REPORT NO. R-10

CANAL WATER DISTRIBUTION AT THE SECONDARY LEVEL IN THE PUNJAB, PAKISTAN

Development of a Simplified Tool to Estimate the Canal Water Distribution at the Distributary Level

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FOREWORD

This report is the thesis or final report for the Master of Science program of Mr. Steven J. Visser. He completed the requirements for an M.S. degree in Civil Engineering from the University of Technology Delft, The Netherlands during July 1996. He spent six months in Pakistan during 1995-96 to complete all of the necessary field work. The reproduction is identical of the document accepted by the University of Technology Delft.

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 one of the studies undertaken regarding unsteady flow hydraulics. In this case, an unsteady flow model was used to simulate a distributary, including all of the outlets. This capability has only recently been developed for application in Pakistan. This particular study focused on testing the sensitivity of various hydraulic parameters in determining the water distribution to the outlets along a distributary. The objective was to simplify the model.

Gaylord V. Skogerboe, Director
Pakistan National Program
International Irrigation Management Institute

ACKNOWLEDGEMENT

This report is the final stage of my study Civil Engineering at the University of Technology Delft. For this purpose I worked two weeks at *Cemagref* in Montpellier, France, 6 months at the *International Irrigation* Management *Institute (IIMI)* in Lahore, Pakistan and another 3 months at the *University of Technology Delft*, The Netherlands. This study would not have been possible without the help and advises of many people involved. I want to thank with all respect the complete IIMI field staff in Bahawalnagar for their enthusiasm and help with the collection of field data and the good times I spend in the fields. I am especially grateful to field team leader Mushtaq Ahmed Khan, and field assistants Muhammad Amin Khan Tareen, Khalid Mahmood and Anwar Iqbal for their technical and social advises to understand the water management situation of the Punjab in a better way.

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I would like to thank Pierre-Olivier Malaterre (Cemagref, Montpellier) for his time spend with me to get acquainted with the SIC software.

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Delft, July 1996

S. J. Visser

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GLOSSARY

Available working head
The minimum difference between the outlet structure upstream and downstream water levels

Discharge
A quantity of water passing in unit time

Distributary (secondary canal)
A canal off taking from either a main canal or branch, to supply canal water to outlet structures and minors, under responsibility of a Water Authority.

Duty
The area irrigated in a period of time, divided by the supplied amount of water in cfs.

Equitability
Equity of water distribution can be defined as a distribution of a fair share of water to users throughout the system, based on the irrigated area served for each outlet structure (expressed in the authorized discharge q_{auth})

Outlet structure
A device at the head of a watercourse off taking either from a distributary or direct from a main canal or branch.

Kharif
The summer flood season (hot season) lasting from 15th April to 15th October.

Main system
The irrigation infrastructure under responsibility of a Water Authority (PIPD), i.e. the river head works, main canals, branches, distributaries and minors, secondary and tertiary off taking structures and cross structures.

Minor
A small canal off taking from a distributary, to supply canal water to outlet structures, under responsibility of a Water Authority.

Modular outlet structures
Those outlet structures which discharge is independent from both the upstream water levels in the distributary as the downstream water levels in the watercourse, between reasonable limits.

Non-modular outlet structures
Those outlet structures which discharge is both depending on the upstream water levels in the distributary, as the downstream water levels in the watercourse.

Non-perennial canal
Canal designed to receive canal water only in a certain period in the year, i.e. the summer season (*Kharif*).

Performance of a distributary
The evaluation of the canal water distribution (to the outlet structures) based on the principles of irrigation in the area of study, i.e. equity and proportionality.

Proportionality
Condition where the sensitivity of a bifurcation is equal to 1, i.e. the change in the distributed discharge to an outlet structure is equal to the change in discharge in the parent canal.

Rabi
Winter season lasting from 15th October to 15th April.

Responsiveness of the system
The re-distribution of canal water to the outlet structures, based on a certain change in one of the input parameters of the model (sensitivity analysis).

Semi-modular outlet structures
Those outlet structures which discharge is depending on the upstream water levels in the distributary, but independent on the downstream water levels in the watercourse, as long as the working head required is available.

Sensitivity analysis
The study of the re-distribution of canal water to outlet structures (responsiveness of the system), based on a change in one of the canal and outlet structure characteristics in the model. The comparison of the model output before any adjustments and after an adjustment results in a study of the sensitivity of the different parameters in the model.

Sensitivity of an outlet structure (S)
The sensitivity ratio S is defined as the variation in an off taking discharge in response to a change in the continuing discharge in the parent canal.

Tertiary unit (or Chak)
Irrigated area served by one outlet structure and corresponding watercourse, supplying canal water to the individual farmers or group of farmers. In general divided in different sections. Within the tertiary unit farmers are responsible for operation and maintenance of the irrigation system.

Water allowance
The amount of supplied discharge (cfs), authorized per 1000 acres of gross or culturable command area. The water allowance not only determines the size of an outlet structure, but also forms the basis for design of the distributaries.

Watercourse (tertiary canal)
Small canal within the tertiary unit (lined or unlined), off taking from a distributary, minor or direct from a main canal or branch, supplying canal water to the farmers and under responsibility of the farmers.

CONVERSION OF UNITS

Length

1 foot (ft)	=	0.3048 m		
1 mile	=	1600.3 m		

Surface or area

1 square foot	=	0.0929 m^2		
1 acre	=	0.4047 ha		

Discharge

1 cfs or cusec (ft^3/s)	=	28.31 l/s		
1 cumec (m^3/s)	=	35.31 cfs		
1 cfs per 1000 acres	=	0.6 mm/day	=	0.07 l/s/ha
1 l/s/ha	=	8.64 mm/day		

SUMMARY

This study is part of the research of the *International Irrigation Management Institute* (IIMI - Pakistan) at the main system level of the Fordwah Eastern-Sadiqia Irrigation Project (Chishtian Sub-Division) in the Punjab, Pakistan. This study is part of the Integrated Approach, conducted by IIMI in the area of study, in order to develop a methodology to evaluate the economic and environmental impact of (changes in) irrigation management. This study is focussing on canal water distribution at the secondary level (distributary level).

Water distribution in Pakistan is mainly based on the principles of proportionality and equity. At present, the water distribution within the distributary, i.e. supply of water to the tertiary outlet structures, is characterized by a high variability and inequity. The main objective of this study is to develop a tool that predicts the canal water distribution to the tertiary units (q) as a function of the inflow (Q), state of the distributary, outlet structure characteristics and interventions therein. By quantifying the effect of these parameters on the water distribution, a better understanding of how to improve the present distribution will be obtained. To develop a tool, i.e. a simplified hydro-dynamic flow model, that predicts the water distribution for the distributaries in the area of study, many parameters must be determined. To minimize the amount of input data for the simplified flow models (thereby enabling an easier application of the model), the sensitivity of these parameters on the canal water distribution was studied, based on simulations with a hydro-dynamic flow model SIC, of one distributary (SIC software is developed by *Cemagref*, Montpellier, France). The first application of these simplified models will be to predict water distribution at the distributary level for all distributaries in the Chishtian sub-division, which will serve as one of the major components of an integrated approach to evaluate the effect of changes in canal water management on salinity/sodicity and agricultural productivity. In future, the (simplified) methodology to set up flow models, will be applied in other research studies.

A methodology was proposed to study the sensitivity of parameters determining the canal water distribution at the distributary level, simulating a defined inflow pattern at the head of the canal using a "low model of a distributary. An indicator was suggested (R-index) to quantify the impact of a change in different parameters on the re-distribution of canal water to the outlet structures. Data which are sensitive and should be defined precisely for the simplified flow model: discharge coefficient, opening height and opening width of outlet structures, and crest level and width of cross structures (drop structures). Data which are insensitive and can be simplified for the flow model: cross sectional profile, crest levels of outlet structures, seepage (inflow and outflow) and Manning's coefficient. The general simplified method to set up a flow model is based on the insensitive parameters. It can be concluded that the simplified method results in a reduction of time and money spend on developing those models, to investigate canal water distribution (accuracies of the simplified method up to 20%).

Besides that, an attempt was made to study the distribution of canal water for the actual and design state of a distributary. It can be concluded that the performance at present is inequitable and non-proportional, based on the design principles of irrigation in the area. At least three irrigation indicators are necessary to study actual canal water distribution: (1) DPR; (2) S, proportionality; (3) MIQR or CV(DPR).

CHAPTER 1 INTRODUCTION

1.1 Irrigation in Pakistan[1]

Pakistan in general

The self awareness was growing in the British ruled Indian Sub-Continent. In 1906 the Muslim League was founded to demand an independent Muslim state but it was until **24** years later that a totally separate Muslim homeland was proposed. Around the same time, a group of England-based Muslim exiles coined the name Pakistan, meaning 'Land **of** the Pure'. The ***Islamic Republic*** of ***Pakistan*** gained its independence on the 14^{th} **of** August 1947, after centuries of British influence in the Sub-Continent. Pakistan is bounded in the north by China and Uzbekistan, in the east by India, in the west by Iran and Afghanistan and in the south by the Indian Ocean. Roughly, Pakistan is situated in between 22.5" and 35° latitude north, and in between 60° and 75" longitude west. The climate in **the** center and the south of Pakistan (Sind, Baluchistan and Punjab) is dominated by hot and *dry* summers with temperatures up to 47°C, and gentle winters with temperatures up to 25°C. In the north, the climate **is** more moderate due to local geographical differences **of** the Himalayan mountains (North-West Frontier Province and Jammu / Kashmir). With an annual rainfall of 300mm a year, the transpiration **of** most of the crops always exceeds the rainfall. Intensive irrigation, both gravity irrigation and tubewell irrigation, is necessary to meet the crop water requirements. With a population of approximately 125 million and a total area of 887.700 km^2, the density of population becomes: 140.9 inhabitants ^{2}/ km . Approximately 70% of the total population is situated in the center of Pakistan (Punjab) along the main rivers of the Indus Plains. With a literacy rate of 35% only, a population growth ratio **of** 3%, and an average annual per capita income **of $380,** Pakistan can be defined as a developing country. In spite of the widespread poverty, Pakistan has the potentials to cope its problems with structural aid and investments in the industrial and agricultural sector. Pakistan economy is dominated by agriculture: 54% **of** the labour is active in the agricultural sector, which forms 26% of the Gross National Product (total GNP: $45.5 billion), and 80% of the total export value.

The Indus system

Pakistan has one of the largest contiguous gravity irrigation systems in the world, situated mainly in the Indus Plain and river Kabul / lower Swat. These days, the agricultural centre **of** Pakistan is situated along the 5 major rivers of the Indus system, and is called the Punjab[2] ***('five rivers'***).

1 *All figures dated 1991, geographical* ***rind topographical*** *information* ***see*** *figure 1.1,* ***page 2.***

7 *With the* ***inception*** *of* ***Pakistan, the*** *Punjab* ***has*** *been* ***divided into two parts: west Punjab (Pakistan) and east*** *Punjab (India). West punjab* ***can be divided*** *into* ***two regions. 7he north-west*** *dry* ***hill region and the*** *Indus* ***plains, an alluvial*** *and flat* ***plain.*** *It slopes almost imperceptibly* ***south-west.*** *West of the Sutlej* ***river, the land rises gradually*** *and fades* ***away into the Thar desert*** *(Annual Report IIMI, 1993).*

The ***Indus Basin River Irrigation System*** is fed by the Indus river and its major tributaries: Jhelum, Chenab, Ravi and Sutlej river (Indian part: Beas River). Three main reservoirs (Mangla dam, Tarbela dam and Warsak dam), 19 barrages (head works), 12 link canals and 46 main canals supplying irrigation water to an area of 16 million hectares and serve about 90,000 tertiary units. The total length of main canals (branches), secondary canals (distributaries, minors and sub-minors) and tertiary canals (watercourses) is about 60,000 km. A great amount of water supplied by the Indus Basin river system is used for irrigation, but although the scarcity of water in the area, there is still an amount of water flowing unused into the Indian Ocean.

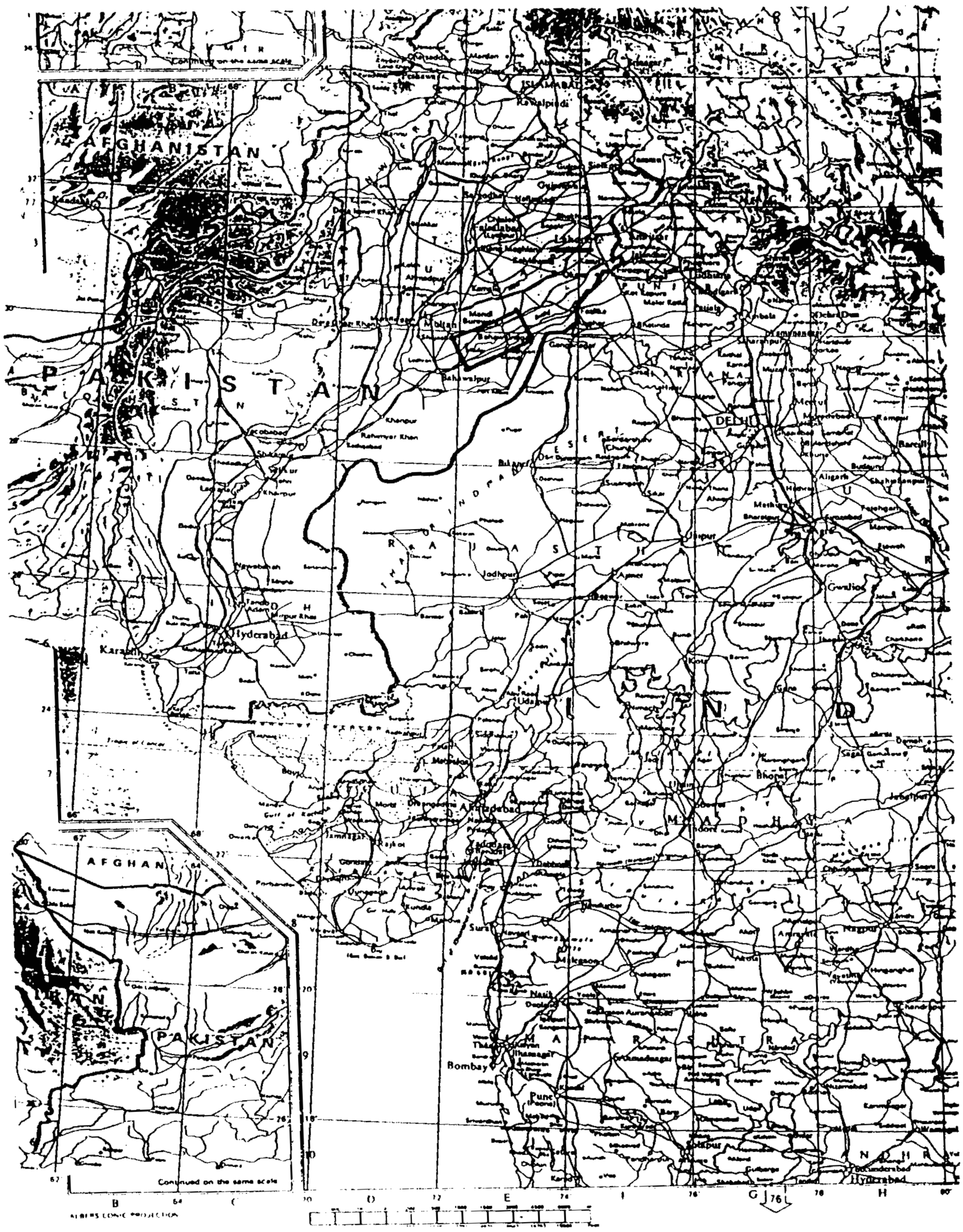

Figure 1.1 **Map of Pakistan and the area of study.**

After the independence of India and Pakistan, two rivers of the Indus system, i.e. the Sutlej river and the Ravi river, which are part of both the Pakistani and Indian irrigation system, resulted into a dispute on water rights. In 1960, the water rights were formally noted down in the Indus Water Treaty. According to this Treaty, Pakistan gained the rights of the three eastern rivers (Indus, Jhelum and Chenab), and India received the rights of the other two rivers (Ravi and Sutlej). The water of the rivers of the Indus system is fully utilized to the extend that in winter there is actually a shortage. To cope the shortage of water, especially of the Ravi and Sutlej rivers and in order to be able to distribute water of the rivers with a maximum advantage, the four rivers (Ravi, Chenab, Jhelum and Sutlej) have been linked by means of feeder canals or link canals. The development of the irrigation system in Pakistan and India **started** about 150 years ago, during British rule (van Essen and van der Feltz, 1992).

The physical layout of **the** irrigation system in Pakistan is based on a classical design approach, and based on protective irrigation management. In general, the classical layout of an irrigation system consists of two major components (for terminology **see** figure 1.2). The **main system** consists of the head works, **link** canals, the main or branch canals and main cross structure devices (cross regulators), the secondary canals (distributaries, minors and sub-minors), secondary off take structures and secondary cross structure devices. The tertiary **system** (tertiary unit) consists of *one* tertiary outlet structure and corresponding tertiary canals (watercourses).

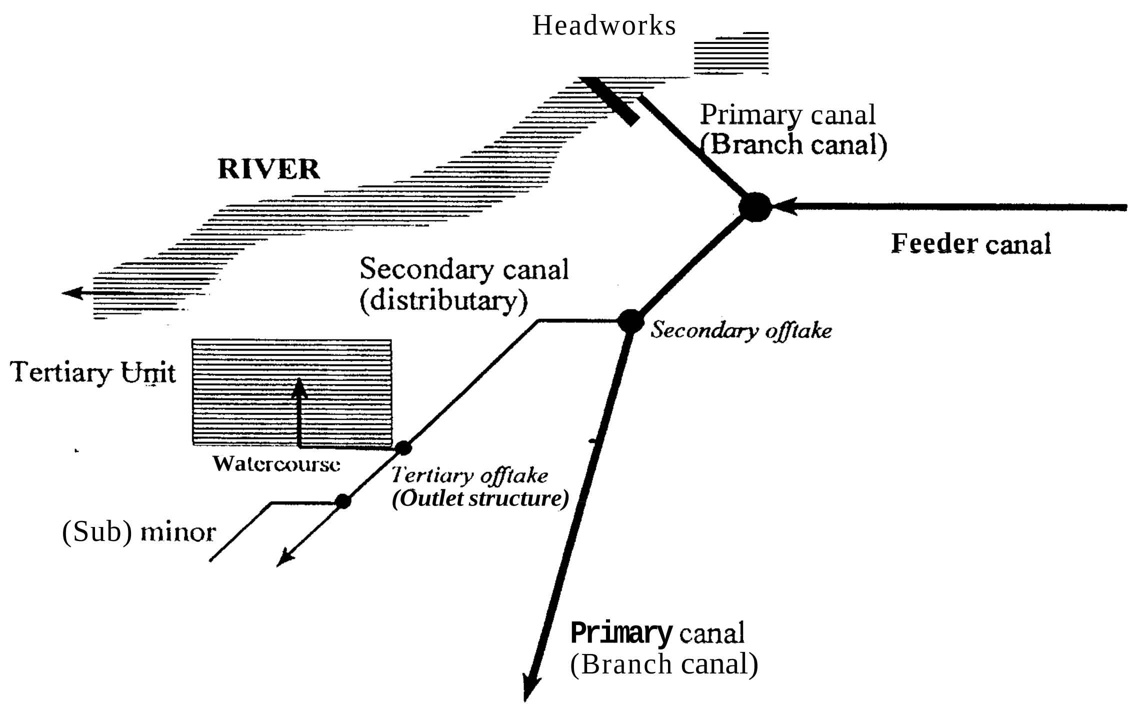

Figure 1.2 **Terminology of the Punjab irrigation system.**

For operation and maintenance, the main system is under responsibility of the Punjab Irrigation and Power Department (PIPD) and the tertiary units are under responsibility of the farmers.

Principles of irrigation

The physical system has been developed over the **years.** The available water **is** spread out over **an area as** large as possible. To **keep** water scarce, as its the major production input for irrigated agriculture, it is assumed that the return per volume water will be maximized. At present, the operation of the **system** is still based on the design principles of irrigation in the Indian Sub-Continent:

- The principle of equitable distribution of canal water. The water is distributed equally over the area in such **a** way that each outlet structure receives an amount of canal water in equal proportion to the size of its command area, i.e. the same amount of water is distributed to every acre (supply-oriented).
- The principle of proportional control of canal water at the secondary level. The available water is distributed along the distributary to the tertiary units with fixed outlet structures that divide the flow into a fixed ratio. Besides a proportional distribution of a steady **flow,** a change in discharge at the parent canal is proportionally distributed along the outlet structures.

Problem identification

I'here are a few processes influencing the overall performance of present irrigation management in the Punjab, resulting in stagnant agricultural production (wheat, rice and sugercane) and less sustainability overall. The main problems effecting the overall irrigation performance are: 1. the increasing demand of canal water supply due to intensified cropping patterns; 2. increase in saline tubewell water use, resulting in a negative effect on production due to increasing salinity and sodicity of the agricultural plots; 3. sever waterlogging at the lower parts of the system due to bad drainage and intensive irrigation; **4.** limited resources **for** proper maintenance and operation of the actual system'by the PIPD, and **5.** non-technical problems due to political and social constraints resulting in water theft and illegal irrigation practices.

This is why the ***International Irrigation Management Institute***[3] (IIMI-Pakistan), based in Lahore, Pakistan, has carried out research on inter-related issues of canal irrigation management, ground water extraction, agricultural production and salinity / sodicity since 1989. Within the overall research program, the work under the Main System Research Component has its main objective to determine the scope for interventions in management of the canal irrigation system at primary and secondary level, in order to improve agricultural production and mitigate soil salinity / sodicity.

This study is part of the Main System Research Component, as it is focussing on canal water distribution at the secondary (distributary) level. In the next section, the context of the study, within **the IIMI program** will be discussed more in detail.

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3 ***7he International Irrigation Management Institute's (IIMI) mission is to strengthen national efforts to improve and sustain the performance of irrigation systems in developing countries. With its headquarters in Colombo, Sri Lanka, IIMI conducts a worldwide program to develop and disseminate improved approaches towards imgation management. IIMI is an autonomous, nonprofit international research and training institute supported by the Consultative Group on International Agricultural Research (CGIAR). The CGIAR is sponsored by the Food and Agriculture Organization (FAO) of the United Nations, World bank, United Nations Development Programme (UNDP) and more then 45 donor countries and private foundations (IIMI annual report, 1993).***

1.2 Context of the study: the integrated Approach

Introduction

Low levels of agricultural productivity in Pakistan have long been associated with a low performance of the management (operation and maintenance) of the Indus Basin River Irrigation System, resulting in inequitable and highly variable canal water supplies along with environmental problems such as salinity, soil degradation and waterlogging (Bhutta and Vander Velde, 1992; Kijne and Kuper, 1995). Recently, IIMI-Pakistan has made a start to integrate all the research components in the area of study[4] in order to develop a methodology to evaluate the economic and environmental impact of (changes in) irrigation management, i.e. to study irrigation from main system level to watercourse (farm) level based on hydraulic, economical, sociological, institutional and agronomical aspects. The so called ***Integrated Approach*** will be based on two case studies (Kuper, Strosser et al, 1996):

- Canal management interventions to mitigate salinity: evaluate the impact of interventions in canal irrigation management (at the main system level) on salinity / sodicity and agricultural production.
- Water Markets Development in Pakistan: evaluate the technical feasibility of water market development and its impact on agricultural production and salinity / sodicity.

The integrated approach is defined as an analytical tool to study the inter-relationships of irrigation, agricultural production and the environment as a dynamical system with different levels and different disciplines. Initially, the integrated approach will be set up for the area of study, but finally, the approach will be generalized in order to study an ***a priori*** evaluation of management interventions and their environmental and economical impact of any irrigation system.

Research components

Representing an irrigation system as: 1. ***a place of collective and individual expectations of different actors that reflect power struggles within each social group*** (Molle and Ruf, 1994), and 2. a more technical bio-physical process (Merkley, 1993), two visions can be formulated:

- The whole process of irrigation has to be divided into different sub-systems (water is distributed from the head works to the farms).
- Irrigation is besides the hydraulic infrastructure an agricultural practice with many actors involved: the farmers community, policy makers and managers (the decision making processes).

Within the Integrated Approach, the different research methodologies for the different sub-systems and decision making processes are schematized, modelled (with analytical models and decision making models), simplified and linked (compatible).

[4] *The research has been concentrated on the Chishtian Sub-Division, an area of 67,000 ha irrigated sub-system, at the lower command area of the Fordwah main canal, in the south-east Punjab (more details: chapter 2).*

The following sub-systems can be distinguished:

Sub-system **1:** main system management
The main system, as it consist the main canal and the corresponding distributaries, is studied for both the physical / hydraulic state using a hydraulic flow model *SIC:* Simulation of Irrigation Canals, and the operational decision making process using the decision-making module *Gateman.* Both models are developed by CEMAGREF[5]. Using these models, both the canal water distribution within the main canal and distributaries, as the operational rules at the main canal (operations at the gated cross regulators and secondary off taking regulators) can be studied.

Activities:	*models SIC_{main}, SIC_{disty}, and Gateman (operational rules).*
Input:	*hydrograph at the head of the system (Suleimanki head works): $Q_{Head\ work}(t)$.*
Output:	*discharge at the head of the distributaries $Q(t)$ and discharge head of the water-courses $q(t)$.*

Sub-system **2:** management **of** tertiary units
The canal water distribution within the tertiary unit is under responsibility of the farmers, who take there share of water according to a pre set warabandi schedule (further discussed in chapter 2). The effect of the state of the watercourse and watercourse discharge fluctuations on the canal water distribution to the farms was studied using **an** analytical *hydraulic (volume-balance) model.* The rules determining the water allocation among the farmers are analysed in an *inter-farm water allocation model.*

Activities:	*Volume-Balance Watercourse model and Inter-Farm Water Allocation model.*
Input:	*discharge at the head of the watercourse $q(t)$, quantity and variability.*
Output:	*discharge at the head of a farm $q_{farm}(t)$ quantity and variability.*

Sub-system **3:** the farm
Farmers using the quantity and variability of canal water available to him into account when planning his cropping pattern and input use (fertilizers, labour, pesticides, water, seeds). 11 farm types are identified, to 'deal with the heterogeneity in farmers' strategies. The relations between available canal water supply, use of tubewell water, water trades in the area and intra-farm water allocation **(based** on plot potentials defined in terms of soil chdracteristics and location), are combined and studied using linear programming (LP) models for the different farm types.

Activities:	*Linear programming models and Intra-Farm Water Allocation models for the 11 farm types.*
Input:	*canal water supply at the head of the farm $q_{farm}(t)$, quantity and variability.*
Output:	*cropping pattern, tubewell water use, intra-farm water allocation (water distribution to the fields, both canal water and tubewell water).*

5 ***CEMAGREF, the French research center for agricultural and environmental engineering, division Irrigation based in Montpellier, France.***

Sub-system 4: the field

The use of different ***'types'*** of water with different qualities, i.e. the relatively good quality canal water and tubewell water of marginal quality (saline), are responsible for a complex process underlying the increase of salinity and sodicity in the area of study[6]. To study the physical processes of water uptake of plants and the evolution of salinity, an analytical solute transport model (SWAP93)[7] will be used. The evolution of sodicity will be analysed by an initially developed deterministic approach (SOD).

Activities: ***modelling of salinity (SWAP93) and sodicity (SOD) at field level.***

Input: ***SWAP93: soil hydraulic parameters, canal water supply at the head of the farm q(t), quantity and variability, crop data, rainfall, evaporation, water table depth; SOD: soil type, present salinity (EC-value) and sodicity (SAR-value).***

Output: ***SWAP93: actual and potential transpiration and evaporation, salt volume per layer soil (predicted EC-value); SOD: predicted SAR-value.***

Table 1.1 Summarv of thematic research studies, o tputs and IIMI's collaboration

THEME	COMPONENT	OUTPUT MODEL	IIMI +
Hydraulics	- Main system irrigation management - Objectives / constraints of the PIPD - Maintenance and water distribution at the distributary level	- Integrated main system model (SIC,, + Gate-man) - Distributaries (SIC,,, , simplified)	PIPD Cemagref TUD
Economics	- Analysing farm systems - Farming practices and agricultural production - Intra-farm water allocation - Inter-farm water allocation - farmers decisions	- Farm typology - Production functions - set of rules - LP models - Aggregated watercourse LP models	Cemagref
Salinity	- Main issues related to salinity / sodicity - Analysing of physical processes - Farmers strategies related to salinity	- Hydro-dynamic solute transport model **(SWAP 93)** - Predictive sodicity model (SOD)	**WAU** SSoP CIRAD

6 *Salinity affects the plant growth (by reducing the transpiration) and germination; salinity affects the hydraulic properties of the soil, i.e. reduction in hydrailic conductivity. Both processes caus reduction in yields (Kuper, Strosser et al., 1996). Salinity can be expressed in the Electric Conductivity (EC-value) and sodicity in the so called Sodium Absorption Ratio (SAR-value).*

7 *SWAP93 is developed at the Wageningen Agricultural University (WAU), The Netherlands.*

THEME	COMPONENT	OUTPUT MODEL	IIMI +
Social and institutional context	- History of the irrigation system - Functioning of the PIPD - Factors influencing the water allocation within the tertiary unit - Political economy and irrigation management	- Social and institutional feasibility of management interventions	WAU CNEARC

The **next** figure 1.3 presents a schematic overview of the **Integrated** Approach, as described above.

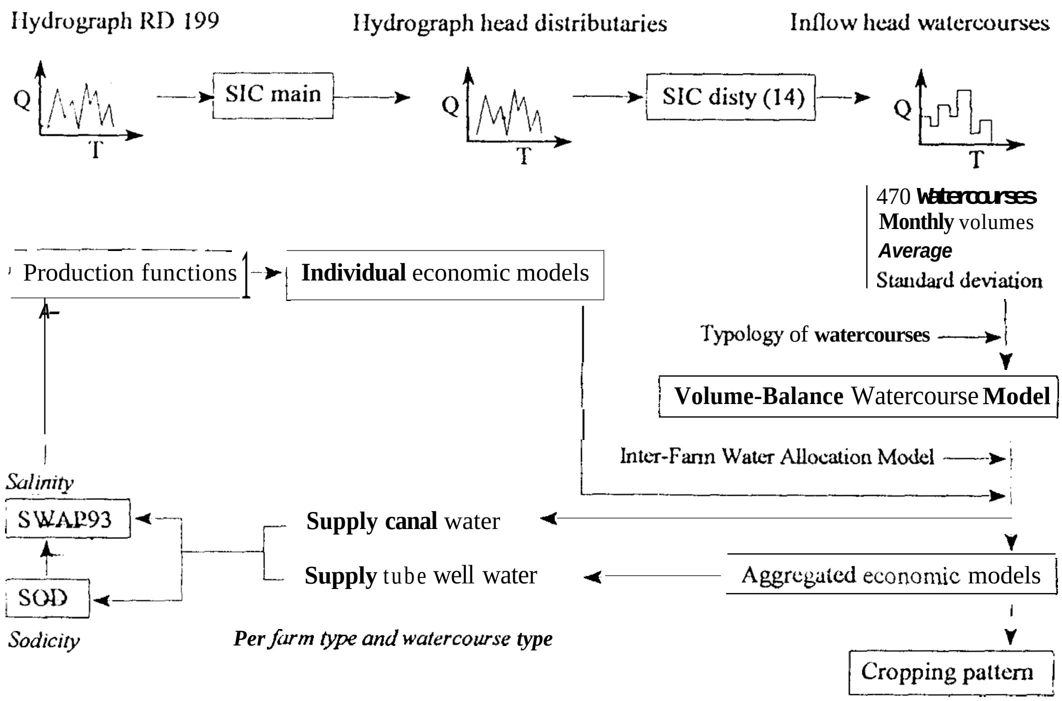

Figure 1.3 **Schematic overview of the Integrated Approach, as studied in the Chishtian Sub-Division, Punjab, Pakistan.**

Based on the presented scheme and the discussed models applied in the approach, different irrigation operation and maintenance scenario's at the main system level will be simulated, in order to evaluate the impact of irrigation practice on salinity, sodicity and agricultural production. For each scenario, a 10 year simulation will be used to be able to recommended different strategies to cope with the initial problems in the area.

Concerning the main system component of the approach, the SIC models of the main canal[8] and distributary canals providing the canal water supply to the **470** watercourses in the area of study. The input at the head of each watercourse to run the simulation loop, i.e. the output of the SIC models for each individual distributary:

- *Monthly volumes in m^3 of canal water* ***supply;***
- *Average volume in m^3* ***for*** *each month;*
- *Standard deviation of the supplied volume for each month (based on day to day supplies).*

1.3 Problem analysis

IIMI is working in the area for almost nine years and most of the studies which have been carried out are focussing mainly on the primary canal level and tertiary farm level. **At** the main canal level several studies focused on operations, while at the tertiary level crop production, agro-economical growth and salinity / sodicity are studied. **At** the secondary level, a study has been carried out to determine possible maintenance strategies to evaluate the effect on canal water distribution to the tertiary outlet structures (Hart, 1995). At the secondary level where *no* control structures exist, interventions are only possible mainly through maintenance activities, i.e. desilting of the canal bed, modifications of outlet and cross structures (drop structures) and cleaning of the canal profile. This study (Hart, 1996) was based on the assumption that the inflow from the main canal (the Fordwah Branch) into the secondary canal (the Fordwah distributary) remains constant at Full Supply Level. At present, the inflow is far from constant, and is varying between almost zero to Full Supply Level, and even more. This has definitely impact on the distribution of different discharges to the tertiary outlet structures.

Initially, canal water distribution at the secondary level in Pakistan is mainly based on the principles of proportionality and equity. At present, the water distribution within the distributary, i.e. the supply of canal water to the tertiary outlet structures, is characterized by a **high variability and inequity.** This study can be seen **as** a part of the Integrated Approach of IIMI, i.e. the investigation of canal water distribution at the secondary level. The canal water supply to the tertiary outlet structures is one of the most important inputs for studies and models within the tertiary unit (sub-system 2, **3** and **4),** as described in section 1.2. Besides that, also the irrigation managers of the Punjab Irrigation and Power Department are interested in the variation of the tertiary canal water supply to evaluate there 'product'.

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[8] *The* ***upstream*** *boundary condition of the SIC model for* ***the*** *main canal* ***is*** *determined by the day to day measured inflow at RD 199, i.e. the Reduced Distance from the head of the system (Suleimanki headworks). 1 RD = 1,000 feet = 304,8 m.*

Thus, the following problem delinition can be formulated:

> ***The canal water distribution within a distributary, to the tertiary outlet structures is depending on many parameters, i.e. canal characteristics, the inflow at the head (Q), state of the distributary and outlet structure characteristics. At the moment (as a part of the Integrated Approach) there is no tool, model or procedure availuble that describes the relationship between those parameters at the secondary level and the impact on the canal water distribution at the tertiary level,***

To link the IIMI studies at the primary level with the studies at the farm level it will be important to develop this tool or procedure. The research is carried out in collaboration with the Punjab Irrigation & Power Department (PIPD), and is part of IIMI research in the area of study.

1.4 Objective and approach of study

This study will focus on the secondary level, addressing the impact of different operational and physical parameters on the water distribution. Some of the outputs of earlier studies at the main canal level will be used for this study: variations of the inflow at the head of the distributary (Q) and impact of maintenance,. methods on the canal water distribution. The main objective of this study following the problem definition, is formulated below:

> ***To develop a tool that predicts the distribution of canal water to the tertiary outlet structures (q) as a function of the inflow (Q), canal characteristics, state of the distributary, outlet structure characteristics und changes (interventions) therein.***

This function can be denoted as follows: $q_i = F(Q, O, C, q_{-1})$, where q stands for the canal water supply **to** outlet structure i, Q stands for the inflow at the secondary level, O the characteristics of the outlet structures and C the canal data. In principle, for analysing canal water distribution and (un)steady flow in irrigation canals, several mathematical hydraulic flow models are available. As described in section 1.2, IIMI is using for this purpose the SIC software[9], so it was most convenient to use this model for this study also. Rased on the existing SIC software, the *tool* that will predict the canal water distribution was developed. From the main objective, two sub-objectives can be formulated:

- Develop a simplified approach to set up a hydraulic flow model (SIC_{disty}), based on a sensitivity analysis of the parameters determining canal water distribution.

In the first stage of this research it was not completely clear which calculation procedure should be used for the development of the tool estimating the canal wafer distribution, i.e. a spreadsheet approach or a mathematical hydraulic flow model (SIC). Based on the necessity of a large database of canal water distribution measurements for a great amount of outlet structures using the spreadsheet approach, and the aim to develop a tool which could be simplified in order to reduce the time spend on field measurements, the mathematical hydraulic flow model SIC was used for this purpose. Besides that, IIMI is working with SIC for several years now and the SIC software and profession is already [here.

- Demonstrate how adjustments (at the distributary level) can be identilied with the hydrodynamic model, which will improve the water management at the secondary level. i.e. a more equitable and proportional water distribution.

The objective leads to the following research questions:

- What kind **of** irrigation outlet structures are used in the Punjab and what are there characteristics determining the canal water distribution.
- What are the main processes and characteristics determining and influencing the distribution of canal water at the secondary level.
- What is the sensitivity of the parameters determining the canal water distribution at the secondary level, i.e. there impact on supply of canal water to the tertiary outlet structures.
- In what way it will be possible to express ***the performance*** **of** a distributary using irrigation indicators, based on the principles of irrigation **in** the area of study.
- Which priorities can be proposed for future development of the research model.

1.5 Methodology of study

1.5.1 Methodology

- **A** literature study must provide some details about the following issues: (1) ***Irrigation in Pakistan in general;*** (2) ***Irrigation in the area of study:*** the Fordwah branch in the Chishtian sub-division; *(3)* ***Previous studies in the area,*** both at the main system level and the secondary level; **(4)** ***The actual physical situation*** of the system, its performance and distribution; *(5)* ***Typical outlet structures*** and characteristics; ***(6) Theoretical principles of the Q/q-variability*** (proportionality, sensitivity, equity) and (7) Characteristics determining the actual canal water distribution at the secondary level.
- To study the parameters determining the canal water distribution, a model of a distributary has been developed. This model is represented by a hydraulic flow model of a **small distributary** modelled in **SIC.** This model has been developed, calibrated and validated based on already existing data and measured data achieved by a field visit. ***The model of the distributary has been calibrated and validated for only one period of the year, i.e. November and December 1995.***
- Use the model of the secondary canal to determine the impact on canal water distribution based on variations of the inflow discharge at the head of the distributary. Determine the minimum and maximum limits in between the inflow **of** the secondary canal fluctuates. based on measurements and physical constraints (no bank overtopping and no dry-tail situation allowed).
- Using the unsteady flow module of **SIC** to conduct a sensitivity analysis: simulate the model for different inflow discharges **at** the head (Q) and evaluate the impact on canal water distribution **(plots** of the **Q-q** relation) based on adjustments of the variable parameters like: inllow pattern. the outlet structure characteristics, canal characteristics and state of the distributary. Analyse the sensitivity of the different parameters and evaluate which parameters have a great impact on the canal water distribution.

- Based on the sensitivity analysis: (1) a simplified approach to **set** up a flow model (a simplified SIC model) has been developed **to** estimate the canal water distribution; and (2) effective adjustments can be proposed to improve water management at the distributary level.
- Calibrate this 'simplified tool' and validate with another distributary.
- The performance of the canal water distribution at the secondary level will be evaluated based on suggested irrigation indicators, which will be tested on a comparison of the actual and design performance of the modelled distributary.
- Hand over the procedure how **to** use this 'tool' for analysing water distribution in the future **to** one of the staff members of IIMI.
- Based on the different simulations, literature study and field practices the conclusions and recommendations are formulated. The research questions are answered and the results are presented to the PIPD and IIMI.

1.5.2 Boundary conditions and constraints

The boundaries of study are defined by the inflow at the head of a distributary and the canal water distribution to the tertiary outlet structures. Besides the boundary condition, the following constraints can be formulated:

- The analysis of the sensitivity and corresponding conclusions are based on computed model output, and not **on** measured processes in the field.
- The study is limited **to** one distributary only.
- Using models to describe physical processes, one must always realize that model output is only an approximation of the reality.
- Constraints of using the SIC software (see section **5.7).**
- The canal water distribution at the secondary level in Pakistan is influenced by social constraints (politics, influential farmers etc.). It is beyond the scope of this report to incorporate this here.

1.6 Structure of the report

In chapter 2 the area of study, The Chishtian Sub-Division, will be discussed more in detail. Chapter 3 is focussing on the theoretical background of proportionality and equity as the main principles of irrigation in the area of study. The typical outlet structures and there design characteristics are described in chapter 4. The development of a flow model of a distributary (field measurements, calibration and validation) is described in chapter **5**. In chapter 6, the methodology for the sensitivity analysis is discussed more in detail. The actual analysis of the parameters determining the canal water distribution is discussed in chapter 7. In chapter 8, the simplified methodology to set up a flow model is discussed and in chapter 9 a method to identify measures for improved water management at the distributary level is treated. Finally, conclusions and recommendations are described in chapter 10.

CHAPTER 2 DESCRIPTION OF THE PROJECT AREA: THE CHISHTIAN SUB-DIVISION

2.1 General description

The command area of the Fordwah-Eastern Sadiqia irrigation project 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. This study is part of the IIMI study conducted at the main system level **of** the Chishtian Sub-Division, part of the Fordwah Division. Figure 2.1 presents a schematic overview of the physical scheme of the Fordwah-Eastern Sadiqia irrigation project.

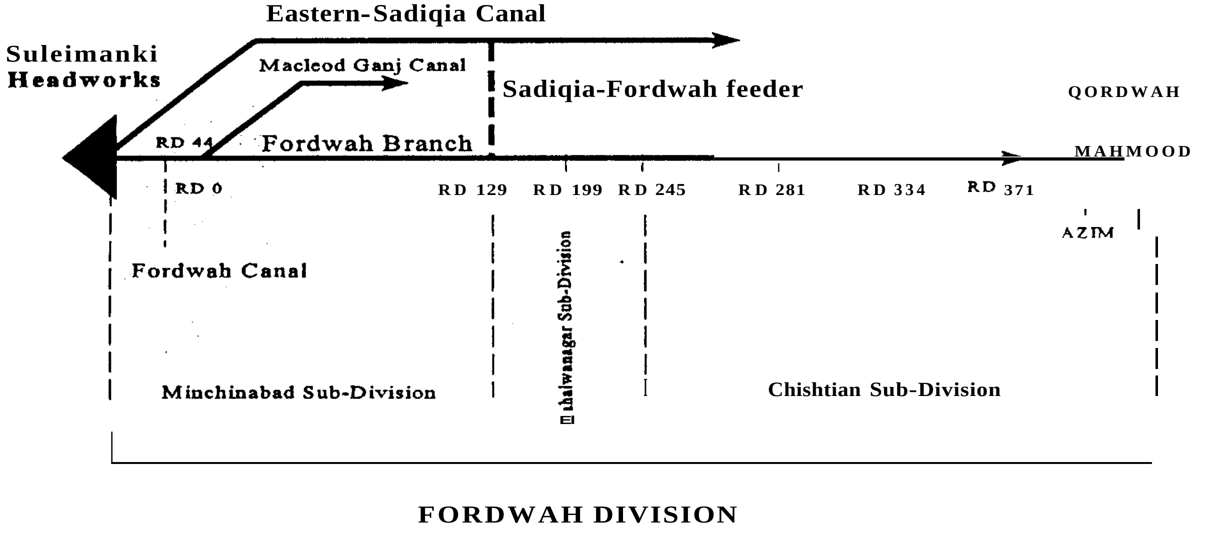


Figure 2.1 **Physical scheme of the Eastern-Sadiqia Irrigation project.**

The system takes off at Suleimanki headwork, a large barrage on the Sutlej River, built in the 1920's by the British. The barrage, with an average width of 600 metres at the head and an average depth of **3** metres. Three primary canals taking off from this barrage: the Fordwah and Easter Sadiqia Primary canals on the left bank, and the Pakpattan Primary Canal on the right bank. Fordwah Canal is diverted in two branches at RD **41.8**: Fordwah Branch and Macleod Ganj Branch (1 RD = 1000 ft = **304.8** metres). The Fordwah Division is divided in three Sub-Divisions: Minchinabad Sub-Division (Fordwah Canal RD 0 to RD 44.8, Fordwah Branch RD 0 to RD 129 and Macleod Ganj Branch), Bahawalnagar Sub-Division (RD 129 to RD 245 of Fordwah Branch) and Chishtian Sub-Division (RD 245 **to** RD 371 **of** Fordwah Branch), see figure 2.1.

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 commended. The climate is semi-arid with annual evaporation (2400mm) far exceeding the annual rainfall (260mm). Most of the rain fall occurs during the Monsoon period, between July and September. The highest temperatures occur during May and June (between 30 C° and 50 C°), and the evaporation rate **is** about 13 mm/day. The cropping pattern is cotton, rice and sugercane in the Kharif season (summer flood season, 15th April to 15th October), and wheat and fodder in the Rabi season (winter season, 15th October to 15th April). This area is part of the Sutlej Valley Project undertaken in 1920's and completed in 1932. Both Fordwah and Eastern Sadiqia canals receive their supply from link canals since partition, as most part off the water **from** the Sutlej River is used by India. In Kharif the supplies are diverted from the Chenab river and conveyed through so called Link canals or feeder canals to the Sutlej River. In Rabi the water comes from the Mangla Dam. Because supply in the winter season is very limited, irrigation canals are divided in perennial and non-perennial canals. Perennial canals receive there water the entire year, while non-perennial canals receive water only during the Kharif season.

2.2 **Fordwah Branch** in **the Chishtian** Sub-Division

Figure 2.2 presents a schematic overview of the Chishtian Sub-Division. The Fordwah Branch has a total length of 123 km, 38.4 km is located in the Chistian Sub-Division (from RD **245** to **RD 371).** The design discharge at RD 199, the hand-over point of the Chistian Sub-Division is 30.3 m^3/s (1282 cfs). The average slope is 0.020% (1/5000). The Gross Command Area (GCA) of the Sub-Division is **74,369 ha,** the Culturable Command Area is approximately **67,000** ha. The Fordwah Branch is as most irrigation systems in Pakistan, a system under upstream control. The Fordwah Branch has six control points, i.e. cross regulators within the Chistian Sub-Division, with distributaries off taking just upstream of five of these regulators. The remaining cross regulator at RD 353 controls the water level in the Iordwah Branch itself with only a very small distributary (5-L) off taking upstream **of** this point. The cross regulators are operated manually. Gauge Readers observe the water levels twice daily at all these Control points and the data are transmitted through signallers to the irrigation officers in charge to take decisions regarding the operation of the irrigation in their (sub)division (Kuper and Kijne, 1992). Discharges are determined by measuring the down stream water level at the structures. With a simplified stage-discharge relationship the discharge downstream of the structure is determined.

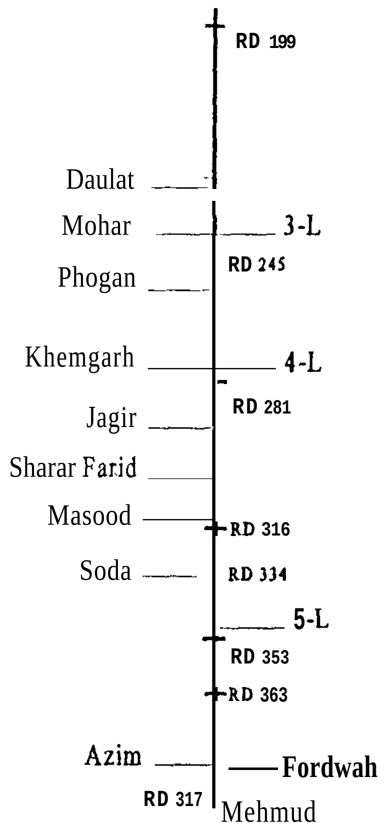


Figure 2.2 **Layout of the Fordwah Branch**

The canal water is delivered to the distributaries through 14 off **take** structures (gates, culverts or open flumes), and to 19 direct outlet structures.

Increased cropping intensities and the development of previously unirrigated area have resulted in the perception that the at present, canal water supplies are insufficient to feed all distributaries at the same time. At the main system, a complex system of rotations has been installed to spread the shortages of canal water in Rabi season. Priorities are given to the Sub-Divisions for certain periods of time. Within the Sub-Divisions, the distributaries are operated on an *on/off basis*. Out of the **14** distributaries, **9** are non-perennial and **5** are perennial. Besides due to shortage of canal water supply during Rabi, also 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).** Besides surface canal water supply, increasing tubewell irrigation supply is used to meet the crop requirements. Almost all tube wells are in private use, **owned** by big farmers, or a group of farmers.

Table 2.1 Physical characteristics of the distributaries

DISTRIBUTARY	STATUS	Q DESIGN (CFS)	LENGTH (FT)	CCA (acres)	NUMBER OF OUTLETS
3-L	NP	18	23,100	2,970	6
Mohar (incl. Hussainabad minor)	NP	38	20,240	5,185	15
Daulat (incl Biluka and Nakewah minor)	NP	209	115,150	40,920	108
Phogan	NP	17.5	8,750	2,210	9
4-L	NP	14	17,350	2,053	7
Khemgarh	NP	24	15,500	5,053	9
Jagir	P'	28	13,830	4,704	9
Shahar Farid (incl. Heerwah minor)	NP	153	74,880	31,550	74
Masood	P	35	52,300	8,099	16
Soda	NP	77	43,700	10,113	33
5-L	P	4	11,300	884	3
Fordwah (incl. Jiwan minor>	P	158	139,780	43,768	109
Mehmud	P	8.25	11,860	20,066	7
Azim (incl. Rathi, Feroze and Forest minor)	NP	244	118,000	33,810	94

NP = Non Perennial; P = Perennial

source: IIMI, 1996

Figure **2.4** on page **17**, presents the whole Chisthian Sub-Division with the Distributary Command Areas (GIS, April **1996).**

The operational irrigation objectives, no different from the rest of the Indus Basin Irrigation System, are to distribute canal water within an area as large as feasible and as equitable as possible (Bandaragoda and Firdousi, 1992). The fanners share the canal water supply among themselves through a flexible roster of turns called ***kacha warabandi***[1]. This system of warabandi was agreed upon by the farmers themselves, with the PIPD interfering only when a dispute arose. About 20 to 30 years ago, due to increased disputes about canal water supply at the watercourse level, the PIPD intervene in the most of the watercourses and fixed an official roster of water turns, ***pacca wurubandi (pacca = official).***

2.3 Institutional context

There are different levels of management units in the Punjab Irrigation System. The ***Zone*** is the largest unit, and a Chief Engineer is in charge. The ***Circle*** is the next unit, under responsibility of a Superintending Engineer (SE). In general, a Circle is divide in different ***Divisions,*** which are the basic irrigation units, headed by an Executive Engineer (XEN). The Divisions are divided in several ***Sub-Divisions,*** under responsibility of an Assistant Executive Engineer, also called a Sub-Divisional Officer (SDO). The Sub-Division itself is divided into different ***Sections,*** each of them headed by a Sub-Engineer. Figure 2.3 presents the organizational setup of the Bahawalnagar Circle.

