Research into the relationship between maintenance and water distribution at the distributary level in the Punjab

Final Report

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Summary

Pakistans agricultural production is largely dependant on irrigation. The hot climate and the low rainfall cause a need for surface irrigation. Luckily the Himalayas in the north feed several large rivers running through Pakistan. This water is used to supply the crops with water. The existing irrigation systems are originally designed and implemented by the British and now face serious problems of several kinds. At the primary level, operations induce a constant state of unsteady flow causing variations in inflow to secondary channels.
At the secondary level, maintenance determines the distribution of water. Due to the heavy silt loads of the water, irrigation channels are posed to the constant threat of siltation. Many distributaries face shortages of water at the tail. It is difficult to control this distribution because the relation between maintenance and distribution is complex. In this study the effects of maintenance on the distribution have been analyzed. For a specific distributary, nl. Fordwah Distributary the characteristics were simulated with a hydraulic software package. Based on extensive measurements in the field a model was made and calibrated. It was found that the performance of the channel needed improvement. The effectiveness of various maintenance scenarios have been compared by using the ratio of overall effectively supplied discharge to total incoming discharge as performance indicator.
Furthermore a theoretical model has been made in which the maintenance of the cros sections of the channel was expressed in one variable only. It was found that the variations of this variable, $A.R^{2/3}$ in the Manning-Strickler equation, have only a slight effect on the overall performance of a distributary. For individual outlets the effect can be large, however.
From the study it can be concluded that the effects of measures of maintenance are difficult to predict.
It was found in both parts of the study that measures of desiltation do not increase overall performance substantially. Controlling the characteristics of individual outlets proves to be more efficient and cost-effective.
Finally it was found that gaining insight in the performance of a distributary is a time consuming and laborative job. A quick approach to assess the distribution of water is necessary to determine the need of correction. A suggestion for such a procedure is made.

Foreword

This study on irrigation in Pakistan is executed as a final thesis of my education at the Faculty of Civil Engineering at the University of Technology, Delft. I would like to thank Professor ir. R. Brouwer and ir. P. Ankum for their help during my study.
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Terminology:

Equitable distribution:	Each tertiary unit receives a discharge proportional to the area being served
Proportional distribution:	Variations in discharge in the parent channel are distributed to the outlets proportional to the areas being served
Sensitivity:	The ratio of the rate of change of an offtaking discharge to the rate of change of discharge in the ongoing parent channel
Modular outlets:	Those outlets whose discharge is independent of the water levels in the distributary and the Water Course provided that the water level in the distributary is higher than the water level in the Water Course.
Semi-modular outlets:	Those outlets whose discharge , although depending on the water levels in the distributary, is independent of the water levels in the water course, so long as the minimum working head required for the semi-module is available
Non-modular outlets:	Those outlets whose discharge is a function of the difference in levels between the water surface in the distributary and the watercourse. Variations in either affect the discharge.
Main system	Primary and secondary canals operated and maintained by the Irrigation Department
Distributary:	A secondary canal taking its supply from a main or branch canal, supplying water to minors and tertiary outlets
Minor (distributary):	A secondary canal taking its supply from a distributary, supplying water to outlets
Tertiary Unit:	area commanded by one outlet;downstream of this outlet the watermanagement is the reponsibility of the water users (farmers)
Water Course:	A canal downstream of the Tertiary Outlet inside the Tertiary Unit
Non-perennial channel:	A channel which is designed to irrigate during only a part of the year (usually Kharif season)
Rabi season:	winter irrigation season: limits fixed by the Irrigation Department for the Rabi flow season are 15 October to 15 April

Kharif season:	summer irrigation season: limits fixed by the Irrigation Department for the Rabi flow season are 15 April to 15 October

List of abbreviations:

AOSM:	Adjustable orifice semi-module
APM:	Adjustable proportional module
CEMAGREF:	French research center for agricultural and environmental engineering
CCA:	Culturable command area
GCA:	Gross command area
Disty:	Distributary
FSD:	Full supply depth
FSL:	Full supply level
ID:	Irrigation Department
IIMI:	International Irrigation Management Institute
OF:	Open Flume outlet
OFRB:	Open Flume with a Roof Block
PCOFRB:	Pipe cum Open flume with a Roof Block
RD:	Reduced distance from the head of the channel 1 RD = 1000 ft. = 304.8 m
SDO:	Sub-divisional Officer
SE:	Superintending Engineer
SIC:	Simulation of Irrigation Canals
XEN	Executive Engineer

Conversion of units

❑ Length
1 inch = 0.0254 m = 25.4 mm
1 foot = 0.3048 m
1 yard = 0.9144 m
1 mile = 1609.3 m

❑ Surface
1 square foot = 0.0929 m^2
1 acre = 0.4047 ha.
1 square mile = 259 ha

❑ Volume
1 cubic foot = 0.028317 m^3 = 28.317 l.
1 acre-inch = 102.8 m^3
1 acre-foot = 1233.5 m^3
1 MAF = 1 million acre-feet = 1,2335 . 10^9 m^3

❑ Discharge
1 cubic foot per second (cusec or cfs) = 0.028317 m^3/s = 28.317 l/s

❑ Discharge per area
1 cfs per 1000 acres = 0.6 mm/day =0.07 l/s.ha
1 l/s.ha = 8.64 mm/day

