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COMPARISON OF DIFFERENT TOOLS TO ASSESS THE WATER DISTRIBUTION IN SECONDARY CANALS WITH UNGATED

Mobin-ud-Din Ahmad
E. G. van Waijjen
Marcel Kuper
Steven Visser

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PAKISTAN NATIONAL PROGRAM
INTERNATIONAL IRRIGATION MANAGEMENT INSTITUTE
LAHORE

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ABSTRACT

The present study invokes the use of three computer modeling tools to assess the water distribution in an irrigation canal with ungated outlet structures, i.e. the hydro-dynamic simulation model, Simulation of Irrigation Canals (SIC), and a Simplified Steady State model (S3), based on the Manning-Strickler equation. SIC was set up using two different approaches; SIC-I approach relies on intensive field data collection, whereas SIC-II distinguishes between sensitive parameters, which are measured in the field, and non-sensitive parameters, which are derived from secondary sources. The S3 model is based on discharges and water levels measured in the field, and on the Manning-Strickler equation. These three tools were compared by applying them to two secondary canals in the Chishtian Sub-division in the southeast of Punjab, Pakistan. The models were compared with respect to data requirements, time and associated costs for data collection, and the pertinence of the tool for specific studies in steady as well as unsteady state situations. It was shown that the time required to collect field data is approximately the same for the S3 model and SIC-II, whereas SIC-I requires about 50% more time. It was further shown that in case of ungated, fixed outlet structure (under steady state conditions) SIC-I and S3 show 2-5 % average absolute error in the simulated discharges for all tertiary outlets; this difference goes up to 7% for SIC-II. The S3 Model is not capable of producing exact hydrographs as it does not take the lag time, nor wave attenuation, into account. This limits its use under unsteady state conditions. However, if the objective of the study is to obtain monthly volumes delivered to tertiary outlets, the S3 model suffices largely to produce the water distribution. In terms of applications, SIC-I is capable for being used in studies of water distribution, canal regulation and planning of maintenance activities; SIC-II can be used for the same purposes as SIC-I, when a somewhat lower accuracy is acceptable, but SIC-II cannot be used for studies related to canal geometry (e.g. desiltation); while S3 can only be used for water distribution assessment.

1. INTRODUCTION

In many countries, especially in the semi-arid tropics, water is a scarce resource with increasing competition between domestic, industrial and agricultural demands. This requires a judicious use of water in all of these sectors, but especially in irrigated agriculture, which is by far the biggest consumer of water. Minimising the existing water use does not only serve to permit its use outside of agriculture, but is also needed to sustain the increasing need for food stuffs with the existing population growth rate. For an efficient use of water resources in the agricultural sector, a thorough understanding of the existing water distribution system is of prime importance in order to be able to deliver just the amount required at the right time and at the right place.

Different methods have been used to analyse the water distribution in irrigation canals. Recently, advances in computer technology have enabled the development of mathematical hydraulic models, based on the discretisation of the St. Venant equations. Thus, the effect of manipulations of gated structures on the water distribution can be studied. The use of these models is relatively time-consuming, however, and requires a substantial data set for their calibration/validation. Waijjen *et al.* (1997) estimate that 8 person-days are required to collect sufficient information for a 10 km stretch of canal in addition to the time that is required for setting up the model on the computer. While the use of these models appears justified in the case of automatic or manual regulation, or control of irrigation canals, these models can be simplified in the absence of gated structures. The simplifications can consist of limiting the inputs of existing models, or of replacing the algorithm, governing the water flow.

The present study endeavours to compare the use of three tools to analyse the water distribution in irrigation canals without gated structures. The first tool is an unsteady state hydraulic model based on the St. Venant equations, referred to as SIC - Simulation of Irrigation Canals (Malaterre and Baume, 1997). The model is developed furnishing all the required input data. The same model is also used with a minimum field-measured data set, based on the work of Visser *et al.* (1997). Thirdly, a simple steady state model in a spreadsheet with the Manning-Strickler equation has been developed to analyse the water distribution. The study is applied to two existing secondary canals in Pakistan's Punjab, where the water distribution depends on the physical state of the canals and the location and dimensions of tertiary outlets.

The aim of the present study is then to make a comparison between the use of the three tools for analysing the water distribution in irrigation canals with ungated offtakes, with a view to intervene in the water distribution. The outputs of these tools will be evaluated and recommendations for the selection of a specific tool for a certain application will be formulated.

2. WATER MANAGEMENT AT THE SECONDARY CANAL LEVEL IN PAKISTAN'S PUNJAB

In this chapter, the water management at the secondary level in Pakistan's Punjab is described and analysed. The water distribution at this level is governed by the inflow, controlled through a gated or ungated structure, and by the state of the physical infrastructure, i.e. the secondary canal, its drop structures and off-taking tertiary outlets. These outlets are not gated. The hydraulic characteristics of canal and outlets are further investigated to determine which parameters are determinant for water distribution. The dimensions, type and settings of the outlets appear to be the most important influencing parameters.

The Indus Basin Irrigation system is one of the largest contiguous irrigation systems of the world. Annually it diverts about 128 billion cubic meters of surface water through an extensive hydraulic network to 45 canal commands. A canal command is typically served though a gated structure on one of Pakistan's rivers that divert water into the main canal. The main canals serve branch canals, which in turn supply water to secondary canals, or distributaries. In general, there are no gated control structures at the secondary canal level in Pakistan's Punjab. Below the gated head structure of a secondary canal, water is distributed by means of fixed tertiary outlet structures, either an orifice, pipe, or an open flume. Canal maintenance and outlet structure modifications are the only 'tools' available to intervene in the existing water distribution. The water delivered at the head of a secondary canal is, thus, distributed to all tertiary outlets and minor canals along the secondary canal. The sum of the distributed discharges to the tertiary outlets, plus the seepage losses, thus equal the incoming discharge at the head of the secondary canal:

$$Q_{head} = \sum_{i=1}^{n} q_i - S_e \qquad (2.1)$$

where:

Q_{head}	=	Discharge at the head of the secondary canal	[m^3/s]
q_i	=	Discharge through an outlet structure	[m^3/s]
S_e	=	Seepage	[l/s/km]
n	=	Number of outlet structures	[-]

2.1. PRINCIPLES OF IRRIGATION AT THE SECONDARY CANAL LEVEL

The Indus Basin Irrigation System was developed more than a hundred years ago for protective irrigation, i.e. to spread the limited water resources over an area as large as possible. The design of irrigation canals and structures followed a number of design principles to ensure the desired water distribution and limit the maintenance requirements of the system. Those principles related to water distribution at the level of the secondary canal, are equitable distribution and proportional control.

2.1.1. Equitable Distribution

Within a secondary canal, the distribution of canal water to the tertiary outlet structures is based on the principle of equitability. Equitability of water distribution can be defined as a distribution of a fair share of water to users throughout the system (Kuper and Kijne, 1992). A discharge is 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 and Vander Velde, 1992). The duty is expressed as a quantity of water per 1000 acres of culturable command area (CCA), i.e. the physical irrigable agricultural area commanded by the outlet structure. 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 to the area served; this discharge is called the *authorised discharge* (q_{auth}).

2.1.2. Proportional Control

The water distribution at the secondary canal level is also based on proportional control, i.e. a flow control method in which the flow is divided into a fixed ratio, irrespective of the flow rate. Thus, disturbances will be proportionally distributed; an increase in discharge at the head of a secondary canal 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:

$$Q = \beta . H^{u}_{c} \ and \ q = \alpha . H^{n}_{w} \tag{2.2}$$

With the assumption that a change in water level in the secondary canal (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 2.1):

$$S = \frac{\frac{dq}{q}}{\frac{dQ}{Q}} = \frac{\frac{n\alpha . H^{n-1}_{w}}{\alpha . H^{n}_{w}}}{\frac{u . \beta . H^{u-1}_{c}}{\beta . H^{u}_{c}}} = \frac{n . H_c}{u . H_w} \tag{2.3}$$

where:

S	=	Sensitivity factor	[-]
q	=	Distributed discharge to outlet structure	[m³/s]
dq	=	Change in distributed discharge to outlet structure	[m³/s]
Q	=	Discharge secondary canal	[m³/s]
dQ	=	Change in discharge secondary canal	[m³/s]

α	=	Depth-discharge coeff. outlet structure[$m^{1.5}/s$ (weir flow); $m^{2.5}/s$ (orifice flow)]	
β	=	Depth-discharge coefficient secondary canal	[$m^{4/3}/s$]
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	[-]

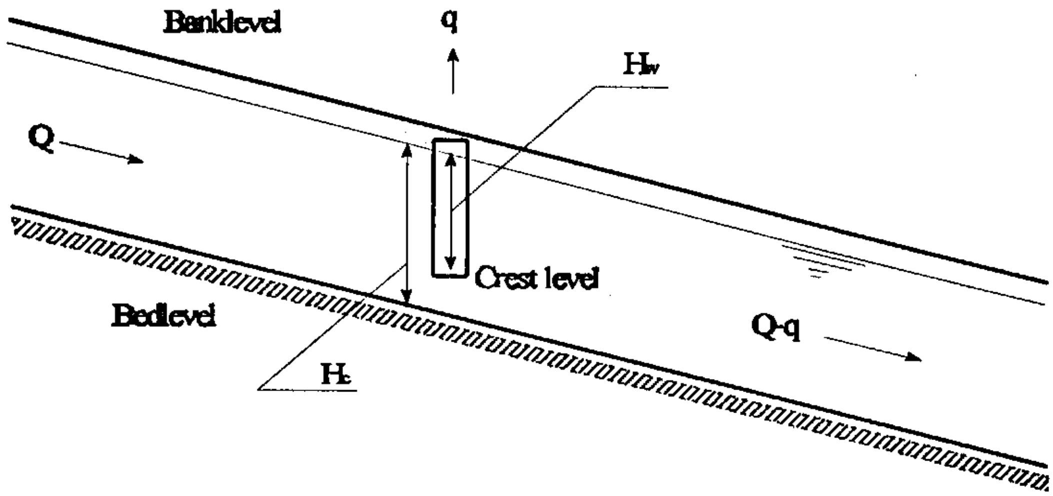


Figure 2.1. Longitudinal profile of a canal with an outlet structure.

Four situations can be distinguished, analysing the sensitivity ratio for a bifurcation of a secondary canal, i.e. a tertiary outlet structure (Ankum, 1993):

S = 0
No sensitivity of the outlet structure discharge to changes in the discharge in the secondary canal. 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 secondary canal. 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 secondary canal. 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 secondary canal are distributed to the head reach outlet structures.

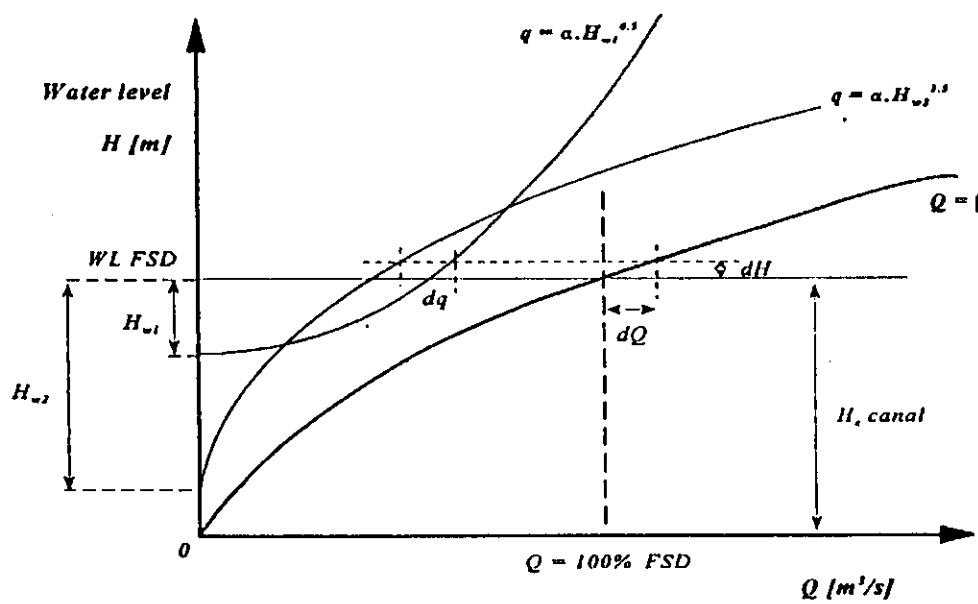


Figure 2.2. Theoretical analysis proportionality for design of outlet structures.

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 secondary canal, a sensitivity ratio of 1 can be obtained in one particular point only (n and u are not similar). When the discharge at the head of the secondary canal is equal to q_{auth}, there is only one combination of ongoing discharge (water level) in the parent canal and allocated discharge to an outlet structure where S = 1: dq/q = dQ/Q. This is illustrated in Figure 2.2.

where:

q	=	Distributed discharge to an outlet structure	[m³/s]
n	=	Exponent; 0.5 for orifice flow and 1.5 for weir flow	[-]
Q	=	Discharge secondary canal	[m³/s]
dH	=	Change in water level in the secondary canal	[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 always be proportional behaviour for different water levels.

So, in order to accomplish proportional behaviour, the discharge-depth relationship of the secondary canal must be related to the discharge-depth relationship of the outlet structure, i.e. S = 1. With the discharge-depth relationship of the secondary canal 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):

$$Q = k.B.H_c.H^{2/3}{}_c.i^{1/2} \Rightarrow Q = \beta.H^{5/3}{}_c \qquad (2.4)$$

$$\frac{dQ}{Q} = \frac{5}{3}.\frac{dH_c}{H_c} \qquad (2.5)$$

where:

Q	=	Discharge secondary canal	[m^3/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:

$$q = \alpha.H^n{}_w \qquad (2.6)$$

$$\frac{dq}{q} = n.\frac{dH_w}{H_w} \qquad (2.7)$$

where:

q	=	Discharge outlet structure	[m^3/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 equalling 1 leads to:

$$S = \frac{\frac{dq}{q}}{\frac{dQ}{Q}} = \frac{n.\frac{dH_w}{H_w}}{\frac{5.dHsubc}{3.H_c}} = 1 \quad (2.8)$$

$$H_w = \frac{3.n.H_c}{5} \quad (2.9)$$

Where n is defined by the type of outlet structure.

For weirs $n = 1.5$, for orifices $n = 0.5$.

- Weir flow : $H_w = H_{w,1} = 9/10.H_c$ (2.10)
- Orifice flow : $H_w = H_{w,2} = 3/10.H_c$ (2.11)

Practically speaking, for weir flow the crest of the open flume should be placed at 1/10 of the depth above bed level of the secondary canal, when the discharge in the parent channel is at its authorised value. For orifice flow, the roof block should be placed at 0.7 of the depth above bed (Ali, 1993; Mahbub and Gulhati, 1951).

For pipe outlet structures, when the parent canal is running at its authorised discharge, the head over the structure should be 0.3 of the water depth 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 tertiary canal, should be approximately at 0.3 of the water depth in the parent canal, below the full supply level in that canal.

By changing the width and height of the opening, the authorised design discharge will be obtained. In Figure 2.3, the design concepts of canal water distribution at the secondary level are presented. Whenever the secondary canal is running at its authorised discharge, the supplied discharge to all the outlet structures equals their authorised discharge, and with these settings, the requirement, S = 1, will be met. To speak with Kennedy (1906), a secondary canal 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 secondary canal*'.

We will see later on (Section 2.2.2, Description of outlets) that the principle of proportionality conflicts with practical problems of siltation in secondary canals. In order to divert the sediments to tertiary canals, the silt draw of tertiary outlets was improved by lowering the crest settings. Thus, proportionality is no longer achieved.

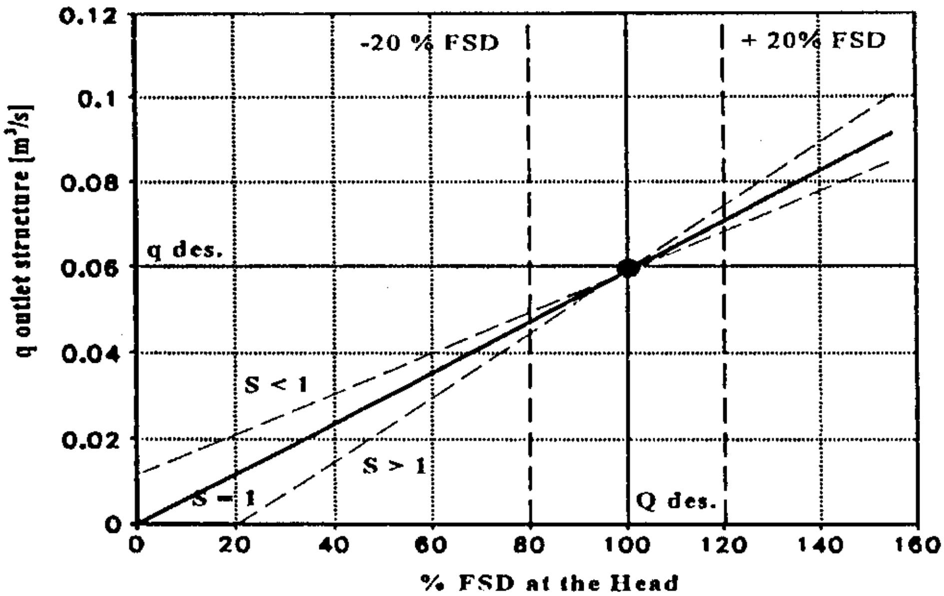


Figure 2.3. Design principles proportionality and equity for an outlet structure.

2.2. OUTLET STRUCTURES IN THE PUNJAB

Outlet structures have a great impact on the water distribution in secondary canals, which explains the need for a thorough understanding of the hydraulic behaviour of the different outlet structures, as they exist in the Punjab irrigation system. An *outlet* or, *mogha,* is a masonry structure through which water is admitted from a state-governed secondary canal into a farmers' tertiary canal. It is the responsibility of the irrigation department to supply water in accordance with the authorised discharges to the tertiary outlets, while the water distribution within the tertiary unit is the responsibility of the farmers.

First, the factors that 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.

2.2.1. Factors Determining the Design of an Outlet Structure

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

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 '*main d'eau*', 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 and Mahbub, 1951). Based on that, a classification of outlet structures can be distinguished:

Table 2.1. 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 severe siltation along the canal, the silt load must be equitably distributed to all the secondary canals; and, within a secondary canal, to all the tertiary canals. Each outlet structure must thus take its fair share of silt. The essential geometric features of outlet structures that determine the silt-drawing capacity, are summarised below (Khan, 1996). The scope of this report does not encompass discussing these concepts in more 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 the bed level of the secondary canal.