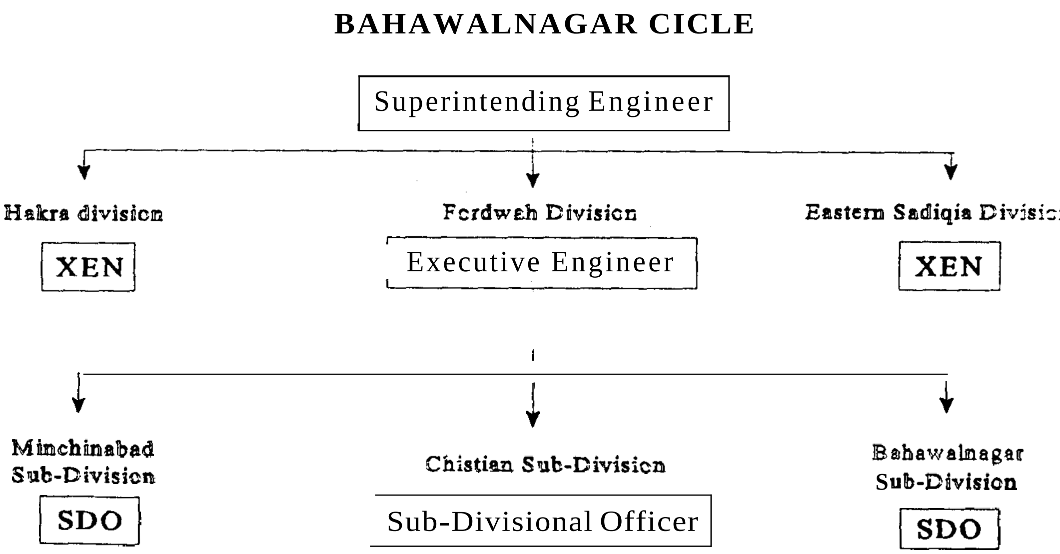
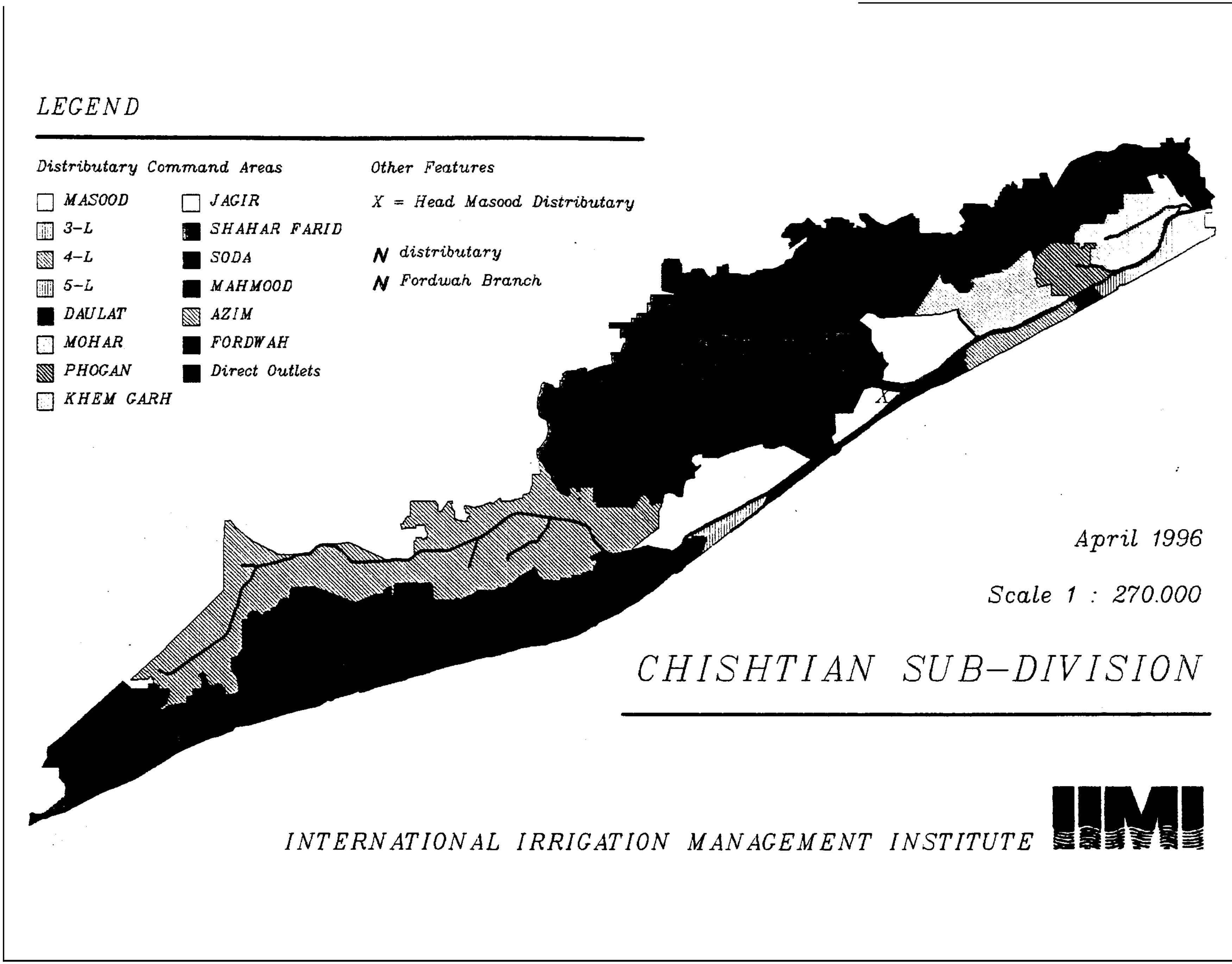


Figure 2.3 Organizational Setup of the Bahawalnagar Circle (source: Litrico, 1995)

1 *kacha = informal; wahr = turn, and bandi = fixed*


LEGEND
Distributary Command Areas
MASOOD
3-L
4-L
5-L
DAULAT
MOHAR
PHOGAN
KHEM GARH
JAGIR
SHAHAR FARID
SODA
MAHMOOD
AZIM
FORDWAH
Direct Outlets
Other Features
X = Head Masood Distributary
N distributary
N Fordwah Branch
X
April 1996
Scale 1 : 270.000
CHISHTIAN SUB-DIVISION
INTERNATIONAL IRRIGATION MANAGEMENT INSTITUTE

CHAPTER 3 WATER MANAGEMENT AT THE DISTRIBUTARY LEVEL

3.1 General

This chapter is about water management at the distributary level in the Punjab. As this study is focussing on canal water distribution at the distributary level, the boundaries are defined by a typical inflow pattern at the head of the distributary and downstream, the distribution of canal water to the tail cluster outlet structures. First, the principles applied for irrigation in Pakistan, as described in section 1.1, will be discussed more in detail. The aim of this chapter is to obtain insight into the theoretical backgrounds of canal water distribution at the distributary level

3.2 Theoretical concept

3.2.1 Flow in a distributary

The Punjab irrigation system was originally designed with a minimum of adjustable control structures. In general, in the area of study, there are no adjustable control structures available at the distributary level. Below the gated head structure of the distributaries, water is distributed by means of fixed tertiary outlet structures, either an orifice, pipe, or an open flume. Actually, canal maintenance and outlet structure modifications are the only 'tools' available to manipulate the existing distribution pattern. So, the amount of water delivered at the head of a distributary, is distributed to all the outlet structures and minor canals along the distributary. There are always seepage losses in the canal, and in general the sum of the distributed discharges to the outlet structures plus the seepage losses must be equal to the incoming discharge at the head of the distributary:

$$Q_{head} = \sum_{i=1}^{n} q_i - S_e$$

Where:

Q_{head}	=	Discharge at the head of the distributary	[m³/s]
q_i	=	Discharge through an outlet structure	[m³/s]
S_e	=	Seepage	[l/s/km]
n	=	Number of outlet structures	[-]

3.2.2 Equitability

Within a distributary the distribution of canal water to the tertiary outlet structures is based on the principle of equity. Equity of water distribution can be defined as a distribution of a fair share of water to users throughout the system (Kuper and Kijne, 1993). The equity principles, as it is used by the PIPD to adjust operational strategies, were originally established as a design parameter for the Punjab canal system. A discharge has to be made available at the head of each ***mogha*** (tertiary outlet structure) for its command area based upon a preset 'duty' or water allowance per unit area (Bhutta, Vander Velde, 1988). The distribution of canal water based on the irrigated area served for each outlet structure has become the prior objective of irrigation in Pakistan. The duty is expressed in a design quantity of water (csf) per 1000 acres of culturable command area (CCA), i.e. the physical irrigable agricultural area commanded by the outlet structure. Besides that it was envisaged that the actual area irrigated by farmers would not exceed 50% to 75 % of the CCA. The discharge for an outlet structure is, therefore, directly related with the area served: the discharge is called the ***authorized discharge (q_{auth})***.

3.2.3 Sensitivity

Theoretical background

Besides an equitable distribution of the canal water supply, the water distribution at the distributary level is also based on proportional control, i.e a flow control method in which the flow is divided into a fixed ratio. Besides proportionality for a steady flow, also disturbances will be proportionally distributed: an increase in discharge at the head of a distributary of approximately 10% will result in an increase of allocated discharge to each individual outlet structure of 10%. The distribution of a disturbance along the canal can be expressed with the so-called ***Sensitivity Ratio S***. The sensitivity ratio S is defined as the variation in an off taking discharge in response to a change in the continuing discharge in the parent canal. The concept of sensitivity is the best basis for evaluation of the performance of a bifurcation under varying discharges. The bifurcation can be without any structure, a free off take in the off taking canal, or with a division structure in the parent canal (Ankum, 1995). The basic equations for flow through the ongoing canal (Q) and off taking outlet structure (q) are:

$$Q = \beta . H^{u}_{c} \quad and \quad q = \alpha . H^{n}_{w}$$

With the assumption that a change in water level in the distributary (dH_c) will lead to an equal change in head over the crest of the outlet structure (dH_w), the sensitivity of an outlet structure can be expressed as follows (clarified in figure 3.1):

$$S = \frac{\frac{dq}{q}}{\frac{dQ}{Q}} = \frac{\frac{n.\alpha.H^{n-1}_{w}}{\alpha.H^{n}_{w}}}{\frac{u.\beta.H^{u-1}_{c}}{\beta.H^{u}_{c}}} = \frac{n \, . \, H_c}{u \, . \, H_w}$$

Where:

S	=	Sensitivity factor	[-]
q	=	Distributed discharge to outlet structure	[m^3/s]
dq	=	Change in distributed discharge to outlet structure	[m^3/s]
Q	=	Discharge parent canal, distributary	[m^3/s]
dQ	=	Change in discharge parent canal, distributary	[m^3/s]
α	=	Depth-discharge coefficient outlet structure	[$m^{1.5}/s$ (weir flow); $m^{2.5}/s$ (orifice flow)]
β	=	Depth-discharge coefficient distributary	[$m^{4/3}/s$]
H_w	=	Head over the outlet structure (above the crest)	[m]
H_c	=	Water level in the canal	[m]
n	=	0.5 for orifices, 1.5 for weirs	[-]
u	=	5/3 (see page 23)	[-]

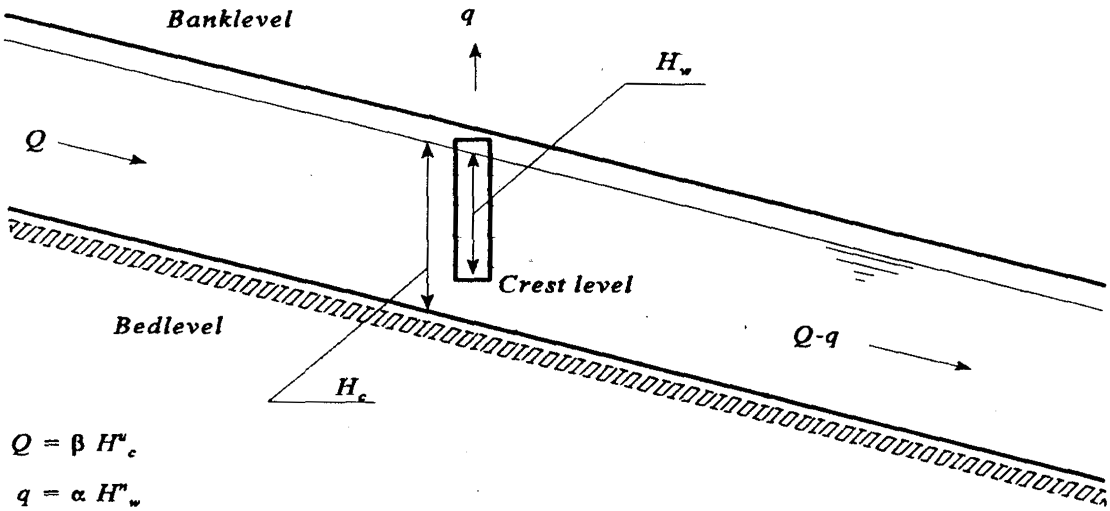


Figure 3.1 Longitudinal profile of a canal with outlet structure.

Four situations can be distinguished, analysing the sensitivity ratio for a bifurcation of a distributary, i.e. a tertiary outlet structure **(Ankum, 1993;** Essen, van der Feltz, **1992):**

s – 0

No sensitivity of the outlet structure discharge to changes in the discharge in the distributary. *Any* variation will be distributed to the tail-end of the system. Either flooding or severe water shortage at the tail due to failure in the supply.

S < 1

Sub-proportional distribution of a disturbance, i.e a low sensitivity of the outlet structure to changes in the discharge in the distributary. The change in the distributed discharge to the outlet structure is less then the change in discharge in the parent canal, The discharge fluctuations are distributed mainly to the tail of the system.

S = 1

Fully proportional distribution of a disturbance, i.e the change in the distributed discharge to the outlet structure is equal to the change in discharge in the parent canal.

S > 1

Super-proportional distribution of a disturbance, i.e a high sensitivity of the outlet structure to changes in the discharge in the distributary. The change in the distributed discharge to the outlet structure is higher then the change in discharge in the parent canal. The variations in the head of a distributary are distributed to the head reach outlet structures.

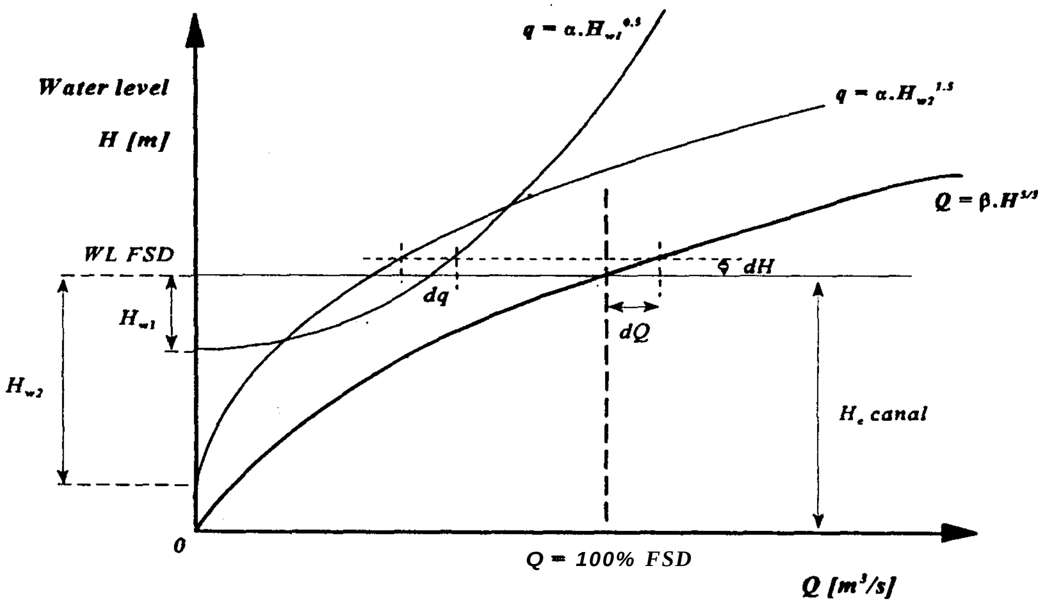


Figure 3.2 **Theoretical analysis proportionality for design of outlet structures.**

The design of outlet structures is based on proportionality, i.e. S = 1. The settings of the outlet structures are related with this design concept. In general, for fixed outlet structures and no control structures in the distributary, a sensitivity ratio of 1 can be obtained in one particular point only (n and u are not similar). For 100% FSD at the head of a distributary, there is only one combination of ongoing discharge (water level) in the parent canal and allocated discharge to an outlet structure: $S = 1$: $dq/q = dQ/Q$. This will be demonstrated in figure 3.2.

Where:

q	=	Distributed discharge to an outlet structure	[m³/s]
		n = 0.5 for orifice flow and n = 1.5 for weir flow.	
Q	=	Discharge distributary	[m³/s]
dH	=	Change in water level in the distributary	[m]
$H_{w,1}$	=	Head over an orifice type outlet structure	[m]
$H_{w,2}$	=	Head over a weir type outlet structure	[m]

For a certain change in water level in the canal (dH), there will be a change in discharge for the ongoing canal (dQ) and distributed discharge to the outlet structure (dq). Only for fixed $H_{w,1}$ and $H_{w,2}$ there will be fully proportional behaviour for orifice and weir flow outlet structures. By changing the settings of the crest level of the weir type outlet structures, and the elevation of the roof block for orifice type outlet structures, this can be obtained in one point only. Whenever n and u are similar, i.e. two weirs or two orifices, there will be always proportional behaviour for different water levels.

<u>Design settings</u>
So, in order to accomplish proportional behaviour, the discharge-depth relationship of the distributary must be related to the discharge-depth relationship of the outlet structure (Hart, 1996), i.e. S = 1. With the discharge-depth relationship of the distributary expressed by the Manning-Stickler equation, **based** on the assumptions that: (1) the hydraulic radius equals the depth (infinite width); and (2) the wetted perimeter is linear with the depth (rectangular cross sections):

$$Q = k.B.H_c.H^{2/3}{}_c.i^{1/2} \Rightarrow Q = \beta.H^{5/3}{}_c$$

$$\frac{dQ}{Q} = \frac{5}{3}.\frac{dH_c}{H_c}$$

Where:

Q	=	Discharge distributary	[m³/s]
k	=	Roughness coefficient (Strickler)	[m^1/3/s]
B	=	Width of the canal	[m]
i	=	Bed slope of the canal	[-]
H_c	=	Water level in the canal	[m]

In general, the outlet structure equations for orifice and weir flow can be simplified as:

$$q = \alpha . H^{n}_{w}$$

$$\frac{dq}{q} = n.\frac{dH_w}{H_w}$$

Where:

q	=	Discharge outlet structure	[m³/s]
H_w	=	Head over the outlet	[m]

As the change of water level in the canal equals the change of head over the crest, i.e. $dH_c = dH_w$, the sensitivity factor equals 1 leads to:

$$S = \frac{\frac{dq}{q}}{\frac{dQ}{Q}} = \frac{n.\frac{dH_w}{H_w}}{\frac{5.dH_c}{3.H_c}} = 1$$

$$H_w = \frac{3.n.H_c}{5}$$

Where n is defined by the type of outlet structure. For weirs n = **1.5**, for **orifices** n = 0.5.

- **Weir flow** : $H_w = H_{w,1} = 9/10.H_c$
- **Orifice flow** : $H_w = H_{w,2} = 3/10.H_c$

Practically speaking, for weir flow the crest of the open flume should be placed at 1/10 of the depth (100% FSD) above bed level of the distributary. For orifice flow, the roof block should be placed at 0.7 of the depth (100% FSD) above bed level (Hart, 1996; Ali, 1993; Mahbub and Gulhati, 1951).

For pipe outlet structures, with the canal is running on 100% FSD, the head over the structure should be 0.3*FSD in the canal. With the crest of the pipe at bed level, to ensure maximum silt draw, the downstream water level, i.e. the water level in the watercourse, should be approximately at 0.3*FSD below FSD in the canal.

By changing the width and height of the opening, the authorized design discharge will be obtained. In figure 3.3, the design concepts of canal water distribution at the secondary level are presented. Whenever the distributary is running at 100% FSD, the supplied discharge to the outlet structure is equal to its authorized discharge, and with the settings the requirement S = 1 will be met. To speak with the words of Kennedy, a distributary should be designed in such a way that *'at each point it will just carry as its full supply a discharge sufficient to supply all the outlets below that point, so that when the proper quantity enters the head all watercourses should just run their calculated allowances with no surplus at the tail of the distributary".*

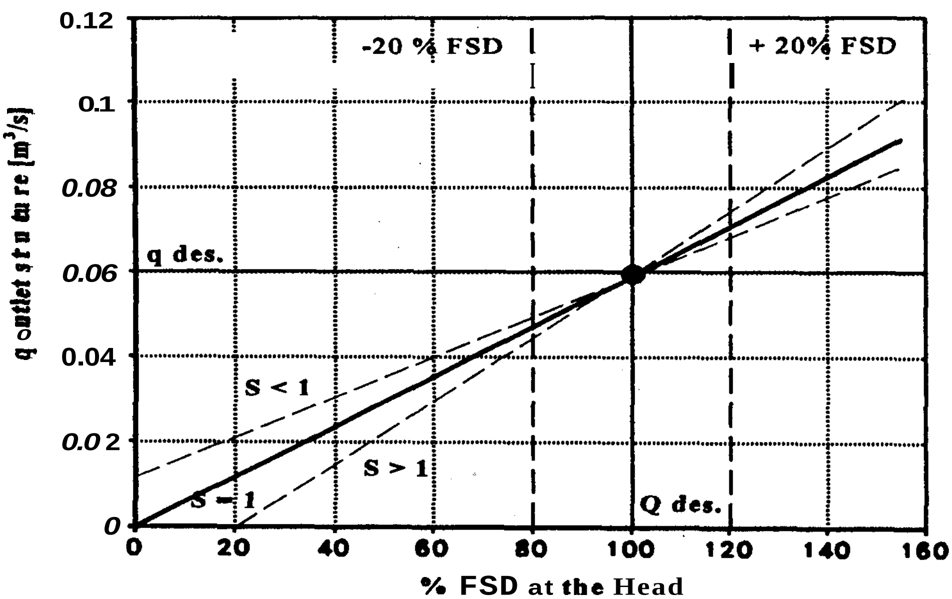


Figure 3.3 **Design principles proportionality and equity for an outlet structure.**

CHAPTER 4 OUTLET STRUCTURES IN THE PUNJAB

4.1 General

There is probably no physical element within an irrigation system which has a greater impact on canal water distribution then outlet structures. Therefore, to study distribution, it is necessary to have a good knowledge of the different types and hydraulic behaviour of outlet structures, as they exist at present in the Punjab irrigation system. In this chapter many details are based on the study on irrigation outlet structures in the Punjab by Mahbub, revised and enlarged by Gulhati, published in 1951. They define an irrigation outlet structure as ***a device at the head of a watercourse off taking either from a distributary or direct from a main canal or branch.*** In general, the distribution of canal water within a tertiary unit or chak, is managed by the beneficiaries them self. Each tertiary unit consist of different plots and is served with canal water through ***one*** outlet structure, supplying the water to the watercourse. ***An*** outlet structure defines the point of contact between the beneficiaries and the higher authorities, i.e. the farmers and the PIPD, and it is therefore the most sensitive part of an irrigation system.

As stated in chapter 3, in large parts of the Punjab, control of the water flow within a distributary is based on canal maintenance and remodelling of outlet structures. After any adjustment, it takes a considerable amount of time before the canal is in its final regime again. To maximize the impact of remodelling outlet structures by the water manager a good understanding of the hydraulic characteristics and there impact on the water distribution will be necessary. This chapter is focussing on the different types of outlet structures that *can* be distinguished in the Punjab. First, the factors determine the design of an outlet structure will be discussed; secondly, the different types of outlet structures will be analysed and finally, the different characteristics determining the canal water distribution will be listed.

4.2 Factors determining the design of an outlet structure

There are several factors having an impact on the design of an outlet structure. They are summarized and discussed below.

O jectives of irrigation

As stated in chapter 2, irrigation in Pakistan is based on the principles of equity and proportionality. In section **4.4.6,** the theoretical design criteria for outlet structures based on the principles of irrigation will be discussed.

Optimum capacity

The optimal discharge through an outlet structure is based on: (1) the amount of water that can be handled efficiently by one farmer and (2) the minimal absorption losses in the watercourse and on the fields. In general, the optimum discharge efficiently used by one farmer is called the 'maind'eau', and is in between 25 to 55 l/s. Studies in the Punjab found that an amount of about 2 cfs[1] (= 56 l/s) is generally the best for cultivating 0.5 acres of irrigated land. Briefly, the optimum discharge through an outlet in cfs, should be 5 times the area in acres to be irrigated (Malhotra, Mahbub, 1951).

Based on that, a classification of outlet structures can be distinguished.

Table 4.1 Classification of outlet structures

Characteristic	Discharge (cfs)	Area (acres)
small outlet	< 0.50	< 0.15
	0.50 - 1.00	0.15 - 0.31
Average outlet	1.00 - 1.50	0.31 - 0.46
	1.50 - 2.00	0.46 - 0.61
Large outlet	2.00 - 4.00	0.61 - 1.23
	> 4.00	> 1.23

source: Mahbub and Gulhati, 1951

Silt drawing capacity

Canal water in the Punjab is heavily loaded with suspended silt, which deposits when the silt carrying capacity of the flow decreases. To avoid sever siltation along the canal the silt load must be equitable distributed to all the distributaries, and within a distributary to all the watercourses. So each outlet structure must take its fair share of silt, which has its impact on the design. The essential geometric features of outlet structures determine the silt drawing capacity are summarized below (Mohammed Hasnein Khan, 1996). It is beyond the scope of this research to discuss these concepts more in detail.

- Position of the inlet structure (wings upstream and downstream) must be so designed that the whole mass of water moves towards the outlet structure, with an approach velocity close to the average velocity of flow in the canal.
- The roof block of orifice-type outlet structures should be as close as possible to the crest, to assure high velocities within the outlet and to increase the silt draw.
- The silt conducting power of an outlet structure is increasing with low settings of the crest, due to intensified silt transport at bed level of the distributary.

[1] *1 cfs = 1 cubic feet per second = 28.31 l/s*

To obtain equitable distribution of silt along all watercourses and due to seepage losses of approximately 10% to 15% (of the inflow) in distributaries, the silt drawing capacity should be at least 110% to 115% to enable them to draw their fair proportional share, compared with the Carrying capacity of the distributary (100%).

Other essential factors

Besides the three major factors determine the design of an outlet structure, described above, there are some other factors, summarized here:

- Outlet structures must be strong and equipped with minimum adjusted and movable parts to avoid expensive maintenance and illegal modifications, i.e. tampering of outlet structures.
- The outlet structure should be functioning with a minimum of working head.
- The costs for design should be as low as possible.

Besides the classification of outlet structures based on a quantitative analysis, a different classification can be distinguished based on flow condition. Outlet structures may be divided into three different classes (Mahbub and Gulhati, 1951; Ankum, 1993):

Modular outlet structures are those outlet structures which discharge is independent from both the upstream water levels in the distributary as the downstream water levels in the watercourse (in between reasonable limits).
Semi-modular outlet structures are those outlet structures which discharge is dependent on the upstream water levels in the distributary, but independent of the downstream water levels in the watercourse, as long as the required working head is available.
Non-modular outlet structures are those outlet structures which discharge is both depending on the upstream water levels in the distributary, as the downstream water levels in the watercourse.

4.3 Hydraulic principles of different types of outlet structures in the Punjab

4.3.1 Types of flow

The two most significant flow conditions are *free flow* (critical depth flow or (semi-)modular flow) and *submerged flow* (drowned flow or non-modular flow). The distinguishing difference between free flow and submerged flow is the occurrence of critical velocity, so the discharge through any constriction is only determined by the depth of head just upstream of the critical section (Skogerboe, 1992). If the difference between upstream water level and downstream water level is decreasing, consequently the velocity becomes less then the critical velocity within the constriction and submergence occurs. The value of the submergence ratio S_f describes the change from free flow to submerged flow; $S_f = h_d / h_u$, also known as the minimum modular head. Free flow and submerged flow are the two major flow types.

$$Q_{ff} = f(h_u)$$

Where:

Q_{ff}	=	Free flow discharge	[m³/s]
Q_{sf}	=	Submerged flow discharge	[m³/s]
h_u	=	Upstream water level above crest	[m]
h_d	=	Downstream water level above crest	[m]
S_f	=	Submergence ratio (= h_d/h_u)	[-]

In between free flow and submerged flow, a few other possible flow conditions can be distinguished, based on a change in S_f. Flow through outlet structures can be discussed based on the possible flow conditions for fixed structures. There are 5 different types of flow that can be distinguished through a fixed outlet structure (Ankum, *1995).* The different types are clarified in figure 4.1 and are discussed for the different types of outlet structures present in the area of study.

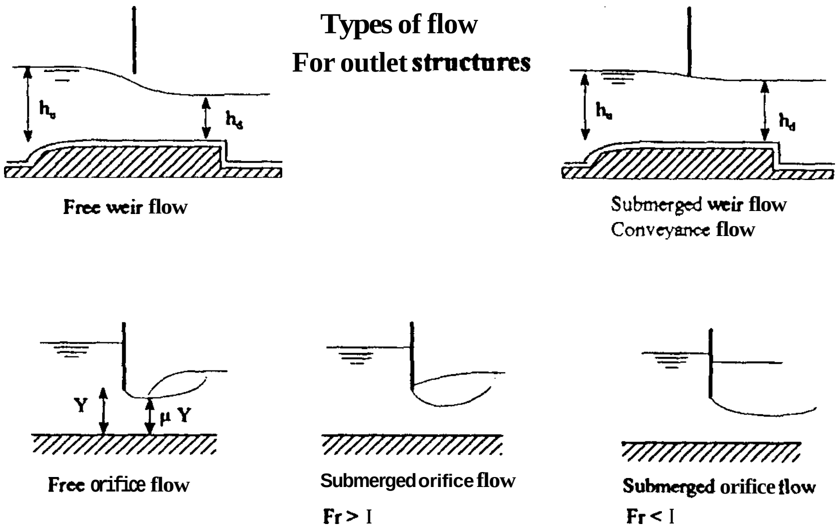


Figure 4.1 **Types of flow condition for weirs and orifice flow.**

4.3.2 Open Flume

General

The design of the open flume outlet structure is based on the ideas 'of the Stoddard-Harvey improved irrigation outlet, whereby the size of the weir has been changed to a long throated flume. The open flume outlet structures are semi-modular as long as the velocity within the throat is above the critical velocity, and the length of the flume should be long enough to ensure strait stream lines above the crest.

In general, the structure is built in brick masonry, provided with an iron frame and steel bed to avoid tampering. The earlier types of outlets structures developed in the Punjab, i.e. the Kennedy's sill outlet, the Kennedy's gauge outlet, the Harvey outlet and the Harvey-Stoddard irrigation outlet have been modified due to there sensitivity of tampering and due to improved designs (FAO, 1982). At present the open flume outlet structures in the Punjab are Crump's type open flume and Jamrao type open flume outlet structures. The length of the throat should be equal to 2.5 times the upstream water level above the crest, with the canal running on FSD. Open flume outlet structures are recommended for use within 300 m upstream of control points, or at tail clusters (FAO, 1982). At the tail, it is useful to distribute the supply proportionally among the watercourses, and to easily absorb an excess of water.

Discharge equation

The discharge through an open flume outlet structure is determined by the free flow weir discharge equation. The depth of water above the crest does not touch the roof block and the downstream water level is sufficient low in order to establish free flow conditions, i.e. the gate opening $Y > {}^2/_3\, h_u$ and in general the downstream water level $h_d < {}^2/_3\, h_u$, or $S_f < 0.67$[2]. The discharge over a weir is determined by the discharge equation:

$$q = C_1 . 1.7 . B . H_u^{\frac{3}{2}}$$

Where:

q	=	Discharge over the weir	[m^3/s]
C_1	=	Discharge coefficient for a weir	[$m^{1/2}/s$]
B	=	Width of the crest	[m]
H_u	=	Upstream energy head above the weir	[m]

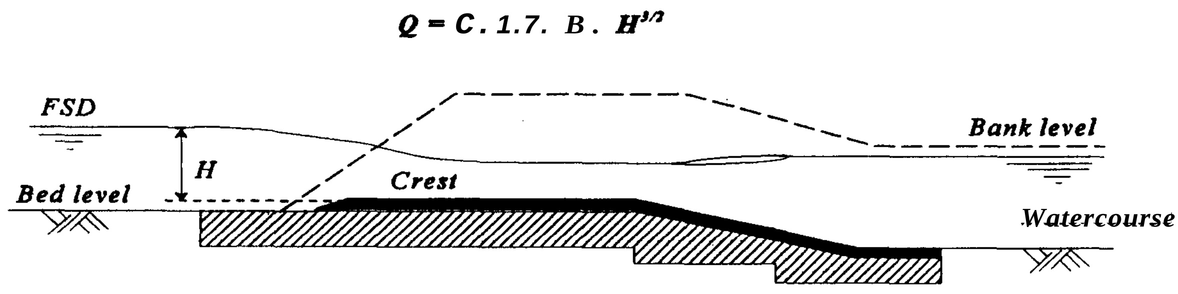


Figure 4.2 **Broad crested weir (Open Flume).**

2

Actually, experimental study by D. G. Romijn proved that even with $h_d = 5/6\, h_u$ ($S_f = 0.83$). and for a high value of h_u/L (L = length of crest): $h_u/L > 0.75$: $C_1 = 1$, so there is still free weir flow.

The coefficient of discharge (C_1) is influenced by several factors like: the side contractions, the shape of the crest, the length of the crest and the head H_u. The difference of a short crested and a broad crested weir is depending on the existence of curved and parallel stream lines above the crest.

The discharge coefficient for a broad-crested weir $C_1 = 1\ m^{1/2}/s$ (theoretical value, in reality it will be approximately $0.95\ m^{1/2}/s$) and for a short-crested weir $> 1\ m^{1/2}/s$.

Table 4.2 Relation between coefficient of discharge and width of an open flume

B (cm)	C_1
6.0 - 9.0	0.94
9.1 - 12.0	0.96
> 12.0	0.98

source: FAO, 1982.

Silt drawing capacity

The higher the crest level of the structure compared with the bed level of the canal, the less its silt drawing capacity. In practice, the width of the throat of the open flume is limited to a minimum of 6 cm, and therefor it becomes necessary to raise the crest of the outlet above bed level and decrease the silt draw.

Submerged weir flow or conveyance flow

The depth of water above the crest does not touch the gate and the downstream water level is as high, so the flow is submerged, i.e. gate opening $Y > H_u$ and downstream water level in general $h_d > {}^2/_3\ h_u$, or $h_d >$ 5/6 h_u for a high ratio h_u / L (L = length of the crest): h_u / L > 0.75. The flow through such a structure is fully submerged, with a head loss in these structures determined by: $z = [\alpha_{in} + \alpha_{out}]v^2/2g$, with entrance head losses α_{in} (approximately 1/3) and exit head losses α_{out} (approximately 2/3).

4.3.3 Open Flume with Roof Block (OFRB)

General

The main disadvantage of the open flume is its sensitivity for illegal blocking when the opening is deep and narrow, and its super-proportional behaviour when the opening is shallow and wide. Besides that, it fails to draw its fair share of silt. Another disadvantage is the increase of discharge through the outlet structure because of a rise in upstream water table, due to siltation. To overcome these negative effects, the PIPD started to place roof blocks above the crest. At present the Open Flume with Roof Block (OFRB) outlet structures are dominant in the area. The roof block is fitted just above the vena contracta of the water flowing over the crest of the open flume at FSD (see figure 4.3). The open flume starts to function as an orifice whenever the upstream water level is raising, which results in a decrease in discharge. The following rules have been approved in the eastern part of the Punjab (formal Bahawalpur State) for the use of roof blocks in open flumes (Mahbub, Gulhati, 1951):

- The roof block should be fixed at a distance equal to h_u (With h_u the upstream water level above crest at FSD) below the upstream end of the throat (length of the throat: **2.5 - 3.0 h_u**).
- The bottom of the roof block should be at a height of **0.75** h_u above the crest.
- The **roof** block should have a square edge at the bottom (to create contraction with a raise in upstream water level).

Discharee **equation**

The discharge through an OFRB outlet structure is determined by either the free flow weir discharge equation when the roof block is out of the water. **As** soon **as** the upstream water level rises, the discharge equation changes to the general equation for orifice flow. The downstream water level is not influencing the discharge through the structure. The hydraulic jump is formed at some distance from the gate. The discharge equation for free orifice flow *can* be given **as (Ankum, 1993):**

$$q = C_d \sqrt{2g}.B.Y.\sqrt{H_u - \alpha.Y}$$

Where:

q	=	Discharge	[m³/s]
C_d	=	Discharge coefficient	[-]
g	=	**9.8 m/s²** (gravity acceleration)	
B	=	Width of the opening	[m]
Y	=	Height of the opening	[m]
H_u	=	Energy head (= approximately the upstream water depth) above the crest	[m]
a	=	Contraction coefficient of the jet (approximately **0.6**)	[-]

The above depth-discharge relation *can* also be expressed in a more general discharge equation (Henderson, **1966):**

$$q = C_d.B.Y.\sqrt{2.g.H_u}$$

The coefficient of discharge is influenced by several factors like: the diameter of the orifice, the shape of the orifice (determining the contraction coefficient a, the head H_u and the degree of turbulence of water approaching. The equation for the discharge coefficient reads:

$$C_d = \frac{\alpha}{\sqrt{1 + \alpha.\frac{Y}{H_u}}}$$

The discharge coefficient ranges for ***free*** orifice flow between **0.5** and **0.6.**

The outlet structure is designed to function **as an** open flume, but due to siltation, i.e. increase in bed level elevation, the water levels at FSD are higher then design water levels, and therefor in most cases the OFRB outlet structures are functioning **as** an orifice.

Partially-submerged underflow (Fr > 1)
The flow is super-critical and the hydraulic jump just touches the gate. The downstream water level is influencing the discharge trough the structure.

Fully-submerged underflow (Fr < 1)
The flow is sub-critical, the structure is completely drowned by the high depth of the downstream water level. When an orifice is submerged, also the downstream water level **determines** the discharge and the discharge equations becomes:

$$q = C_d.B.Y.\sqrt{2.g.(H_u - H_d)}$$

Where:

H_u	=	Upstream water level (measured from the crest)	[m]
H_d	=	Downstream water level (measured from the crest)	[m]

4.3.4 Adjustable Orifice Semi-Module **(AOSM)**

General
Adjustable orifice semi-module outlet structures **(AOSM),** or the early Adjustable Proportional Module **(APM)** presented by Crump in 1922, are widely used in the Punjab **(Pakistan** and India). To ensure full proportionality, Crump's design was originally based on fitting the crest at 0.6*FSD and the bottom of the roof block at 0.3*FSD (measured from FSD water level). After installing these APM's, problems occur due to limited silt draw and a bad siltation of the canals. The silt drawing capacity **was** to low and other types were developed. At present, all APM's are removed and replaced by AOSM outlet structures, which are not fully proportional due to lower crest settings, but ensure **a** fair share of distribution of silt. The **AOSM** exists of a long throated flume (approximately 0.60 m) with **a** roof block, capable of vertical adjustments and with a rounded roof to prevent contraction and ensure straight stream lines. The structure is built fiom reinforced cement (roof block), brick masonry (side walls) and cast iron (adjustable rounded).

Discharge equation
The discharge through an ***APM*** / AOSM outlet structure is determined by either the free flow weir discharge equation when the roof block is out of the water. ***As*** soon **as** the upstream water level rises, the discharge equations changes to the equation for **APM / AOSM** orifice **flow.** The downstream water level is not influencing the discharge through the structure. The hydraulic jump is formed at some distance fiom the gate.

The discharge equation for **free** orifice through an AOSM outlet structure ***can*** be given **as:**

$$q = C_d.B.Y.\sqrt{2.g.z}$$

Where z is defined by: $[H_u - Y]$ and according to Crump, **the** coefficient of discharge remains constant at approximately 0.90 **(FAO, 1975).**

Silt drawing **capacity**

Research has shown that remodelled AOSM outlet structures draw with the crest **at** bed level about **14%** and below bed level at 12/10*FSD about 29% more silt than it would draw at the originally designed **6/10*FSD** setting. With **these** changes in settings, the outlet structure loses its proportionality.

Table 4.3 Improved silt drawing capacity of AOSM

Settings ref. FSD	6/10 setting	8/10 setting	10/10 setting
silt drawing capacity	99.5 %	109.7 %	113.7 % to 121.9 %

source: FAO, 1975; Ali, 1993.

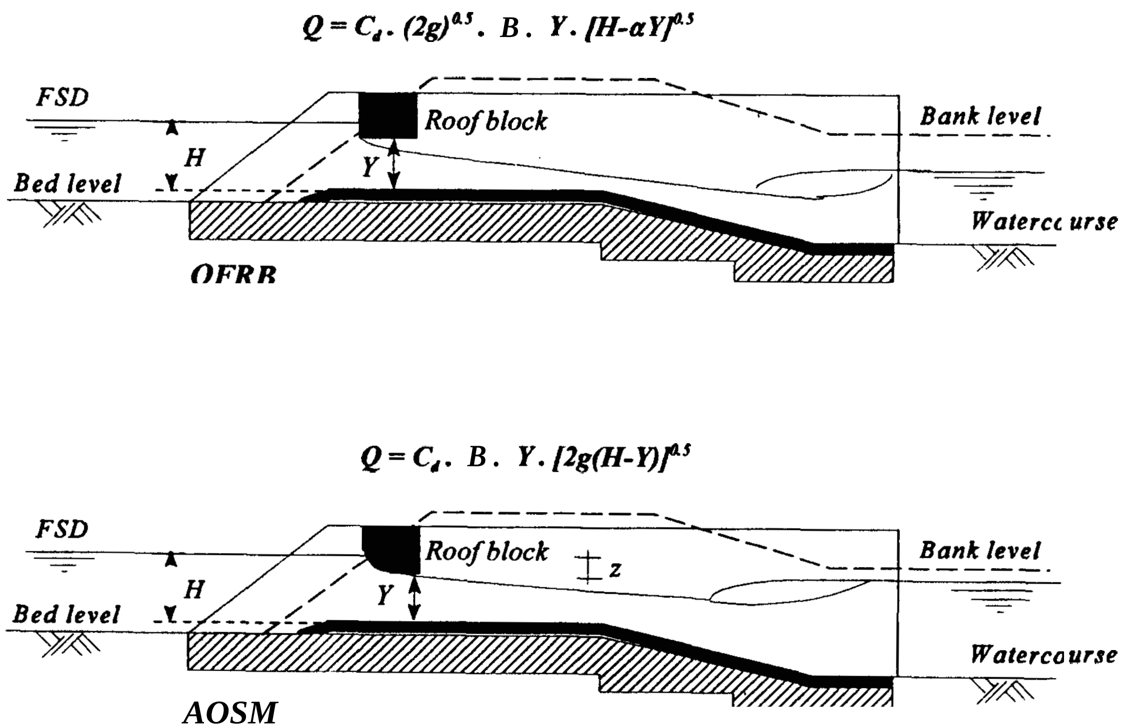


Figure 4.3 Orifice flow for OFRB and AOSM outlet structures.

4.3.5 Pipe outlet structure

General

Pipe outlet structures are the most simple and oldest known types in the Punjab. In early days, the pipe outlet structures were constructed of earthenware, but at present they are replaced by masonry pipes and cast iron and concrete pipes. Pipes are used at places where the available head is low, and therefor most of the outlet structures are running submerged. The pipe outlet structure consist of an upstream head wall, a pipe and a downstream head wall. The entrance is usually at bed level or just above bed level.

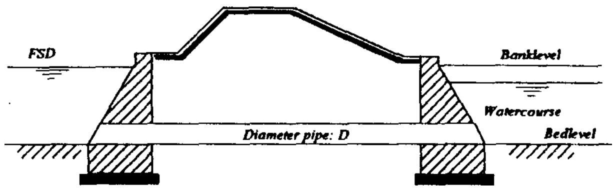


Figure 4.4 **Pipe outlet structure**

The pipe is placed either horizontally, or with a slope 1:12 downstream. Both ends of the pipe outlet structure are built in masonry, which quite offend is dam aged due to bad maintenance, illegal tampering and eroded canal banks. Experimentally, it is found that with the crest at bed level the outlet structure is taking its fair share of silt and (sub) proportional behaviour is achieved. Special merit of the (non-modular) pipe outlet structure is their operation with a very low working head (minimum 2.5 cm, with which no semi-module *can* function).

<u>Discharge equation</u>

For a tube or pipe having a length of 2.5 to 3 times the diameter of the orifice, the discharge equation reads:

$$q = C_p.A.\sqrt{2.g.z}$$

Where:

q	=	Discharge	[m³/s]
C_p	=	Discharge coefficient of a pipe outlet structure	[-]
g	=	9.8 m/s² (gravity acceleration)	[m/s²]
A	=	Area of the opening	[m²]
z	=	Energy head measured from (see figure 4.5):	[m]

1. Centre of the pipe to the water level in the parent canal, when flow enters in the free air
2. The difference in the water level in the watercourse and the distributary, when the pipe discharges into a watercourse in which the water level is above the top of the pipe.

$$q = C_p.A.[2gz]^{0.5}$$

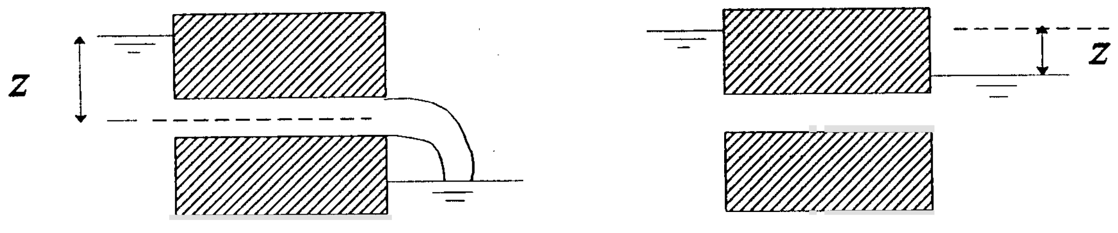


Figure 4.5 **Energy head z for pipe outlet structures.**

Experiments resulted in a C_p coefficient of **0.63** for free flow to **0.74** for submerged flow, with a head loss of **0.33H.** By means of rounding the edge of the entrance of the pipe, suppression of the contraction leads to higher values for C_p.

4.3.6 Pipe-Crump Semi Module

General

This type of outlet structure can also be regarded as a development of the Stoddard-Harvey improved irrigation outlet structure. Upstream of this structure, a pipe is taking off from the parent canal and opening into an approximately 3 square feet (round) tank on the other side of the bank. From the tank, the different types of semi-modular outlet structures can be seen, discharging into the watercourse. Either a pipe working free fall, an open flume or an orifice type. In the area of study, only the so-called Open-Crump OFRB (OCOFRB) and Open-Crump AOSM (OCAOSM) are installed. The same design characteristics and proportional settings for normal OFRB and OASM are applied here.

Discharge equation

The discharge equation of the outlet structure is equal to the type of outlet structure installed at its downstream end. The upstream water level above the crest (h_u) will be determined within the cistern and not in the canal. The head loss through the pipe is minimal, due to the size of the pipe or barrel.

Silt drawing capacity

Special merit of this type of outlet structure is the improved silt drawing capacity, as the opening of the pipe can be placed at bed level or even below bed level. There is no time for the silt to settle, due to high turbulence in the tank. Other advantages of this type of outlet structure are (Mahbub; Gulahti, 1951):

- Large range of modularity;
- Cheap in construction, especially in large canal banks;
- Easy to adjust the settings, when the canal is running;
- Protected for sever interference due to the possibility of early detection, by closing the pipe at the upstream end so the tank will be empty and the actual outlet structure is visible.

4.4 Sensitivity and proportionality

Sensitivity as defined here and discussed in section 3.2.3, is similar to the principles of *flexibility* mentioned in many textbooks. Sensitivity has been defined as:

$$S = \frac{\frac{dq}{q}}{\frac{dQ}{Q}} = \frac{n \cdot H_c}{u \cdot H_w}$$

Obviously any fluctuation, i.e. change in depth of water in the canal will cause an equal change in the head over the outlet: $dH_c = dH_w$. As proportionality is defined by: S = 1, the design settings of outlet structures can be easily expressed in H_w and H_c. In the following table the settings for proportional behaviour with the distributary at FSD are listed (H_c = FSD).

Type	n	Head over the crest: H_w	Crest elevation (from bed level)	Roof block (from bed level)
Open Flume	1.5	0.9 * FSD	0.1 * FSD	-
OFRB	0.5	0.3 * FSD	0.1 * FSD	0.7 * FSD
(OC) OFRB	0.5	0.3 * FSD	0.1 * FSD	0.7 * FSD
AOSM	0.5	0.3 * FSD	< 0.4 * FSD[3]	0.7 * FSD
(OC) AOSM	0.5	0.3 * FSD	< 0.4 * FSD	0.7 * FSD
Pipe	0.5	0.3 * FSD	-[4]	-

3 *Crest level of the (OC)APM was originally set at a height of 0.4 * FSD, but due to bad silt draw, the (OC)AOSM outlet structures (improved (OC)APM's) were designed with a lower crest (no further specified rules).*

4 *Pipe outlet structures were originally designed with the crest at bedlevel, or just a few centimeters above bedlevel, to ensure maximum silt drawing capacity. With a head of 0.3 * FSD, z (= h_u - h_d) should be 0.3 * FSD.*

The sensitivity ratio S will be approximately **1,** with the canal running on FSD:

- for an Open Flume outlet structure, with the crest level placed at **0.9** of the depth of the canal, i.e. **0.1** * FSD above bed level;
- for an Open Flume with Roof Block outlet structure, when the bottom of the roof block is placed at **0.3** of the FSD of the canal and the crest level at 0.9 of the FSD of the canal, i.e. crest **0.1** * FSD above bed level and roofblock 0.7 * FSD above bed level.

When the upstream water level is increasing, orifice type outlet structures **are** becoming less sensitive, i.e. sub-proportional. On the other hand, a decrease in upstream water level results in super-proportional behaviour. When the upstream water level is increasing, weir type outlet structures are becoming more sensitive, i.e. super-proportional. On the other hand, a decrease in upstream water level results in sub-proportional behaviour.

4.5 Outlet structure characteristics determining the distribution

The delivery of canal water to any type of outlet structure is based on the corresponding discharge equation and actual flow condition. For **free flow conditions** the distribution is determined by the upstream water level above the crest, which is related to the elevation of the crest level. The amount of water distributed is related to the discharge coefficient C, the width B and opening height Y, **as** defined in the typical outlet structure equation. For **submerged outlet structures,** besides the characteristics mentioned above, the discharge is depending on the downstream water level above the crest, i.e the water level at the head of the watercourse.

Although the discharge coefficient is fixed for a calibrated situation, the value is changing between certain limits for **free** flow (OC)OFRB outlet structures and pipe outlet structures. For submerged outlet structures the discharge coefficient is variable and quite difficult to determine. The above characteristics, flow conditions and **types** of outlet structures, **are** listed in table **4.5**

Table 4.5 Outlet structure characteristics

Types	Free flow	Submerged flow
Open Flume	- upstream water level - crest level - B - C	- upstream water level - crest level - B - C - downstream water level
(OC)OFRB / OFRB	- upstream water level - crest level - B - Y - C	- upstream water level - crest level - B - C - Y - downstream water level
(OC)APM / APM	- upstream water level - crest level - B - Y	- upstream water level - crest level - B - C -Y - downstream water level
Pipe	- upstream water level - crest level - Y - C	- upstream water level - crest level - B - C -Y - downstream water level

CHAPTER 5 MODELLING A DISTRIBUTARY

5.1 General

To study canal water distribution of a real irrigation system, the use of hydrodynamic software is widely applied by Consultants, Research Institutes and even Water boards and Irrigation Agencies. In this chapter, the development of such a model with the SIC software, for a distributary in the area of study will be discussed. The model will be used to study the canal water distribution and the parameters determining the canal water distribution. In the end, the results of the simulations will be used for the development of the simplified method (SIC $_{disty}$), as described in chapter 1.

The SIC software is a mathematical model which can simulate the hydraulic behaviour of most of the irrigation canals, under steady and unsteady flow conditions. The main purposes of the model are: (1) to provide a research tool to gain in-depth knowledge of the hydraulic behaviour of the main canal and distributaries, within an irrigation system; (2) to identify, through the model, appropriate operational practices at regulating structures with a view to improving the present canal operations; (3) to evaluate the influence of possible modifications to some design parameters with a view to improve and maintaining the capacity of canal to satisfy the discharge and water targets.

The development of a flow model requires a lot of real measured field data for calibration and validation of the model parameters. In annex A, the field measurements and activities are listed. In annex B, the results of the calibration and validation procedures are listed. *Annex* C presents the structure equations as the are used within SIC. In annex D and E, the actual input data for structures and cross sections of listed.

5.2 Simulation of Irrigation Canals (SIC)

5.2.1 Main components

The hydro-dynamic software SIC is built around three main components (computer programs TALWEG, FLUVIA and SIRENE). The three units are:

Unit I

The topographical and geometrical lay out of the canal will be specified here. The topographical and geometrical files are used by unit II and III. The canal will be divided in separate reaches connected by nodes. A **node** is a point where either the canal flow is divided in different directions, or when there is a lateral in- or outflow. Practically, a node is either the head or tail end of the canal, or a secondary or tertiary outlet structure. At least two cross sections have to be entered for every reach, to describe the geometry of the canal.

The cross sections are expressed in an elevation referred to the head of the canal. So, the bed slope of the canal is incorporated in the cross sections.

Unit II

Steady flow computations can be carried out with unit II. The hydraulic characteristics of the canal have to be entered here. Unit II also allows to determine the off take gate openings and adjustable regulator gate settings.

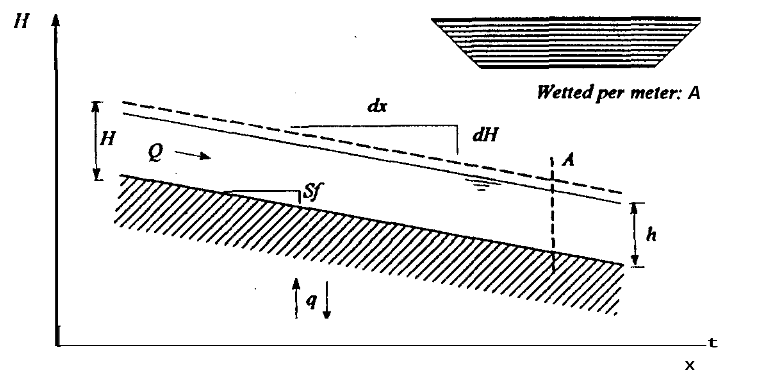


Figure 5.1 Longitudinal profile of a canal: steady flow

Unit II computes the water surface profile for a given constant discharge at the head. The steady state flow computations are based on the Manning-Strickler equation expressed in a differential equation of the water surface profile (see figure 5.1).

$$\frac{dH}{dx} = -S_f + (k-1).\frac{qQ}{gA^2}$$

Where:

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

And:

H	=	Energy head	[m]
x	=	Abscissa	[m]
S_f	=	Bed slope	[-]
k	=	Constant	[-]
q	=	Lateral inflow (> 0) or outflow (< 0) (q > 0: k = 0; q < 0: k = 1)	[m^2/s]
Q	=	Canal discharge	[m^3/s]
A	=	Wetted perimeter	[m^2]
n	=	Manning's coefficient	[$m^{-1/3}$/s]
R	=	Hydraulic radius	[m]
g	=	9.81	[m/s^2]

For solving this equation, an upstream boundary condition in terms of a discharge and a downstream boundary condition in terms of a water surface elevation are required. The water surface profile will be solved step by step starting from the downstream end.

