (Figures 41 and 42). The figures 43 and 44 display the upstream elevations with reference to FSD (target upstream level), and the gate openings at each regulator for the 90 hours of the simulation.

Table 6.3.2. Number of operations at cross regulators for scenarios 1 and 4 (situation b).

Regulators	**Scenario 1 : Improved operations**	**Scenario 4: Feed forward control**
RD 245	9	1
RD 281	7	1
RD 316	8	1
RD 353	9	1
RD 371	13	1

The feed-forward control has the advantage of reducing drastically the number of operations (only one operation is performed at each regulator), and therefore reducing the time needed to stabilize the levels in the canal. The effect on the discharge at Fordwah distributary is more important for the feed forward control, because upstream levels are allowed to fluctuate more. But this perturbation stays in the limits defined for the possible variation of discharge.
In improved operations, we see that a great number of operations is required to stabilize the levels in the canals, with closing and opening operations at each regulator.

The feed-forward control is possible only with a good maintenance of the banks, as water levels are allowed to fluctuate; if the banks are weak, the safety margin for operations has to be increased.

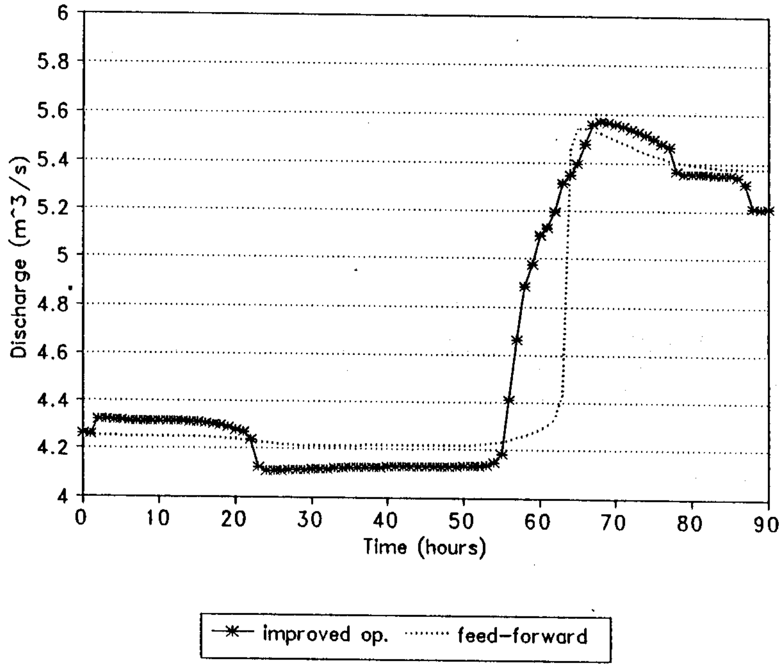


Figure 41: Discharge at Azim disty

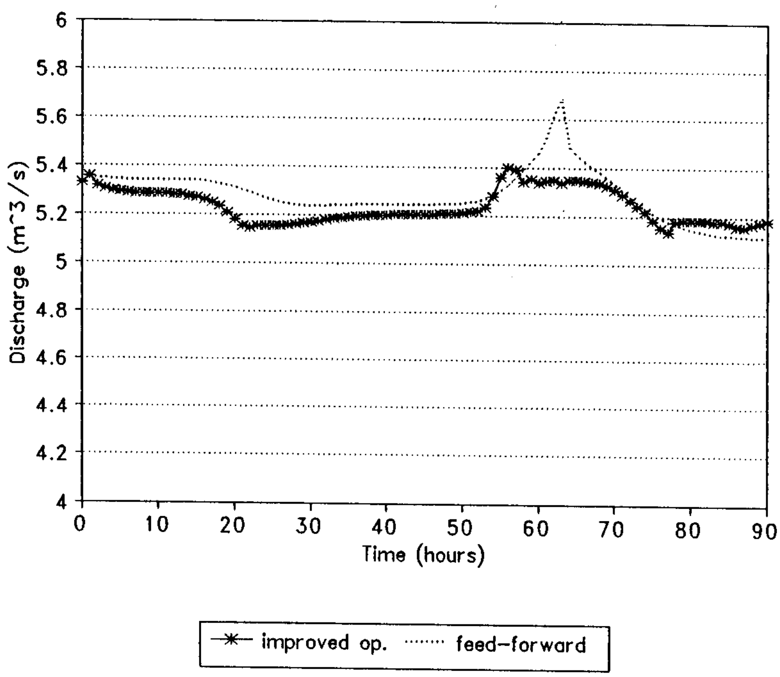


Figure 42: Discharge at Fordwah disty

6.3.3. Situation c: negative step of discharge at the head

With a negative step of 2 m^3/s at RD 199, we compare two options:

- case 1: operation of the cross regulator at RD 245 to keep the upstream level and the discharge in Daulat constant,

- case 2: operation at Daulat distributary to take the perturbation and keep the discharge downstream the main branch constant.

If the gauge reader acts without any special order, he will operate the cross regulator, therefore pass all the fluctuation downstream. If all gauge readers act the same way, it will end at the tail, where one of the two disties Azim or Fordwah will have to suffer. This is clear from the Figure 47: with all other gauge readers acting as usual, i.e. keeping their upstream level constant, the result is a shortage at the tail. To follow the preference order, the discharge should be kept constant in the main branch, and the gauge reader can perform such operation by operating only Daulat disty. This is the case when the regulator at RD 245 is not operated, and Daulat is used to take the fluctuation.

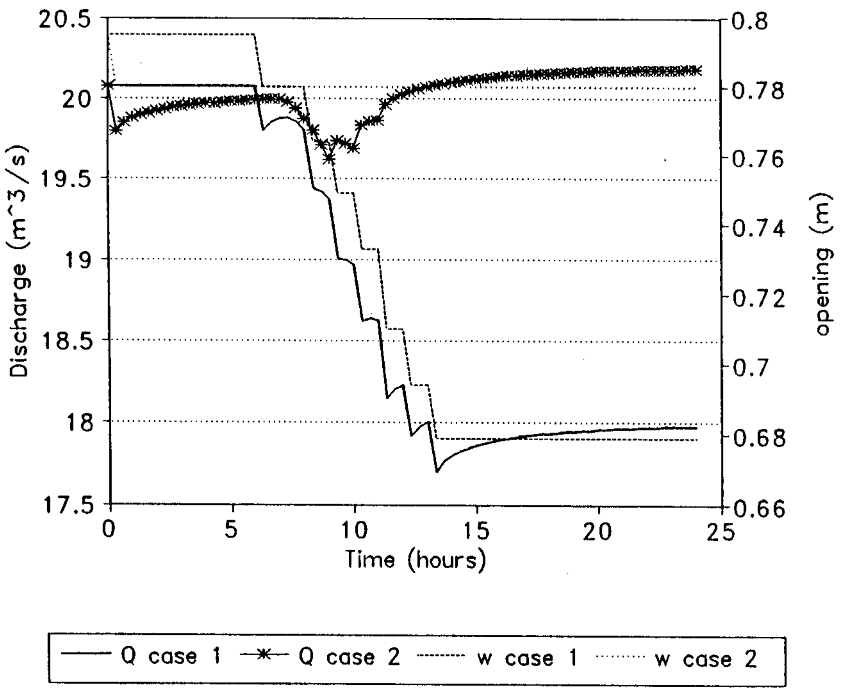


Figure 45: Discharge at RD 245

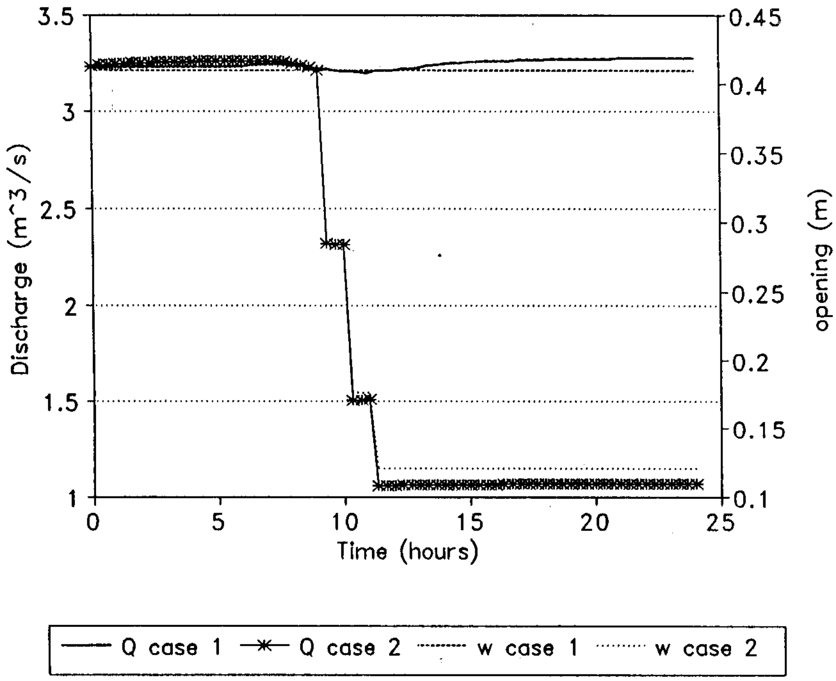


Figure 46: Discharge at Daulat disty

The result of this absorption of fluctuation can be seen at the tail, where the discharge passed through Azim distributary is almost constant, and does not provoke any reaction from the gauge reader at this point (Figure 48).

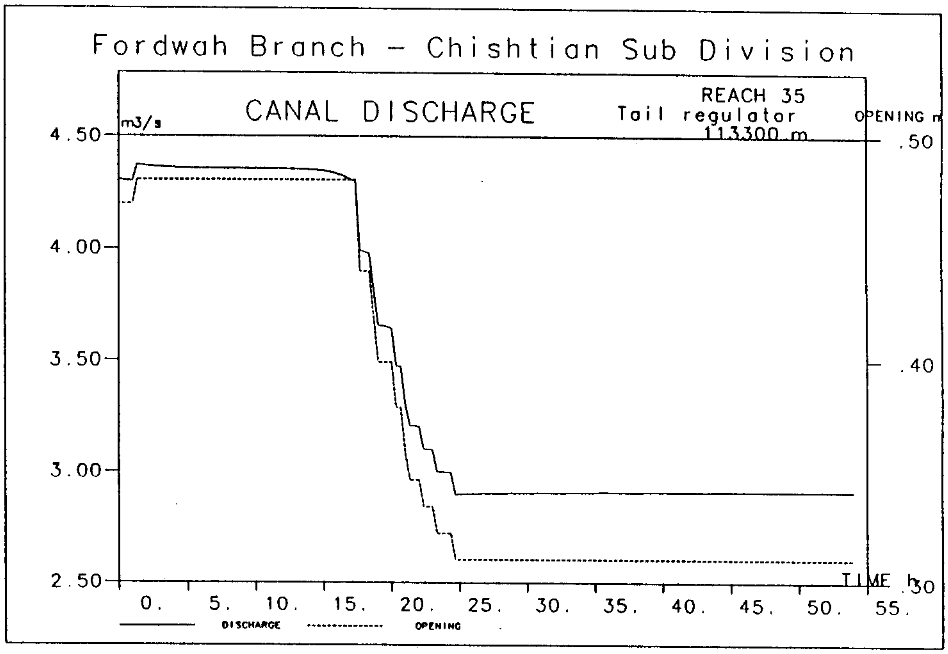


Figure 47: Discharge at Azim disty, case 1

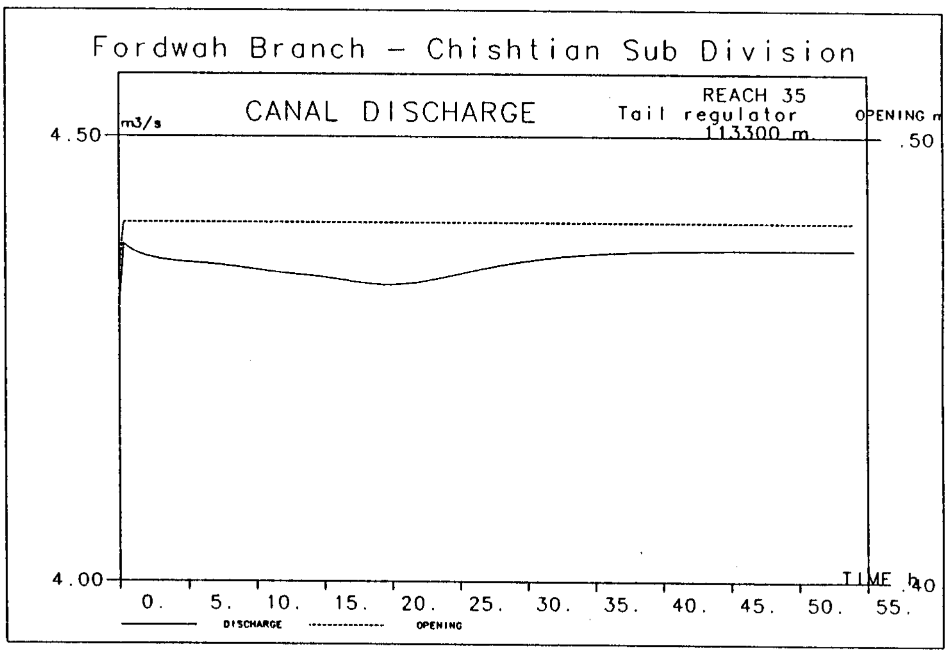


Figure 48: Discharge at Azim disty, case 2

6.3.4. Situation d: shift of rotation from Daulat to Azim

For this situation, Daulat distributary is closed at the time 00:00, beginning of the simulations. The inflow at the head is constant, and only cross regulators are operated along the canal.

<u>Scenario 1: Improved operations</u>

If we use the calibrated regulation module Gateman to simulate actual improved operations, we see that the response to the wave is amplified the further it propagates downstream. The operators perform a lot of operations (even if they are all justified hydraulically), because they only react to local variables, and keep a safety margin in their operations. The number of operations at cross regulators increases the further we go downstream, to end with 18 operations at RD 371. The target is reached after 13 hours, but the levels in the canal are still not stabilized 24 hours after the release at Daulat. The peak of discharge passed to Azim disty is 7.25 m^3/s (256 cusecs).

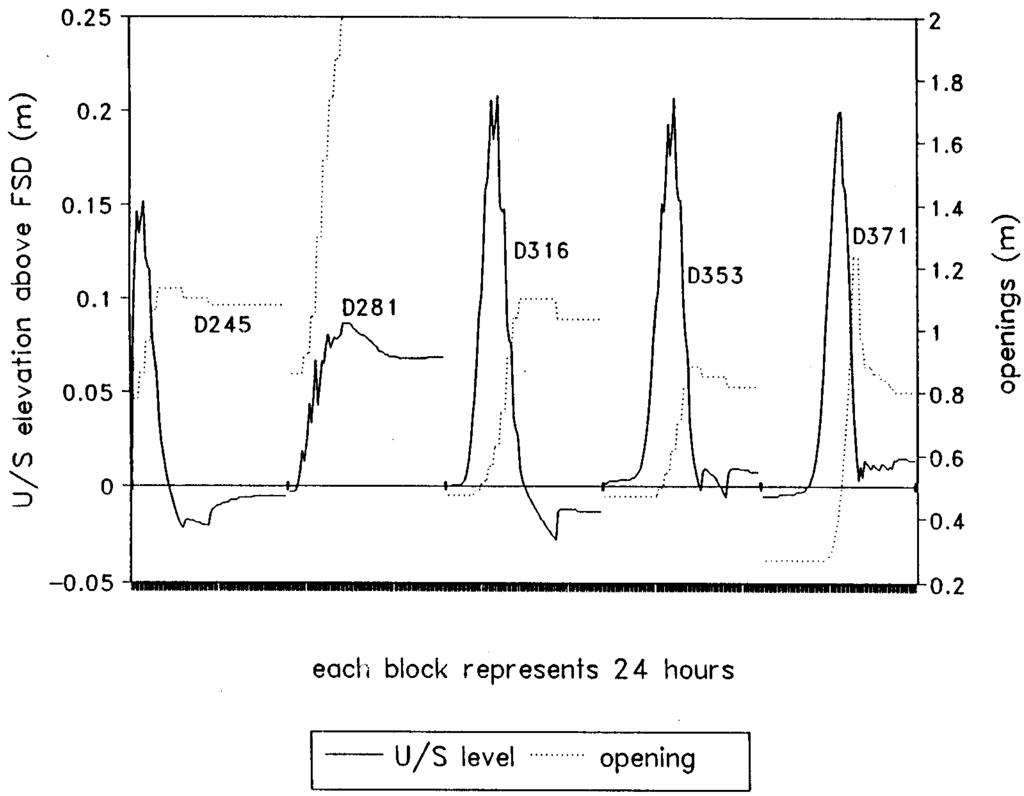


Figure 49: Improved operations (Scenario 1)

Scenario 2: Buffer capacity

In this scenario, operators are told to let the upstream level fluctuate more, until the limit defined by the minimum critical free board in their upstream reach, and then to operate with their usual rule. This needs a communication system, as the operators need to know whether the wave coming is important or not, and short or long. The information should consist in the approximate amplitude of the wave, and the time it will last. With these order and information given to all the gauge readers, the number of operations is reduced from 7 to 1 at RD 281, and from 18 to 11 at the tail. The target is reached one hour later, but the amplitude of the wave is not amplified, and the levels are stabilized quicker. By using the buffer capacity of each reach, we manage to reduce the amplitude of the wave at the tail of 5 cm (0.16 ft) in elevation and of 1.25 m^3/s (44 cusecs) in discharge. The final time of stabilization is about 16 hours.