To obtain equitable distribution of silt along all watercourses, and due to seepage losses of approximately 10% to 15% (of the inflow) in secondary canals, 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 secondary canal (100%).

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

Other Essential Factors

- 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 where discharge is independent on both, the upstream water levels in the secondary canal, and the downstream water levels in the watercourse (in between reasonable limits).

Semi-modular outlet structures are those outlet structures where discharge is dependent on the upstream water levels in the secondary canal, 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 where discharge is both, dependent on the upstream water levels in the secondary canal, and the downstream water levels in the watercourse.

2.2.2. Hydraulic Principles of Different Types of Outlet Structures

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 the upstream water level and the 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; $S_f = h_u / h_d$, also known as the minimum modular head. Free flow and submerged flow are the two major flow types.

$$Q_{ff} = f(h_u) \qquad Q_{sf} = f(h_u, h_d) = f(h_u - h_d, S_f) \tag{2.12}$$

where:

Q_{ff}	=	Free flow discharge	[m^3/s]
Q_{sf}	=	Submerged flow discharge	[m^3/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 2.4., and are discussed for the different types of outlet structures present in the area of study.

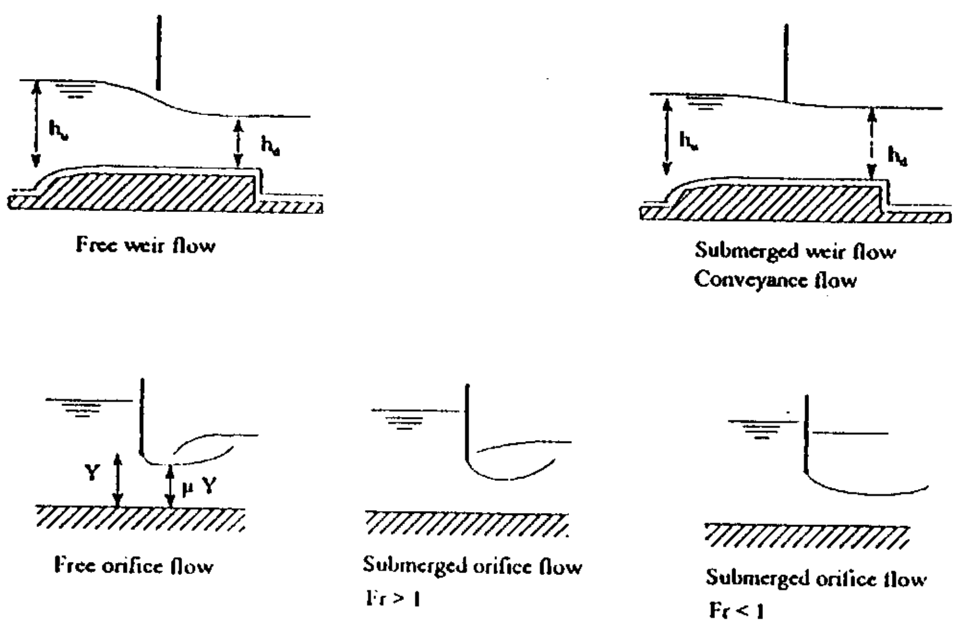


Figure 2.4. Types of flow condition for weirs and orifice flow.

Open Flume

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 straight 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 outlet 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 sensitivity to tampering and improved designs (FAO, 1982). At present, the open flume outlet structures in the Punjab are Crump's type and Jamrao type 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 absorb an excess of water with ease.

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 sufficiently 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:

$$q = C_f . 1.7 . B . H^{3/2} \tag{2.13}$$

where:

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

Q=C. 1.7. B. H$^{3/2}$

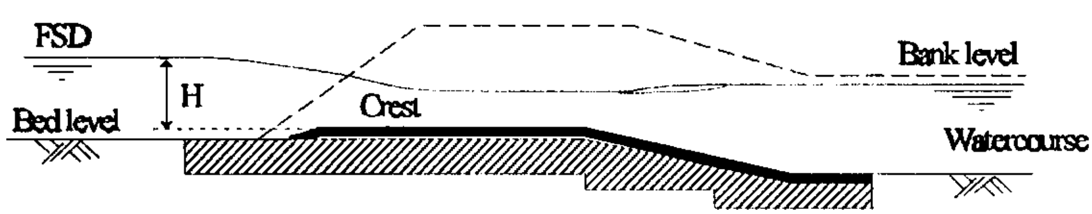


Figure 2.5. Broad Crested weir (Open Flume).

[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): $h_u / L > 0.75$; $C_f = 1$, so there is still free weir flow.*

The coefficient of discharge (C_1) is influenced by several factors; the side contractions, the shape of the crest, the length of the crest, and the head, H_u. The difference between 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 is $C_1 = 1\ m^{1/2}/s$ (theoretical value, in reality it will be approximately $0.95\ m^{1/2}/s$) and for a short-crested weir is $C1 > 1\ m^{1/2}/s$.

Table 2.2. 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 therefore, it becomes necessary to raise the crest of the outlet above the bed level, and to 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).

Open Flume with Roof Block (OFRB)

The main disadvantage of the open flume is its sensitivity to 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 PID started placing 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 2.6). The open flume starts to function as an orifice whenever the upstream water level rises, which results in a decrease in discharge. At present, practically all OFRBs are functioning as orifices in full supply conditions. The following rules have been approved in the eastern

$$C_d = \frac{\alpha}{\sqrt{1 + \alpha . \frac{Y}{H_u}}} \tag{2.16}$$

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

The outlet structure is designed to function as an open flume, but due to siltation, i.e. an increase in bed level elevation, the water levels at FSD are higher than design water levels, and therefore, in most cases, the OFRB outlet structures function 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 influences the discharge trough of 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 also determines the discharge and the discharge equations becomes:

$$q = C_d . B . Y . \sqrt{2 . g . (H_u - H_d)} \tag{2.16}$$

where:

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

Adjustable Orifice Semi-module (AOSM)

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 too 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 silt distribution. The AOSM consists of a long- throated flume (approximately 0.60 m) with a roof block, capable of vertical adjustments, and of a rounded roof to prevent contraction and ensure straight stream lines. The structure is built from 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 or 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 does not influence the discharge through the structure. The hydraulic jump is formed at some distance from the gate.

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

$$q = C_d . B. Y. \sqrt{2.g.z} \qquad (2.18)$$

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 with the crest at bed level draw 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.

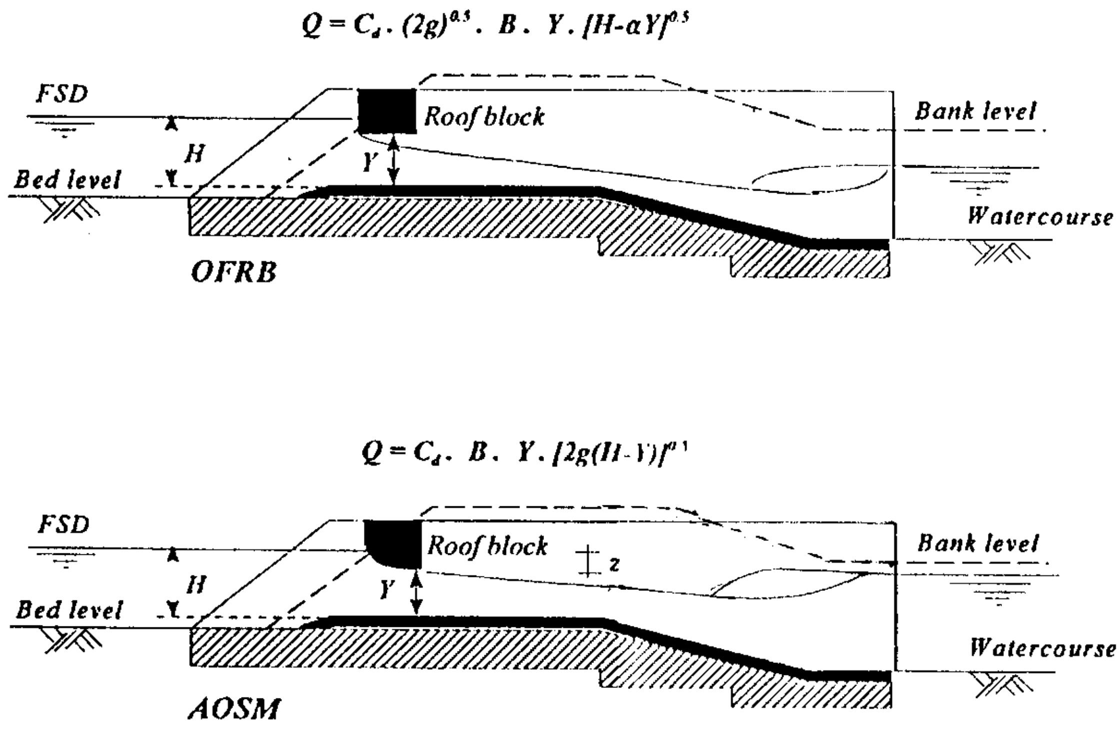


Figure 2.6. Orifice flow for OFRB and AOSM outlet structures.

Table 2.3. 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.

Pipe Outlet Structure

Pipe outlet structures are the most simple and oldest known types in the Punjab. In early days, 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 therefore, most outlet structures are running submerged. The pipe outlet structure consists of an upstream head wall, a pipe, and a downstream head wall. The entrance is usually at bed level, or just above bed level.

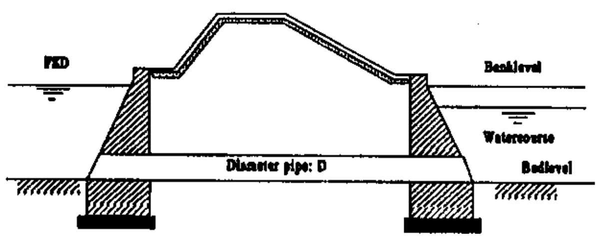


Figure 2.7. Pipe outlet structure.

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 often is damaged 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 its operation with a very low working head (minimum 2.5 cm, with which no semi-module can function).

Discharge Equation

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

$$q = C_p . A . \sqrt{2.g.z} \qquad (2.19)$$

where:

q	=	Discharge $[m^3/s]$	
C_p	=	Discharge coefficient of a pipe outlet structure	[-]
g	=	9.8 m/s^2 (gravity acceleration)	$[m/s^2]$
A	=	Area of the opening	$[m^2]$
z	=	Energy head measured from	[m]

1. Centre of the pipe to the water level in the parent canal, when flow enters in the free air; and
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.

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

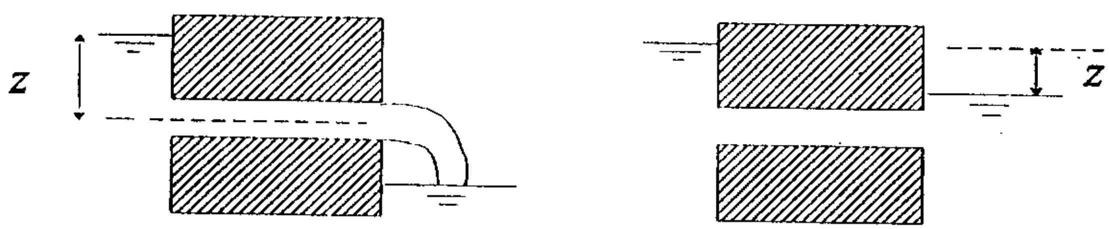


Figure 2.8. Energy head z for pipe outlet structure.

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.

Pipe/Crump Semi-module

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 takes off from the parent canal and opens into an approximate 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 by any one of 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 and Gulhati, 1951):

- Large modularity/rangees;
- Cheap construction, especially in large canal banks;
- Easily adjustable settings, when the canal is running; and
- Protected from 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.

2.2.3. 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 the opening height Y, as defined in the typical outlet structure equation. For **submerged outlet structures**, besides the characteristics mentioned above, the discharge is dependent 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 changes 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 2.4.

Table 2.4. 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 and (OC)AOSM / AOSM	- upstream water level - crest level - B - Y - C	- 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

3. METHODOLOGY

3.1. RESEARCH APPROACH

The study compares the use of three tools to analyse the water distribution in irrigation canals without gated structures. The first tool is an unsteady state hydraulic model based on the St. Venant equations, referred to as SIC, Simulation of Irrigation Canals (Malaterre and Baume, 1997). The model is developed furnishing all the required input data. The same model is also used with a minimum (sensitive) field-measured data set, thus, limiting the input (time) requirements, based on the work of Visser *et al.* (1997). Thirdly, a Simple Steady State model in a spreadsheet with the Manning-Strickler equation has been developed to analyse the water distribution.

The comparison is carried out with respect to the development of the model, with parameters such as data requirements and efforts required to set up the models, and to the application of the models. The models will, therefore, first be set up and then calibrated and validated with reference to field measurements of water levels and discharges. Finally, the models will be applied to a number of water distribution scenarios in steady as well as unsteady states in order to illustrate the use of these models.

3.2. RESEARCH LOCALE

For the comparative study of different models, the Chishtian Sub-division with the gross command area of 75,000 has been selected. Geographically, it forms part of the Fordwah/Eastern Sadiqia Irrigation System, which is confined by the Sutlej River in the north-east, the Indian border in the east, and by the Cholistan desert in the south-east. The climate is semi-arid with an average annual rainfall of 260 mm, far lower than the annual evaporation of 2,400 mm.

The main source of irrigation water is the river water distributed to farmers through an extensive network of 14 secondary canals, or *distributaries*, and 510 tertiary canals, or *watercourses* (shown in Figure 3.1). Because of limited water supplies in winter (*rabi*) season, these distributaries have been divided in perennial and non-perennial (receive water only in summer season, mid April to mid October) channels. Farmers augment canal water by pumping groundwater through more than 4,400 tubewells (about 6 tubewells per 100 ha). The main crops cultivated in the area are cotton, rice, sugarcane (annual), and fodder during the summer (*kharif*) season and, wheat and fodder during winter (*rabi*) season.

Two distributaries, viz. Masood and Fordwah Distributaries were chosen for detailed comparison in the middle and tail reach of the Chishtian Sub-division. Masood Distributary runs all the way along the main canal, Fordwah branch, whereas, the Fordwah distributary is one of the last distributaries of this system. Masood Distributary with 14 outlets (all on the right bank), and Fordwah Distributary, with 89 outlets, exemplify a typical small and large secondary canal system in Pakistan. Their basic characteristics are summarised in Table 3.1 (Iqbal, 1996). Both distributaries are unlined,

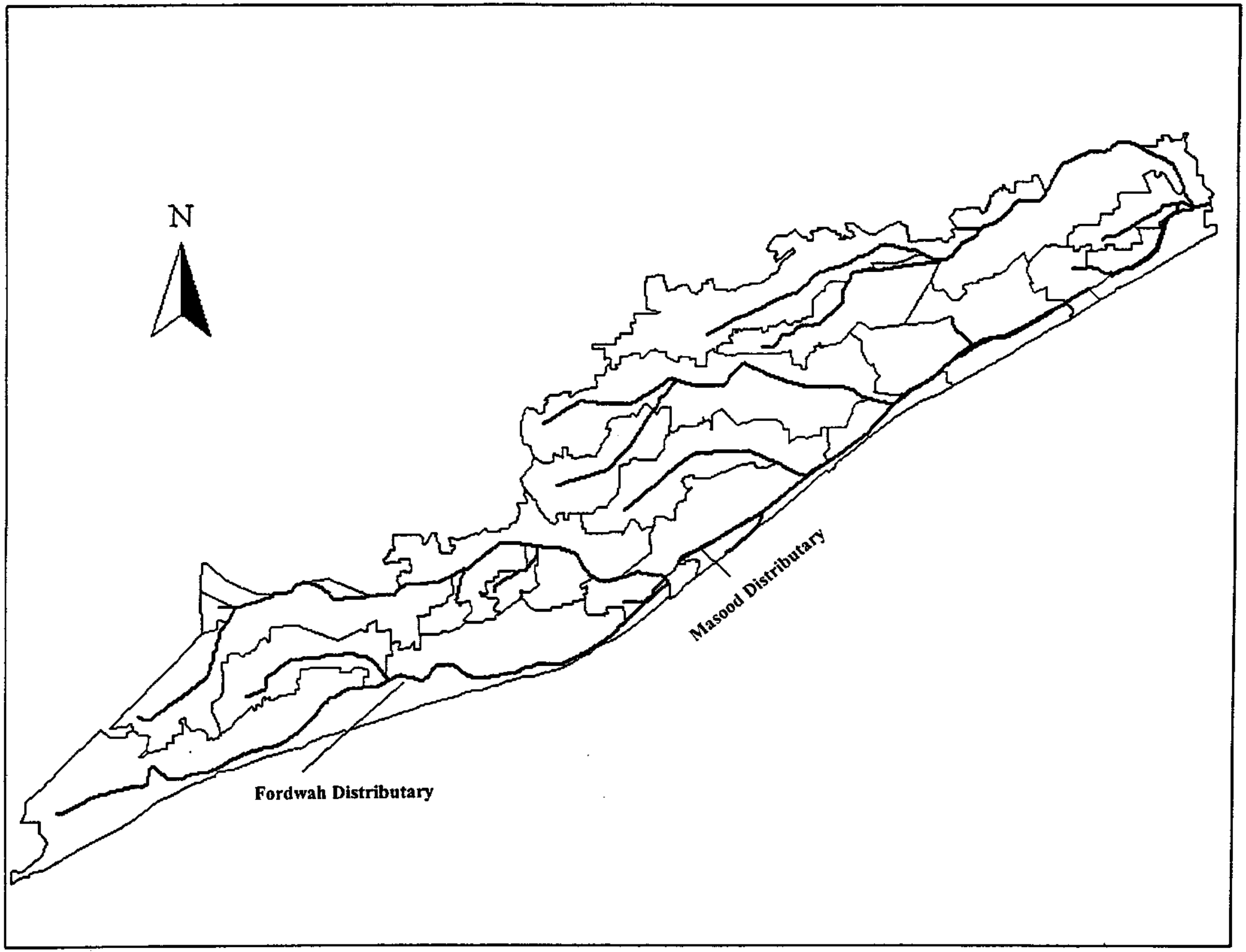


Figure 3.1. Distributaries and their command areas in the Chishtian Sub-division.

and are perennial channels with design discharges of 0.99 cumecs (35 cusecs) and 4.47 cumecs (158 cusecs) for Masood and Fordwah Distributaries, respectively. The design slope of these distributaries ranges from 0.00015 to 0.00028. The OFRB and AOSM types of outlets are dominant in these distributaries.

Table: 3.1. Basic characteristics of Masood and Fordwah distributaries.