Unit III

Unsteady flow computations can be carried out with unit III. It allows to test various scenario's of water demand schedules and operations at the head works and control structures. Unit III starts from an initial steady state regime, generated by unit II. The unsteady flow computation are based on the Saint Venant's equations. They are solved numerically by discretizing the equations. The discretization scheme used in SIC, in order to solve the equations is a four-point semi-implicit scheme known as Preissmann's scheme (Baume and Malaterre, 1995).

Saint Venant's equations (SIC user guide part II: Theoretical concepts, 1995):

Continuity (conservation of mass of water):

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

Dynamic equation:

$$\frac{\delta Q}{\delta t} + \frac{\delta(Q^2/A)}{\delta x} + g.A.\frac{\delta h}{\delta x} = -g.A.S_f + k.q.V$$

Where:

h	=	Vertical depth of flow	[m]
V	=	Mean fluid velocity	[m/s]
k	=	1 (lateral outflow); 0 (lateral inflow)	[-]

The variation in momentum due to lateral inflow or outflow is expressed by the term: $k.q.V$. The constant k is equal to 1 for a lateral outflow ($q < 0$), and 0 for a lateral inflow ($q > 0$). The partial differential equations must be completed by initial and boundary conditions in order to be solved. The initial condition is the computed water surface profile generated by the steady flow computation. The boundary conditions consist of the hydrograph's at the upstream nodes of the reaches, and a rating curve at the downstream node of the model.

5.2.2 Input data modelling a distributary with SIC

To develop a hydrodynamic model of an irrigation canal, a lot of input data is necessary. Most of the data was already available and was collected from the PIPD and the database of the IIMI field measurement survey's. The following input data is necessary:

Physical data

The topographical files used in **SIC** requires information about the topographical and geometrical layout of the distributary, i.e. a longitudinal profile and various typical cross sections. For each node, both upstream **as** downstream, the following data must be add: **abscissa** (location measured **from** the head of the canal) and **geometrical description** of a reach expressed in typical cross section. It will also be possible to add more cross sections between two nodes. When there are cross structures along the canal, both the upstream and downstream cross section must be add.

Outlet structures and cross structure2

Along a distributary, several different types of outlet and cross structures *can* be distinguished. The input data concerning outlet structures and cross structures along a distributary must be entered in Unit II of **SIC.**

OUTLET STRUCTURES	**CROSS STRUCTURES**
Type of outlet structure	**Type of cross structure**
Crest elevation (reference elevation, m)	**Crest elevation (reference elevation, m)**
Height of the opening (m)	**Height of the opening (m)**
Width of the opening (m)	**Width of the opening (m)**
Authorized discharge (m^3/s)	**Weir length (m)**
Discharge coefficient	**Discharge coefficient**
	Number **of openings**

Upstream boundary condition

The inflow at the head of the distributary must be given as **an** input. To run Unit 11 of **SIC,** the inflow must be constant. To compute dynamical changes of different inflow patterns Unit III starts with the initial state computed in Unit II.

Downstream boundary condition at the tail

The outflow at the tail of the distributary must be determined **as** a function of the upstream water level above the crest. **A** rating curve must be given at this node.

Downstream boundary condition of the outlet structures

When **SIC** has to compute the off taking discharge trough the outlet structures (not an imposed discharge) the downstream boundary condition of the corresponding watercourse must be put in. The downstream water level will be used by **SIC** to select the proper discharge equation based on the present flow condition through the outlet structure. Three options are available:

- constant downstream water level (input: d/s water level);
- weir type downstream condition, i.e a theoretical free flow weir is defined just downstream of the outlet structure (input: weir discharge coefficient, weir crest elevation, weir length and targeted discharge), and;
- user defined downstream condition expressed as a theoretical rating curve.

Canal data

Besides the discharge-depth relation and bed slope expressed in the cross sectional outline of a distributary, there are two major parameters which are determining the hydraulics:

1. the ***roughness coefficient*** expressed as the Manning's coefficient n or Strickler coefficient k [$m^{1/3}/s$], with $k = 1/n$, and;
2. the rate of ***seepage losses*** S_e (l/s/km).

The Manning's coefficient is related with the resistance of the canal profile and will be further determined in the calibration process. The initial value for n will be taken uniform if possible. The seepage can be calculated by means of an ***inflow-outflow test*** and is based on a simplified water-balance methodology. Within a typical reach the seepage can be determined as follows:

$$S_e = Q_{in} - Q_{out} - \Sigma q_{outlet}$$

Where:

S_e	=	Seepage losses	[l/s/km]
Q_{in}	=	Inflow discharge of reach or canal	[m^3/s]
Q_{out}	=	Outflow discharge of reach or canal	[m^3/s]
q_{outlet}	=	Allocated discharges to outlet structures	[m^3/s]

The value for the seepage will be negative, inflow seepage or gain, if there is an inflow due to high phreatic ground water tables or leaching from neighbouring canals. Most canals in the Punjab are subject to frequent fluctuations in water levels over relatively short periods of time (Kuper et al., 1994, Bhutta and Vander Velde, 1990). To obtain reliable seepage data using the inflow-outflow method, a steady state flow period (SFP) in the canal is desirable.

5.2.3 Structure equations used in SIC

The structure equations for both cross structures and off taking structures used in SIC are based on experimental studies, and therefor they differ from the theoretical orifice flow and weir flow equations as presented in chapter 4. Especially for the transition phase between free flow and submerged flow, the equations are based on several different ranges where different equations are used. The different ranges are defined by the relation between the ratio h_1/Y and the ratio h_2/Y, where: h_1 determines the upstream water level above the crest [m], h_2 determines the downstream water level above the crest [m], and Y determines the gate opening [m].

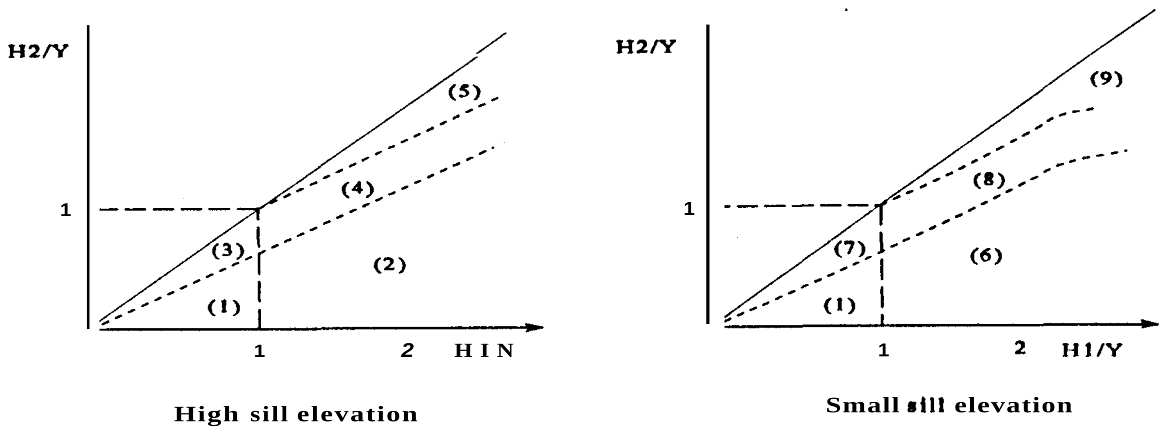


Figure 5.2 **Validity ranges of SIC equations.**

The different ranges where the different equations used in SIC are applied are presented in figure 5.2. From the graphs, it can be concluded that one of the simplifications is based on the assumption that the transition between open flow and orifice flow occurs for h, = Y.

1. Openflow, free flow
2. Orifice flow, free flow
3. Openflow, submergedflow
4. Orificeflow, partially submergedflow
5. Orificeflow, completely submergedflow
6. Orifice flow, free flow
7. Openflow, submergedflow
8. Orificeflow, partially submergedflow
9. Orificeflow, completely submergedflow

The corresponding structure equations are listed in annex C.

5.3 Development of a SIC model of a distributary

5.3.1 Masood distributary

From the 14 distributaries in the area of study (see section 2.2), only one is modelled for this study. Masood distributary was chosen for this purpose, based on the following points:

- The length of the distributary should be in between 35,000 and 55,000 feet and should not exceed the amount of 15 to 20 outlet structures, to reduce the amount of time necessary for developing, calibration and validation of the model.
- The major types of outlet structures with different types of flow conditions should be present, i.e. OFRB outlet structures and PIPE outlet structures.

- To be able to do measurements in the period of study (November **1995** - May **1996)** the canal should be perennial, with a variable inflow pattern, to cover **a** representative range of discharge fluctuations at the head of the distributary.

The **Masood** distributary is **a** small secondary canal, off **takes** at RD **316.38** of Fordwah Branch **(see** figure **2.2,** chapter 2) **.** It is running all the way just along the Fordwah Branch. All the tertiary outlet structures are located on the right bank. It is a perennial canal and originally designed for an inflow of approximately **1** m^3/s **(35** cfs) at the head, with a total length of **15.9 km (52,300 A).** At present the canal water flow hardly reaches **RD 45.95.** The physical condition of the **Masood** distributary is sufficient. At some places the right bank is damaged by cattle and much vegetation is found in the middle reach and tail reach. The left bank is not damaged, mainly because it is part of the right bank of the Fordwah Branch. No cuts were observed and no outlet structures were damaged. The general design characteristics of the **Masood** distributary are listed in table **5.2.** The listed design **data** of Masood distributary are based on the PIPD records.

Table 5.2 General design characteristics of Masood distributary

Outlet no.	RD	G.C.A. (acres)	C.C.A. (acres)	Crest (ft)	B (ft)	Y (ft)	Q[1] (cfs)	Type
1	1.10	268	268	488.47	0.20	1.11	0.96	OFRB
2	3.70	408	408	487.90	0.25	1.24	1.47	OFRB
3	7.30	292	292	487.26	0.20	1.23	1.10	OFRB
4	13.50	379	379	486.78	0.83	-	1.32	PIPE
drop 1	18.00	-	-	485.50	-	-	-	DROP
5	24.00	474	464	484.22	0.92	-	1.67	PIPE
drop 2	24.05	-	-	483.50	-	-	-	DROP
6	27.20	493	489	482.22	0.33	1.10	1.76	OFRB
7	28.75	260	250	482.32	0.30	0.77	0.90	OFRB
8	34.86	452	447	480.49	0.34	1.09	1.82	OFRB
9	35.59	573	567	480.30	0.38	1.11	2.04	OFRB
10	35.60	441	440	480.69	0.43	1.00	1.67	OFRB
11	36.62	494	494	480.25	0.38	0.98	1.81	OFRB
12	37.15	462	443	480.02	0.39	0.90	1.96	OFRB
drop 3	37.25	-	-	-	-	-	-	DROP
13	44.32	615	544	475.32	0.55	0.78	1.96	OFRB
14	45.95	615	612	474.99	0.62	0.78	2.20	OFRB
15	50.20	512	490	473.07	0.59	0.83	1.76	OF
Tail	50.20	480	476	472.65	0.56	16	1.71	OF

1 *q =Authorized discharge (in cfs)*

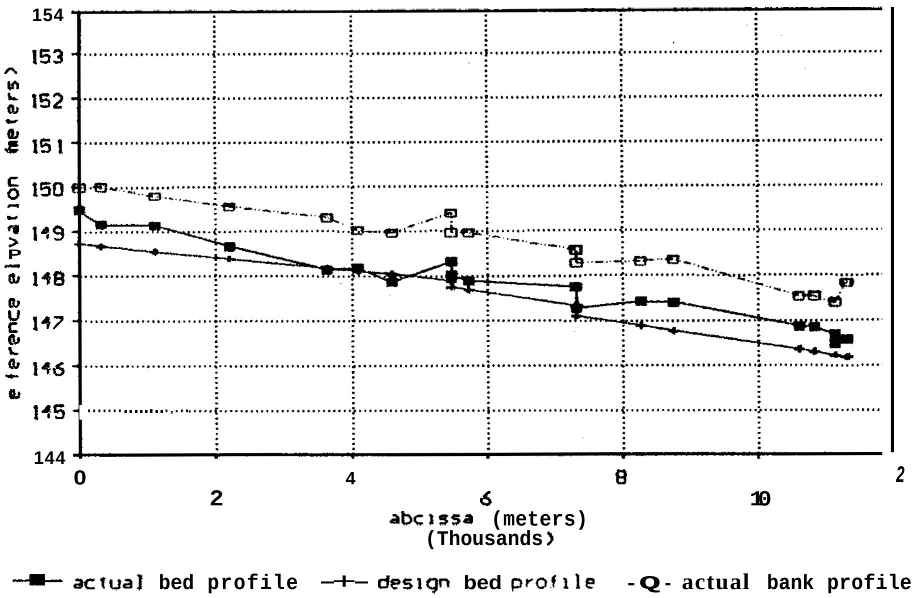


Figure 5.3 **Longitudinal profile Masood distributary: design and actual state.**

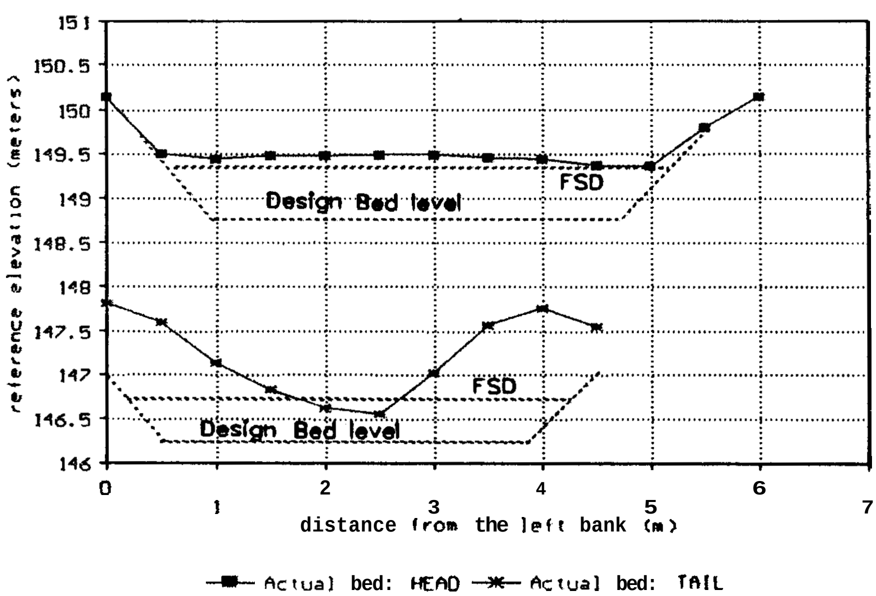


Figure 5.4 **Actual cross sections Head and Tail of Masood distributary.**

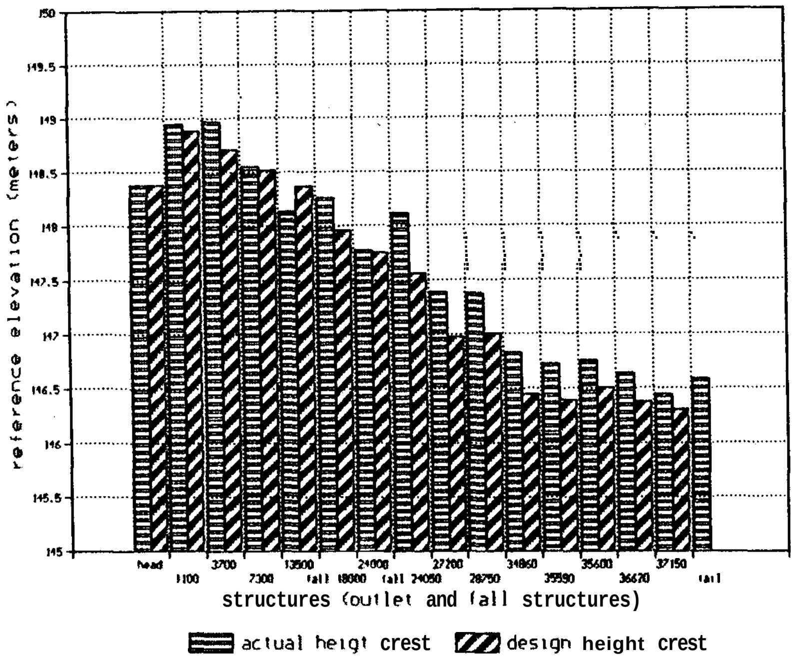


Figure 5.5 **Crest elevations: actual and design levels of Masood distributary.**

It is interesting to compare the design state of the distributary, based on information from the PIPD and original construction drawings, with the measured topographical and geometrical data. The following figures 5.3, 5.4 and 5.5, are presenting the differences between the design and actual state of Masood distributary. Due to siltation and a lack of proper maintenance, there is a substantial difference between the actual bed level of the canal and the design bed level. At the head and downstream reaches there is up to 0.75 metre siltation (figure 5.3 and 5.4).

When looking at figure 5.5, it can be stated that comparing the actual crest levels of the outlet structures and cross structures, with the design levels, almost all levels are increased up to 0.30 metre. The crest of the drop structure at RD 24.05 has been increased even with 0.40 metres. There are two possibilities to clarify this: (1) to tackle the problems of heavy siltation and lacking distribution, the crest levels were increased by the PIPD or the farmers themselves, and / or (2) the initial starting point of the conducted topographical survey of Masood distributary was not correct.

Table 5.3 Actual characteristics of Masood distributary: comparison of actual B and Y with design

Outlet no.	RD	Crest (m)	B (m)	dB[2] (m)	Y (m)	dY[2] (m)	Type
1	1.10	148.947	0.067	-	0.393	+0.055	OFRB
2	3.70	148.980	0.110	+0.034	0.488	+0.110	OFRB
3	7.30	148.547	0.070	+0.009	0.378		OFRB
4	13.50	148.133	-	-	0.271	+0.018	PIPE
drop 1	18.00	148.270	1.219	-	-	-	DROP
5	24.00	147.781	-	-	0.268	- 0.012	PIPE
drop 2	24.05	148.113	3.277	-	-	-	DROP
6	27.20	147.392	0.116	+0.015	0.332		OFRB
7	28.75	147.397	0.104	+0.013	0.253	+0.018	OFRB
8	34.86	146.838	0.098	- 0.006	0.329		OFRB
9	35.59	146.733	0.116	-	0.354	+0.016	OFRB
10	35.60	146.756	0.128		0.256	- 0.050	OFRB
11	36.62	146.644	0.119		0.296		OFRB
12	37.15	146.453	0.177	+0.058	0.253	- 0.021	OFRB
drop 3	37.25	146.594	0.628				DROP

Comparison of the actual and design width and height of the outlet structures is showing that there are differences. Especially outlet structure no. 2, 4, 5, 6, 7 and 12 are substantially tampered, either by illegal manipulations of farmers, or re-modelling construction works by the PIPD. The changing dimensions do have there impact on the distribution of canal water, and will be further discussed in chapter 7.

5.3.2 Reaches and nodes

The total length of the canal modelled in SIC is 11,354 m (37,250 ft). The modelled part of the canal consists of 14 nodes, i.e. 1 head node, 12 nodes at the tertiary outlet structures and 1 tail node. Between these nodes in total 13 reaches can be distinguished. In the next table 5.4 the length of the different reaches determined by the SIC model of Masood distributary are listed.

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[2] *dB and dY representing the measured difference between actual and design (see table 5.2) width (B) and height (Y) of the outlet structure opening.*

Table 5.4 Reaches and nodes

Outlet No.	Node	Reach	Length (ft)	Length (m)	Distance from the head (ft)	Distance from the head (m)
	Head		0	0	0	0
1	1100-R	1	1100	335.3	1100	335.8
2	3700-R	2	2600	792.5	3700	1127.8
3	7300-R	3	3600	1097.3	7300	2225.0
4	13500-R	4	6200	1889.8	13500	4114.8
5	24000-R	5	10500	3200.4	24000	7315.2
6	27200-R	6	3200	975.4	27200	8290.6
7	28750-R	7	1550	472.4	28750	8763
8	34860-R	8	6110	1862.3	34860	10625.3
9	35590-R	9	730	222.5	35590	10847.8
10	35600-R	10	10	3.0	35600	10850.9
11	36620-R	11	1020	310.9	36620	11161.8
12	37150-R	12	530	161.5	37150	11323.3
	Tail	13	100	30.5	37250	11353.8

In figure 5.6 a schematization of the modelled canal is presented.

5.3.3 Data collection

Most of the hydraulic data was already available, but to obtain two more data sets to analyse the existing data and to validate the calibrated SIC model of Masood distributary measurements took place for different discharges at the head. Due to a lack of reliable geographical and topographical data of Masood distributary a topographical survey was set up. The field measurements took place during a three week visit from 23-11-1995 to 11-12-1995.

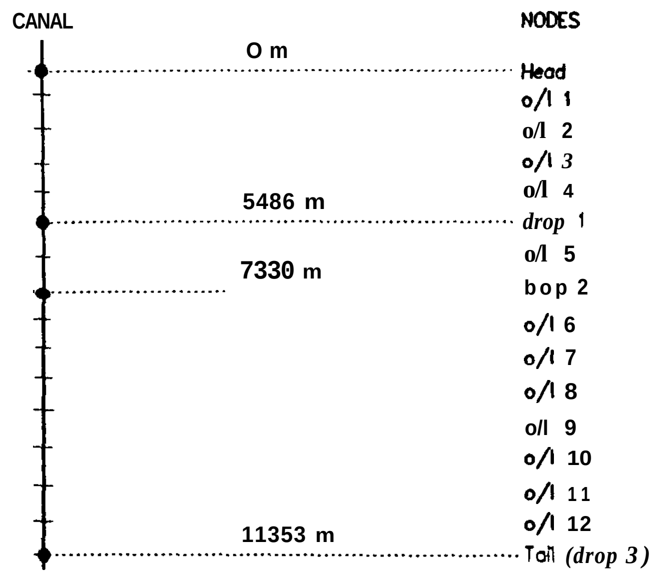

Figure 5.6 Schematization Masood distributary.

The initial data set, necessary to start the simulation and to provide reliable input data for the model calibration procedure was obtained by an outlet structure calibration survey of all the **14** distributaries of the Fordwah branch, conducted by IIMI in **1995.** The IIMI study of the Masood distributary was started at **15-11-1995.** Hydraulic data for calibration and validation were measured on **Masood** distributary on **27-11-1995** and **30-11-1995.** To be able to develop rating ***curves*** for the downstream condition for submerged outlet structures, several measurements took place on **11** and **12** February **1996.**

Activities, methodology and results of the field measurements are listed in annex **A.**

5.3.4 Overall results and conclusions of the field measurements

First of all the hardware has to be calibrated, i.e. determine the discharge coefficients for the outlet structures and cross structures. Also canal data must be determined: the rate of seepage losses and a reliable initial value for the roughness coefficient (n) must be determined. The field data from **15-11-1995** will be used to calibrate the SIC model of Masood distributary for both water levels in the canal and discharge coefficients of the structures. The field data from **27-11-1995** and **30-11-1995** will be used to validate the calibrated SIC model **(see** annex **A):**

Field data 27-11-1995: validation of water levels, discharge coefficients of cross structures and supplied discharges to the outlet structures.

Field data 30-11-1995: validation of inflow, outflow and typical discharges along the canal.

The next figures **5.7** and **5.8** are presenting the results of the different field measurements. The upstream water levels (h_u) above the crest of the outlet structures are depending on the inflow at the head of the distributary. The difference in the raise in upstream water level for certain outlet structures **is** due to geometrical differences of the cross sections. Figure **5.8** presents the calibrated discharge coefficients for the outlet structures of Masood distributary, both for the **15-11-1995** and **27-11-1995** measurements. Only outlet structures no. **4, 5, 7, 11** and **14** were recalibrated during the exercise on **27-11-1995.** It can be stated that for the **2** submerged pipe outlet structures (no. **4** and **5)** the calibrated discharge coefficients differ from the theoretical value of approximately **0.74.** The difference *can* be due to errors in the measurements and unsteady canal behaviour upstream the flume during measurement. In general it *can* be stated that calibration of submerged pipe outlet structure is a difficult task. For the 3 OFRB structures (flow condition: o.m.) the re-calibration is successfully. The difference **(14%)** at outlet structure no. **14** is due to unsteady canal behaviour during the **27-11-1995** discharge measurement at the tail. Except for outlet structure no. **2,** the discharge coefficients of all the OFRB outlet structures **are** approximately in between **4.0** and **5.0**[3] **(C_d = 0.5** to **0.6).** Free weir flow conditions were observed **at** OFRB no. **11:** discharge coefficient approximately **3.0 $ft^{1/2}/s$** (C, = **0.97).** The discharge coefficient of OFRB no. **2** is to low, which can be due to errors in the measurements or **free** weir flow condition during measurement. The outlet structure was closed during the **27-11-1995** measurements.

3 **Measured m feet (A) and cusec (csf), $(2*g)^{0.5}$ incorporated (values used by IIMI).**

- 15-11-1995: $Q_{head} = 23.1\ cfs = 0.65\ m^3/s$
- 27-11-1995: $Q_{head} = 28.0\ cfs = 0.80\ m^3/s$
- 30-11-1995: $Q_{head} = 18.1\ cfs = 0.51\ m^3/s$

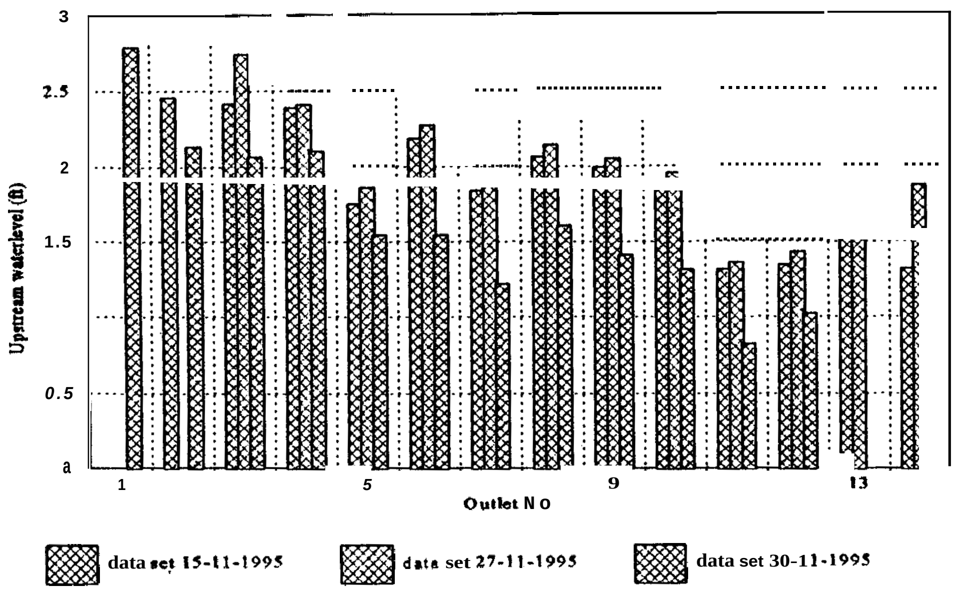

Figure 5.7 Upstream water levels (above the crest) for different discharges at the head of Masood Distributary.

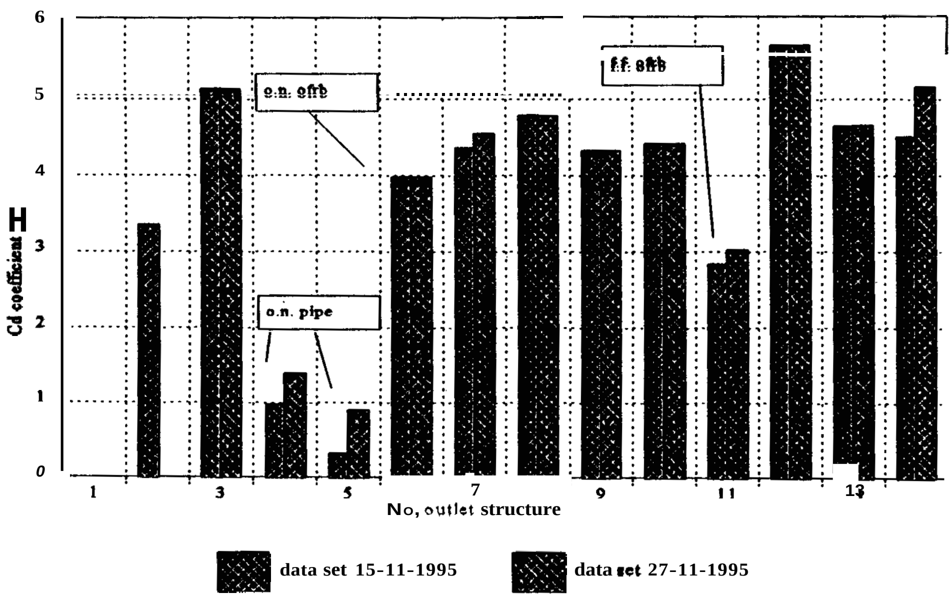

Figure 5.8 Calibrated and computed discharge coefficients of outlet structures along the Masood distributary.

Figure ***5.9*** presents the ratio of ***actual supplied discharge to target (authorized) discharge*** for all outlet structures. The ratio q_{actual} / q_{design}, is related to the inflow at the head of the distributary. Observing the graph, three reaches can be distinguished, i.e. a ***head reach*** , a ***middle reach*** and a ***tail reach.*** The performance[4], of the distributary is dominated by an unequitable distribution of canal water. To much water is distributed to the outlet structures in the head reach, and less water or no water is distributed to the tail-enders. This distribution pattern can be observed in many distributaries (Bhutta, 1991; Vander velde, 1991; Hart, 1995). Due to modified outlet characteristics, the actual supplied discharge to outlet structure no. **2, 4, 7** and **12** is far above design. For the submerged pipe outlet structure and **OFRB** (no. **5** and 6) the supplied discharges are less then design. In figure 5.10, the distribution of canal water to the outlet structures is presented, based on different inflow at the head of the distributary.

Calibration of the hardware

To calibrate the **3** drop structures in a proper way, it is important to have a steady state situation in the canal. During the **measurements:** (1) the discharge just downstream of the drop structures were measured; **(2)** the flow condition was determined and (3) h_u and h_d were measured. Drop 1 at RD 18.00 (combined drop structure and bridge) appeared to be fully submerged and therefor it **was** not possible to calibrate this structure properly: submergence ratio of almost 1. Actually, the flow through this structure is due to heavy siltation just downstream the drop transformed from free flow to conveyance flow. For the second and third drop structures at **RD 24.05** and at RD **37.25,** the flow condition appeared to be free weir **flow,** determined by the rating curve for a broad-crested weir. In the next table the results of the calibration are listed.

Table **5.5** Results calibration cross structures along Masood distributary.

Drop structure	**C_d [ft and cfs] 15-11-1995**	**C_1 [$m^{1/2}$/s] 27-11-1995**	**C_d [ft and cfs] 15-11-1995**	**C_1 [$m^{1/2}$/s] 27-11-1995**
RD 18.00	-	-	-	-
RD 24.05	2.91	0.95	3.05	0.99
RD 37.25	2.57	0.83	2.53	0.82

4 *The 'performance' of a distributary is defined as the evaluation of the canal water distribution (to the outlet structures) based on the principles of irrigation, i.e. equity and proportionality.*

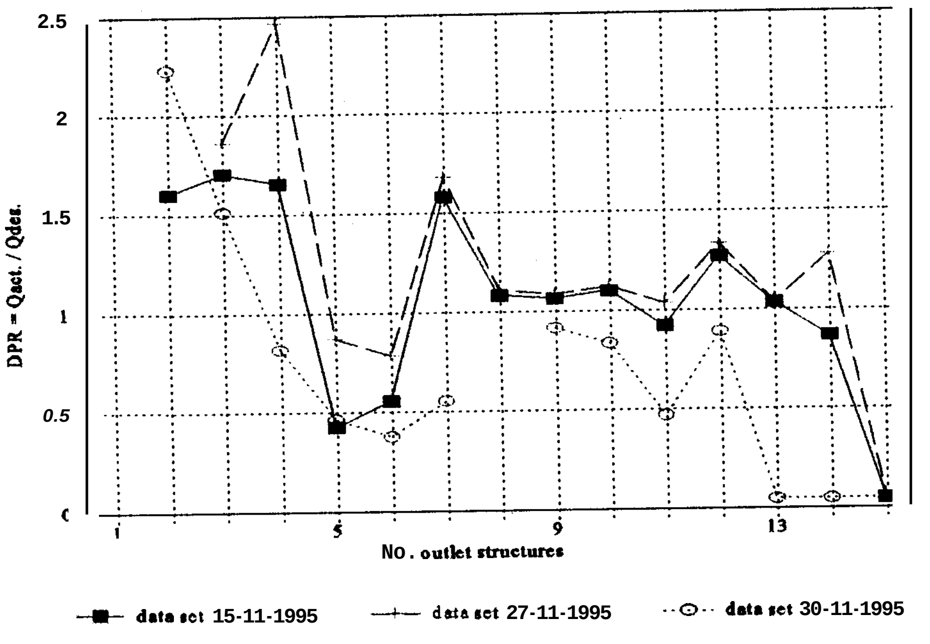

Figure 5.9 **Delivery performance ratio of the outlet structures based on different inflow at the head.**

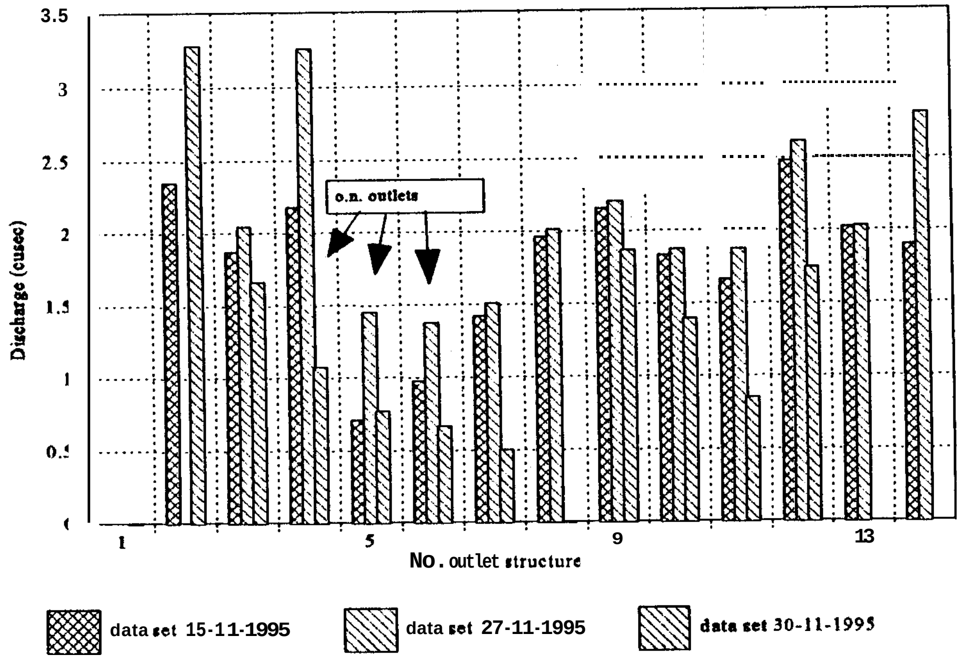

Figure 5.10 **Distribution to the outlet structures based on different inflow.**

The initial downstream boundary condition for all the outlet structures will be a fixed downstream water level based on the water level measured in the field. The downstream water level, i.e. the water level at the head of the watercourse, is directly related to discharge fluctuations in the parent distributary.

For outlet structures it is difficult to use the user's defined type of downstream boundary condition. The development of a rating curve exceeds several water level measurements related to different discharges. For all outlet structures this will be a difficult task. Besides that, in practice most of the outlet structures (except no. 4, 5 and 6) were operating under free flow conditions, so downstream effects do not influence the distribution of canal water. Whenever the model will be used for unsteady flow simulations, rating curves for the downstream end of submerged outlet structures are definitely necessary, because a change in discharge in the parent canal results in a change in water level (in the watercourse) just downstream the submerged outlet structures and therefor in a change in distributed discharge.

5.3.5 initial input for the SIC model of Masood distributary

To obtain a proper set of initial input data for the model. It is important to start the simulation before the actual calibration of the model with a set of reliable data which is valid for the actual situation of the distributary. All the input data must correspond with the actual field data. Only then, it will be possible to compare the computed model output with the measured data.

Table 5.7 Results field data 15-11-1995: initial hydraulic input for the SIC model.

Outlet No.	RD	h_u (ref. level, m)	h_d (ref. level, m)	q_{target} (m^3/s)	C_d (SIC)	Flow condition
1	1.10	-	-	-	-	closed
2	3.70	149.71	149.09	0.067	0.42	o.m.
3	7.30	149.28	148.80	0.053	0.63	o.m.
4	1.350	148.86	148.80	0.062	0.60	o.n.
5	2.400	148.31	148.27	0.020	0.38	o.n.
6	2.720	148.06	147.95	0.028	0.49	o.n.
7	2.875	147.96	147.77	0.040	0.54	o.m.
8	3..486	147.47	147.18	0.056	0.60	o.m.
9	35.59	147.34	147.04	0.061	0.54	o.m.
10	35.60	147.33	147.03	0.052	0.55	o.m.
11	3662	147.04	146.78	0.047	0.35	f.f.
12	37150	146.86	146.49	0.070	0.70	o.m.

Practically, the simulation starts with the same ***upstream boundary condition,*** i.e. measured constant inflow, same ***downstream boundary condition*** for both the outlet structures and the distributary, ***calibrated discharge coefficients*** (outlet structures and cross structures), ***rate of seepage losses*** and ***Manning coefficient.*** The results of the 'hardware' calibration (discharge coefficients for the outlet structures and cross structures), downstream boundary condition for each outlet structure (fixed downstream water level) and target discharge, i.e. the measured discharge in the watercourse, based on the field data from 15-11-1995, are listed in table 5.7

For the fully submerged drop structure at RD 18.00, the discharge coefficient is fixed at a value of 1.00 (this is the maximum value that *can* be entered in SIC, and the results seems to be good). SIC computes at this point a head loss, also found in the field, of approximately 2 cm.

Table 5.8 Calibration of the drop structures: 15-11-1995

drop	location	H_u (ref. level)	h_d (ref. level)	Q (m^3/s)	C_d[5] (SIC)	Flow
1	RD 18.00	-	-	-	1.00	o.n.
2	RD 24.05	0.64	-	16.00	0.36	o.m.
3	Rd 37.25	0.925	-	4.71	0.32	o.m.

Based on observations, the initial value for the **roughness coefficient** is fixed for reach 1 to 7: n = 0.028 (k = 35.7 $m^{1/3}/s$); and reach 8 to 13: n = 0.045 (k = 22.2 $m^{1/3}/s$). Seepage is entered as computed and presented in table 5.6. The downstream boundary condition of the model will be represented by the drop structure at RD 37.25. This drop structure works as a free flow weir, determined by a depth-discharge relation above the crested. In the following table the downstream rating curve is presented.

5 *The discharge coefficient for free weir flow reads in SIC:* $C_d * (2g)^{0.5}$ (= 1.7 * C_d).

Table 5.9 **Downstream rating curve for the model of Masood distributary**

Q (m^3/s)	h_s (m above crest level)	Elevation referred from crest (m)
0	0	146.59 (crest level)
0.01	0.05	146.64
0.028	0.10	146.69
0.052	0.15	146.74
0.082	0.20	146.79
0.111	0.25	146.84
0.146	0.30	146.89
0.184	0.35	146.94
0.225	0.40	146.99
0.269	0.45	147.04
0.315	0.50	147.09
0.363	0.55	147.14
0.414	0.60	147.19
0.467	0.65	147.24
0.522	0.70	147.29

Where: the discharge equation reads: $Q = 1.7 . C_1 . B . H^{1.5}$; B = 0.628 m; and C_1 the calibrated discharge coefficient the weir: 0.83 $m^{1/2}/s$.

5.4 Calibration of the model

By means of model calibration, the SIC model of the Masood distributary will be changed by adjusting several variables until the output of the model, i.e. computed water levels and discharges, match the real measured values. When the model is properly calibrated for a typical situation observed in the field, the model will be validated with another set of field date, to check the calibration results and the validity of the model output. After calibration and validation the model can be used to simulate different situations without disturbing the actual functioning of the system. For this purpose only unit II of SIC was used. In principle, the variables listed below are used to calibrate flow models:

- Discharge coefficients of outlet and cross structures
- Roughness coefficient (n or k)
- Seepage (S_e)
- Downstream boundary condition for the outlet structures

The used methodology to develop a proper model of the Masood distributary is summarized below. The results of the different calibration steps are presented in annex B.

step 1
After developing the flow model of the Masood distributary in SIC with the initial data set as described in section 5.4.5, based on the field data of 15-11-1995, the simulation can be started. Characteristics: inflow at the head 23.1 cusec (0.65 m^3/s) and outlet structure 1 was closed. After simulation based on the actual situation, the computed discharges and upstream water levels are compared with the actual measured data: the calibration of the model starts.

step 2
Use the calibration module of SIC to compute Manning's coefficients for different reaches based on several water level measurements along the canal.

step 3
Run the model with the calibrated Manning's coefficient and compare the computed water supplies to the outlet structures with the measured discharges in the field. Adjust the discharge coefficients of the outlet structures in such a way that the computed discharges match the measured discharges.

step 4
Final step in the calibration process is to evaluate the calibration results and present the final calibrated coefficients. In the following figures all the results are presented.

The model is accurate for the measured discharges and water levels dated 15-11-1995. After examining figure 5.11 and figure 5.12, it can be stated that the calibrated Manning's coefficients for the SIC model of Masood distributary are accurate, i.e. the computed water levels are matching with the measured water levels in the field (maximum deviation for outlet 12: 0.04 m).

Figure 5.13, the adjusted (calibrated) discharge coefficients in SIC are compared with the initial values, i.e. the discharge coefficients of the outlet structures based on the measurements. It can be concluded that:

- for all OFRB outlet structures, except the submerged one (no. 6), the calibrated discharge coefficients are close to the initial values (maximum deviation for outlet structure 7: 7.4 %);
- the discharge coefficient for submerged outlet structures (4, 5 and 6) is variable;
- outlet structure no. 11 is functioning free open flow during the measurements, but orifice flow during the simulations due to the theoretical transition between open weir flow and orifice flow used in SIC: H = Y. Therefor, the calibrated discharge coefficient for the SIC model reaches 0.65 (orifice flow).

The overall deviation between computed and measured discharges supplied to the outlet structures, as seen in figure 5.14, is varying up to 5%. **With an accuracy of 5%, it can be stated that the SIC model of Masood distributary is calibrated in a proper way.**

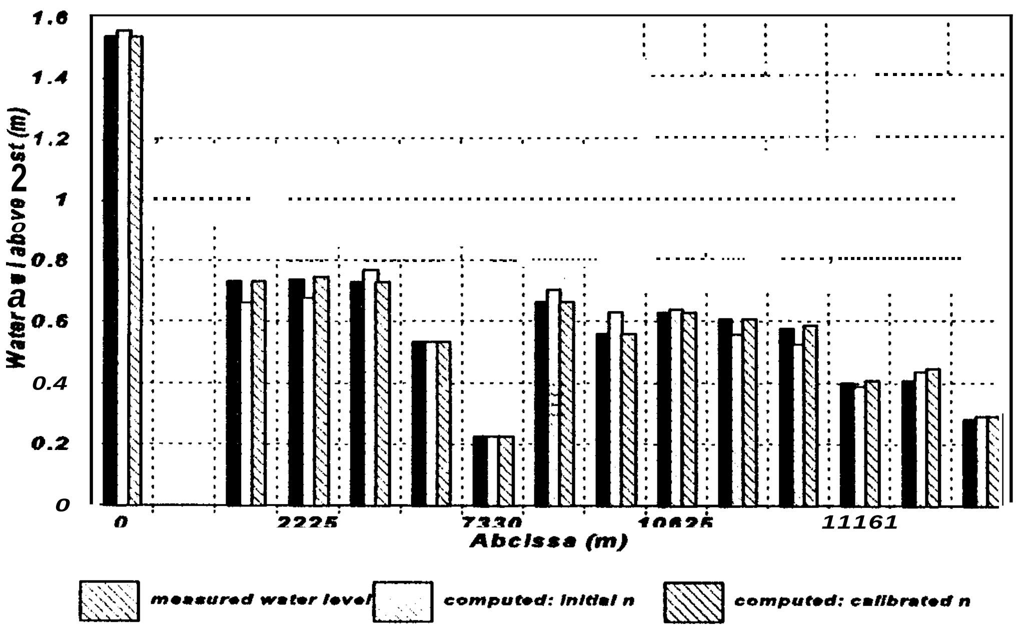

Figure 5.11 Water levels along the canal: measured values, values for initial Manning's coefficient (n) and calibrated Manning's coefficient.

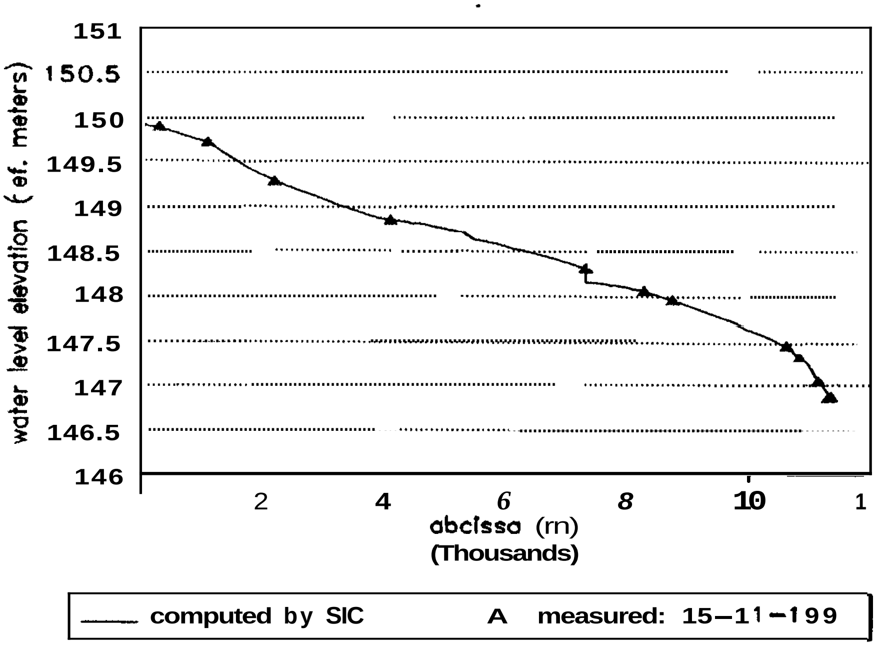

Figure 5.12 Computed water profile and measured water levels along the canal.

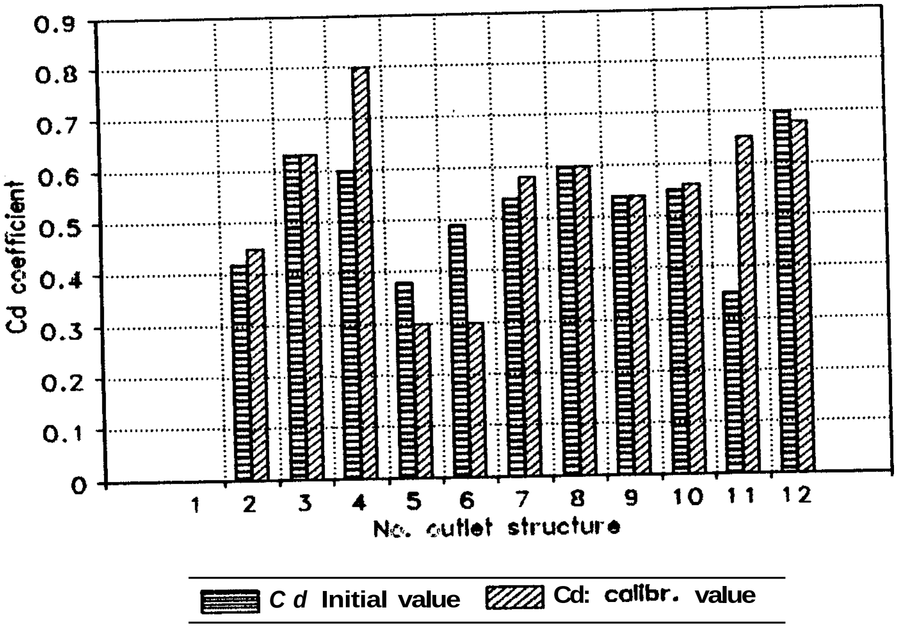

Figure 5.13 Discharge coefficients before (initial value) and after model calibration.

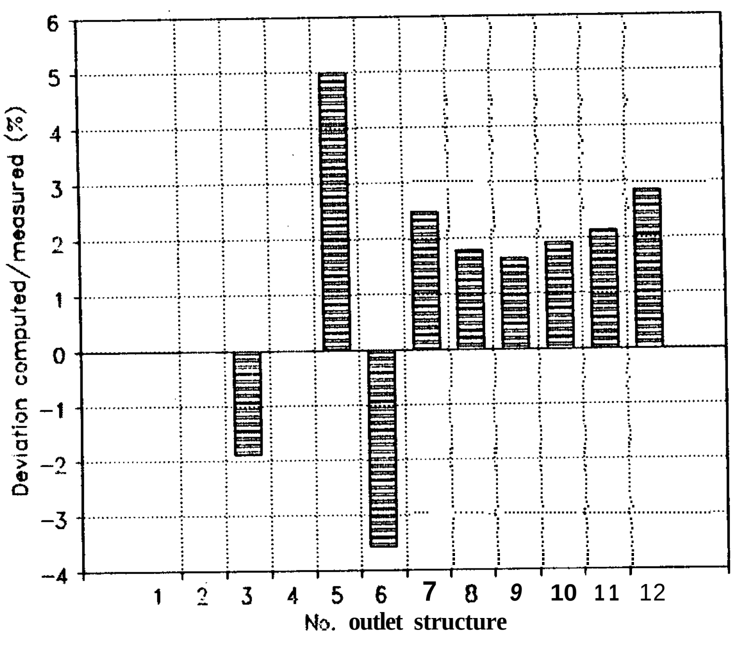

Figure 5.14 Deviation between computed discharges and actual measured discharges.

5.5 Validation of the model

5.5.1 Input data for the validation

The calibrated model of Masood distributary will be validated in a steady state situation based on the steady state data sets from 27-11 and 30-11-1995. For this purpose only unit II of SIC was used. The used methodology to validate the model of the Masood distributary is described below. The results of the different validation steps are presented in annex B. Figure 5.15 and 5.16 on page 65 presenting the results of the different validation scenario's

step I

The model is running a simulation based on the input data of 27-11-1995, i.e. a constant inflow discharge of 0.80 m^3/s (28 csf) at the head of Masood distributary. All other input data and calibrated parameters are kept constant **(validation 1).** It can be stated that the computed discharge supply to the outlet structures compared with the measured discharges is varying up to 29% (outlet structure 5). More accurate discharge computation will be obtained by the model, if the downstream boundary condition of the submerged outlet structures is set on the real measured downstream water levels in the corresponding watercourses **(validation 2).** Still, there are differences between the computed and measured discharges (up to 16% for outlet structure 12). The differences are due to higher computed water levels along the canal, compared with the measured values. Proposed adjustment: use the actual seepage values of 27-11-1995 instead of the seepage values of the model based on the 15-11-1995 measurements. The main difference is that there is inflow seepage (27-11) instead of outflow seepage (15-11). Both, the fixed downstream water level as the measured downstream water level of the submerged outlet structures will be simulated **(validation 3 and 4).**

So, 4 different validation scenario's are evaluated and the results are presented in figure 5.15:

- **validation 1:** no changes in the model.
- **validation 2:** real measured d/s water level as a d/s boundary condition for the submerged outlet structures 4, 5 and 6.
- **validation 3:** seepage as computed for 27-11-1995.
- **validation 4:** **both seepage as measured d/s water level for a d/s boundary condition for the submerged outlet structures 4, 5 and 6.**

step 2

The model is running a simulation based on the input data of 30-11-1995, i.e. a constant inflow discharge of 0.51 m^3/s (18.13 *csf*) at the head of Masood distributary. The validation is to check different measured discharges along the canal, and measured water levels upstream of the outlet structures, with the computed output of the model. Based on the conclusions of the 27-11-1995 validation, the seepage will be taken as the 27-11-1995 measurements (because of low water levels of Fordwah Branch, also outflow seepage) and the measured downstream water levels for the submerged outlet structures (no. 4,5,6 and 7) will be used in the model.