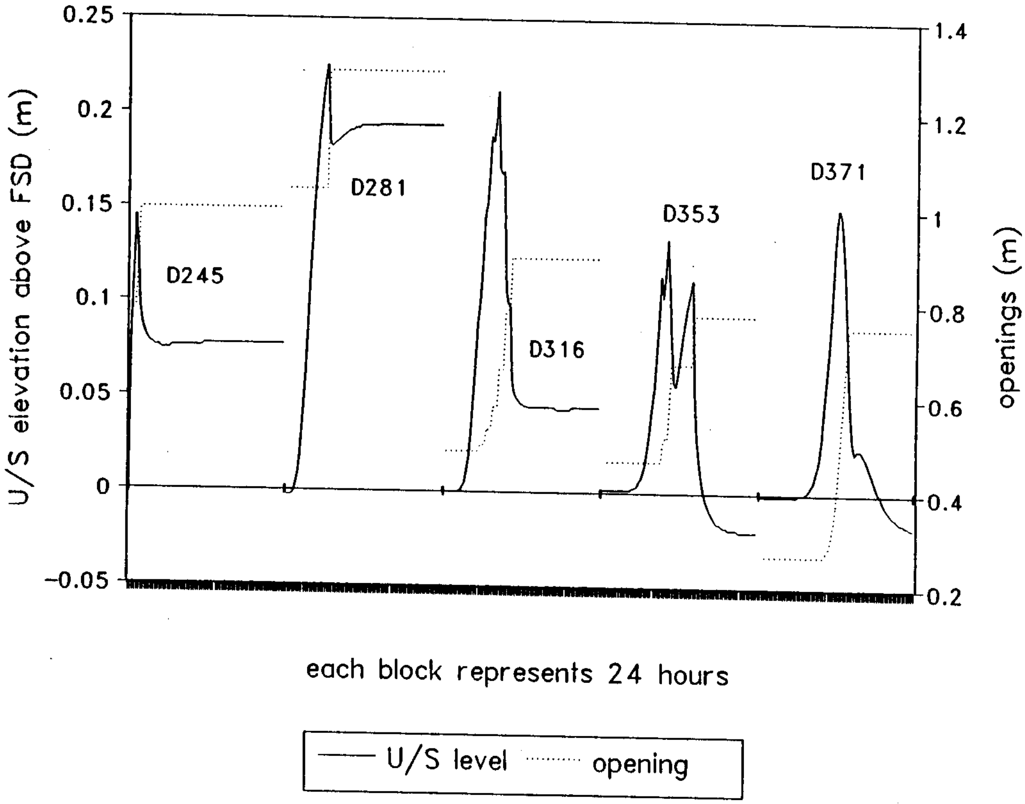


Figure 50: Use of buffer capacity (scenario 2)

Scenario 3: Information at the tail

In this scenario, only the two gauge readers at the tail of the system are in communication with each other. The gauge reader at RD 353 informs the gauge reader at RD 371 when a wave is arriving; the latter decreases the upstream level at his point (if all the disties are not already at full supply), while the former uses the capacity of his reach to store some water for a short time. The other gauge readers located upstream act as usual (improved operations, scenario 1).
The number of operations at the two last regulators are reduced (from 7 to 4 at RD 353, and from 18 to 10 at RD 371), and the amplitude of the wave is reduced, compared to scenario 1. The time of stabilization is not reached in 24 hours, but the wave is passed safely downstream. With information only at the tail, the amplitude of the wave passed to Azim is reduced of 5 cm in elevation, and of 0.4 m^3/s (14 cusecs) in discharge.

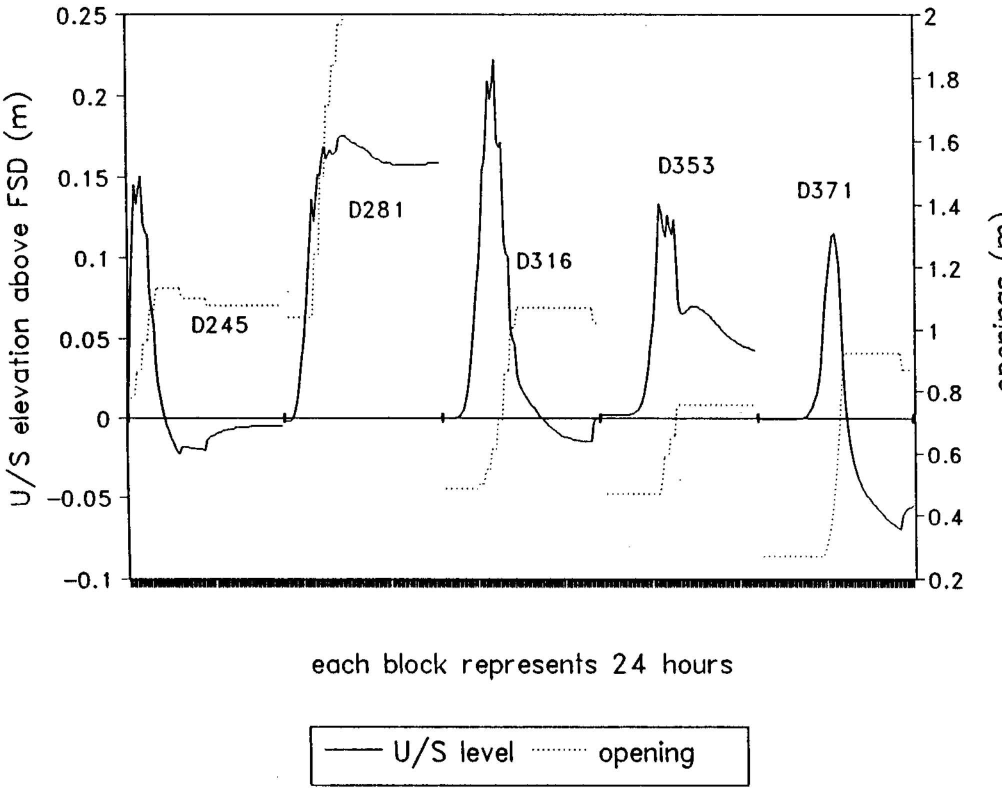


Figure 51: Linked operations (scenario 3)

Scenario 4: Feed forward control

In this scenario, orders in terms of time and amplitude of operations are given to gauge readers. The amplitude of operation is computed with the Unit 2 of SIC, by simulating initial and final states. The time of operation is computed with Unit 3 of SIC, with successives simulations to find the time lags for each regulator.

This scenario is the best one in terms of time to reach the target, time of stabilization, and non amplification of the wave. One operation is performed at each point, and the wave is 'pushed' downstream, so that the fluctuation is shorter in time. The maximum elevation is limited to the elevation corresponding to the minimum critical free board. The amplitude of the wave at Fordwah disty is 0.1 m^3/s (3.5 cusecs) less than the one passed with actual operations, and much shorter in time. The target is reached in 11 hours, and the number of operations at regulators is reduced from 18 to 1 at RD 371 (with reference to scenario 1).

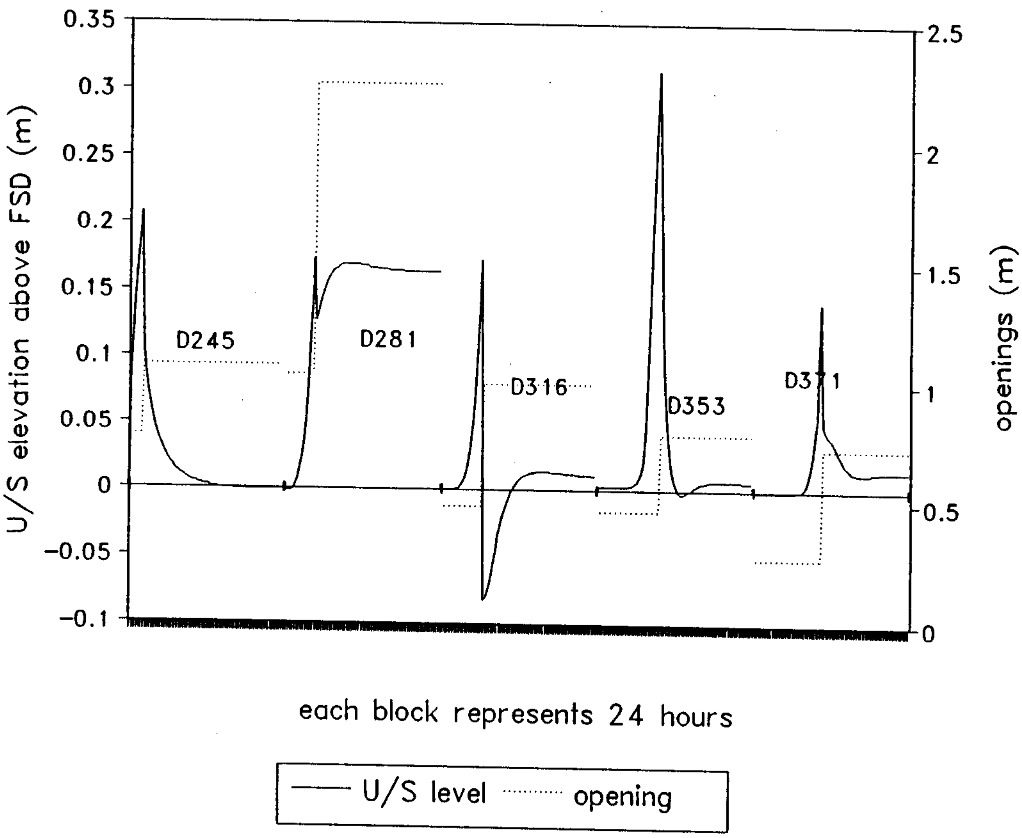


Figure 52: Feed forward control (scenario 4)

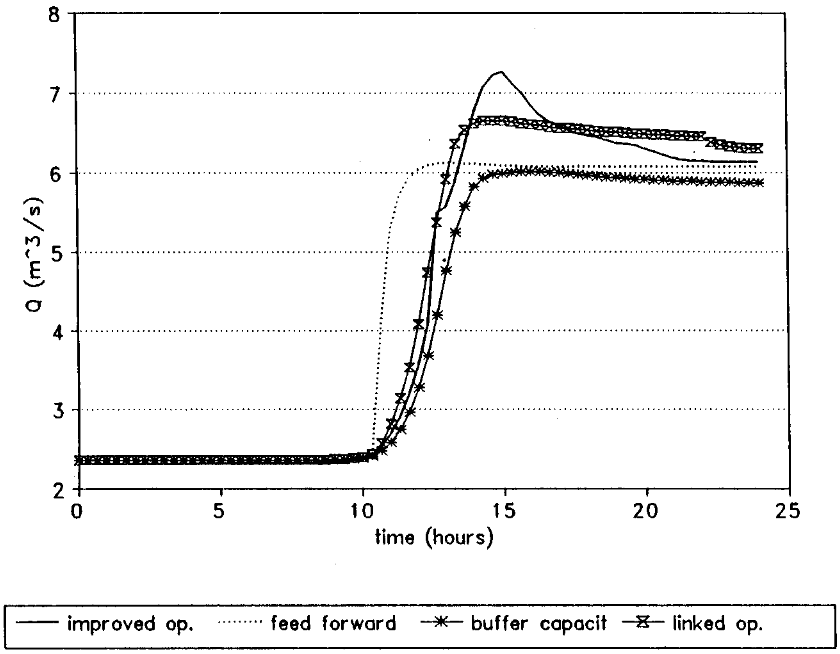


Figure 53: Discharge at Azim disty

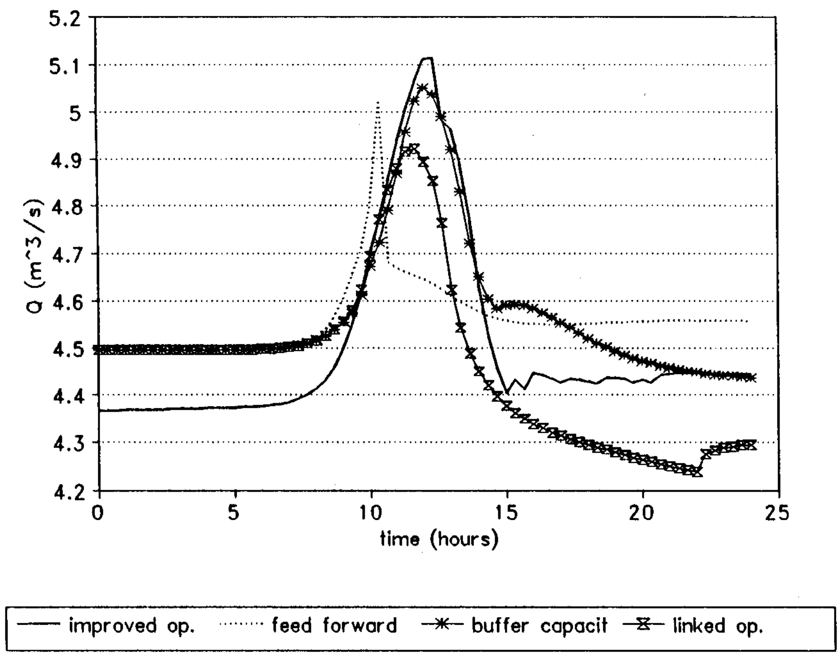


Figure 54: Discharge at Fordwah disty

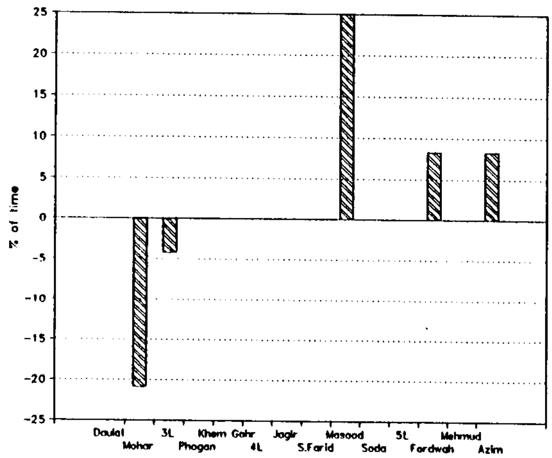


Figure 55: Difference between scenarios 1 and 4, % of time

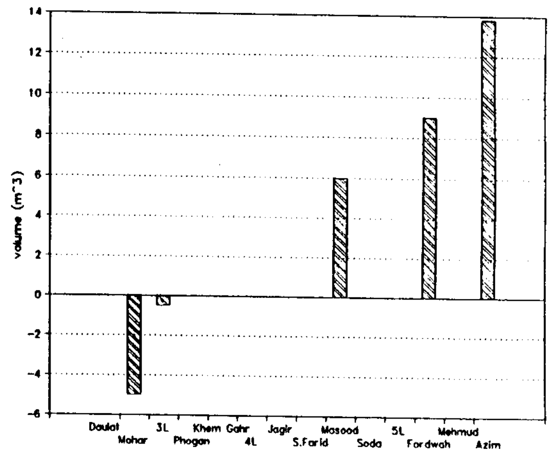


Figure 56: Increase in volume between scenarios 4 and 1

The comparison of indicators for the scenarios 1 and 4 shows that the improvement is felt at the tail of the subdivision, where Azim is fed according to the defined criterion two hours more in the feed forward scenario than in the improved operations, for a total duration of simulation of 24 hours. The increase in percentage of time is of 8.3% for Azim. The excess that was given to Fordwah and Mohar with improved operations is reduced to zero with the feed forward control.

This represents an increase of 23 m^3 of useful volume for the distributaries at the tail (Fordwah and Azim).

Table 6.3.3. Number of operations at cross regulators for the scenarios 1 to 4 (situation d).

Regulators	Scenario 1: Improved operations	Scenario 2: Buffer capacity	Scenario 3: Linked operations	Scenario 4: Feed forward
RD 245	6	2	6	1
RD 281	7	1	6	1
RD 316	9	6	8	1
RD 353	7	4	4	1
RD 371	18	11	10	1

6.3.5. Situation e: local maintenance at RD 363

A local maintenance at RD 363 gives a substantial increase in storage capacity for the last reach, and therefore more flexibility in operations: the upstream level at the end of that reach (RD 371) can be allowed to fluctuate more.

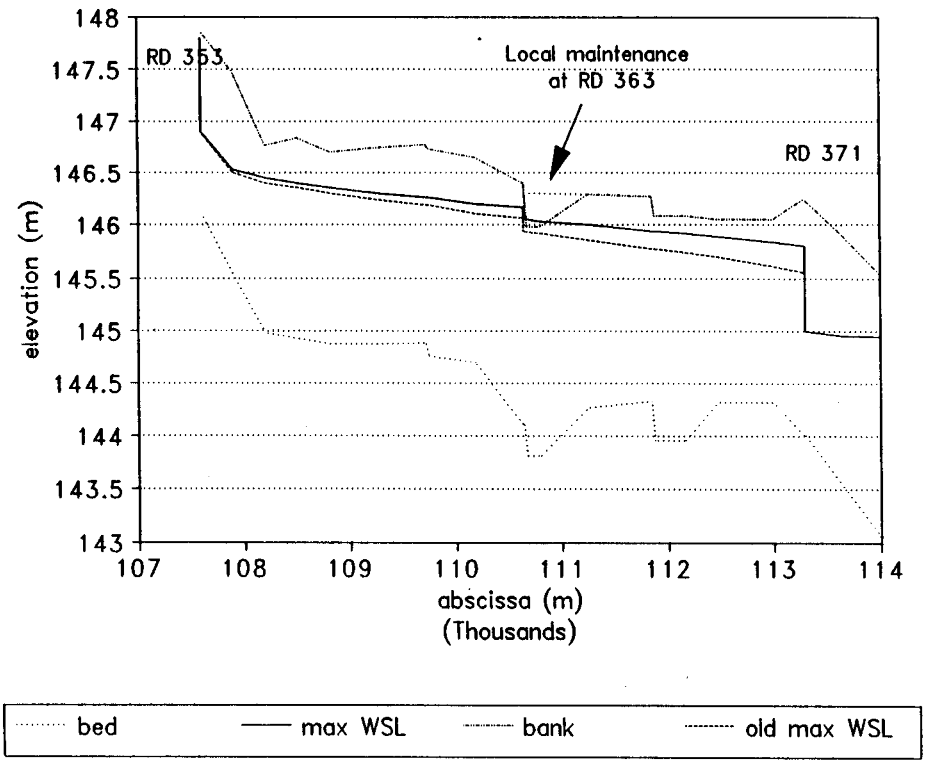


Figure 57: Water levels, Reach 5

With a local maintenance, an increase of the height of the left bank of 32 cm (1.1 ft) enables the gate operator at RD 371 to have a supplementary security margin of 25 cm (0.82 ft) for his operations (if we keep a minimum critical free board of 20 cm or 0.65 ft).
The maintenance can be done over a distance of 500 m (1640 ft), and mainly on the left bank, which is the lower one.
The maximum storage with this new geometry will be equal to 131.5 thousands of m^3. This represents an increase of 11% compared to the maximum storage capacity in actual situation.

Evaluation of the cost of this local maintenance:

The length, height and width that have to be maintained represent:

Length	= 500 m
Height	= 0.25 m
Width	= 7 m

This represents a volume of earth of 875 m^3, or 30900 cft[17].

As the cost for earth work in Fordwah Branch is evaluated to 675 Rs per 1000 cft, the total cost would be of 21,000 Rs (about 700 US$).

[17] cft = cubic foot

6.3.6. Automated gates

a. Results of the calibration

The controllers are calibrated from upstream to downstream, with the Ziegler Nichols ultimate sensitivity method (non parametric calibration). These coefficients are then reduced, from downstream to upstream, in order to reduce instabilities. This is a necessary step, because of important coupling and non-linear effects.

We get the following K (proportional coefficient) and Ti (integral time) coefficients.