	Masood Distributary	Fordwah Distributary
Status	Perennial	Perennial
Design Discharge	0.99 $m^3 s^{-1}$	4.47 $m^3 s^{-1}$
Length	15.90 km	42.60 km
CCA	3279ha	14850 ha
Number of Drop Structures	2	3
Number of Minors	0	1
Types and Number of Outlets		
AOSM	-	39
OFRB	12	12
OCAOSM	-	4
OCOFRB	-	20
Pipe	2	7
Flume	-	5

3.3. DESCRIPTION OF SIC (SIMULATION OF IRRIGATION CANALS) MODEL

Main Components

The hydro-dynamic software SIC is built around three main components (computer programs TALWEG, FLUVIA and SIRENE) that, respectively, generate topography, compute the steady and the unsteady flows. For the present study, Version 3.0 of SIC was used, which allows the study of looped or branched networks and allows the iteration of the discharge calculation for all the off-taking nodes. The three units are:

Unit I

The topographical and geometrical layout of the canal is specified in this unit. The topographical and geometrical files are used by unit II and III. The canal is 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 as 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.

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 3.2).

$$\frac{dH}{dx} = -S_f + (k-1).\frac{qQ}{gA^2} \tag{3.1}$$

And: $$S_f = \frac{n^2 Q^2}{A^2 R^{4/3}} \tag{3.2}$$

where:

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 which allows testing various scenarios 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 is based on the Saint Venant's equations solved numerically by discretising the equations. The discretisation 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).

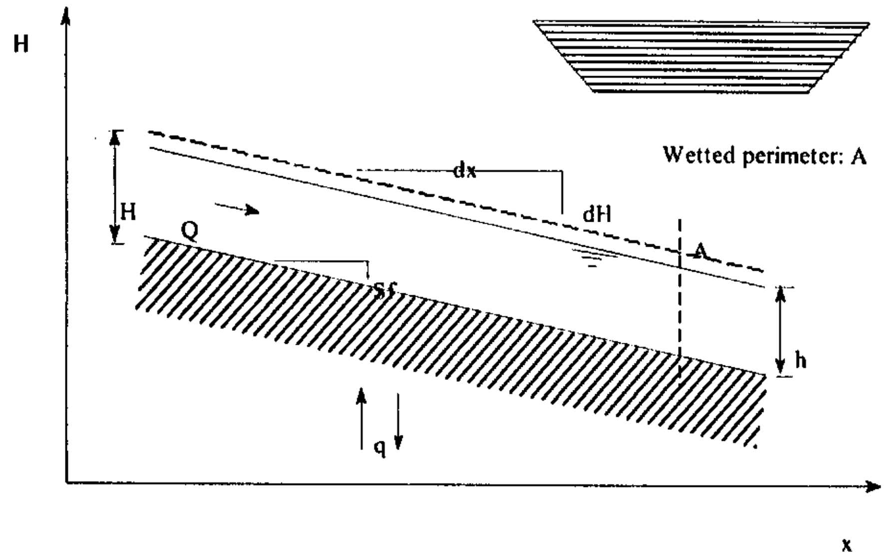


Figure 3.2. Longitudinal profile of a canal.

Saint Venant's equations (SIC User Guide Part II: Theoretical concepts, 1995):

Continuity (conservation of mass of water):

$$\frac{\delta A}{\delta t} + \frac{\delta Q}{\delta x} = q \tag{3.3}$$

Dynamic equation:

$$\frac{\delta Q}{\delta t} + \frac{\delta (Q^2/A)}{\delta x} + g.A.\frac{\delta h}{\delta x} = -\,g.A.S_f + k.q.V \tag{3.4}$$

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 hydrographs at the upstream nodes of the reaches, and a rating curve at the downstream node of the model.

SIC has been used in a number of countries around the world and its computational accuracy has been verified against the benchmarks of the American Society of Civil Engineers (Malaterre and Baume, 1997). In Pakistan, the model has been used by IIMI for various purposes, mainly related to the manual operation of irrigation canals (Kuper *et al.*, 1994; Litrico, 1995; Kuper *et al.*, 1997) and to maintenance and sediment transport (Habib *et al.*, 1992; Hart, 1996; Visser *et al* ., 1997 ; Belaud, 1996 ; Vabre, 1996). In most of these cases the SIC set up is based on an extensive set of field measurements (topographic survey to determine the geometry and elevation of the canal sections, location, dimensions and crest levels of structures (drops, outlets)). Similarly, the model was generally calibrated and validated using field measurements of water levels and discharges at strategic locations (outlets, drops). Since the model was used extensively, a study was carried out by Visser (1996) to study the possibility of reducing the field data required to set up, calibrate and validate the model. The results of this study were encouraging, and have been included in the present study. The data intensive and data-extensive use of SIC will be referred to as SIC-I and SIC-II :

SIC-I

According to this approach, all the input/information required (see section 3.5.1) by the model is actually collected from the field. Based on field data, the model is calibrated and validated.

SIC-II

Based on a sensitivity analysis of the input parameters, the outlet dimensions were found to be very sensitive parameters affecting the water distribution in secondary canals, while the geometry and absolute levels of the canal were found to be much less sensitive. The cross-sectional profile and the crest level of the cross structures are, therefore, based on the design crest level of outlets. This minimises the extent of the topographic survey, by taking only a few cross-sections (at locations of outlets, and up- and down-stream of every drop structure) with reference to *design* benchmarks. Finally, the model can be calibrated and validated by refining the Manning-Strickler coefficient and the discharge coefficient of every outlet to match actually measured discharge.

3.4. DESCRIPTION OF SIMPLIFIED STEADY STATE (S3) MODEL

3.4.1. Field Data Set

In the framework of an integrated model of irrigated agriculture in the Chishtian Sub-division, Punjab, Pakistan, a methodology was developed and applied to determine the supply to outlets along a secondary irrigation canal as a function of the inflow at the head of the canal. The objective was a model that can be set up quickly, without a topographic survey, as is needed by dynamic hydraulic models, such as SIC. Initially, the name 'volume balance model' was used. However, more complicated models also contain volume balance calculations, so the name 'Simplified Steady State model', or 'S3 model' for short, was found more appropriate.

The methodology that is described below was developed for use with an existing data set, which proved to be very suited for the purpose. In 1995 and 1996, IIMI conducted hydraulic surveys for all distributaries in Chishtian Sub-division. For each distributary, all outlets were calibrated by measuring discharge, orifice width (B), orifice height (Y), upstream head (Hu), and in case of submerged flow, also downstream head (Hd). This took several days for the bigger distributaries. Then, on one day, the inflow was kept constant, and an inflow-outflow test was conducted. During the inflow-outflow test, for each outlet, Hu, and where necessary, Hd, were recorded, *and also the water depth in the canal (D).* Q was only measured if Hu was different from the previous value. Although the water depth was observed with the objective of assessing the condition of the canal (siltation), an additional benefit is that it makes the calculation of changes in D possible, and thus, changes in Hu for each outlet.

3.4.2. Volume Balance and Outlet Sensitivity

The basic equation of a volume balance of a canal is:

$$Q_{inflow} = \sum_{i=1}^{n} q_i + \text{seepage} \tag{3.5}$$

where Q_{inflow} is the supply at the head of the distributary, q_i the supply to outlet no. i, and n the total number of outlets. Evaporation losses are ignored, and changes in storage can be ignored if there is a steady state, or if the observation period is long enough.

Initially, it was proposed to base the procedure for setting up the S3 model on the concept of 'outlet sensitivity', describing how the outlet discharge responds to changes in discharge in the distributary, depending on the hydraulic characteristics of the outlet and the canal.

Outlet sensitivity is defined in this way:

$$S = (dq / q) / (dQ / Q) \tag{3.6}$$

Where q = outlet discharge and Q = canal discharge

If S = 1, the water distribution is proportional, i.e. a 1% change in the distributary discharge leads to a 1% change in the outlet discharge.

However, this concept proved calculations to be either inaccurate, or very cumbersome. The sensitivity of an outlet is not constant, but changes when the discharge in the distributary changes, because

- the ratio between the depth in the canal and the working head of the outlet changes, and
- the flow condition of the outlet can change, e.g. from orifice to flume.

Therefore, it was preferred to calculate changes in water level in the distributary first and then to calculate the change in outlet discharge with formulas based on the structure's hydraulics.

Most unlined canals in the Punjab have a cross-section with a wide flat bottom (bed) and steep side slopes.

The Manning-Strickler formula for discharge in a rectangle canal is:

$$Q = k * B * D * D^{2/3} * i^{1/2} \quad \text{or} \tag{3.7}$$

$$Q = C_1 * D^{5/3} \tag{3.8}$$

Where k = roughness coefficient; B = width of the canal; D = water depth in the canal; and i = slope.

Hydraulic radius is taken as equal to the depth, assuming width is much greater than depth.

From a set of measured values of canal discharge and depth (Q_0 and D_0), the new depth can be calculated for any given discharge with the formula.

$$D = D_0 * (Q / Q_0)^{3/5} \tag{3.9}$$

The change in the water level $\Delta D = D - D_0$. To calculate outlet discharges, a similar procedure is followed. The general discharge formula for an outlet is:

$$q = C_2 * H^n \tag{3.10}$$

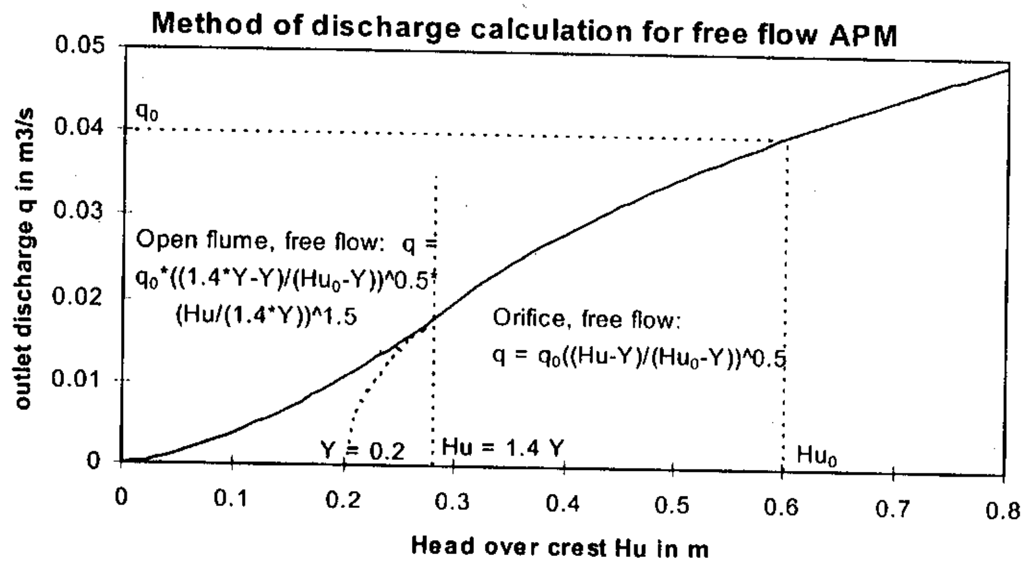
Where H = hydraulic head; n = 0.5 for orifices, and 1.5 for flumes and weirs.

From a set of measured values of outlet discharge and hydraulic head (q_0 and H_0), the new outlet discharge can be calculated for any given hydraulic head with the formula:

$$q = q_0 * (H / H_0)^n. \tag{3.11}$$

The change in the hydraulic head can be calculated from the change in the canal water depth.

Complications arise from the variety of outlet types and flow conditions. In Figure 3.3 below, is shown how discharges calculations in case of free flow APM and OFRB are shown. The free flow open flume is handled in a similar way, but it has no change in the flow type when the water level drops. Submerged APM and OFRB offer more of a



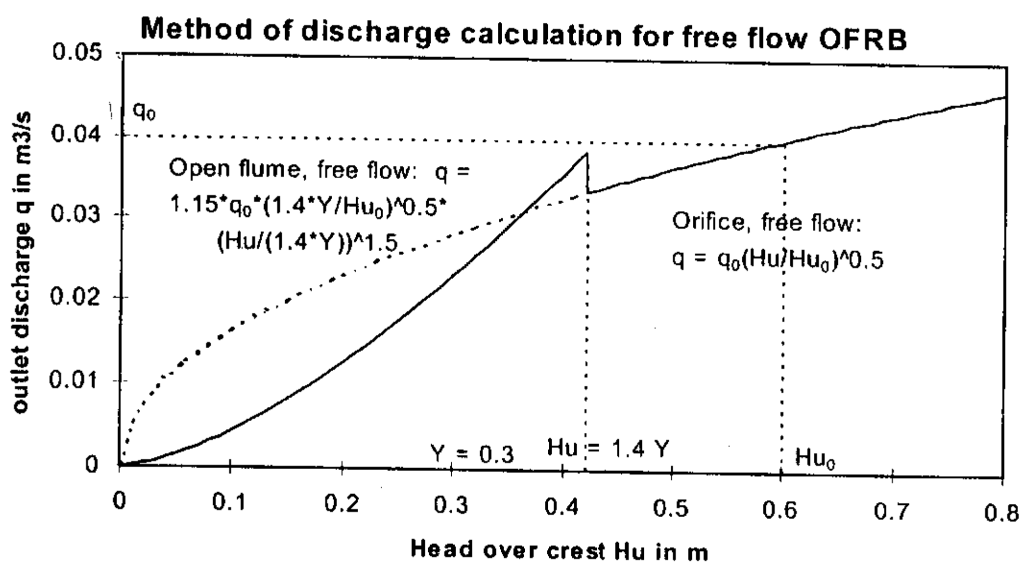


Figure 3.3. Methods of discharge calculation for free flow APM and OFRB.

problem, because it is not known how the downstream water level, Hd, changes with a changing discharge. The question is: what is the minimum water level in the watercourse, below which no water can be supplied to the fields? In the SIC model, this minimum Hd is taken as equal to the crest level of the outlet, often more than 40 cm below the observed Hd near full supply discharge. In the S3 model, it is assumed that fields can no longer be irrigated if Hd drops more than 15 cm. (Of course this will depend on whether a farmer with a high or a low field, close or far from the *mogha* has his irrigation turn.) The hydraulic head of the submerged orifice is taken as $Hs_0 = Hu_0 - Hd_0 + 0.15$.

The discharge through submerged APM and OFRB outlets is calculated with:

$$q = q_0 * ((Hs_0+\Delta D) / Hs_0)^{0.5} \tag{3.12}$$

The transition from submerged orifice to submerged flume is assumed to take place at Hu = Y.

Below this level,

$$q = q_0 * ((Hs_0+Y-Hu_0) / H_0)^{0.5} * ((Hs_0+\Delta D) / (Hs_0+Y-Hu_0))^{1.5} \tag{3.13}$$

A special case, is a proportional division structure where a minor branches off. If the crest levels of the minor and the on-going distributary are equal, and both are free-flow, the sensitivity of the minor is 1. Unfortunately, many proportional dividers have become submerged. In case of Jiwan Minor of Fordwah Distributary, the minor is free-flow, but the control structure in the distributary is completely submerged. The distributary behaves almost as if there is no control structure, so for Jiwan Minor the same formulas for D and for q can be used as for outlets.

3.4.3. Structure of the Model

The relevant field data are copied into a spreadsheet (VOLUMBAL.XLS), one row per watercourse. The following data are needed, entered in one column each: q_0, Hu, Hd, D, Y, outlet type (APM, OFRB, pipe, flume), flow condition (orifice or open flow, submerged or free flow). The subscript $_0$ means: as measured on the day of the inflow-outflow test. Apart from the outlet data, two more parameters are known; inflow at the head, and seepage losses in $m^3/s/km$ (calculated from the inflow-outflow test).

The volume balance is applied to calculate the local Q_0 just upstream of each outlet, moving downstream along the canal. Each time, q_0 of an outlet and the seepage for the reach between two outlets is subtracted from the canal discharge to obtain the canal discharge at the next outlet. Discharge after the last outlet will be exactly zero, since the same data were used to calculate the seepage.

In the same way, the canal discharge Q just upstream of each outlet can be calculated for a new value of the inflow. With the new local canal discharge, the new water depth and the new outlet discharge are calculated, as described above. The canal discharge just upstream of an outlet is used to avoid circular calculations. In reality, water level at an outlet depends on the canal discharge downstream of that outlet, which is affected by the supply to the outlet, which depends on the water level. But the relative change in the downstream Q will not be much different from that of the upstream Q, because outlet supplies are much smaller than the canal discharge and outlets are not operated.

3.5. COMPARISON OF MODELS

The different models to assess the water distribution differ from each other in terms of data requirement, output, complexity and prospects of potential use (implementation). Comparison of these models can be made based upon the following categories related to model development and its uses for different applications.

- Development of Models
  - *Data Requirement*
  - *Implementation Aspects*
- Comparison of Application

3.5.1. Development of Models

Data Requirement

The input data requirement determines the number of parameters measured, along with the required accuracy and associated time and space steps; an important criterion for model evaluation, as it will determine the overall capability/performance of a model, and will have a big impact on the practical utility of a model. An overview of the input requirements for the models is presented in Table 3.2 for the various steps involved in modelling.

To set up a model of an irrigation canal, generally two types of information are essential, i.e. topographic/geometric and hydraulic data. The first is required to define the irrigation network, while the second is needed to quantify the water flows in the canals. Additional hydraulic data sets are needed to calibrate and validate the models.

Table 3.2. Input data requirement for different models.

Data Requirements	SIC-I	SIC-II	S3
Setting up Models			
Topographic Information/Survey	Detailed	Semi-Detailed[3]	No
Type and Actual Dimension of Outlet	All	All	All
Water levels in Distributary at Outlets	No	No	All
Actual Discharges of Outlets	No	No	Yes
Calibration of Models			
Discharges of Outlets/Distributary	Few Locations	All Locations	Few Locations
Water Levels	Few Locations	No	No
Validation of Models			
Discharges of outlets/distributary	Few Locations	Few Locations	Few Locations
Water levels	Few Locations	No	No

[3] Few cross-sections with reference to design crest level of outlet and drop structure (up- and down-stream)

As described in the previous sections, the process of setting up the model is quite different for the S3 model as compared to setting up SIC. S3 is based directly on the measurement of water levels at the off-taking outlets, whereas, in SIC, the water levels are required only at the stage of calibration and validation. The S3 model requires much less topographic information, as it is structured around the water levels in the canal at the time of an inflow-outflow test.