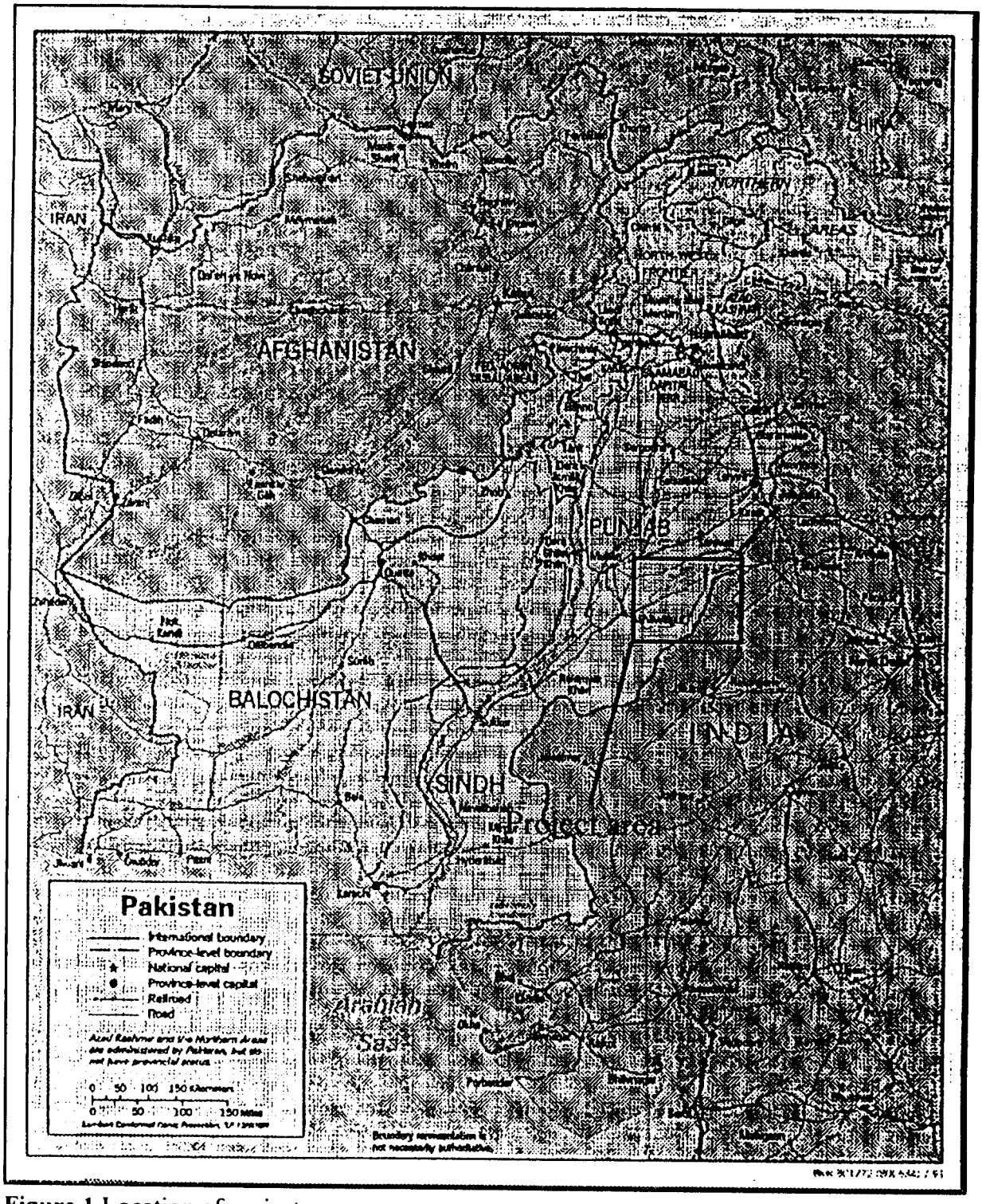

Figure 1 Location of project area

1 Introduction

1.1 Background information

1.1.1 Pakistan

Pakistan lies in between Iran, Afghanistan, China, India and the Indian Ocean. It covers an area of approximately 770,000 km^2. The total population numbers around 123 million and is growing at a rate of 3% per year. The Islam is the dominating religion. The climate is mostly hot and dry in the middle and south. In the north it is more moderate. Official languages are Urdu and English. The literacy rate is only 35%.
The Islamic Republic of Pakistan gained its independence from the United Kingdom on the 14th of August 1947. Its legal system is based on English common law with provisions to accommodate Pakistans stature as an Islamic state.
The Gross National Product amounts to $ 45.4 billion. The average annual per capita income is around $ 380. The real growth rate is estimated at 4.8% (all figures 1991).
Agriculture is very important in Pakistan. 54 % of the labor force is active in agriculture. Furthermore, agricultural production provides 26% of the Gross National Product and accounts for 80% of exports value.

1.1.2 Irrigation in Pakistan

Pakistan has one of the largest contiguous irrigation system in the world. Supplies are diverted from the Indus river and its major tributaries through 19 barrages or head works, 12 Link Canals and 46 canals to command an area of 16 million hectares. The total length of canals is about 60,000 km.
After the independence in 1947 a dispute arose between Pakistan and India on the water rights of the rivers which cross the boundaries between the two countries. In 1960 the water rights were formally settled in the Indus Water Treaty.
According to this treaty, Pakistan gained the rights of the three western rivers Indus, Jhelum and Chenab, while India received the rights of the three eastern rivers Ravi, Beas and Sutlej. With foreign aid Pakistan has built large link canals to transport water to the areas which were deprived of water by the treaty. A schematic diagram of this situation is given in appendix A Agricultural production for three major crops (wheat, rice and sugarcane) has been stagnant for twenty years. At this moment it is generally recognized that water management is one of the main constraints for the further growth of agricultural production. The water resources are limited and the distribution is not functioning optimally. If agricultural production is to be raised, water management has to be improved.

The irrigation systems in Pakistan have a classical layout (see figure 2). The main system, operated and maintained by the Irrigation Department, consists of primary and secondary canals. Along the secondary canals lie the Outlets. These are the points of contact between the Irrigation Department and the users. Here the water is transferred from the main system into the Tertiary Units. Within the Tertiary Units the users are responsible for further distribution

and maintenance.

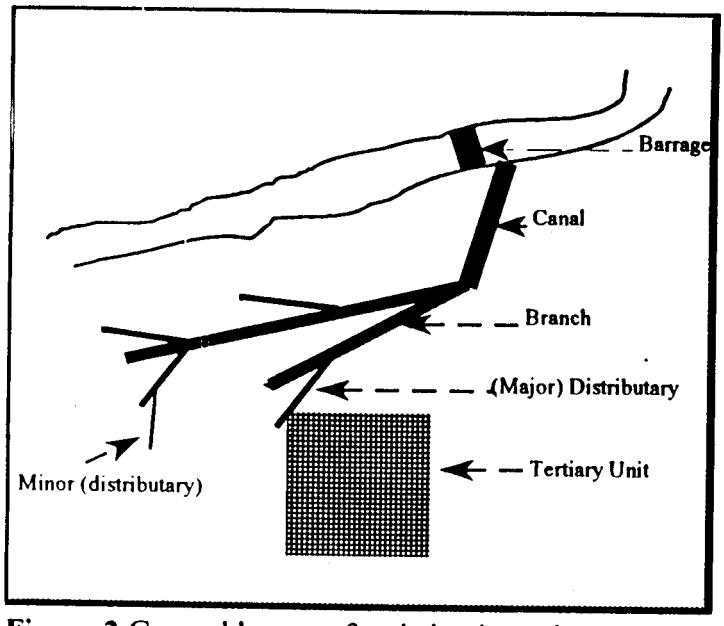

Figure 2 General layout of an irrigation scheme

1.2 Background of the study

1.2.1 Relevance

Protective irrigation is the concept underlying Pakistans irrigation network. The available water is spread over as large an area as possible. By keeping the water scarce it is assumed that it will be used in the most efficient way. In this way the return per unit of water is to be maximized.