Table 6.3.4. Coefficients for the PI controllers

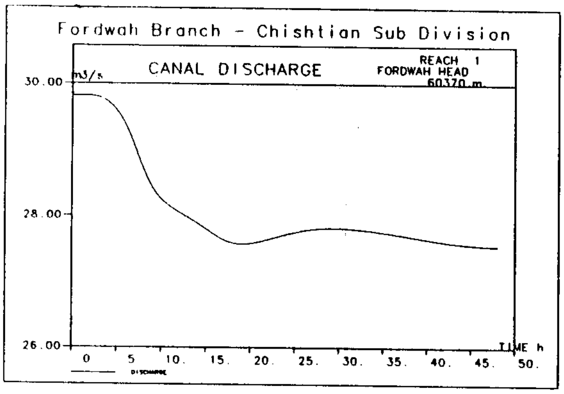
Controllers	K	Ti (seconds)
Q head	4.20	4000
W RD199	0.45	70900
W RD245	1.40	60900
W RD281	1.65	70300
W RD316	1.00	35800
W RD353	1.25	44000

b. Results of the simulation

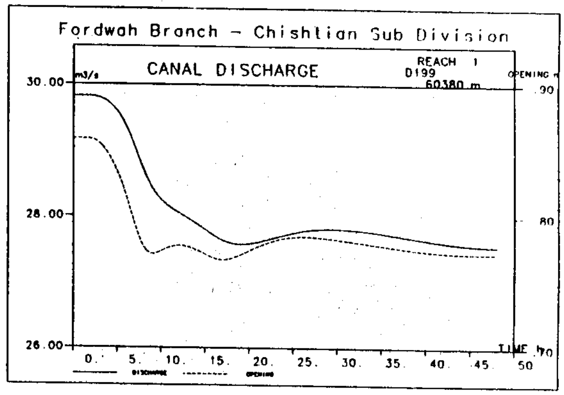
Situation: from the initial steady state of 25/08/95 (see field test 2), the 2 gates of Azim are closed from 0.43 to 0.23 m. The simulation lasts 48 h.

Results are displayed in Figure 58. All controllers react in series to the closing of Azim, and reduce corresponding regulator gate openings to stabilize their controlled variables (water level at the downstream end of the pool). The discharge at the head decreases in order to stabilize the water level upstream of the regulator at RD199.

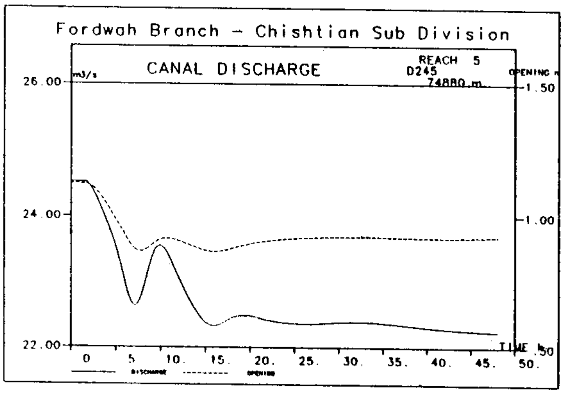
The controllers are stable and precise, even if they are quite slow. The total stabilization seems not to be completely reached after the 48 hours of simulation. This stems from the fact that we have reduced the coefficients, to avoid too big oscillations.



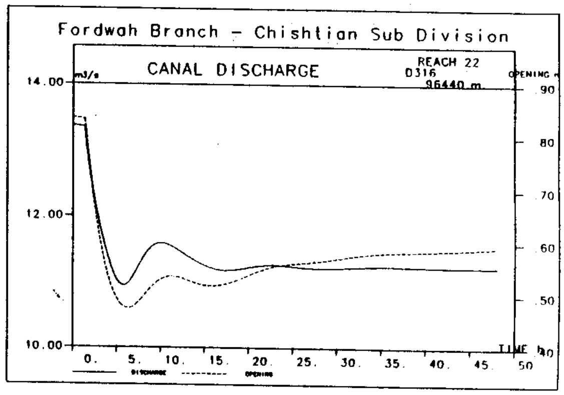
Discharge at the head



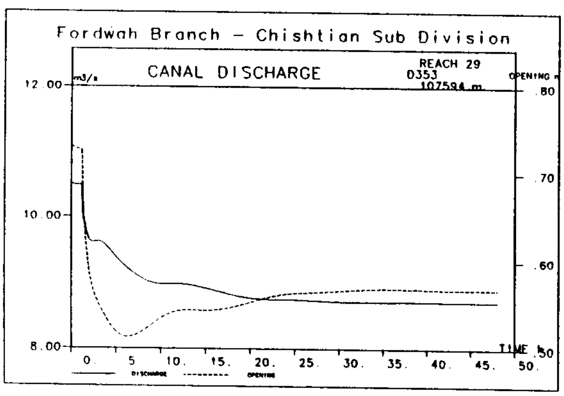
Command at RD199



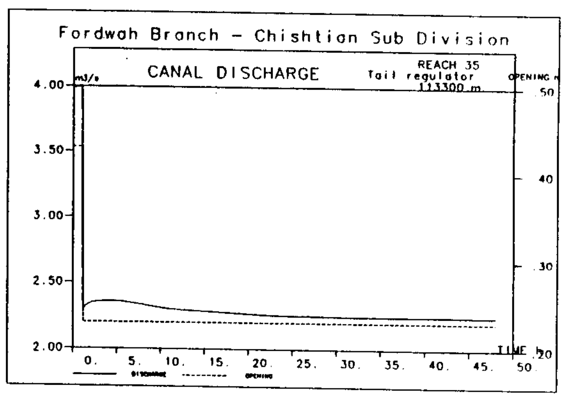
Command at RD245



Command at RD316



Command at RD353



Operation at Azim disty

Figure 58: Results of the automated gates

Table 6.3.5. Maximum deviation of controlled variables.

Location	Maximum deviation
Zup RD199	8 cm
Zup RD245	16 cm
Zup RD281	15 cm
Zup RD316	32 cm
Zup RD353	26 cm
Zup RD371	13 cm

The maximum deviations at offtakes is given through Ind1 computed by SIC (Ind1=supplied volume/demanded volume for the entire simulation period of 48 h). Demanded volume is computed assuming a constant demand discharge equal to the initial discharge. The offtakes located in the middle of a reach suffer because the discharge decreases from upstream, and some offtakes located just upstream of cross regulators benefit from the increase in water level.

Table 6.3.6. Maximum deviation for offtakes.

Offtakes	Maximum deviation (volume)
Mohar	+8%
4L	+9%
Masood	+24%
Phogan	-8%
Jagir	-8%
Soda	-17%
5L	-9%
Total for all offtakes	+2.4%

6.4. Field tests

Two field tests were performed on Fordwah Branch, Chishtian Subdivision. The aim was to work together with the Irrigation Department, to see if the scenarios proposed could bring an improvement in the management of the system. The objectives and results of the field tests are described below.

6.4.1. Objectives of the field tests

a. The first objective of these tests was to reach a **stable flow** in the Subdivision. This was done while the Subdivision was in first preference in the rotational program, so that fluctuations coming upstream of RD 199 could have been passed in Sikandar or in Rojahanwala disties, located just upstream of RD 199. The operations inside the Subdivision had to be kept to a minimum so that no wave would be created by operations inside the Subdivision.

b. Once this stable flow was reached, after 2 or 3 days, the objective was to **pass a wave** from RD 199 to the tail (Azim disty). The gauge readers were to be informed about the approximate time of arrival and the amplitude of the wave (number of cusecs). The propagation of the wave, its amplification or attenuation were to be observed.

Steady flow: to reach a steady flow in the canal, we needed to have:

- a stable inflow
- a minimum of operations at cross regulators and offtakes.

The steady inflow could be ensured when the Subdivision was in first preference according to the official warabandi schedule. This was the case from August 4 to 13, and from August 24 to September 2.

To keep the operations at a minimum, the gauge readers at each control point were told not to operate the gates if the upstream level at their point is within a range of 6 cm (0.2 ft) around the pond level. By following this rule, the unjustified operations are eliminated, and no wave is artificially provoked inside the Subdivision.

Inputs needed for the exercise: to be performed successfully, this field test needed some important inputs:

- orders were supposed to be given to the gauge reader at RD 199 to keep the discharge constant into the main branch during the duration of the field test
- orders had to be given to gauge readers not to operate cross regulators and offtakes during the steady flow period, except at RD 199 and 245.

6.4.2. Results

Due to weather conditions, the field test scheduled at the beginning of August had to be stopped before the end of the experiment. However, this first test was not a failure, as some good results have been achieved. A second field test took place from the 23rd to the 26th of August, with good weather conditions.

6.4.2.1. Field test 1

This field test took place from the 3rd to the 6th of August 1995. As specified in the objectives, a stable inflow during the experiment was a pre-requisite. However, as the indent was decreased from 1300 to 900 cusecs[18] (36.8 to 25.5 m^3/s), and then to 500 cusecs (14.2 m^3/s), the discharge at RD199 fell from 28.3 to 11.3 m^3/s (1000 to 400 cusecs) in two days. It was then difficult to ensure a constant flow in the Subdivision.

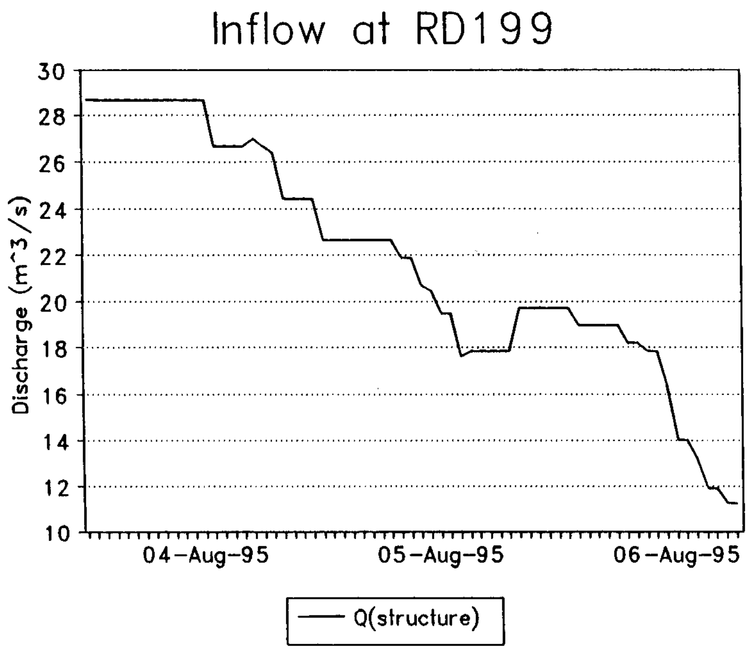


Figure 59: Inflow during the field test

18 The indent is given according to I&PD rating tables, that have not been updated.

In order to prevent this fluctuation from affecting the rest of the Subdivision, the gauge reader at RD245 was asked to operate the gates of his distributaries instead of moving the gates of the cross regulator, to keep a constant discharge downstream the main branch.
He succeeded quite well during the first 24 hours of the experiment, and passed down a very stable discharge from the 4th to the 5th of August. All the perturbations (due to the decrease at RD199) were passed into Daulat disty, which was closed twice during 2 hours. This disty could therefore absorb a decrease of 5.6 m^3/s (200 cusecs) at the beginning of the test. The 5th in the morning, the gauge reader received an order to open Daulat disty, and could no longer perform this control. This day, the experiment was also made difficult because of heavy rains.

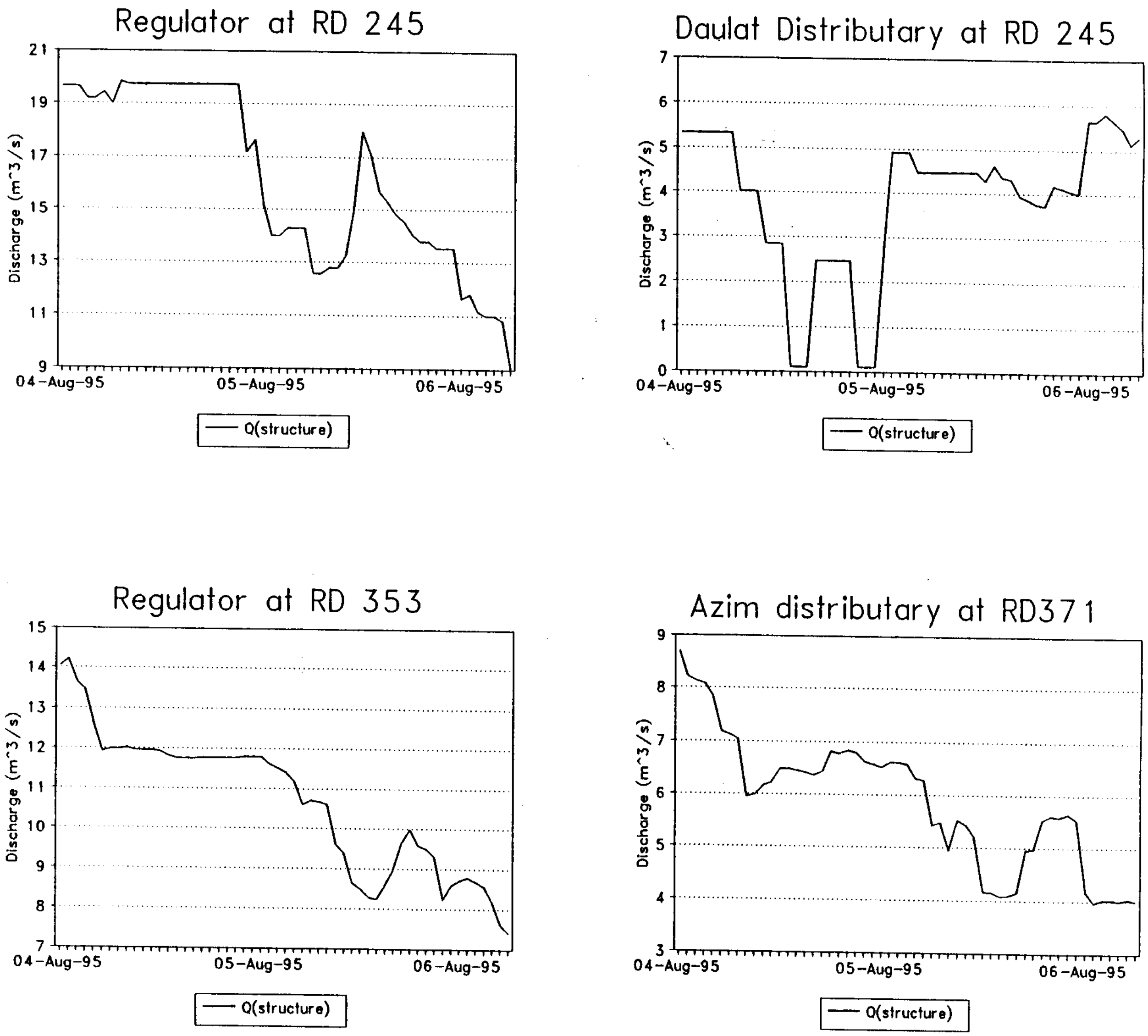


Figure 60: Results of field test 1

The stable flow created by this good control was felt in the downstream parts of the canal: the discharges at RD353 and even at Azim disty are quite stable for the first part of the monitoring period compared to the situation when the cross regulator is operated at RD245 (from 5th to 6th).

The number of operations performed at regulators has decreased during this period of stable flow, as can be seen in the figure below. Azim distributary is the only point where gates were operated during this period of stable flow. First he opened them, and then he had to close again, showing that the operations could have been avoided. At all cross regulators, the gates were not moved, and the discharge is then very constant. This has a beneficial impact on the distribution of water to offtaking distributaries: when the upstream level is constant and offtake gates are not operated, the discharge to offtaking disties is constant.

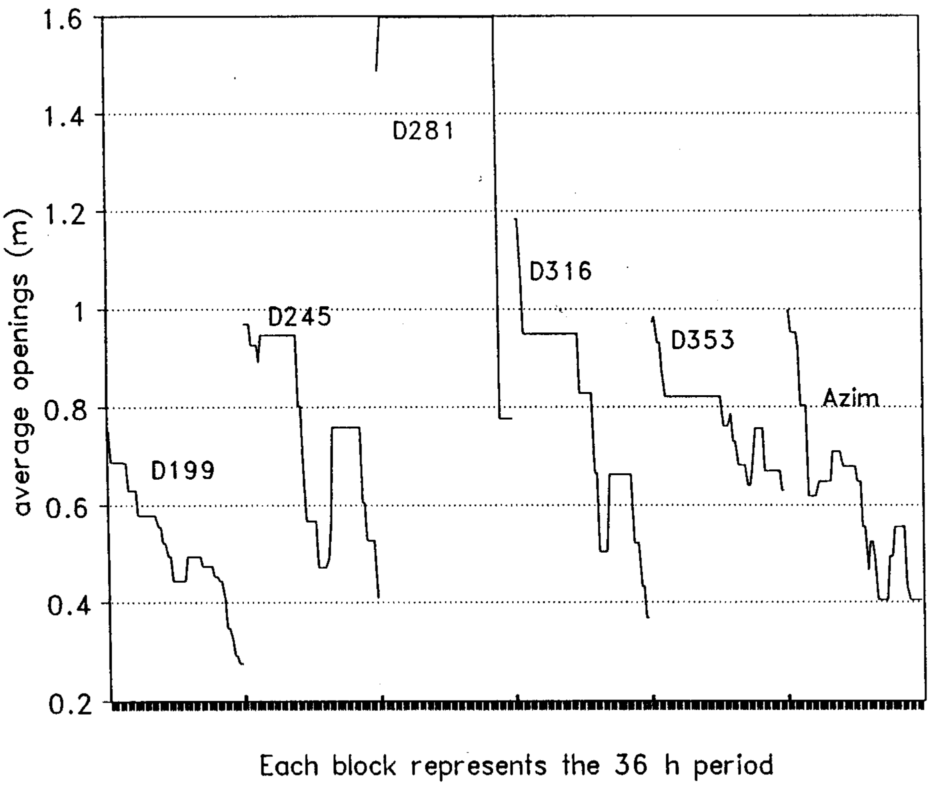


Figure 61: Operations at regulators (average openings)

6.4.2.2. Field test 2

The same objectives were kept for this second field test, that took place from the 23rd to the 26th of August 1995.

The SDO Chishtian kept his indent constant for the three days of the experiment (from the 23rd to the 26th) at 1350 cusecs (38.2 m^3/s).

As the indent was kept constant, the inflow at RD 199 was rather constant around 30 m^3/s (between 1350 and 1400 cusecs according to I&PD discharge table). The SDO Chishtian sent the Sub-engineer Tahkt Mahal to order the gauge reader at RD 245 to keep the discharge downstream his point at full supply, by closing some disties if necessary. He ordered the gauge reader not to operate the gates of the regulator, but to operate the gates of Daulat or Mohar disties instead. These two disties are offtaking just upstream of the cross regulator at RD 245, and can take a maximum discharge of about 7 m^3/s.
This way, the discharge in the main branch downstream of this point was to stay constant as long as the inflow at RD 199 did not fluctuate too much.