SIC-II requires the same topographic information as SIC-I, but they differ in the way the data is collected. In the case of SIC-II, the topographic survey is more limited than in case of SIC-I, which is not surprising, as the approach of SIC-II was especially developed to minimise the topographic survey efforts. Minimising the topographic survey is possible because it was demonstrated that the water distribution in secondary canals is not much affected by the canal topography (Visser, 1996). In this approach, geometric information of the canal is based on the design crest level of outlets and drop structures. Cross-sections are determined up- and downstream of each drop structure, and near few outlets along the canal with reference to these design levels, thus avoiding a time-consuming benchmark survey.

In conclusion, the S3 model depends entirely on the quality of the field data and representativeness of discharge measurements at one point in time, whereas, the SIC model can claim some wider validity (non-uniqueness) by using more permanent parameters and hydraulic laws.

Implementation aspects

In this section, the time and skills required to implement the models are evaluated. The implementation relates to aspects of field data collection, hardware and computational requirements.

Time requirement

The time requirements for collecting field data to set up the model, is summarised in Table 3.3.

Table: 3.3. Time required to collect information from the field.

Type of information	Extent surveyed by two persons in a day
Benchmark Survey	3-3.5 km
Hydraulic Survey	20-25 outlets
Calibration of Drop Structure	2 structures
Calibration of Outlets	6-8 outlets
Inflow-outflow Test	30 outlets

Two experienced field staff, performing a benchmark survey, can cover approximately 3-3.5 km per day, using a levelling instrument (Dumpy Level) with the range of about 100 m (300-350 feet). The hydraulic survey, i.e. determining the location, dimensions and type of the outlets, can be conducted for 20-25 outlets per day, depending on their condition. The calibration of outlets and drop structures is a rather more time-consuming task, and requires more precision. Around 6 - 8 outlets and 1 - 2 drop structures can be calibrated by a team of two field staff within a day, depending on the flow conditions. Once the outlets have already been calibrated, then at least 30 outlets can be monitored by the team of two field staff during an Inflow-outflow test to estimate seepage and distribution losses. In case of distributaries with a larger number of outlets, a larger field team will be required.

Once the data collected through the field survey is checked and converted in digital form, the SIC model (for both approaches) can be developed for 7-8 km per day. This is much easier in case of the S3 model, which requires only 1-2 hours for the same length of canal.

In summary, it can be estimated that for the Masood and Fordwah Distributaries, the following amounts of time are required to collect the field data and to set up the model on the computer:

- SIC-I : 10 days (Masood); 30 days (Fordwah)
- SIC-II : 6 days (Masood); 20 days (Fordwah)
- S3 : 5 days (Masood); 18 days (Fordwah)

Another important parameter, while looking at time requirement, is the computational/simulation time required. As the SIC model is based on complex equations and routines, more time is required to run a specific scenario for a distributary as compared to the S3 Model. In addition, SIC requires a computer with a high speed processor, requiring a high capital investment in hardware. The S3 Model is a spreadsheet, whereby changing the value of the head discharge in a cell, the discharges at the head of all the watercourses, are computed within almost no time. For SIC, the computation time differs from steady state to unsteady state simulation, depending on the length of the canal and the number of outlets. In the case of Fordwah Distributary, SIC (both approaches) takes around 10 minutes for steady state, and 4 hours to simulate the water levels and discharges for three days of flow, with 15 iterations and a time step of 10 minutes under unsteady state calculations. These calculations were carried out on a Pentium PC with 16 Mb RAM and 133 MHz Processor. Increasing the time step in an unsteady state situation reduces the time requirements dramatically. In the case of Fordwah Distributary, the model took only 15 minutes with the time step of one hour for the same three-day calculation. However, increasing the time step has a big impact on the numerical stability of the model, and thus, on the discharge estimation results. Malaterre (1994) advises to take a time step equal to, or less than 10 minutes. This result was confirmed for the study area by Kuper (1997).

A last remark related to costs; the costs for field data collection are directly related to the time requirement, which means that the S3 model, SIC-II and SIC-I are increasingly expensive to implement. In addition, SIC is a commercial product at a cost of about 15,000 US $, and protected with a hardware lock.

Skill Required to Setting Up These Models

These models also differ in terms of skills and knowledge of the system required. To develop SIC-II model, more knowledge of the irrigation system is required in order to understand and use a more limited set of data in an effective way. When a complete and precise set of information is available, it is less difficult for an engineer to set up a complete SIC-I model and use it for the evaluation of various interventions. Using the Simplified Steady State model approach is relatively straightforward and less time consuming. However, the interpretation of results requires a thorough hydraulic knowledge of the system; an important emphasis, as there is a danger that the S3 model is applied by professionals with relatively little hydraulic skills, due to the lower hydraulic and informatic barrier.

3.5.2. Comparison of Applications

A comparison of applications refers to the potential use of models to address a specific objective. The models under consideration differ from each other in their scope of application, as presented in Table 3.4.

Table 3.4. Comparison of application of different models.

Applications	SIC-I	SIC-II	S3
Water Distribution	Yes	Yes	Yes
Canal Regulation	Yes	Yes	No
Remodelling of the Outlet Structure	Yes	Yes	No
Desiltation of the Channel	Yes	No	No

The various applications of SIC-I, around the world, have shown that the model can be used:

1. To estimate the water supply to tertiary outlets based on head discharge of a distributary (water distribution);
2. To analyse the propagation of a perturbation and assess the impact of a gate manipulation on the water levels and discharges; and
3. To assess the impact of maintenance activities by:
    - Desiltation of the channel; and
    - Remodelling of outlet structures.

SIC-II is developed with only a rudimentary knowledge of the geometry of the canal, which implies that the model cannot be used to assess the impact of a change in the geometry, provoked by, e.g. maintenance, on the water distribution. The S3 Model is developed around the water levels and discharges of the current system and its use for a change in infrastructure would require a new model, based on a new set of field data.

The analysis of Chapter 3 has dealt with defining the three models that are evaluated in this study, the hydraulic unsteady state model SIC, used with a full set of field data (SIC-I) and a simplified set of field data (SIC-II), as well as the Simplified Steady State S3 Modell. In addition, the field data requirements of the models were compared showing that 30, 20 and 18 days were required, respectively, to set up each of these three models for the Fordwah Distributary. The main difference in field data requirements between the models is the detail with which the topography and geometry of the canal is determined. In terms of computational time, the S3 model is far more rapid in use than the SIC model, as the latter is based on the iterative discretisation of a fairly complex algorithm. An assessment of the range of applications of the models shows that SIC-I can be used for a wide variety of studies, i.e. water distribution, canal regulation, remodelling of the outlet structures, desiltation of the channel. SIC-II cannot be used for studies related to canal geometry (e.g. desiltation), whereas, S3 can be used for studies on the water distribution with fixed outlet structures only.

4. MODEL DEVELOPMENT

4.1. SIC-I MODEL

4.1.1. Setting up the Model

A topographic file representing the canal layout was prepared for Masood and Fordwah Distributaries[4]. In order to define the actual geometry of the canal, a detailed topographic survey (benchmark survey) of the distributaries was performed, and cross-sections were measured near outlet and drop structures. This cross sectional information was used for the computation of canal geometry and the development of the longitudinal profile of the canal.

Once the topography and geometry of the Fordwah and Masood Distributaries were developed, these files were further elaborated in the steady state module of SIC, adding data related to the types of outlets and their dimensions, flow conditions and discharge coefficients of drop structure, based on field measurements (Tareen *et al*, 1996). For the modelling of OFRB's, there is no separate option available under SIC, so they are entered as AOSM outlets. In order to compensate for the difference between the hydraulic behaviour of OFRB and AOSM, discharge coefficients of 0.53 and 0.79 have been used for most of the free flow OFRB and AOSM, respectively (Hart, 1996). In case of submerged flow conditions, the outlet discharge is a function of the downstream water level. Thus a user-defined rating curve was defined, based on a stage-discharge field measurement.

The reach-wise seepage losses for both the distributaries were estimated through Inflow-outflow tests, conducted as part of the water distribution study at the secondary level in the Chishtian Sub-division (Tareen *et al* 1996, Visser, 1996). The results are presented in Table 4.1.

Table 4.1. Seepage losses in Masood and Fordwah distributaries.

Reach (km)	Seepage Losses ($l\ s^{-1}\ km^{-1}$)
Masood Distributary	
0 - 4.42	1.92
4.42 - 7.62	-2.74
7.62 - 11.35	-9.06
Fordwah Distributary	
0 - 15.42	5.3
15.42 - 28.65	7.6
28.65 - 42.61	1.0

[4] The Fordwah and Masood Distributaries were modelled in two different studies by Hart (1996) and Visser (1996), respectively, using version 2.1 of SIC. The present study, which uses Version 3.0 of SIC, has used some of the field data of these studies. The topographic survey of the Fordwah and Masood Distributaries was done, for instance, in 1995. Additional information was collected, when necessary. Topographic survey of the Fordwah and Masood Distributaries was done in 1995

In Masood Distributary, the seepage losses are very low in the head reach, as it is running all the way along the Fordwah Branch. In the second and third reaches, there is even negative seepage (inflow) due to the positive backwater curve upstream of the gated cross regulator at RD 316 of Fordwah Branch. In the case of Fordwah Distributary, there is seepage in all the reaches.

4.1.2. Calibration of SIC-I

Once the information related to dimensions and characteristics of outlets and drop structure characteristics are entered, calibration of the model can start, using the steady state module of SIC. The calibration of a hydrodynamic model is a difficult and time-consuming task, and is perhaps the most difficult stage in using a model like SIC. The calibration of the Fordwah Distributary proved to be considerably more tedious due to the length of this canal, and the large number of outlets.

The discharge and water level measurements of all outlets during the Inflow-outflow tests were used for the calibration of SIC. The observed head discharges of 0.65 and 4.76 m^3 s^{-1} were adopted for the calibration of the Masood and Fordwah Distributaries, respectively. The discharge coefficients of the outlet structures and the roughness coefficient of the Manning-Strickler formula for the canal are refined during the calibration of the model to match them with the observed outlet discharges and water levels along the distributary. In case of the Fordwah Distributary, a few broken outlets were found during the field survey. The width of these outlets was adjusted to match with the actual discharge measured during Inflow-outflow test. A Manning-Strickler coefficient of 0.025-0.057 and 0.024-0.026 was found for the Masood and Fordwah Distributaries, respectively.

To compare the actual and simulated discharges, the percentage error in discharge estimation i.e. the ratio of the difference in simulated and actual supply to actual supply, was determined for all the individual outlets. The results are shown in Figures 4.1 and 4.2 for the Masood and Fordwah Distributaries. The deviation between computed and measured discharges supplied to the outlets goes up to 5 % in the Masood Distributary, with an average absolute error of 2%. For the Fordwah Distributary, this difference is less than 10 % for most outlets, as evidenced by the frequency distribution of percentage error shown in Figure 4.3. The simulated discharge of Jiwan Minor, and at the tail, are quite accurate with the percentage error of 3.28 and 1.49, respectively. The higher errors (more than 10 percent) are found in case of submerged outlets, such as 14710R and 135180R. Figure 4.2 also indicates that most of the problems in discharge estimation are at the tail reach. This implies that the small rough estimation of discharge estimation in the head / middle reach is causing/propagating high errors in the tail reach of the distributary (because of the length of the distributary). Another important factor, while looking at the percentage difference in discharge estimation one should keep in mind, is the magnitude of the value. A higher accuracy is more crucial for outlets with a high discharge rate such as Jiwan Minor (0.76 m^3/sec); because a little error in the prediction may lead to higher errors in the overall prediction. The average absolute error in discharge estimation is about 5.7 % for the Fordwah Distributary.

For the water distribution study, it is preferable to have a high accuracy in discharge supplied to outlets, whereas, the water level in the distributary level is less important. However, both depend on each other. The difference in actual and simulated water level in the distributary for each outlet is plotted in Figures 4.4 and 4.5 for the Masood and Fordwah Distributaries, respectively. The absolute error in actual and simulated water levels in the Masood and Fordwah Distributaries is around 1.18 and 7.5 cm respectively. The model estimates a rather low water level in the middle and tail reach of the Fordwah Distributary. The main reason for these inaccuracies is the fact that the topography and geometry of the Fordwah Distributary was determined in 1995, whereas, the Inflow-outflow test was conducted in 1996. In between, changes have occurred in the canal geometry. This hypothesis was confirmed during the validation process, where an Inflow-outflow test conducted in 1995, was used. The validation results turned out to be very accurate, as will be shown in the next section.

The calibration results for the Masood and Fordwah Distributaries can be considered acceptable, with average errors in the discharge estimation in the range of 2-5 %.

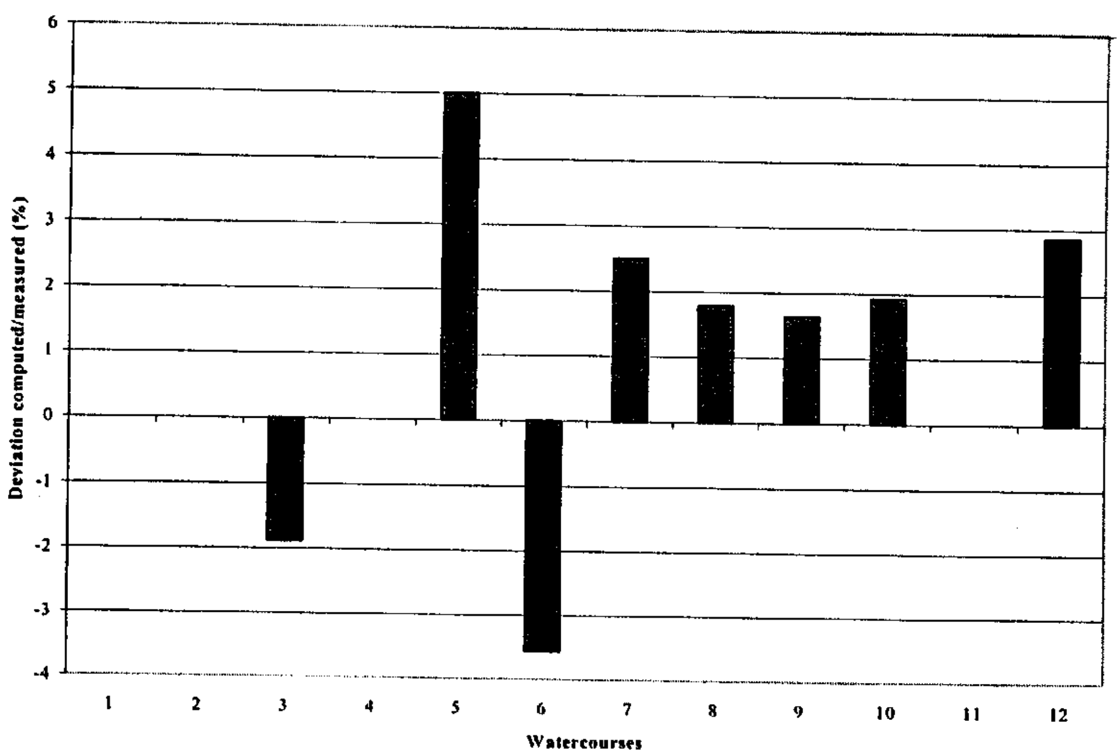


Figure 4.1. Deviation between computed discharge by SIC-I and actual measured discharge, Masood Distributary.

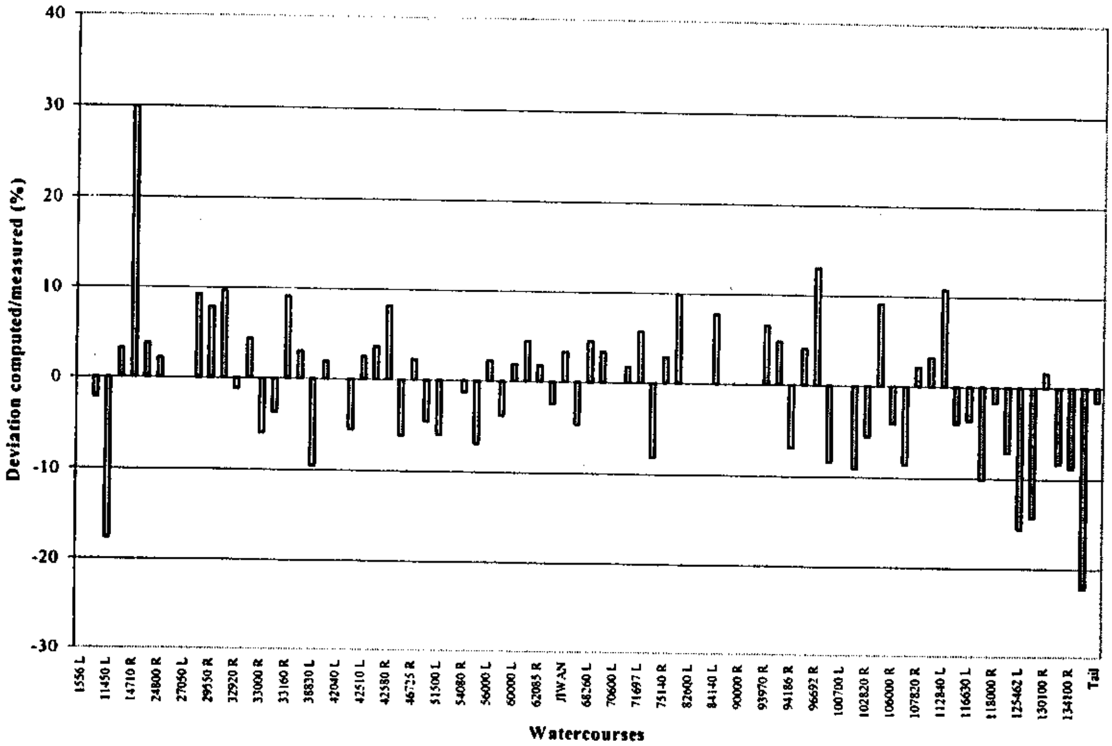


Figure 4.2. Deviation between computed discharge by SIC-I and actual measured discharge, Fordwah Distributary.