Integral part of this concept is the idea of equitable distribution of water. The water is spread equally over the area, such that each acre of land receives the same amount of water. In this way everyone has to cope with the same shortage. This supply-oriented distribution enables relatively constant discharges to the tertiary units.

Reality however, is far different. At all levels, primary and secondary and tertiary, there are factors, both technical and non-technical, which induce unsatisfactory water distribution in terms of quantity, variability and equity. This has a negative impact on crop production.

IIMI is conducting research to establish a relation between the water distribution at the main canal level and the crop production. Different research activities are performed at the different levels in the irrigation system.

- At the strategic level, studies are conducted to assess the criteria used for the water allocation to different canal commands (Primary Units).

- At the main system level studies are performed to understand the irrigation scheduling. A complex system of preferences determine which irrigation channels will receive water at which times and which will not. The validity of these targets can be questioned if adverse effects of these schedules can be determined. Next to that, research is done on operational rules at the primary level to realize these targets.

- At the Watercourse level, studies are done to develop a relation between the discharge through the Outlet and the discharge arriving at the farm gate

- At the farm level, models are developed to determine the relation between water supply and crop production

This study focusses on the secondary level. The function of secondary canals is to distribute water equitably over the command area. For two reasons it is important to maintain this concept of equitable water supply:

1. To maintain the maximization of the return per unit of water.
2. To prevent the distribution from falling into anarchy and chaos. There is no alternative for the concept of equitable water supply in this respect.

1.2.2 Difficulties with maintenance

The design of secondary canals (distributaries) creates some particular difficulties in maintaining the equity. The distribution is generally not controlled by operational actions, since there is no hardware to operate. The distribution of water is controlled by **maintenance**. The objective of the maintenance is to distribute the water equitably over the irrigated area by taking the right measures such as desilting, bermcutting or periodical adjustments of Outlets.
This is not an easy task. Changes in the characteristics of secondary channels, such as changes in roughness, width, slope or bed level all lead to a different pattern of water distribution and loss of equity.
Next to the characteristics of the channel, the properties of the Outlets play an important role in the distribution of water.

1.3 Problem definition

In short, the main problem in the secondary canals can be defined as:

The performance of the water distribution in the secondary canals is inadequate to deliver the water to the Tertiary Units in the required manner.

Inadequate performance can be attributed to:
1. Influence of sedimentation in canals
2. Influence of inadequacy of hydraulic operation structures
3. Influence of non-stationary discharge conditions

1.4 Objectives of the study

The objective of the study is:

To identify the maintenance measures with which the water distribution from the secondary canal to the Tertiary Unit can be improved.

Next to this main objective, the following sub-objectives have been formulated:

1) The identification of the required manner of water distribution to the Tertiary Units

2) The identification of the different components of maintenance that influence the water distribution in the secondary canal system to the Tertiary Units.

3) To determine and quantify effective and cost-efficient maintenance

4) To develop a tool for the manager to rapidly assess the performance of Distributaries, determine causes of possible deficiencies and determine effective solutions

5) To develop recommendations for long-term improvements for Distributaries

1.5 Constraints of the study

The study has the following constraints:

1) The boundary conditions of the studied channel are as follows. The upstream boundary condition is the head of the Distributary. Although the inflow from the primary channel into the Distributary may show deficiencies, this is not further studied.

2) The downstream boundary conditions are the tertiary units. The distribution through the Outlets to the Tertiary Units is part of the study, but not the distribution within the tertiary unit.

3) Maintenance which does not have a direct hydraulic impact or which cannot be simulated with the help of a hydraulic model is not taken into account.

1.6 Approach of the study

First a description of the project area is given. In this part the general features of the area under study are presented.
The theoretical backgrounds of the design of distributaries are examined next. The result of this part is a definition of the required performance of distributaries. This is the reference to the rest of the study
Third, the maintenance factors which influence the performance are defined. The different kinds of maintenance are the tools with which the manager of the system has to work.
In the next step one particular channel, Fordwah Distributary, is taken as an example for further study. Of this channel, a hydrodynamic model is made. This model is calibrated with field measurements. The result of this segment of the study is the performance of the water

distribution of the chosen channel.
With the model effects of several scenarios of maintenance on the distribution are simulated and quantified. These results of the different maintenance scenarios are compared, using the required performance defined in the first part of the study as a reference.

1.7 Structure of the report

In chapter two a description of the present situation is given. The setup of the whole irrigation system is given, as well as the location of Fordwah Distributary and the particular problems connected to the location. Next to that a detailed description of the channel itself is given.
In chapter three the design of distributaries is described. Also the required performance and the parameters which are used to measure the performance are given. In chapter four maintenance in general is treated. The different components are described. In chapter five the hydraulic model of the system is described. Input, calibration and the distribution pattern in the current situation are presented.
In chapter six the simulations which have been carried out are given. Effects of different maintenance scenarios on the water distribution are calculated. In chapter seven a theoretical approach is given. Based upon a slightly different concept the effect of maintenance variables on performance are compared. In chapter eight the reader will find conclusions and recommendations.