Positive wave propagation: starting from the steady state obtained in a first step, the objective was to create a wave of about 2 m^3/s (70 cusecs) at RD 199, to follow its propagation while informing the gauge readers, and to see the effect of this wave on the hydraulic state of the canal. Because of a drop in the discharge upstream of RD 199, it was not possible to create a significant wave, and only 0.5 m^3/s (17 cusecs) were released downstream. Little waves were also created at other control points, as the first one was not big enough to create a significant fluctuation.

Steady flow:
With an almost constant flow at RD 199, and a constant discharge downstream of RD 245, the fluctuations in the discharge passing downstream were reduced. There was almost no operations at cross regulators, allowing to stabilize the levels in the canal in less than 20 hours.
The canal stayed in steady flow for more than 36 hours. The discharge downstream the cross regulator at RD 245 was kept constant by operating Daulat disty, which could take the fluctuations coming from RD 199.

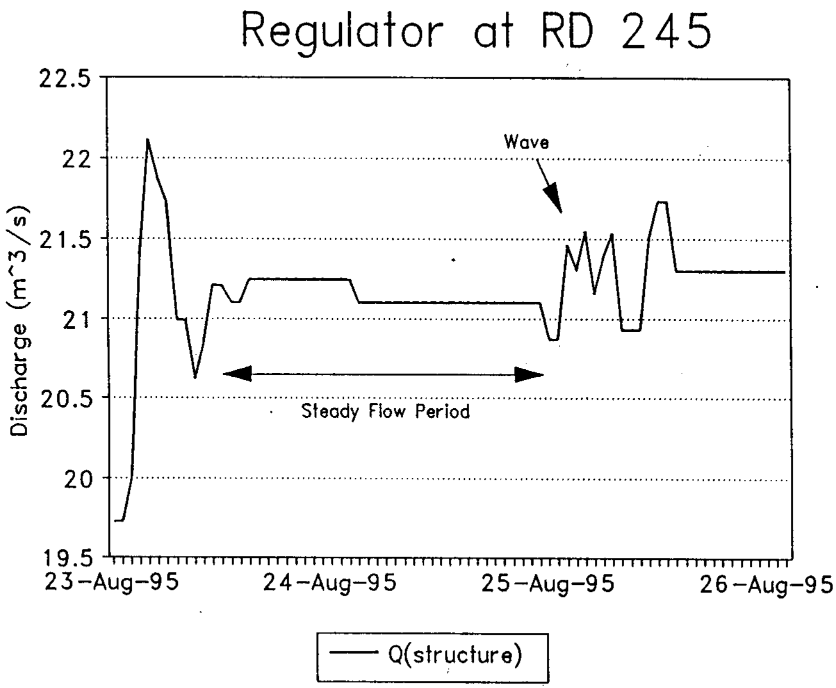


Figure 62: Discharge at RD 245

The discharge at RD 353 stays within a range of 0.5 m^3/s during about 40 hours, allowing the gauge reader at RD 371 not to operate during more than 36 hours.
The discharge in the distributaries at the tail of the system is fluctuating around an average value, but the gauge reader did not need to operate the gates. The fluctuations stay in a range of about 0.5 m^3/s.

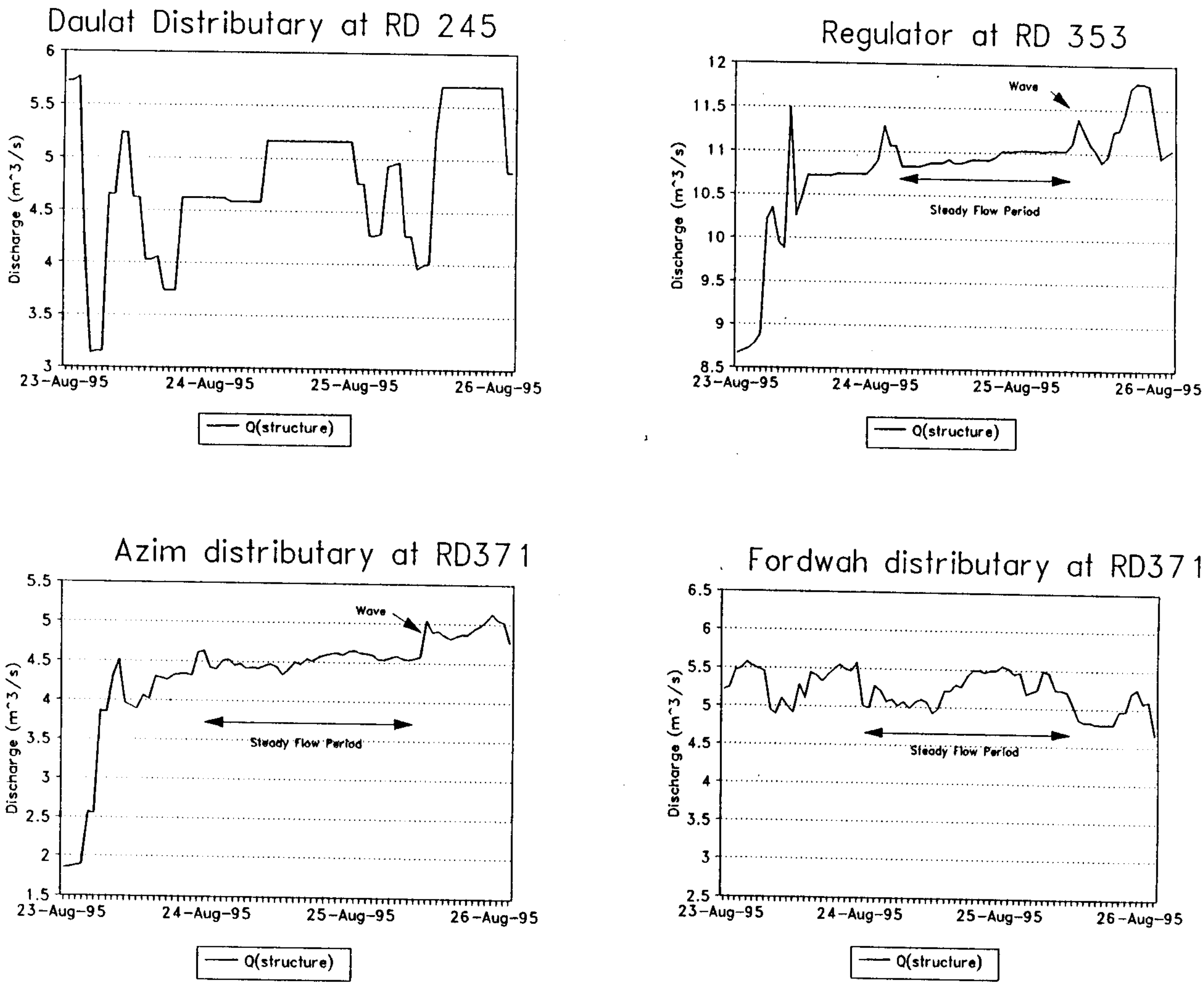


Figure 63: Results of field test 2

The operations at cross regulators are displayed in the figure below. The steady flow period can be clearly identified, as very few operations were performed during this time.
The wave propagation is less clear, as 4 little waves were created at different points along the canal. The time lags in the reaches predicted with SIC and Gateman were checked in the field, and showed a good fit with observations.

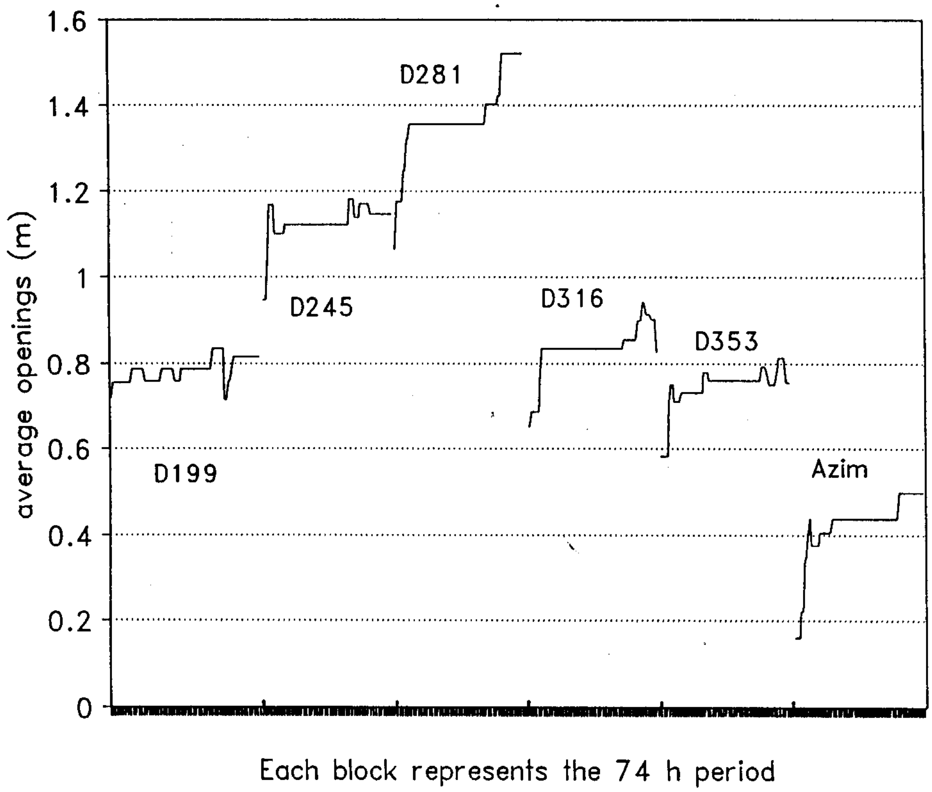


Figure 64: Operations at regulators

The table below compares the number of operations at regulators during the field test and another period of 60 hours monitored between the 3rd and 5th of June 1995[19]. During this period also, the inflow at the head was not too much fluctuating, at least for the first 30 hours. The decrease in number of operations is clearly visible at the tail, RD 371, where fluctuations usually provoke many operations.

[19] This is the hourly monitoring period used for the calibration of the models.

Table 6.3.7: Number of operations at cross regulators during two 60 hour periods.

Regulators	Monitoring period June 1995	Field test 2 August 1995
RD 199	7	9
RD 245	11	7
RD 281	11	5
RD 316	5	4
RD 353	11	7
Azim	20	7

The number of operations at cross regulator decreased dramatically during this steady flow period. The gauge reader at RD 371 did not move his gates during more then 36 hours, a situation which never occurred before in 7 years.

Effect of information on the operations:

The gauge reader at the tail of the system (RD 371) is usually the one operating the most often, as he receives all the fluctuations created by operations upstream. During this steady flow period, he did not operate the gates, but the upstream level at his point was nonetheless fluctuating a little bit.
This is represented in Figure 65, where the upstream level at RD 371 with reference to the targeted upstream level (FSD) is plotted over time, with the usual limits of intervention used by this gauge reader (+/- 2 cm).
It is clear that he has allowed bigger fluctuations in the upstream level without operating the gates (maximum = +/- 5 cm) during this steady flow period. This can be explained by the fact that during this time, he had some information about the future state of the canal: he knew that no big fluctuation was suppose to come, therefore he could allow his level to fluctuate more without operating the gates.
The information was the key point in this experiment.

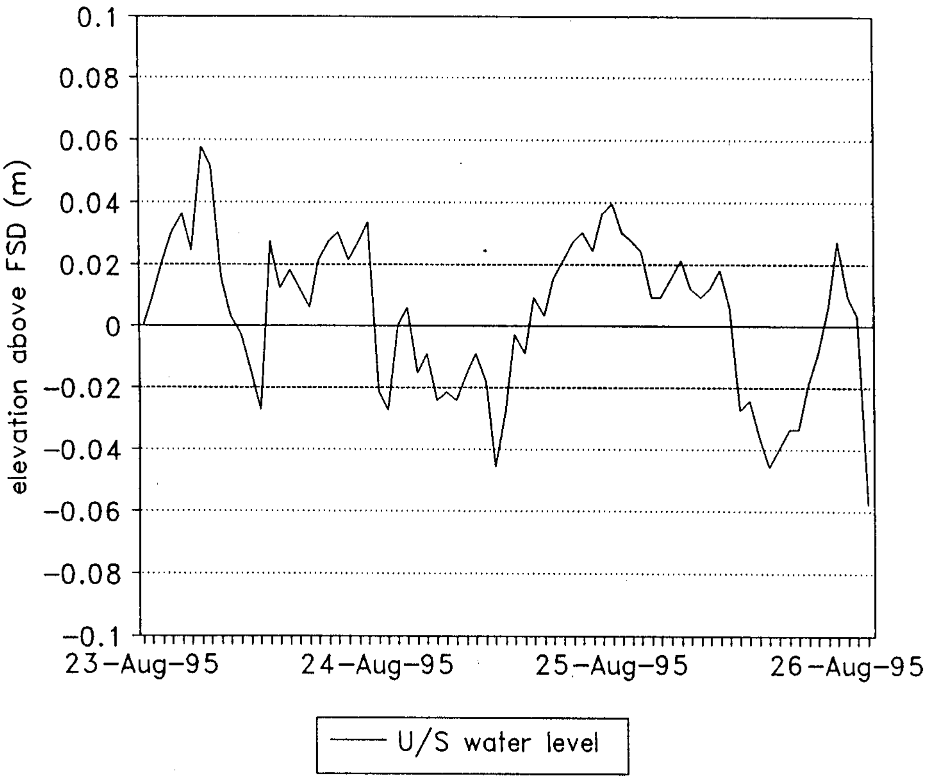


Figure 65: Water elevation above FSD at RD 371

Discussions with operational staff:

The results of this experiment were restituted to the gauge readers, who identified the different conditions that made this test successful, and ranked them in order of priority.
The conditions identified were as follows:

1. Specific orders were given by the managers (XEN, SDO, Sub-engineer) to gauge readers.
2. The inflow at RD 199 was rather stable.
3. Communication was provided inside the system (IIMI vehicle), and information was given to the gauge readers (number of cusecs and time of arrival of the wave).

- No rain occurred during this test.
- High water demand for the crops.
- Trained gauge readers.
- No intervention from farmers took place during the test.

The gauge readers emphasized the fact that they requested some guidance from their managers, especially in times where changes occur in the hydraulic state of the canal (rain, change of rotation,...).
They felt rather insecure when changes occur and they are not aware of them. The security margin they take in those cases sometimes lead to an increased risk of breaches.

The results were also jointly discussed with the manager (SDO Chishtian). He stressed the fact that this study was limited to the main canal level, and did not take into account problems encountered at the distributary level. At the main canal level, this field test showed some possible improvement that can be implemented.

Summary of results:

With a rather stable inflow and a few number of operations inside the system, the canal reached a steady state in less than 20 hours. Having information on the future state of the canal, the gauge readers were able to minimize the number of operations, tempering the little fluctuations they received instead of amplifying them. As a result, the water distribution to secondary canals was constant at full supply. As gauge readers had information on future perturbations, the safety margin could be kept at each point, with the canal running at full supply.

Starting from this very good steady flow, we tried to disrupt the system by creating some waves, and we studied the reactions of gauge readers to these waves. Each time, the gauge reader downstream of the point where the wave was coming from was informed about the time of arrival and the amplitude of the wave.They showed a very good hydraulic knowledge stemming from experience, and were able to temper these waves without any problem. It also came out of the experiment that it is quite difficult to pass water to the tail in times when the demand of water is high. Any increase in the supply coming from upstream is taken by the disties in the middle, and only a small portion reaches the tail.

7. SCENARIOS FOR IMPROVED OPERATIONS: DISCUSSION OF RESULTS

From a hydraulic approach, using a hydraulic simulation model and a regulation module, we managed to identify the physical and operational constraints of the system.
The analysis of the system showed that strategic targets were not clear, and that the official strategy was not implemented. Moreover, the dominant strategy is a combination of rare operational orders by the managers, local control by gauge readers, and interferences by farmers.

The possible improvement in the operations was shown to be at two levels: a local level, with improved operations at hydraulic structures, and a global level, with the introduction of information in the system.
The simulations were done at a tactical level, assuming that the official strategy was effectively followed. Improvement in actual strategy is a necessity, but it involves many actors, and would need a greater implication of all of them.

7.1. Local improvement: improved operations

The four simulated scenarios showed that it was possible to improve the water distribution in the system by changing rules of operations at a local level. They were tested on five situations describing operational problems that are presently addressed by the gauge readers, as the official strategy is not implemented.

7.1.1. Comparison with real operations

The scenario 1 "improved operations" proposes to eliminate non hydraulically justified operations at cross regulators. It only requires a change in the habits of gauge readers, not in their rules of operation. The relevant hydraulic criterion is a significant change in the upstream level at their point (more than 2 cm around FSD). With these limits of operations, we have shown that an improvement is possible in terms of stabilization of the levels in the canal, and of the waves created by operations. Non justified operations are avoided, reducing the number of operations, and smoothening the variations in discharge passed downstream.

The interesting part in this comparison is that it has been done with the regulation module Gateman, enabling an a priori evaluation of operations. The results showed that the trend in the operations is the same, as the calibration of Gateman showed good results.

Some operations observed in the field were not performed by Gateman, showing that they were not useful to stabilize the level. In fact, some of those unjustified operations upstream provoke a reaction downstream, as the next gauge reader sees the level fluctuate; this multiplies the

number of operations at the tail, and creates a lot of artificial fluctuations.
We used the aggregated indicator defined in the methodology to compare the two scenarios. It gives the total useful volume delivered to the distributaries in the Subdivision.

Table 7.1. Aggregated indicator for the situation a

Scenarios	Scenario 0	Scenario 1
Indicator (m^3)	114	147.85

The total useful volume is not very important in this situation, because the discharge at the head was not very high (25.45 m^3/s). The improvement brought by this local change in operations is clear, as it increases by almost 30% the useful volume in this situation.

This local improvement is nonetheless not enough to tackle some delicate situations, as a variation in the discharge at the head of the system (situation b and c), or a shift in rotation (situation d).

7.1.2. Field test 1

The first field test, although made difficult because of changes in the discharge at the head of the Subdivision, was a sort of verification of the results obtained with Gateman in situation c. This simulation showed that when a fluctuation was coming from upstream, the discharge could be kept constant in the main branch by operating an offtake instead of a cross regulator.

It was also confirmed in the field. During this field test, the gauge reader at RD 245 operated Daulat instead of his cross regulator, and the discharge was stabilized downstream of this point during more than 20 hours, whereas he was receiving a negative wave of more than 5 m^3/s during this time.