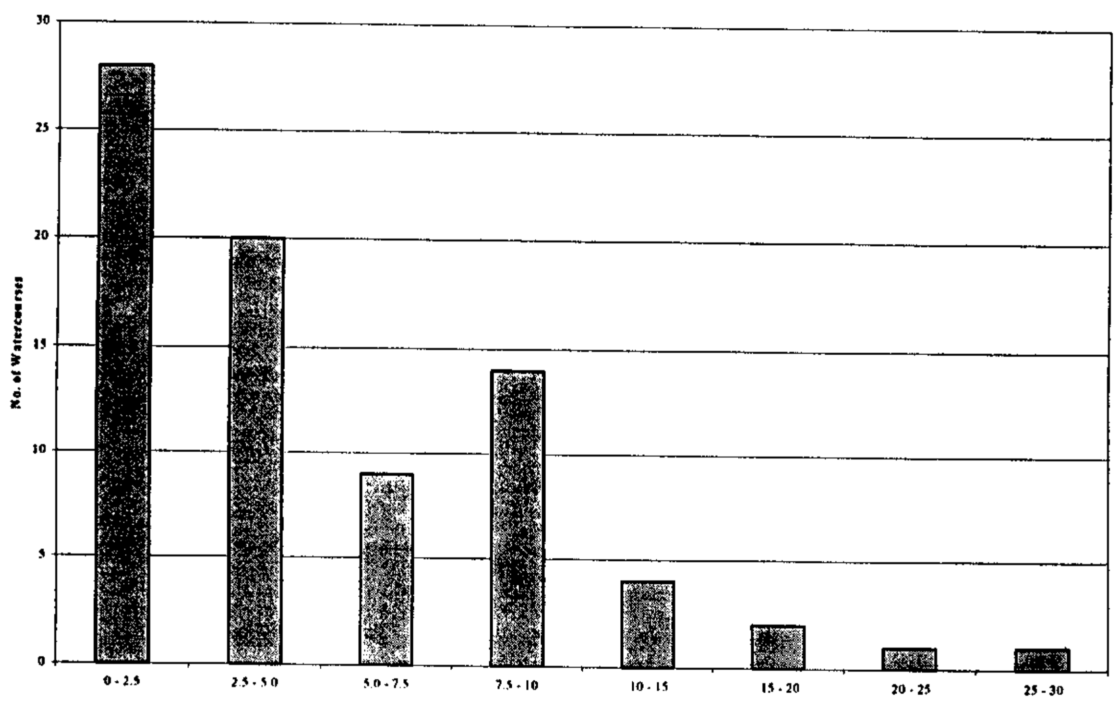


Figure 4.3. Frequency distribution of absolute error in discharge estimation, Fordwah Distributary.

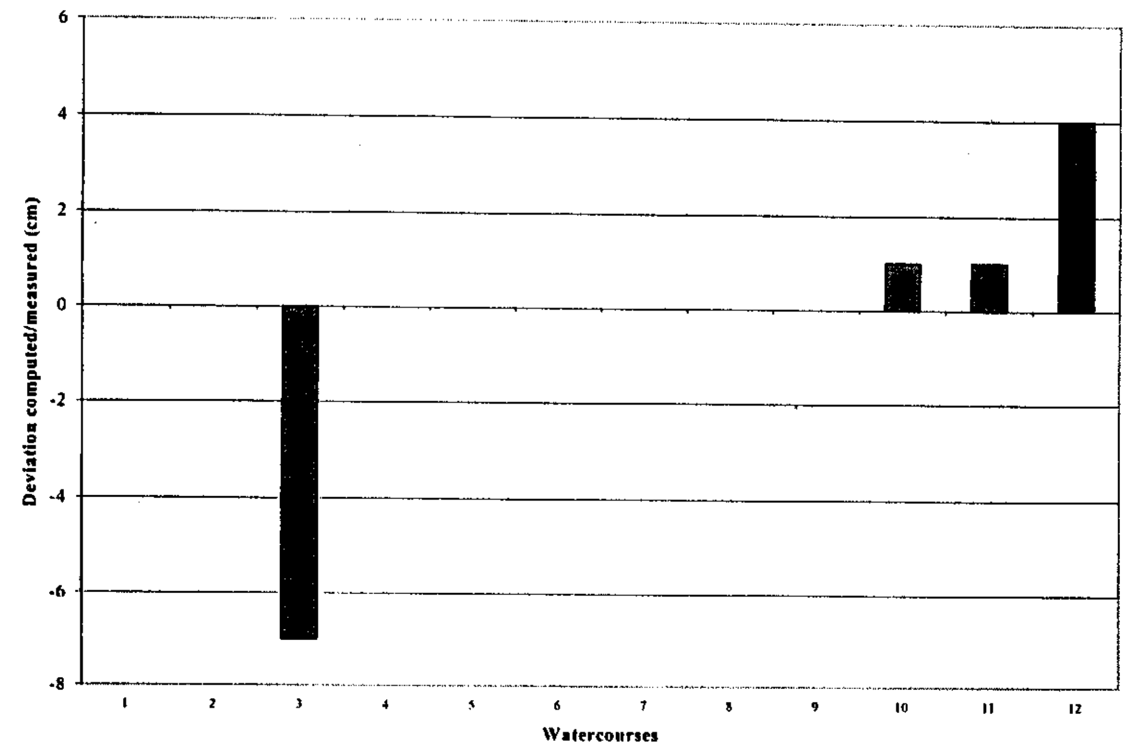


Figure 4.4. Deviation by SIC-I compared and actual measured water level, Masood Distributary.

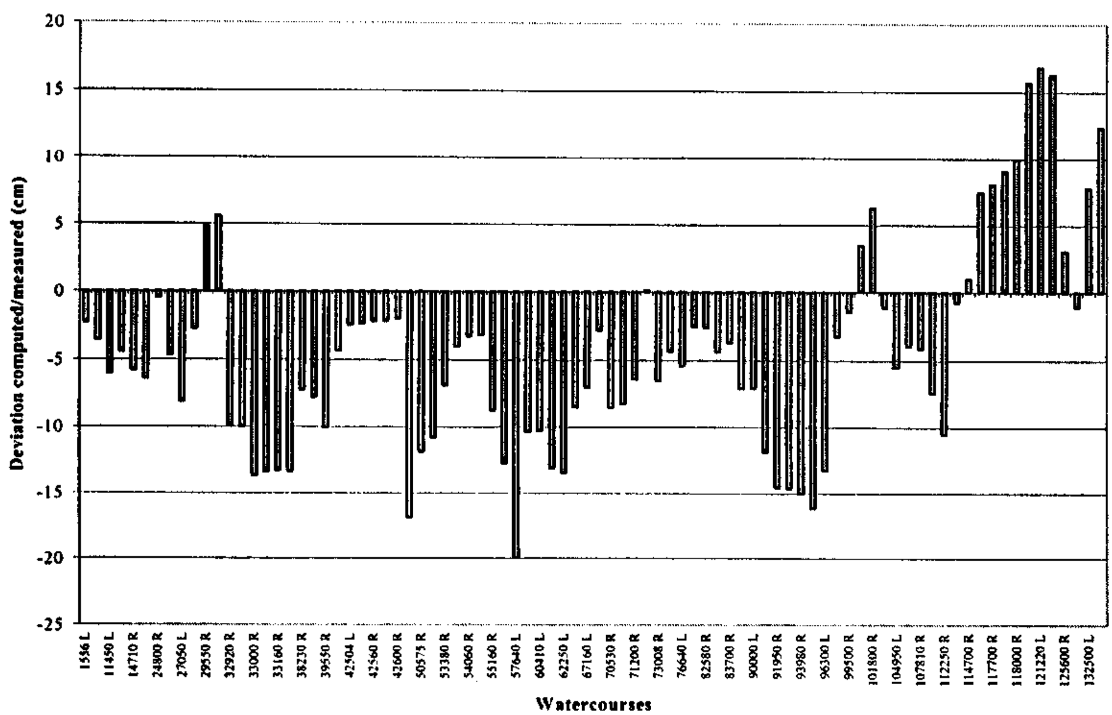


Figure 4.5. Deviation by SIC-I computed and actual measured water level, Fordwah Distributary.

4.1.3. Validation of SIC-I Models

Even though the driving algorithm of SIC has been validated by applying it to different canals around the world (Malaterre and Baume, 1994), a hydraulic model for a specific canal needs to be validated (after calibration) in order to be able to use the model for different discharges. In addition, the earthen canals in Pakistan experience some variation in their geometry, due to sediment deposition/removal. Testing the validity of a model for a different geometry, thus, extends the potential use of the model. In the case of the Masood Distributary, the model was tested for different discharges for the same geometry, while the model of the Fordwah Distributary was tested for a similar discharge, but a different geometry. The procedure of validation was slightly different for both canals, since the Masood Distributary was validated in steady state conditions, whereas, the Fordwah Distributary was validated in unsteady state conditions.

Masood Distributary

For Masood Distributary the model was validated under steady state conditions using the field collected data of Nov. 27 and Nov. 30, 1995. This was done by Visser (1996). The relevant information regarding the validation is summarised here:

Step 1

The model was run with the input data of Nov. 27, 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 were kept constant (validation 1). An observation is that the computed discharges supplied to the outlet structures are approximately 0.06 to 0.011 m^3/s too high, compared with the measured water levels converted into discharges. The computed discharges of the submerged pipe outlet (no. 5) and submerged OFRB (no. 6) especially, are not very precise. The computed discharges are more accurate if the downstream boundary conditions of the submerged outlet structures is set on the real measured downstream water levels in the watercourses (Validation 2). The remaining differences are due to higher computed water levels along the canal. The proposed adjustment is to use the computed seepage values of Nov. 27, 1995 (seepage instead of gain). This represents Validation 3. In Validation 4, the measured downstream water level of the submerged outlet structures will be simulated, adopting the measured seepage losses of Nov. 27, 1995. In Table 4.2, the results of the 4 different validations are listed.

Table 4.2. Validation results of Masood Distributary: Nov. 27, 1995.

Outlet No.	Validation 1	Validation 2	Validation 3	Validation 4	Measured 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; all parameters are equal to the values determined during calibration*

Validation 2: *Measured downstream water level as a downstream boundary condition for the submerged outlet structures 4, 5 and 6.*

Validation 3: *Seepage as computed for Nov. 27, 1995*

Validation 4: *Seepage as computed for Nov. 27, 1995; measured downstream water level for the downstream boundary condition for the submerged outlet structures 4, 5 and 6.*

Step 2

The model is run using the input data of Nov. 30, 1995, i.e. a constant inflow discharge of 0.51 m^3/s (18.13 csf) at the head of Masood Distributary. The different measured discharges along the canal, and measured water levels upstream of the outlet structures, were then compared with the computed output of the model. Based on the conclusions of the Nov. 27, 1995 validation, the input parameters were identical to those of Validation 4.

Table 4.3 presents the results of the comparison of measured water levels (measured above the crest) and computed water levels. Table 4.4 presents the validation results of discharges along the canal.

Table 4.3. Validation results of Masood Distributary: Nov. 30, 1995 (water levels above the crest).

Outlet No.	Computed Hu	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 4.4. Validation results of Masood Distributary: Nov. 30, 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 correctly. Thus, concluding that the validation of the SIC model of Masood Distributary has been successful.

Fordwah Distributary

Whereas the discharge of the Masood Distributary is relatively constant, the discharge at the head of the Fordwah Distributary is highly variable, related to its position at the tail of the Fordwah Branch, and the fact that the rotation in the sub-division is based, mainly, on the four larger distributaries (Kuper *et al* 1997). To use the model with varying head discharges, it was attempted to validate the model in unsteady state conditions. A second reason for validating the model in unsteady state conditions is the length of this distributary, which implies a relatively important time lag (estimated in the order of 12-15 hours). During the validation exercise[5], the water levels upstream of the cross structure at RD 15 were measured every half-hour and converted into discharges with the help of a stage-discharge relationship determined in the field. This was used as the inflow for the model. The discharge at the head of the Fordwah Distributary is difficult to establish, due to varying flow conditions.

The water levels were collected in the field every hour at three drop structures along the Fordwah Distributary for two days, i.e. RD 15, RD 33 and RD 65. These drop structures were calibrated, allowing to establish stage-discharge relationships. In addition, discharge measurements were taken four times during the measurement period at RD 107 with the help of a current meter. Finally, the water levels were measured at the tail of the distributary and converted into discharges through a stage-discharge relationship.

The actual measurements of discharges and water levels were compared with the results that the model produced with the inflow file over these two days. The comparison was made for the second day only, to account for the time lag in this distributary. The results are shown in Figures 4.6 and 4.7. The average temporal absolute error of measured and computed discharges and water levels is presented in Table 4.5. The obtained results are within the same range as found during model calibration, so that model can be considered well validated.

[5] The field data collected were originally used for the calibration of SIC of the Fordwah Distributary by Hart (1996). In the present study, SIC was calibrated with data collected in 1996 and validated with the data collected by Hart (1996).

Table 4.5. Temporal difference between simulated and actual discharge, and water level at various reaches of Fordwah Distributary.

Absolute Error	RD 33	RD 65	RD 107
Discharge Difference (percent)	5.4	3.3	8.4
Water Level Difference (cm)	1.8	3.2	2.7

The model simulations during the calibration and validation phases showed that the model does not function below a minimum head discharge of 4.5 $m^3 s^{-1}$. This is related to the computational difficulties in calculating the water profiles under dry tail conditions, an issue that has been raised before in relation to the use of SIC (e.g. Hart, 1996), and constitutes one of the limitations of SIC in the context of secondary irrigation canals in Pakistan.

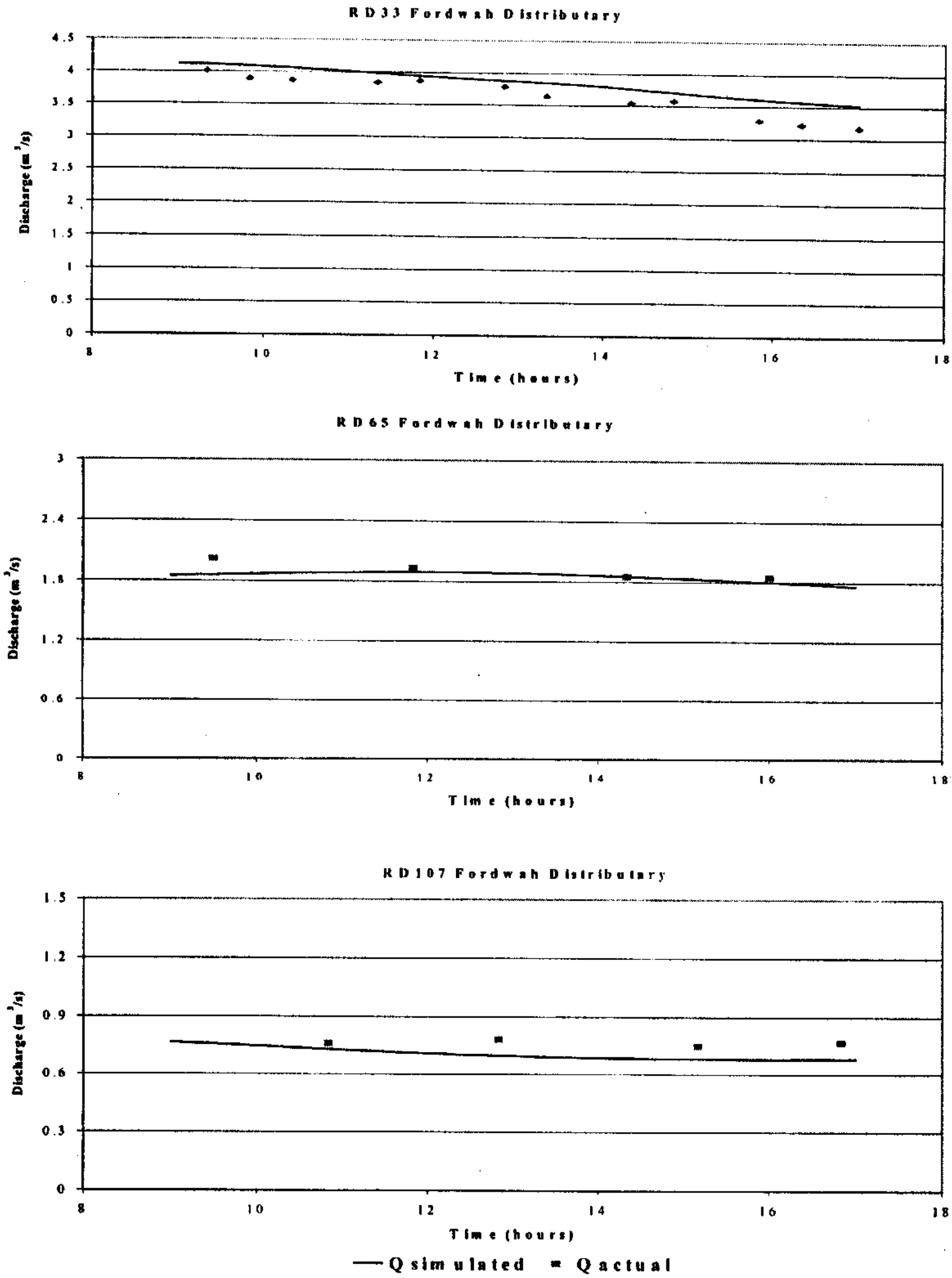


Figure 4.6. Comparison of measured and computed discharge by SIC-I at RD 33, RD 65 and RD 107 of Fordwah Distributary.