2 Description of the project area

2.1 Fordwah Branch

2.2.1 General layout of the system

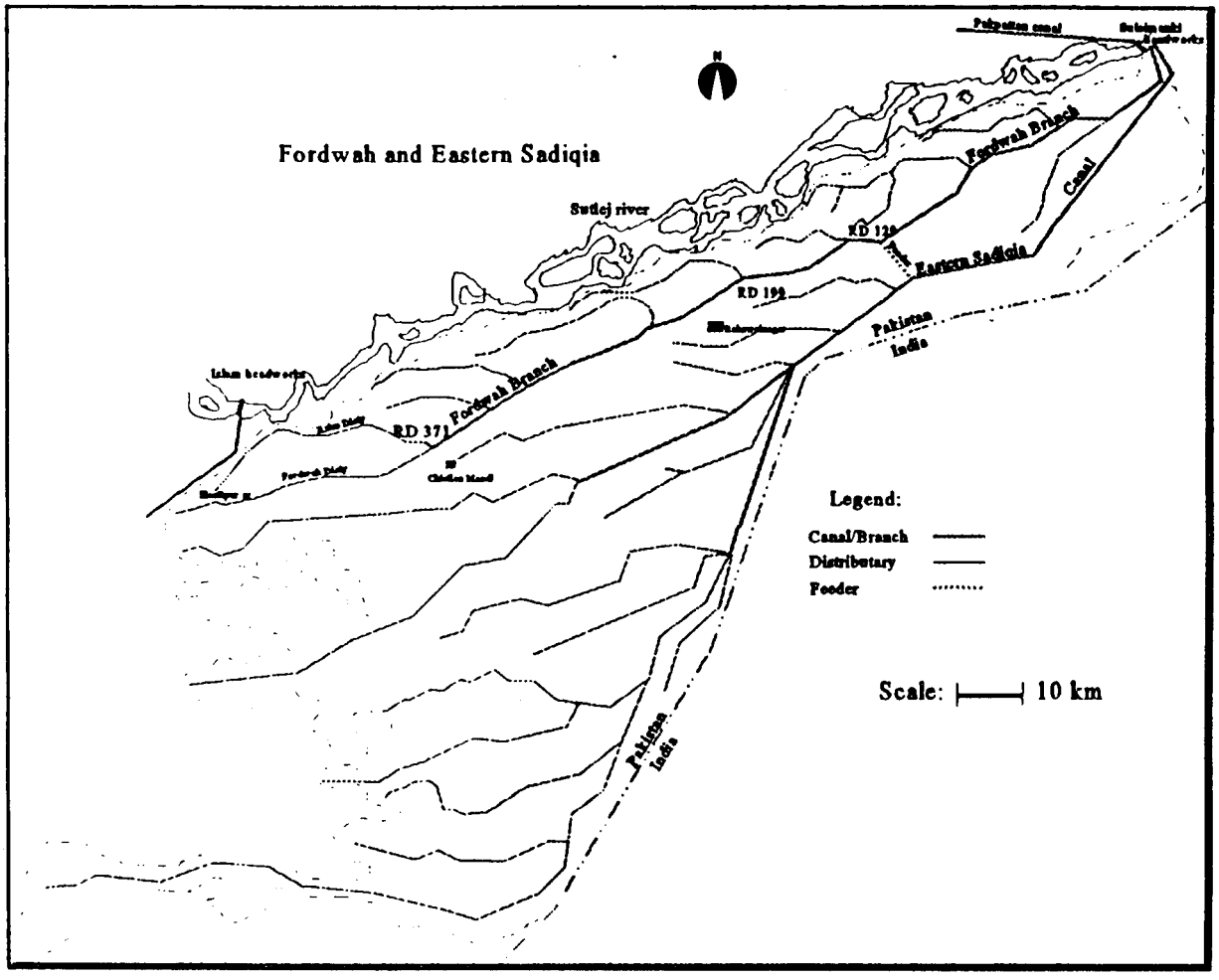

Figure 3 Fordwah area

The Fordwah area is located in the Punjab province. Fordwah Branch takes off at Suleimanki head works, on the left side of the Sutlej river. The Fordwah Branch is a primary canal in the Fordwah Division of Bahawalnagar Circle in the Bahawalpur Irrigation zone.
Following the Indus Water Treaty in 1960 India has control of the Sutlej river. It has diverted this water for its own use. Pakistan has since then provided other sources of water for the areas formerly supplied by the Sutlej river. In the summer season (Kharif season) the water is diverted from the Chenab river and conveyed through so called Link Canals to the Sutlej river. In the winter season (Rabi season) water comes from Mangla Dam and is also transported

through these link canals to the Sutlej river. Because supply in the winter season is very limited, irrigation channels have been divided in perennial and non-perennial channels. Perennial channels receive water the entire year, while non-perennial channels receive water only in the summer season (15th of April to 15th of October).

From Suleimanki head works three canals take off. Pakpattan canal starts from the right bank. Fordwah Canal and eastern Sadiqia Canal take off from the left bank.
At RD 44.8 Fordwah Main Canal is divided into Fordwah Branch and Macleod Ganj Branch. Therefore the Fordwah Division is divided in three subdivisions:

1. Minchinabad Subdivision (Fordwah Canal RD 0 to RD 44.8, Fordwah Branch RD 0 to RD 129 and Macleod Ganj Branch)
2. Bahawalnagar Subdivision (RD 129 to RD 245 of Fordwah Branch and off takes)
3. Chistian Subdivision (RD 245 to RD 371 of Fordwah Branch)

In the primary channel a complex system of rotations has been installed to spread the shortages of water in winter season. Priorities are given to the subdivisions for certain periods of time. Within the subdivisions, distributaries are operated on an on/off basis.

Minchinabad subdivision is non-perennial. In Rabi season the supply to Bahawalnagar subdivision and Chistian Subdivision does not come through Fordwah Canal. Instead it is diverted from Eastern Sadiqia Canal through a feeder canal and enters Fordwah Branch at RD 129. The reach from RD 0 to RD 129 is then completely dry.

The Chistian Subdivision, in which Fordwah Distributary is located, was designed for a discharge of about 29 m^3/s. The Gross Command Area (GCA) of the subdivision is 74369 ha. The Culturable Command Area is 67693 ha.

Fordwah Branch is as most irrigation systems in Pakistan, a system under upstream control. Gates are operated manually. Discharges are measured by measuring the downstream water level at structures. With a simplified Manning-Strickler formula (stage-discharge relationship) a relation between the downstream water level and the discharge is determined.

Because of the inaccuracies in the used methods of determination of the discharges it is possible that at the tail of the channel a surplus remains. In the past there used to be an escape channel at the tail of Fordwah Branch to divert any excess of water to Bahawalpur Canal. This escape channel is no longer in use.
Any surplus of water at the tail of Fordwah Branch now has to be given to either Azim Distributary or Fordwah Distributary (see figure 4). This means that in case of a surplus at the tail of Fordwah Branch a breach is likely to occur in either Azim or Fordwah Distributary.