5.5.2 Conclusions of the validation

In figure 5.15, the results of the 4 different scenario's are presented. It can be concluded that scenario 4 is the most accurate (deviation between computed and measured discharge up to 10%). It can be concluded that the downstream boundary condition for submerged outlet structures is an important characteristics. Besides that, the existence of either inflow seepage or outflow seepage has its impact on the distribution. Also the water levels upstream of the outlet structures, measured from the crest, are computed correctly, compared with the real measured values: deviation up to 0.08 m for outlet structure 4 (figure 5.16).

For the 30-11-1995 validation (step 2) it can be concluded that both computed upstream water levels above the crest of the outlet structures (maximum deviation up to 0.07 m) as the computed discharges along the canal match with the measured values. In table 5.10, the 30-11 measured discharges and computed discharges along the canal are presented.

Table 5.10 Comparison of the measured discharges and computed discharges along the canal (validation step 2)

Location	Q measured (15 - 11 - 1995) (m^3/s)	Q computed (SIC) (m^3/s)
Head at RD 0	0.513	0.513
Drop 1 at RD 18.00	0.340 (o.n.)	0.342 (o.n.)
Drop 2 at RD 24.05	0.299 (f.f.)	0.296 (f.f.)
Tail at RD 37.25	0.035 (f.f.)	0.039 (f.f.)

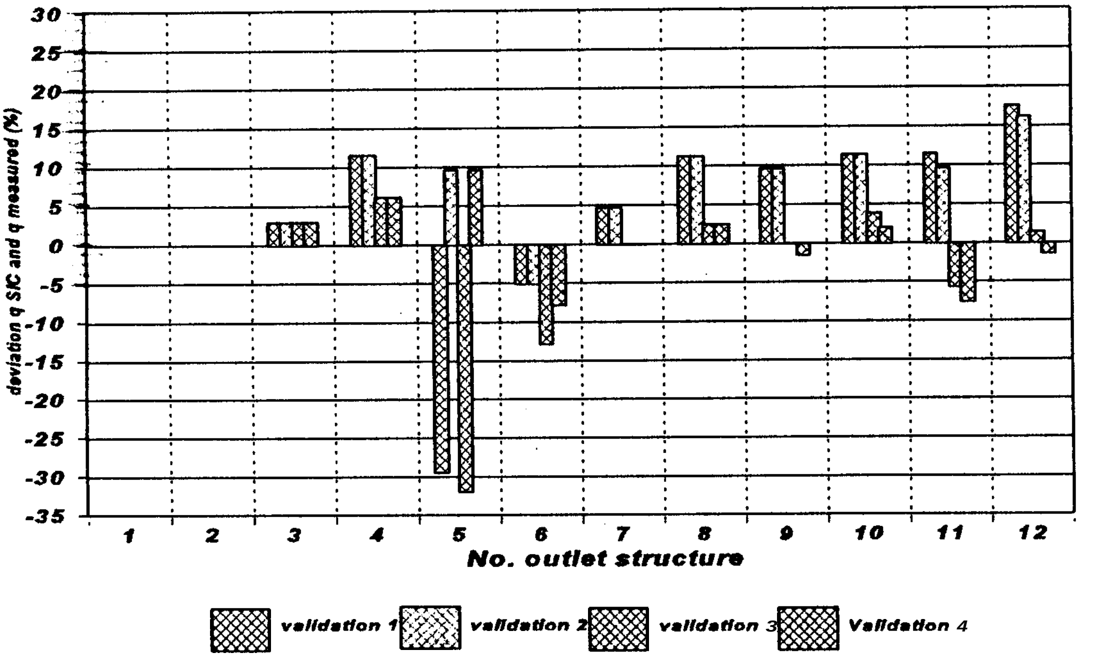

Figure 5.15 **Validation results of the 4 different scenario's.**

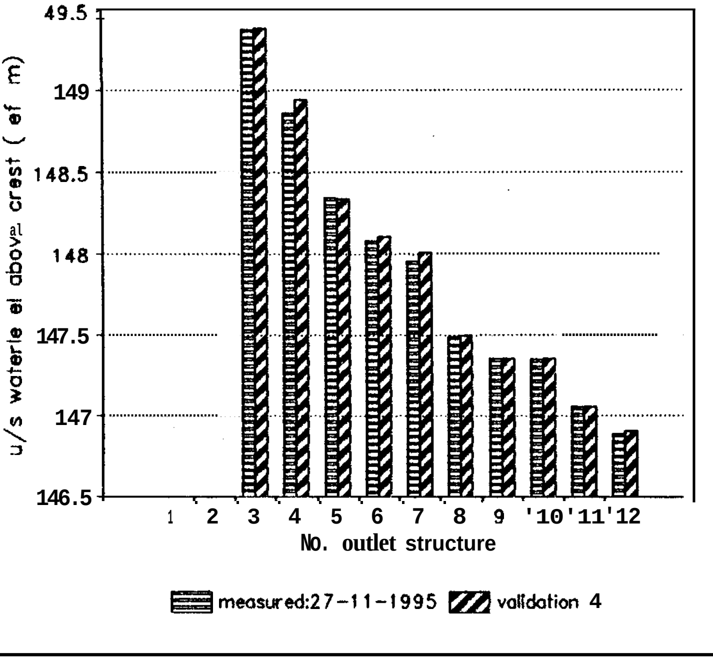

Figure 5.16 **Water levels along Masood distributary.**

5.6 Limitations using the hydrodynamic flow model SIC

Using models to predict physical processes one must always take into account that the output of the model is an approximation of the real process taking place. Besides that, a number of problems occurred using the SIC software modelling Masood distributary. Most of them are also mentioned by Hart (1996) and Litrico (1995).

- The number of nodes which can be entered in SIC is limited to 80. When modelling distributaries with a large number of outlet structures, it will not be possible to model the whole canal with SIC. At present, new versions of the software are available which can handle more then 80 nodes.

- In SIC, the transition between free open flow and orifice flow through outlet structures takes place for $h_1 = Y$ (see figure 5.2), with h_1 the upstream water level measured above the crest. However, in reality there will be critical flow above the crest of the weir, so the water level will touch the gate only when $Y = 2/3 . h_1$. Transition between free open flow and orifice flow will take place for $h_1 = 1.5$ Y. Practically, when for example in the field an outlet structure is functioning as a weir, it is possible that SIC computes the distributed discharge with the orifice flow equations.

- Within SIC, there is no difference between an OFRB and an AOSM outlet structure. As stated in chapter 4, there is a difference in hydraulic behaviour of an AOSM (improved APM) and an OFRB outlet structure which is not completely covered by the SIC structure equations. (1) The AOSM was so designed to have a constant discharge coefficient and a rounded roofblock to prevent the jet from contracting. In SIC however, the discharge coefficient for an AOSM or OFRB is depending on the upstream water level and is not constant. (2) The transition between free open flow and orifice flow for an OFRD is not continuous, although in SIC the transition is always continuous. This results in inaccurate prediction of canal water distribution for outlet structures within the region of transition from free open flow to orifice flow[6].

- Up to now, the computation stops whenever the tail of the canal (downstream boundary condition of the model) runs dry. As this is a common phenomenon in distributaries in the Punjab. this will create problems simulating actual discharge supplies to the distributaries. This problem can be tackled by moving the downstream boundary condition more upstream, as done with the SIC model of Masood distributary. Improvement is advisable at this point.

- Besides dry tail problem, the computation also stops whenever super-critical flow occurs. Especially running the model with low discharges at the head, super-critical flow is possible above the crest of drop structures.

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6 *The inaccuracies for OFRB outlet structures with the SIC equations compared with the 'theoretical equation' as stated in chapter 4, can be minimized by adjusting the initial discharge coefficient during the calibration phase of modeling canals. Additional study (Cemagref) proved that SIC seems to overestimate the discharge for free overflow and underestimates the discharge for (AOSM) orifice flow.*

CHAPTER 6 A METHODOLOGY TO STUDY THE CHARACTERISTICS DETERMINING CANAL WATER DISTRIBUTION

6.1 General

In this chapter, first a methodology will be defined, to study ***the impact on the canal water distribution, i.e. a redistribution of canal water to the outlet structures*** (= responsiveness of the system), based on ***adjusted canal and outlet structure characteristics*** (= sensitivity analysis). The sensitivity analysis will be conducted **for** different inflow at the head of the distributary using the SIC model of Masood distributary. Therefore, it will be necessary to defme a typical inflow pattern based on the actual variability of discharge at the head of the distributary. Secondly, a comparison will be made between the irrigation performance of the **actual** situation and the **design** situation of a distributary, based on the output of the SIC model of Masood distributary. It will be necessary to defme different irrigation indicators to be able to evaluate the irrigation performance. The aim of this chapter will be:

- to set up **a** methodology to analyse the responsiveness of the system, based on adjustments ***of*** the different parameters;
- to test the different suggested irrigation indicators, based on an analysis of the actual and design performance of Masood distributary;
- to define the different parameters, determining the canal water distribution.

6.2 Methodology of the sensitivity analysis

The methodology that has been used to determine the responsiveness of the system, i.e. the re-distribution of canal water is listed below and represented in figure **6.1.** The analysis is based on the output of the different simulations with the calibrated and validated SIC model of Masood distributary. The analysis is based on adjustments of different parameters determining the canal water distribution (the various input parameters of the model). ***Any*** adjustment will result in a change of canal water distribution, i.e. a re-distribution (dq). Compared with the initial output of the model (without adjustments: '0-option') an indicator will be used to quantify the impact on the canal water distribution. This indicator is called the ***Responsiveness Index.***

For a certain adjustment of a parameter, a **substantial impact'** [1] on the canal water distribution is expressed **in a high value for** the responsiveness index, and results in:

[1] *A 'substantial' and 'small' impact of the canal water distribution will be quantified in section 6.4.2.*

- the conclusion that the parameter have its impact on canal water distribution and will form an important input parameter for the simplified methodology to set up a flow model;
- the conclusion that the parameter will be interested for adjustment, in order to improve the canal water distribution.

On the other hand, for a certain adjustment of a parameter, **a small impact'** on the canal water distribution is expressed in a low value for the responsiveness index, and **results** in:
- the conclusion that the parameter has a limited impact on distribution and *can* be simplified within the simplified methodology to set up a flow model;
- the conclusion that the parameter will not be that interesting to adjust, in order to improve the canal water distribution, because the impact on distribution will be **limited.**

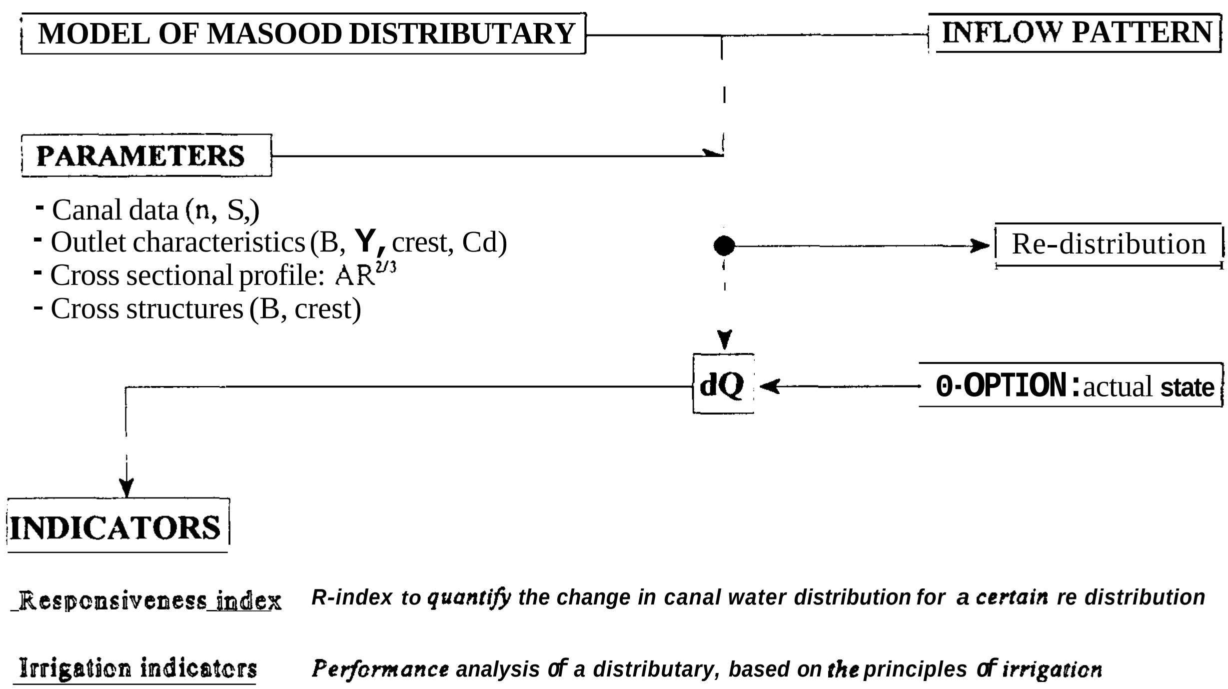


Figure 6.1 **Methodology of the sensitivity analysis.**

The following procedure is used:

- Run a steady state simulation with the SIC model of Masood distributary: inflow at the head **1.0** m^3/s. The steady state results **are** used **as** the *initial* input for the unsteady state computations. The different simulations will be carried out with the unsteady state module of SIC, based on a pre-defined inflow pattern. The inflow pattern **covers** the actual variability of inflow at the head of Masood distributary.
- Run an unsteady flow simulation of the actual situation, without **any** changes in the input parameters: so called **'0-option'.**

- Define both an indicator to express the ***rate of change*** in the output with respect to a rate of change of one of the parameters (responsiveness index) and several irrigation indicators to express the ***performance*** of both individual outlet structures as the total canal system, based on the design concepts of equity and proportionality.
- Define several scenario's in which the different parameters will be adjusted in a certain range and with a certain step function. A distinction has been made between a theoretical analysis of outlet structure characteristics (B, Y, C_d and Crest Level), and other parameters, further described in chapter 7.
- The impact on the canal water distribution will be analysed locally, i.e only a few outlet structures will be studied, or global: all outlet structures and tail.
- Results of the simulations: for different discharges at the head the steady state water profile ***after*** the adjustment is computed and the corresponding upstream water levels above crest can be determined. For different discharges at the head the canal water distribution ***after*** the adjustment is computed. The amount of canal water is re-distributed (dq).
- Adjusted parameters do have a high responsiveness, when there is a substantial impact on the water distribution. The behavior of an outlet structures is called sensitive, when there is a substantial impact on the water distribution: compute the responsiveness index (R-index). Based on the evaluation of the simulations, those parameters with a substantial impact on the canal water distribution can be distinguished and will be further analysed.

6.3 Settings of the Model of Masood distributary for the sensitivity analysis

6.3.1 Introduction

In order to apply the validated model of Masood distributary for the above described analysis the following remarks must be taken into account:

- to study the distribution for different inflow at the head of the distributary, the upstream boundary condition of the model must be re-defined;
- to extrapolate the results of the analysis to other distributaries, all types of outlet structures must be evaluated;
- for various discharges in the canal, the downstream boundary condition for the submerged outlet structures is changing;
- the initial input for the seepage losses is based on the 15-11-1995 field measurements;
- and the target discharges of the outlet structures are set at their authorized discharge.

In the next sections the points mentioned above are discussed more in detail.

6.3.2 Outlet structures

Flow condition

Based on the results of the validation it was found that the downstream water level for submerged outlet structure for different discharges in the parent canal is of great importance. As the model will be used in an unsteady flow situation, a constant downstream water level as downstream boundary condition for submerged outlet structures is not sufficient anymore. In order to simulate a change in water level in the watercourse based on a change in supplied discharge trough a submerged outlet structure, theoretical rating curves are developed (for outlet structures: 4, 5 and 6). A theoretical rating curve used as a ***user defined downstream boundary condition*** in SIC reads:

$$q(h_d) = q_0\left(\frac{h_d - h_{crest}}{h_0 - h_{crest}}\right)^n$$

Where:

$q(h_d)$	=	supplied discharge as a function of the downstream water level (watercourse)	[m^3/s]
q_0	=	measured discharge	[m^3/s]
h_d	=	downstream water level (watercourse)	[m]
h_{crest}	=	theoretical downstream crest elevation	[m]
n	=	coefficient	

The coefficient n will set at a value of 1.5, as we can schematize a watercourse as a rectangular constriction. The discharge through a submerged outlet structure with the theoretical rating curves will be iteratively computed, by solving the equation of the form:

$$q_T = F(h_1\ ,\ h_2(q_T))$$

q_T	=	Target discharge for the outlet structure	**[m^3/s]**
h_1	=	Upstream water level	**[m]**
$h_2(q_T)$	=	Downstream water level as a function of the discharge	**[m]**

In table 6.1, the input parameters for the rating curves are mentioned.

Table 6.1 Theoretical rating curves for the submerged outlet structures

Outlet structure	q_0 [m^3/s]	h_d [m]	h_{crest} [m] (= crest outlet)	n
4.	0.062	0.67	148.133	1.50
5.	0.020	0.49	147.781	1.50
6.	0.028	0.56	147.392	1.50

It can be stated that the rating curves do not compute the proper downstream water levels. This is due to the fact that the downstream water levels not only depending on the discharge fluctuations in the parent canal, but are correlated **with** the dynamical behavior in the watercourse. ***Anyway, the analysis will be conducted with the validated model with the theoretical rating curves.*** The dynamical behavior of a watercourse is almost impossible to model but at least the impact of submerged outlet structures is obtained.

Types of outlet structures

The proposed method of simulating the impact on water distribution with theoretical rating curves for the submerged outlet structures **(4, 5** and **6) does** not take into account the impact on water distribution with other types of outlet structures, i.e. (OC)AOSM and OF outlet structures. In order to evaluate the impact on the water distribution for different **types** of outlet structures, **based** on the proposed adjustments, a theoretical analyses will be conducted. In the end the supply of canal water is determined by the upstream water level above the crest and the discharge equation for a **certain** type of structure. The analysis will be discussed more in detail in chapter 7.

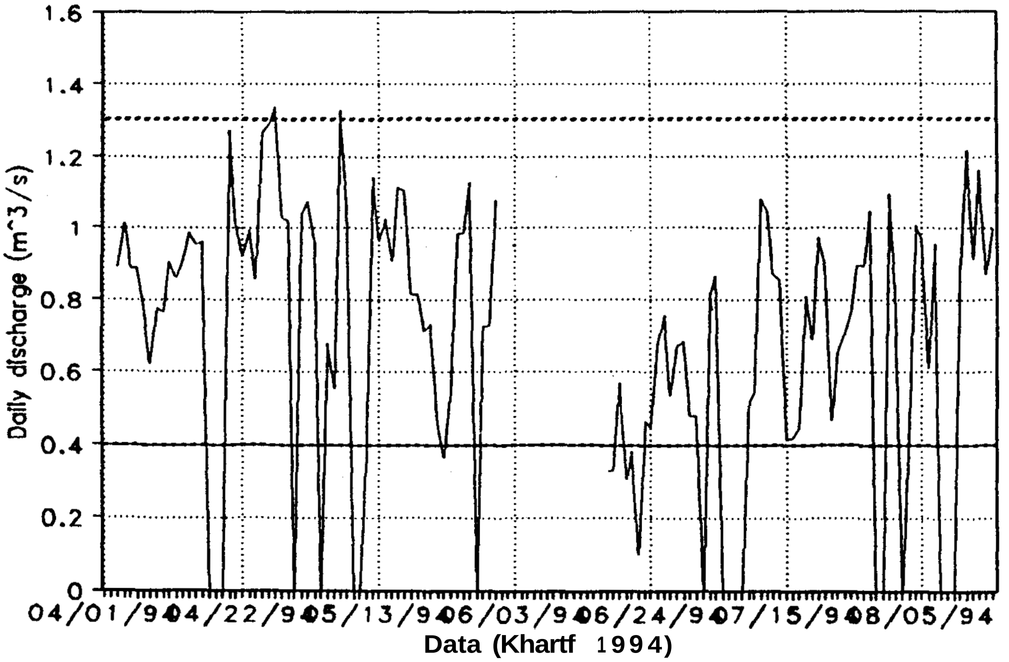


Figure 6.2 **Daily discharges of Masood distributary for Kharif 1994 (01/04/1994 - 05/08/1994).**

6.3.3 Canal water inflow: upstream boundary condition

The high variability of **canal** water supply to the tertiary outlet structures is related to upstream fluctuations at the main system level. It has already been stated that due to operations at the primary level, the inflow of canal water to the distributaries is characterized by daily fluctuations.

The inflow pattern of Kharif **1994** of **Masood** distributary was used in order to derive a typical inflow to run the analysis with the unsteady state unit of SIC (figure 6.2). The computed discharges are based on daily water level measurements both upstream and downstream of the fully submerged gated regulator at the head of **Masood** distributary. The calibrated discharge coefficient of the regulator used for computing the discharge[2] (C, = **0.48)** is almost similar to the value found during the field measurements in November **1995** (C, = **0.45).**

Maximum and minimum discharges

Besides a few measurements in Kharif **1993,** the inflow ranges in between approximately **0.4** m^3/s and **1.3** m^3/s. Besides that, to simulate the SIC model, no ***dry tail*** and no ***bank overtopping*** may occur. Based on that, the inflow is pre-defined at **0.5** m^3/s to 1.2 m^3/s, which determines the boundaries of the simulated inflow pattern. Besides the minimum and the maximum values of the inflow, a certain step function with a certain duration must be introduced in order to simulate different typical discharges in the canal.

Step function

To cover a wide range of different discharges on the canal, and study the corresponding distribution, a step function of 0.10 m^3/s will be suggested. This means that for **8** different values for the inflow at the head, the distribution of canal water can be studied.

Duration

The ***duration*** T is defined **as** the time between two different discharges at the head of the distributary. **A** change of inflow results definitely in a change of water distribution **to** the outlet structures. In order to study the a constant supply of canal water to an outlet structure, based on a certain inflow at the head, **a** steady flow at the canal is necessary. In principle, a change in discharge is followed by a wave in the canal with a certain ***travel time*** T_w, defined **as** *(Ankum,* **1995):**

$$T_w = \frac{L}{c + v_0} = \frac{L}{\sqrt{g y_0} + v_0}$$

Where:

T_w	=	Travel time of the wave	[s]
c	=	Wave velocity	[m/s]
y_0	=	Previous water depth in the canal	[m]
v_0	=	Previous flow velocity	[m/s]
L	=	Length of the canal reach	[m]
g	=	Gravitational acceleration	[9.8 m/s²]

2 ***IIMI field measurements***

The travel time of a wave does not take into account the time necessary for filling or emptying the in-canal or dynamical storage, i.e. the volume of water add or released after a change in discharge to require the new steady state situation. In principle, the ***response time T_r*** of a system is the time required for a canal system to transit from the previous steady state into the new steady state situation, can be approximated by (Ankum, 1995):

$$T_r = \frac{2.V_{dyn}}{Q_n - Q_0} - T_w$$

Where:

T_r	=	Response time of the canal reach	[s]
V_{dyn}	=	Volume of dynamical storage (storage wedge)	[m^3]
Q_n	=	New discharge	[m^3/s]
Q_0	=	Previous discharge	[m^3/s]

The theoretical concept of travel time and response time of a canal reach is listed in figure 6.3.

In order to require a proper steady state situation in the canal for each typical inflow at the head: T > T,. The response time of the system increases when the change in discharge incrcascs. During thc simulation inflow, the largest change in discharge reaches 0.6 m^3/s (from **0.5** m^3/s to 1.1 m^3/s). Therefore, the response time of the canal for a change in discharge (from 0.5 m^3/s to 1.3 m^3/s) is calculated.

For both a steady state condition with an inflow of 0.5 m^3/s and 1.3 m^3/s at the head of Masood distributary, the storage of water in the canal was calculated.

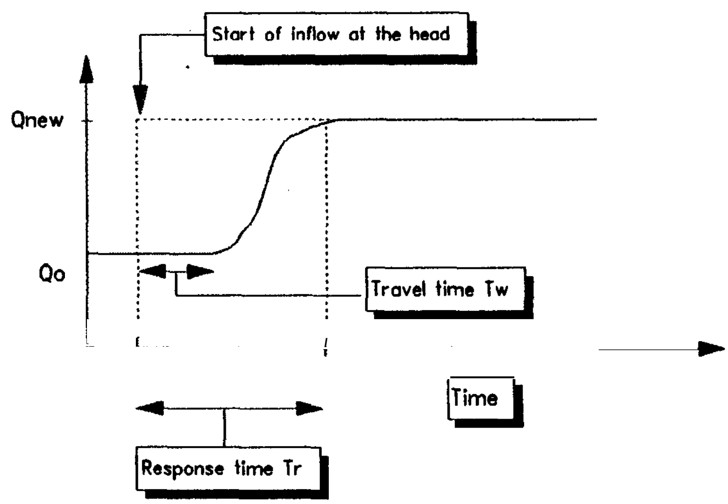


Figure 6.3 Discharge/time diagram for the tail of the canal: Travel and response times in a canal reach.

$$V_{dyn} = V_{1.3m^3/s} - V_{0.5m^3/s} = 33194.2\ m^3 - 17739.7\ m^3 = 15454.5\ m^3$$

With an average water depth $y_0 = \mathbf{0.40}$ m and an average flow velocity $v_0 = 0.27$ m/s, the T_w becomes about **5044** s (from head to tail). Given the T_w and the T_{syn}, the T_r for a change of discharge for the Masood distributary becomes 33592.25 s: 9.3 **hours.** Based on the computed response time, the duration of each steady state simulation is fixed at **24 hours.**

Considering the maximum inflow, the minimum inflow, the step function and duration; the simulations with Unit II of SIC are based on an inflow pattern listed in figure **6.4.**

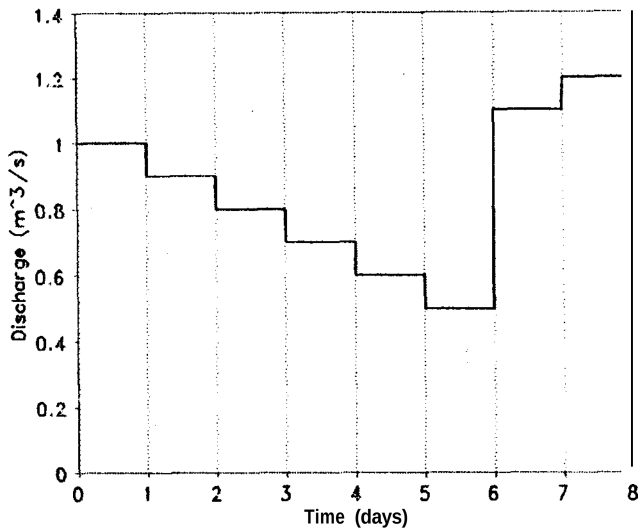


Figure 6.4 **Inflow pattern used for the simulations.**

6.4 Evaluation of the analysis

6.4.1 General

As the investigation is based on two major parts, i.e. **(1)** the analysis of the responsiveness of the system and (2) the impact on the operational performance of a distributary, for a certain change in a parameter determining the canal water distribution, indicators have to be introduced. For the analysis of the responsiveness, a so called Responsiveness Index will be suggested.

It has already been stated that special requirements can be formulated, related to the distribution of irrigation water in Pakistan, sufficient distribution is the one which can meet the following requirements:

- adequacy to meet the targets;
- efficiency losses at a minimum;
- reduced variability of water flow indicates the reliability of the system;
- equitable distribution among the beneficiaries;
- proportionality of the distributed water flow.

In order to decide whether the system performance, i.e. the canal water distribution along a distributary, is acceptable, it is necessary that an agreed and rational set of indicators be identified, based on the specified principles for irrigated agriculture in Pakistan. Using the indicators, the effectiveness of the present operation of the distributary, a used management tool (maintenance), or different suggested improvements can be evaluated. Evaluation has to be made with reference to a certain base, i.e. the authorized design targets for the outlet structures, reflecting the proposed equitable and proportional water distribution. In this study, indicators based on the volume concept will be used: a quantitative analyses of water need and supply.

6.4.2 Responsiveness index

The responsiveness of the system is the rate of change in the output, i.e. computed discharges to the outlet structures, with respect to a rate of change in the value of the parameter while keeping the other parameters constant (McCuen, 1973). The index R, proposed by Loomis (Maheshwari et al, 1990), was adopted for this study and is calculated as:

$$R = \frac{100}{N} \sum_{i=1}^{N} \frac{(X_{ni} - X_{ci})}{X_{ci}} \cdot \Delta^{-1}$$

Where:

N	=	Number of points in an output, i.e. number of outlet structures (local analysis means: N = 1);
X_{ni}	=	New value of output for the ith point after a simulation with an adjusted parameter;
X_{ci}	=	Value of output for the ith point for the '0'-option;
A	=	Absolute value of change of a parameter, expressed as a percentage of its value for the '0'-option.

Actually, the expression of R is a measure of elasticity, i.e. the percentage change in the output referred to the '0-option' based on a 1% change in the value of an input parameter. For *example:* R = + ***0.3 would mean that for + 1% change in the input, the output increases by 0.3%, and R = - 0.3 would mean that for + 1% change in the input, the output decreases by 0.3%.***

Example

For the simulation of the '0-option', i.e. no change of one of the input parameters. the output of outlet structure 5 is: 0.038 m³/s. In the sensitivity analysis. one of the input parameters will be adjusted. For example, the width of the drop, just downstream of oritlet structure 5, will be increased with 25% (from 3.28 m to 4.10 m). After simulation of the 'new' situation, the distributed discharge to oritlet structure 5 has become: 0.035 m³/s. The R-index reads (with N = 1. local analysis):

$$R = \frac{100}{1} \sum_{i=1}^{1} \frac{(0.035 - 0.038)}{0.038} \cdot (+25)^{-1} = -0.32$$

R = -0.32 means: +1% change of the input (width of the drop structure) results in -0.32% change (decrease) in the output (decrease of distributed discharge to outlet structure 5).

The R-index will be used to quantify the impact of a certain change in the input parameter. The change in an input parameter will have a substantial impact on the distribution when the R-index > 0.5 and a small impact when the R-index < 0.5. The characteristic difference of the R-index values found in the sensitivity analysis: the computed R-index values ranged in between 0.05 to 0.40 and 0.65 to 1, so the difference between a sensitive and an insensitive parameter is based on the R-index value of 0.5.

Classification: **R < 0.5:** low responsiveness, **R > 0.5:** high responsiveness.

6.4.3 Irrigation indicators

Adequacy and Efficiency

The performance of an off take (secondary and tertiary) can be described by considering three volumes of water (Schuurmans, 1991):

1. The intended volume of water (V_i), the volume of water in m' per irrigation period, to be supplied to the off take. Here, the volume is based on the authorized design volume (q_{auth} * T). The intended volume of water is always described within a allowable range of variation in flow rate.
2. The effective volume of water (V_e), the volume in m^3 per irrigation period, which is effective, i.e. the moment of supply is within the defined period and flow rate, with respect of response time and operational losses (Ankum, 1995).
3. The actual supplied volume of water (V_a), the volume in m^3 per irrigation period, actually supplied to an off take.

In this study the effective volume is equal to the supplied volume, because due to water shortage, all the water supplied to an off take will be used by the farmers. There is no minimum or maximum range in between the supplied water flow is effective. The ratio actual supplied over intended has many applications (Bos et al, 1990), for instance the division of flow over a scheme and the performance of water distribution to the tertiary outlet structures. Based on the above defined volumes of water, two performance indicators can be distinguished:

$$DPR = \frac{V_e}{V_i} * 100\% = \frac{V_a}{V_i} * 100\%$$

$$e_o = \frac{V_e}{V_a} * 100\% = 100\%$$

Where:

DPR	=	Delivery Performance Ratio	[-]
e_0	=	Operation efficiency (= 100%. in this case)	[-]
V_e	=	Volume effectively delivered = Volume actually delivered	[m^3]
V_i	=	Volume intended to be delivered	[m^3]
V_a	=	Volume actually delivered	[m^3]

The e_0 determines the operational losses at the off take. The operational efficiency, due to a lack of sufficient water supply, is 100%. The DPR determines the quality of the actual supplied amount of water, i.e. has the flow satisfied the effective flow. The hydro-dynamical performance of an off take is described by both the DPR and the e_0 (Ankum, 1995).

Also the overall performance for example of a distributary *can* be expressed by the above mentioned indicator: DPR_{sys}.

$$DPR_{sys} = \frac{\sum V_e}{\sum V_i} * 100\ \% = \frac{\sum V_a}{\sum V_i} * 100\%$$

Studies in the Chistian sub-division, conducted by IIMI, has shown the relation between the DPR at the head of the distributary and the DPR of individual outlet structures: the distribution to the outlet structures is more correlated to the distribution to the distributary for head reach outlet structures, then for more downstream tail outlet structures (Wahaj, **1995).**

Performance classes DPR: **0.90 - 1.10:** good; **0.75 -** 0.90 and **1.10 - 1.25:** fair; **< 0.75** and **> 1.25:** poor.

Variability and Reliability
The above mentioned indicators do not tell anything about the uniformity of the supply in relation to the design discharge over a specific period of time. The variability and reliability of the distribution *can* be expressed **as** the Coefficient of Variation of the DPR (Wahaj, **1995).** The **mean is** the centre of gravity of the distribution density function and the variance is **a** measure of the spread of the observations, The ratio of the standard deviation over the mean is known **as** CV, Coefficient of Variation. The CV(DPR) is similar to the dependability indicator, P_D, given by Molden and Gates **(Rao, 1993).**

$$P_D = \frac{1}{T} \sum CV_T(DPR)$$

Where:

DPR	=	Delivery Performance Ratio
CV_T	=	Temporal coefficient of variation (ratio of standard deviation to ***mean)*** over time period T

The DPR values are either based on an individual outlet structure (local), or all outlet structures of a distributary (global). Vander Velde **(1991)** showed that the variability of **canal** water supplied **to** the tertiary outlet structures along a distributary, indicated by the CV of discharges, increased more downstream. The increasing CV indicates the increased variability of supplied discharges to outlet structures with increasing distance from the distributary head.

Performance classes: ***0.00* - 0.10:** good, low variability; **0.11 - 0.20** fair; **>0.20:** poor, high variability **(Rao, 1993).**

Equity

The equity of water distribution is an expression of the fair share for each farmer or group of farmers. As expressed by Kuper and Kijne (1992), the fairness of a share may be based on legal water rights or on the delivery of a fixed rotation of a water supply based on extend of the irrigated land served by each (focussing on individual outlet structures: authorized discharge). A system that is considered fair by most farmers is more likely to be productive and efficient than one that the state has designed on basis of productivity and efficiency but which is considered unfair by the farmers (Levine and Coward, 1989). Besides the authorized discharge, focussing on the distributary level, equity may also be expressed as defined by Bos et al (1994). Equity indicator: Modified **Interquartile Ratio (MIQR)** is the ratio between the average DPR of the best 25% of the system and the average DPR of the worst 25% of the system.

$$MIQR = \frac{\frac{1}{n}\sum DPR_{best 25\%}}{\frac{1}{n}\sum DPR_{worst 25\%}}$$

Performance classes: 1.00 - 1.50: good; 1.50 - 1.75: fair; >1.75: poor (Bos et al, 1994).

In fact, the variability indicator P_d can also be used as a measure for the equity (IIMI) of the distribution along a distributary, when computing the CV(DPR) of al outlet structures along a canal, for a certain inflow. The difference between equity expressed as the intended volume (authorized discharge) of canal water and the CV(DPR) or MIQR indicators is based on the fact that the authorized discharge is the equitable amount of water for each individual outlet structure and the CV(DPR) and MIQR indicators expresses equity for a whole distributary. Practically, there could be an equitable distribution when all outlet structures receiving canal water less or above authorized discharge.

Proportionality

The proportionality of the water flow distribution *can* be expressed by the sensitivity factor of a bifurcation. Outlet structure behavior is fully proportional when the sensitivity factor S = 1, sub-proportional when S < 1, and super-proportional when S > 1. When S << or >> then 1 proportional control of the off taking outlet structure is not obtained. As fully proportionality only *can* be reached in one point (for the ongoing canal and an offtaking outlet structure), the S_{outlet} for FSD in the canal (1 m^3/s) is calculated with the equation for the R-factor:

$$S = \frac{100}{N} \sum_{i=1}^{N} \frac{(X_{ni} - X_{ci})}{X_{ci}} \cdot \Delta^{-1}$$

Where:

S	=	Sensitivity ratio;
N	=	1;
X_{ni}	=	q outlet structure for Q = 0.9 m^3/s;
X_{ci}	=	*q* outlet structure for Q = 1.1 m^3/s;
A	=	- 18.1818 %.

Fully proportionality means: 1% change in discharge at the head of the distributary results in 1% change in supplied discharge to the outlet structure. Also the overall proportionality for example a distributary can be expressed by the above mentioned indicator: S,

$$S_{sys} = \frac{\sum S_{outlet}}{n_{outlets}}$$

Where:

n = number of outlet structures

Performance classes: 0.85 - 1.15: good, fully proportional; 0.70 - 0.85 and 1.15 - 1.30: fair, (sub / super) proportional; and < 0.70 and > 1.30: poor.

6.5 Comparison between the actual and design performance of a distributary

This section presents the results of the analysis described in section 6.3 for the actual situation and the design situation of Masood distributary. The actual situation means the validated model with the theoretical rating curves for the submerged outlet structures (the output of the model is equal to the '0-option' results). The output of the design model can only be used for a theoretical interpretation. The design characteristics of a distributary are based on the initial outline of the canal. It takes many years before the canal is in regime, actually the outline of the canal is always changing due to siltation, erosion and physical adjustments in time. The aim of this section is:

- to obtain a better understanding of the performance of a distributary based on proportionality and equity;
- to evaluate the proposed irrigation indicators;
- and to present the outlet structures which will be studied more in detail during the analysis.

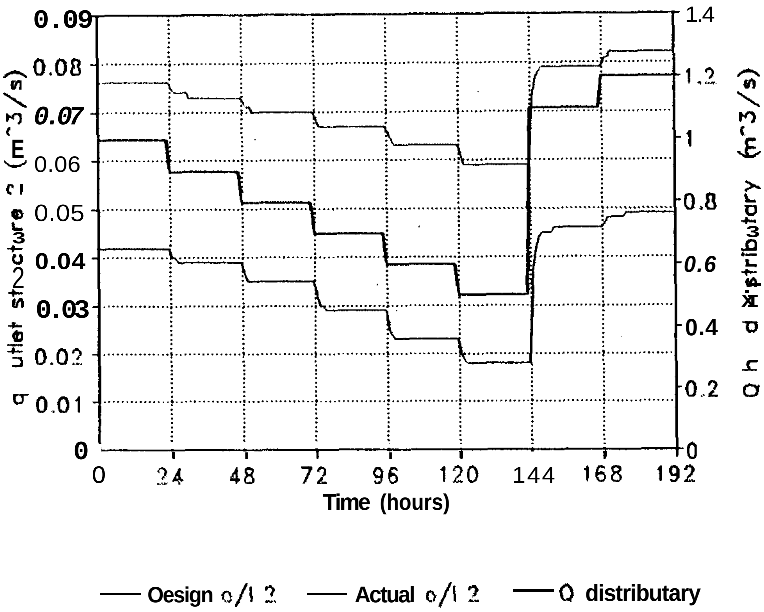


Figure 6.5 **Supply of canal water to outlet structure 2 for both design and actual state of Masood distributary, and inflow at the head of the distributary.**

To have a clear look at the graphics and to reduce time in analysing data, the presentation of the comparison of actual and design performance will be conducted for 7 outlets only:

- ***Headreach:*** outlet structure **1** and **2** (OFRB, o.m.), **4** (PIPE, o.n.) and 5 (PIPE, o.n.);
- ***Middle reach:*** outlet structure 6 (OFRB, o.n.) and **10** (OFRB, o.m.);
- ***Tailreach:*** outlet structure **12** (OFRB, o.m.).

As described in section 6.4, the analysis will be conducted with a pre-defined inflow pattern at the head of the distributary based on (1) the limits of inflow, i.e. dry tail problems and over topping, and **(2)** a time step between a change in inflow to establish different steady state situations in the canal for different inflow at the head. In figure **6.4** both the inflow at the head of Masood distributary and the supplied discharge (actual and design) to outlet structure 2 is shown. Thus, for the design discharge of **1** m^3/s at the head of Masood distributary, the supplied discharge to outlet structure 2 is: 0.042 m^3/s (authorized discharge in the design situation) and **0.076** m^3/s for the actual situation. The change in discharge at the head of the distributary is reflected in the change in supplied discharge to the outlet structure.

First thing that can bc concluded is at present, the outlet structure is receiving far to much water compared with the authorized discharge. Main cause is the remodelling of this structure: width B +3.5 cm and opening heigh Y +11 cm (see table **5.3**).

In table **6.2,** the supplied discharge as a function of the discharge at the head of Masood distributary is listed. **As** already mentioned in chapter **4,** in the end the supplied discharge is depending on the upstream water level above the crest of an outlet structure.

The relation between discharge at the head (Q) and supplied discharge to the outlet structure (q) reflects the typical discharge curve related to type and flow condition of the outlet structure, and is presented in figure 6.6.

Table 6.2 Supplied discharges (outlet structure no. 2) as a function of discharge at the head in m^3/s

Q_{head}	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2
q_{Design}	0.018	0.023	0.029	0.035	0.039	0.042	0.046	0.049
q_{Actual}	0.059	0.063	0.067	0.07	0.073	0.076	0.079	0.082

The results can be plotted in a graph which the x-axis coutains the discharge at the head of the distributary and the y-axis the supplied discharge to the outlet structure. The graph presents information on both the ***equity*** as the ***proportionality*** of an outlet structure, at **FSD of** the canal.

- **100%** proportionality: the tangent at the curve for Q = 1 m^3/s (FSD) crosses the origin.
- **100%** Equity: for Q = 1 m^3/s (FSD), the supplied discharge equals the authorized discharge.

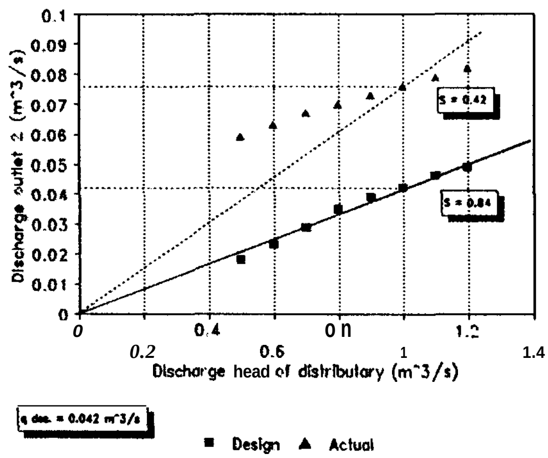


Figure 6.6 Relation between Q and **q,** for outlet structure **2** (both design and actual state).

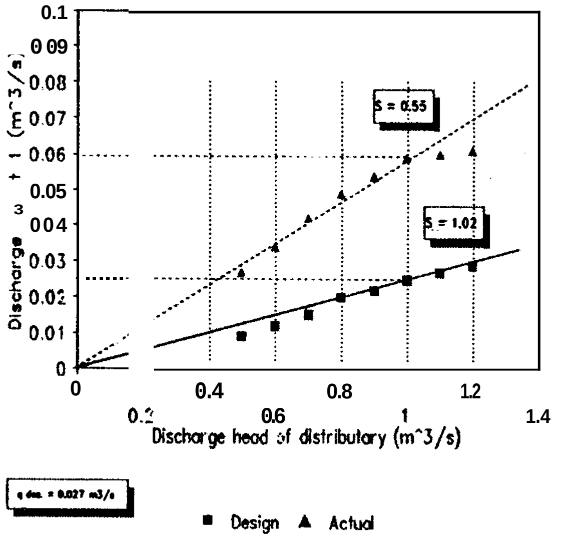
proportionality

The lines in the graph are representing the theoretical tangents for both the actual **as** the design situation for fully proportional behavior of the outlet structure. The actual tangents are represented by the sensitivity factor S. Actual situation: S = 0.42 (no proportional behavior), actual tangent crosses the y-axis. Design situation: S = 0.84 (fair (sub)proportional behavior), actual tangent crosses the y-axis. In case of (super)proportional behavior **(2.25** < *S* < *1.30*), the tangent crosses the x-axis.

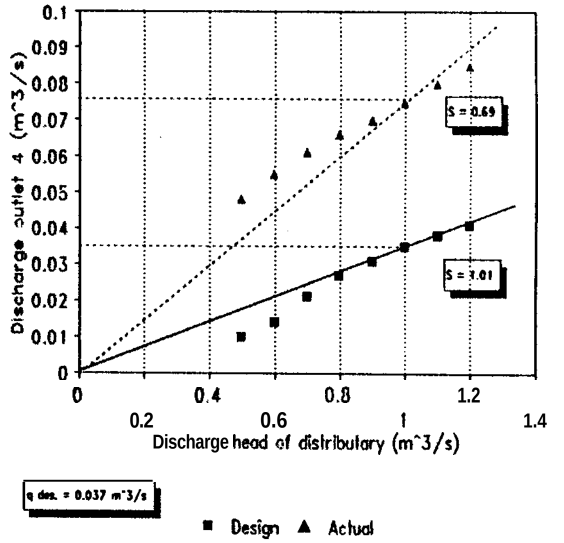
equity

For both the actual **as** the design situation, the supplied discharges to the outlet structure with the canal running on FSD (1m^3/s) are listed in the graph. Based on the principles of equity expressed in an authorized discharge for each outlet structure the following can be concluded. Actual situation: q = 0.076 m^3/s, which is >> $q_{auth.}$ = 0.042 m^3/s, so no equitable distribution. Design situation: q = 0.042, equals the $q_{auth.}$ = 0.042 m^3/s, so 100% equitable distribution. The graph is also printed for the other 6 outlet structures.

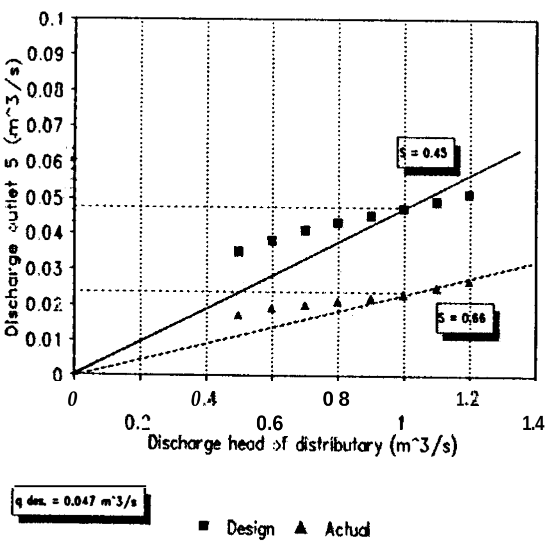
The next 6 graphs presenting the same curve as figure 6.6 for outlet structure 1, 4, 5, 6, 10 and 12, both the actual and the design situation.



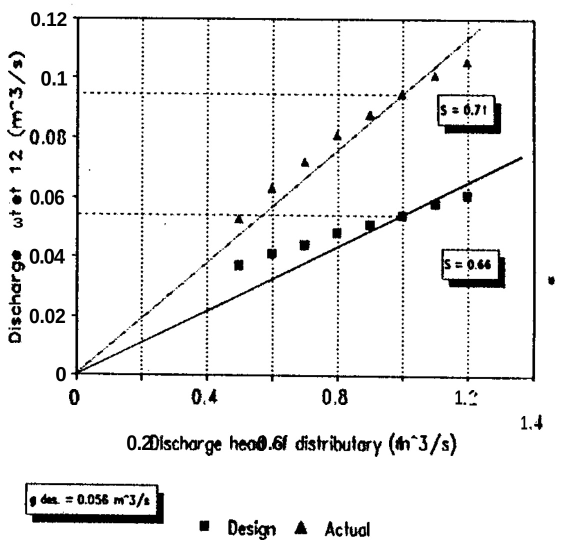
outlet structure 1



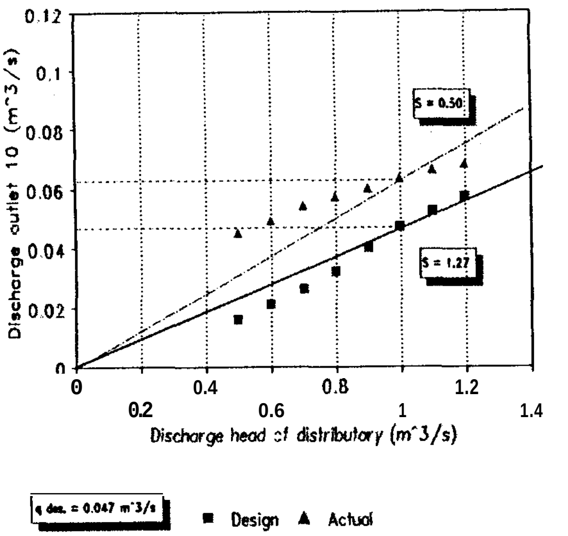
outlet structure 4



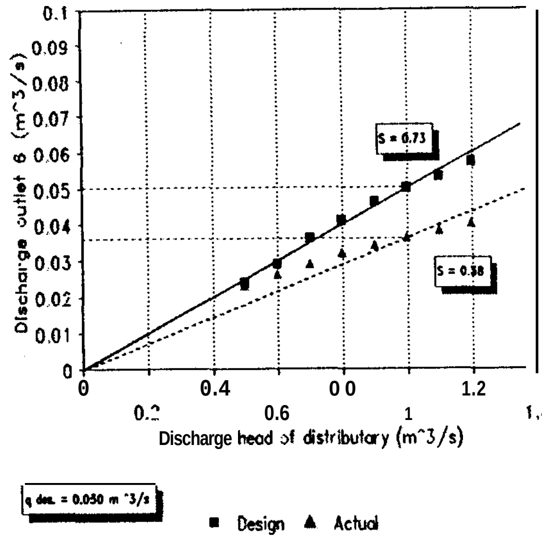
outlet structure 5



Outlet structure 6



Outlet structure 10



Outlet structure 12

and the actual state (100% FSD).

Outlet structure	q m^3/s		S		DPK		CV (DPR)		$Q_{tot.}$ m^3	
	Des.	Act.	Des.	Act.	Des.	Act.	Des.	Act.	Des.	Act.
1	0.025	0.059	1.02	0.55	0.93	2.19	0.30	0.21	13710	38802
2	0.042	0.076	0.84	0.42	1.00	1.81	0.26	0.10	24170	56506
3	0.029	0.060	0.71	0.44	0.94	1.94	0.23	0.10	16971	44127
4	0.035	0.075	1.01	0.69	0.95	2.03	0.35	0.16	18703	54283
5	0.047	0.023	0.45	0.66	1.00	0.49	0.11	0.13	30229	17440
6	0.050	0.036	0.73	0.58	1.00	0.72	0.24	0.15	28955	25933
7	0.024	0.047	1.06	0.45	0.96	1.88	0.34	0.11	12770	34395
8	0.049	0.066	0.74	0.48	0.94	1.27	0.23	0.11	28514	48555
9	0.060	0.074	0.85	0.50	1.03	1.28	0.23	0.12	35023	53304
10	0.047	0.063	1.27	0.50	1.00	1.34	0.35	0.12	24979	45866
11	0.050	0.064	0.90	0.80	0.98	1.25	0.21	0.21	29682	44581
12	0.054	0.095	0.66	0.71	0.96	1.70	0.14	0.19	33839	66247
Tail	0.449	0.299	1.11	1.87	1.12	0.75	0.23	0.51		

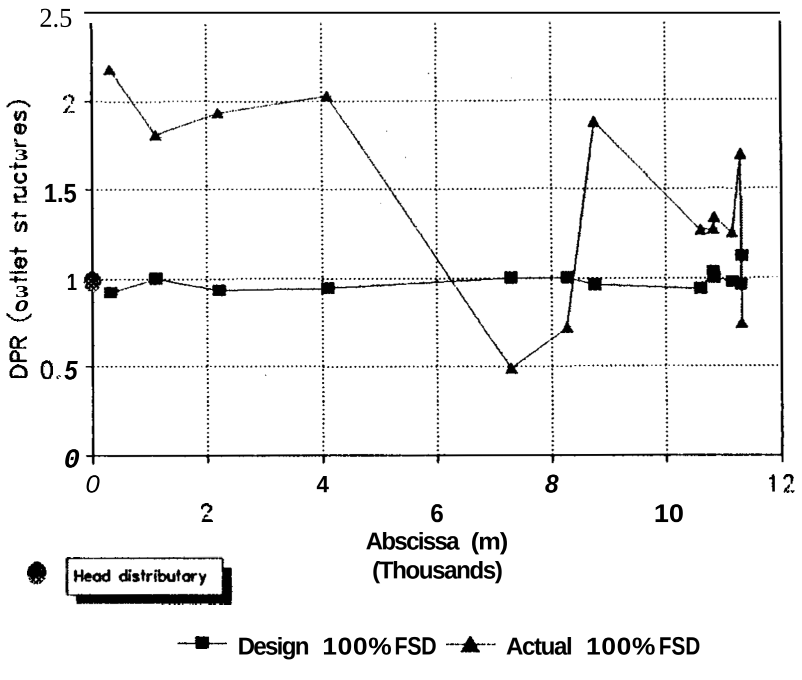


Figure 6.7 DPR values along the distributary (100% FSD).