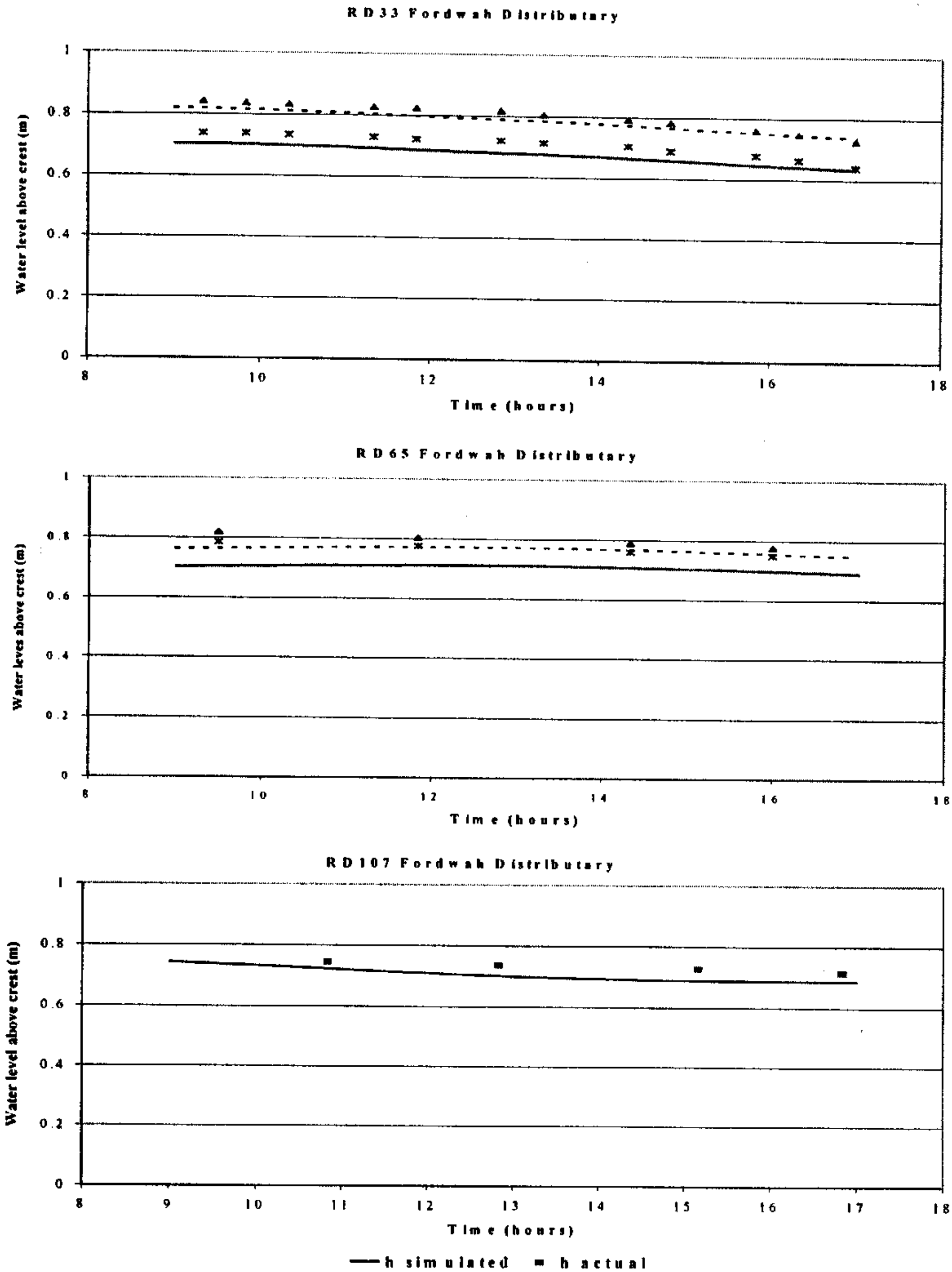


Figure 4.7. Comparison of measured and computed water level by SIC-I at RD 33, RD 65 and RD 107 of Fordwah Distributary.

4.2. SIC-II MODEL

4.2.1. Setting up the Model

Based on a sensitivity analysis of the different input parameters, a simplified data set was proposed by Visser (1996). Actual collection of field information was only required for «sensitive » parameters, i.e. those parameters that have a big impact on the water distribution. Other parameters, such as the canal topography and geometry, were approximated, by taking design values of crest settings of outlets and canal bed levels for the *canal topography*, and only a few cross-sections for determining the *geometry* in lieu of a detailed field survey. The design information related to these distributaries was collected from the Punjab Irrigation Department (Tareen *et al* 1996). The input data collection is illustrated for the Fordwah Distributary in Table 4.6.

Table 4.6. Illustration of the input requirements for SIC-I and SIC-II for the Fordwah Distributary.

	SIC-I	SIC-II
Topography		
• Bed level canal	Benchmark survey	With reference to design crest levels structures
Geometry		
• Cross sections	80	24
• Crest levels structures	Field survey	Design values

The example of Table 4.6 shows the reduction in field-collected information, particularly related to the determination of benchmarks. The actual dimensions of outlet and drop structures was collected through a field survey, as these are sensitive parameters. In case of submerged outlets, the downstream rating was defined by the actual downstream water level of the outlet structure with reference to its design crest. The seepage rate for the various reaches of the distributary was calculated and used in SIC-II, based on the Inflow-outflow test (Table 4.1). The design discharge coefficient was used for the different outlets, based also on the work of Tareen *et al.* (1996), who showed that the actual coefficients, Cd, were not much different from the design values. Manning's coefficient was taken to be 0.026 (average for earthen canal in Punjab).

4.2.2. Calibration of SIC-II

For the calibration of the SIC-II model under the steady state, the head discharge of the Inflow-outflow tests were used, 0.65 and 4.76 m^3/sec for Masood and Fordwah Distributaries, respectively. The models for these distributaries are calibrated by means of adjusting/refining the Manning's coefficient, and discharge coefficients of outlet structure in order to match the computed outlet discharges with the measured discharges. In case of broken outlets in Fordwah Distributary, the width of the outlets is adjusted to match the actual discharges. The deviation between actual and simulated discharges for each outlet is estimated and presented in Figure 4.8 and Figure 4.9. Figure 4.8 shows that the deviation of the simulated discharge from the actual discharge for the Masood

Distributary goes up to 7 %. The overall average absolute error in the estimation is 1.9%. For the Fordwah Distributary, the frequency distribution of the error in estimation is depicted in Figure 4.9, which shows that most of the outlets have a deviation of about 5 %. The average absolute error in case of Fordwah Distributary is 7.3 %. The higher errors are found in pipe and submerged outlets, such as 100700L and 104950L. Similar to SIC-I, the percentage error is much higher at the tail reach of the distributary. This reveals that a small error in the estimation of discharge at the upper reach results in an aggravated error in the tail reach of the distributary, showing that the length of the distributary and the number of outlets are important factors. The deviation in discharge estimation at two important locations, i.e. Jiwan Minor and the tail of the distributary is 1.83% and 0%, respectively. The calibration results that are obtained are slightly less favourable than those obtained for SIC-I, but are acceptable as they remain in the range of 5-7 %.

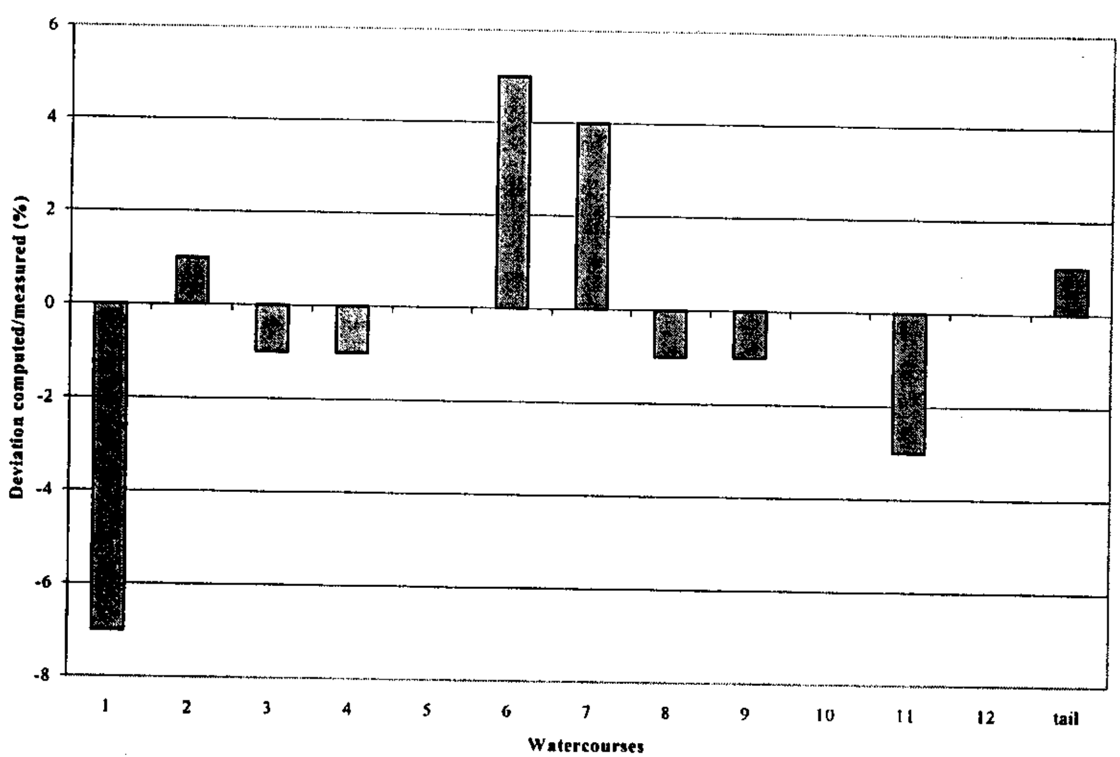


Figure 4.8. Deviation between computed discharges by SIC-II and actual measured discharges, Masood Distributary.

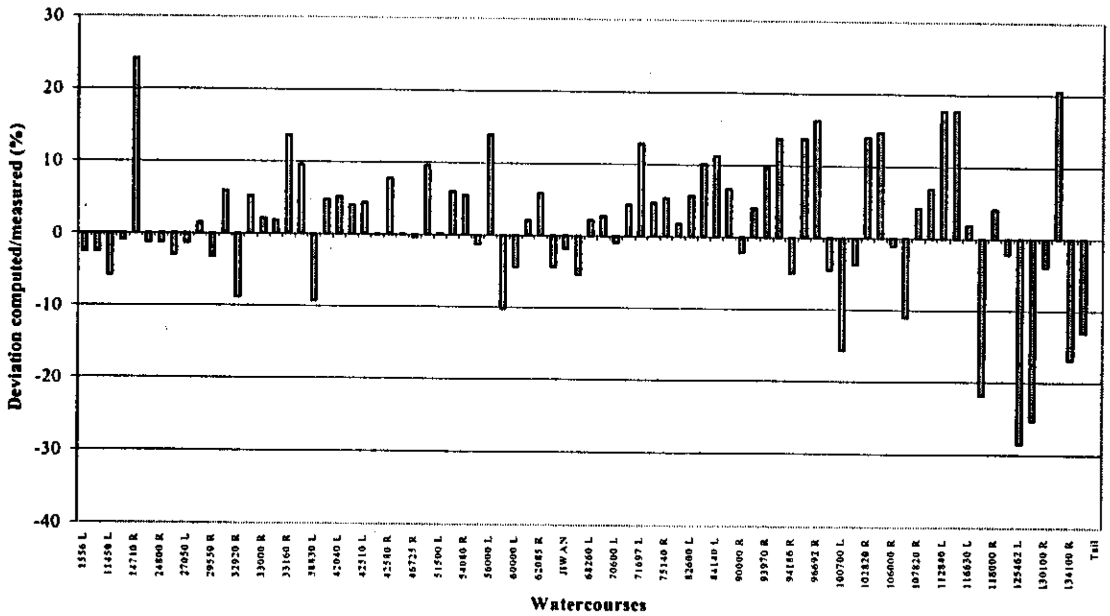


Figure 4.9. Deviation between computed discharges by SIC-II and actual measured discharges, Fordwah Distributary.

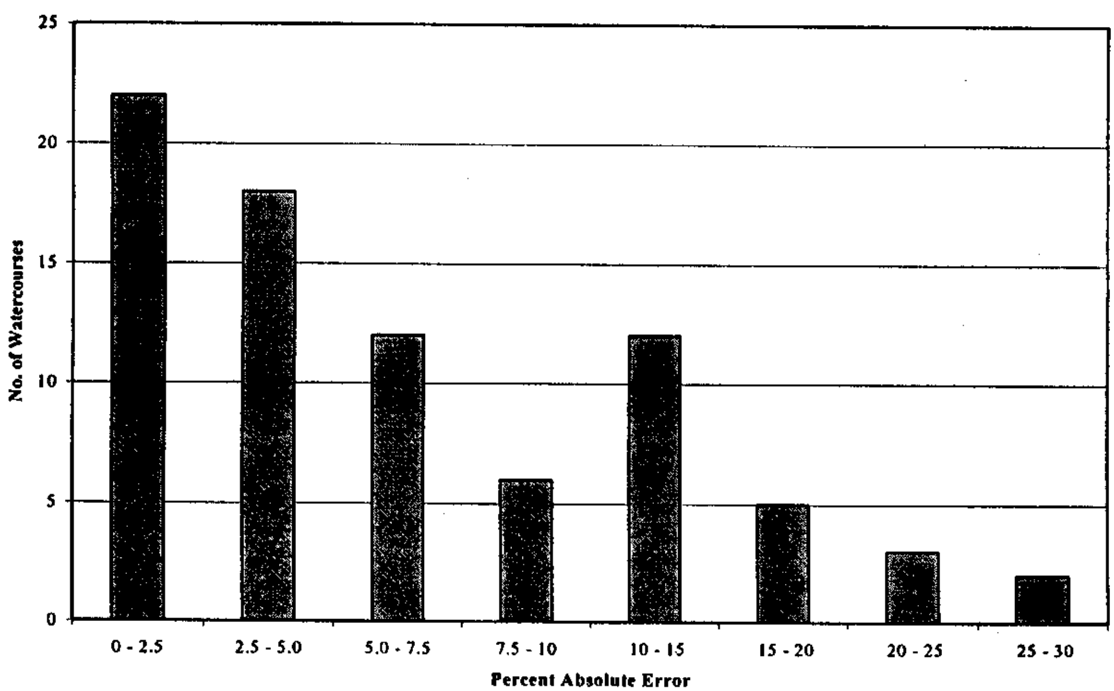


Figure 4.10. Frequency distribution of absolute error in discharge estimation by SIC-II, Fordwah Distributary.

4.2.3. Validation of SIC-II

Like SIC-I, it is important to validate the SIC-II model for these distributaries. The validation procedure is similar to the one adopted for SIC-I. In the case of Masood Distributary SIC-II is calibrated in a steady state situation at three different discharges, i.e. 60%, 80% and 120% of the design discharge (1 m^3 s^{-1}). The lessons learnt during validation of SIC-I were directly applied here. Seepage losses were adjusted and the downstream water levels of submerged outlets were determined in the field. The outlet discharges estimated by this model were then compared with the actual measured discharges (see Figure 4.11). The deviation of computed discharges from actual, is about 10%, except for the first outlet, which shows aberrant results at low discharges. This has implications for the use of the model, as it is clear that the model cannot be used for discharges as low as 60 % of the design discharge.

The SIC-II model for Fordwah Distributary is validated using the same flow pattern under unsteady state condition as used for the validation of SIC-I. The computed discharge at three different points in the distributary, i.e. RD33, RD65 and RD107, were compared with actual measured discharges, as shown in Figure 4.12. The average absolute temporal error at these points in discharge estimation is 2.3 %, 8.3 % and 19.7 % respectively. The computed discharge of this model will, thus, vary up to 20 per cent from the actual measurements, which is quite a considerable difference, appearing quite clearly from the calibration and validation procedure that the approach adopted under SIC-II should be applied with great care in the case of long canals with a large number of outlets.

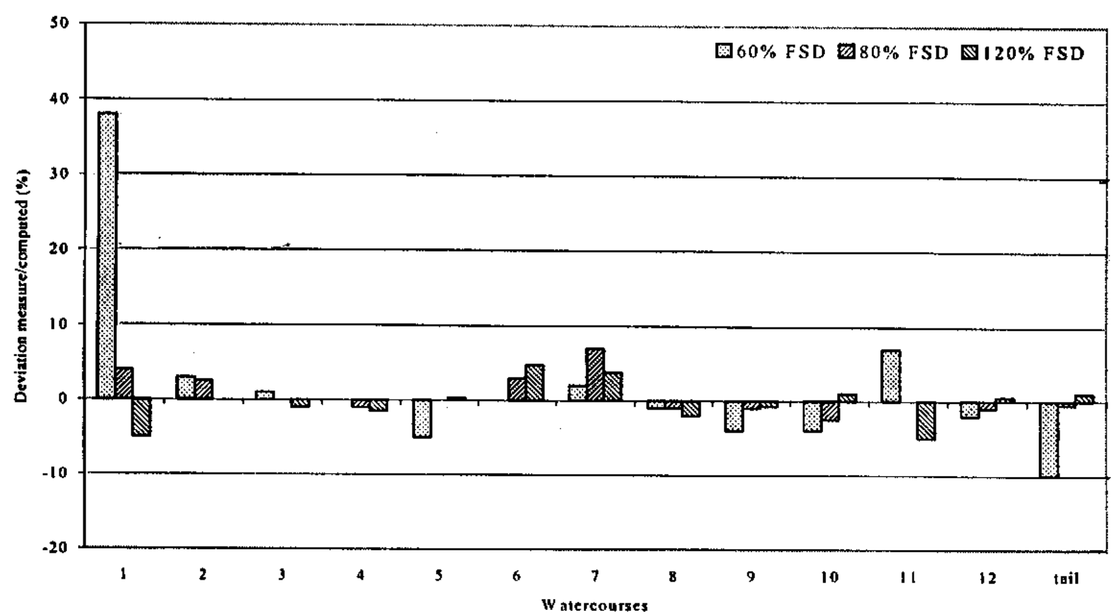


Figure 4.11. Validation of SIC-II for Masood Distributary.

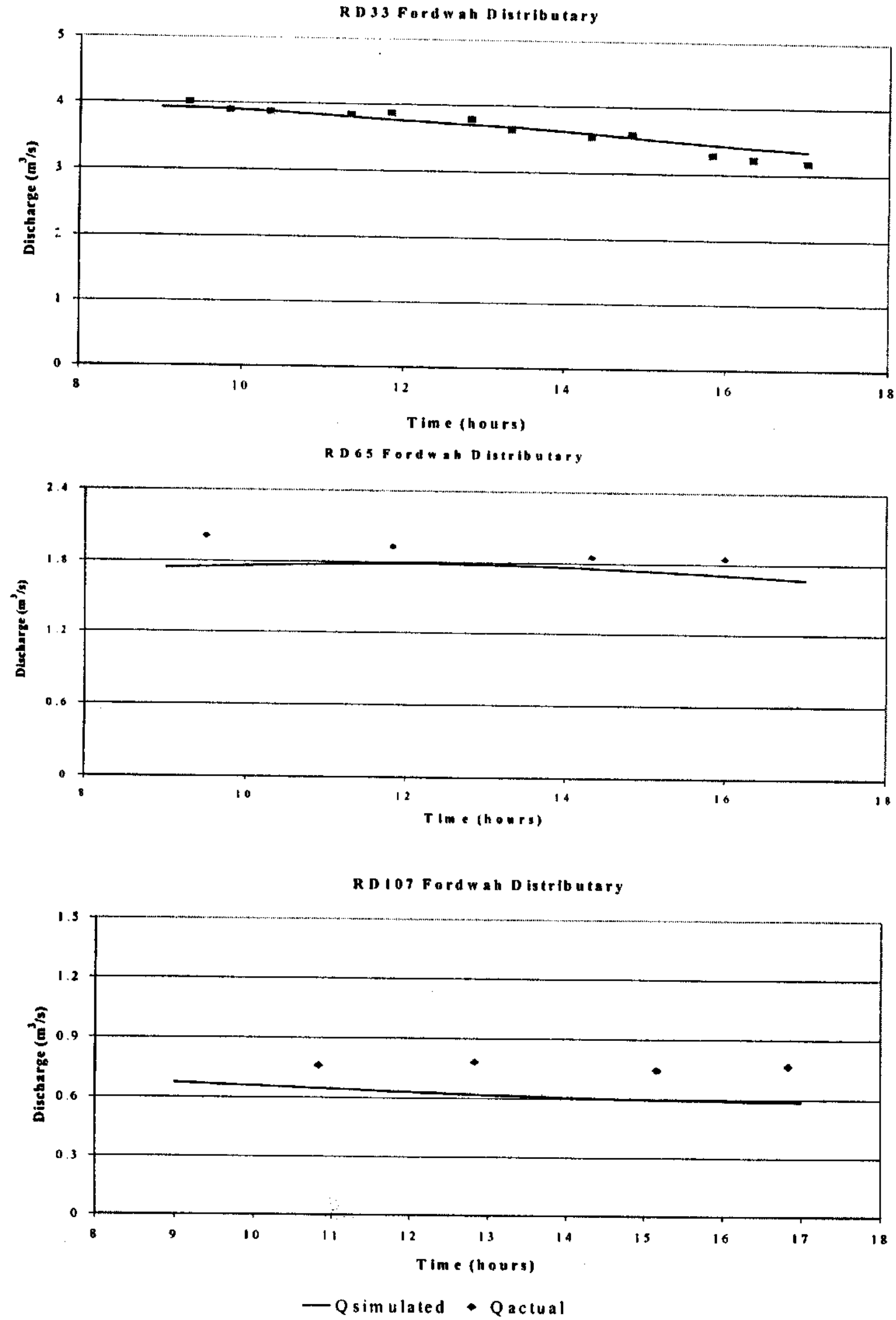


Figure 4.12. Comparison of measured and computed discharge by SIC-II at RD 33, RD 65 and RD 107, Fordwah Distributary.