This result may seem obvious, but it did not seem to be so clear for gauge readers at first. Their first reaction was to say that as the bed level of some disties are higher than the one of the branch, closing a disty would provoke a "big pressure wave" downstream the main branch, which is absolutely not the case. They used this reason as an excuse not to operate their disties. In fact, the operation of a cross regulator has a much bigger impact on the discharge in the main branch than the operation of an offtake: the opening of a cross regulator provokes a peak in the discharge, which stays for some hours before the discharge goes back to the steady state value. If an upstream offtake is operated, then only the consequent change in upstream level will affect the discharge downstream. The wave created will be continuous, and not abrupt, like in the case of the operation of the cross regulator.

The conclusion from this experiment is that it is possible to stabilize a flow in this canal by operating offtakes instead of cross regulators (if the fluctuations remain in a reasonable range). Therefore, such a practice, implemented at control points in Bahawalnagar Subdivision (the Subdivision located directly upstream of Chishtian Subdivision) could ensure a constant inflow to the Chishtian Subdivision when it is in first preference.
The same practice should also be implemented inside the Subdivision, when there is a rotation between disties. As it has been simulated in the situation c, this practice will improve especially the situation of distributaries located at the tail of. the system.

7.2. Global improvement: Information in the system

7.2.1. Simulated scenarios

To get a better performance in situations where the inflow is changing (situation b and c), or when a wave is passed through the system (situation d), a better communication inside the system is necessary. The implementation of a communication system between gate operators was simulated in the scenarios 2 and 3, "Buffer capacity", and "Linked operations at the tail". The capacity of the reaches of the canal (reach 4 only for scenario 3) is used in both scenarios to temper fluctuations coming from upstream.
Both scenarios showed a significant improvement for an extreme situation when a wave of 5 m^3/s is passed through the system. The risk of overtopping is decreased as the amplitude of the wave is not amplified.
These scenarios are performed with the transmission of global information inside the Subdivision. Gauge readers keep their role of dealing with local variables, but they do it with a knowledge about future perturbations, and they are also supposed to convey information downstream. This would perhaps create a sense of responsibility of upstream gauge readers for waves that are passed on downstream. The number of breaches could also be reduced.
The role of the manager in this would be to give orders for the implementation of the strategy.

The "feed forward control" (scenario 4) gave the best results for the shift of rotation. This is a centralized tactical control, as the manager gives orders to gauge readers for time and amplitude of operations. This would need the implementation of an improved communication system between the manager and gauge readers, and also a closer communication between the manager and higher echelons. The manager should also have an estimation of future perturbations, as well as an intimate knowledge of his system.

Table 7.2. Aggregated indicator for situation d

Scenarios	Scenario 1 Improved op.	Scenario 2 Buffer	Scenario 3 Linked op.	Scenario 4 Feed forward
Indicator (m^3)	344.65	333.7	336.3	367.95

In this situation, as the initial state of the canal was good in terms of discharge delivered to the distributaries, the difference between scenarios is less visible. We see that with the scenarios 2 and 3, less useful volume is delivered to the distributaries than with the scenario 1. It is due to the fact that levels are allowed to fluctuate more in those scenarios, and for a rather long period. We have seen that the result was beneficial in terms of reducing the risk of breaches, as the peak in the discharge at the tail was reduced. These scenarios are therefore interesting for emergency cases. For routine management, the field test 2 showed that a significant improvement can be taken from the introduction of information in the system.
Performance can be further improved by introducing a feed forward control (scenario 4). The increase in useful volume represents about 7% of the useful volume delivered with scenario 1. The improvement is also in terms of reducing the risk of breaches, as the wave was not amplified.

The feed back control performed by gauge readers is an essential feature of this system, that should not be lost. A feed forward + feed back control could be applied by giving average time and amplitude of operations to gauge readers, and also the expected amplitude of fluctuations in upstream levels at regulators. The operators would have to apply the orders, but to operate if the upstream level goes beyond the given limits. This information should be given each day to each gauge reader. Their task then could be more focused on observing the state of the canal, which is more in accordance with their denomination.

7.2.2. Field test 2

This field test was very productive because it was done in collaboration with the I&PD; the results were discussed with gauge readers and the manager, and the conditions that made this test successful were identified and ranked by order of importance.

The three most important conditions were:

- implication of managers
- information in the system
- stable flow at the head

The implications of the managers was a very important point, as well as the information inside the system. As the supply to distributaries was constant at full supply and the demand of crops was high, the farmers were satisfied, and very few interferences occurred during these three days.

The system was found to be relatively robust. It was not perturbed by little fluctuations provided gauge readers were informed about future perturbations (timing, number of cusecs, origin).

The test showed that a global improvement is possible if gauge readers are informed about future perturbations. They have a very good knowledge of the system, not only at their control point.

Some of them had an idea about the time lags in the system, and they could predict the amplitude of their operation if they were told the number of cusecs coming.
These gauge readers are an essential component of this system. They represent a considerable amount of experience that should not be lost. Therefore, the introduction of an information system between the gauge readers would be a way to use this knowledge and to provide them better working conditions. They would be able to perform a better control on the canal, as it was shown during this test.

7.3. Automated gates

The scenario 5 was simulated with the hypothesis of the removal of one constraint of this system (discharge at the head). The local automatic controllers reacted quite well, but slowly. Due to coupling and non-linear effects, it was not possible to increase their speed without destabilizing them.

It is possible to improve such controllers:

- coupling and non-linear effects can be reduced by using Q as the control action variable instead of w.
- non-linear effects can be reduced by using multiple models or autoadaptative techniques.
- the selection of the PI coefficients can be improved by using a parametric design (based on a transfer function of the process).
- the use of a Smith predictor, an internal model based technique or a predictive technique could speed up the controller.
- the use of a MIMO (Multiple Inputs, Multiple Outputs) technique (eg. multivariable PI, optimal control) would guaranty a better stability.
- implementation constraints like minimum gate movements, limits of intervention, high frequency filter, can be taken into account for a more realistic simulation.

7.4. Indicator: Output to the secondary level

The indicators we used for this study were meant to be a link with the secondary level. We used two indicators:

- the percentage of time during which the discharge supplied to a distributary is within the range of [85%, 110%] of the design discharge,
- the volume delivered during this time (called 'useful volume').

The limits defined for the acceptable range of discharge were derived from an on-going study at the distributary level (Hart, 1995). It is part of a broader objective, that is to assess the effect of operations on crop production.
We worked with the underlying hypothesis that an improvement at the main canal level would have a positive impact on crop production.

Our study was confined to tactical operations at the main canal level. It was therefore important to have clear targets on which to evaluate the performance of actual situation and of proposed changes.

The indicator used was very dependent on the strategic choice of the manager. As an example, the design discharges for distributaries, that were taken as a reference, were not updated for some disties where big changes have occurred; number, sizes of outlets, size of the offtake structure, etc...
This remark highlights the fact that an improvement at a local level depends a lot on the strategic option chosen. A first improvement in the management of this Subdivision would be to clarify the strategy implemented, so that an evaluation based on clear targets would be possible. Starting from this situation, the implementation of new local rules could be successful.

IIMI is presently working on the development of a Rapid Assessment Procedure (RAP) for the secondary canals. It aim is to evaluate the water distribution to tertiary outlets inside a distributary by means of a short time monitoring period. The results of this work could be useful to establish a new set of targets for the managers, as these would represent the actual state of the canal.

8. CONCLUSIONS AND RECOMMENDATIONS

8.1. Conclusions

1. The use of a hydraulic simulation tool to improve management of an irrigated system proved to be very interesting and useful. The hydraulic package SIC is a powerful tool, rather simple of use for basic simulations. After its calibration, the physical limits of the system can be determined (capacity of reaches, maximum discharge, minimum free board, maximum water levels). It can also show the marginal effect of different operations through simulations.

2. The regulation module Gateman provided an a priori evaluation of operations, and was used in SIC (Unit 3) to simulate manual operators. The rule introduced in this module to compute the openings of a cross regulator was found to match rather accurately manual operations performed by the gauge readers in Fordwah Branch. It seems to validate the method tested by Malaterre in Sri-Lanka (1989).

3. The scenario improved operations simulated a local improvement in the operations of hydraulic structures, cross regulators and offtakes. The improvement was clearly shown for a field test where observed operations were compared to operations simulated with Gateman. As non hydraulically justified operations are avoided, the discharge in the main branch is smoothened, and this results in a more stable discharge delivered to distributaries.

4. The introduction of a communication system represented a global improvement; it was simulated in three scenarios, and also tested in the field. It showed that the system was rather robust, because little perturbations did not disrupt the system, when the gauge readers were informed about future perturbations.

5. The very good knowledge of the system displayed by the gauge readers is an essential feature of this system, that should be fully used by giving them information on the state of the canal.

6. Baume et al. (1993) intended the SIC model to become a "decision-support tool in a manually operated canal system" (p.2). As a conclusion of this study, we can say that this step seems to be quite far away from present possibilities, at least in the Fordwah Branch system. The usefulness of such a package combined with a regulation module was clearly demonstrated in the diagnosis and the elaboration of alternative operational rules. The field tests, first step towards implementation, was also a undeniable achievement. Nonetheless, the introduction of a hydraulic model in the routine management of this system seems to be rather unrealistic in present situation. It could however profitably be used for the training of managers (SDOs).

8.2. Proposed improvements in SIC and Gateman

8.2.1. SIC package

* Difficulties encountered in the use of SIC package:

- No super critical flow allowed: this limitation was felt in two cases; first, some sections downstream of RD 353 had to be deleted because they provoked a super critical flow during the unsteady state simulations, and secondly, the gate opening of Azim for one operation had to be raised from 4 to 6 cm because a too small gate opening provoked super critical flow in sections downstream.

- Discharge coefficients: the determination of Cds for submerged cross structures appeared to be difficult, and the simulated water levels in unsteady state deviate from the observed values when gates are operated.

- The outputs of SIC (graphs) were found difficult to use for the comparison of measured with simulated values.

* Proposed improvements:

- Introduce the possibility to enter a set of levels along time at a section, to enable the comparison of measured and simulated values. This would facilitate the calibration process, as we could determine directly by visualizing the results of a simulation in unsteady flow if a change in a coefficient is needed, without having to export the data as an ASCII file in a spreadsheet.

- Write the name of the file on the operational print out in steady flow. This operational print out was used very often before carrying the simulation in unsteady state, and printing the name of the .FLU file in a header would clarify this print out.

- Give the wetted area in a reach as an output of the steady flow unit. This data is widely used to calculate the seepage in terms of cusecs per million square feet, or l/s/ha.

8.2.2. Gateman

- The use of this regulation module was found very helpful for the evaluation of present practices, and the simulation of actual and alternative rules. The good results showed by the rule chosen to compute the openings of cross regulators could be checked on other manually operated irrigation systems. The calibration of the method for the computation of offtake openings was not possible in our study. This calibration could be done in order to test the reliability of the method, and its accuracy.

- The use of such a module can be recommended for researchers who would like to test some changes in strategic rules or some new situations. The reactions of the gauge readers can be simulated in a realistic way, and repetitions of simulations are allowed, as it is integrated to a computer model.

- This regulation module Gateman could be interfaced to become more user friendly. It could then be used by managers if they already have a good mastery of SIC for routine management.

8.3. Recommendations

The recommendations are delivered at two levels: for the manager, in an operational view, and for further research on this subject.

8.3.1. Management

1. Clarify objectives: the present strategy requires some clarification, as the official rotational schedule is not implemented. A clear strategy would allow to set the frame for a work on the operational side, at a tactical level.

2. Information: IMIS (or any other means to improve communication, data collection, analysis and evaluation) could provide very valuable data to the manager. The rehabilitation of the old communication system could be a first step towards a better information in the Subdivision.

3. Create stability in the system: it can be done by using the local rule already tested (limit the operations to the hydraulically necessary ones), and providing guidance to gauge readers in case of hydraulic changes in the canal.

4. Introduce a communication system between gauge readers: the second field test has clearly shown that a considerable improvement was possible in terms of reduced risks of breaches, and stability of flow in the canal. The communication between gauge readers is something that used to exist in this system, as the managers used to visit their operational staff when there was any risk of emergency, to give them advises. Baildars were also more numerous to look after the banks conditions.

5. Field tests like the ones performed during this study should become a normal matter: the collaboration with the Irrigation Department proved to be very productive and interesting. These field tests could be extended to a longer period: it could be done again for a 10 day period, when the Chishtian Subdivision is in first preference. Information should be conveyed to the gauge readers from the manager during this period. They requested for some guidance, especially when changes occur (rain, change in rotation, increase in indent). They need specific information concerning the time, the amplitude (number of cusecs), the reason for this fluctuation and its provenance.

8.3.2. Research

1. Assess the effect of fluctuation at the head of a distributary on water distribution to tertiary outlets: this would give a dynamic objective for the water distribution at the main canal level.

2. Work at the strategic level: some research at the strategic level is required for a realistic implementation of alternative tactical rules. We worked in a moving frame, as the actual strategy was not clearly defined. A study on actual strategy would set a clear frame for a work at tactical level.

3. Test the methodology: the methodology described in this study is not a generic one, as it has only be applied to Fordwah Branch. Such an approach could be profitably tested and improved in other irrigated systems.

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ANNEXES

A: Data on Fordwah Branch

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A.3. Warabandi Program for Kharif season 1995, Fordwah Division, Bahawalnagar
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B: Results of calibration

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D: Miscellaneous

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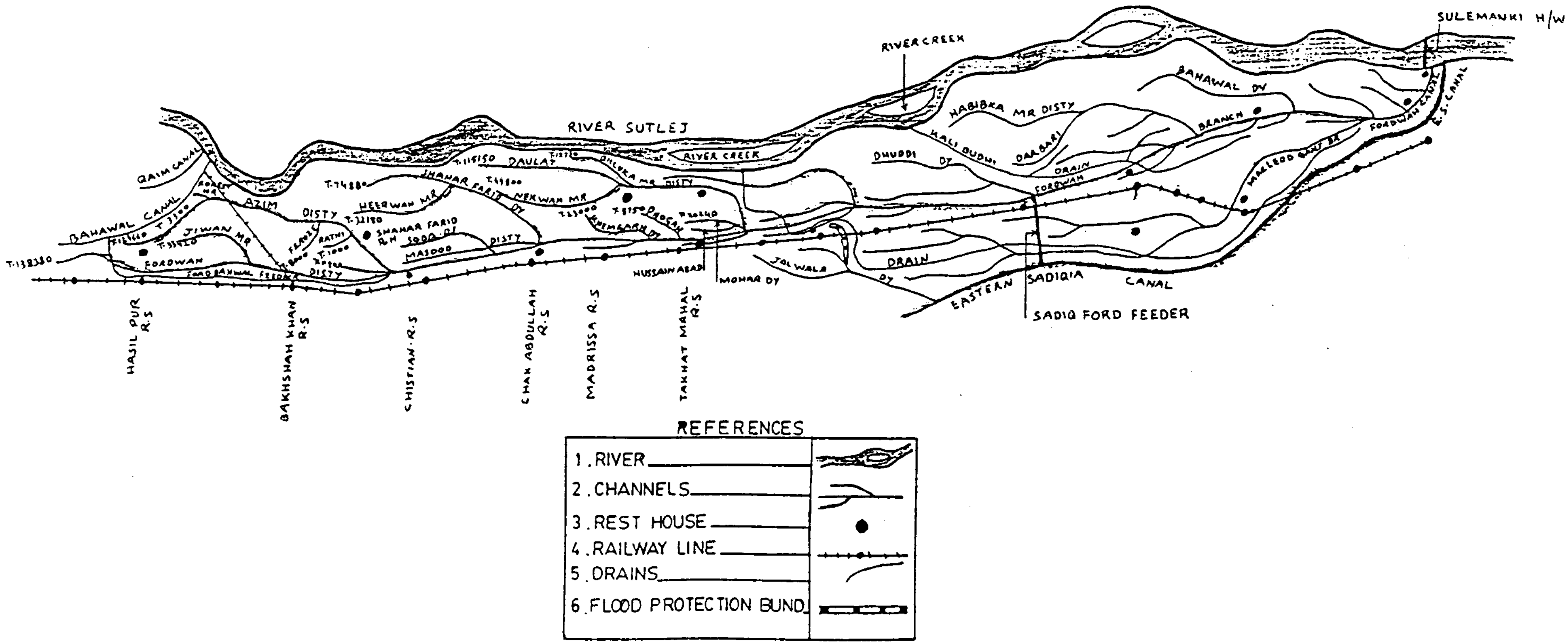
D.3. Computation of the opening of a circular pipe with a circular gate

ANNEX A.1. Plan of Fordwah Division



ANNEX A.2. Issue tree of Fordwah Branch, Chishtian Subdvision

Cross Structures	Gates	Weirs	Offtakes
D 199	___ ___ ___ ___ ___ ___		
			<> <> Daulat
			<> Mohar
			] 3L
D 245	___ ___ ___ ___		
			] Phogan
			<> Khem Gahr
			] 4L
D 281	___ ___		
			<> Jagir
			<> <> S.Farid
			<> Masood
D 316	___ ___ ___		] Soda
Weir 334		]	
			] 5L
D 353	___		
Weir 363		]	
			<> <> Fordwah
			<> Mehmud
			<> <> Azim

The distributaries are supplied with water through offtakes situated along the main branch; these offtakes are either open-flumes (]) or orifices equipped with flat sliding gates (< >).

ANNEX A.3. WARABANDI PROGRAM FOR KHARIF 1995, FORDWAH DIVISION BAHAWALNAGAR

Discharges are given in cusecs.

Group A: Bahawalnagar Subdivision

A1:
Bahawal disty=117, Ahmad pur disty=7, Khalsana disty=22, Darbary disty=270, Total Discharge=284 Cusecs.
A2:
Mirza disty=41, Behkan disty=31, Dhuddi disty=170, 1-L disty =19, 2-L disty=23, Total discharge=284 Cusecs.
A3:
Bahawalnagar disty=31, Rojhanwala=70, Sikandar disty=171, Total Discharge=272 Cusecs.

Group B: Chishtian Subdivision

B1:
Daulat disty=209, Mohar disty=38, 3_L disty=23, Phogan disty=18, Khemgarh=30, 4-L disty=16, Total discharge=334 Cusecs.
B2:
Jagir disty=28, Masood disty=35, Shahar Farid disty=153, Soda disty=77, Total discharge=286 Cusecs.
B3:
5-L disty=4, Fordwah disty=158, Azim disty=244, Mahmood disty=9, Total discharge=418 Cusecs.