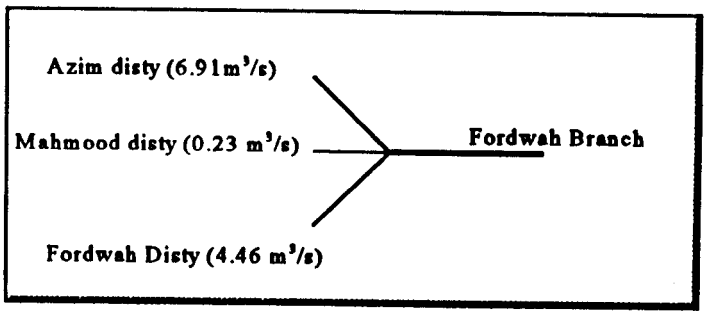

Figure 4 Situation at tail of Fordwah Branch

Extra complications in the operation

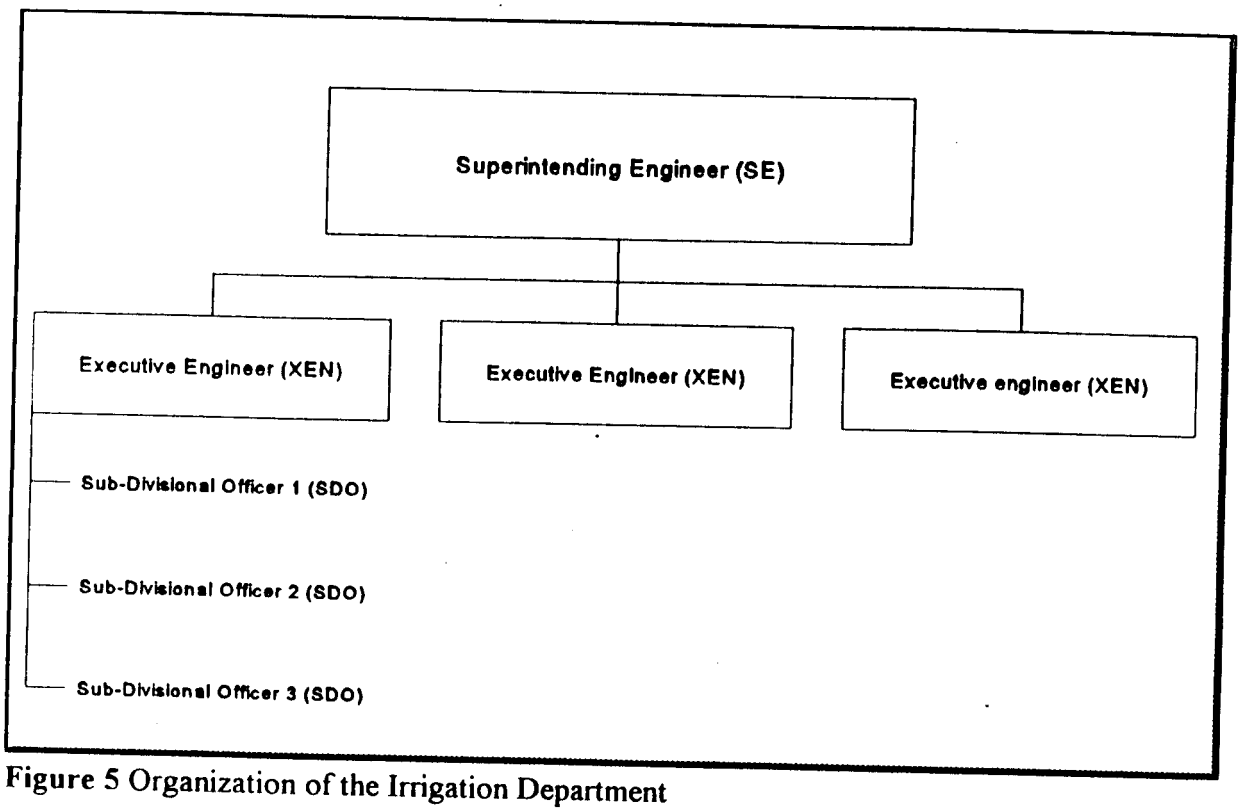

Figure 5 Organization of the Irrigation Department

of Fordwah Branch are caused by the submergence of structures. A proper communication system is also not installed.

The institutional framework which runs the system is the Irrigation Department (ID). At the head of the Bahawalnagar Circle is the Superintending Engineer. The different divisions are headed by the Executive engineers (XEN). The Sub-divisional officers (SDO) are responsible for the subdivisions. In figure 5 this setup is given.
Operation and maintenance is the responsibility of the Irrigation Department. This responsibility begins at the head works and ends at the outlet.

One of the problems this irrigation system has to cope with is the distribution of silt. The problem of silt entering the system are not new. In Fordwah Branch so called King's Vanes have been installed at the off takes of several distributaries to keep the silt out of the distributaries and in the primary channel. Nevertheless Fordwah Branch copes with serious problems of siltation. The main cause of this siltation is probably the fact that the channel is running at less than 50% of its design discharge during Rabi season.

The selected canal for this study is Fordwah Distributary, a secondary canal which starts at the tail of Fordwah Branch, at RD 371.

2.2 Fordwah Distributary

2.2.1 Schematization

Fordwah Distributary has a total length of 42.6 km. The design discharge at the head is 4.46 m^3/s (158 cusecs). It serves an area of 14800 ha. The water allowance is 0.28 l/s.ha. The average slope of the bed is 21cm/km. The distributary is perennial. There are 87 outlets along the channel. Over the whole length there are 3 drop structures (see figure 6).
The tertiary units do not have a standard size. All tertiary units have different sizes. This can vary between 43 ha and 496 ha. This implies that all tertiary units have a different water allowance.
At RD 65 a minor distributary takes off from the right bank. This minor distributary is called Jiwan Minor.