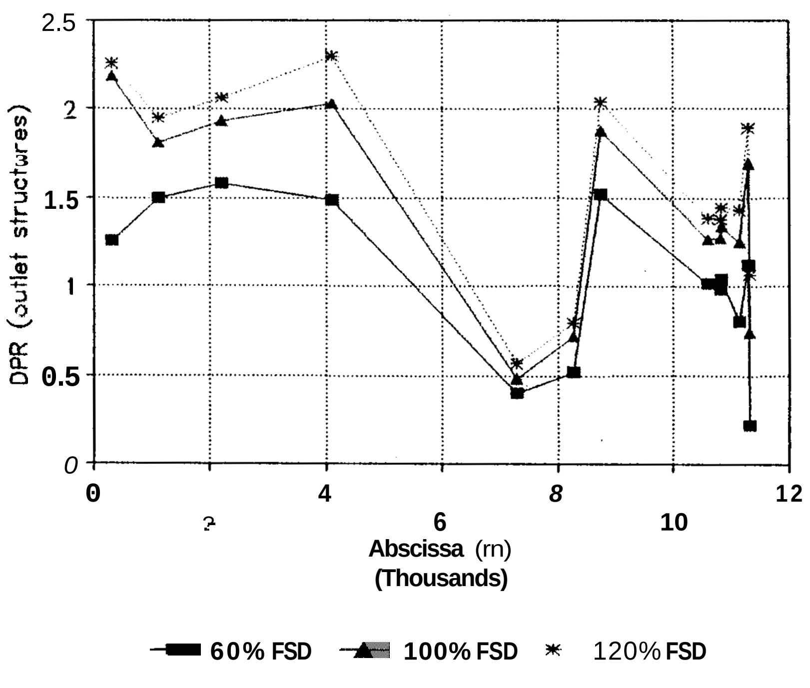


Figure 6.8 DPR values along the distributary for different discharges at the head.

In table 6.3 on page 79, the hydraulic behavior for both the design and the actual state of the distributary, for all outlet structures is listed. As the downstream boundary of the model of Masood distributary is fixed at RD 37.25, and at present the tail outlet structure is located at RD 50.20, also the 'performance' of the tail is taken into account. In the design state with the canal is running on FSD, the supplied discharge at the tail equals 0.4 m^3/s, in order to distribute canal water to the 5 tail outlet structures.

Supplied discharge

With the canal at FSD, generally it can be stated that, except for the submerged pipe (5), OFRB (6) and the tail, the supplied discharges to the outlet structures for the actual state is far above the equity based authorized discharge (supplied discharges in the design state). The main reasons are:

- the actual full supply level is substantially above the design level for approximately the upper 75% of the canal, due to much siltation at the head of the distributary (in many cases crest level < bed level canal), i.e. an increase of h_u above crest;
- increase of canal water supply upstream causes water shortage at the tail;
- remodelling of outlet structures: outlet structure 2, 4, 5, 6, 7, and 12.
- change in flow condition (free flow to submerged flow) due to siltation in the watercourse.

Sensitivity

Looking at the sensitivity ratio for outlet structures with the canal at FSD it can be stated that for the design state, most of the outlet structures do have proportional behavior. At present almost all outlet structures are non-proportional, according the proposed classification. Main reasons:

- change in outlet structure settings (crest elevation referred to bed level, B and Y);
- change in the cross sectional profile ($AR^{2/3}$) of the canal due to erosion, sedimentation and bank cuts.

Table 6.4 Performance evaluation on proportionality

Performance classes	Design situation: outlet No.	Actual situation: outlet No.
0.85 - 1.15: good, fully proportional	1, 4, 7, 9, 11, tail	11
0.70 - 0.85 / 1.15 - 1.30: fair, (sub/super)proportional	2, 3, 6, 8, 10	-
< 0.70 and > 1.30: poor, no proportionality	5, 12	1, 2, 3, 4, 5, 6, 7, 8, 9, 10, tail
S_m	0.87 : proportional	0.67: no proportionality

Delivery Performance Ratio

The adequacy of the distribution of canal water for the actual and design situation as discussed above can be expressed in the Delivery Performance Ratio for both individual outlet structures and the total distributary. In figure 6.7 the DPR value for each outlet structure is plotted in a graph, With the canal running on FSD. From this graph the inadequate distribution can be extracted easily. Figure 6.8 presents the same DPR curve for 100% FSD, but also the curve for 60% FSD and 120% FSD. From these graphs it can be concluded that:

- at present there is an inadequate distribution of canal water to the outlet structures;
- submerged structures 5 and 6 suffer from a lack of water, and are quite insensitive to a change in discharge in the canal;
- the supply of water is decreasing more downstream of the distributary: the tail-enders suffer the most, especially when the discharge drops in the parent canal;
- for the design situation the DPR value is almost 1, i.e. the supplied discharges equals the authorized discharges for outlet structures;
- a drop in discharge at the head resulting in a drop in supplied discharge to outlet structures, when in the end, the supply to the tail outlet structures fall dry.

The relation between the actual supply and the authorized supply to outlet structures is dominantly a function of the physical distance between an outlet structure and the head of a distributary. The characteristic distribution pattern of the two graphs (high DPR at the head and decreasing DPR in flow direction) are similar to earlier studies at the distributary level, conducted by IIMI in 1987, in the Lower Chenab Canal system in the Punjab (Bhutta, Vander Velde, 1987).

Table 6.5 Performance evaluation on adequacy of supply

DPR performance (at FSD)	Design situation	Actual situation
DPR_m	99 % : good	144 % : poor
E-index (Hart, 1996)[3]	97 % : good	92 % : good

3

Hart is using a parameter with the ratio of effectively supplied discharge to the sum of the authorized discharge of the outlet structures. Per definition. this ratio has a maximum of one (ideal performance). If qi < qi, authorized then qi, effective = qi else qi. effective = qi, authorized:

$$E = \Sigma \frac{q_{i,effective}}{q_{i,authorised}} * 100\%$$

It can be concluded that the evaluation of the adequacy performance proposed by Hart (1996) in a study of different maintenance methods at the distributary level, is not sufficient in this case. Due to removed outlet structures and a shorted tail, the E-index assumes that there is a good performance on adequacy, with the canal at FSD. Partly it is true, but it does not take into account the actual irrigation practice, i.e. removed outlet structures, tail shortening, closed branches and illegal practices (supply at the head greater then FSD). The E-index seems to be good, but the reality is far from acceptable. That is why the simple DPR indicator as proposed in the previous section is preferable. ***The adequacy is not sufficient, i.e. overall supply is higher then authorized.***

Coefficient of variation of the Delivery Performance Ratio

In order to skip the problems with the above mentioned $DPR_{sys.}$ (supply of all water to one outlet structure only, and the indicator seems to be good !), the proposed indicator of Molden and Gates was suggested. The Coefficient of Variation reflects an unequal distribution to an outlet structure, i.e. it is an expression of its variability. Both local as global the CV(DPR) is computed, see figure 6.9 and 6.10. The computed CV(DPR) for each outlet structure (local) is not very much reliable: (1) to compute a CV of 4 situations only is not very precise, and (2) the computed DPR for different discharge in the canal are based on model output, and not on measured values. Due to the fact that at present more outlet structures are sub-proportional, the actual CV(DPR) is lower then for the design situation. At present, disturbances are transplanted towards the tail (tail: CV(DPR)= 0.51).

Looking at figure 6.10, the global CV(DPR) values are printed for different discharges at the head of the distributary.

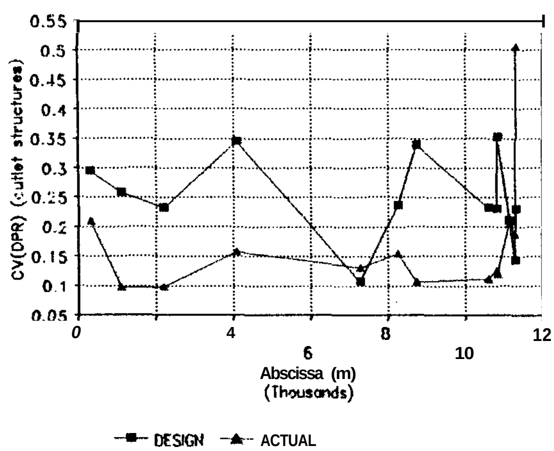


Figure 6.9 **Local CV(DPR) for actual and design state of the distributary.**

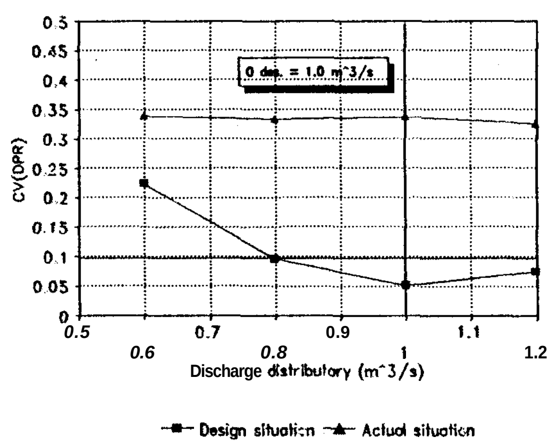


Figure 6.10 **Global CV(DPR) for actual and design state of the distributary.**

For the design state of the distributary, it can be concluded that the distribution becomes more variable when the discharge at the head drops. Minimum variability, performance class: good (0 - 0.1), is obtained for the design state of the distributary when running on 100% FSD (1 m^3/s). At present, the global CV(DPR) is not depending on the inflow at the head of the distributary.

Table **6.6** Performance evaluation on variability of supply

Performance indicator	Design situation	Actual situation
CV(DPR) 60% FSD	0.22	0.34
CV(DPR) 80% FSD	0.10	0.33
CV(DPR) 120% FSD	0.07	0.33
CV(DPR) FSD	0.05: good variability	0.34: poor variability

Equity
As equity can be expressed local for each outlet structure by means of the authorized discharge, nothing is mentioned about the equitable performance of an distributary. The distribution of canal water can be equitable, although the individual outlet supplies are less or more then authorized discharge. Therefor, besides the authorized discharge, the Modified Inter Quartile Ratio (MIQR) will be used. In figure **6.11** the MIQR is printed as a function of the discharge at the head of the distributary.

It can be concluded, that the performance becomes more inequitable when the discharge at the head drops. Maximum equitability is obtained for the design state of the distributary when running on FSD.

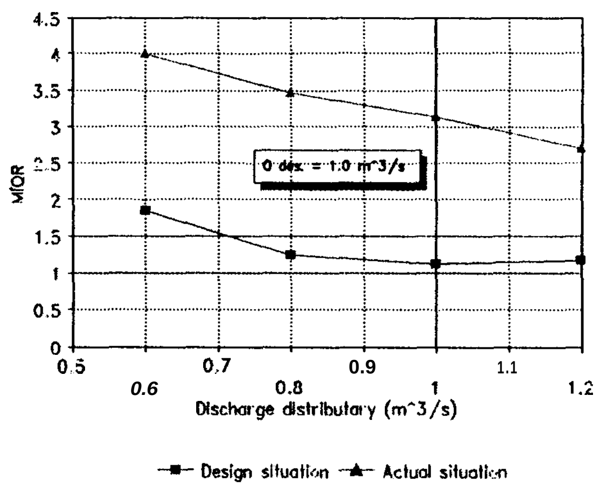


Figure6.11 MIQR value for both the actual and the design state of the distributary for various inflow at the head.

Performance indicator	Design situation	Actual situation
MIQR 60% FSD	1.857	4.003
MIQR 80% FSD	1.218	3.473
MIQR 120% FSD	1.181	2.709
MIQR FSD	1.126: good equitability	3.142: poor equitability

Looking at the results obtained by using the MIQR value, the same conclusion can be drawn when using the variability indicator: (global) CV(DPR), as used by IIMI as the indicator for equity.

conclusions

Referred to the objectives of this section and after analysing the performance of the design and actual state of Masood distributary it can be stated that:

- Although most of the outlet structures running above authorized discharge due to removed outlets and a shorted tail, the actual performance based on the principles of proportionality and equitability is poor.
- To evaluate proposed adjustments on the impact on water distribution, the performance of a distributary at FSD can not be expressed by one indicator only. In this case, at least three indicators are necessary. For proportionality S local and global, i.e. S_{outlet} and S_{sys} , and for equity DPR local for individual outlet structures, DPR global and MIQR for the whole distributary.
- The outlet structures which will be studied more in detail during the analysis of the responsiveness of the system are: outlet No. 2 (head reach), outlet No. 5 (close to a drop and middle reach), outlet No. 8 (middle reach), and outlet No. 12 (tail reach).

In figure 6.12, the proposed set up of the analysis with the irrigation indicators is summarized. Parameters with an substantial impact on the canal water distribution (high R-value), are interesting to study in order to improve the water management at the distributary level. Based on the above suggested parameters, any physical adjustment can be evaluated using these irrigation indicators, based on the principles of irrigation in the area: proportionality and equity.

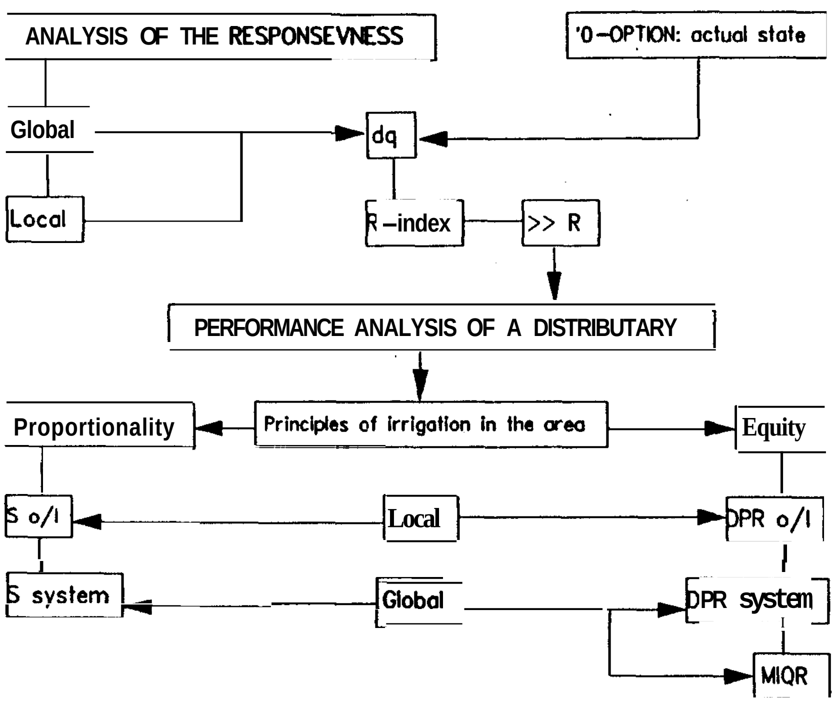


Figure 6.12 **Performance analysis of a distributary, based on the principles of irrigation: proportionality and equity.**

6.6 Characteristics determining the canal water distribution

This section presents the different characteristics, which will be analysed With the sensitivity analysis, further discussed in chapter 7. In figure **6.13,** a distributary is schematized **as** a linear canal with n off taking outlet structures.

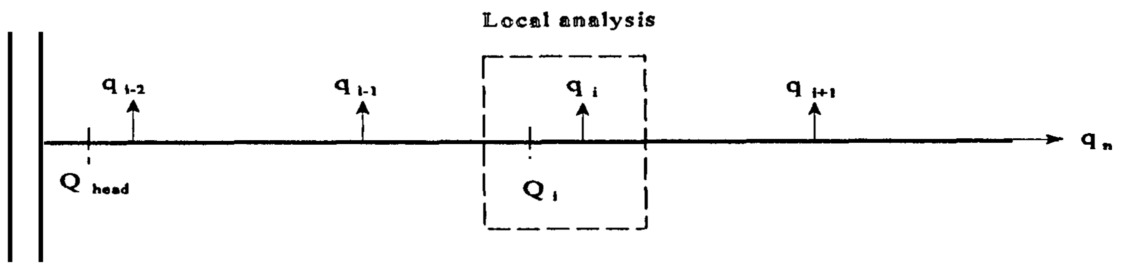


Figure 6.13 Schematic presentation of a distributary

The canal water distribution to outlet structure i is determined by:

- Distributed discharge **to** outlet structure i (local analysis): the type of outlet structure and flow condition determines the different outlet structure characteristics: ***opening width*** (B), ***opening heighf (Y), crest level, discharge coefficient, upstream and downstream water level above the crest.***
- The outlet structure characteristic which is related with the discharge Q_i **in** the canal is the upstream water level above the crest (and partly the downstream water level for submerged outlet structures). Q_i is related with the discharge at the head of the distributary Q_{head}. The relation between Q_{head} and the water level in the canal in front of outlet structure i is determined by: ***(1) Manning's coefficient n, seepage losses S_e and the bed slope of the canal i (canal characteristics); (2) cross sectional profile expressed as $A.R^{2/3}$; (3) impact of upstream located outlet structures.***
- The existence of cross regulators (drop structures), with the characteristics: ***width of the drop (B), crest level, discharge coefficient and flow condition.***
- Besides the distribution of canal water for a steady inflow at the head of a distributary, a change in inflow (AQ) results in a re-distribution (Aq) of canal water.

$$AQ = \sum_{i=l}^{n} \Delta q_i$$

In general, canal water distribution to an individual outlet structure can be denoted as:

$$q_i = F\,(Q_{head}(t)\ ,\ O\ ,\ C\ ,\ D\ ,\ q_{i-1})$$

q_i	=	Canal water distribution to outlet structure i	[m³/s]
$Q_{head}(t)$	=	Inflow discharge at the head of a distributary as a function in time	[m³/s]
O	=	Outlet structure characteristics	
C	=	Canal characteristics	
D	=	Drop structure characteristics	
q_{i-1}	=	Upstream outlet structures	

In chapter 7, al these parameters will be evaluated on there sensitivity (expressed in the R-index) using the proposed methodology of section 6.2.

CHAPTER 7 RESPONSIVENESS OF THE SYSTEM: A SENSITIVITY ANALYSIS

7.1 Theoretical approach

A distinction has been made between a theoretical analysis of the impact on the canal water distribution adjusting outlet structure characteristics, and an analysis of the impact on the canal water distribution adjusting other parameters based on simulations. This chapter presents the analysis of the responsiveness of the system, i.e. ***a re-distribution of canal water,*** based on pre-defined adjustments of the parameters determining the distribution and an inflow pattern at the head of the distributary. The pre-defined adjustments are organized in different scenario's to be able to study:

- (1) The responsiveness expressed in the R-index for the different parameters, determined in section 6.6; (2) to quantify the impact of a certain adjustment and (3) to divide the input parameters for the methodology to set up a simplified model into two groups: ***insensitive parameters,*** which can be generalized and simplified for all distributaries, and ***sensitive parameters*** which have to be measured and calibrated for each individual distributary.
- Based on the analysis, the sensitive parameters are interesting to study more in detail, when it comes to improve water management at the distributary level.

In table **7.1,** the different scenario's and the range of adjustments are listed.

Table **7.1** Scenario's for the analysis of the responsiveness

SCENARIO'S	PARAMETERS	RANGE OF ADJUSTMENTS	
Remodelling of outlet structures	**- Width opening: B - Height opening: Y - Discharge coefficient: C, - Crest level**	**Theoretical analysis**	
Remodelling of cross structures	**- Width: B - Discharge coefficient: C, - Crest level**	**- 25% / + 25% - - 40% / + 40%**	**SIC**
Hydraulic canal data	**- Manning's coefficient: n - Seepage: S**	**- 20% / + 20% - 100% / + 100%**	**SIC**
Canal maintenance	**- Area: $AR^{2/3}$**	**- 20% / +40%**	**SIC**

7.2 Outlet structure characteristics

A change in inflow at the head of the distributary results in a change in water levels along the canal. For a new steady state in the canal, the water table follows Manning-Strickler. Focussing on a local outlet structure, a change in distribution (dq) based on a change in inflow at the head (dQ) is given in figure 7.1. It is obvious, that the delivery of canal water to the different types of outlet structures is determined by the input parameters of the corresponding discharge equation and the corresponding flow condition.

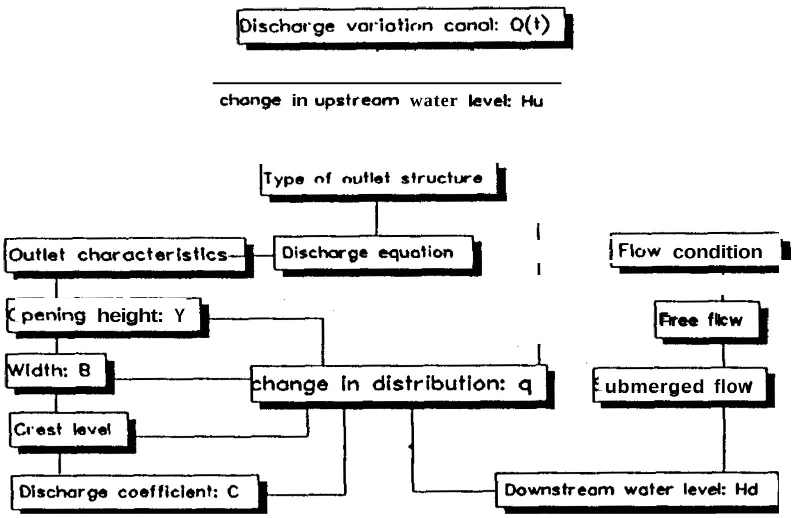


Figure 7.1 Outlet structure characteristics.

To study the responsiveness of the system for adjustments in outlet structure characteristics, a theoretical analysis will be sufficient. As discussed in section **6.4.2,** the responsiveness will be expressed in the so-called Responsiveness-index (R-index), defined as a percentage change in supplied discharge to an outlet structure as result of a percentage change in one of the characteristics. For the theoretical analysis, the R-index values were taken absolute. For example: an increase of the width B with 25%, results in an increase of distributed discharge with **25%,** a decrease **of** B with **25%** will result in a decrease of distributed discharge of **25%.** For both adjustments, the R = 1..

The R-index = 1: 1% change in the adjusted characteristic results in 1% change in distributed discharge.

The analysis of the outlet structure characteristics is based on the ***discharge equations (see chapter 4)*** *for the different types of outlet structures, and the physical characteristics of outlet structure no. 11 of Masood distributary (table 7.2).*

Table 7.2 **Outlet structure characteristics (based on outlet structure 11)**

Type	Discharge coefficient	Width B (m)	Height Y (m)	Crest Level (m) (above bed level)
(OC)OFRB	F(h_u, Y)	0.12	0.30	0.06
(OC)AOSM	0.90 (o.m.)	0.12	0.30	0.06
OF	0.95 (f.f.)	0.12	-	0.06
PIPE	0.80 (o.n.)	-	0.30	0.06

(OC)OFRB outlet structures

For the analysis, the general vertical gate discharge equation for orifice flow, as discussed in section 4.3.3 is used. The coefficient of discharge C_d is a function of the gate opening **Y** and the upstream water level above the crest h_u. With the initial contraction coefficient μ = **0.60,** the C_d ranges between **0.50** and **0.60.** The R-index for the C_d-coefficient and the width B of an (OC)OFRB outlet structure reads I. confirm the discharge equation.

The R-index for the opening height Y is a function of the upstream water level h_u and the change in Y (dY), as shown in figure 7.2. For high upstream water levels, R is increasing up to approximately **0.95.**

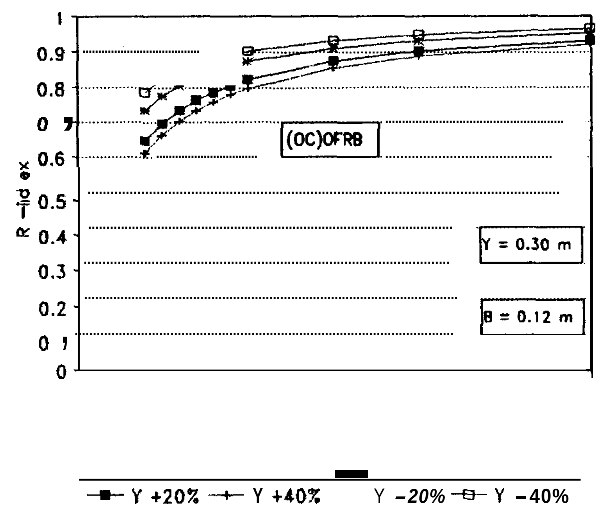


Figure 7.2 Opening height Y for (OC)OFRB.

For low upstream water levels the impact on distribution *for **a change in*** Y is substantially higher then for high upstream water levels. **As** the upstream water levels in general reaching up to **1.0** to 1.5 metres, one can say that the opening height is a sensitive parameter: **0.6** < R < **0.95.** Besides that, the impact on water distribution is more sensitive for a decrease in Y then for an increase, i.e. $R_{-40\%} > R_{+40\%}$.

Figure 7.3 presents the R-index values for a change in crest level. For example, with an initial crest level of **0.06** m (above bed level), the R-index curve does not change for an increase or decrease up to 40 % **(0.036** m - **0.06 m - 0.084 m).** On the other hand, the R-index depends on the ***initial crest level*** and the ***water level*** in the canal. For a higher initial crest level, for example **0.24 m,** a change in crest level results in higher R-values.

For high water levels in the canal, the impact on distribution for a change in crest level is decreasing. The impact on distribution can be expressed with the ratio: **Initial Crest level ($CL_{initial}$) / Water level (WL).** For a high ratio $CL_{initial}$ / WL, the impact on the canal water distribution increases: the K-index < 0.5 when $CL_{initial}$ / WL < **0.35.**

It can be concluded that for (OC)OFRB outlet structures the B and the C_d coefficient do have a substantial impact on water distribution, as the R-index = 1. Also, a change in Y results in a substantial impact on distribution. Crest level adjustments are insensitive when $CL_{initial}$ / WL < 0.35.

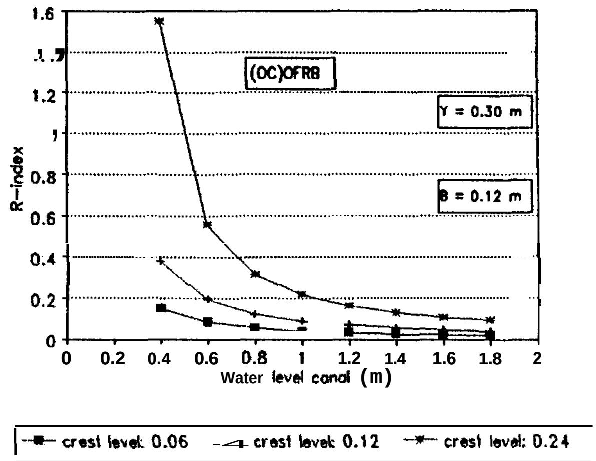


Figure 7.3 Crest levels for (OC)OFRB.

(OC)AOSM outlet structures

For the analysis, the general discharge equation for an AOSM, as discussed in section **4.3.4** is used. The coefficient of discharge C_d is not depending on the upstream water level, as the roof block has a rounded top and the initial coefficient of contraction $\mu = 1$: the C_d approximately reads 0.90.

The R-index for the C_d-coefficient and the width B of (OC)AOSM outlet structures reads 1, based on the discharge equation. The R-index for the opening height Y is a function of the upstream water level h_u and the change in Y (dY), see figure **7.4.** Compared with the (OC)OFRB, the impact on water distribution for a change in Y is more substantial: opening height is a more sensitive parameter for an (OC)AOSM. For high upstream water levels, R is increasing up to approximately 0.95. For low upstream water levels the impact on distribution for a change in Y is substantially higher then for high upstream water levels. As the upstream water levels in general reaching up to 1.0 to **1.5** metres, one can say that the opening height is a sensitive parameter: 0.2 < R < 0.95. The impact on water distribution is more sensitive for a decrease in Y then for an increase, i.e. $R_{-40\%} > R_{+40\%}$.

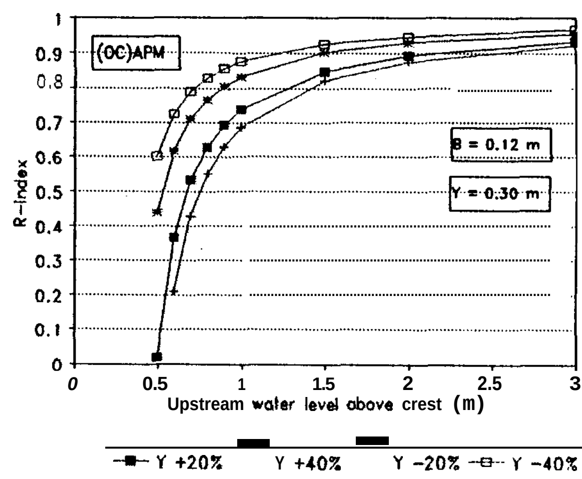


Figure 7.4 Opening height Y for (OC)AOSM.

Figure **7.5** presents the R-index values for a change in crest level. The same curves are found as for (OC)OFRB outlet structures. For example, with an initial crest level of 0.06 m, the R-index curve does not change for an increase or decrease up to 40 % **(0.036** m - **0.06 m** - 0.084 m). The R-index depends on the ***initial crest level*** and the ***water level*** in the canal. For **a** higher initial crest level, for example 0.24 m, a change in crest level results in higher R-values. Compared with figure 7.3, it can be stated that a change in crest level for (OC)AOSM structures is more sensitive then for (OC)OFRB structures: R < 0.5 when the ratio $CL_{initial}$ / WL < 0.30.

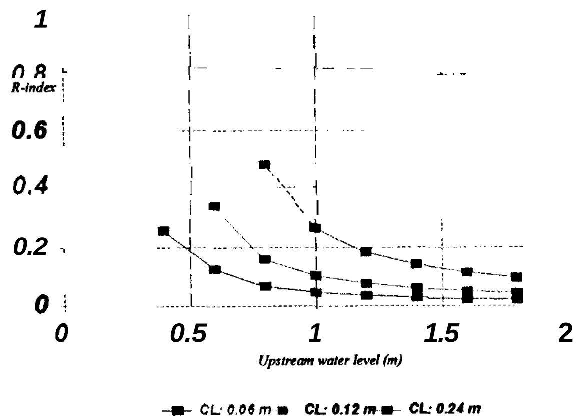


Figure 7.5 **Crest levels for (OC)AOSM.**

It can be concluded that for (OC)AOSM outlet structures the B and the C_d coefficient do have a substantial impact on water distribution, as the R-index = 1. Also, a change in Y results in a substantial impact on the canal water distribution. Crest level adjustments are insensitive when** $CL_{initial}$ **/ WL < 0.30.

<u>Open Flume outlet structures</u>

For the analysis, the general discharge equation for an Open Flume, as discussed in section 4.3.2 is used. The coefficient of discharge C_L is not depending on the upstream water level, and is fixed at 0.95 (theoretical value **1).**

The R-index for the discharge coefficient and the width B of Open Flume outlet structures reads 1, confirm the discharge equation. Figure **7.6** presents the R-index values for a change in crest level. The same curve is found, as for (OC) OFRB and (OC)AOSM outlet structures. For example with an initial crest level of 0.06 m, the R-index curve does not change for an increase or decrease up to 40 % (0.036m - **0.06m** - 0.084m).

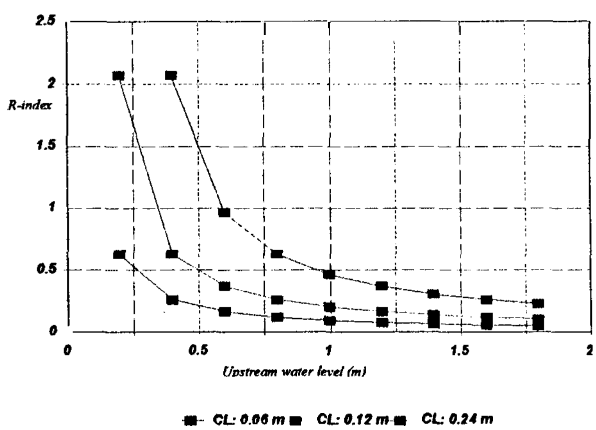


Figure 7.6 **Crest levels Open Flume.**

The K-index depends on the *initial crest level* and the *water level* in the canal. Compared with figures 7.3 and 7.5, it can be stated that a change in crest level for OF structures is more sensitive then for (OC)AOSM and (OC)OFRB structures: R < 0.5 when the ratio $CL_{initial}$ / WL < *0.24. It can be concluded that for an OF outlet structures the B and the C_d coefficient do have a substantial impact on water' distribution, as the R-index = 1. Crest level adjustments are insensitive when $CL_{initial}$ / WL < 0.24.*

Pipe outlet structures

For the analysis, the general discharge equation for a pipe, as discussed in section 4.3.5 is used. As the pipe outlet structures most of the time functioning under submerged conditions, the o.n.-pipe equations is used. The coefficient of discharge C_p is fixed at the theoretical value 0.80. The R-index for the diameter Y is depending on the change in Y, is independent from the head over the structure (z), and is listed in table 7.3. In general, it can be stated that the sensitivity of the diameter Y for pipe outlet structure reaches R = 2.

Change in diameter Y (m)	R (z = 0.05 to 0.45)
+ 20%	2.2
+ 10%	2.1
- 10%	1.9
- 20%	1.8

Figure 7.7 presents the R-index values for a change in crest level. The same curve is found for the other outlet structures. For example, with an initial crest level of 0.06 m, the R-index curve does not change for an increase or decrease up to 40 % (0.036m - 0.06m - 0.084m). The R-index depends on the *initial crest level* and the *water level* in the canal. Compared with the other figures , it can be stated that a change in crest level for pipe structures is the most sensitive, i.e. has the most substantial impact on the water distribution. ***It can be concluded that for pipe outlet structures the Y and the C_p coefficient do Jiave a substantial impact on water distribution, as the R-index is respectively 2 and 1. Crest level adjustments are insensitive when the ratio $CL_{initial}$ / WL < 0.15.***

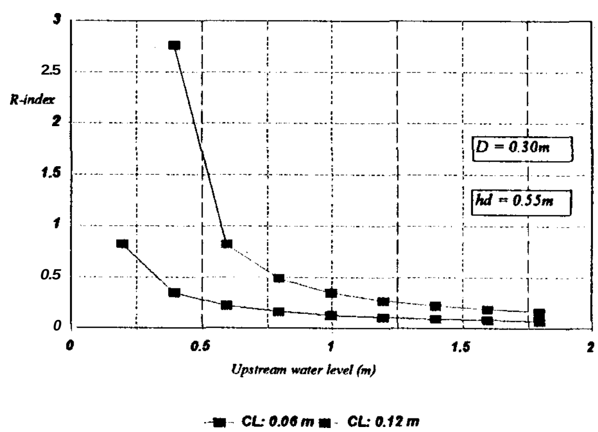


Figure 7.7 crest levels for (o.n.) pipe outlet structures.

expressed by the R-index for a change in upstream water level for (OC)OFRB, (OC) AOSM and OF outlet structures, and a change in the head z for submerged pipe outlet structures is listed in figure 7.8. The R-index is depending on the percentage change and is determined by the value of the power (0.5 or 1.5).
The same conclusion can be drawn as formulated for the R-index for the diameter Y for pipe structures:

- *(OC)OFRB / (OC)AOSM: $R_{hu} = 0.5$;*
- *OF: R_{hu} = **1.5**;*
- *PIPE: $R_z = 0.5$.*

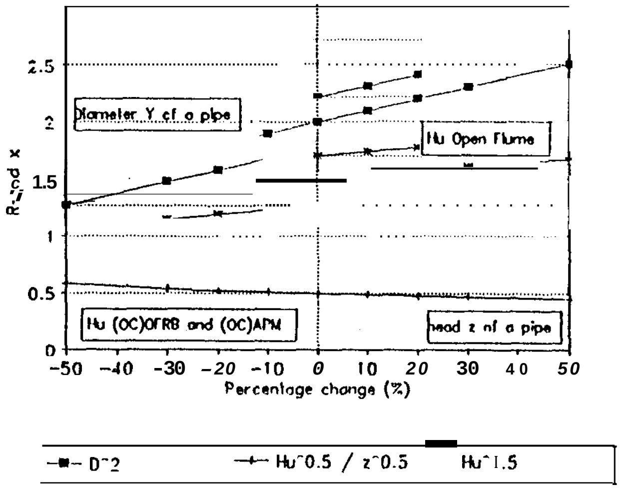


Figure 7.8 **R-index values for upstream water levels and head for different types of outlet structures.**

The analysis on the impact on distribution of different parameters will be carried out by the model of Masood distributary, which contains (OC)OFRB and submerged pipe outlet structures only. To take all types of outlet structures into account, the change in upstream water level due to a certain change of input parameter will be studied, and based on the results the impact on distribution will be explained. For a certain change in upstream water level, the following relations can be stated: see table 7.4.

Table 7.4 **Impact on a change in water level on distribution to different types of outlet structures**

	(OC)OFRB	(OC)AOSM	OF	PIPE	comparison
+ h_u or dz	+ q_1	+ q_2	+ q_3	+ q_4	$q_1:q_2:q_3:q_4$ = 1:1:3:1
- h_u or dz	- q_{11}	- q_{22}	- q_{33}	- q_{44}	$q_{11}:q_{22}:q_{33}:q_{44}$ = 1:1:3:1

In general: the impact of any adjustment on the canal water distribution for open flume type of outlet structures is higher then for (OC)OFRB or (OC)AOSM-type of structures. For submerged pipe and (OC)OFRB and (OC)AOSM outlet structures: the increase of water level resulting in an increase in discharge (%) is less then the decrease in discharge (%) due to a decrease in water level. It goes the other way around for open flume-types.

Submergence

Except for the submerged pipe outlet structure, the analysis is conducted for free flow conditions only. For a submerged outlet structure, the same hydraulic principles are there. The difference is, due to the submergence, the discharge through an outlet structure is also depending on the downstream water level.

The overall results of the theoretical analysis of the responsiveness of the outlet structure characteristics, i.e. the impact on water distribution, based on a certain change in input is listed in table 7.5.

Table 7.5 Results theoretical responsiveness analysis of outlet structure characteristics

CHARACTERISTICS	DISCHARGE EQUATION	RELATION	R-INDEX
OPEN FLUME			
C	$q = C_f \cdot 1.7 \cdot B \cdot h_u^{1.5}$	dq :: dC_f	R = 1
B		dq :: dB	R = 1
h_u		dq :: dh_u	R = 1.5
Crest Level		dq :: d crest	R = F(H_{canal}, CL[1])
(OC)OFRB			
C_d	$q = C_d \cdot B \cdot Y \cdot (2 \cdot g \cdot h_u)^{0.5}$	dq :: dC_d	R = 1
B		dq :: dB	R = 1
Y	$C_d = u / (1 + (u \cdot Y / h_u))^{0.5}$	dq :: dY	R = F(dY, H_{canal})
h_u		dq :: dh_u	R = 0.5
Crest Level		dq :: d crest	R = F(H_{canal}, CL)
(OC)AOSM			
C_d	$q = C_d \cdot B \cdot Y \cdot (2 \cdot g \cdot (h_u - Y))^{0.5}$	dq :: C_d	R = 1
B		dq :: dB	R = 1
Y		dq :: dY	R = F(dY, H)
h_u		dq :: dh_u	R = 0.5
Crest Level		dq :: d crest	R = F(H_{canal}, CL)
PIPE			
C	$q = C_p \cdot A \cdot (2 \cdot g \cdot z)^{0.5}$	dq :: C	R = 1
D		dq :: dD	R = 2
z (= h_u - h_d)	$A = \frac{1}{4} \cdot pi \cdot D^2$	dq :: dz	R = 0.5
Crest Level		dq :: d crest	R = F(H_{canal}, CL)

1 *CL = Initial Crest Level measured above bed level*

7.3 Cross structures

Cross structures are used to control the flow of water in the canal, i.e. maintain certain upstream water levels and divide the canal in sections to decrease the slope of the water profile. The cross structures placed in the distributaries of the Chishtian Sub-Division are ungated fixed control structures, i.e drop structures, so no manual operations are possible. The characteristics of drop structures are: the width B, the discharge coefficient C_d, the crest elevation and the flow condition. **At** present, there are three drop structures in Masood distributary, one submerged drop located at 5.5 km from the head, and two free flow drops located at 7.3 km and 11.4 km from the head. To analyse the responsiveness of the system based on adjustments of drop structure characteristics, the following scenario's are simulated:

- ***Scenario 1: free flow drop structure.*** Adjust B and CL. No adjustments on the submerged drop structure. The discharge coefficient is not analysed, as for free flow drop structures, the coefficient of discharge will be approximately constant ($C_d = 0.95$).
- ***Scenario 2: submerged drop structure.*** Analyse the impact on the distribution when the flow condition turns to free flow, by increasing the crest level and change the discharge coefficient. No adjustments on the free flow drop structure.

<u>Scenario 1</u>
To analyse the impact on the water distribution for a change in the width and the crest level, the value of B and CL has been changed as follows:

Table 7.6 Adjustments of B and CL of the free flow drop structure in the model at RD 24.04

Width (B)						
- 25%			0%	+ 25%		
2.46 m			3.28 m	4.10 m		
- 40%	- 25%	- 10%	0%	+ 10%	+ 25%	+ 40%
0.22 m	0.27 m	0.32 m	0.36 m	0.40 m	0.45 m	0.50 m

After analysing the results, listed in annex F, of the simulations for the outlet structures No. 2, 5, 8 and 12, the next conclusions can be stated:

- There is only **a** local impact on the canal water distribution for adjustments on drop structures, i.e the outlet structures within the reach of the back water curve upstream **a** drop structure (outlet structure no. 5). Disturbances are transported to the tail, **as** the discharge over the tail drop structure is varying due to upstream adjustments.

- An *increase of 25%* in the width of the drop structure is resulting in **a** decrease in upstream water level (flow condition changed from free flow to submerged flow) and therefor in a decrease in supplied discharge to outlet structure 5. **A** negative backwater curve results in suction of water over the crest of the drop. **dB +25% => R_{mean}[2]= (-)0.26**.

- **A** *decrease of 25%* in the width of the drop structure is resulting in an increase in upstream water level and therefor in an increase in supplied discharge to outlet structure 5. **dB -25% ==> R_{mean} = 0.37**

- For outlet structure 5, the R-index for **a** change in crest levels are presented in table 7.7. **An** increase of the crest level results in an increase **of** q, **R_{mean} = 0.78.** Decrease of the crest level: decrease of q (decreasing R) R_{mean} = (-)0.67.

- The value of the R-index for a change in crest level (based on a change in canal water distribution to outlet structure 5) is depending on the discharge at the head (as **a** percentage of the **FSD).** For low discharges, the R is substantially higher. For **a** large adjustment of the crest level, the value for the R-index also depends on the adjustment itself: R-index for an increase in CL > R-index for a decrease (see figure **7.9).**

- Downstream water level of the drop structure remains constant after the adjustments, for all discharges at the head.

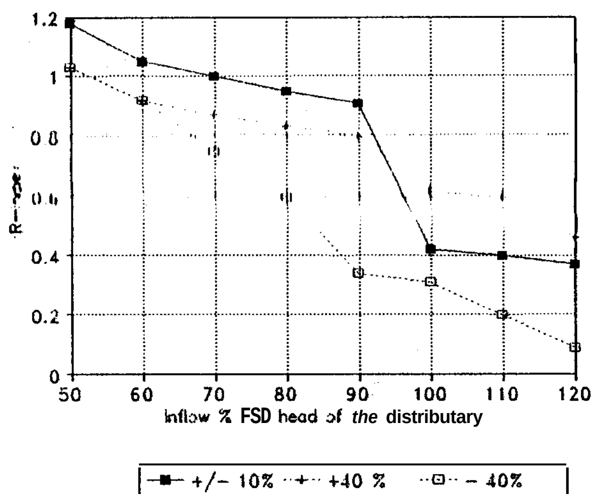


Figure 7.9 **Crest level adjustment on drop structure for different inflow at the head.**

Outlet structure	-40%	-25%	-10%	+10%	+25%	+40%	-d CL	+d CL
5	(-)0.53	(-)0.64	(-)0.84	**0.78**	**0.80**	**0.77**	**(-)0.67**	**0.78**

2 *R_{mean} is based on the computed R-index for the distributed discharge to outlet structure 5 for different inflow at the head*

The analysis is conducted for a submerged pipe outlet structure only. Table 7.9 presents the upstream water levels and the percentage of change referred to the '0-option'. It *can* be concluded that:

- ***An* increase of the crest level,** results in an increase of the water level for the outlet structure.
- **A decrease of crest level,** results in **a** decrease of the water level for the outlet structures.
- The change in water level due to a decrease of crest level of the drop is more substantial for low discharges (60% **FSD).**
- The impact on the water distribution for (OC)AOSM, (OC)OFRB, and definitely for Open flume outlet structures will be more significant, because the impact on submerged pipe outlet structures is always slightly attenuated.

1

Crest level adjustment	- 40 %	+ 40 %
Outlet No.	5	5
60% FSD - h_u (m) - dh_u (%)	 0.38 -25.50%	 0.64 +25.50%
100% FSD - h_u (m) - dh_u (%)	 0.55 -8.33%	 0.73 +21.67%
120% FSD - h_u (m) - dh_u (%)	 0.64 -4.48%	 0.78 +16.42%

Scenario 2

To analyse the impact on the water distribution for a change in the flow condition of **a** drop structure, the crest level and the discharge coefficient has been changed, with respect **to** avoid overtopping of the upstream banks.

	Old value	New value
Crest level	148.27 m	148.47 m
Discharge coefficient	2.61	0.95

After analysing the result of the simulation for the outlet structures No. 2, 5, **8** and 12, the next conclusions can be stated:

- Again, there **is** only a local impact on the water distribution for adjustments on drop structures. Only **a** change in the back water curve is responsible for a change in distribution of canal water to outlet structures.
- Free weir flow **is** resulting in significant higher upstream water levels (+0.20 m).
- The discharge over the weir slightly decreases (1% for 120% FSD to 3.5% for 50% FSD at the head) due to an increase in discharge to outlet structure no. **4.**
- Downstream water level of the drop structure remains constant afier the adjustments, for all discharges at the head.

Backwater effects

The flow profile represents the surface curve **of** the flow. There will be a positive backwater curve if the depth of flow increases in the direction of flow, and a draw down or negative backwater curve if the depth of flow decreases in the direction of flow. The reach of a backwater curve *can* be computed numerically.

7.4 Hydraulic canal data

There are two parameters that can be distinguished **as** hydraulic canal data, which will be analysed here: the *coefficient ofroughness* expressed **as** the Manning's coefficient n or Strickler coefficient k; and **the** *seepage losses* expressed **as** S,. Both have their impact on distribution as they are influencing the water table in the canal and therefor the upstream water level above the crest of the outlet structures.

7.4.1 Coefficient of roughness

The Manning's coefficient is **part** of the well-known Manning'sequation for uniform flow in open canals. The equation is usually expressed in the Manning-Strickler equation, with the Strickler coefficient **k** defined as 1/n. k will have the dimension of [$m^{1/3}$/s].

$$Q = k.A.R^{\frac{2}{3}}.i^{\frac{1}{2}}$$
$$R = \frac{A}{O}$$

Where:

Q	=	Discharge	[m^3/s]
k	=	Coefficient of roughness	[$m^{1/3}$/s]
A	=	Wettedarea	[m^2]
R	=	Wetted radius	[m]
O	=	Wetted perimeter	[m]
i	=	Energy line (slope)	[-]

At present, there are no exact methods to determine the value of k or n, and therefor selecting a proper value for k or n is based on experience. There are a lot of factors affecting the coefficient of roughness (Ven Te Chow, 1973): (1) surface roughness of the outline of the canal (determined by grain size and bed material); (2) height, density and distribution of bed, bank and floating vegetation; (3) canal irregularities; (4) canal alignment; (5) silting and scouring of the canal bed; (6) canal obstructions, and (7) change in discharge (for low discharges the irregularities get more substantial and k slightly decreases).

In table 7.10, the canal characteristics as described by Ven Te Chow (1973) are applied to the calibrated n-values of the model of Masood distributary. An ***increase in roughness of the canal results in an increase of n and a decrease of k.***

Table 7.10 Canal characteristics and coefficient of roughness (Masood distributary n-values)

Canal Characteristic	n	k (= 1/n)
Earth canal excavated in alluvial silt soil, slightly vegetated with grass	0.025 - 0.030	33.333 - 40.000
Earth canal excavated in alluvial silt soil, irregular bed, vegetated with long grass	0.030 - 0.035	28.571 - 33.333
Earth canal excavated in clay and loam, irregular bed, vegetated with long grass	0.035 - 0.040	25.000 - 28.571
Earth canal excavated in clay and loam, irregular cross sections and bed, vegetated with brushes	0.040 - 0.050	20.000 - 25.000
Earth canal excavated in clay , very irregular bed and side slopes, vegetated with heavy weeds and grass	0.050 - 0.060	16.667 - 20.000

To analyse the impact on the water distribution for a change in the roughness coefficient as defined in the different reaches of the model, the value of n has been changed as follows (table 7.11):

Table 7.11 Adjustments of the value n i

- 20%	- 10%	- 5%	initial n value	+5%	+ 10%	+20%
0.020	0.023	0.024	**0.025**	0.026	0.028	0.030
0.029	0.032	0.034	**0.036**	0.038	0.040	0.043
0.020	0.023	0.024	**0.025**	0.026	0.028	0.030
0.023	0.026	0.028	**0.029**	0.030	0.032	0.035
0.028	0.032	0.033	**0.035**	0.037	0.039	0.042
0.034	0.039	0.041	**0.043**	0.045	0.047	0.052
0.046	0.051	0.054	**0.057**	0.060	0.063	0.068
0.039	0.044	0.047	**0.049**	0.051	0.054	0.059

The results of the simulations for the outlet structures No. 2, 5, 8 and 12 are listed in annex F. After analysing the result the following conclusions can be stated:

- Head reach outlet structures

An increase of the value of n results in an increase of water levels along the canal and therefor in an increase in canal water delivery to the head outlet structures. Although q increases, there is a decreasing responsiveness of the system for an increasing n. A decrease of the value of n results in a decrease of water levels along the canal and therefor in a decrease in canal water delivery to the head outlet structures. *In general, an increase of the value of n: **R** = **0.36;** a decrease of the value of n: **R** = (-)0.33.*

- Submerged pipe

Can be characterized by a low responsiveness to a change of the value of n. In general: a decrease of the value of n: **R** = (-)0.05; an increase of the value of n: **R** = 0.11. The low response of outlet structure no. **5** for a change of n, is due to the close presence of a drop structure **just** downstream. The depth discharge relationship defined by the drop structure keeps the upstream water level approximately constant, when the water level in the canal is changing.

- Middle reach outlet structures

An increase of the value of n results in an increase of water levels along the canal and therefor in an increase in canal water distribution to the middle reach outlet structures. A decrease of the value of n results in a decrease of water levels along the canal and therefor in a decrease in canal water distribution to the middle reach outlet structures. In general **an** increase of the value of n: **R** = **0.18;** a decrease of the value of n: **R** = (-)0.17.

- **Tail reach outlet structures**

An increase of the value of n results in a decrease of water levels along the tail end of the canal and therefor in a decrease in canal water distribution to the tail outlet structures. A decrease of the value of n results in an increase of water levels along the tail end of the canal and therefor in an increase in canal water distribution to the tail outlet structures. In general an increase **of** the value **of** n: **R = (-)0.26**; a decrease of the value of n: R = **0.26.**

- **Increase of the value of n**

More downstream along the canal, the impact on the canal water distribution changes from an increase in supply to a decrease in supply.

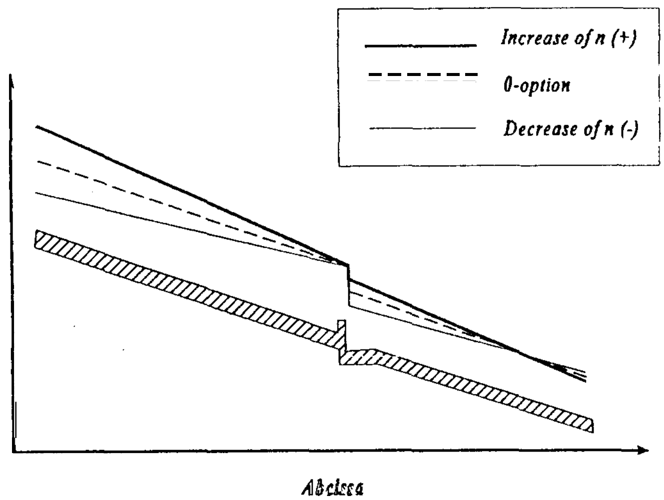


Figure 7.10 Impact on a change in Manning's coefficient on the water table of a distributary.

There is a breakpoint were the water line drops below original water line, i.e. '0-option', due to an increase of distribution to head and middle reach outlet structures. The opposite counts for a decrease of the value of n. The tail-enders suffer either from a lack of distribution or an increased distribution (bank overtopping). This is explained in figure 7.10. In table 7.12 the overall mean R-index values are listed. The analysis is conducted for (OC)OFRB outlet structures and a submerged pipe outlet structure only. As mentioned already, the impact on the distribution for other types of outlet structures will be evaluated based on a change in upstream water level in the canal.