4.3. SIMPLIFIED STEADY STATE (S3) MODEL

The methodology of setting up the S3 model has been described in section 3. There is no need for calibration of the model, since the field measurements of the outlet calibration exercise and inflow-outflow test are used to build the model. If the model is fed with the inflow of this data set, it will reproduce exactly the measured outlet discharges. However, it is necessary to validate the model at different inflows. For this purpose, all outlet discharges would have to be measured at a different inflow during a period of approximate steady state flow. Such data are not available for Fordwah Distributary, but they are for Masood Distributary.

4.3.1. Validation of S3

The validation of the S3 model was applied to data for the Masood Distributary collected by Visser (1996). Apart from the data of the inflow-outflow test (that are used to build the model), two more data set for different inflows at the head are available for Masood Distributary. This allows an assessment of the prediction by the model of the changes in the outlet discharges as a function of changes in the inflow at the head. In his work on the SIC model of Masood Distributary, Visser (1996) found that two modifications had to be made in the calibrated model to get a reasonable fit for the two validation data sets:

1) change the seepage losses in the model to reflect the observed transition from seepage inflow to seepage outflow, and 2) change the downstream boundary condition of submerged outlets according to the observations. Only the former has been done in the S3 model. The fluctuations in the seepage are due to the fact that Masood distributary runs parallel to Fordwah Branch Canal, and is influenced by its water level. In most other distributaries, this refinement will not be necessary. However, the problem of changing downstream conditions of submerged outlets is encountered in all distributaries. A model that claims general applicability cannot incorporate refinements during validation that cannot be reproduced when applied with other inflows, or to other canals. The S3 model has, therefore, not incorporated this refinement in the validation procedure.

In the two Figures below, the validation results are presented, plotting the increase of outlet discharges (both measured and model) with the data of Nov. 15, 1997 (inflow-outflow test) as reference. The first outlet was closed on all three days; the second outlet was closed on Nov. 27, 1995. The match between the measured values and the model predictions is reasonable for the free-flow outlets, but poor for the submerged outlets. The average absolute error between model and observations is 12.2% at 28 cfs inflow, and 30.7% at 18 cfs inflow.

In case of the higher inflow (28 cfs), the outlet discharges do not increase as much as the inflow because the outlets react sub-proportionally to the increase in discharge. Thus, the extra discharge is supplied to the tail of the distributary.

In case of the lower inflow (18 cfs), the second outlet shows an interesting increase of the supply instead of the expected decrease. This is due to the fact that its flow type changes from orifice (OFRB) to open flume. When the water surface in the outlet no longer touches the roofblock, contraction of the water jet ceases, and discharge increases abruptly (see Figure 2.4).

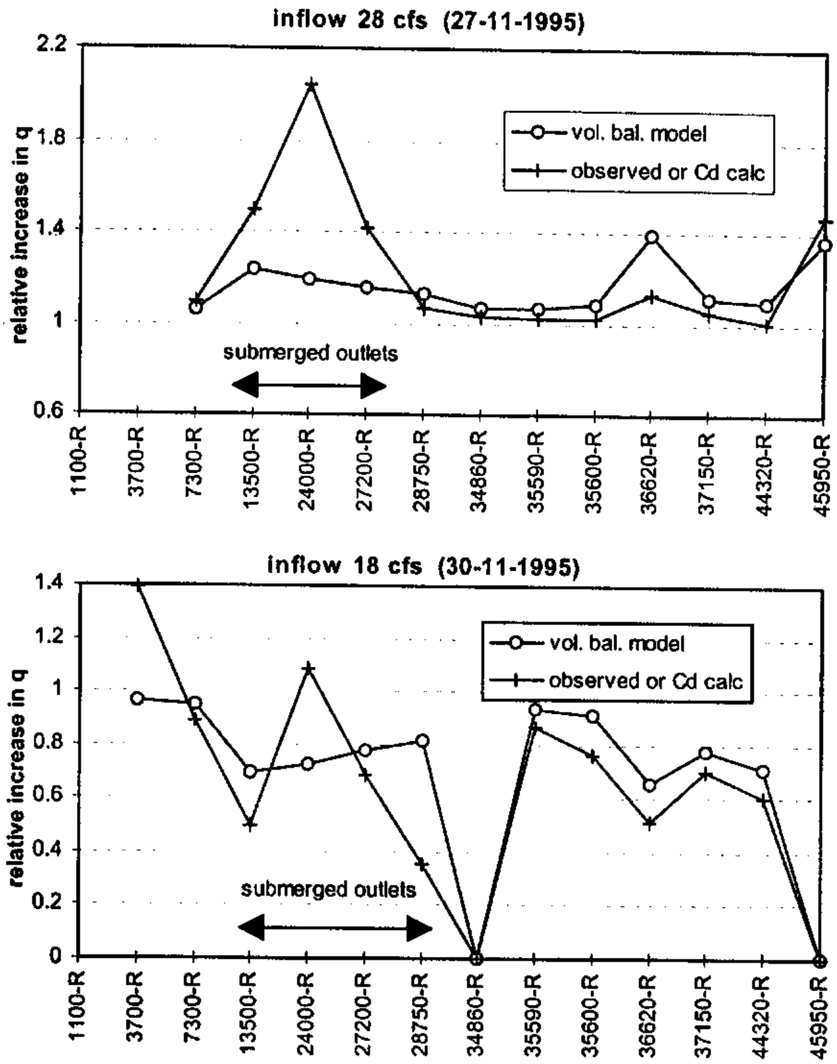


Figure 4.13. Validation of S3 model for Masood distributary.

Reference situation was measured on 1 Nov.15, 1995 (inflow 23 cfs).

A caveat about the 'measured' values; only the first data set of the inflow-outflow test was obtained by taking discharge measurements with current meter or cut-throat flume. Most of the outlet discharges in the other two data sets were obtained by measuring water levels and applying structure formulae with discharge coefficients, Cd (referred to in the legend of the graphs), derived from the first measured discharge.

It can be concluded that the S3 model is a simplification of reality that correctly predicts trends in the observed discharges, but cannot catch all the variations in the observations. The same applies to the SIC model; the next step is to compare these two tools.

5. COMPARISON OF SIC-I, SIC-II AND S3 MODEL RESULTS

5.1. STEADY STATE

To compare results of the S3 model of Fordwah distributary with both SIC models, three steady state scenarios were selected with inflows of 4.5, 5.0 and 5.5 m^3/s. Below 4.5 m^3/s, the SIC models face computational problems, though the tail only falls dry around an inflow of 3.7 m^3/s.

For this comparison, the results of the SIC-I model have been taken as a reference because of the use of field-measured data has been used for calibration and validation purposes. The Figures 5.1a and 5.2a present a direct comparison of outlet discharges. The match between S3 model and SIC-I is, of course, rather good (6% average absolute error), because both models have been calibrated with the same data set. Figure 5.2a shows that the match between SIC-II and SIC-I is less good (15% average absolute error). In the lower middle reach, the SIC-II model over-estimates the outlet discharges. This means that the water level in the SIC-II model is too high, or that the design crest levels used in the simplified model are all lower than the actual crest levels. In that case, the bed in the model will also be lower than actual, and thus, also the water level. Too much water reaches the tail in the SIC-II model.

Rather than comparing the predicted outlet discharges directly, it is more interesting to compare the way in which the outlet discharges change as a function of a change in inflow at the head of the distributary. In other words, comparing the sensitivities of the outlets. This has been done in Figures 5.1b and 5.1c for the S3 Model, and in Figures 5.2b and 5.2c for the SIC-II model. In the S3 model, the change in q of almost all the outlets is very near to the one predicted by the full SIC model. Near to the tail, the sensitivity of two outlets is too high and two outlets have too low a sensitivity (more than 10% difference). In the SIC-II model, the prediction of the sensitivities of almost all the outlets are also very close to the one of the SIC-I model, but five outlets are much too sensitive. Due to these over-sensitive outlets, the discharge at the tail is not changing as much as in the full SIC model.

Due to calibration with the same data set, the S3 model predicts almost the same outlet discharges as the SIC-I model, when both are run at an inflow that lies near the calibration inflow. The real test lies in the comparison of the predicted outlet sensitivities; the average absolute error is only about 2.8%. For the SIC-II model, there is clearly a lower accuracy of the predicted outlet discharges when run at the calibration inflow, but the prediction of the outlet sensitivities is still quite good at an average absolute error of 4.4%.

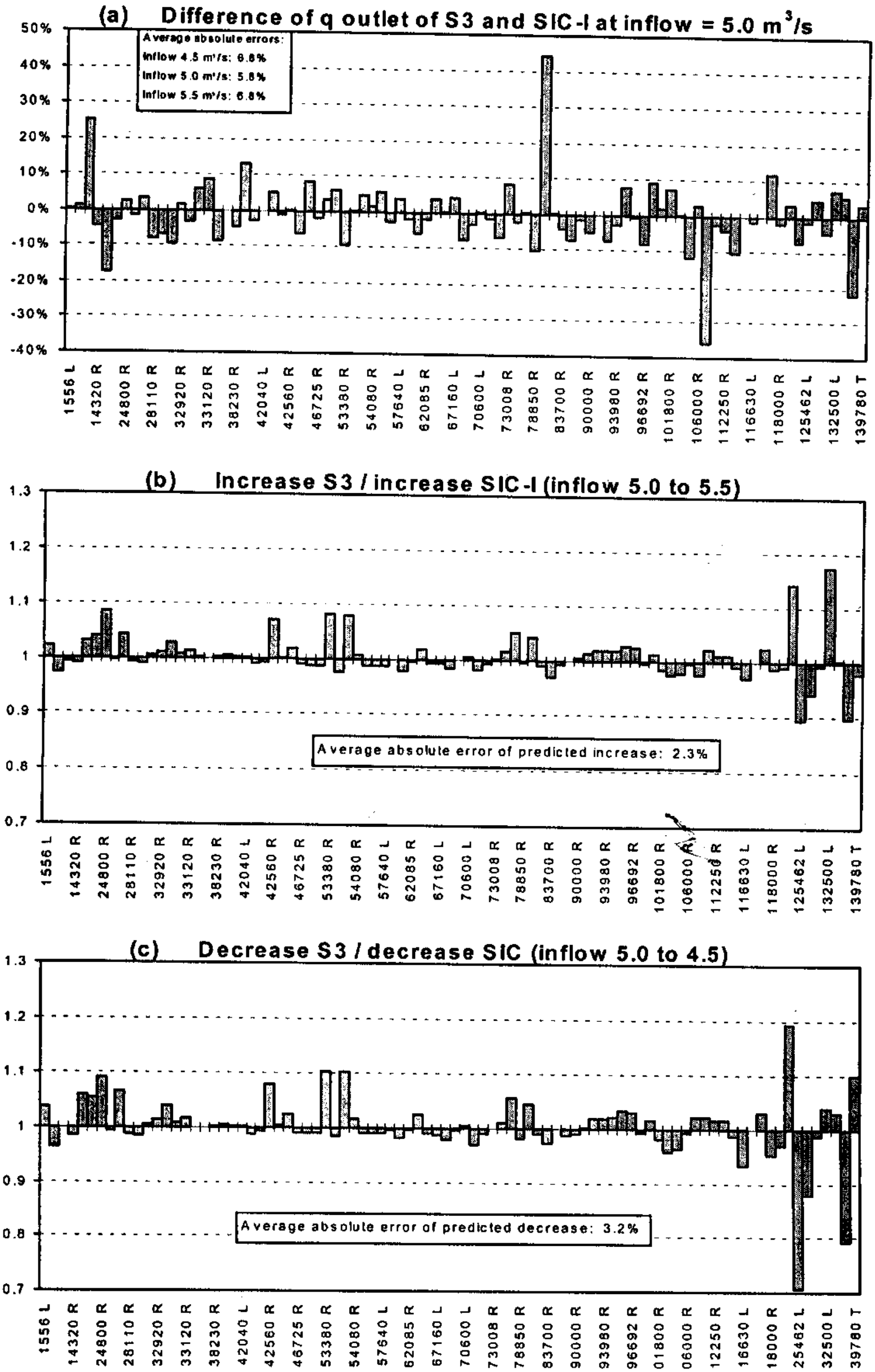


Figure 5.1. Comparison of S3 model and SIC-I Model.

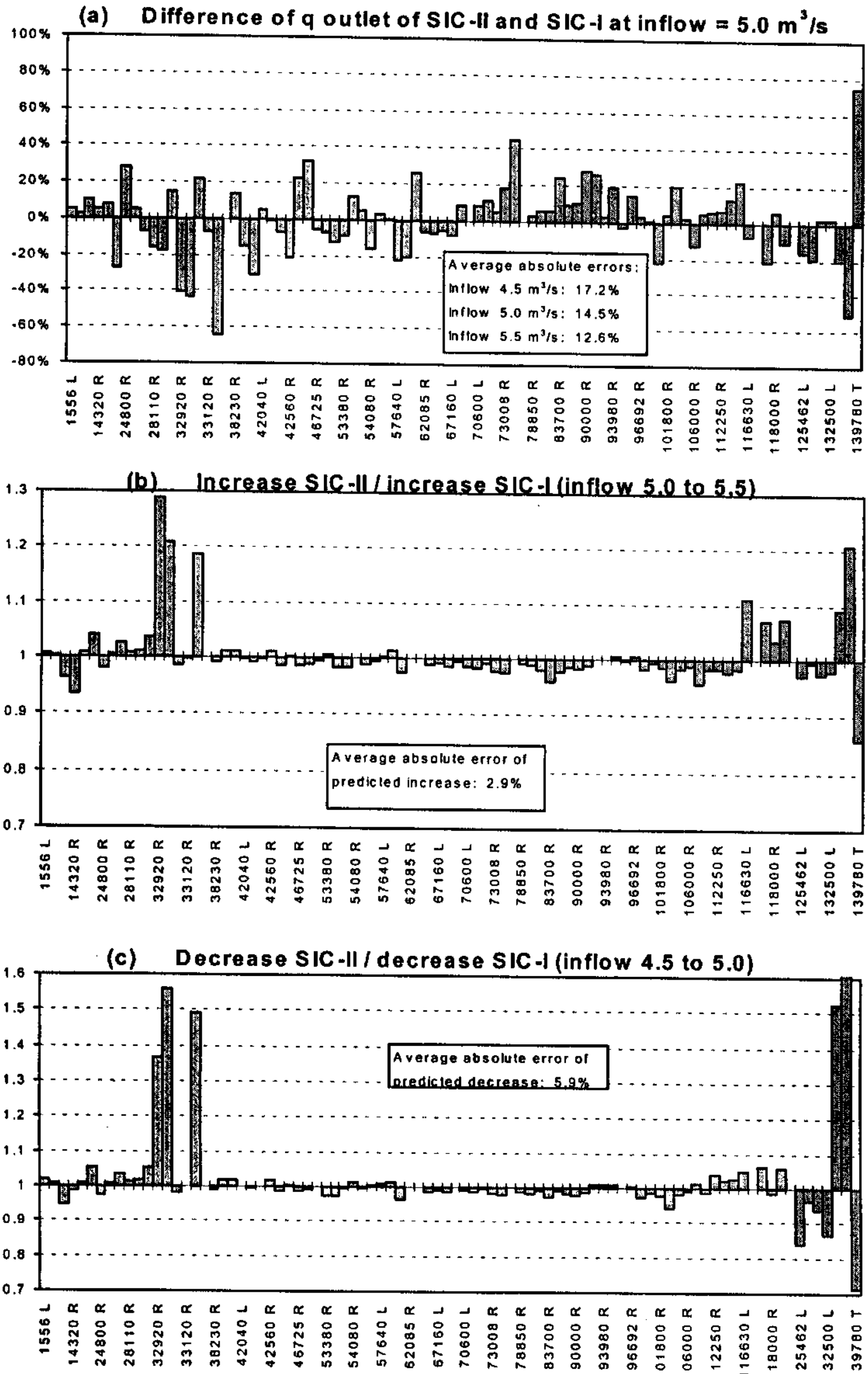


Figure 5.2. Comparison of SIC-II model and SIC-I model.

5.2. UNSTEADY STATE

In a typical application of the models, the observed daily inflow into a distributary during a certain period (e.g. a month) is used as input for the model to calculate the water distribution to the outlets. These results can then be summarised in a monthly average supply, and variability of supply for each outlet. For this purpose, unsteady state module of SIC can be used, but two remarks have to be made:

- To fully benefit from the high accuracy of SIC, inflow data at intervals less than 24 hours should be used; and
- An unsteady SIC simulation of one month takes about 48 hours on a Pentium PC.

The S3 model assumes that the distributary has steady state flow during 24 hours with the inflow for that day. To assess the inaccuracy that this assumption gives rise to, simulations of Fordwah Distributary were done with all three models, taking an inflow pattern of 3 days (see Figure 5.3a). The inflow starts at 4.5 m^3/s, increase to 5.5 m^3/s during 24 hours, then fall to the starting level again. The period is taken long enough to return to steady state after the wave passes, even at the tail.