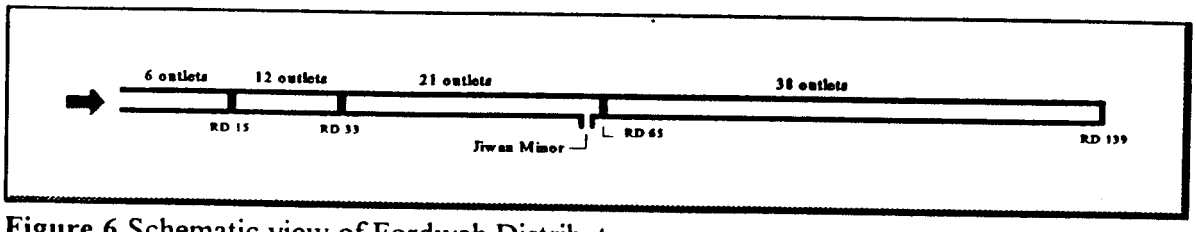

Figure 6 Schematic view of Fordwah Distributary

discharge at head	4.46 m^3/s
average slope	0.21 m/km
side slope (v:h)	varies between nearly vertical to 1:5
width at head	6.10 m
depth at head	1.19 m
width at tail	1.52 m
depth at tail	0.52 m

Table 1 General figures on Fordwah Distributary

2.2.2 Cross structures

The cross structures in Fordwah Distributary are all broad crested weirs (see figure 7). There are four cross structures (RD 15, RD 33.3, RD 42.8 and RD 65). Originally designed to operate under free flow conditions, three of the four now are submerged. Only the cross structure at RD 15 is running under free flow conditions. The cross structures at RD 33 and RD 65 are both submerged due to siltation downstream of the structures. The cross structure at RD 42 is no longer functional after a redesign of the distributary in the past.

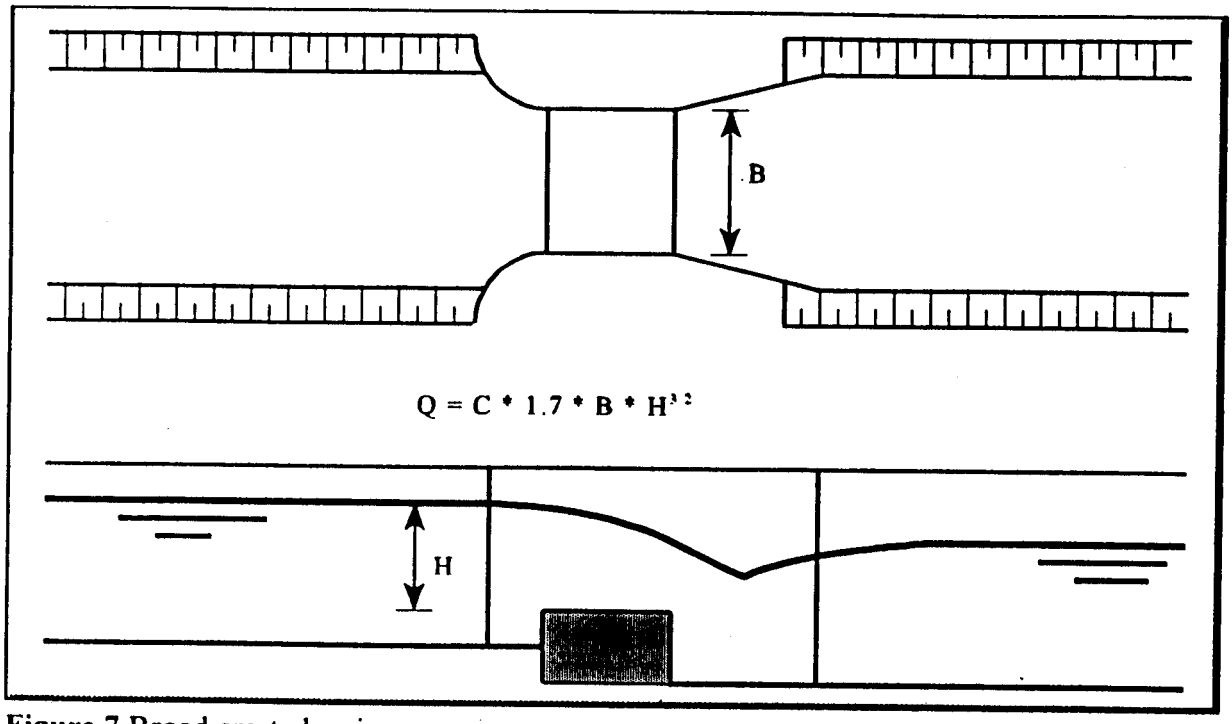

Figure 7 Broad crested weir

2.2.3 Outlets

There are different kinds of outlets applied in Fordwah Distributary. In fact the history of the development of outlets is an interesting subject of its own. The three types of outlets along Fordwah Distributary are:

1. Adjustable Orifice Semi-Module (AOSM), also known as Adjustable Proportional Module (APM) see figure 8
2. Open Flume with Roof block (OFRB), see figure 9
3. Pipe outlets see figure 10

The AOSM is the predecessor of the Crump-de Gruijter gate. The orifice has a rounded upstream face to prevent the jet from contracting. The outlet is not really adjustable. For adjustments a masonry key has to be broken out. Then the roof block can be adjusted and the key rebuilt. This takes several hours.

The Open Flume with Roof Block is originally designed as an open flume. The roof block has been added to prevent high discharges in case of high water levels in the distributary. During normal conditions the outlet is supposed to operate as an open flume. However, in Fordwah Distributary the water level touches the roof block in most cases.
The Open Flume with Roof Block along Fordwah Distributary have a modification. The outlet is not situated along the channel, but on the other side of the bank. The water leaves the channel through a pipe and flows into a tank on the other side of the bank. The outlet is placed in this tank. This construction is applied to increase the silt draw of the outlet. The silt draw is presumed to be higher because the outlet takes the water close to the bed of the distributary, instead of the top layer.