Outlet structure	- 20%	-10%	- 5%	+ 5%	+ 10%	+ 20%	- n	+ n
2	(-)0.33	(-)0.36	(-)0.31	+ 0.42	+ 0.37	+ 0.30	(-)0.33	+ 0.36
5	(-)0.00	(-)0.05	(-)0.09	+ 0.11	+ 0.08	+ 0.15	(-)0.05	+ 0.11
8	(-)0.21	(-)0.18	(-)0.11	+ 0.16	+ 0.18	+ 0.19	(-)0.17	+ 0.18
12	+ 0.28	+ 0.22	+ 0.28	(-)0.24	(-)0.30	(-)0.24	+ 0.26	(-)0.26

Table 7.12 presents the upstream water levels and the percentage of change referred to the '0-option'. It can be concluded that:

- An **increase of n,** i.e the canal becomes rougher, results in an increase of the water level for head and middle reach outlet structures and a decrease of the water level for tail outlet structures.

- **A** decrease **of n,** i.e the canal becomes smoother, results in a decrease of the water level for head and middle reach outlet structures and an increase of the water level for tail outlet structures.

Adjusted n	**-20 %**				**+20 %**			
Outlet No.	**2**	**5**	**8**	**12**	**2**	**5**	**8**	**12**
60% FSD - h_u (m) - dh_u (%)	0.64 -7.25%	0.51 0%	0.55 -5.45%	0.41 +7.89%	0.73 +5.80%	0.50 -1.96%	0.60 +3.45%	0.34 -10.53%
100% FSD - h_u (m) - dh_u (%)	0.78 -7.69%	0.60 0%	0.73 -7.60%	0.67 +4.69%	0.90 +7.14%	0.63 +5.00%	0.83 +5.06%	0.62 -3.13%
120% FSD - h_u (m) - dh_u (%)	0.84 -8.70%	0.64 -4.69%	0.82 -7.32%	0.80 +3.90%	0.97 +5.43%	0.72 +7.46%	0.93 +5.68%	0.75 -2.60%

7.4.2 Seepage

Seepage consists of all the losses in the canal, and is expressed in l/s/km. The losses can be positive, i.e. inflow seepage, and negative or outflow seepage. To analyse properly the impact on the canal water distribution to the outlet structures based on a change in seepage losses two scenario's were used:

- The model with the seepage data from 15-11-1995 all positive, i.e. no outflow seepage.
- The model with the seepage data from 15-11-1995 all negative, i.e. no inflow seepage.

The seepage data used are valid for the analysis, as the S_e as percentage of the inflow ranges between approximately 3% to 16% (see table ***5.6).*** In general, in the Punjab seepage losses ranges in between ***5%*** to 20% of the inflow at the head of a distributary. Therefor, the results of the analysis will be valid for other distributaries too.

Reach (km)	**S_e (l/s/km)**	**S_e *(l/s)* (for the whole canal)**	**S_e in % of the inflow**	**Outlet structures**
0 - 4.4	1.9	21.47	3.3 %	1, 2, 3, 4
4.4 - 7.6	2.7	30.51	4.7 %	5
7.6 - 11.3	9.1	102.83	15.82 %	6, 7, 8, 9, 10, 11, 12

To analyse the impact on the water distribution for a change in the rate of seepage losses, for both inflow and outflow seepage, the value of S, has been changed as follows:

- 100%	- 40%	0%	+ 40%	+ 100%
0	1.1	1.9	2.7	3.8
0	1.6	2.7	3.8	5.4
0	5.5	9.1	12.7	18.2

The analysis is conducted for outlet structure No. 2, 5 and 12. After the simulations it was found that there were hardly any changes in the distribution for outlet structure 2 and 5. For no. 12 only, the adjustments of the seepage rate resulted in a change in distribution. The results of the simulations for the outlet structures No. 2, 5 and 12 are listed in annex F. After analysing the result, the following conclusions can be stated (see figure 7.11):

- Impact on the canal water distribution for a change in seepage, i.e a value for R, is depending on the value for S_e.
- The impact on the canal water distribution for a change in outflow seepage is more significant then for inflow seepage Outflow seepage: R_{mean} = 0.11; Inflow seepage: R_{mean} = 0.065.
- Impact on the canal water distribution for a change in seepage, is depending on the inflow at the head of the distributary, i.e. the value for S, as a percentage of the inflow. For example S_e +100%: Q_{head} = 0.5 m³/s, R = 0.13 (S, inflow) and R = (-)0.22 (S, outflow); Q_{head} = 1.2 m³/s, R = 0.04 (S, inflow) and R = (-)0.04 (S outflow)

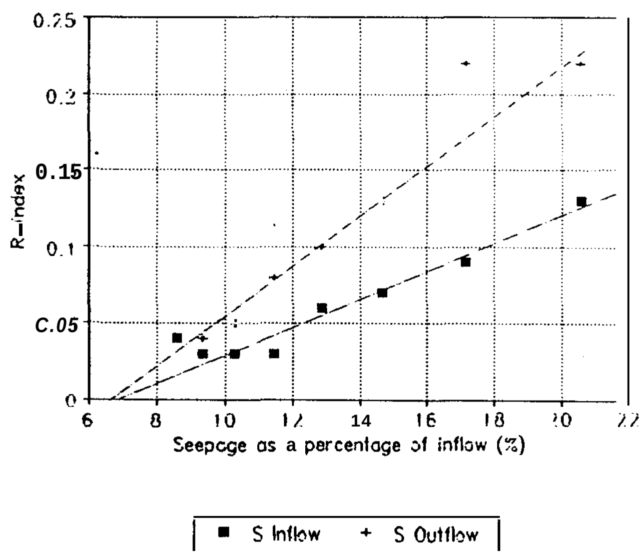


Figure 7.11 R-index of a change in seepage as a percentage of the inflow.

- Impact on the canal water distribution for a change in inflow or outflow seepage is limited, but the impact is significant for a change from inflow to outflow. Therefor, it is important to use inflow (+) or outflow (-) data to simulate properly the distribution. This was also found in the validation of the SIC model of Masood distributary.

It can be concluded that the impact on the canal water distribution of a change in seepage is more significant for low discharges at the head. It was found that there was a proper correlation between the seepage rate as percentage of the inflow at the head, both outflow ($R^2 = 0.94$) and inflow seepage ($R^2 = 0.92$), and the value for the R-index, listed in figure 7.11. Below ***6%*** to **8%** of seepage losses in the canal, a change in seepage does not effect the distribution to the outlet structures. That is way there **was** no change in distribution for outlet structure 2 and 5.

In general, **as** seepage losses ranges between 10% and 20% of the inflow at the head, it can be stated that the impact on the water distribution for change in seepage is low (seepage is an insensitive parameter):

- Inflow seepage: $0.05 < K < 0.20$;
- Outflow seepage: $0.03 < R < 0.12$

The analysis is conducted for (OC)OFRB outlet structures and **a** submerged pipe outlet structure only. Table 7.16 presents the upstream water levels (for outlet structure 12 only) and the percentage of change referred to the '0-option'. It ***can*** be concluded that:

- An increase respectively decrease **of** inflow seepage, results in an increase respectively decrease **of** the water level in the canal, which is more significant for low discharges at the head of the distributary.
- An increase **of** outflow seepage, results in a decrease of the water level in the canal, which is more significant for low discharges at the head of the distributary.
- **A** decrease **of** outflow seepage, results in **an** increase of the water level in the canal, which is more significant for low discharges at the head of the distributary.

Table **7.16** Change in water level (h_u) in metres and percentage ***(%)*** related to a change in the

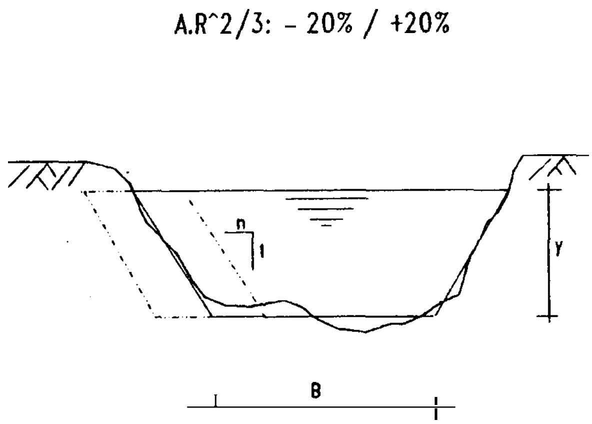
Adjusted seepage	- 100%		+ 100%	
Outlet No. 12	**Inflow seepage**	**Outflow seepage**	**Inflow seepage**	**Outflow seepage**
60% FSD				
- h_u (m)	0.34	0.31	0.43	0.26
- dh_u (%)	-12.8 %	+3.33 %	+10.26 %	-13.3 %
100% FSD				
- h_u (m)	0.61	0.55	0.70	0.53
- dh_u (%)	-6.15 %	+0.54 %	+7.69 %	**-1.85** %
120% FSD				
- h_u (m)	0.75	0.72	0.82	0.67
- dh_u (%)	**-3.85** %	+1.41 %	+5.13 %	-5.63 %

7.5 Canal maintenance

Considering the Manning-Strickler equation, keeping the slope i and the roughness coefficient k constant, the factor $A.R^{2/3}$ determines the impact of large canal maintenance, i.e. desilting and remodelling the canal cross sections. Maintenance of the canal cross section plays an important role in the distribution of canal water to the outlet structures, as the factor $A.R^{2/3}$ determine the relationship between discharge and water level within a distributary (Hart, 1996).

During the simulations, the factor $A.R^{2/3}$ is increased or decreased over the whole canal reach (global), by means of scaling the width of the canal (real measured cross sections were used). It was found that for 60% FSD, 100% FSD and 120% FSD, an increase of $A.R^{2/3}$ with for example 20% can be obtained by increasing the width B of the canal by 20%. The applied re-dimensioning of the canal cross sections is figured (figure 7.12). The initial value for the factor $A.R^{2/3}$ is based on the water level with the canal running on 100% FSD.

To analyse the impact on the water distribution for a change in the cross section of the canal, the value of $A.R^{2/3}$ has been changed as follows, see table 7.17.



A = (B + ny).y
R = A / O = A / (B t 2.y.(1+n^2)^0.5)

Figure 7.12 **Change in cross sectional profile based on adjusting the width (B) of the canal.**

A change of the initial cross section results definitely in a change in water level, as the slope and the roughness coefficient are kept constant. The analysis is conducted for the defined inflow pattern at the head, ***without the discharge at 50% FSD*** and ranges from -20% up to +40% in order to avoid sever bank over topping for a decreased factor $A.R^{2/3}$. The results of the simulations for the outlet structures No. 2, 5, 8 and 12 are listed in annex F. After analysing the results the following conclusions can be stated:

- Head and middle reach

Increase of $A.R^{2/3}$ results in a decrease of discharge to the outlet structures (no. 2: R_{mean} = (-)0.13; no. 8: R_{mean} = (-)0.20). Decrease of $A.R^{2/3}$ results in an increase of discharge to the outlet structures (no. 2: R_{mean} = 0.26; no. 8: R_{mean} = 0.33): so distribution of canal water responses more for a decrease in the cross section then for an increase. The responsiveness of the system is independent on a variation of discharge at the head of the distributary or the value of the adjustment (%).

Table 7.17 Adjustments of the value A.R$^{2/3}$ (m^2) in the model (change of B)

Location	- 20%	- 10%	0%	+ 20%	+ 40%	n
Head	1.90	2.13	2.36	2.82	3.29	1
o/l 1	2.34	2.61	2.89	3.45	4.01	1
o/l 2	1.07	1.20	1.32	1.57	1.83	1.2
o/l 3	2.18	2.33	2.48	2.77	3.07	2.5
o/l 4	2.22	2.43	2.63	3.05	3.47	1.5
cross 1	2.12	2.27	2.42	2.72	3.02	1.5
drop 1 (u/s)	1.00	1.09	1.18	1.38	1.57	2
drop 1 (d/s)	1.43	1.53	1.63	1.83	2.03	3
cross 2	1.54	1.64	1.74	1.95	2.16	2
o/l 5	1.06	1.17	1.28	1.51	1.73	1.5
drop 2 (ds)	1.06	1.17	1.28	1.51	1.73	1.5
drop 2 (d/s)	2.60	2.88	3.17	3.74	4.33	1
o/l 6	1.30	1.35	1.41	1.54	1.66	2.5
o/l 7	1.47	1.60	1.73	2.00	2.27	2
o/l 8	0.96	1.05	1.13	1.31	1.48	1.25
o/l 9	0.63	0.70	0.76	0.89	1.03	1
o/l 10	0.61	0.70	0.74	0.87	1.00	1
o/l 11	0.84	0.95	1.06	1.29	1.51	0.5
o/l 12	0.77	0.83	0.90	1.04	1.17	1.5
Tail	0.72	0.78	0.85	0.98	1.11	1.5

- **Upstream cross structure (outlet structure 5)**

It can be concluded that the presence of a cross structure (drop structure) reduces the impact on the distribution due to a change in the cross section of the canal. For inflow at the head between 60% FSD and 100% FSD, an increase of A.R$^{2/3}$ does not result in a change in allocation of canal water to the upstream outlet structure (R_{mean} = (-)0.02). The cross structure maintains the upstream water level, as defined by its discharge-depth relation. For a decrease of A.R$^{2/3}$ the distribution slightly changed (R_{mean} = 0.17). With the canal running above 100% FSD, there is more impact on the distribution, for a decrease in A.R$^{2/3}$.

- **Tail reach**

Increase of A.R$^{2/3}$ results in an increase of discharge to the outlet structures (12: R_{mean} = 0.14), due to less distribution more upstream in the canal. Decrease of A.R$^{2/3}$ results in a decrease of discharge to the outlet structures (12: R_{mean} = (-)0.18), due to more supply to the upstream outlet structures. There is not enough water left for the tail-enders. Distribution of canal water to tail outlet structures responses more for a decrease in the cross section then for an increase. The responsiveness of the system is depending on a variation of discharge at the head of the distributary: for a low discharge (60% to 80% FSD) the impact on the distribution is substantially higher then for 80% to 120% FSD.

In general: the impact on the canal water distribution for a distributary is low for a change in the cross sectional area of the canal (-20% to + 40%): the factor AR$^{2/3}$ is an insensitive parameter.

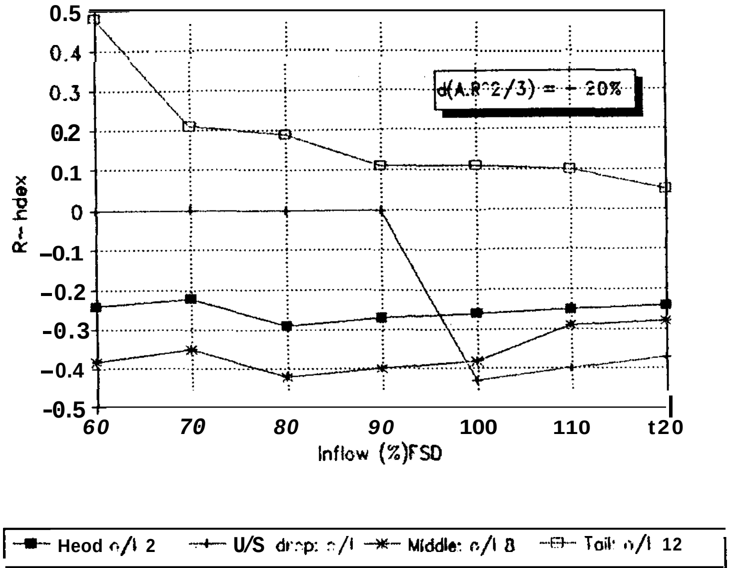


Figure 7.13 K-index values as a function of the inflow at the head for a change in cross sectional profile with: - 20%.

Figure 7.13 presents the computed R-index values as a function of the inflow at the head of the distributary, expressed as a percentage of the FSD. For head, middle and tail reach, the conclusions as stated above are visualized. In table 7.18 the overall mean R-index values are listed.

Table 7.18 Mean R-index values for a change in $A.R^{2/3}$

Outlet structure	R: - 20%	R: - 10%	R: + 20%	R: + 40%	R_{mean} -$AR^{2/3}$	R_{mean}+$AR^{2/3}$
2	+ 0.25	+ 0.28	(-)0.15	(-)0.11	**+ 0.27**	**(-)0.13**
5	+ 0.17	+ 0.17	(-)0.03	(-)0.00	**+ 0.17**	**(-)0.02**
8	+ 0.36	+ 0.29	(-)0.18	(-)0.21	**+ 0.33**	**(-)0.20**
12	(-)0.18	**(-)0.17**	+ 0.14	+ 0.13	**(-)0.18**	**+ 0.14**

The above conclusions are corresponding with the results obtained by Hart (1996), simulating a change in the cross sectional area (between -60% to + 130% of the design value) of the Fordwah distributary. It indicates that the response of the system, either a large distributary (Fordwah) or a small distributary (Masood), due to adjustments of the cross sectional area is similar.

The analysis is conducted for (OC)OFRB outlet structures and a submerged pipe outlet structure only. Analysing the change in upstream water levels. It can be concluded that:

- An increase of $A.R^{2/3}$, results in a decrease of the water level for head and middle reach outlet structures and an increase of the water level for tail outlet structures.
- A decrease of $A.R^{2/3}$, results in an increase of the water level for head and middle reach outlet structures and a decrease of the water level for tail outlet structures.
- Within the reach of the backwater curve upstream of drop structures, water levels are kept constant for an increase of $A.R^{2/3}$. A decrease of $A.R^{2/3}$ can affect the discharge-depth relation of a drop structure. For high inflow at the head (> 90% FSD), water levels are raising.

7.7 Evaluation

Evaluation of the sensitivity analysis of the different parameters in the model results in the following list of so-called sensitive and insensitive parameters, according to the classification suggested in section 6.4.2 (insensitive: $R < 0.5$) and the range of adjustment used during the analysis. Based on the results of the sensitivity analysis, the 'insensitive parameters' are used to suggest different scenario's to develop a method for simplifying a flow model for a distributary. This will be discussed in chapter 8. When it comes to suggest scenario's to improve the canal water distribution at the secondary level with limited resources, it will be useful to start with adjusting the 'sensitive parameters', as the have the most substantial impact on the water distribution. This will be further discussed in chapter 9.

- Sensitive parameters: discharge coefficients of outlet structures, height and width of the opening of outlet structures, crest levels of drop structures (only a local impact).

- Insensitive parameters: crest level of outlet structures (measured above bed level), for a low ratio CL/WL canal (OFRB: < 0.35; AOSM: < 0.30; OF: < 0.24; PIPE: < 0.15), width of a drop structure, Manning's coefficient, Seepage (inflow and outflow) and cross sectional profile (AR^2/3).

The sensitivity of 'crest levels of outlet structures' is increasing for low discharges at the head of the distributary, as the ratio CL/WL increases. Besides that, due to the non-proportionality of most of the outlet structures (see section 6.5), disturbances are transported to the tail end of the canal. It can be expected that inaccuracies of the simplified flow model simulations occur for low discharges at the head (low water table in the canal) and for the distributed discharges to the tail outlet structures.

Manning's coefficient	- 20 %	- 10%	+ 10%	+ 20 %
outlet 2:	(-)0.33	(-)0.36	0.37	0.30
outlet 5:	(-)0.00	(-)0.05	0.08	0.15
outlet 8:	(-)0.21	(-)0.18	0.18	0.19
outlet 12:	(-)0.28	(-)0.22	(-)0.30	(-)0.24
Inflow seepage losses	- 100 %	- 40 %	+ 40 %	+ 100 %
outlet 12:	(-)0.07	(-)0.07	0.06	0.06
Outflow seepage losses	- 100 %	- 40 %	+ 40 %	+ 100 %
outlet 12:	0.10	0.11	(-)0.11	(-)0.11
Cross sectional area	- 20 %	- 10%	+ 20 %	+ 40 %
outlet 2:	0.25	0.28	(-)0.15	(-)0.11
outlet 5:	0.17	0.17	(-)0.03	(-)0.00
outlet 8:	0.36	0.29	(-)01.8	(-)0.21
outlet 12:	(-)0.18	(-)0.17	0.14	0.13

- **Sensitive parameters:** discharge coefficients of outlet structures, height and width of the opening of outlet structures. crest levels **of** drop structures (only a local impact).

- **.Insensitive parameters:** crest level of outlet structures (measured above bed level), for a **low** ratio CL/WL canal (OFRB: < 0.35; **AOSM:** < 0.30; OF: < 0.34: PIPE: < 0.15), **width** of a **drop** structure, Manning's coefficient, Seepage (inflow and outflow) and cross sectional profile ($AR^{2/3}$).

The sensitivity **of** 'crest levels **of** outlet structures' is increasing for low discharges **at** the head **of the** distributary, as the ratio CL/WL increases. Besides that, due **to** the non-proportionality **of** most **of** the outlet structures (see section 6.5), disturbances are transported **to** the tail end **of** the canal. It can be **expected** that inaccuracies of the simplified flow model simulations occur for **low** discharges at the head **(low** water table in the canal) and for the distributed discharges **to** the tail outlet structures.

The analysis is conducted for (OC)OFRB outlet structures and a submerged pipe outlet structure only. Analysing the change in upstream water levels. It can be concluded that:

- An increase **of** $A.R^{2/3}$, results in a decrease of the water level for head and middle reach outlet structures and an increase of the water level for tail outlet structures.
- **A decrease of** $A.R^{2/3}$, results in an increase of the water level for head and middle reach outlet structures and a decrease of the water level for tail outlet structures.
- Within the reach of the backwater curve upstream of drop structures, water levels are kept constant for an increase of $A.R^{2/3}$. **A** decrease of $A.R^{2/3}$ can affect the discharge-depth relation of a drop structure. For high inflow at the head (> 90% FSD), water levels are raising.

7.7 Evaluation

Evaluation of the sensitivity analysis of the different parameters in the model results in the following list of so-called sensitive and insensitive parameters, according to the classification suggested in section 6.4.2 (insensitive: R < 0.5) and the range of adjustment used during the analysis. Based on the results of the sensitivity analysis, the 'insensitive parameters' are used to suggest different scenario's to develop a method for simplifying a flow model for a distributary. This will be discussed in chapter 8. When it comes to suggest scenario's to improve the canal water distribution at the secondary level with limited resources, it will be useful *to* start with adjusting the 'sensitive parameters', as the have the most substantial impact on the water distribution. This will be further discussed in chapter 9. In table 7.19, all interventions with the corresponding mean R-index values are listed.

Intervention and outlet structure	**+ + % adjustment** **R mean**	**+ % adjustment** **R mean**	**- % adjustment** **R mean**	**- - % adjustment** **R mcnn**
Width B drop structure outlet: 5	-	- 25% 0.37	+ 25% (-)0.26	
Crest level drop strurture outlet 5:	- 40 % (-)0.53	- 25 % (-)0.64	+ 25 % 0.80	+ 40 % *0.77*

CHAPTER 8 DEVELOPMENT OF A SIMPLIFIED HYDRAULIC FLOW MODEL

8.1 General

The overall aim of this chapter is to develop a so-called ***simplified method*** to set up a hydraulic flow model for a distributary, based on the results of chapter 7, i.e. the 'insensitive parameters'. When it comes to simplifying input data of flow models, it will be useful to simplify only those parameters, which impact on canal water distribution is limited (as discussed in chapter 7). The approach will be a general description which steps to be taken, and will be more specified for distributaries in the area of study. **As** the traditional way of developing flow models takes **a** lot of time and requires a lot of field data, a simplified model could be useful to save ***time*** and ***money*** for an initial analysis of the actual canal water distribution and performance of a distributary. In this chapter the development, limitations, boundary conditions and calibration of the simplified approach will be discussed.

8.2 Simplified scenario's

In chapter 5, the initial input data of the hydraulic flow model **(SIC)** is discussed. To evaluate different simplifications of the input parameters of the model, ***5*** scenario's are tested. The simplifications of the different scenario's are based on the results of the sensitivity analysis of chapter 7. **As** the simplification has to result in a methodology to set up a flow model with limited resources (time and money). The time consuming aspects of developing flow models will be analysed and simplified, i.e. the topographical survey (bench marks and cross sectional profile measurements) and calibration of outlet structures and drop structures. The results of the scenario's will be used for the development of the simplified method, described in section **8.3.** The following scenario's are simulated, **based** on the 'insensitive parameters of section 7.7:

- ***Actual physical state of the distributary with theoretical crest levels for outlet structures and drop structures.***
- ***Simplification of the geographical inputfiles: minimize the number of cross sections.***
- ***Actual physical state of the distributary with the calibrated (IIMI) discharge coefficients and theoretical coefficients for the outlet structures and drop structures.***

Besides that, two more scenario's are simulated, to check the possibility to simplify the downstream rating curve of the model and the necessity to incorporate illegal closure **of** outlet structures:

- ***Simplification of the downstream rating curves for submerged outlet structures and tail of the distributary.***
- ***Illegal closure of outlet structures.***

8.2.1 Actual physical state of the distributary with theoretical crest levels for outlet structures and drop structures.

The topographical layout (longitudinal profile) of a canal determines the horizontal slope **of** the canal bed. Crest levels of outlet structures, cross structures and elevation of typical cross sections are all related to the topographical profile of the canal. In general, the elevations are expressed in a certain height referred to the head of the canal. To determine the reference elevations of the crest levels of outlet structures, cross structures and typical cross sections, bench marks should be established by means of a topographical survey along the canal. Such a survey takes a long time: *approximately* 2 ***to 3*** *days for 10 kilometres*[1]. To avoid these types of elaborate surveys would be an achievement for the simplified approach.

Two methods were tested to be able to determine the crest levels of outlet structures, drop structures and the elevation of cross sections, *without* ***a*** *topographical survey.*

Method 1
For a certain constant discharge at the head of the distributary, there will be a steady state water profile in the canal (for example the 15-11-1995 measurement data set: Q_{head}, h_u). With the assumption that the slope of the water profile i_c equals the bed slope i_b, there will be uniform flow in a certain canal section. $A.R^{2/3}$ can be computed after measuring the cross section near the outlet structure. After determining a proper value for the coefficient of roughness, the slope i_c can be computed with the Manning-Strickler equation. **It** will be possible to find the reference elevation of the crest levels of outlet structures with the following equation (explained in figure 8.1):

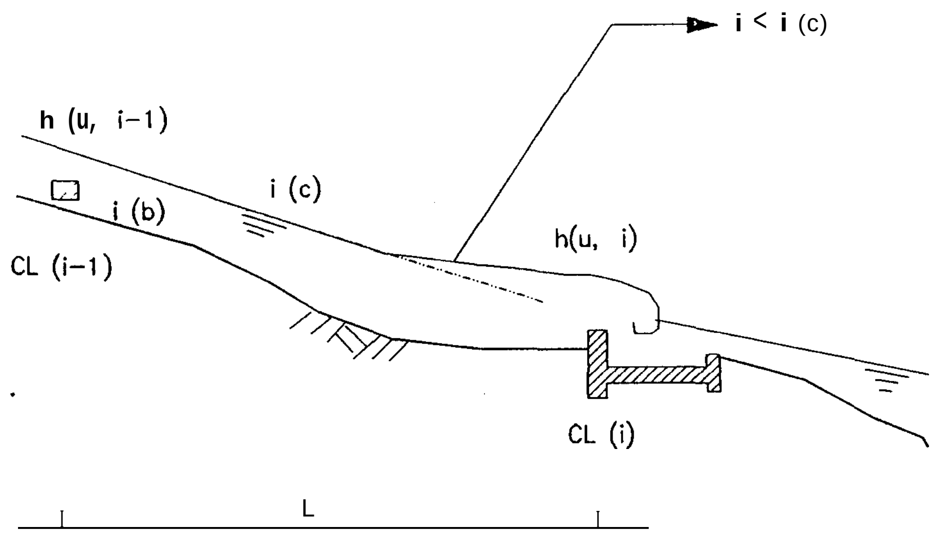


Figure 8.1 **Schematic representation of a canal to determine crest levels (method 1).**

1 *Based on the survey held for Masood distributary.*

$$CL_i = CL_{i-1} + h_{u,i-1} - (L*i_c) - h_{u,i}$$

Whcrc:

CL_i	=	Crest lcvcl i^{th} outlet structure	[m]
CL_{i-1}	=	Crest lcvcl $(i-1)^{th}$ outlet structure	[m]
$h_{u,i}$	=	Upstream water lcvcl i^{th} outlet structure	[m]
$h_{u,i-1}$	=	Upstream watcr level $(i-1)^{th}$ outlct structure	[m]
i_c	=	Slope watcr profilc canal	[-]
L	=	Distance bctwccn tlic outlet structures	[m]

When the crcst lcvcls of outlct structurcs arid cross structures are dctennincd, it will bc possiblc to measure the typical cross sections of the canal referred to the corresponding crest levels. Limitations of **this** mcthod arc:

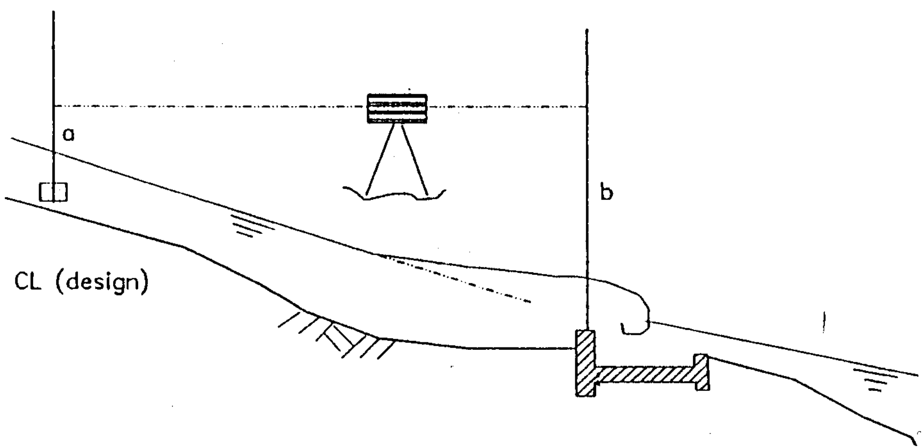
- Still, a lot of data arc necessary: a steady state data set will be necessary for **all thc** canals which must be modelled.
- The assumption that thcre will be uniform flow in the canal docs not count for the sections just upstream cross structures. Positive and negative back water curves resulting in a false prediction of the theoretical crcst level values: $i </> i_c$ (as shown in figure 8.1).
- In gcncral, crest levels of outlet structures (free flow, o.m.) are not that sensitive for changes coniparcd with crcst levels of drop structurcs and submerged pipe structures, **as** found in thc thcorctical analysis. This mcthod is especially inaccurate for drop structures (and for Masood distributary a submcrged pipe outlet structure), so inaccuracieg will occur.
- Errors upstream are transplanted downstream and accumulation of errors resulting in a low accurate prediction at the tail.

Method 2

As found in the analysis, in the end only the reference elevation of the water level above the crest of a structure determines the discharge, so crest levels can be expressed in a theoretical value if the water level in thc canal is related with this value. For a certain discharge at the **head** and a certain value for the roughness coefficient, water levels are determined by the factor $A.R^{2/3}$ (slope i is determined by the reference elevation of the cross sectional profile). As the impact on the water distribution is limited for slight changes of the crest levels of outlet structures (sensitivity related to the ratio CL/WL), an increase in height of the outlet structure crest level together with the same increase in height of $A.R^{2/3}$ does not effect the distributed discharge. The initial (theoretical) crest levels of the outlet structures and drop structures are defined by the *design* **crest levels.** In general, the values of the design crest levels are available with the Irrigation Agency (PIPD). In case of the Chishtian Sub-Division, all the design crest levels of outlet structures are available. The design values **for** cross structures are not always available, and are not very reliable. ***The cross sections close to the outlet structures are measured in the field, referred to the design crest levels of the corresponding outlet structures.*** **For** drop structures thcre are two possibilities:

- Drop structure close (in between 75 metres) to an outlet structure (see figure 8.2):
  The crest level of the outlet structure is the initial elevation point. The elevation of the drop *can* be measured referred to the crest of **the** outlet. The cross sections upstream and downstream of the drop can be measured, with reference to the crest of the outlet structure.

- Drop structure *not* close (> 75 metres) to an outlet structure (see figure 8.2);
  The crest level of the outlet structure nearby is the initial elevation point. The elevation **of** the drop **can** be measured referred to the crest of the outlet, by means of **a small** survey. The cross sections upstream **and** downstream of the drop can be measured, with reference to the crest of the outlet structure.



CL (theoretical) = **CL (design) + a** − b

Figure **8.2** **Schematic representation of a canal to determine crest levels (method 2).**

To be able to **test** this approach with the model of Masood distributary, the following adjustments were implemented:

- Al the crest levels of **the** outlet structures were set at the design value (source: PIPD).
- The cross sections nearby outlet structures were adjusted proportional with the adjusted value of the crest level. For example outlet structure no. 2: CL_{design} = 148.71 m, CL_{actual} = 148.98 m, *so* both the crest as the complete cross section was reduced with 0.27 m.
- The two cross section in between outlet structures were deleted.
- The crest level of drop 1 was reduced with 0.03 m (referred to the new crest level **of** outlet structure 4).
- The crest level of drop 2 was reduced with 0.19 m (referred to the new crest level of outlet structure 5).
- The crest level of drop **3** (down stream boundary condition) **was** reduced with 0.14 m (referred to the new crest level of outlet structure 12).

- The same counts for the values of the theoretical crest level and downstream water level of the rating curves for the submerged outlet structures. They were all reduced proportional with the difference between design and actual crest level.

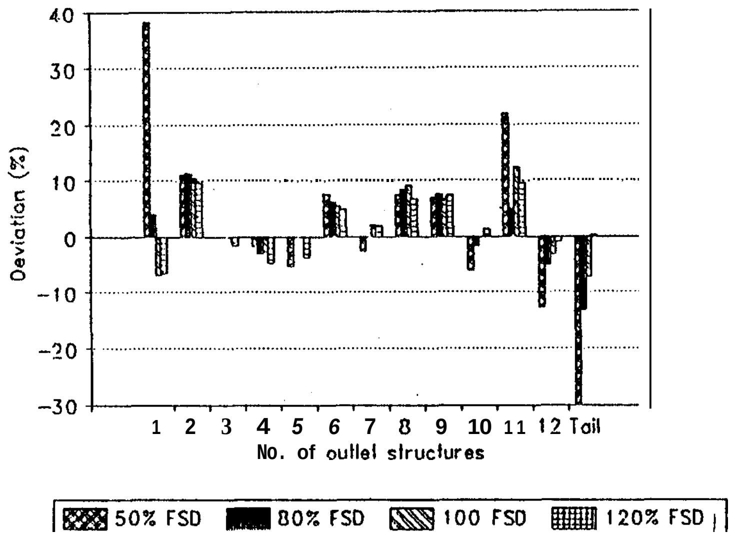


Figure 8.3 **Difference between computed discharges (%) for the outlet structures of the actual SIC model and the simplified model, for different inflow at the head.**

Based on the adjustments listed above, the geographical files of SIC are transformed into *simplified files* **as** if there **was** no topographical survey, but only detailed measurements of the cross sections with reference to the corresponding design crest levels of the outlet structures. In figure **8.3,** for different inflow **at** the head of the distributary, the difference (expressed as a percentage) between computed discharge with the actual SIC model and the simplified model is listed.

It can be concluded that for low discharges at the head the accuracy is poor. Between 80% FSD and 120% FSD the accuracy is in between 0% and 12%. This means that with the proposed method, i.e. without a topographical survey, the distributed discharges to the outlet structures are computed properly[7] by the model for Masood distributary.

During the simulations, it was found that the dynamical computation was not possible for discharges **below** 60% FSD, due to fatal errors in the computation. At a certain moment, there occurred super critical **flow** during the computation in reach **6** (at the location of drop structure no. 2). During a normal steady flow computation, the Froud number reads > 1, above the crest of the drop structure **($v > (g.D)^{0.5}$**, shooting water at the drop).

7

The classification 'properly' will be used whenever the deviation between computed discharges of the simplified method compared with the actual computed discharges by the model are in between 0 % and 20 %. The accuracies are not fixed. It depends on the riser of the simplified models what the accuracies should be.

The super critical behavior is due to the adjusted cross sections up and down stream of the reach and adjusted crest level of the drop structure. Up to now, SIC can not handle this problem. Nevertheless, it does not imply that for all canals super critical flow will occur for low discharges, and therefor the simplification method is still interesting to use.

8.2.2 Simplification of the geographical input files: minimize tlie number of cross sections.

Using SIC, at least two cross section for each reach have to be defined. To reduce the amount of field work, simulations were done to minimize the amount of cross sections, and after that to simplify the measurcments. The next steps were taken:

- Remove all cross sections (2) in between outlet structures.
- Reduce the amount of cross sections using the actual topographical files.
- Reduce the amount of cross sections using the simplified topographical files.

Remove all cross sections (2) in between outlet structures.

There is no change in the distribution pattern, after removing two cross sections in between outlet structures (at 1406.7 m and 1748.6 m).

Reduce the amount of cross sections using the actual topographical files.

To remove different cross sections it is important to maintain the typical changes in the cross sectional profile. All cross sections are studied and based on a visual analysis of the cross sectional shape of the canal. There are three types of cross sections which can not be removed: (1) head cross section just downstream of the secondary inlet, (2) upstream and downstream cross section of cross structures, and (3) tail cross section just upstream of the downstream boundary condition. In figure 8.4, the systematical simplification is drawn. Tlie aniount of cross sections is reduced to 7 (approximately 1 cross section : 1.5 km canal).

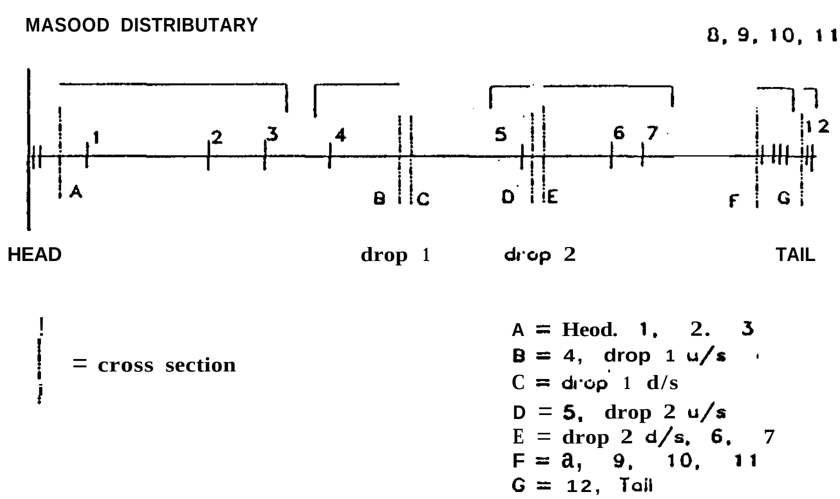


Figure 8.4 Schematic visualization of the reduced amount of measured cross sections.

The simplification **is** modelled in SIC by means of extrapolating the cross sections **(A** to G) to the other points including an initial lowering of the elevation, based on the difference of (1) **mean bank level** or (2) **mean bed level,** between two cross sections. In this way, the actual slope of the canal is maintained. In table 8.1, the results of the simulations are presented, using the ***actual topographicalfiles*** with the reduced amount of cross sections (based on reduction using difference in bed level).

Outlet No.	50% FSD		80% FSD		100% FSD		120% FSD	
	O-option	Simpl.	O-option	Simpl.	O-option	Simpl.	O-option	Simpl.
2	0.043	0.058	0.07	0.07	0.076	0.076	0.082	0 081
5	0.017	0.017	0.021	0.021	0.023	0.023	0.027	0.026
8	0.049	0.054	0.060	0.061	0.066	0.066	0.072	0.070
12	0.053	0.056	0.081	0.083	0.095	0.096	0.106	0.108

It can be concluded thatfor low discharges at the head the qccuracy is poor. Between 80% FSD and 120% FSD the maximum deviation between the actual and simplified model of 2.5 %. This means that with the proposed method, i.e. a reduction of the number of cross sections to approximately 1 measurement every 1.5 km, the distributed discharges to the outlet structures are computed properly by the model, for Masood distributary.

<u>Reduce the amount of cross sections using the simplified topographical files.</u>

The same reduction **of** cross sections are simulated with the simplified topographical files defined in the previous section. The design crest levels of the outlet structures are used as the initial elevations along the canal. To maintain the slope the actual slope between two **cross** sections, the following procedures are tested (see figure 8.5):

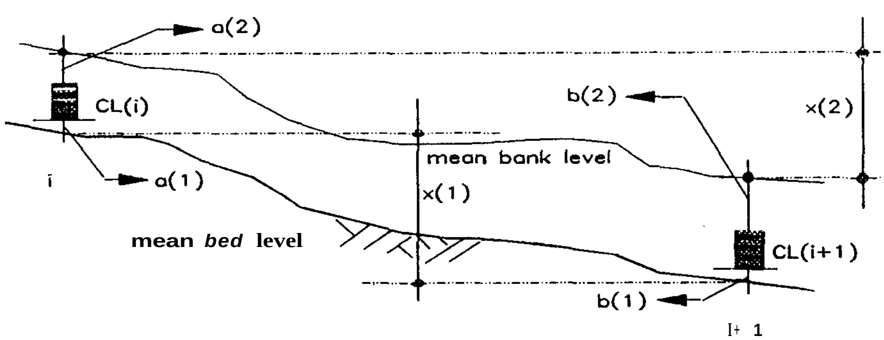


Figure 8.5 **Reduction of the amount of cross sections. Maintain slope using two methods: (1) reduction based on bed level or (2) based on bank level.**

Bed level:

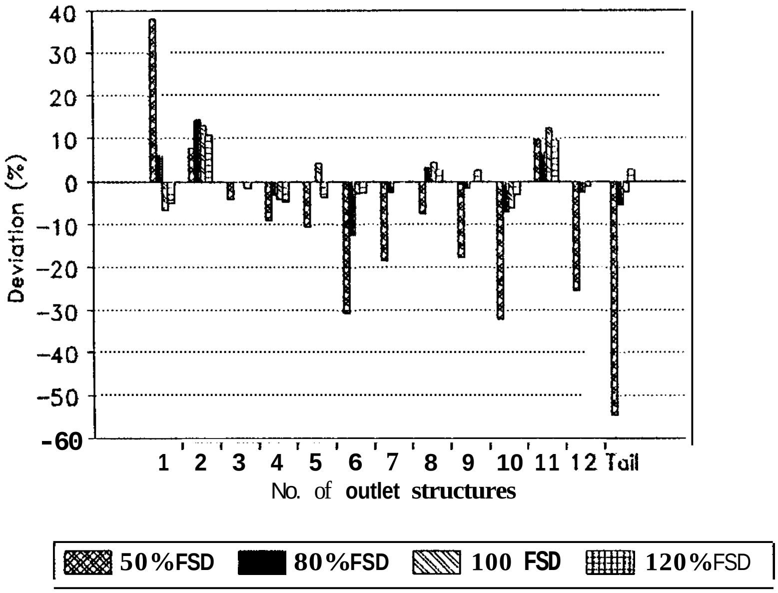


Figure 8.6 **Deviation (%) between actual and simplified model output: simplified topographical files and reduction of cross sections.**

Bunk level:

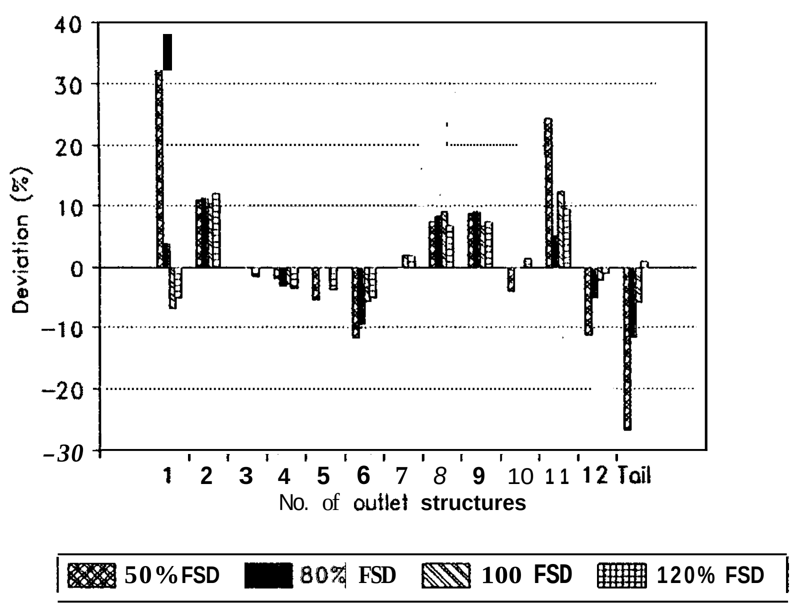


Figure 8.7 **Deviation (%) between actual and simplified model output: simplified topographical files and reduced amount of cross sections.**

For different inflow at the head of the distributary, the difference (expressed as a percentage) between computed discharge with the actual **SIC** model and the simplified model are listed.

- The extrapolation of cross section i to i+1 can be based on a reduction of the elevation, by means of extracting the difference in **bank level:** x_1 (see figure **8.5).** The difference in bank level can be obtained by measuring a, and b,. The reduction factor x_1 will be: $\mathbf{x_1 = (CL, + a,) - (CL,,, + b_1)}$.
- The extrapolation of cross section i to i+1 can be based on a reduction of the elevation, by means of extracting the difference in **bed level:** x_2 (see figure **8.5).** The difference in bed level can be obtained by measuring a, and b,. The reduction factor x_2 will be: $\mathbf{x_2 = (CL, - a,) - (CL_{i+1} - b_2)}$.

Both the methods are simulated and the results are plotted in figure **8.6** and **8.7**

Again. it can be concluded that for low discharges at the head the accuracy is poor. Between 80% FSD and 120% FSD the accuracy is in between 0% and 13%, using the difference in bank level, and between 0% and 12% using the difference in bed level. This means that with the proposed method, i.e. without a topographical survey and minimal number of cross sections (based on a difference in bed level), the distributed discharges to the outlet structures are computed properly by the model, for Masood distributary.

As mentioned earlier, the simplified topographical files causing super critical flow for discharges lower then **60%** FSD above the crest of drop structure 2, for the Masood model. The way the cross sections will be defined in the model is depending on the available time and the possibility of proper data collection. **Cross** sectional profile measurements every 0.5 to 1 metre are recommended (as discussed in annex D).

8.2.3 Actual physical state of the distributary with the calibrated discharge coefficients for the outlet structures.

Especially for large canals, the calibration of the discharge coefficients of the outlet structures is a labour intensive work. To reduce the amount **of** time and field work, the next 2 simplifications are suggested and tested:

- **Method 1:** Use the calibrated discharge coefficients of the outlet structures, as defined by the IIMI field measurements.
- **Method 2: As** the discharge coefficient for outlet structures is approximately constant around its design value for free flow and o.m. flow conditions, take the design coefficient for the simplified model. The values are the initial values (μ: default values) used by **SIC. A** distinction must be made between outlet structures under free flow (and o.m.) or submerged flow conditions.

Free flow (o.m):		**Submerged flow:**	
APM:	*0.60*	*APM:*	*measured in the field*
OFRB:	*0.53*	*OFRB:*	*measured in the field*
OF:	*0.37*	*OF:*	*measured in the field*
PIPE:	*0.70*	*PIPE:*	*measured in the field*

The two proposed methods are tested with the actual topographical files of Masood distributary and the **results** are listed in figure **8.8** and **8.9. It** can be concluded that the accuracy for both methods is poor for a low discharge at the head (50% FSD). Between **80%** FSD and **120%FSD,** the accuracy is fair. There is a difference up to 20% between actual computed and simplified computed discharges. Method 2 (figure **8.9) will** be recommended, to minimize the amount of field measurements and reduce the chance of errors and inaccuracies. **Conclusion: use the theoretical values and calibrate only the submerged outlet structures.**

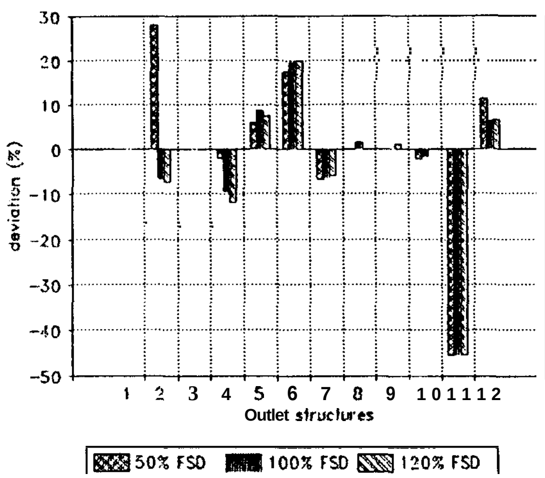


Figure 8.8 **Discharge coefficients based on the IIMI field calibration measurements.**

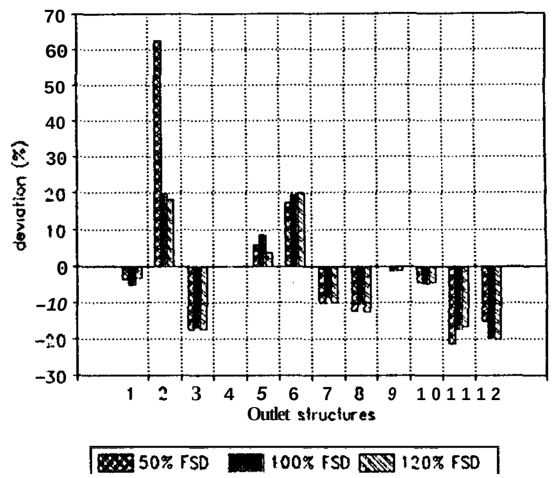


Figure 8.9 **Discharge coefficients based on theoretical values and measurements for submerged types.**

To measure in a quick way the distribution to all the outlet structures this method can be used too: (1) Steady state in the canal; (2) Measure discharge at the head and tail of the canal; (3) Free flow and o.m. flow condition outlet structures: measure upstream water level above crest (compute the discharge); (4) Submerged flow condition outlet structures: calibrate the structures (measure discharge). This rapid Inflow - Outflow method can be used for the calibration of the simplified method to set up a flow model (see section 8.4) and to define the seepage losses in the canal.

8.2.4 Simplification of the downstream rating curves for submerged outlet structures and tail of the distributary.

Outlet downstream boundary condition

As the downstream rating curve for submerged outlet structures is already a simplification of the actual dynamical behavior of watercourse variation and discharge variation in the parent canal, no further simplifications are suggested.

The method of developing a rating curve is described in section 6.3.2, and it will take one measurement for each submerged outlet structure only. The simplifications of the crest levels and topographical files must correspond with the values for the theoretical crest elevation and downstream water level defined in the rating curve. Looking at the results of the simplifications up to now, the rating curves of the outlet structures **4**, 5 and **6** are computing the distributed discharge properly (up to 20% deviation for outlet structure 6, figure 8.8).

Tail downstream boundary condition

Each model requires a proper defined downstream rating curve at the tail of the distributary. In general, there are three possibilities, depending on the physical lay out of the canal.

(1) The downstream boundary condition consist of a tail drop structure, as used in the Masood model. The rating curve will be defined by the discharge-depth relation above the crest for free weir flow. Data necessaiy: width B, simplified crest level and discharge coefficient for free flow (C, approximately 0.95).

(2) 'I'he downstream boundary condition consist of a cluster of tail outlet structures. The rating curve will be defined by the discharge-depth relation above the crest for free weir flow in case of an open flume, or orifice flow in case of an AOSM or OFRB. The final tail outlet structure will be defined as the downstream boundary condition. Data necessary: width B, simplified crest level, opening height Y and discharge coefficient for free flow (C_d approximately 0.95 for weir flow; $C_d = 0.53$ (OFRB), and 0.90 (AOSM) for orifice flow).

(3) The downstream boundary condition consist of a depth-discharge relationship of the canal itself. Data necessary: width B, bank slope' n, bed slope **i** and roughness coefficient k (Strickler).

As the first two options are already defined in the SIC model of Masood, only the last option will be simulated and analysed what the impact is on the distribution. The rating curve will be defined based on Manning-Strickler, with:

- width B = 1.5 m
- bank slope n = 1.5
- bed slope i $= 4.9 * 10^{-3}$
- roughness coefficient k $= 20\ m^{1/3} / s$

Replacing the old downstream **boundary** condition and simulating the actual model, it can be concluded that **the** impact is limited to the last two to three outlet structures at the tail (see figure 8.10). 'I'he impact is depending on the discharge in the parent canal.