Note:
Sadhu disty, Dona disty, Noshera disty, Bonga disty, 2-L disty: These disties will not included in the warabandi shedule because their crest levels are higher.
1. At the tail of Fordwah canal, Head haddi wala water will be distributed between Fordwah branch and MacLeod Ganj branch in ratio of 87:13.
2. When MacLeod Ganj branch is at full supply from head then at tail of MacLeod Ganj branch water will be distributed to Bearwala and Meard at the ratio of 62:38. But when water will be less then 50% then one disty will be at full supply for ten days and then other for next ten days. S.D.O. Minchinabad will responsible for this.
3. Executive Engineer will be in charge of warabandi.
In case excess or shortage of water warabandi program will be changed with the permission of S.E. Bahawalnagar.
4. In group "B" when water will less, then disties will be closed from right to left.
For example, when group "B" will be second on dated on 16-4-95 to 25-4-95 and then in case of shortage of water in group "B3" first 5-L then Mahmood then Azim then at the last Fordwah disty will closed.

5. In the dates when Darbari disty will closed, at that time the Karries will not be used in Fordwah Branch at R.D. 77500.
6. Water should not be heading up unnecessarily.

From	To	1ST	2nd	. 2nd 1	2nd 2	2nd 3
16-4-95	25-4-95	A	B	B1	B2	B3
26-4-95	5-5-95	B	A	A1	A2	A3
6-5-95	15-5-95	A	B	B2	B3	B1
16-5-95	25-5-95	B	A	A2	A3	A1
26-5-95	4-6-95	A	B	B3	B1	B2
5-6-95	14-6-95	B	A	A3	A1	A2
15-6-95	24-6-95	A	B	B1	B2	B3
25-6-95	4-7-95	B	A	A1	A2	A3
5-7-95	14-7-95	A	B	B2	B3	B1
15-7-95	24-7-95	B	A	A2	A3	A1
25-7-95	3-8-95	A	B	B3	B1	B2
4-8-95	13-8-95	B	A	A3	A1	A2
14-8-95	23-8-95	A	B	B1	B2	B3
24-8-95	2-9-95	B	A	A1	A2	A3
3-9-95	12-9-95	A	B	B2	B3	B1
13-9-95	22-9-95	B	A	A2	A3	A1
23-9-95	2-10-95	A	B	B3	B1	B2
3-10-95	12-10-95	B	A	A3	A1	A2
13-10-95	22-10-95	A	B	B1	B2	B3

Fordwah Branch, Chishtian Subdivision

ANNEX A.

Dimensions of structures in meters

Structures	Description	Dimensions (m)			WM ab. crest (m)		gate WM	Spindle	Cd
		width	height	crest	U/S	D/S	ab. crest	Zero	
D199	Cross regulator	2.29	1.75	156.02	-0.05 *	-2.83 *	2.78	1.03	0.57
	6 gates	2.29	1.75	156.03	-0.05 *	-2.83 *	2.75	1	
		2.3	1.75	156.04	-0.05 *	-2.83 *	2.85	1.1	
		2.29	1.75	156.04	-0.05 *	-2.83 *	2.74	0.99	
		2.28	1.75	156.03	-0.05 *	-2.83 *	2.79	1.04	
		2.27	1.75	156.04	-0.05 *	-2.83 *	2.77	1.02	
Daulat	2 gates	2.27	1.71	152.93	1.58	0.87 *	2.87	1.16	0.55
		2.26	1.27	152.93	1.57	0.86 *	2.58	0.87	
Mohar	1 gate	1.21	1.37	152.74	1.76	1.61	2.13	0.76	0.65
3L	Open flume	1.21	-	153.23	1.27	1.26	-	-	0.13
D245	Cross regulator	2.25	2.57	151.71	2.78	2.25	3.06	0.49	0.72
	4 gates	2.28	2.57	151.71	2.77	2.23	3.04	0.47	
		2.27	2.57	151.71	2.79	2.26	3.08	0.51	
		2.28	2.57	151.71	2.78	2.25	3.06	0.49	
Phogan	Open flume	0.69	-	151.84	0.89	0.58	-	-	0.28
Khem Gahr	1 gate	1.18	1.41	151.01	1.03	0.79	2.42	1	2.16
4L	Open flume	0.89	-	150.66	1.38	1.37	-	-	0.4
D281	Cross regulator	4.56	1.98	150.25	1.82	1.62	3.5	1.52	0.94
	2 gates	4.6	1.98	150.25	1.82	1.62	3.37	1.39	
Jagir	1 gate	1.5	1.71	149.1	2.1	1.91	2.72	1.01	0.75
Shahar Farid	1 gate	2.6	1.82	148.26	2.25		2.25	0.43	0.57
Masood	1 gate	1.81	1.47	148.4	2.14	2.14	2.74	1.27	0.86
D316	Cross regulator	2.27	2.06	148.02	2.53	-0.39 *	3.21	1.15	0.47
	3 gates	2.27	2.06	148.02	2.55	-0.37 *	3.15	1.09	
		2.27	2.06	148.03	2.54	-0.38 *	3.24	1.18	
Soda	Open flume	1.43	-	147.47	1.36	0.78	-	-	0.24
Weir 334	1 Weir	6.12	-	147.47	-	1.42	-	-	0.48
5L	Open flume	0.75	-	146.91	1.17	1.2	-	-	-
D353	Cross regulator								0.58
	1 gate	6.1	1.86	146.23	1.39	0.64	2.8	0.94	
Weir 363	1 Weir	6.11	-	144.65			-	-	-
Fordwah	2 gates	1.8	1.37	144.01	2.01	1.4	2.79	1.42	0.87
		1.82	1.37	144.01	2.01	1.4	2.78	1.41	
Azim	2 gates	1.81	1.52	144.01	2	0.97	2.93	1.41	0.53
		1.82	1.52	144.01	2	0.97	2.93	1.41	
Mehmud	1 gate/culvert	1.25	0.73	144.47	1.55	1.45	1.7	-0.36	0.5

* : gauge

5. In the dates when Darbari disty will closed, at that time the Karries will not be used in Fordwah Branch at R.D. 77500.
6. Water should not be heading up unnecessarily.

From	To	1ST	2nd	2nd 1	2nd 2	2nd 3
16-4-95	25-4-95	A	B	B1	B2	B3
26-4-95	5-5-95	B	A	A1	A2	A3
6-5-95	15-5-95	A	B	B2	B3	B1
16-5-95	25-5-95	B	A	A2	A3	A1
26-5-95	4-6-95	A	B	B3	B1	B2
5-6-95	14-6-95	B	A	A3	A1	A2
15-6-95	24-6-95	A	B	B1	B2	B3
25-6-95	4-7-95	B	A	A1	A2	A3
5-7-95	14-7-95	A	B	B2	B3	B1
15-7-95	24-7-95	B	A	A2	A3	A1
25-7-95	3-8-95	A	B	B3	B1	B2
4-8-95	13-8-95	B	A	A3	A1	A2
14-8-95	23-8-95	A	B	B1	B2	B3
24-8-95	2-9-95	B	A	A1	A2	A3
3-9-95	12-9-95	A	B	B2	B3	B1
13-9-95	22-9-95	B	A	A2	A3	A1
23-9-95	2-10-95	A	B	B3	B1	B2
3-10-95	12-10-95	B	A	A3	A1	A2
13-10-95	22-10-95	A	B	B1	B2	B3

Fordwah Branch, Chishtian Subdivision

Dimensions of structures in feet

Structures	Description	Dimensions (ft)			WM ab. crest (ft)		gate WM	Spindle	Cd
		width	height	crest	U/S	D/S	ab. crest	Zero	
D199	Cross regulator	7.51	5.74	511.88	-0.16 *	-9.28 *	9.12	3.38	0.57
	6 gates	7.51	5.74	511.91	-0.16 *	-9.28 *	9.02	3.28	
		7.54	5.74	511.94	-0.16 *	-9.28 *	9.35	3.61	
		7.51	5.74	511.94	-0.16 *	-9.28 *	8.99	3.25	
		7.48	5.74	511.91	-0.16 *	-9.28 *	9.15	3.41	
		7.45	5.74	511.94	-0.16 *	-9.28 *	9.09	3.35	
Daulat	2 gates	7.45	5.61	501.74	5.18	2.85 *	9.41	3.80	0.55
		7.41	5.61	501.74	5.15	2.82 *	8.46	2.85	
Mohar	1 gate	3.97	4.49	501.12	5.77	5.28	6.99	2.49	0.65
3L	Open flume	3.97	-	502.72	4.17	4.13	-	-	0.13
D245	Cross regulator	7.38	8.43	497.74	9.12	7.38	10.04	1.61	0.72
	4 gates	7.48	8.43	497.74	9.09	7.31	9.97	1.54	
		7.45	8.43	497.74	9.15	7.41	10.10	1.67	
		7.48	8.43	497.74	9.12	7.38	10.04	1.61	
Phogan	Open flume	2.26	-	498.16	2.92	1.89	-	-	0.28
Khem Gahr	1 gate	3.87	4.62	495.44	3.37	2.61	7.93	3.31	2.16
4L	Open flume	2.92	-	494.29	4.52	4.51	-	-	0.4
D281	Cross regulator	14.96	6.49	492.95	5.98	5.31	11.48	4.99	0.94
	2 gates	15.09	6.49	492.95	5.98	5.31	11.05	4.56	
Jagir	1 gate	4.92	5.61	489.17	6.88	6.28	8.92	3.31	0.75
Shahar Farid	1 gate	8.53	5.97	486.42	7.37		7.38	1.41	0.57
Masood	1 gate	5.94	4.82	486.88	7.03	7.03	8.99	4.17	0.86
D316	Cross regulator	7.45	6.76	485.63	8.30	-1.28 *	10.53	3.77	0.47
	3 gates	7.45	6.76	485.63	8.36	-1.21 *	10.33	3.58	
		7.45	6.76	485.66	8.33	-1.25 *	10.63	3.87	
Soda	Open flume	4.69	-	483.83	4.47	2.58	-	-	0.24
Weir 334	1 Weir	20.07	-	483.83		4.66	-	-	0.48
5L	Open flume	2.46	-	481.99	3.84	3.94	-	-	-
D353	Cross regulator 1 gate	20.01	6.10	479.76	4.56	2.10	9.18	3.08	0.58
Weir 363	1 Weir	20.04	-	474.57			-	-	-
Fordwah	2 gates	5.90	4.49	472.47	6.59	4.59	9.15	4.66	0.87
		5.97	4.49	472.47	6.59	4.59	9.12	4.62	
Azim	2 gates	5.94	4.99	472.47	6.56	3.18	9.61	4.62	0.53
		5.97	4.99	472.47	6.56	3.18	9.61	4.62	
Mehmud	1 gate/culvert	4.10	2.39	473.98	5.08	4.76	5.58	-1.20	0.5

* : gauge

ANNEX A.5.: Direct Outlets in Fordwah Branch, Chishtian Subdivision

Name	Location		Type	measured Q		Size		Cd
	RD	m		cusecs	m^3/s	inch	m	
Out1R	260115	79283	pipe	0.73	0.021	5.5	0.14	0.53
Out2R	263186	80219	pipe	2.53	0.072	6	0.15	-
Out3R	272600	83088	pipe	2.25	0.064	6.7	0.17	0.68
Out1L	273200	83271	pipe	2.08	0.059	9	0.23	0.57
Out4R	296500	90373	pipe	5.68	0.161	8	0.2	1.42
Out2L	303000	92354	pipe	1.23	0.035	4.7	0.12	0.68
Out3L	305500	93116	pipe	1.87	0.053	5	0.13	0.94
Out4L	308855	94139	pipe	2.06	0.058	5.5	0.14	1.36
Out5R	311620	94982	pipe	3.38	0.096	8.5	0.22	0.64
Out5L	313384	95519	pipe	1.31	0.037	7.5	0.19	0.57
Out6R	314050	95722	pipe	2.6	0.074	5.5	0.14	1.22
Out6L	316250	96393	pipe	3.33	0.094	12	0.3	0.74
Out7L	316350	96423	pipe	-	0	9	0.23	-
Out8L	333500	101651	pipe	2.1	0.059	7.5	0.19	0.75
Out9L	342275	104325	pipe	2.29	0.065	6.5	0.17	2.09
Out7R	352700	107503	APM[1]	3.14	0.089	0.28	0.09	1.07
						0.8	0.24	
Out10L	363500	110795	pipe	0.53	0.015	3	0.08	0.16[2]
Out8R	368000	112166	pipe	8.83	0.25	12	0.3	0.66
Out11L	370742	113002	pipe	1.58	0.045	5.8	0.15	0.38

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[1] For this outlet, the "Size" column displays first the width and then the height of the APM.

[2] This values stands for Cd*A, with A=area of the outlet

ANNEX A.6. Hydraulic parameters (.FLU file)

```
*---------->  CROSS DEVICE
V  1  2  1     2.29    156.02        .01        .56                  1.75
V  1  2  2     2.29    156.03        .87        .56                  1.75
V  1  2  3     2.30    156.03        .88        .56                  1.75
V  1  2  4     4.57    156.03        .84        .56                  1.75
V  1  2  5     2.27    156.03        .56        .56                  1.75
V  5  2  1     2.25    151.71        .80        .51                  2.57
V  5  2  2     2.28    151.71        .80        .51                  2.57
V  5  2  3     2.27    151.71        .79        .51                  2.57
V  5  2  4     2.28    151.71        .79        .51                  2.57
V 12  2  1     4.56    150.25        .87        .60                  1.98
V 12  2  2     4.60    150.25        .93        .60                  1.98
V 22  2  1     2.27    148.03        .88        .58                  2.06
V 22  2  2     2.27    148.03        .84        .58                  2.06
V 22  2  3     2.27    148.03        .89        .58                  2.06
D 26  2  6     6.12    147.47                   .48
V 29  3  1     6.10    146.23        .76        .63                  1.83
D 29 14  6     6.11    144.65                   .40
V 35  2  1     1.82    144.01        .47        .53                  1.52
V 35  2  2     1.81    144.01        .47        .53                  1.52
*---------->  NODE - OFFTAKE or IMPOSED DISCHARGE
PHEAD  Q0Q    25.450    .000    .00     .00     .00 .00   .00 .00   .00    .00
PDAUL1 V2Q    -3.240   4.530 152.93    1.27     .41 .40 152.87 .40  3.00 153.43
PDAUL2 V2I      .000   2.270 152.93    1.71     .48 .40 152.87 .40  1.50 153.43
PMOHAR V2Q     -.890   1.210 152.74    1.37     .50 .65 153.37 .40  1.00 153.43
P3L    D3Q     -.381   1.210 153.23     .00     .00 .08 152.61 .99   .44 154.17
POUT1R P2I     -.021    .140 151.42     .00     .14 .53 151.42 .40  1.00    .00
POUT2R P2I     -.072    .150 151.36     .00     .15 .50 151.36 .40  1.00    .00
PPHOGAND2Q     -.490    .690 151.84     .00     .00 .32 151.85 .40   .80 152.90
POUT3R P2I     -.064    .170 150.68     .00     .17 .68 150.68 .40  1.00    .00
POUT1L P2I     -.059    .230 150.68     .00     .23 .57 150.68 .40  1.00    .00
PKHEM  V3Q    -1.150   1.180 151.01    1.41     .132.16 151.171.50   .44 151.32
P4L    D3Q     -.360    .890 150.66     .00     .00 .10 150.902.99   .44 151.80
POUT4R P2I     -.161    .200 149.33     .00     .201.42 149.33 .40  1.00    .00
PJAGIR V3Q     -.836   1.500 149.10    1.71     .45 .75 149.101.28   .73 150.55
POUT2L P2I     -.035    .120 149.02     .00     .12 .68 149.02 .40  1.00    .00
POUT3L P2I     -.053    .130 149.24     .00     .13 .94 149.24 .40  1.00    .00
POUT4L P2I     -.058    .140 148.68     .00     .141.36 148.68 .40  1.00    .00
POUT5R P2I     -.096    .220 148.89     .00     .22 .64 148.89 .40  1.00    .00
POUT5L P2I     -.037    .190 148.78     .00     .19 .57 148.78 .40  1.00    .00
POUT6R P2I     -.074    .140 148.78     .00     .141.22 148.78 .40  1.00    .00
PSFARIDV2Q     -.080   2.600 148.26    1.82     .01 .56 147.34 .40  1.00    .00
PMASOODV3Q     -.954   1.810 148.40    1.47     .52 .50 148.422.80  1.20 149.92
POUT6L P2I     -.094    .300 148.03     .00     .30 .74 148.03 .40  1.00    .00
POUT7L P2I      .000    .230 147.92     .00     .23 .50 147.92 .40  1.00    .00
POUT8L P2I     -.059    .190 147.13     .00     .19 .75 147.13 .40  1.00    .00
PSODA  D2Q    -1.650   1.430 147.47     .00     .00 .28 147.67 .44  1.00    .00
POUT9L P2I     -.065    .170 146.90     .00     .172.09 146.90 .40  1.00    .00
P5L    D2Q     -.310    .750 146.91     .00     .00 .10 147.27 .44  1.00    .00
POUT7R A2I     -.089    .090 146.11     .24     .241.07 146.11 .40  1.00    .00
POUT10LP2I     -.015    .080 143.81     .00     .08 .16 143.81 .40  1.00    .00
POUT8R P2I     -.250    .300 143.96     .00     .30 .66 143.96 .40  1.00    .00
POUT11LP2I     -.045    .150 144.32     .00     .15 .38 144.32 .40  1.00    .00
PFORD1 V3Q    -5.200   3.620 144.01    1.37     .74 .56 144.021.99  5.00 145.17
PFORD2 V3I      .000   1.810 144.01    1.37     .59 .56 144.021.99  2.50 145.17
PMEHMUDV2Q     -.700   1.250 144.47     .73     .41 .38 144.67 .40  2.00    .00
*---------->  STRICKLER
*---->  REACH   1
K  1  1 49   52.6316
*---->  REACH   2
K  2  1  2   52.6316
*---->  REACH   3
K  3  1  2   52.6316
*---->  REACH   4
K  4  1  2   52.6316
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*----> REACH   5
K  5  1 18   52.6316
*----> REACH   6
K  6  1  5   52.6316
*----> REACH   7
K  7  1  4   52.6316
*----> REACH   8
K  8  1  6   43.4783
*----> REACH   9
K  9  1  3   43.4783
*----> REACH  10
K 10  1  9   43.4783
*----> REACH  11
K 11  1  2   43.4783
*----> REACH  12
K 12  1  2   43.4783
K 12  3 18   50.0000
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K 13  1  2   50.0000
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K 14  1  6   47.6190
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K 15  1  4   47.6190
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K 16  1  3   47.6190
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K 17  1  5   47.6190
*----> REACH  18
K 18  1  3   47.6190
*----> REACH  19
K 19  1  2   47.6190
*----> REACH  20
K 20  1  3   47.6190
*----> REACH  21
K 21  1  2   47.6190
*----> REACH  22
K 22  1  3   47.6190
K 22  4  4   41.6667
*----> REACH  23
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K 29  4 17   50.0000
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K 32  1  2   50.0000
*----> REACH  33
K 33  1  2   50.0000
*----> REACH  34
K 34  1  2   50.0000
*----> REACH  35
K 35  1  5   50.0000
*----------> SEEPAGE
*----> REACH   1
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L 21  1  2   -60.0000
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L 22  1  4   -60.0000
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L 23  1  2   -60.0000
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L 24  1 18  -60.0000
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L 26  1 12  -60.0000
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L 27  1  5  -60.0000
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L 28  1  5  -60.0000
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L 29  1 17  -60.0000
L 29  1 17  -60.0000
*---->  REACH  30
L 30  1  5  -60.0000
L 30  1  5  -60.0000
*---->  REACH  31
L 31  1  3  -60.0000
L 31  1  3  -60.0000
*---->  REACH  32
L 32  1  2  -60.0000
L 32  1  2  -60.0000
*---->  REACH  33
L 33  1  2  -60.0000
L 33  1  2  -60.0000
*---->  REACH  34
L 34  1  2  -60.0000
L 34  1  2  -60.0000
*---->  REACH  35
L 35  1  5  -60.0000
L 35  1  5  -60.0000
*---------->  DOWNSTREAM CONDITION
AAZIM           .000   143.070
AAZIM           .504   143.260
AAZIM          1.343   143.560
AAZIM          2.000   143.660
AAZIM          2.820   143.960
AAZIM          3.490   144.160
AAZIM          4.267   144.320
AAZIM          5.262   144.470
AAZIM          6.177   144.660
AAZIM          7.007   144.860
AAZIM          8.000   145.060
AAZIM         10.000   145.210
```

```
L  1  1 49  -60.0000
L  1  1 49  -60.0000
*---->  REACH   2
L  2  1  2  -60.0000
L  2  1  2  -60.0000
*---->  REACH   3
L  3  1  2  -60.0000
L  3  1  2  -60.0000
*---->  REACH   4
L  4  1  2  -60.0000
L  4  1  2  -60.0000
*---->  REACH   5
L  5  1 18  -60.0000
L  5  1 18  -60.0000
*---->  REACH   6
L  6  1  5  -60.0000
L  6  1  5  -60.0000
*---->  REACH   7
L  7  1  4  -60.0000
L  7  1  4  -60.0000
*---->  REACH   8
L  8  1  6  -60.0000
L  8  1  6  -60.0000
*---->  REACH   9
L  9  1  3  -60.0000
L  9  1  3  -60.0000
*---->  REACH  10
L 10  1  9  -60.0000
L 10  1  9  -60.0000
*---->  REACH  11
L 11  1  2  -60.0000
L 11  1  2  -60.0000
*---->  REACH  12
L 12  1 18  -60.0000
L 12  1 18  -60.0000
*---->  REACH  13
L 13  1  2  -60.0000
L 13  1  2  -60.0000
*---->  REACH  14
L 14  1  6  -60.0000
L 14  1  6  -60.0000
*---->  REACH  15
L 15  1  4  -60.0000
L 15  1  4  -60.0000
*---->  REACH  16
L 16  1  3  -60.0000
L 16  1  3  -60.0000
*---->  REACH  17
L 17  1  5  -60.0000
L 17  1  5  -60.0000
*---->  REACH  18
L 18  1  3  -60.0000
L 18  1  3  -60.0000
*---->  REACH  19
L 19  1  2  -60.0000
L 19  1  2  -60.0000
*---->  REACH  20
L 20  1  3  -60.0000
L 20  1  3  -60.0000
*---->  REACH  21
L 21  1  2  -60.0000
L 21  1  2  -60.0000
*---->  REACH  22
L 22  1  4  -60.0000
L 22  1  4  -60.0000
*---->  REACH  23
L 23  1  2  -60.0000
L 23  1  2  -60.0000
```