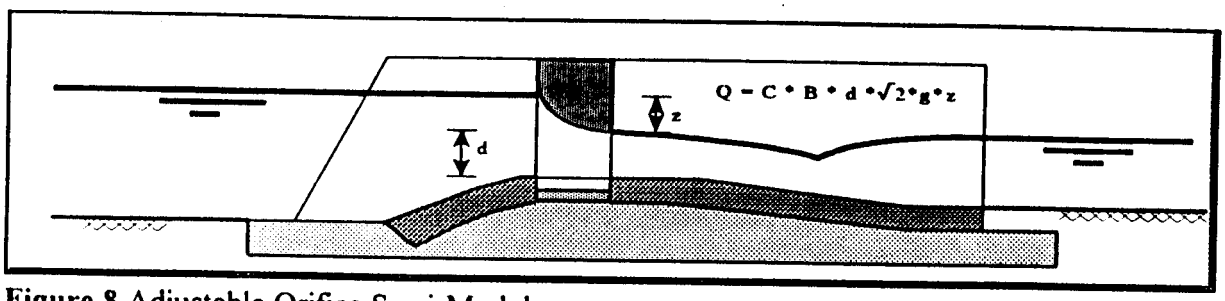

Figure 8 Adjustable Orifice Semi-Module

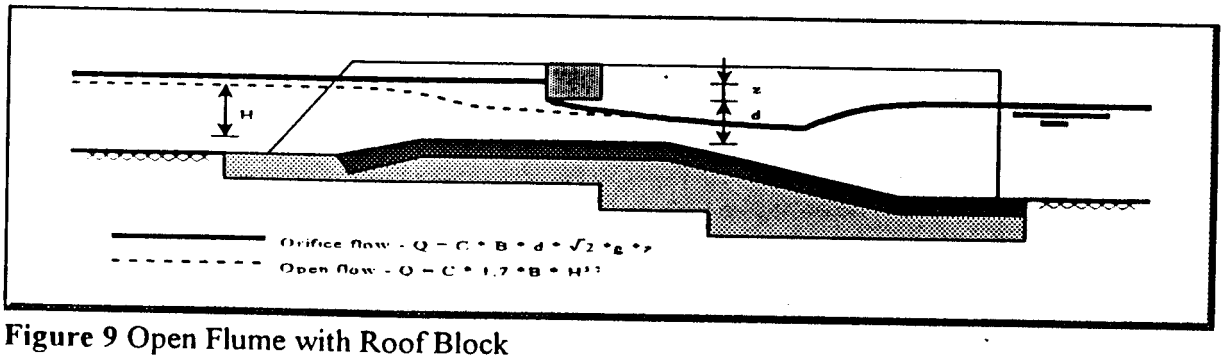

Figure 9 Open Flume with Roof Block

Pipe outlets (see figure 10) are installed where the available head is not enough to implement a semi-modular outlet. Pipes therefore usually operate as submerged structures. In this channel two are installed.

The outlets of the type OFRB are mainly located from RD 0 to RD 65. The AOSM outlets are located in the tail portion of Fordwah Distributary, from RD 65 to RD 139. There are only two pipe outlets at RD 32.

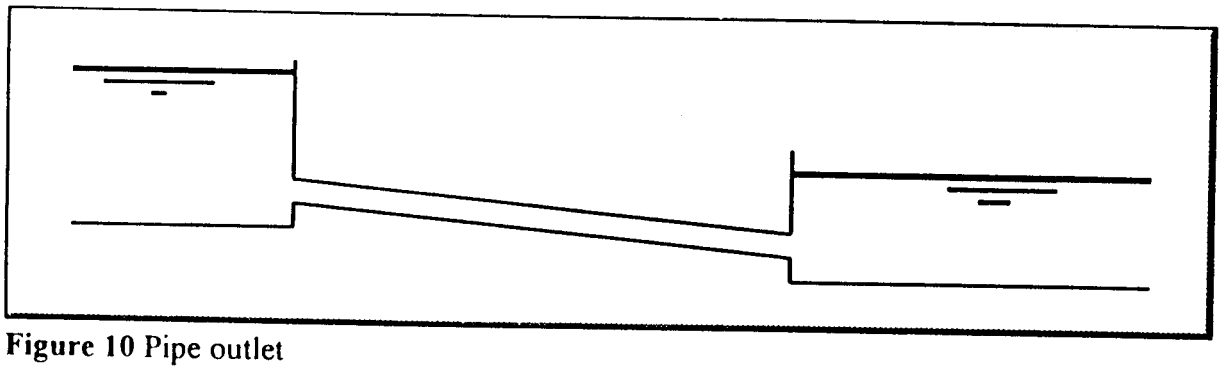

Figure 10 Pipe outlet

2.2.4 Channel

Fordwah Canal, Fordwah Branch and Fordwah Distributary are all unlined, earthen canals. They are designed with Lacey's equations for non-scour, non-silting channels.

The canal is unlined. The slopes of the banks vary from nearly vertical to 5:1 (h:v) at places where cattle is drenched. The bed of the channel is even, although at some places local variations occur. These variations in bed level over a short distance are mostly due to local

In figure 12 the inflow for one month, the month of March 1995 has been given. This shows that for this particular month there was no guarantee for any discharge at all. If the measured values are interpolated the total volume of water which was supplied in this month was only 53% of the design value.

Another graph which shows the variability in the inflow is one with half-hour measurements during a two day period in the month of April (figure 13). This shows that even during short periods of time, the discharge can vary considerably.

Another feature of the current system is that the discharge in Fordwah Distributary is sometimes higher than authorized. This can also be seen in the graph in figure 13. As has been explained before the way of operation of Fordwah Branch inevitably leads to this phenomenon.