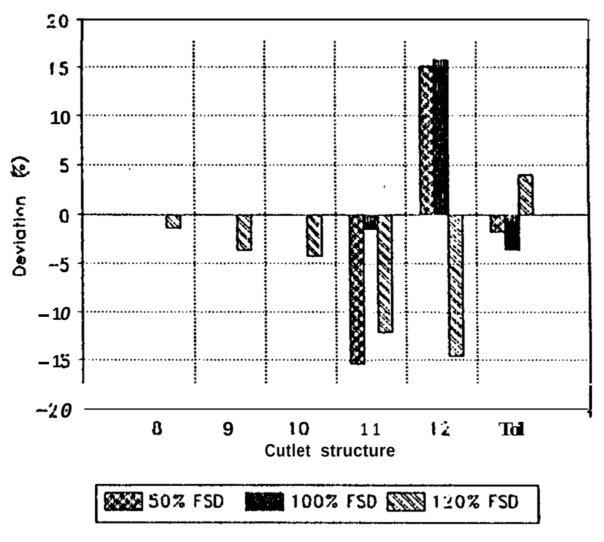


Figure 8.10 **Impact of a change in tail downstream boundary condition.**

8.2.5 Illegal closure of outlet structures.

The impact on the distribution, when ever an outlet structure is illegally closed by the farmers is listed in figure 8.11. For this purpose, outlet structure 1 and 2 were closed during simulation. There is an increase of distributed discharge to the outlet structures (approximately 10%).It can be concluded, that the upstream disturbance is distributed towards the tail due to non-proportional behaviour of the outlet structures. The impact on the canal water distribution at the tail is substantially higher.

Further, it can be stated that the impact on the canal water distribution is depending on the inflow at the head of the distributary. For low discharges in the canal, the impact is more pronounced.

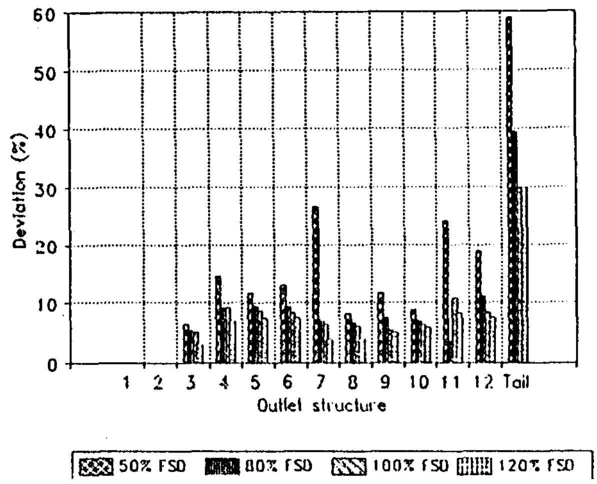


Figure 8.11 Impact closure of an outlet structure (1 and 2) on canal water distribution.

8.3 A simplified flow model general approach

The analysis of the responsiveness of the system for general hydraulic characteristics is described in chapter 7. Section 8.2 presents the analysis of different simplified scenario's based on the sensitivity analysis of chapter 7. Based on that, the next simplified approach to develop a hydro dynamical flow model for distributaries, is suggested:

1. **Inventory of the** topographical layout **of the** distributary

For each distributary the different nodes must be defined, therefor information is necessary about: (1) total length of the distributary; (2) location of al the outlet structures, inlet structure and tail structure (nodes abscissa); (3) location of the cross structures, and (4) location of off taking minors and subminors. The topographical module can be developed in SIC !

2. **Simplified geometrical** module

The geometrical files are based on the simplified approach of section 8.2.1 : cross sectional profiles and the crest levels of the cross structures based on the design crest levels of the outlet structures. Two options are available, either (1) cross sectional measurements for all nodes and cross structures, or (2) minimize the amount of cross sectional measurements and apply the proposed method of section 8.2.2.

3. **Cross devices description**

In general, the cross structures are normal drop structure, without gated openings. Input parameters for drop structures: measured width B (m), crest level elevation as defined in point 2., and discharge coefficient based on mean flow condition.

Whenever the drop structure is working under free flow conditions, the theoretical discharge coefficient is sufficient: Cd = 0.37 (= initial value μ: default value used by SIC). Whenever the drop structure is working under submerged conditions, the discharge coefficient has to be calibrated based on field measurements.

4. Nodes description

In general, the *outlet structures* are either (OC)AOSM, (OC)OFRB, OF or PIPE outlet structures. Input parameters for outlet structures: measured width B (m), measured opening height Y (m), **design** crest level elevation (PIPD), and discharge coefficient based on mean flow condition. Whenever the outlet structure is working under free flow or o.m. conditions, the theoretical discharge coefficient is sufficient (see section 8.2.3), and the downstream boundary condition does not play any role. Whenever the outlet structure is working under submerged conditions, the discharge coefficient has to be calibrated based on field measurements. The downstream boundary condition should be modelled by means of a theoretical rating curve as discussed in section 6.3.2. The ***upstream boundary condition*** (head node of the distributary) exists either of a constant inflow (m^3/s), or a typical inflow pattern Q(t) defined in Unit III of **SIC.** The ***downstream boundary condiiion*** of the model must be a rating curve **as** described in section **8.2.4. If** the tail condition is a drop structure or outlet structure, the discharge coefficient must be determined based on measurements for both free flow and submerged conditions.

5. Manning's coefficient

The initial input of the roughness coefficient, expressed as the Manning's coefficient n, will be **based** on a visual analysis of the physical state of the distributary. Using the descriptive state of a distributary, based on the classification defined by Ven Te Chow (1973), n values **for** certain reaches in the canal can be obtained. It will be suggested to define n values between: ***head - drop structure 1; drop structure 1 - drop structure 2; drop structure 2 - drop structure x; drop structure x*** - *tail.*

6. Seepage

The rate of seepage losses can be simplified taking the seepage as a percentage of the inflow (10% to 20%), or computed by means of a rapid Inflow - Outflow study of the canal: measure the discharge at the head and tail of the canal (steady state in canal **is** necessary), the upstream water levels above the crest of the free flow and o.m. flow condition outlet structures and calibrate the submerged outlet structures to compute the distribution pattern. In this case, the rate of seepage losses within a distributary will be based on the IIMI measurements which took place for all distributaries. **For** small distributaries (**< 15** km): the mean value for S_e for the whole canal will be add in **SIC.** For large distributaries (**> 15** km): the mean value for S_e for reaches up to 10 to **15 km** will be add in SIC. Attention must be paid on the difference of inflow and outflow seepage. For ***outflow seepage a negative value*** must be add. For ***inflow seepage a positive value*** must be add.

In general, the more one simplify, the less accurate the output of the simplified model will be. The proposed different simplified scenario's, with there individual accuracies, will give the user the possibility to select the most appropriate methodology. For example, a global and general prediction of canal water distribution could be **less** accurate then a close examination of the actual performance of a distributary. Four different simplified scenario's are suggested. The estimated accuracies are experimentally based on simulations with the SIC model **of** Masood distributary (see section 8.4), the time spend on developing the simplified models **is** a rough indication **based** on the time spend on developing the model **for** Masood distributary (with 2 man):

1. Simplified hydraulic input file and simplified geometric input file with a limited amount of measured cross sections.

This method is the most simplified and therefor the less accurate. Accuracies for estimating the allocated discharge referred to a fully developed SIC model are in between *10%* to **35%.** In general it will take about 2 day's for every 10 to 15 kilometres canal length to set up these kinds of simplified models. One day will be spend in the field: measuring the cross sections, crest levels of drop structures, visual analysis of the canal to determine the initial roughness coefficient n, and the calibration of the submerged structures and the downstream boundary condition of the model. The other day will be spend on entering the data in SIC. It can be stated that the time spend on developing these models is depending on the skills of the people involved and the availability of the necessary data (design crest levels and a map with the abscissa of the nodes).

2. Simplified hydraulic input file and simplified geometric input file with actual cross sections at the nodes.

This method is less simplified and therefor the accuracy will be better. Accuracies for estimating the allocated discharge referred to a fully developed SIC model are in between *10%* to 25%. In general it will take about 2 to 3 day's for every 10 to 15 kilometres canal length to set up these kinds of simplified models. Extra time will be used for measuring all the cross sections along the canal, referred to the design crest levels of the outlet structures..

3. Simplified hydraulic input file and simplified geometric input file with a limited amount of measured cross sections. Calibration of the simplified model by means of the 'model calibration mode' of SIC: calibration of n-values.

This method is also less simplified and even calibrated for a certain situation, and therefor the accuracy will be better. Accuracies for estimating the distributed discharge referred to a fully developed SIC model are in between *10%* to 20%. In general it will take about 3 to **4** day's for every 10 to 15 kilometres canal length to set up these kinds of simplified models. Extra time will be necessary to collect the field data for the calibration of the Manning's coefficient (n-value) with the SIC model. The calibration procedure is based on several water level measurements along the canal nearby the head of the distributary, several outlet structures and the tail of the distributary (approximately every 3 to 5 km). The calibration method is explained in chapter 5 and annex B.

4. Simplified hydraulic input file and simplified geometric input file with a limited amount of measured cross sections. Calibration of the simplified model by means of adjusting discharge coefficients of outlet structures in the model.

This method is also less simplified and calibrated for a certain situation. The accuracy is the best of all the other methods. Accuracies for estimating the allocated discharge referred to a fully developed SIC model are in between 0% to ***10%.*** In general it will take about ***4*** day's for every 10 to 15 kilometres canal length to **set** up these kinds of models. The calibration procedure is based on an elaborate calibration procedure of **al** the outlet structures along the distributary. A data set 'of the distribution pattern for all outlet structures along the canal will be necessary for a steady state situation in the canal (reach). During the calibration procedure, the discharge coefficients in the model will be adjusted until the computed output of the model match with the field data set. The calibration of al the discharge coefficients is time consuming, but accurate.

8.4 A simplified flow model for Masood distributary

8.5.1 General approach

The four simplified scenario's, as discussed in the previous section are tested and simulated for Masood distributary. First, the simplified input files will be discussed. The listings of these files are printed in annex F.

1. Inventory of the topographical layout of the distributary
   The topographical data for the simplified approach are the same as for the actual model of Masood distributary.

2. Simplified geometrical module
   The geometrical files are based on the simplified approach of section 8.4.

3. Cross devices description
   - drop structure 1 (o.n.): B = 1.22 m; crest level = 148.24 m; discharge coefficient = 1.00.
   - drop structure 2 (o.m.): B = 3.28 m; crest level = 147.92 m; discharge coefficient = 0.37.

4. Nodes description
   - discharge coefficient: theoretical values for the free flow and o.m. flow condition
   - rating curves for submerged outlet structures: No. 4, 5 and 6
   - B, Y, crest: design value (PIPD)
   - upstream: 100% FSD = 1 m^3/s / inflow pattern.
   - downstream: original calibrated drop 3 + rating curve + simplified crest level (146.45 m)

5. Manning's coefficient
   - up to drop 2: 0.025
   - up to o/l 8: 0.030
   - up to the tail: *0.050*

6. Seepage
   Small distributary (< 15 km): mean seepage rate based on the 15-11-1995 IIMI data is + 3.31 *l/s/km*.

1. Simplified hydraulic input file and simplified geometric input file with a limited amount of measured cross sections (without calibration).

This method is the most simplified and therefor the less accurate. Accuracies in estimating the allocated discharge referred to a fully developed SIC model are in between ***10%*** lo **35%.**

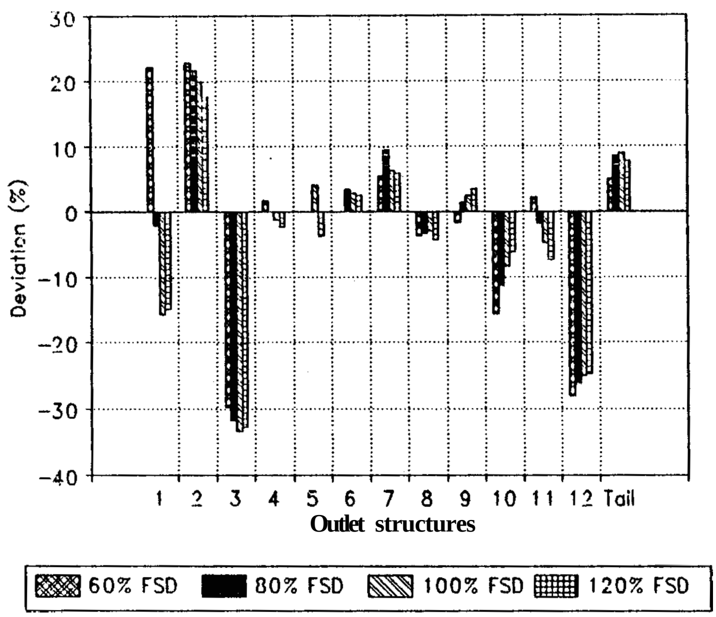


Figure 8.12 Deviation between actual and fully simplified model output, for different inflow at the head.

2. Simplified hydraulic input file and simplified geometric input file with actual cross sections at the nodes (without calibration).

This method is less simplified and therefor the accuracy will be better. Accuracies in estimating the allocated discharge referred to a fully developed SIC model are in between 20% *to* 25%.

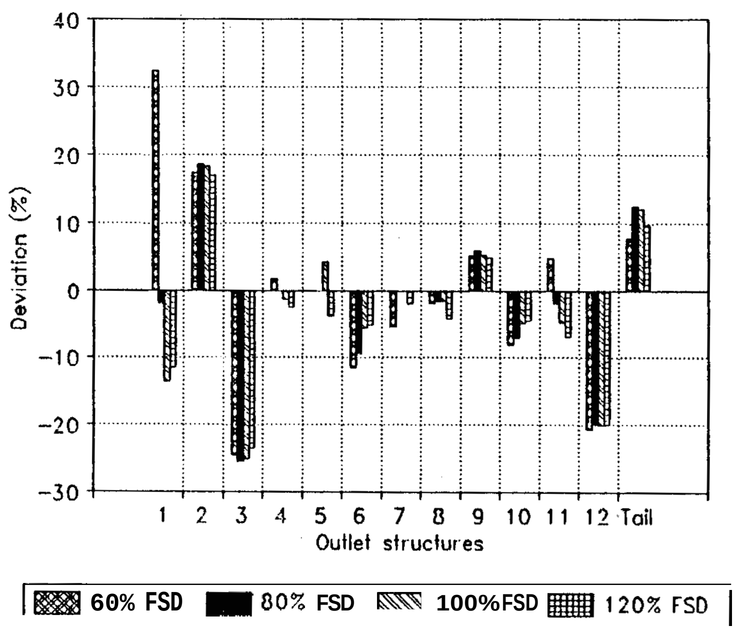


Figure 8.13 deviation between actual and less simplified model output, for different inflow at the head.

3. Simplified hydraulic input file and simplified geometric input file with a limited amount of measared cross sections. Calibration of the simplified model by means of the 'model calibration mode' of SIC: calibration of n-values.

Accuracies in estimating the allocated discharge referred to a fully developed SIC model are in between 10% to 20%. The calibration procedure is based on several water level measurements along the canal nearby the head of the distributary, several outlet structures and the tail of the distributary (approximately every 3 to 5 km). Procedure:

- Using the 15-11-1995 data (IIMI) of Masood distributary: discharge at the head, distributed discharges to all the outlet structures and outflow tail.
- Water level in the canal close to outlet structure: 2, 4, 5, 8 and tail drop structure.
- Using the model calibration mode of SIC to calibrate n based on: (1) pre defined water levels along the canal; (2) all off taking discharges as an imposed value based on there measured values, and (3) head discharge fixed at the measured value.
- Results n calibration: 0.025 up to drop 2; 0.033 up to o/l 8 and 0.042 up to the tail.

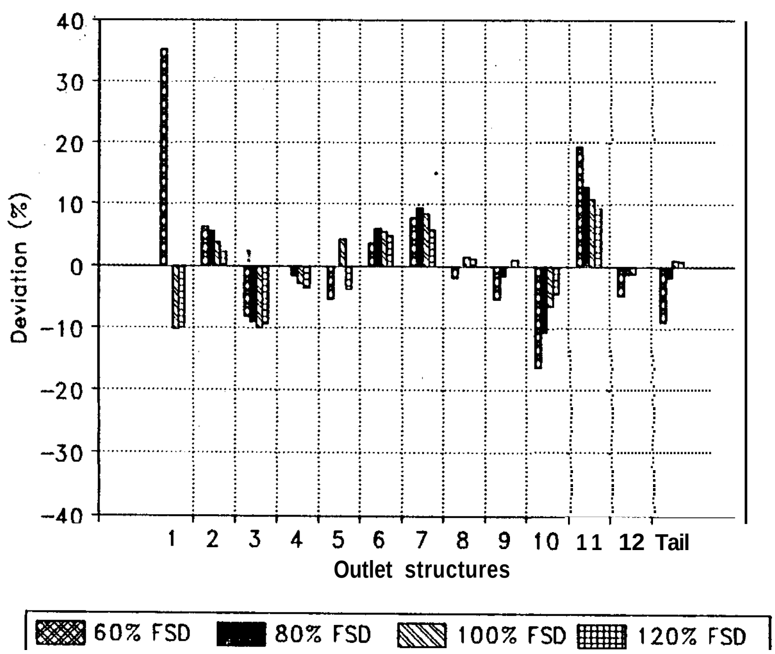


Figure 8.14 Deviation between actual and calibrated and simplified model output, for different discharges at the head. Calibration: Manning's coefficient (n).

4. Simplified hydraulic input file and simplified geometric input file with a limited amount of measured cross sections. Calibration of the simplified model by means of adjusting discharge coefficients of outlet structures in the model.

The accuracy is the best of all the other methods. Accuracies in estimating the allocated discharge referred to a fully developed SIC model are in between 0% to *10%* (except for low discharges at the head). The calibration procedure is based on an elaborate calibration procedure of al outlet structures along the distributary. Procedure:

- Using the **15-1**1-1995 data (IIMI) of Masood distributary: discharge at the head, distributed discharges to all the outlet structures and outflow tail.
- Compare the model output of the simplified model of Masood distributary, i.e. supplied discharges to the outlet structures, with the measured values in the field.
- Adjust the discharge coefficients in the model to match the model output with the measured values.

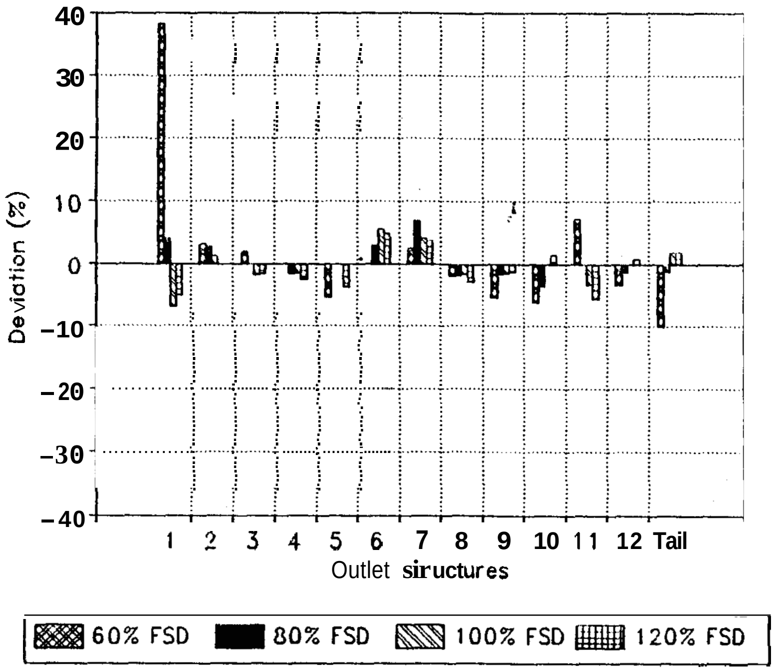


Figure 8.15 **Deviation between actual and calibrated simplified model output, for different inflow at the head. Calibration: discharge coefficients of outlet structures.**

8.5 Validation of the simplified method

To check whether the simplified method for developing a flow model of a distributary, as discussed in section 8.3, is practical when it comes to developing this model for other distributaries, a validation is proceeded for 3-L distributary of the Chishtian Sub-Division. It took 4 hours with two persons to collect all the necessary data, and 2 hours to develop the model in SIC. Evaluating the computed distribution, with the IIMI measurements of 08/10/1995 on 3-L distributary, the accuracy is in between 10% to 25% with the Manning's coefficient (n) calibration method and in between 0% and 10% with the discharge coefficient calibration method. The development of the simplified SIC model of 3-L distributary is described in annex G.

Most of the input parameters changing in time. Due to siltation, erosion, maintenance activities and outlet structure modifications, the parameters changing. To update the existing flow models of distributaries, a validation data set will be necessary (for example: a rapid Inflow - Outflow measurement). Such a validation of existing models should be based on a decrease in accuracy of the output of the models. It was beyond this study to determine the time in between two re-validation procedures of existing (simplified) flow models.

8.6 Evaluation and conclusions

The different simplified scenario's are evaluated in table 8.2. The user can make its own decision, based on the amount of time, money and required accuracy of the simplified models.

Table 8.2 Evaluation of the simplified scenario's

No.	Scenario	Accuracy[3]	Characteristics
1	Simplified hydraulic input file and simplified geometric input file with a limited amount of measured cross sections	10% - 35%	- simplified topographical files (see 8.2.1) - minimum amount of cross sections (see 8.2.2) - simplified hydraulic files: cross structures, nodes description, Manning's coefficient, Seepage (see 8.4)
2	Simplified hydraulic input file and simplified geometric input file with actual cross sections at the nodes.	10% - 25%	- simplified topographical files (see 8.2.1) - simplified hydraulic files: cross structures, nodes description, Manning's coefficient, Seepage (see 8.4)

3 *The accuracy is measured based on the percentage difference between computed distributed discharge of the traditional calibrated model and the computed distributed discharge of the simplified scenario.*

3	As scenario 1 + Calibration of Manning's coefficient in the model.	10% - 20%	- simplified topographical files (see 8.2.1) - minimum amount of cross sections (see 8.2.2) - simplified hydraulic files: cross structures, nodes description, Manning's coefficient, Seepage (see 8.4) - calibration of the manning's coefficient based on measured water levels and distributed discharges along the canal - use the calibration module of the SIC software
4	As scenario 1 + Calibration of discharge coefficients of outlet structures.	0% - 10%	- simplified topographical files (see 8.2.1) - minimum amount of cross sections (see 8.2.2) - simplified hydraulic files: cross structures, nodes description, Manning's coefficient, Seepage (see 8.4) - calibration of the discharge coefficients of all outlet structures, based on measurements of distributed discharge to all outlet structures - calibration by means of adjusting the discharge coefficients in order to match the computed discharge with the measured discharge

CHAPTER 9 IMPROVED WATER MANAGEMENT AT THE DISTRIBUTARY LEVEL

9.1 Discussion

In this chapter, briefly, adjustments at the distributary level will be identified based on the results of the sensitivity analysis (chapter 7), using the hydrodynamic flow model SIC, in order to improve the canal water distribution performance. Here, the performance of a distributary is defined as the evaluation of the canal water distribution to the outlet structures, based on the principles of irrigation in the area of study, i.e. equity and proportionality. The canal water distribution is functioning properly , whenever the supplied discharges to the outlet structures are at there authorized discharge for 100% FSD in the parent canal, the variability of the distribution along the canal for various inflow at the head is sufficient low (CV(DPR < 0.10 or MIQR in between 1 and 1.5, as discussed in section 6.4.3), and the available canal water supply at the head of the distributary is distributed proportionally to all the outlet structures. In other words, the canal water supply is classified as 'good', when the distributary is functioning as designed.

The 'improvement' as described above is a pure technical subject, and therefor relatively. For example, a farmer receiving his canal water from outlet structure 2 of Masood distributary once said to me, that he blamed the PIPD of a lack of interest and capability to operate and maintain the irrigation system. Approximately 15 years ago he used to receive enough canal water supply to meet his crop water requirements, but at present the canal water supply is not sufficient anymore, according the farmer. These day's he has to buy tubewell water to irrigate his fields, although the outlet structure has been modified and is functioning at 1.81 times its authorized discharge with the canal is running on 100% FSD. Due to intensified cropping patterns and variable discharge fluctuations at the head of the distributary, at present the canal water supply does not meet the demand of the farmers anymore. Besides that, the 'improvement' will not only reduce the amount of canal water supply to the head outlet structures, but will increase the canal water supply to the tail-enders. As it is quite common that the more influential farmers in the Punjab are located at the head reach of the distributaries, the 'improvement' could provoke political anxiety in the area. This practical example illustrates the contradiction speaking of improved water management at the distributary level, looking at this individual farmer. The problems of canal water distribution are either upstream (variable inflow at the head of the system) or at field level (intensified cropping patterns). Another contradiction seems the national goal to up boost the agricultural sector (increase the agricultural production) in the Punjab, by means of intensified cropping patterns, High Yielding Variety crops, increase in available crop production inputs and to cultivate more dry lands, all with the limited resources (scarcity of fresh canal water). In order to meet the crop water requirements, an increase in (saline) tubewell irrigation is necessary and therefor the problems of salinity and sodicity increases. So, speaking of improvements at the distributary level, it is necessary to look at the broad context. The common interests of equitable and proportional canal water supply above individual interest, variability at the head of the system and the operations at the main system level.

It is beyond this chapter to analyse this complex situation, and provide an answer on all questions, but one has to take this brought context in mind dealing with improved scenario's at the distributary level in the Punjab.

9.2 Possible measures in order to adjust the actual distribution pattern

The study conducted by Hart (1996), to find a relationship between maintenance and canal water distribution at the secondary level, resulted in the conclusions that:

- measures of desiltation do not results in better results then simple adjustments of outlet structures;
- the best performance (based on the E-index proposed by Hart, see section 6.5, page 86) was obtained by decreasing in size of a limited number of outlet structures together with the raising of the crest of one submerged drop structure, and;
- the distributary is relatively insensitive to variations of the factor $A.R^{2/3}$ (canal maintenance does not improve its performance significantly).

The above conclusions found by Hart, studying Fordwah distributary are similar with the results of the sensitivity analysis of Masood distributary. Adjustments on parameters which will result in a reasonable change of the canal water distribution (with the R-index > 0.5) are: *discharge coefficient, opening height and opening width of outlet structures and crest levels of drop structures.* Besides the question, what has to be improved, from a cost-effective point of view, it will be advisable to start the improvements by adjusting these sensitive parameters in order to change the distribution pattern.

9.3 Scenario's to improve the water management at the distributary level

Before suggesting any scenario for improved canal water distribution, the next points have to be answered:

- Define the boundary's in between the proposed measures should result in any change in canal water distribution. Upstream: actual inflow at the head; tail: actual tail outlet structures.
- Quantify the impact of the adjustment based on an irrigation indicator (or more indicators, as proposed in section 6.4).
- Define the term 'improvement', based on the principles and objectives of irrigation in the area (national goals) to classify an adjustment as 'positive' or 'negative'.

The three points mentioned above, should be defined by the authority responsible for the improvements, in this case the PIPD. Without answering these initial questions, the next scenario's for improved water management are briefly discussed, based on the results of the sensitivity analysis and the study of Hart (1996):

- *Back to design*

As shown in section 6.4, the distribution pattern based on the initial objectives of irrigation in the area, can be obtained whenever the distributary is in its design state. Design crest levels, cross sections, outlet structure dimensions and longitudinal profile result in a more equitable and proportional distribution pattern. Especially, the tail-enders will benefit from these measures.

- *Improved maintenance*

Besides the regular hydraulic maintenance activities like desiltation, kila bushing, berm cutting and non-hydraulic maintenance activities like strengthening of banks and closing breaches (Hart, 1996) during the closure period of the canals (January), it was proposed from a cost-effective point of view to invest in maintenance that does have its impact on canal water distribution: adjusting outlet structure dimensions and adjust crest levels of (submerged) drop structures.

- *Moving towards demand-based operations (change of irrigation objective)*

The idea to match water supplies with the crop water requirements has been debated in Pakistan for quite some time (Bandaragoda, Badruddin, 1992). The initial irrigation objectives in the area were based on a delivery system that assumed full supply level flow of canal water in distributaries, and equitable and proportional delivery to the outlet structures. At present, the rotation schedules are preset (the warabandi system) and do not allow any user control. A more demand-based schedule provide the user with water as he needs it, within the limits of the system capacity. In other words, a shift in irrigation objective will be suggested form 'protective irrigation' to 'productive' irrigation'. Demand-based operations of an irrigation system involves the supply of variable quantities of water during the cropping season. Besides the feasibility in institutional point of view, the following adjustments at the distributary level can be suggested: (1) gate the outlet structures and implement a within distributary rotation that allows each watercourse a reduced time of essential flows, and (2) install gated cross regulators within the distributary to control the water flow (to ensure enough head with the canal running less then design). Constraints affect the possibility of a demand-based operation (IIMI, 1992): (1) the limited storage available is inadequate for intra-seasonal supply regulation; (2) time necessary for adequate response on a change in demand is due to no in-system storage quite long; (3) due to a minimum slope designed for carrying its required discharge and a small number of gated cross regulators at the main system level, there is no scope for increasing the in-system storage, and (4) the canals are unlined and this restricts the range of discharges that can be run so to avoid problems of scour and siltation.

- *Increase the number of drop structures*

As found in the analysis and by the study of Hart (1996), a drop structure clearly makes the portion of the canal directly upstream insensitive for a change in cross sectional profile, depending on the upstream back water curve. The positive effect of drop structures is a more stable water profile for various discharges at the head for a change in cross sectional profile of the canal reach. At present however, the extra head loss necessary for installing more drop structures is not available.

9.4 Rapid assessment of the performance of a distributary, based on the canal water distribution

Again, without define the necessary performance of a distributary, for both additional research on canal water distribution in a certain area and the evaluation of delivered canal water by an Irrigation Agency, it will be useful to have a simplified and quick procedure to determine the actual canal water distribution. Based on practice, it can be concluded that it takes a long time to develop a flow model (with SIC) for a distributary: approximately 3 month's for Masood distributary and 6 month's for Fordwah distributary (Hart, 1996). The simplified method and rapid Inflow - Outflow procedure, suggested in this study, will be useful whenever flow models are necessary to evaluate actual performances and suggested improvements at the distributary level.

Irrigation agencies can use this tool to evaluate there 'product', and research institutes are able to study inter-related processes in irrigated agriculture.

Advantages of this method: less time and money necessary for development and without changing the actual system different scenario's and improvements can be evaluated for various discharges at the head of the distributary. **Disadvantage** of this method: the use of flow models requires knowledge, specialized equipment (personal computers) and expensive software. In general, these are not the appropriate technologies which can be used by Irrigation Agencies in developing countries. On the other hand, cooperation between funded International Research Institutes and National Irrigation Agencies could lead to a solution to cope with the structural lack of resources within Irrigation Agencies, and in the end will result in fruitful cooperation and a better understanding of the system, for both parties.

CHAPTER 10 CONCLUSIONS AND RECOMMENDATIONS

10.1 Conclusions

The conclusions of this study are presented based on the different parts of this study:

1. The analysis of the impact on the canal water distribution for different parameters: the sensitivity analysis.
2. The analysis of the irrigation performance of both the actual and the design state of a distributary.
3. The simplified method to set up a flow model.
4. The analysis of improved water management at the distributary level.

1. The analysis of the impact on the canal water distribution for different parameters: sensitivity analysis. Conclusions:

- A methodology was proposed to study the sensitivity of parameters determining the canal water distribution at the distributary level, simulating a defined inflow pattern at the head of the canal using a flow model of a distributary. An indicator was suggested (R-index) to quantify the impact of a change in different parameters on the re-distribution of canal water to the outlet structures. It can be concluded that the proposed methodology is an easy tool to gain insight in the sensitivity of the different parameters determining the canal water distribution at the distributary level.
- Based on a theoretical analysis to study the impact on the canal water distribution for a change in outlet structure characteristics it can be concluded that: (1) opening width, and discharge coefficient for all types of outlet structures are sensitive parameters, as the R-index = 1; (2) opening height for (OC)OFRB and (OC)AOSM outlet structures is depending on the water level in the canal and the change in opening (dY), and is a sensitive parameter, as the R-index > 0.5; (3) height of the crest level above bed level for all types of outlet structures is an insensitive parameter (R < 0.5) whenever the ratio Crest Level / Water Level < 0.35 (OFRB), < 0.30 (OASM), < 0.24 (OF) and < 0.15 (PIPE).
- Open flume outlet structures are more sensitive for any changes of the parameters compared with other types of outlet structures.
- An increase in Manning's coefficient (the roughness of the canal bed increases) results in an increase of water levels, and therefor in an increase of canal water supply to the head and middle reach outlet structures, and a decrease in canal water distribution to the tail outlet structures due to increased distribution upstream. A decrease in Manning's coefficient (the roughness of the canal bed decreases) results in a decrease of water levels, and therefor in a decrease of canal water supply to the head and a middle reach outlet structures, and an increase of canal water distribution to the tail outlet structures due to decreased distribution upstream. The tail-enders suffer either from a lack of distributed canal water or an abundance of distributed canal water. The impact on the canal water distribution for a change in the Manning's coefficient is limited (R < 0.36).

- Rate of seepage losses do have a limited impact on canal water distribution. Sensitivity of outflow seepage ($0.03 < R < 0.12$) is larger then for inflow seepage ($0.05 < R < 0.20$).
- An increase in the cross sectional profile ($A.R^{2/3}$) results in a decrease of water levels along the canal, and therefor in a decrease of canal water supply to the head and middle reach outlet structures, and an increase of canal water supply to the tail outlet structures due to decreased distribution upstream. A decrease in the cross sectional profile ($A.R^{2/3}$) results in an increase of water levels, and therefor in an increase of canal water supply to the head and a middle reach outlet structures, and a decrease to the tail outlet structures due to increased distribution upstream. Again, the tail-enders suffer either from a lack of distributed canal water or an abundance of distributed canal water. The impact on distribution for a change in the cross sectional profile is limited ($R < 0.33$).
- Impact on water distribution for adjustments on cross structures (drop structures) only within the reach of the upstream backwater curve.
- A drop structure clearly makes the portion of the canal upstream, within the reach of the back water curve insensitive for a change in cross sectional profile ($A.R^{2/3}$). Drop structures maintain water levels upstream for any adjustment when water levels will drop, i.e. an increase in outflow seepage, canal maintenance, or cleaning of the bed profile. More drop structures are resulting in a more stable water profile (less water fluctuations), and therefor in a better canal water distribution.
- In general, the sensitivity of the various parameters is increasing for a low discharge in the canal (< 60% FSD).

- ***Data which are sensitive and should be defined precisely for the simplified flow model*: discharge coefficient, opening width and opening height of outlet structures, crest level and width of cross structures (drop structures).**
- ***Data which are insensitive and can be simplified for the flow model*: cross sectional profile, crest levels outlet structures, seepage (inflow and outflow) and Manning's coefficient.**

2. **The analysis of the irrigation performance of both the actual and the design state of a distributary. Conclusions:**

- At present, the physical layout of the irrigation system is based on the design principles of irrigation in the Punjab, and is still the actual basis for operation and maintenance by the PIPD.
- The modelling of a distributary with the SIC software is accurate for studying canal water distribution: accuracy for the calibrated flow model of Masood distributary up to 5%, based on the difference between computed distribution and measured distribution.
- At least three irrigation indicators are necessary to study canal water distribution (both primary and secondary level), based on the principles of irrigation in the Punjab: (1) Delivery Performance Ratio (DPR); (2) S, proportionality, and (3) MIQR or CV(DPR).
- The performance as per design of Masood distributary is equitable and proportional: the original design of distributaries in the area is optimal.
- The performance at present of Masood distributary is inequitable and non-proportional. Due to modified outlet structures, siltation and variable inflow pattern at the head, the actual performance of canal water distribution at the secondary level is far from optimal (referred to the design principles of irrigation in the area of study: equity and proportionality).

- At present, canal water distribution is characterized by highly variable supply: surplus to the head outlet structures and therefor a lack of canal water supply to the tail outlet structures. Tail-enders are suffering from high variability and irregular supply.
- The principle of equity (expressed by the MIQR and CV(DPR)) is optimal for the design inflow at the head of the distributary (100% FSD), for the design state of the canal.
- At present, changes in inflow at the head of the canal and disturbances are transplanted to the tail, due to sub-proportionality and non-proportionality of most of the outlet structures.

3. The simplified method to set up a flow model for a distributary. Conclusions:

- Theoretical discharge coefficient is accurate for free flow or o.m. flow condition outlet structures: no calibration measurements in the field necessary. Measurement of the upstream water level (above crest) is sufficient to compute the distributed discharge with the corresponding discharg equation.
- General simplified method to set up a flow model for a distributary, based on the simplifications of the insensitive parameters (see conclusions point 1): reduction of time and money to investigate canal water distribution with accuracies up to 10% for the simplified method with calibration of the discharge coefficients and accuracies up to 20% for the simplified method with calibration of the Manning's coefficient. Accuracies are based on the percentage difference between computed distributed discharge with the traditional model and the computed distributed discharge of the simplified model.
- The accuracy of the simplified methodology is depending on the inflow. For low inflow at the head of the canal the accuracy is decreasing, due to increased sensitivity of the simplified parameters for low discharges at the head (low water tables in the canal).

4. The analysis of improved water management at the distributary level. Conclusions:

- Speaking of improvements at the distributary level, it is necessary to look at the broader context: the common interest of equitable and proportional canal water distribution above individual interests, and the variability at the head of a distributary is related to the upstream fluctuations of inflow in the system and the operations at the main system level.
- Whenever any scenario for improved canal water distribution will be suggested, the next points have to be discussed: (1) define the boundary's in between the proposed measures should result in any change in canal water distribution; (2) quantify the impact of the adjustment based on an irrigation indicator, and (3) define the term 'improvement', based on the principles and objectives of irrigation in the area (national goals) to classify an adjustment as 'positive' or 'negative'.

10.2 Recommendations

- Based on the sensitivity analysis and the conclusions draw at point 1 of section 10.1, it can be recommended from a cost effective point of view to invest in maintenance or small measures to improve canal water distribution for those parameters only, which do have a reasonable impact on the canal water distribution: *discharge coefficient, opening height and opening width of outlet structures and crest levels and width of cross structures (drop structures)*. It was found that the factor $A.R^{2/3}$ should be adjusted (back to design) in order to obtain fully proportional and equitable distribution. The impact of a small adjustment of the cross sectional profile does not change the canal water distribution sufficient, therefore this parameter is characterized as an insensitive parameter. To restore the design principles of irrigation of canal water distribution, this parameter should be adjusted though.

- Based on the analysis of the actual and design state of a distributary, it can be recommended whenever an improvement should result in a canal water distribution as per design, i.e equitable and proportional distribution, the best strategy to be followed will be adjusting the different characteristics back to design.
- To analyse the actual canal water distribution pattern of a distributary, or to evaluate different suggested improvements in order to change the actual canal water distribution pattern based on the principles of irrigation (equity and proportionality), one should use three irrigation indicators. To express the variability of distribution along the canal, the Delivery performance Ratio (DPR) can be used. To express the equity of the canal water distribution pattern either the MIQR or CV(DPR) indicator can be used. Finally, to express the proportionality of distribution the sensitivity factor S can be used. The different irrigation indicators can be applied local, i.e. for one outlet structure only, or global, for all outlet structures along a distributary.

Recommendation for a rapid assessment of the canal water distribution at the distributary level

- In order to obtain a quick insight in the actual distribution pattern of canal water to the outlet structures and to compute the seepage losses in the canal, a Rapid Inflow-Outflow method is suggested. The inflow during the measurements should be constant, in order to obtain a steady state in the canal. (1) Measure discharge at the head and tail of the distributary; (2) calibrate the submerged outlet structures; (3) measure the upstream water levels above the crest of free flow and orifice modular flow condition outlet structures; (4) determine the characteristics of the outlet structures (opening width and opening height); (5) compute the discharge to the free flow outlet structures based on the discharge equation and the corresponding theoretical discharge coefficient, and (6) compute the seepage (Inflow = Outflow + distribution to outlet structures + seepage).
- For Research Institutes and Irrigation Agencies it can be recommended to make use of the simplified method to set up a flow model, to evaluate canal water distribution based on different measures (actual pattern, maintenance strategies, improvements). The simplified method saves time and money whenever hydrodynamic flow models will be developed of distributary canals.

Recommendation for further research

- More research should be done on determining the validation of developed simplified flow models of distributary canals. To what extend, the different parameters in the model have to be re-measured in the field in order to increase its accuracy.
- Up to now, the sensitivity analysis did not analyse the sensitivity of adjustments on gated cross structures. In order to incorporate gate cross structures in the simplified method, new simulations and measurements should be done.
- The simplified method of developing a flow model is a quick tool in order to investigate actual canal water supply. It will be useful, to investigate the possibilities how to make this kind of technology available for local Irrigation Agencies (PIPD, WAPDA), to evaluate there product, i.e. the distribution of canal water, and to obtain more knowledge about the system.

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ANNEXES

ANNEX A FIELD MEASUREMENTS

Topographical survey Masood distributary

In the files of the PIPD, only one detailed longitudinal section of the Masood distributary was available. This map, dated 13-11-1963, presented the Masood distributary off taking at RD 31,638 of the Fordwah Branch with the design tail at RD 52.30. Due to a lack of reliable topographical data, i.e. crest elevations and cross sections of the actual situation, a four day survey was held. Three days were spend (03, 04 and 05-12-1995) on establishing bench marks along the canal with the crest elevation of the (intake) head structure of Masood as initial point: **486.82 ft (148.38 m)**. In total 17 bench marks were established and they will also be used by future IIMI field activities. The bench marks are listed in table A.1.

Table A.1 Bench marks along Masood distributary

Benchmark No.	Description	Location (RD)	Elevation (ft)	Elevation (m)
0	head level	0	486.820	148.383
1	o/l 1	1100	491.750	149.885
2	o/l 2	3700	491.540	149.821
3	cutout tree	4700	492.395	150.082
4	o/l 3	7300	490.275	149.436
5	RD stone	8500	494.295	150.661
6	cutout tree	10500	493.760	150.498
7	RD stone	12000	492.830	150.215
8	o/l 4	135000	488.860	149.005
9	cutout tree	15140	491.820	149.907
10	drop 1	18000	489.470	149.190
11	o/l 5	24050	487.305	148.531
12	o/l 6	27200	486.300	148.224
13	o/l 7	28750	485.808	148.074
14	o/l 8	34860	483.873	147.484
15	o/l 9, 10	35600	484.148	147.568
16	o/l 11	36620	483.180	147.273
17	o/l 12, tail	37250	483.280	147.304

On 09-12-1995 based on the different benchmarks along the canal, 20 typical cross sections were established. Also the elevations of the White Marks ($Ha_{,wm}$ upstream and $H_{b,\,wm}$ downstream) and the crest levels of the 12 outlet structures, the 2 drop structures and the tail drop structure, with reference to the benchmark elevation were measured (for an explanation of White Marks see figure A.1). The measured data are listed in table A.2.

Table A.2 White Mark and crest elevations along Masood distributary

Outlet No./ head/ tail and cross structures	Ha, WM (m)	Hb, WM (m)	Crest level (m)
Head	-	-	148.383
1	149.861	149.727	148.947
2	149.977	149.855	148.980
3	149.454	149.079	148.547
4	-	-	148.133
drop 1	149.276	148.869	148.270
5	-	-	147.781
drop 2	148.512	-	148.113
6	148.133	148.172	147.392
7	148.095	147.930	147.397
8	147.481	147.387	146.838
9	147.379	147.644	146.733
10	147.448	147.305	146.756
11	147.193	147.153	146.644
12	147.029	147.084	146.453
Tail	147.351	-	146.594

Hydraulic data collection for the calibration and validation of the model

IIMI field measurements 15-11-1995

The first set of data, used for calibration of the SIC model of Masood distributary, was obtained by the IIMI calibration survey of all the 14 distributaries of the Chistian Sub-Division. The exercise of the Masood distributary was started at 06:00, 15-11-1995.

Methodology

- At least 5 hours before starting the exercise the gate reader was instructed not to change the gate settings (gated orifice-type off take) at the head of Masood distributary, in order to establish a steady state condition in the canal during the exercise.

- White Marks (WM) were established both upstream and downstream of all outlet structures, width, height and elevations of the upstream and downstream water level were measured (referred to the WM). Almost all structures in the Chishtian sub-division have been provided with White Marks to measure h_u and h_d. The White Marks are referred to the crest. The advantages of this method is that both the measurements of the upstream and downstream water levels are independent of the physical condition of the parent canal and the corresponding watercourse. By means of a simple equation both h_u and h_d can be obtained: $h_u = H_{a,WM} - h_a$ and $h_d = H_{b,WM} - h_b$. In figure A.1 the White Marks on structures in the Chistian Sub-Division are presented.
- For each outlet structure the discharge was measured by means of a current metering (for lined watercourses) or a fluming test. After that, the discharge coefficients were computed using the corresponding equation, based on the measurements, type of outlet structure and typical flow condition.
- To check if the inflow of water in the distributary was constant and the canal is in steady state, every half an hour the upstream and downstream water elevation referred to the WM's of the gate at the head of Masood distributary was measured. Also the discharge some distance downstream of the head was measured by means of a current metering.
- To calculate the seepage, a water balance based Inflow-Outflow method was used: *Seepage = Inflow - Outflow - Total supplies to the outlets.*
- The tail of the distributary was assumed to be at about RD 45.95. Further downstream the canal hardly received any water and the supply to the outlet structures is neglectful.

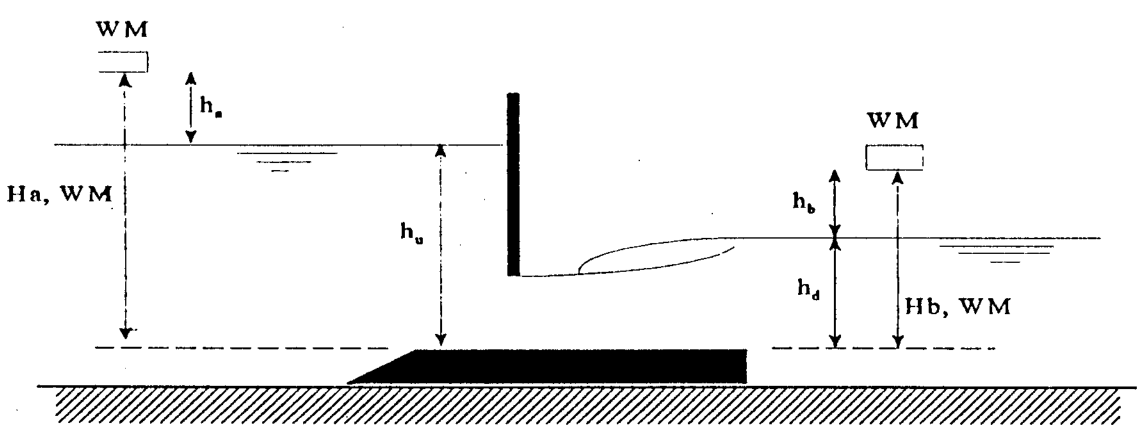


Figure A.1 **White Marks on outlet structures in the Chishtian Sub-Division.**

Hydraulic situation

At the head of Masood distributary the discharge was constant throughout the day from 07:00 a.m. to 5:30 p.m., at 23.1 cfs. The flow through the head structure of Masood was submerged. Also the discharge at the tail (RD 45.95) was constant. Both discharges are presented in figure A.2.

Measurement results

The results of the measurements and hardware calibration are listed in the next table. Table A.3 contains the measured discharges q, measured h_a and h_b, computed h_u and h_d, corresponding discharge coefficients and observed flow condition. To calculate the different Cd coefficients the equations presented in chapter 4 were used.

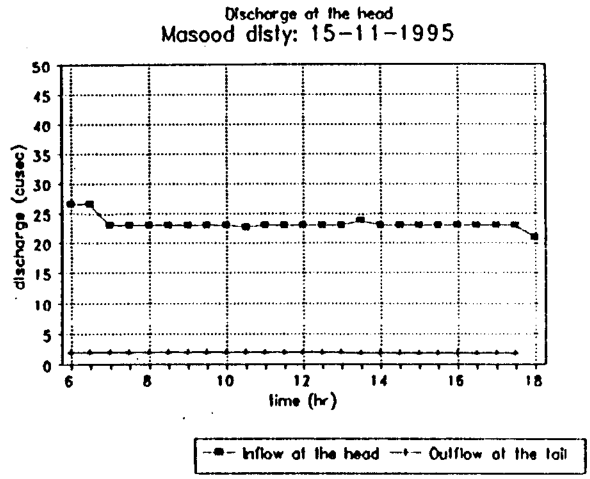


Figure A.2 **Discharge at the head and tail during the 15-11-1995 measurements.**

Table A.3 **Discharge measurements and calibration of outlets: 15-11-1995**

Outlet No.	Location (RD)	h_a (ft)	h_b (ft)	h_u (ft)	h_d (ft)	q (cfs)	C_d	Flow condition
1	1.10	-	-	-	-	-	-	closed
2	3.70	0.81	2.52	2.46	0.35	2.35	3.35	o.m.
3	7.30	0.56	0.94	2.415	0.83	1.87	5.09	o.m.
4	13.50	-	-	2.39	2.19	2.18	0.98	o.n.
5	24.00	-	-	1.75	1.604	0.71	0.38	o.n.
6	27.20	0.25	0.73	2.18	1.83	0.974	3.97	o.n.
7	28.75	0.45	0.52	1.84	1.23	1.42	4.36	o.m.
8	34.86	0.05	0.68	2.06	1.12	1.96	4.79	o.m.
9	35.59	0.13	1.98	1.99	1.01	2.16	4.33	o.m.
10	35.60	0.38	0.90	1.89	0.90	1.83	4.42	o.m.
11	36.62	0.49	1.22	1.31	0.45	1.66	2.84	f.f.
12	37.15	0.55	1.95	1.34	0.12	2.48	5.64	o.m.
13	44.32	0.78	2.06	1.556	0.18	2.02	4.66	o.m.
14	45.95	1.03	0.88	1.315	0.87	1.9	4.51	o.m.

To calibrate the 3 drop structures in a proper way, it is important to have a steady state situation in the canal. During the 15-11 measurements, the discharge just downstream of the drop structures, h_u and h_d were measured and the flow condition was determined. Drop 1 at RD 18.00 (combined drop and bridge structure, was not noticed during 15-11-1995 measurements) is totally submerged and therefor it was not possible to calibrate this structure properly: submergence ratio S_f (= h_d/h_u) almost 1 . Actually, the flow through this structure is due to heavy siltation just downstream the drop transformed from free flow to conveyance flow.

For the second and third drop at RD 24.05 and at RD 37.25 the flow condition was completely free overflow, determined by a rating curve for a broad-crested weir, with the discharge formula **q = Cd.B.H$^{1.5}$**; where q is the discharge in cfs, B the width of the structure in ft, H is the upstream energy head (equals to h_u) and Cd the discharge coefficient for a broad-crested weir [$ft^{1/2}/s$]. In the following table A.4 the results of the calibration are presented.

Table A.4 Calibration of the drop structures: 15-11-1995

drop	location	h_a (ft)	h_b (ft)	h_u (ft)	h_d (ft)	B (ft)	Q (cfs)	Cd ($ft^{1/2}/s$
1	RD 18.00	-	-	-	-	4.00	-	-
2	RD 24.05	0.67	-	0.64	-	10.75	16.00	2.91
3	~~Rd~~ 37.25	1.56	-	0.925	-	2.06	4.71	2.57

Conclusions

Most of the outlet structures were taking excess water compared to there design discharges, due to higher actual water depth above crest. Inflow seepage occurs in the middle and tail reaches of the distributary. The data set is accurate, and will be used for calibration of the model parameters of the SIC model of Masood distributary.

Field practice: 27-11-1995

Methodology

- During the night the inflow remains constant, in order to establish a steady state condition in the canal during the exercise. The last operation on the gated inlet structure was at 01:00, according the gate reader.
- From 06:00 until 22:00 h_a and h_b were measured at the head and tail (= assumed tail at RD 45.95), every 30 minutes.
- During the day, 5 outlet structures were re-calibrated with a fluming test or a current metering. These 5 outlets were either submerged or disputable during the 15-11-1995 experiment.
- By means of current metering the discharge in the canal was measured at 5 points, i.e. the head of the distributary, the tail and close to the tree drop structures.

Table A.5 Discharge measurements and calibration of outlets: 27-11-1995

Outlet No.	Location (RD)	h_a (ft)	h_b (ft)	h_u (ft)	h_d (ft)	q (cfs)	C_d	Flow condition
1	1.10	-	-	-	-	-	-	closed
2	3.70	-	-	-	-	-	-	closed
3	7.30	0.23	0.40	2.745	1.345	**2.05**	5.09	o.m.
4	13.50	-	-	2.41	2.19	**3.26***	1.39	o.n.
5	24.00	-	-	1.86	1.75	**1.45***	0.90	o.n.
6	27.20	0.16	0.99	2.27	1.57	**1.38**	3.97	o.n.
7	28.75	0.40	0.65	1.89	1.10	**1.51***	4.55	o.m.
8	34.86	-0.03	0.97	2.14	0.83	**2.02**	4.79	o.m.
9	35.59	0.07	1.94	2.05	1.05	**2.21**	4.33	o.m.
10	35.60	0.32	0.95	1.95	0.85	**1.87**	4.42	o.m.
11	36.62	0.44	1.08	1.36	0.59	**1.87***	3.02	f.f.
12	37.15	0.46	-	1.43	-	**2.61**	5.65	o.m.
13	44.32	0.77	2.08	1.57	0.16	**2.03**	4.66	o.m.
14	45.95	0.48	0.89	1.865	0.85	**2.80***	5.16	o.m.