ANNEX B.1. Results of SIC model calibration, Cd and D/S conditions for offtakes

Offtakes	Cd	D/S condition in SIC			
		crest	n or Cweir	Qo or Lweir	Zo
Daulat	0.40	152.87	0.40	3	-
Mohar	0.65	153.37	0.40	1	-
3L	0.08	152.61	0.99	0.44	154.17
Phogan	0.32	151.85	0.40	0.80	-
Khem Gahr	2.16	151.17	1.50	0.44	151.32
4L	0.10	150.90	2.99	0.44	151.80
Jagir	0.75	149.10	1.28	0.73	150.55
Shahar Farid	0.56	147.34	0.40	1	-
Masood	0.50	148.42	2.80	1.20	149.92
Soda	0.28	147.67	0.44	1	-
5L	0.10	147.27	0.44	1	-
Fordwah	0.56	144.02	1.99	5	145.17
Mehmud	0.38	144.67	0.40	2	-
Azim [1]	0.53	-	-	-	-

The D/S condition for offtakes in SIC can be defined between 3 types:

- Constant D/S level
- Weir type condition
- User defined condition

The last option enables to define a D/S condition as a rating curve, with an equation:

$$Q = Qo * [(Z-Zs)/(Zo-Zs)]^n$$

where Zs is the crest elevation,
Zo and Qo the measured elevation and discharge,
and n the power of the relation.

[1] Azim is taken as the tail of the system, therefore modelled as a cross regulator. The boundary condition is specified as a Q(H) relation, defined with a maximum of 16 points.