In Figures 5.3b and 5.3c, the resulting water supplies to Jiwan Minor, and to the tail are plotted, and indicators are given for comparison. The result of the S3 model is, of course, a block wave, with a time lag of zero. SIC-I and SIC-II models produce gradual changes in supply with very similar time lags. The amplitude of the wave at the tail for SIC-II is smaller than for SIC-I, indicating that, on average, outlets have a higher sensitivity in SIC-II.

In spite of the fact that the hydrographs produced by the S3 model look quite different from those of the two SIC models, indicators for the three-days period are almost equal. The small differences in average supply are mainly due to differences that already existed in the steady state that is taken as starting point. The temporal variability of the S3 is higher than that of the SIC models, especially in case of the supply to the tail. This follows from the definition of the coefficient of variation, where deviations from the mean have a quadratic weight. In the S3 model, the maximum deviation is maintained 24 hours, whereas, in SIC the maximum deviation is maintained for a shorter time due to the fact that the supply wave becomes less steep towards the tail.

Run in its unsteady state mode, the SIC-II model predicts time-lag and wave attenuation very similar to the SIC-I model. The S3 model does not have this capability. If we are interested in water supply indicators over a longer period, and not in instantaneous values, the S3 model <u>can</u> be used to calculate water supply to outlets, as a function of a fluctuating inflow into the distributary. The average supply to outlets is equal to the one calculated with SIC. The temporal variability is somewhat higher than the SIC one (up to 23% at Fordwah tail), because supply fluctuations are not as smooth as in SIC.

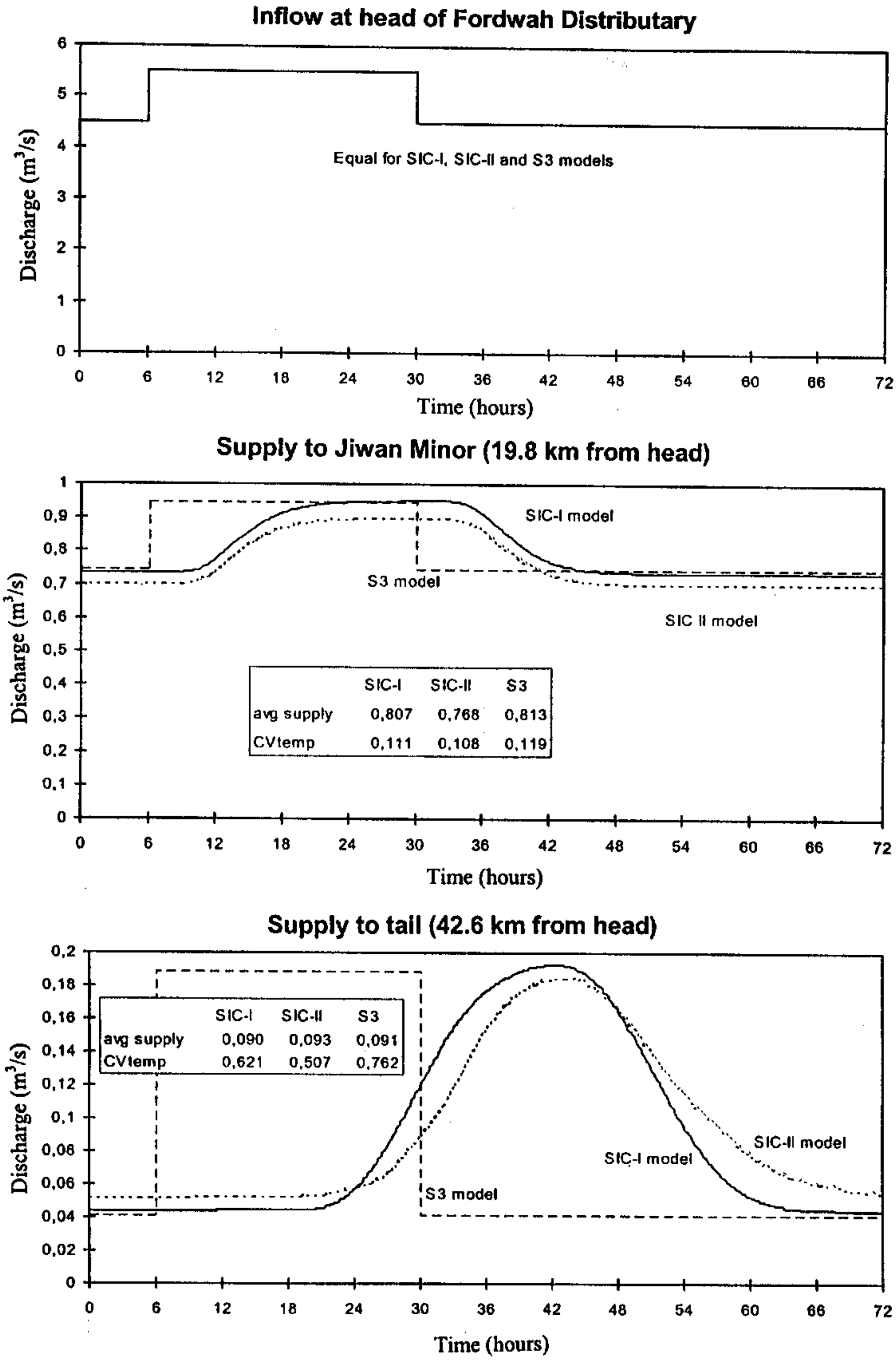


Figure 5.3. Comparison of unsteady state simulations

6. CONCLUSIONS AND RECOMMENDATIONS

A hydraulic unsteady state model, SIC - Simulation of Irrigation Canals, was developed for two secondary irrigation canals in south-east Punjab, Pakistan. Two ways of setting up the model were undertaken, one with a full data set (SIC-I), obtained from field measurements, and one with a limited data set (SIC-II), obtained from field measurements for « sensitive » parameters, and derived from secondary data sources for less sensitive parameters. A Simplified Steady State model - S3 -, based on the Manning-Strickler equation, was developed for the same canals. These models were compared in terms of data requirements, time and cost required to set up the model, and the application of the models to particular case studies related to canal water distribution. The following conclusions can be obtained from the comparison:

1. Setting up the three models require different *time and cost investments* for data collection. For the Fordwah Distributary, a 47 km long secondary canal with 87 tertiary outlets, 30, 20 and 18, respectively, days were required to set up SIC-I, SIC-II, and S3. The main difference in field data collection between the models is the detail with which the topography and geometry of the canal is determined. In terms of computational time, the S3 model is far more rapid in use than the SIC model, as the latter is based on the iterative discretisation of a fairly complex algorithm.

2. For studies of *canal water distribution with a constant inflow (steady state) to ungated, fixed outlet structures*, the S3 model shows a similar *degree of accuracy* as SIC-I, i.e . an absolute average error in the range of 2-5 % for the discharge estimation to tertiary outlets. SIC-II has a similar accuracy for the smaller Masood Distributary, but has an absolute average error of 7 % for the Fordwah Distributary. The errors due to a more limited field data set, are propagated by SIC to the tail of the canal, where errors in discharge estimation are considerable. In the case of S3, the errors are much less propagated, since the model is not entirely based on physical laws. The choice of a model is fairly straightforward in this case, as the S3 model is less time-consuming and produces equal results as compared to SIC.

3. For studies of *canal water distribution with a variable inflow (unsteady state) to ungated, fixed outlet structures*, SIC-I and SIC-II are shown to reproduce the time lag and wave attenuation of the water levels and discharges quite well. This is not reproduced by S3, where changes in the inflow are abruptly translated in changes in off-taking discharges. The choice of the model will depend here on the objectives of the study. If one is interested only in monthly volumes delivered to tertiary outlets, S3 may well suffice. In case one wants to know more about the discharge variability, or about the exact hydrographs, it will be necessary to employ SIC.

An assessment of the range of applications of the models shows that SIC-I can be used for a wide variety of studies, i.e. water distribution, canal regulation, remodelling of the outlet structures, desiltation of the channel. SIC-II can be used for the same purposes as SIC-I, when a somewhat lower accuracy is acceptable, except studies related to canal geometry (e.g. desiltation). Whereas, S3 can be used for studies on the water distribution with fixed outlet structures only.

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ANNEXURE

PRACTICAL DETAILS OF SIMPLIFIED STEADY STATE (S3) MODEL IN SPREADSHEET FORM

The theory behind and the structure of the S3 model have been treated in the main text of the report. Below, some practical details are given of the S3 model; i.e. how the spreadsheet is set up, and its application to Fordwah and Masood distributaries. This will enable interested researchers to use the model; to modify it or to apply it to other distributaries. The S3 model was also developed in the programming language 'Matlab', and applied to the other 12 distributaries of Chishtian sub-division, for utilisation in the Integrated Model of Chishtian subdivision (Strosser, 1997). These other 12 distributaries and the Matlab version of the S3 model are not treated here.

DETAILS OF SPREADSHEET SET UP

The basis for the spreadsheet for each distributary (14 in Chishtian sub-division) is the information collected by IIMI-Pakistan in 1995 and 1996, presented in the report "Water distribution at the secondary level in the Chishtian sub-division" (Tareen *et al.* 1996). Additional, mainly descriptive, information can be found in "Hydraulic characteristics of Chishtian sub-division, Fordwah Canal Division" (Iqbal 1996).

In the annexures of the former report, for each distributary three tables are given containing 1) outlet data, 2) calibration of outlets, and 3) inflow-outflow test. These tables were available as computer files in word processor format, and they could be imported without modification into a spreadsheet (MS Excel 7.0). Since the tables for all distributaries were built in the same way, the structure of the spreadsheet model could be equal for all distributaries.

In case one wants to apply the model to other data sets, which are likely to be in a different format, it is advised to use the Matlab version of the S3 model, that was developed in the framework of the "integrated model" of Chishtian sub-division. The Matlab version offers a more structured approach with data files, program and output file. It automatically adapts to the number of outlets and handles water distribution at the tail, while setting up the spreadsheet model requires copying formulas by hand etc.

However, the spreadsheet version of the model is more flexible. It allows easy editing of data and formulas, e.g. in case of special outlet types, bifurcation of minors, etc., or for different 'what if' scenarios. Results can be seen on the same screen as the data and formulas.

The first working name for the model was "volume balance model"; this old name can still be found in the spreadsheets. Later, this name was changed, because even the SIC model contains a volume balance, and the S3 model also does hydraulic calculations.

In columns A through J, data of the calibration exercise are imported, and in columns L through T, data of the inflow-outflow test. On the right of this, in columns V thru AH, is the calculation block, with the resulting outlet discharges in column AG. Outlet data are placed below these blocks, because they are not used in the calculations, except for the actual Y (orifice height). Actual B and Y are not in the upper two tables, because they were measured in the field during canal closure in January, and not during the calibration and inflow-outflow exercises.

The tables contain more data than needed for the model, but they are left intact to cross-check data etc.. The following data are actually required for the calculations:

DATA ENTRY

Column A: name of the watercourse (for calculation of location)
Column B: outlet type
Column N: measured discharges (inflow-outflow test)
Column Q: measured Hu (upstream head) (inflow-outflow test)
Column S: measured D (depth in distributary) (inflow-outflow test)
Column T: observed flow condition (inflow-outflow test)
Columns F and G (Hu and Hd measured during calibration exercise) are needed to calculate the working head for submerged outlets, because Hd data are not available for the inflow-outflow test.

Information about actual Y (orifice height) is copied from the third table (lower down in column M) to column Z in the calculation block.

Data are in the foot and cusec format, still commonly used in Pakistan. The codes in column T "flow condition" have the following meaning:

ON = Orifice Non-modular (submerged orifice)
OM = Orifice Modular (free-flow orifice)
FF = Flume Free-flow
FS = Flume Submerged.

The fifth allowed flow condition is "closed". Observations from the calibration exercise (in column J), such as "broken" or "P. closed" (partially closed) should not be entered in column T.

CALCULATIONS

In the calculation block, the model calculations are implemented in a number of columns, so that the calculations can be followed step by step, and to avoid formulas in cells from becoming too complex. This block can be built by simply copying the first line downwards as far as there are outlets. Only tail outlets require different formulas.

In column V, the distance of the outlet from the distributary head in km is determined from the name of the outlet (e.g. "28110 R"), to be used for seepage calculation. For irregular outlet names, such as minors or tail outlets, this has to be entered by hand.

Column W repeats the outlet discharge as measured or calculated in the inflow-outflow test, and in the next column the discharge in the distributary just upstream of each outlet is calculated, starting from the inflow at the head. This represents the steady state flow during the inflow-outflow test. The seepage outflow in cfs/km is calculated in cell AE1 from the difference between the inflow and the sum of the outlet discharges, divided by the length of the canal.

In column Y, the exponent n in the discharge formula $q = C H^n$ is determined from the flow condition; $n = 1.5$ for open flume flow (with vertical sides); $n = 0.5$ for orifice flow.

Actual Y is copied from the outlet data table into column Z. It is used to determine the water level at which the flow through APM's and OFRB's changes from orifice flow to open flume flow.

The working head during inflow-outflow test (Ho) is calculated in column AA. It is smaller than the measured upstream head (in column Q) in case of submerged outlets and in case of APM outlets (H = Hu – Y).

In column AB the important calculation of the change in water level in the distributary takes place, dependent on the new local discharge in the disty calculated in column AE. The new local discharge depends on the new inflow at the head, entered in cell AE3, and on the new supplies to the outlets situated upstream. This is where one of the simplifications of the model comes into play: the supply to the outlet itself is not taken into account in the calculation of the local water level, to avoid circular references.

The change in water level is used to calculate the new upstream head and the new working head in columns AC and AD. In column AF, the outlet supply is calculated, testing for (a change in) flow condition. It is called "Q trial outlet" because at low water levels the calculated outlet Q might be negative or bigger than the discharge in the distributary. In that case the outlet Q is set to zero in column AG, containing the final outlet supplies.

At the tail, the outlet supply does not depend on the water level, but on the discharge reaching the tail. If there is more than one outlet at the tail (tail cluster), the water reaching the tail is distributed in the same proportion as was measured during the inflow-outflow test.

Column AG serves to visualise the new flow condition that was used in the outlet Q calculation.

CALCULATION OF MONTHLY AVERAGES AND VARIABILITY

This is done in a second worksheet for each distributary, linked to the model worksheet. It contains three table blocks. In the first table, daily water supply to each outlet is calculated dependent on the daily inflow at the head. Below this are two smaller tables for the average monthly outlet supplies and the monthly variability of the outlet supplies.

Before the table of daily outlet supplies can be made, the model worksheet has to be linked to the worksheet with the tables. This is done by entering a reference to cell C3 of the tables worksheet in the cell that contains the new inflow in the model worksheet (cell AE3).

Because a worksheet can only have 256 columns, the days for one year have to be placed vertically in different rows. So the outlets come horizontally instead of vertically as in the model worksheet.

The first three columns of the table contain the month number, the date, and the observed inflow (in m^3/s) on that date. The first row contains the outlet names, the second row copies the outlet supply (in cusec) from the model worksheet, and this is converted into m^3/s in the third row.

Using the table function of Excel, the daily inflow values of column C are input one by one in cell C4. Cell C3 converts the inflow into cusec. As described above, the value in cell C3 is taken as the new inflow in the model worksheet, which calculates the outlet supplies. These are then copied to the row in the table from which the inflow was taken.

The tables are built with the Table command in the Data menu of Excel. This command creates a block filled with formulas {TABLE(,input_cell)}. To avoid that the recalculation of the workbook becomes very slow, formulas should be converted to values after building a table. This means using the Data/Table command each time when the inflow data (or the model) are (is) changed.

DETAILS OF MODELS OF INDIVIDUAL DISTRIBUTARIES

The Simplified Steady State model was applied to all 14 distributaries in Chishtian sub-division except Azim. Minor adjustments were needed for individual distributaries as described below. Fordwah and Masood disty are treated first because they were used to build, calibrate and validate the model. Their spreadsheets are combined with Azim's in the workbook file "Volbal_4.xls". Other distributaries are found in the file "Othervb.xls". Azim will be treated separately at the end.

Fordwah Distributary

The supply to Jiwan minor was not given in the tables nor elsewhere in the report mentioned above (Tareen *et al.* 1996), so it had to be calculated from the inflow, outlet supplies and seepage losses (table 3.12.1 on page 93 in the same report).

In the table of the calibration exercise, for outlet 107820 R, Hd has been entered as 0.11. This does not match the flow condition "ON", so it has been changed to 1.11.

No other 'manual' adjustments are needed.

Sensitivity of tail supply w.r.t. inflow at the head is about 10 (1% change in the inflow gives rise to 10% change in the tail supply), illustrating the sub-proportional behaviour of the outlets in the upper and middle reaches. The tail falls dry at an inflow of about 130 cfs; this is 82% of the authorised inflow.

Masood Distributary

The Simplified Steady State Model was validated for Masood, using the same data that Visser used to validate the SIC II model. A problem with Masood was the observed variable seepage loss (sometimes seepage inflow). Normally, seepage loss is taken as constant both in the SIC model and in the S3 model. For Masood, seepage in the original data set is calculated in cell W20 (total seepage in cfs), and a new seepage can be entered in cell AG20.

In the data set used to build the model (14 or 15-11-95, inflow 23 cfs), one outlet is closed. In the data set of 27-11-95 (inflow 28 cfs), a second outlet is closed. This can be simulated by entering the text "closed" in column T, or replacing the formula in column AG with the value 0. It is more difficult to open an outlet in the model that was closed in the original data set, because no discharge information is available for such an outlet.

The command area of the original tail outlet of Masood (50200-TR) is now irrigated by direct outlets from Fordwah Branch Canal and is not supposed to receive water from Masood anymore. Tail outlet is now 45950-R. However, at the inflow of 28 cfs, water was observed to flow beyond outlet 45950-R. In the model, the formula for the supply to 45950-R was changed in such a way that the outlet gets the full discharge in the distributary up to 3 cfs; any excess discharge flows to the old tail.

During the inflow-outflow test of Fordwah distributary, all APM and OFRB outlets were behaving as orifices, so in the formula for the outlet supply no test was introduced for a change in flow condition from FF to OM. However, OFRB outlet 36620-R of Masood is FF in the model-building data set (inflow 23 cfs), and is OM at an inflow of 28 cfs. An extra test for this change is introduced in the formula in cell AF14. This longer formula is used in the models of all the following distributaries.

Sensitivity of tail supply w.r.t. inflow at the head is about 6. Masood tail falls dry at an inflow of 18 cfs; this is 57% of the authorised supply (taken as 31.65 cfs because of the truncated tail).

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