* Discharge measured with a fluming test or a current metering, discharge coefficient re-calibrated.

To re-calibrate the 3 drop structures in a proper way, again it is important to have a steady state situation in the canal. During the 27-11 measurements, the discharge just downstream of the drop structures were measured, the flow condition was determined and h_u and h_d were measured. Again, drop 1 at RD 18.00 was totally submerged and therefor it was not possible to calibrate this structure properly: submergence ratio $S = h_d/h_u = 0.963$. For the second and third drop at RD 24.05 and at RD 37.25 the flow condition was completely free overflow, determined by a rating curve for a broad-crested weir. In the following table A.6. the results of the re-calibration are presented.

Table A.6 Re-calibration of the drop structures: 27-11-1995

drop	location	h_a (ft)	h_b (ft)	h_u (ft)	h_d (ft)	B (ft)	Q (cfs)	Cd ($ft^{1/2}/s$)
1	RD 18.00	1.25	-0.01	2.05	1.975	4.00	21.68	-
2	RD 24.05	0.59	-	0.72	-	10.75	20.00	3.05
3	Rd 37.25	1.48	-	1.005	-	2.06	5.25	2.53

Conclusions

After examine the results of the data collection on 27-11-1995, the following can de stated: (1) Data set is not reliable for calibration, due to unsteady behavior in the canal and a lack of precise discharge measurements at the outlet structures; (2) Data set is reliable for validation in steady flow (unit II of SIC): use measurements between 08:00 and 14:00; (3) The assumed tail at RD 45.95 is not sufficient and reliable to model in SIC: *new tail and downstream boundary condition for the model at the free flow drop structure at RD 37.25*; (4) The SIC model of Masood distributary consists at present of a canal of 11.3 km, 12 outlets (2 submerged pipe outlets and 12 OFRB's), one free flow drop and one submerged drop.

field practice 29-11-1995

Visited different farmers along Masood distributary to explain my investigation and field practices. I asked them if they could open there outlet structure and do not close it before 14:00, 30-11-1995. Result: positive, only outlet no. 1 remained closed, according the farmers.

field practice 30-11-1995

Methodology

- During the night the inflow remains constant, in order to establish a steady state condition in the canal during the exercise.
- Inflow constant at approximately 75% of the discharge at 15-11-1995.
- From 06:00 until 14:00, h_a and h_b were measured at the head, new tail (= assumed tail at RD 37.25), and the two drop structures every 30 minutes.
- Between 08:00 and 14:00 permanent control of the outlets, measurement of h_a and check the typical flow condition.
- Discharge measurements at the head, drop 1 (RD 18.00) and the tail.

Problems

The following problems occurred during this exercise: (1) Discharge to low for a current metering at the tail; (2) Very small wave in the inflow; (3) Outlet no. 8 at RD 34.86 was closed by farmers just after the exercise started.

Hydraulic situation

At the head of Masood distributary the discharge was approximately constant throughout the day from 06:00 to 14:00, at 18.13 cfs (0.51 m³/s). The flow through the head structure of Masood was submerged.

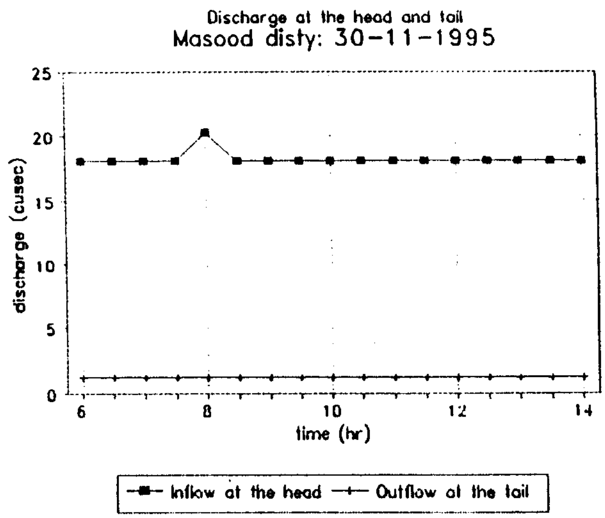


Figure A.4 **Discharge at the head and tail during the 30-11-1995 measurements.**

The discharge at the tail (RD 37.50) was constant during the day, at 1.22 cfs. Both head and tail discharges are presented in figure A.4.

Measurement results

The results of the measurements are presented in the next table. Table A.7 contains the computed discharges q, measured h_a and h_b, computed h_u and h_d, corresponding Cd values (calibrated) and flow condition.

Table A.7 Water level measurements and discharge computations of outlets: 30-11-1995

Outlet No.	Location-(RD)	h_a (ft)	h_b (ft)	h_u (ft)	h_d (ft)	q (cfs)	C_d (mean)	Flow condition
1	1.10	0.21	-	2.79	-	-	-	closed
2	3.70	1.14	-	2.13	-	3.28	2.93	f.f.
3	7.30	0.91	-	2.07	-	1.66	5.09	o.m.
4	13.50	-	-	2.10	2.07	1.08	1.19	o.n.
5	24.00	-	-	1.54	1.48	0.77	0.64	o.n.
6	27.20	0.89	1.18	1.54	1.38	0.67	3.97	o.n.
7	28.75	1.08	0.73	1.21	1.02	0.50	3.97	o.n.
8	34.86	0.5	-	1.61	-	-	-	closed
9	35.59	0.71	-	1.41	-	1.87	2.93	f.f.
10	35.60	0.96	-	1.31	-	1.39	4.42	o.m.
11	36.62	0.98	-	0.82	-	0.85	2.93	f.f.
12	37.15	0.87	-	1.02	-	1.74	2.93	f.f.

Table A.8 presents discharges measured at several places along the canal.

Table A.8. Measured discharges along the canal

Location	RD	Q (m^3/s)	Q (cfs)	Flow condition
Head	0	0.51	18.13	o.n.
drop 1	18.00	0.34	12.02	f.s.
drop 2	24.05	0.30	10.56	f.f.
Tail	37.25	0.035	1.22	f.f.

Conclusions

- Data set reliable for validation of water levels and discharges along the canal.
- Data set useful for validation of the seepage calculation of 15-11-1995 and seepage is related with inflow at the head and surrounding water levels.

ANNEX B CALIBRATION AND VALIDATION RESULTS OF THE MASOOD MODEL

Calibration results

This part of the annex presents the results of the 4 different calibration steps to calibrate the SIC model of Masood distributary based on the field data from 15-11-1995.

step 1
After developing the flow model of the Masood distributary in SIC with the first set of actual field data as described in section 5.4.5 (initial data input for SIC), the simulation can be started. Characteristics: inflow at the head 23.1 cusec (0.65 m^3/s) and outlet 1 was (illegally) closed. After simulation of the situation based on the actual situation the computed discharges and upstream water levels are compared with the actual measured data.

step 2
Use the calibration module of SIC to calculate the Manning's coefficients for the different reaches based on several measured water levels along the canal. Run the model with all outlet structures as an imposed discharge (measured discharge) and define the measured water levels along the canal. Table B.1 and B.2 presenting the results of the Strickler (k) and Manning's coefficient (n) calibration:

Table B.1 Computed values for the Strickler - Manning's coefficient

Reach	Computed k	Input SIC model: k	Input SIC model: n
1	40.6870	40.00	0.025
2	40.6870	40.00	0.025
3	27.6344	27.78	0.036
4	28.6256	27.78	0.036
5	39.2706	40.00	0.025
6	40.0000	40.00	0.025
7	35.1430	34.48	0.029
8	28.7811	28.57	0.035
9	23.3602	23.26	0.043
10	17.5383	17.54	0.057
11	17.5383	17.54	0.057
12	20.4291	20.41	0.049
13	20.4291	20.41	0.049

In table B.2 the computed water levels for both the computations with n initial and n calibrated are listed. Also the computed supplied discharges to the outlet structures with calibrated n values and the initial discharge coefficients are listed[1].

Table B.2 Measured and computed water levels and discharges during the Manning-Strickler calibration procedure

Node no.	measured water level (m)	measured discharge (m^3/s)	computed water level: n initial (m)	computed water level: calibrated n (m)	computed discharge (m^3/s)
head	149.92	-	149.94	149.92	-
1	-	closed	149.88	149.87	closed
2	149.71	0.067	149.64	149.71	0.041
3	148.28	0.053	149.22	149.29	0.044
4	148.86	0.062	148.89	148.86	0.055
5	148.31	0.020	148.31	148.31	0.049
drop 2	148.31	-	148.31	148.31	-
6	148.06	0.028	148.10	148.06	0.058
7	147.96	0.040	148.03	147.96	0.055
8	147.47	0.056	147.48	147.47	0.045
9	147.34	0.061	147.29	147.34	0.048
10	147.33	0.052	147.28	147.34	0.044
11	147.04	0.047	147.03	147.05	0.034
12	146.86	0.070	146.89	146.90	0.061
tail	146.87	-	146.88	146.88	-

step 3

Run the model again with the calibrated Manning's coefficients and compare the computed water supplies to the outlet structures with the measured supplies in the field. Adjust the discharge coefficients of the outlet structures in such a way that the computed discharges match the measured discharges. This procedure is listed in table B.3.

1

Initial n = initially defined Manning's coefficient (n = 0.028 for reach 1 to 7 and n = 0.045 for reach 8 to 13).
Initial Cd = initial discharge coefficient based on the calibrated outlet structures with the measurement set of 15-11-1995.

Table B.3 Calibration of C_d values outlet structures of the SIC model of Masood distributary

No.	C_d start	q(1)	C_d(2)	q(2)	C_d(3)	q(3)	C_d(3)	q(3)	C_d end	q end
1	-	-	-	-	-	-	-	-	(0.55)	-
2	0.42	0.062	0.44	0.065	0.45	0.067	-	0.067	**0.45**	**0.067**
3	0.63	0.052	-	0.052	-	0.052	-	0.052	**0.63**	**0.052**
4	0.60	0.050	0.70	0.056	0.80	0.062	-	0.062	**0.80**	**0.062**
5	0.38	0.028	0.30	0.022	-	0.021	-	0.021	**0.30**	**0.021**
6	0.49	0.044	0.40	0.036	0.30	0.027	-	0.027	**0.30**	**0.027**
7	0.54	0.038	0.58	0.041	-	0.044	-	0.041	**0.58**	**0.041**
8	0.60	0.057	-	0.057	-	0.057	-	0.057	**0.60**	**0.057**
9	0.54	0.061	-	0.061	-	0.062	-	0.062	**0.54**	**0.062**
10	0.55	0.052	0.56	0.053	-	0.053	-	0.053	**0.56**	**0.053**
11	0.35	0.025	0.50	0.036	0.65	0.047	-	0.047	**0.65**	**0.047**
12	0.70	0.077	0.65	0.071	-	0.069	0.68	0.072	**0.68**	**0.072**

2. Validation results

This part of the annex presents the results of the steps to validate the calibrated SIC model of Masood distributary, based on the field data from 27-11-1995 and 30-11-1995.

step 1

The model is running a simulation based on the input data of 27-11-1995, i.e. a constant inflow discharge of 0.80 m^3/s (28 csf) at the head of Masood distributary. All other input data and calibrated model data is kept constant (validation 1). It can be stated that the computed discharges supplied to the outlet structures are approximately 0.06 to 0.011 m^3/s to high, compared with the measured and computed discharges. Especially the computed discharges of the submerged pipe outlet (no. 5) and submerged OFRB (no. 6) are nor very precise. The computed discharges slightly become more accurate, if the downstream boundary condition op the submerged outlet structures is set on the real measured downstream water levels in the watercourses (validation 2). Still there are differences between the computed and measured discharges. The differences are due to higher computed water levels along the canal. Proposed adjustment: use the computed seepage values of 27-1-1995 (now there is seepage in stead of gain). Both the model downstream water level as the measured downstream water level of the submerged outlet structures will be simulated (validation 3 and 4). In table B.4, the results of the 4 different validation are listed.

Table B.4 Validation results : 27-11-1995

Outlet No.	Validation 1	Validation 2	Validation 3	Validation 4	Targeted discharge
1	-	-	-	-	-
2	-	-	-	-	-
3	0.058	0.058	0.058	0.058	0.058
4	0.103	0.103	0.098	0.098	0.092
5	0.029	0.045	0.028	0.045	0.041
6	0.037	0.037	0.034	0.036	0.039
7	0.045	0.045	0.043	0.043	0.043
8	0.063	0.063	0.058	0.058	0.057
9	0.069	0.069	0.063	0.062	0.063
10	0.059	0.059	0.055	0.054	0.053
11	0.059	0.058	0.050	0.049	0.053
12	0.087	0.086	0.075	0.073	0.074

validation 1: no changes in the model
validation 2: measured d/s water level as a d/s boundary condition for the submerged outlet structures 4, 5 and 6.
validation 3: seepage as computed for 27-11-1995
validation 4: both seepage as measured d/s water level for a d/s boundary condition for the submerged outlet structures 4, 5 and 6.

step 2

The model is running a simulation based on the input data of 30-11-1995, i.e. a constant inflow discharge of 0.51 m^3/s (18.13 csf) at the head of Masood distributary. The validation is to check different measured discharges along the canal, and measured water levels upstream of the outlet structures, with the computed output of the model. Based on the conclusions of the 27-11-1995 validation, the seepage will be the same as 27-11-1995 (because of low water levels of Fordwah Branch) and measured downstream water levels for submerged outlet structures (no. 4,5,6 and 7) will be used in the model.

Table B.5 presents the results of the comparison of measured water levels (measured above the crest) and computed water levels. Table B.6 presents the validation results of discharges along the canal.

Table B.5 Validation results: 30-11-1995, water levels above the crest

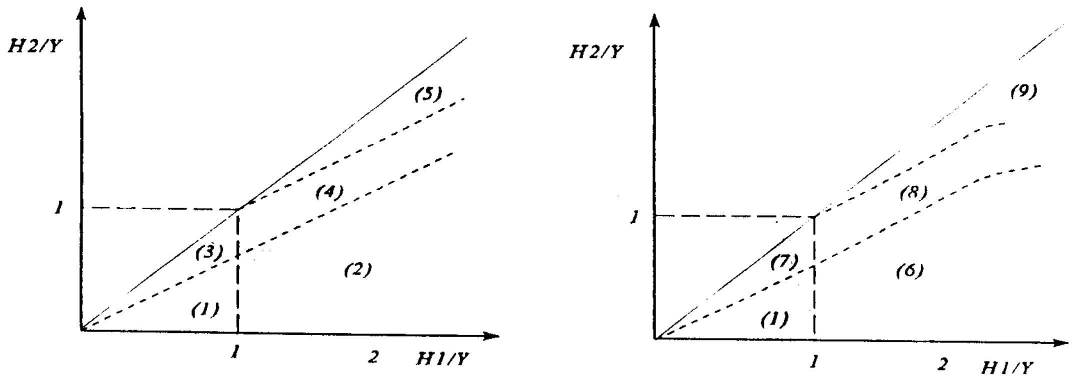
Outlet No.	Computed H_u	H measured
1	149.80	149.80
2	149.63	149.64
3	149.18	149.20
4	148.77	148.76
5	148.25	148.26
6	147.86	147.93
7	147.77	147.82
8	147.33	147.34
9	147.16	147.19
10	147.16	147.18
11	146.89	146.92
12	146.76	146.73

Table B.6 Validation results: 30-11-1995, measured discharges along the canal

Location	Q measured	Q computed
Head at RD 0	0.513	0.513
drop 1 at RD 18000	0.340 (o.n.)	0.342 (o.n.)
drop 2 at RD 24050	0.299 (f.f.)	0.296 (f.f.)
tail at RD 37250	0.035 (f.f.)	0.039 (f.f.)

Both computed upstream water levels near the outlet structures as the computed discharges along the canal match with the measured values. The water levels are computed correct. It can be concluded that the validation of the SIC model of Masood distributary has been successful.

ANNEX C STRUCTURE EQUATIONS USED IN SIC



High sill elevation ***Small sill elevation***

1. *Open, free flow*
2. *Orifice, free flow*
3. *Open, submerged flow*
4. *Orifice, partially submerged flow*
5. *Orifice, completely submerged flow*
6. *Orifice, free flow*
7. *Open, submerged flow*
8. *Orifice, partially submerged flow*
9. *Orifice, completely submerged flow*

Structure equations:

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

and: $0<h_1<Y$, $h_2<2/3.h_1$

2. $Q = \mu . L . \sqrt{2g} . (h_1^{3/2} - (h_1 - Y)^{3/2})$

and: $h_1>Y$, $h_2<2/3.h_1$

Where: $\mu = \mu_F$

3. $Q = \mu_s.L.\sqrt{2g}.(h_1 - h_2)^{1/2}.h_2$

and: $0<h_1<Y$, $2/3.h_1<h_2<h_1$

Where: $\mu_s = \frac{3\sqrt{3}}{2}.\mu_F$

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

and: $h_1>Y$, $2/3.h_1<h_2<2/3.h_1 + Y/3$

5. $Q = \mu'.L.\sqrt{2g}.(h_1 - h_2)^{1/2}.Y$

and: $h_1>Y$, $h_2<2/3.h_1 + 1/3.Y$

Where: $\mu' = \mu_s = \frac{3\sqrt{3}}{2}.\mu_F$

6. $Q = L.\sqrt{2g}.(\mu.h_1^{3/2} - \mu_1.(h_1 - Y)^{3/2})$

and: $h_1>Y$, $h_2<\frac{h_1}{1 + 0.14.\frac{h_1}{Y}}$

Where: $\mu = \mu_F + 0.08.(1 - \frac{1}{\frac{h_1}{Y}})$

Where: $\mu_1 = \mu_F + 0.08.(1 - \frac{1}{\frac{h_1}{Y} - 1})$

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

and: $h_1 < Y \ , \ h_1 < \dfrac{h_1}{1 + 0.14.\dfrac{h_1}{Y}} < h_2 < h_1$

Where: $x = \sqrt{1 - \dfrac{h_2}{h_1}}$

x > 0.2: $k_F = 1 - (1 - \dfrac{x}{\sqrt{1-\alpha}})^{\beta}$

x < 0.2: $k_F = 5x.(1 - (1 - \dfrac{0.2}{\sqrt{1-\alpha}})^{\beta})$

Where: $\beta = -2\alpha + 2.6$

8. $Q = L.\sqrt{2g}.(\mu.h_1^{3/2} - \mu_1.(h_1 - Y)^{3/2})$

and: $h_1 > Y \ , \ \dfrac{h_1}{1 + 0.14.\dfrac{h_1}{Y}} < h_2 < \dfrac{h_1 + Y.(0.14.\dfrac{h_1}{Y} + 0.14)}{1.14 + 0.14.\dfrac{h_1}{Y}}$

With k_F, mu and mu_1 as defined above.

9. $Q = L.\sqrt{2g}.(k_F.\mu.h_1^{3/2} - k_{F1}.\mu_1.(h_1 - Y)^{3/2})$

and: $h_1 > Y \ , \ \dfrac{h_1 + Y.(0.14.\dfrac{h_1}{Y} + 0.14)}{1.14 + 0.14.\dfrac{h_1}{Y}} < h_2 < h_1$

With k_F, mu and mu_1 as defined above.

For k_{F1}: $$x = \sqrt{1 - \frac{h_2 - Y}{h_1 - Y}}$$

x > 0.2: $$k_F = 1 - \left(1 - \frac{x}{\sqrt{1-\alpha}}\right)^{\beta}$$

x < 0.2: $$k_F = 5x.\left(1 - \left(1 - \frac{0.2}{\sqrt{1-\alpha}}\right)^{\beta}\right)$$

Where: $$\beta = -2\alpha + 2.6$$

ANNEX D INPUT STRUCTURE DATA ACTUAL SITUATION

Cross structures

No.	Name	Location (ft)	Location (m)	Width B (ft)	Width B (m)	Crest Level (m)
1	Drop 1	18,000	5,486	4	1.219	148.270
2	Drop 2	24,050	7,330	10.75	3.277	148.113
3	Tail	37,250	11,354	2.06	0.628	146.594

Nodes (outlet structures)

No.	Name	Location (ft)	Location (m)	Width B (m)	Opening Y (m)	Crest Level (m)
1	1100-R	1,100	335.8	0.067	0.393	148.947
2	3700-R	3,700	1127.8	0.110	0.488	148.980
3	7300-R	7,300	2225.0	0.070	0.378	148.547
4	13500-R	13,500	4114.8	-	0.271	148.133
5	24000-R	24,000	7315.2	-	0.268	147.781
6	27200-R	27,200	8290.6	0.116	0.332	147.392
7	28750-R	28,750	8763	0.104	0.253	147.397
8	34860-R	34,860	10625.3	0.098	0.329	146.838
9	35590-R	35,590	10847.8	0.116	0.354	146.733
10	35600-R	35,600	10850.9	0.128	0.256	146.756
11	36620-R	36,620	11161.8	0.119	0.296	146.644
12	37150-R	37,150	11323.3	0.177	0.253	146.453

ANNEX E INPUT CROSS SECTIONS OF MASOOD DISTRIBUTARY ACTUAL SITUATION

Survey done by: Anwar Iqbal and Steven Visser.
Date: December, 1995.
Method: Cross sections measured from the left bank of Masood distributary, with x-step of 0.50 metres, referred to a Bench Mark (figures in metres and reference elevation: initial point crest level of the gated inlet structure of Masood distributary), see figure F.1.

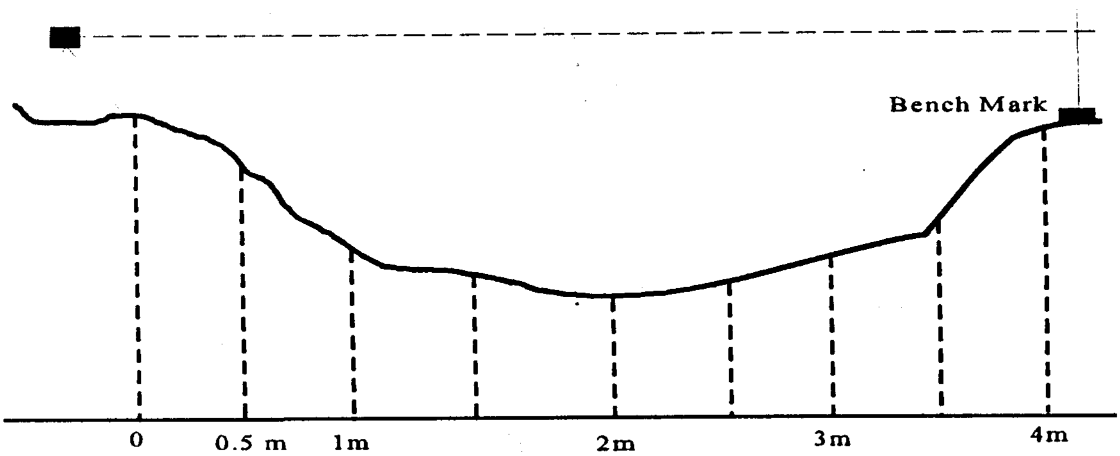


Figure F.1 **Measurements of cross sections of Masood distributary**

REACH : HEAD - 1100R

Masood Head

0.00	0.50	1.00	1.50	2.00	2.50	3.00	3.50
150.14	149.50	149.45	149.48	149.49	149.49	149.49	149.46
4.00	4.50	5.00	5.50	6.00			
149.44	149.37	149.36	149.80	150.14			

o/l 1 (u/s)

0.00	0.50	1.00	1.50	2.00	2.50	3.00	3.50
150.01	149.81	149.34	149.20	149.16	149.22	149.30	149.38
4.00	4.50	5.00	5.50	6.00			
149.46	149.52	149.58	149.73	149.97			

REACH : 1100R - 3700R

o/l 1 (d/s)

0.00	0.50	1.00	1.50	2.00	2.50	3.00	3.50
150.01	149.81	149.34	149.20	149.16	149.22	149.30	149.38
4.00	4.50	5.00	5.50	6.00			
149.46	149.52	149.58	149.73	149.97			

o/l 2 (u/s)

0.00	0.50	1.00	1.50	2.00	2.50	3.00	3.50
149.81	149.64	149.23	149.24	149.27	149.27	149.28	149.27
4.00	4.50	5.00	5.50	6.00	6.50		
149.27	149.27	149.46	149.66	149.62	149.81		

REACH : 3700R - 7300R

o/l 2 (d/s)

0.00	0.50	1.00	1.50	2.00	2.50	3.00	3.50
149.81	149.64	149.23	149.24	149.27	149.27	149.28	149.27
4.00	4.50	5.00	5.50	6.00	6.50		
149.27	149.27	149.46	149.66	149.62	149.81		

o/l 3 (u/s)

0.00	0.50	1.00	1.50	2.00	2.50	3.00	3.50
149.50	149.35	149.10	149.00	148.74	148.59	148.56	148.60
4.00	4.50	5.00	5.50	6.00			
148.81	148.90	149.06	149.06	149.50			

REACH : 7300R - 13500R

o/l 3 (d/s)

0.00	0.50	1.00	1.50	2.00	2.50	3.00	3.50
149.50	149.35	149.10	149.00	148.74	148.59	148.56	148.60
4.00	4.50	5.00	5.50	6.00			
148.81	148.90	149.06	149.06	149.50			

o/l 4 (u/s)

0.00	0.50	1.00	1.50	2.00	2.50	3.00	3.50
149.01	148.92	148.45	148.15	148.14	148.08	148.11	148.12
4.00	4.50	5.00	5.50				
148.25	148.84	148.95	149.28				

REACH : 13500R - 24000R

o/l 4 (d/s)

0.00	0.50	1.00	1.50	2.00	2.50	3.00	3.50
149.01	148.92	148.45	148.15	148.14	148.08	148.11	148.12
4.00	4.50	5.00	5.50				
148.25	148.84	148.95	149.28				

cross 1

0.00	0.50	1.00	1.50	2.00	2.50	3.00	3.50
148.96	148.79	148.73	148.71	148.12	147.95	147.90	147.99
4.00	4.50	5.00	5.50				
148.50	148.78	148.99	149.13				

drop 1 (u/s)

0.00	0.50	1.00	1.50	2.00	2.50	3.00	3.50
149.41	149.14	148.82	148.55	148.30	148.25	148.12	148.26
4.00	4.50	5.00	5.50	6.00			
148.26	148.38	148.61	148.93	149.14			

drop 1 (d/s)

0.00	0.50	1.00	1.50	2.00	2.50	3.00	3.50
148.97	148.86	148.79	148.72	148.56	148.32	148.13	148.03
4.00	4.50	5.00	5.50	6.00	6.50	7.00	7.50
148.02	148.08	148.16	148.22	148.32	148.38	148.54	148.69
8.00	8.50						
148.83	149.03						

cross 2

0.00	0.50	1.00	1.50	2.00	2.50	3.00	3.50
148.97	148.87	148.53	148.23	147.98	147.89	147.92	148.06
4.00	4.50	5.00	5.50				
148.25	148.63	149.04	149.10				

o/l 5 (u/s)

0.00	0.50	1.00	1.50	2.00	2.50	3.00	3.50
148.57	148.48	148.14	147.84	147.75	147.76	147.80	147.83
4.00	4.50	5.00	5.50				
148.02	148.34	148.64	148.70				

REACH : 24000R - 27200R

o/l 5 (d/s)

0.00	0.50	1.00	1.50	2.00	2.50	3.00	3.50
148.57	148.48	148.14	147.84	147.75	147.76	147.80	147.83
4.00	4.50	5.00	5.50				
148.02	148.34	148.64	148.70				

drop 2 (u/s)

0.00	0.50	1.00	1.50	2.00	2.50	3.00	3.50
148.57	148.48	148.14	147.84	147.75	147.76	147.80	147.831
4.00	4.50	5.00	5.50				
148.02	148.34	148.64	148.70				

drop 2 (d/s)

0.00	0.50	1.00	1.50	2.00	2.50	3.00	3.50
148.28	147.92	147.62	147.58	147.50	147.38	147.28	147.33
4.00	4.50	5.00	5.50				
147.52	147.92	148.44	148.72				

o/l 6 (u/s)

0.00	0.50	1.00	1.50	2.00	2.50	3.00	3.50
148.30	148.21	148.37	147.86	147.82	147.60	147.47	147.41
4.00	4.50	5.00	5.50	6.00			
147.49	147.59	147.76	148.10	148.31			

REACH : 27200R - 28750R

o/l 6 (d/s)

0.00	0.50	1.00	1.50	2.00	2.50	3.00	3.50
148.30	148.21	148.37	147.86	147.82	147.60	147.47	147.41
4.00	4.50	5.00	5.50	6.00			
147.49	147.59	147.76	148.10	148.31			

o/l 7 (u/s)

0.00	0.50	1.00	1.50	2.00	2.50	3.00	3.50
148.35	148.22	148.00	147.86	147.65	147.44	147.49	147.43
4.00	4.50	5.00	5.50	6.00	6.50		
147.39	147.45	147.55	147.96	148.11	148.35		

REACH : 28750R - 34860R

o/l 7 (d/s)

0.00	0.50	1.00	1.50	2.00	2.50	3.00	3.50
148.35	148.22	148.00	147.86	147.65	147.44	147.49	147.43
4.00	4.50	5.00	5.50	6.00	6.50		
147.39	147.45	147.55	147.96	148.11	148.35		

o/l 8 (u/s)

0.00	0.50	1.00	1.50	2.00	2.50	3.00	3.50
147.63	147.31	147.18	146.87	146.87	146.87	146.90	147.16
4.00	4.50	5.00					
147.48	147.55	147.73					

REACH : 34860R - 35590R

o/l 8 (d/s)

0.00	0.50	1.00	1.50	2.00	2.50	3.00	3.50
147.63	147.31	147.18	146.87	146.87	146.87	146.90	147.16
4.00	4.50	5.00					
147.48	147.55	147.73					

o/l 9 (u/s)

0.00	0.50	1.00	1.50	2.00	2.50	3.00	3.50
147.52	147.47	147.20	146.84	146.85	146.86	146.85	147.23
4.00	4.50	5.00					
147.46	147.65	147.80					

REACH : 35590R - 35600R

o/l 9 (d/s)

0.00	0.50	1.00	1.50	2.00	2.50	3.00	3.50
147.52	147.47	147.20	146.84	146.85	146.86	146.85	147.23
4.00	4.50	5.00					
147.46	147.65	147.80					

o/l 10 (u/s)

0.00	0.50	1.00	1.50	2.00	2.50	3.00	3.50
147.52	147.47	147.20	146.84	146.85	146.86	146.85	147.23
4.00	4.50	5.00					
147.46	147.65	147.80					

REACH : 35600R - 36620R

o/l 10 (d/s)

0.00	0.50	1.00	1.50	2.00	2.50	3.00	3.50
147.52	147.47	147.20	146.84	146.85	146.86	146.85	147.23
4.00	4.50	5.00					
147.46	147.65	147.80					

o/l 11 (u/s)

0.00	0.50	1.00	1.50	2.00	2.50	3.00	3.50
147.38	147.31	146.89	146.69	146.58	146.71	146.75	146.79
4.00	4.50	5.00	5.50				
146.87	147.01	147.27	147.49				

REACH : 36620R - 37150R

o/l 11 (d/s)

0.00	0.50	1.00	1.50	2.00	2.50	3.00	3.50
147.38	147.31	146.89	146.69	146.58	146.71	146.71	146.71
4.00	4.50	5.00	5.50				
146.70	147.71	147.27	147.49				

o/l 12 (u/s)

0.00	0.50	1.00	1.50	2.00	2.50	3.00	3.50
147.82	147.60	147.14	146.83	146.63	146.56	146.44	146.57
4.00	4.50						
147.36	147.56						

REACH : 37150R - TAIL

o/l 12 (d/s)

0.00	0.50	1.00	1.50	2.00	2.50	3.00	3.50
147.82	147.60	147.14	146.83	146.63	146.56	146.44	146.57
4.00	4.50						
147.36	147.56						

tail

0.00	0.50	1.00	1.50	2.00	2.50	3.00	3.50
147.82	147.60	147.14	146.83	146.63	146.51	147.01	147.57
4.00	4.50						
147.76	147.56						

ANNEX F RESULTS SENSITIVITY ANALYSIS

- all discharges in m^3/s
- outlet structure 2, 8 and 12: no effect

Width B Drop Structure

outlet structure 5

	-25 %		0 %	+25 %	
Q	q	R	q	q	R
1.0	0.026	0.33	0.024	0.022	-0.33
0.9	0.025	0.55	0.022	0.021	-0.18
0.8	0.023	0.38	0.021	0.020	-0.19
0.7	0.022	0.40	0.020	0.018	-0.40
0.6	0.020	0.21	0.019	0.017	-0.42
0.5	0.019	0.47	0.017	0.016	-0.24
1.1	0.027	0.32	0.025	0.024	-0.16
1.2	0.029	0.30	0.027	0.025	-0.15
R_{mean}		**0.37**			**-0.26**

Crest level Drop Structure

outlet structure 5

	-40 %		-25%		0 %	+25 %		+40%	
Q	q	R	q	R	q	q	R	q	R
1.0	0.021	-0.31	0.021	-0.50	0.024	0.028	0.67	0.030	0.62
0.9	0.019	-0.34	0.019	-0.55	0.022	0.027	0.91	0.029	0.80
0.8	0.016	-0.60	0.017	-0.76	0.021	0.026	0.95	0.028	0.83
0.7	0.014	-0.75	0.015	-1.00	0.020	0.024	0.80	0.027	0.87
0.6	0.012	-0.92	0.014	-1.05	0.019	0.023	0.84	0.026	0.92
0.5	0.010	-1.03	0.013	-0.94	0.017	0.022	1.18	0.024	1.03
1.1	0.023	-0.20	0.024	-0.16	0.025	0.029	0.64	0.031	0.60
1.2	0.026	-0.09	0.026	-0.15	0.027	0.030	0.44	0.032	0.46
R_{mean}		**-0.53**		**-0.64**			**0.80**		**0.77**

Manning's coefficient

outlet structure 2

	-20 %		-10%		0 %	+10 %		+20%	
Q	q	R	q	R	q	q	R	q	R
1.0	0.071	-0.33	0.074	-0.26	0.076	0.079	0.39	0.081	0.33
0.9	0.069	-0.27	0.071	-0.27	0.073	0.076	0.41	0.078	0.34
0.8	0.066	-0.29	0.068	-0.29	0.07	0.073	0.43	0.074	0.29
0.7	0.063	-0.30	0.064	-0.45	0.067	0.069	0.30	0.071	0.30
0.6	0.059	-0.32	0.061	-0.32	0.063	0.065	0.32	0.067	0.32
0.5	0.054	-0.42	0.056	-0.51	0.059	0.061	0.34	0.062	0.25
1.1	0.074	-0.32	0.076	-0.38	0.079	0.082	0.38	0.084	0.32
1.2	0.076	-0.37	0.079	-0.37	0.082	0.085	0.37	0.086	0.24
R mean		**-0.33**		**-0.36**			**0.37**		**0.30**

outlet structure 5

	-20 %		-10%		0 %	+10 %		+20%	
Q	q	R	q	R	q	q	R	q	R
1.0	0.024	-0.22	0.023	-0.00	0.023	0.024	0.43	0.025	0.43
0.9	0.022	-0.00	0.022	-0.00	0.022	0.022	0.00	0.023	0.23
0.8	0.021	-0.00	0.021	-0.00	0.021	0.021	0.00	0.021	0.00
0.7	0.02	-0.00	0.02	-0.00	0.02	0.02	0.00	0.02	0.00
0.6	0.019	-0.00	0.019	-0.00	0.019	0.018	-0.53	0.018	-0.26
0.5	0.017	-0.00	0.017	-0.00	0.017	0.017	0.00	0.017	0.00
1.1	0.025	-0.00	0.025	-0.00	0.025	0.026	0.40	0.027	0.40
1.2	0.026	-0.19	0.026	-0.37	0.027	0.028	0.37	0.029	0.37
R mean		**-0.00**		**-0.05**			**0.08**		**0.15**

outlet structure 8

	-20 %		-10%		0 %	+10 %		+20%	
Q	q	R	q	R	q	q	R	q	R
1.0	0.063	-0.23	0.065	-0.15	0.066	0.068	0.30	0.069	0.23
0.9	0.061	-0.16	0.062	-0.16	0.063	0.065	0.32	0.066	0.24
0.8	0.058	-0.17	0.059	-0.17	0.06	0.061	0.17	0.063	0.25
0.7	0.054	-0.26	0.056	-0.18	0.057	0.058	0.18	0.059	0.18
0.6	0.051	-0.19	0.052	-0.19	0.053	0.054	0.19	0.055	0.19
0.5	0.047	-0.20	0.048	-0.20	0.049	0.049	0.00	0.05	0.10
1.1	0.066	-0.22	0.068	-0.14	0.069	0.07	0.14	0.072	0.22
1.2	0.068	-0.28	0.07	-0.28	0.072	0.073	0.14	0.074	0.14
R mean		**-0.21**		**-0.18**			**0.18**		**0.19**

outlet structure 12

	-20 %		-10%		0 %	+10 %		+20%	
Q	q	R	q	R	q	q	R	q	R
1.0	0.097	0.11	0.096	0.11	0.095	0.093	-0.21	0.092	-0.16
0.9	0.095	0.40	0.089	0.11	0.088	0.086	-0.23	0.085	-0.17
0.8	0.084	0.19	0.082	0.12	0.081	0.079	-0.25	0.078	-0.19
0.7	0.077	0.35	0.074	0.28	0.072	0.07	-0.28	0.069	-0.21
0.6	0.068	0.40	0.065	0.32	0.063	0.06	-0.48	0.058	-0.40
0.5	0.059	0.57	0.056	0.57	0.053	0.049	-0.75	0.046	-0.66
1.1	0.103	0.10	0.102	0.10	0.101	0.1	-0.10	0.099	-0.10
1.2	0.109	0.14	0.108	0.19	0.106	0.105	-0.09	0.105	-0.05
R mean		**0 .28**		**0.22**			**-0.30**		**-0.24**

Seepage losses

- outlet structure 2 and 5: no effect

outlet structure 12: INFLOW SEEPAGE

	-100 %		-40%		0 %	+40 %		+100%	
Q	q	R	q	R	q	q	R	q	R
1.0	0.092	-0.04	0.094	-0.05	0.096	0.097	0.03	0.099	0.03
0.9	0.085	-0.06	0.087	-0.08	0.090	0.091	0.03	0.093	0.03
0.8	0.077	-0.06	0.080	-0.06	0.082	0.084	0.06	0.087	0.06
0.7	0.068	-0.08	0.072	-0.07	0.074	0.076	0.07	0.079	0.07
0.6	0.057	-0.12	0.062	-0.12	0.065	0.068	0.12	0.071	0.09
0.5	0.047	-0.15	0.052	-0.14	0.055	0.058	0.14	0.062	0.13
1.1	0.098	-0.04	0.100	-0.05	0.102	0.103	0.02	0.105	0.03
1.2	0.104	-0.03	0.106	-0.02	0.107	0.109	0.05	0.111	0.04
R mean		**-0.07**		**-0.07**			**0.06**		**0.06**

outlet structure 12: OUTFLOW SEEPAGE

	-100 %		-40%		0 %	+40 %		+100%	
Q	q	R	q	R	q	q	R	q	R
1.0	0.092	0.06	0.089	0.06	0.087	0.085	-0.06	0.083	-0.05
0.9	0.085	0.06	0.082	0.06	0.080	0.077	-0.09	0.074	-0.08
0.8	0.077	0.08	0.073	0.07	0.071	0.069	-0.07	0.064	-0.10
0.7	0.068	0.11	0.064	0.12	0.061	0.058	-0.12	0.053	-0.13
0.6	0.057	0.14	0.053	0.15	0.050	0.046	-0.20	0.039	-0.22
0.5	0.047	0.31	0.041	0.35	0.036	0.032	-0.28	0.028	-0.22
1.1	0.098	0.04	0.096	0.05	0.094	0.093	-0.03	0.090	-0.04
1.2	0.104	0.03	0.102	0.02	0.101	0.099	-0.05	0.097	-0.04
R mean		**0.10**		**0.11**			**-0.11**		**-0.11**

Cross sectional area: A.R$^{2/3}$

outlet structure 2

	-20 %		-10%		0 %	+20 %		+40%	
Q	q	R	q	R	q	q	R	q	R
1.0	0.080	0.26	0.078	0.26	0.076	0.074	-0.13	0.073	-0.10
0.9	0.077	0.27	0.075	0.27	0.073	0.071	-0.14	0.070	-0.10
0.8	0.074	0.29	0.072	0.29	0.070	0.068	-0.14	0.067	-0.11
0.7	0.070	0.22	0.069	0.30	0.067	0.065	-0.15	0.064	-0.11
0.6	0.066	0.24	0.065	0.32	0.063	0.061	-0.16	0.060	-0.12
1.1	0.083	0.25	0.081	0.25	0.079	0.077	-0.13	0.075	-0.13
1.2	0.086	0.24	0.084	0.24	0.082	0.079	-0.18	0.078	-0.12
R mean		**0.25**		**0.28**			**-0.15**		**-0.11**

outlet structure 5

	-20 %		-10%		0 %	+20 %		+40%	
Q	q	R	q	R	q	q	R	q	R
1.0	0.025	0.43	0.024	0.43	0.023	0.023	0.00	0.024	0.11
0.9	0.022	0.00	0.022	0.00	0.022	0.022	0.00	0.022	0.00
0.8	0.021	0.00	0.021	0.00	0.021	0.021	0.00	0.021	0.00
0.7	0.020	0.00	0.020	0.00	0.020	0.020	0.00	0.020	0.00
0.6	0.019	0.00	0.019	0.00	0.019	0.019	0.00	0.019	0.00
1.1	0.027	0.40	0.026	0.40	0.025	0.025	0.00	0.025	0.00
1.2	0.029	0.37	0.028	0.37	0.027	0.026	-0.19	0.026	-0.09
R mean		**0.17**		**0.17**			**-0.03**		**-0.00**

outlet structure 8

	-20 %		-10%		0 %	+20 %		+40%	
Q	q	R	q	R	q	q	R	q	R
1.0	0.071	0.38	0.068	0.30	0.066	0.064	-0.15	0.061	-0.19
0.9	0.068	0.40	0.066	0.48	0.063	0.061	-0.16	0.058	-0.20
0.8	0.065	0.42	0.062	0.33	0.060	0.058	-0.17	0.055	-0.21
0.7	0.061	0.35	0.058	0.18	0.057	0.055	-0.18	0.052	-0.22
0.6	0.057	0.38	0.054	0.19	0.053	0.051	-0.19	0.048	-0.24
1.1	0.073	0.29	0.071	0.29	0.069	0.066	-0.22	0.064	-0.18
1.2	0.076	0.28	0.074	0.28	0.072	0.069	-0.21	0.066	-0.21
R mean		**0.36**		**0.29**			**-0.18**		**-0.21**

outlet structure 12

	-20 %		-10%		0 %	+20 %		+40%	
Q	q	R	q	R	q	q	R	q	R
1.0	0.093	-0.11	0.094	-0.11	0.095	0.096	0.05	0.098	0.08
0.9	0.086	-0.11	0.087	-0.11	0.088	0.090	0.11	0.092	0.11
0.8	0.078	-0.19	0.079	-0.25	0.081	0.083	0.12	0.085	0.12
0.7	0.069	-0.21	0.071	-0.14	0.072	0.075	0.21	0.077	0.17
0.6	0.057	-0.48	0.060	-0.48	0.063	0.067	0.32	0.070	0.28
1.1	0.099	-0.10	0.100	-0.10	0.101	0.102	0.05	0.104	0.07
1.2	0.105	-0.05	0.106	-0.00	0.106	0.108	0.09	0.109	0.07
R mean		**-0.18**		**-0.17**			**0.14**		**0.13**

ANNEX G PROPOSED APPROACH SIMPLIFIED SIC MODEL FOR 3-L DISTRIBUTARY

General approach

The next procedure to develop a simplified hydrodynamical flow model (SIC) for distributaries, is suggested:

1. **Inventory of the topographical layout of the distributary**
   For each distributary the different nodes must be defined, therefor information is necessary about: (1) total length of the distributary; (2) location of al the outlet structures, inlet structure and tail structure (nodes abscissa); (3) location of the cross structures, and (4) location of off taking minors and sub-minors. The topographical module can be developed in SIC !

2. **Simplified geometrical module**
   The geometrical files are based on the following simplified approach: *cross sectional profiles* and the *crest levels of the cross structures* are based on the design crest levels of the outlet structures. Minimize the amount of cross sectional measurements by taking cross sectional measurements close to outlet structures and head structure only.

3. **Cross devices description**
   In general, the cross structures are normal drop structure, without gated openings. Input parameters for drop structures: measured width B (m), crest level elevation as defined in point 2., and discharge coefficient based on mean flow condition. Whenever the drop structure is working under free flow conditions, the theoretical discharge coefficient is sufficient: Cd = 0.37 (= initial values mu, default values, used by SIC). Whenever the drop structure is working under submerged conditions, the discharge coefficient has to be calibrated based on field measurements.

4. **Nodes description**
   In general, the *outlet structures* are either (OC)AOSM, (OC)OFRB, OF or PIPE outlet structures. Input parameters for outlet structures: measured width B (m), measured opening height Y (m), *design* crest level elevation (PID), and discharge coefficient based on mean flow condition. Whenever the outlet structure is working under free flow conditions, the theoretical discharge coefficient is sufficient, and the downstream boundary condition does not play any role. Whenever the outlet structure is working under submerged conditions, the discharge coefficient has to be calibrated based on field measurements. The downstream boundary condition must be modelled by means of a theoretical rating curve. The *upstream boundary condition* (head node of the distributary) exists either of a constant inflow (m^3/s), or a typical inflow pattern Q(t) defined in Unit III of SIC. The *downstream boundary condition* of the model must be a rating curve. If the tail condition is a drop structure or outlet structure, the discharge coefficient must be determined based on measurements for both free flow and submerged conditions.

5. Manning's coefficient

The initial input of the roughness coefficient, expressed as the Manning's coefficient n, will be based on a visual analysis of the physical state of the distributary. Using the descriptive state of a distributary, based on the classification defined by Ven Te Chow (1973), n values for certain reaches in the canal can be obtained. It will be suggested to define n values between: *head - drop structure 1; drop structure 1 - drop structure 2; drop structure 2 - drop structure x; drop structure x - tail.*

6. Seepage

The rate of seepage losses within a distributary will be based on the IIMI measurements which took place for all distributaries. For small distributaries (< 15 km): the mean value for S_e for the whole canal will be add in SIC. For large distributaries (> 15 km): the mean value for S_e for reaches up to 10 to 15 km will be add in SIC. Attention must be paid on the difference of inflow and outflow seepage. For *outflow seepage a negative value* must be add. For *inflow seepage a positive value* must be add.

Data input

The necessary data input to develop a simplified SIC model for 3-L distributary is presented in the following table. The data are based on the measurements on 3-L dated 08/10/1995, conducted by IIMI (Bahawalnagar).

Table F.1 Data input simplified SIC model 3-L distributary

1. TOPOGRAPHY		
Nodes	**Name**	**Abscissa [m]**
Head	Head 3-L	0
1	outlet 1	3
2	outlet 2	1036
3	outlet 3	1524
4	outlet 4	2134
5	outlet 5	3353
6	outlet 6	4974
Tail	Tail 3-L	7041
Cross structures	Name	Abscissa [m]
-	-	-
2. GEOMETRY		
Longitudinal profile	(field)	
Cross sections	(field)	
3. CROSS STRUCTURES		
-	-	-

4. NODES DESCRIPTION

Outlet structures: B [m]	Y [m]	CL [m]	Type	Free flow (o.m.)	Submerged (o.n)
1 0.186	0.308	153.15	OFRB	Y: $C_d = 0.53$ D/S WL = 153.2	N
2 0.162	0.308	152.87	OFRB	N	Y: field
3 0.180	0.320	152.73	OFRB	(field)	N
4 -	0.250	-	PIPE	N	Y: field
5 0.213	0.264	152.31	OFRB	Y: $C_d = 0.53$ D/S WL = 152.4	N
6 0.146	0.262	151.86	OFRB	Y: $C_d = 0.53$ D/S WL = 152.0	N

Upstream boundary condition: Discharge [m³/s]	

Downstream boundary condition:	Outlet / drop structure:		Rating curve:
Rating curve: Q(H)	type:	OF	$q(H) = 0.459 * H^{3/2}$
	flow:	f.f.	
	C_1:	1.34[2]	
	width B [m]:	0.201	
	height Y [m]:	-	
	crest level:	151.29 m	

MANNING'S COEFFICIENT

Visual analysis for every reach		
	0 m - 884 m :	- banks were good
	884 m - 1067 m :	- banks damaged
		more visual field data

SEEPAGE LOSSES

Inflow - Outflow test (8/10/1995)

Inflow: 0.57 m³/s (20.12 cfs)
Outflow: 0.12 m³/s (4.10 cfs)

S_e [0 m - 5000 m]: **1.7 l/s/km (inflow)**
S_e [5000m - tail]: **- 11.6 l/s/km (outflow)**

q outlet structures [m³/s]		Upstream water levels [m]	
1	0.116	0.76	153.91
2	0.054	0.74	153.61
3	closed	-	-
4	0.108	0.64	-
5	0.102	0.57	152.88
6	0.059	0.59	152.45

2

Discharge coefficient is based on the measurements on 3-L (8/10/1995): $C_1 = q / 1.7.B.H^{3/2}$, where: $q = 0.116\ m^3/s$, $B = 0.201\ m$ and $H = 0.4\ m$. C_1 is slightly to high (theoretically 1.0), but for the simplified model the measured value will be used.

Field measurements
Besides the inflow - outflow test, conducted by IIMI, the following measurements are necessary to set up the simplified model for 3-L:

1. Cross sectional measurements close to all outlet structures and the tail structure (open flume), with reference to the crest of the corresponding outlet structure.
   TIME: 2 hours, 2 man
2. Survey to establish the crest level of outlet structure 4 (submerged pipe), with reference to the crest level of outlet structure 3. (only approximately 600 metres levelling).
   TIME: 1 hour, 2 man
3. For outlet structure 2 and 4: measure *width of the watercourse [m], lined or unlined,* h_u, h_d, *depth of the watercourse [m].*
   TIME: 0.25 hour, 1 man
4. For outlet structure 3: check the general flow condition. If submerged, measure the same items as stated in point 3. If free flow (o.m.), its ok.
   TIME: 0.25 hour, 1 man
5. Describe the state of the distributary in between every node: vegetation, banks, cuts, in fill / in cut etc.
   TIME: 0.5 hour, 1 man

Total time and manpower for the exercise: **4 hours with 2 man.**

FIELD MEASUREMENT RESULTS

- Overall, the condition of the canal is bad. Many cuts and a weak bank was found at the left side of the canal. Vegetation was found al along the canal, therefor the initial n-value for the model will be set at 0.04 (k = 25). Especially the tail reach is heavy vegetated.
- Looking at the measurements of IIMI, outlet no. 4 (illegal pipe) is supposed to be free flow. Only a rating curve will be set up for outlet no. 2. Width of the watercourse: 0.75 m, unlined.
- Crest level of outlet no. 4 (illegal pipe) and cross section of outlet no. 2 (crest was covered with to much silt) are measured referred to the design crest of outlet no. 3.

Table F.2 Topographical survey results

Crest 3 ==> cross section 2 BS	FS	Crest 3 ==> crest 4 BS	FS
crest 3 + 8.66		crest 3 + 9.56	
+ 7.04	- 5.32	+ 3.54	- 5.01
+ 4.76	- 6.135	+ 5.085	- 4.48
+ 3.83	- 3.90	+ 4.825	- 5.26
+ 4.08	- 4.195		- 9.07 (= crest 4)
	- cross section 2		

SIMPLIFIED SIC MODEL OF 3-L

$Q_{head} = 0.57\ m^3/s$

Table F.3 SIC output and IIMI data

O/L	$q_{measured}$	$q_{initial}$	q (n)	q (C_d)	h_u (IIMI)	h_d (IIMI)	C_d (IIMI)
1	0.116		0.101	0.111	0.76	0.20	0.52 (o.m.)
2	0.054		0.051	0.052	0.74	0.66	0.86 (o.n.)
3	-		-	-	-	-	-
4	0.108		0.136	0.117	0.64	0.40	0.79 (pipe, o.m.)
5	0.102		0.082	0.097	0.57	0.37	0.54 (o.m.)
6	0.059		0.068	0.064	0.59	0.16	0.45 (o.m.)
Tail	0.116		0.113	0.015	0.40	0.24	1.34 (f.f.)

Calibration

n: For outlet 1, 2, 4, 5, 6 pre-define the measured upstream water levels in the model and all the measured discharges as imposed discharge. Simulate the model and the calibrated n will be computed.

Reach 1 - 5: $k = 22.5 / n = 0.044$
Reach 6: $k = 48.0 / n = 0.021$
Reach 7: $k = 25.0 / n = 0.040$

C_d: Simple adjustments of the initial discharge coefficient input was necessary.

1: *0.53 ==> 0.60*
2: *0.86 ==> -*
3: *0.53 ==> -*
4: *0.75 ==> 0.65 (pipe o.m.)*
5: *0.53 ==> -*
6: *0.53 ==> 0.50*