ANNEX B.2. Graphs of SIC calibration and validation

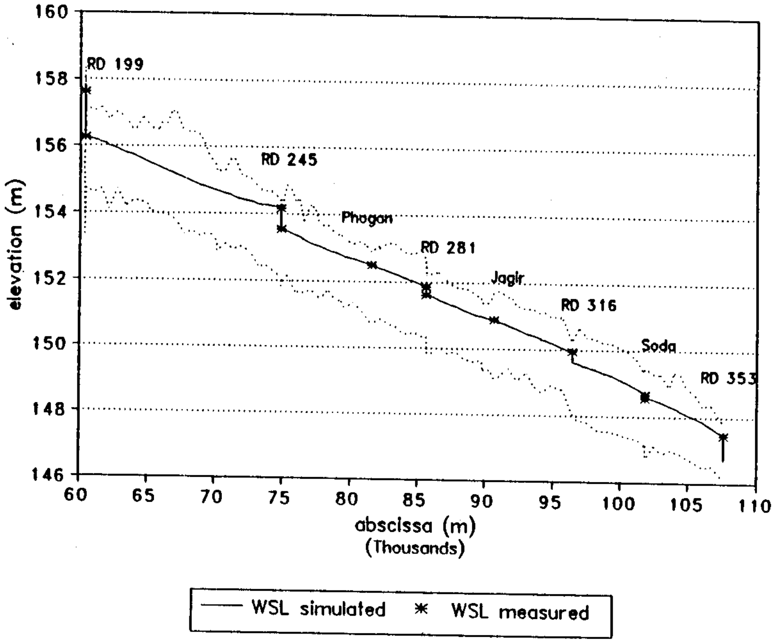


Figure I: SIC calibration, Reaches 1-4, SFP 1 2-3 June 1995

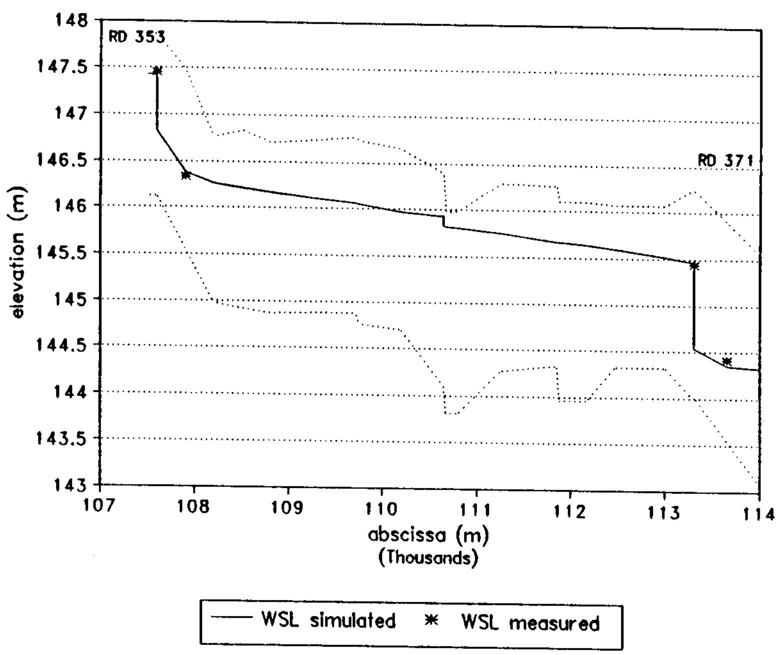


Figure II: SIC calibration, Reach 5, SFP 2, 3-4 June 1995

Validation in unsteady state

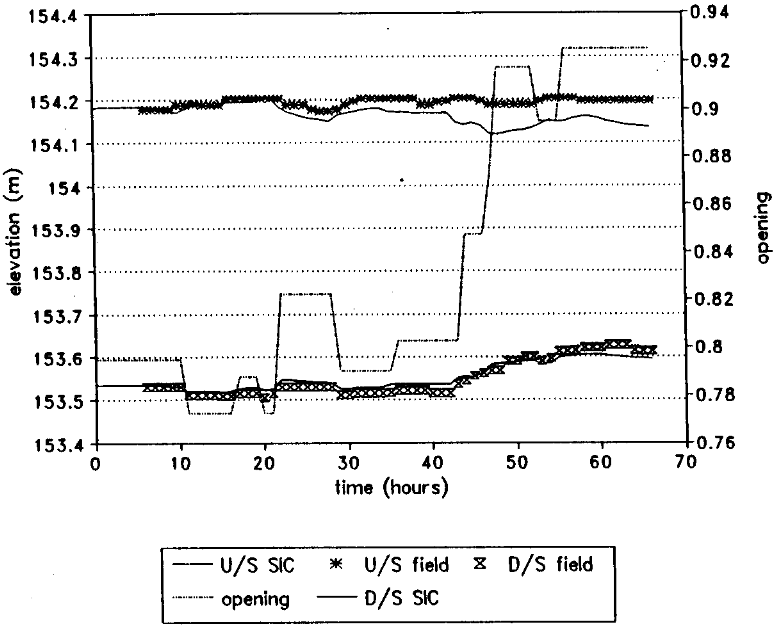


Figure III: Validation at RD 245

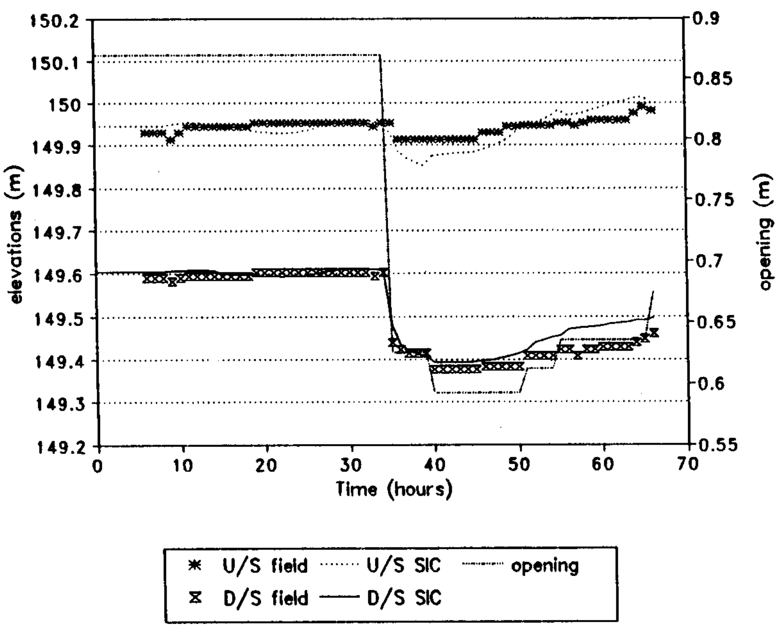


Figure IV: Validation at RD 316

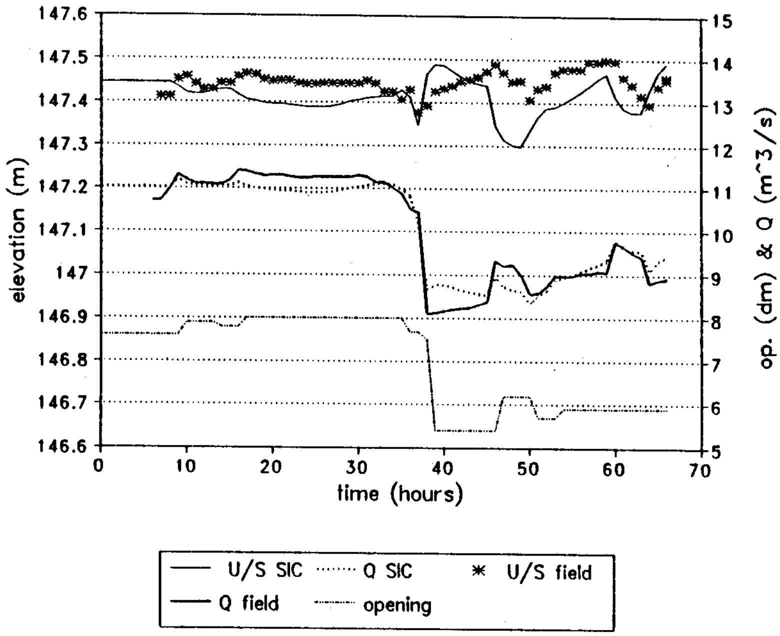


Figure V: Validation at RD 353

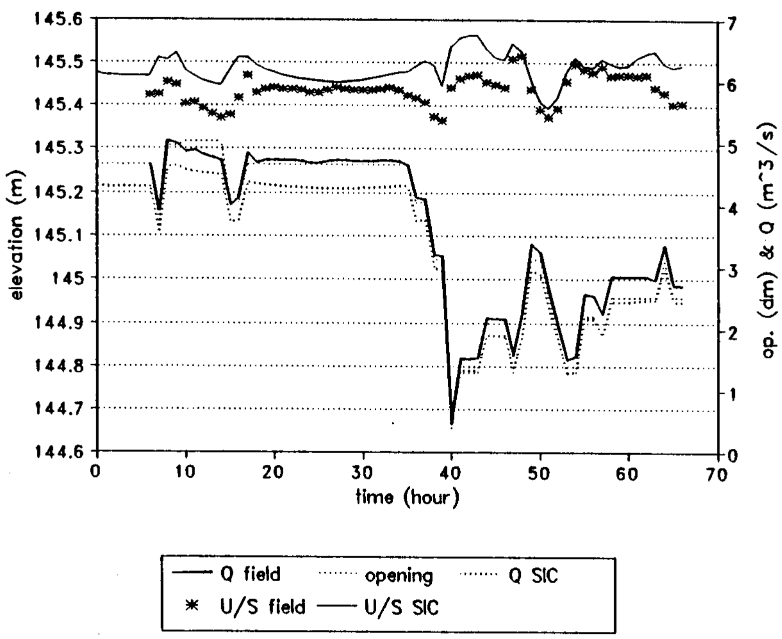


Figure VI: Validation at Azim disty

ANNEX B.3. Graphs of regulation module calibration

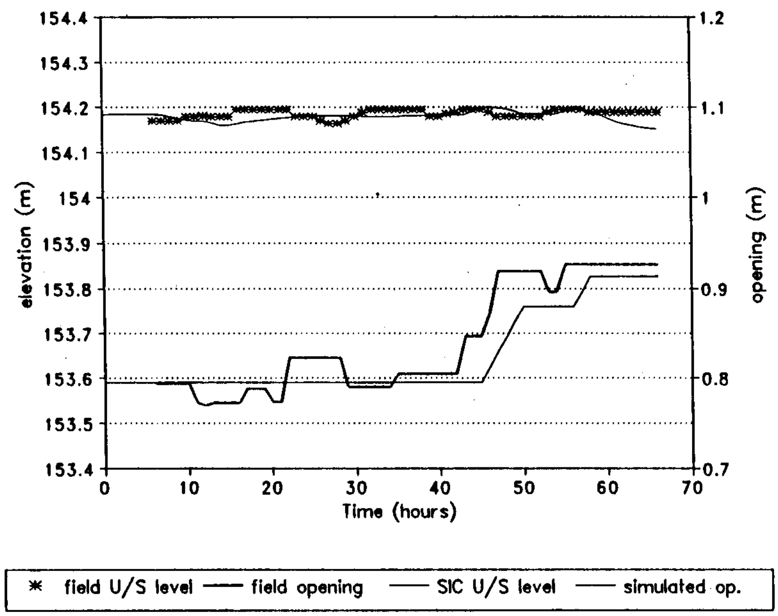


Figure VII: RD 245

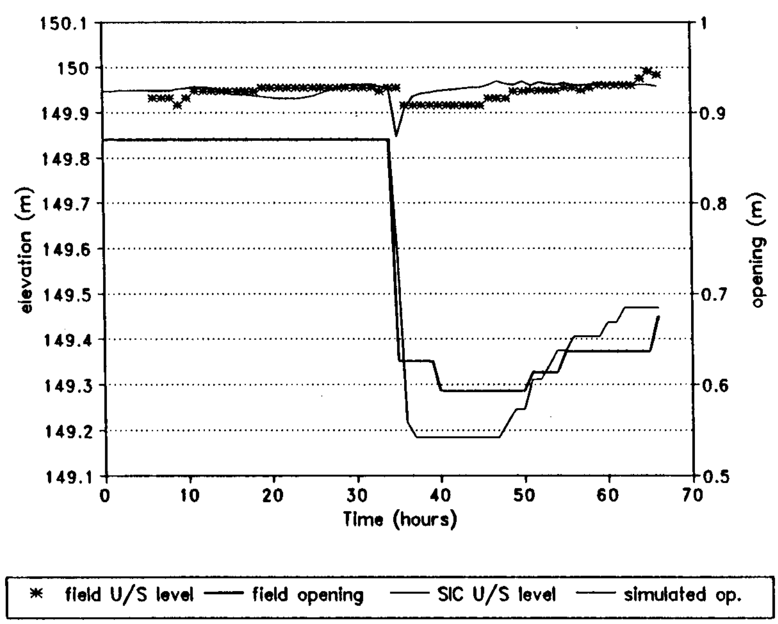


Figure VIII: RD 316

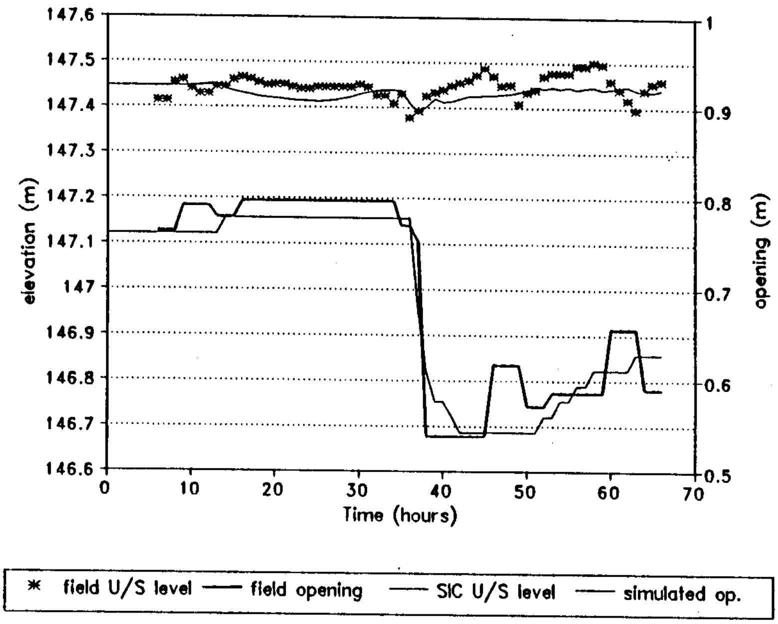


Figure IX: RD 353

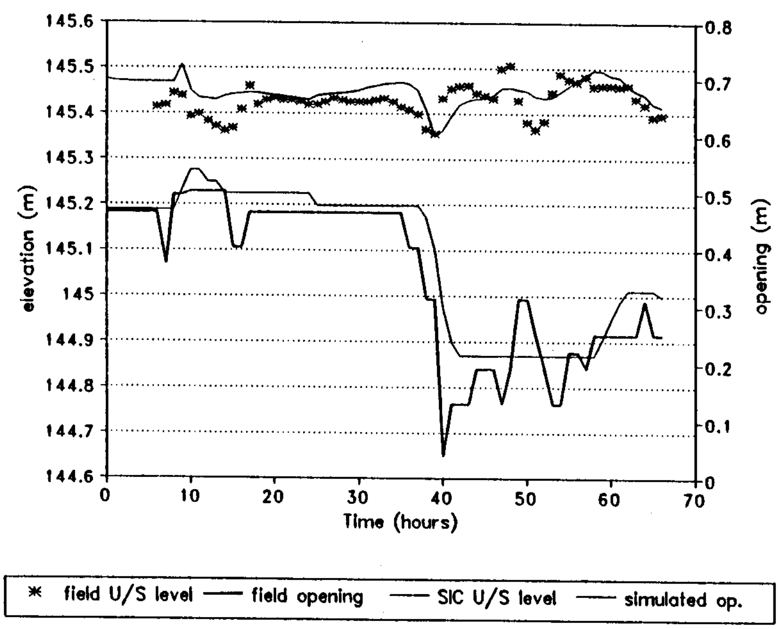


Figure X: Azim disty

ANNEX C.1. Examples of .REG files

Operations at cross regulators

```
* Fichier de regulation pour Fordwah Branch
*
R  1
*NReg Tder DTop Top Tdebj Djou Inf Sup OuMax OuMin  Fma  Fmi  Couv  Cferm CONS
P 0.   6.   1.  10.   0.  24.  -2.   2.  25.   1.  -25.  -1.   1.    1.  0002.
R  2
*NReg Tder DTop Top Tdebj Djou Inf Sup OuMax OuMin  Fma  Fmi  Couv  Cferm CONS
P 6.   1.   1.  10.   0.  24.  -2.   2.  25.   1.  -25.  -1.   1.08  0.99-154.21
R  3
*NReg Tder DTop Top Tdebj Djou Inf Sup OuMax OuMin  Fma  Fmi  Couv  Cferm CONS
P 6.   1.   1.  10.   0.  24.  -2.   2.  25.   1.  -25.  -1.   1.09  1.05-151.87
R  4
*NReg Tder DTop Top Tdebj Djou Inf Sup OuMax OuMin  Fma  Fmi  Couv  Cferm CONS
P 6.   1.   1.  10.   0.  24.  -2.5  2.5 25.   1.  -25.  -1.   1.01  1.05-149.94
R  5
*NReg Tder DTop Top Tdebj Djou Inf Sup OuMax OuMin  Fma  Fmi  Couv  Cferm CONS
P 0.   6.   1.  10.   0.  24.  -2.   2.  25.   1.  -25.  -1.   1.    1.  0002.
R  6
*NReg Tder DTop Top Tdebj Djou Inf Sup OuMax OuMin  Fma  Fmi  Couv  Cferm CONS
P 6.   1.   1.  10.   0.  24.  -2.   2.  25.   1.  -25.  -1.   1.09  1.07-147.45
R  7
*NReg Tder DTop Top Tdebj Djou Inf Sup OuMax OuMin  Fma  Fmi  Couv  Cferm CONS
P 0.   6.   1.  10.   0.  24.  -2.   2.  25.   1.  -25.  -1.   1.    1.  0002.
R  8
*NReg Tder DTop Top Tdebj Djou Inf Sup OuMax OuMin  Fma  Fmi  Couv  Cferm CONS
P 6.   1.   0.3310.   0.  24.  -2.   2.  25.   1.  -27.  -1.   1.68  2.03-145.45
```

Operations at offtakes

```
* Regulation file for Fordwah Branch
*
R  1
*NReg Tder DTop Top Tdebj Djou Inf Sup OuMax OuMin  Fma  Fmi  Couv  Cferm CONS
P 0.   0.   1.5 10.  0.  24.  -2.   2.  25.   1.  -25.  -1.   1.    1.  0002.0
R  2
*NReg Tder DTop Top Tdebj Djou Inf Sup OuMax OuMin  Fma  Fmi  Couv  Cferm CONS
P 6.   0.   1.5 10.  0.  24.  -2.   2.  25.   1.  -25.  -1.   1.    1.  -154.21
*Offtakes parameters
*     Inf Sup OuMax OuMin  Fma  Fmi  Couv  Cfer  WPC  WRC
O 3.  -2.   2.  25.   1.  -25.  -1.   1.    1.   0.   0.
*Priorities
*          JJ  HH MM  PRIO
           00  05:00  -4
           00  15:00   0
           00  24:00  -4
R  3
*NReg Tder DTop Top Tdebj Djou Inf Sup OuMax OuMin  Fma  Fmi  Couv  Cferm CONS
P 6.   1.   1.5 10.  0.  24.  -2.   2.  25.   1.  -25.  -1.   1.    1.  -151.87
*Offtakes parameters
*     Inf Sup OuMax OuMin  Fma  Fmi  Couv  Cfer  WPC  WRC
O 3.  -2.   2.  25.   1.  -25.  -1.   1.    1.   0.   0.
*Priorities
*          JJ  HH MM  PRIO
           00  24:00  -2
R  4
*NReg Tder DTop Top Tdebj Djou Inf Sup OuMax OuMin  Fma  Fmi  Couv  Cferm CONS
P 6.   0.   1.5 10.  0.  24.  -2.   2.  25.   1.  -25.  -1.   1.    1.  -149.94
*Offtakes parameters
*     Inf Sup OuMax OuMin  Fma  Fmi  Couv  Cfer  WPC  WRC
O 3.  -2.   2.  25.   1.  -25.  -1.   1.    1.   0.   0.
*Priorities
*          JJ  HH MM  PRIO
           00  15:00  -2
           00  24:00   0
R  5
*NReg Tder DTop Top Tdebj Djou Inf Sup OuMax OuMin  Fma  Fmi  Couv  Cferm CONS
P 0.   0.   1.5 10.  0.  24.  -2.   2.  25.   1.  -25.  -1.   1.    1.  -147.47
R  6
*NReg Tder DTop Top Tdebj Djou Inf Sup OuMax OuMin  Fma  Fmi  Couv  Cferm CONS
P 6.   0.   1.5 10.  0.  24.  -2.   2.  25.   1.  -25.  -1.   1.    1.  -147.47
R  7
*NReg Tder DTop Top Tdebj Djou Inf Sup OuMax OuMin  Fma  Fmi  Couv  Cferm CONS
P 0.   0.   1.5 10.  0.  24.  -2.   2.  25.   1.  -25.  -1.   1.    1.  -144.98
R  8
*NReg Tder DTop Top Tdebj Djou Inf Sup OuMax OuMin  Fma  Fmi  Couv  Cferm CONS
P 6.   0.   1.5 10.  0.  24.  -2.   2.  25.   1.  -25.  -1.   1.    1.  -145.45
```

ANNEX C.2. Gateman User's Guide

The .REG file displays the coefficients described in chapter 3.
First Character:

<u>* for a comment</u>

<u>R for a regulator</u> (cross structure). Cross weirs are considered as regulators. The numbers are given from upstream to downstream.

<u>P for parameters for the regulator.</u>

- column 1: NReg = Number for the method of regulation (see listing of Gateman for different methods of computation of the openings of a cross regulator)
- column 2: Tder = Time of the first operation (h)
- column 3: Dtop = Time between two operations (h)
- column 4: Top = Time of an operation (min)
- column 5: Tdebj = Time of the beginning of the day work (h)
- column 6: Djou = Duration of a day work (h)
- column 7: Inf = lower limit of intervention (cm)
- column 8: Sup = upper limit of intervention (cm)
- column 9: OuMax = Maximum amplitude for opening operation (cm)
- column 10: OuMin = Minimum amplitude for opening operation (cm)
- column 11: Fma = Maximum amplitude for closing operation (cm)
- column 12: Fmin = Minimum amplitude for closing operation (cm)
- column 13: Couv = Amplification coefficient for opening operation
- column 14: Cferm = Amplification coefficient for closing operation
- column 15: CONS = Targeted upstream level or FSD (m) (give -FSD)

<u>O for parameters for offtake operations.</u>

- column 1: Number for the method of regulation (see listing of Gateman for different methods of computation of the openings of an offtake)
- columns 2 to 9: same signification as the parameters for regulator
- column 10: WPC = Opening to apply to the offtake to get the targeted discharge with the upstream level equal to FSD (cm)
- column 11: WRC = Opening to apply to the regulator to get the targeted discharge with the upstream level equal to FSD (cm)

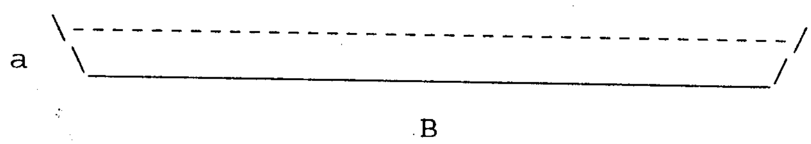
<u>Timing for priorities:</u>

- column 1: JJ = Days
- column 2: HH = Hours
- column 3: MM = Minutes
- column 4: PRIO = 0 if the regulator is operated until the time JJ, HH, MM.

  -N if the node located N nodes upstream of the regulator is operated until the time JJ, HH, MM.

ANNEX D.1. Computation of the total wetted perimeter using SIC's results

By running the steady state flow module, SIC computes the volumes of water in each reach as well as the water surface level (which will give us the water depth).
We thus have the volume of water V, the length of each reach L, and the average depth of water D in a reach. As canals in the area are generally very wide and flat, we can assume a trapezoidal section in which the bed width $B >> a$ (see sketch). The side slope is assumed to be 45°.



If $D = \frac{a}{\sqrt{2}}$ and the wetted perimeter $P = B + 2*a$

then $P = B + 2\sqrt{2}*D = B + 2.83*D$

The area S is given by: $S = B * D + D^2$

By replacing B by P - 2.83*D we find:

$S = P * D - (2.83 - 1) * D^2 = P * D - 1.83*D^2$

Since we know V (= S * L) from SIC results, we can then compute
P * L for each reach.

<u>Application</u>: The total wetted area calculated with this method for Fordwah Branch, Chishtian Subdivision with an inflow of 25.45 m^3/s is equal to 12.4 million square feet, or 1.15 million square meters.

ANNEX D.2. Error margin in seepage computation by inflow-outflow method

During a SFP in a given reach, the seepage can be estimated as the ratio of the inflow and the outflows by the reach length:

q = - 10E6*[Qu/s - (Qd/s+Qofftakes)]/(reach length)

with q = seepage outflow [l/s/km]
Qu/s = inflow discharge [m^3/s]
Qd/s = outflow discharge d/s [m^3/s]
Qofftakes = outflow discharge at offtakes [m^3/s]
reach length [m]

This method gives results with a rather big margin of error, especially when the discharges are not measured, but computed from levels and openings.

The error margin in seepage computation stems from different sources:
- systematic error (topographic survey for White Marks[1])
- error on the water levels readings: in a study focused on the roughness coefficient, Male adopts an accuracy of +/- 1.5 cm in upstream water levels, taking into account the systematic error from the topographic survey (Male, 1992)
- error on the openings readings: accuracy of +/- 1 cm
- error on the Cd: this is the most important source of error, but also the most difficult to estimate. It stems from measures of discharges with a current metering, and for big canals, the measure can take a few hours, during which the flow is rarely constant.

a. Error on the Cd

If we assume an error of 5% in the discharge measurement, and the above values for levels and opening readings, the absolute error on the Cd for a gate will be expressed as:

dCd/Cd = dQ/Q + dw/w + 1/2*dH1/H1

If we take typical values of w = 1m, and H1 = 1.5m, dw = +/- 1 cm, dH1 = +/- 1.5 cm, then:

dCd/Cd = 5% + 1% + 0.5*1% = 6.5%

We see that the error on the Cd is mainly determined by the error on the discharge measurement.

---

[1] The White Marks are used as references for water levels readings

These errors are responsible for the global error in the calculation of discharges with water levels and openings. The seepage evaluated by inflow-outflow method is therefore calculated with an important error margin.

<u>b. Discharge accuracy at structures:</u>

if $Q = a*Cd*w*L*H1^{0.5}$, then

$$dQ/Q = dw/w + dCd/Cd + dL/L + 0.5*dH1/H1$$

Then the accuracy on the discharge is:

$$dQ/Q = 2\% + 6.5\% + 0.5*1\%$$ (dL is assumed to be nil)

or:

$$dQ/Q = 9\%$$

<u>c. Error on the seepage computation</u>

If the computation of the seepage is done in a reach with one structure at the head and one at the tail of the reach, and discharges calculated from water levels readings, we get:

$$Q'1 = Q1 + DQ1$$

$$Q'2 = Q2 + DQ2$$

with Q'1 : calculated discharge at the head
Q1 : real discharge at the head
DQ1 : error on the discharge
and the same notations for the tail (number 2).

The calculated seepage is therefore:

$$q' = q + 10E6*(DQ1+DQ2)/(\text{reach length})$$

if we note q the real seepage, and q' the calculated one.

It is clear that the error on the seepage will decrease if the length of the reach increases. But if the seepage is low, the error of the calculated seepage in terms of percentage of the real seepage can be very high.

Example of application:

For a 10 km long reach, with an real inflow of 25 m^3/s, and a real seepage of 50 l/s/km, we have:

$Q1=25m^3/s$
$Q2=24.5m^3/s$

The error is supposed to be of 9% for the discharge calculations:
DQ/Q=9% for 1 and 2.

The maximum error on the seepage is then:

$$Dq/q = 10E6*(DQ1+DQ2)/(\text{reach length})/(Q1\text{-}Q2)$$

or $Dq/q = 10E6*(0.09*25 + 0.09*24.5)/(10*1000)/(0.5)$

that is:

$$Dq/q = 891\ \%$$

There is a maximum coefficient of 9 between the real and calculated seepage. This is a maximum value, if the errors on the calculation do not compensate one for the other.

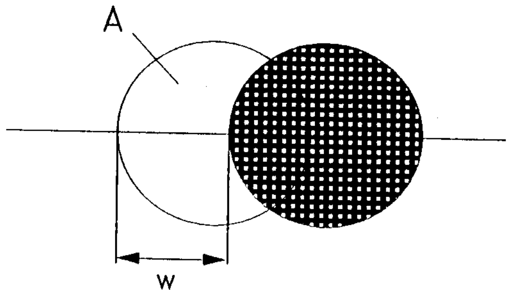
This margin will become higher as the number of structures where the flow is calculated this way increases.

Therefore, any seepage calculation based on inflow-outflow method with calculation of discharge from water levels is subject to a high error margin.

The seepage values calculated this way in the present study were found rather realistic, and as the calibration of the model was good, these values were not changed.

ANNEX D.3. Computation of the opening of a circular pipe with a circular gate.

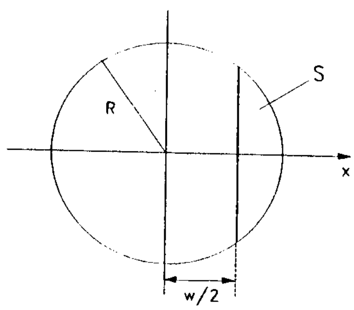
We assume the radius of the pipe and the radius of the gate are equal, and this value is noted R (in meters).
We are looking for A, the area of the opening, when the gate is up-lifted of w meters.



First we have:

$$A = \pi R^2 - 2S$$

with S the area between the circle and the vertical line at x=w/2.



The computation of S can be done by integrating the function

$$f(x) = 2\sqrt{R^2 - x^2}$$

between w/2 and R.

We have:

$$S = 2\int \sqrt{R^2 - x^2}\, dx$$

With two changes of variables, first u=x/R, then t=Arccos(u), we get the result:

$$A = R^2 \left[\pi - 2Arccos\left(\frac{w}{2R}\right) + \frac{w}{R} \cdot \sqrt{1 - \frac{w^2}{4R^2}}\right]$$

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RESEARCH REPORTS

Report No.	Title	Author	Year
R-1	**Crop-Based Irrigation Operations Study in the North West Frontier Province of Pakistan** Volume I: Synthesis of Findings and Recommendations	Carlos Garces-R D.J. Bandaragoda Pierre Strosser	June 1994
	Volume II: Research Approach and Interpretation	Carlos Garces-R Ms. Zaigham Habib Pierre Strosser Tissa Bandaragoda Rana M. Afaq Saeed ur Rehman Abdul Hakim Khan	June 1994
	Volume III: Data Collection Procedures and Data Sets	Rana M. Afaq Pierre Strosser Saeed ur Rehman Abdul Hakim Khan Carlos Garces-R	June 1994
R-2	Salinity and Sodicity Research in Pakistan - Proceedings of a one-day Workshop	IIMI-Pakistan	Mar 1995
R-3	Farmers' Perceptions on Salinity and Sodicity: A case study into farmers' knowledge of salinity and sodicity, and their strategies and practices to deal with salinity and sodicity in their farming systems	Neeltje Kielen	May 1996
R-4	Modelling the Effects of Irrigation Management on Soil Salinity and Crop Transpiration at the Field Level (M.Sc Thesis - pulished as Research Report)	S.M.P. Smets	June 1996
R-5	Water Distribution at the Secondary Level in the Chishtian Sub-division	M. Amin K. Tareen Khalid Mahmood Anwar Iqbal Mushtaq Khan Marcel Kuper	July 1996
R-6	Farmers Ability to Cope with Salinity and Sodicity: Farmers' perceptions, strategies and practices for dealing with salinity and sodicity in their farming systems	Neeltje Kielen	Aug 1996
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