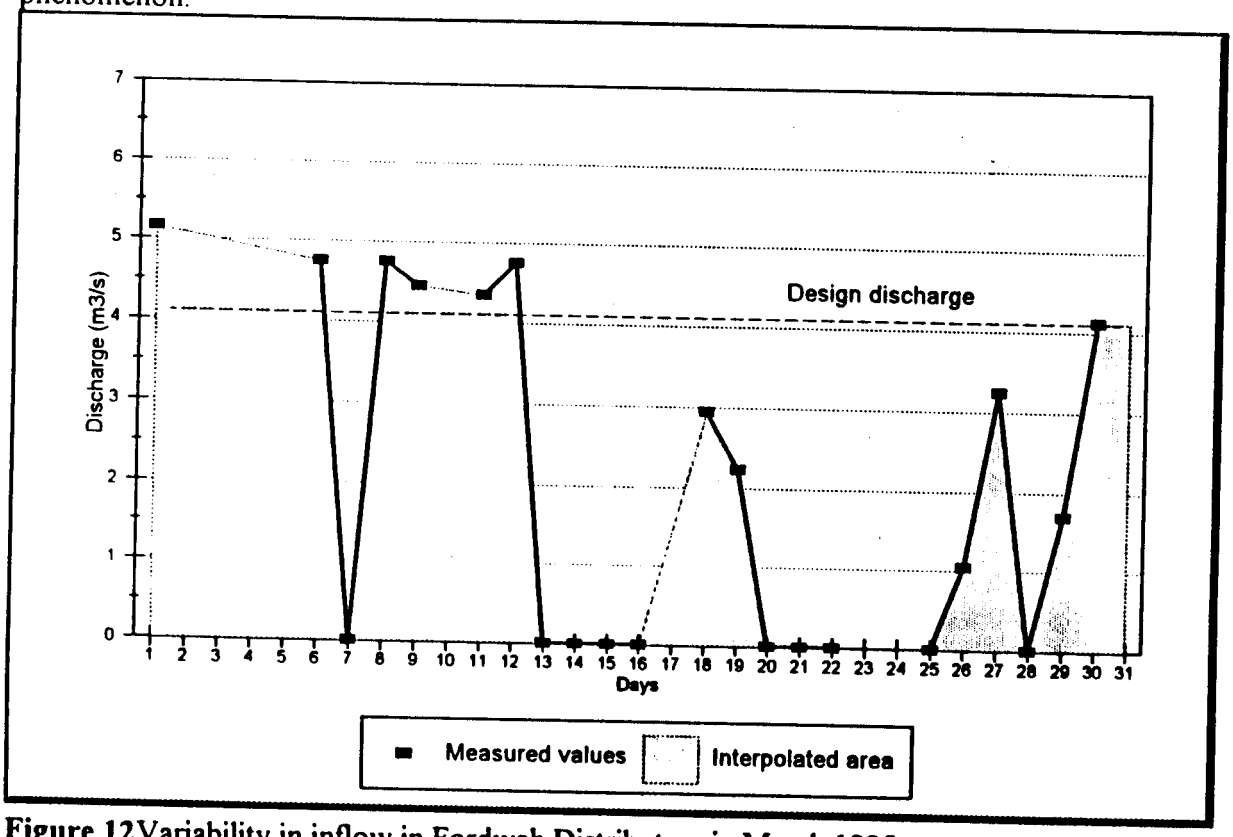

Figure 12 Variability in inflow in Fordwah Distributary in March 1995

The main conclusion that can be drawn from this is that in the case of Fordwah Distributary deficiencies in maintenance can only be second-order problems. The main problem is the enormous variability of the inflow.

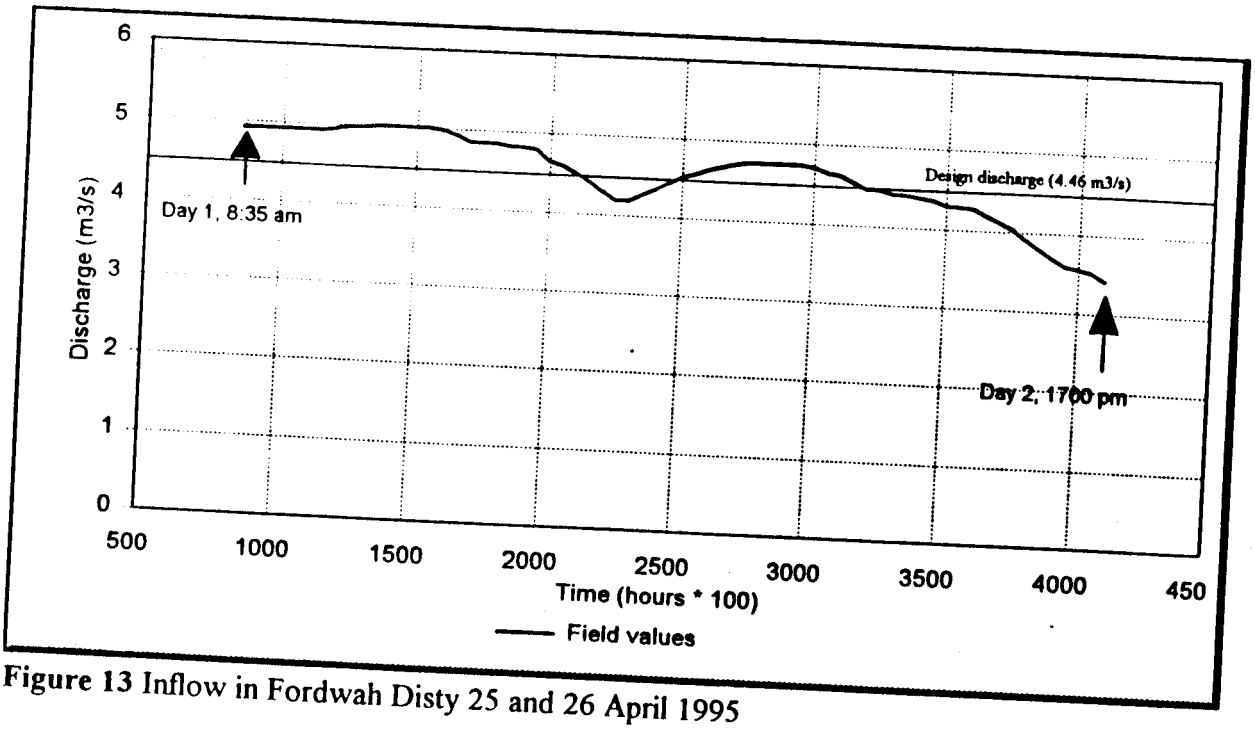

Figure 13 Inflow in Fordwah Disty 25 and 26 April 1995

This, however is not the subject of this study. For the further study the inflow is assumed to be equal to the design discharge of the channel.

3 Flow control in distributaries

3.1 Theoretical backgrounds

A distributary should be so designed and maintained 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 the watercourses should just run their calculated allowances with no surplus at the tail of the distributary"[1]

The inflow is distributed among the outlets. A small part is lost to seepage. When the seepage is neglected, the sum of the outgoing discharges must be equal to the incoming discharge:

$$Q_{Inflow} = \sum_{i=1}^{n} q_i$$

With:

Q_{inflow} : Incoming discharge at head of the Distributary
q_i : Discharge through outlet

When we define C_i as:

$C_i = q_i / Q_{inflow}$,

Then : $\sum_{i=1}^{n} C_i = 1$ must be valid for all values for Q_{inflow} .

The proportionality S is defined as:

$$S = \frac{dq_i/q_i}{dQ/Q}$$

With:

Q : Local discharge in Distributary

If C_i is independent from Q_{inflow} , the behavior of the channel would be totally proportional. In that case S=1 for all values of Q_{inflow} .
Ideally, the inflow would be distributed proportionally. This is not easy to accomplish.
When at some point sub-proportional behavior (S<1)exists, consequently super-proportional behavior (S>1) must exist somewhere else. This behavior has been given in figure 14.

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[1] Kennedy, R.G., "Distribution of water for irrigation by measurement" page 5. Punjab Irrigation Branch Paper No. 12.

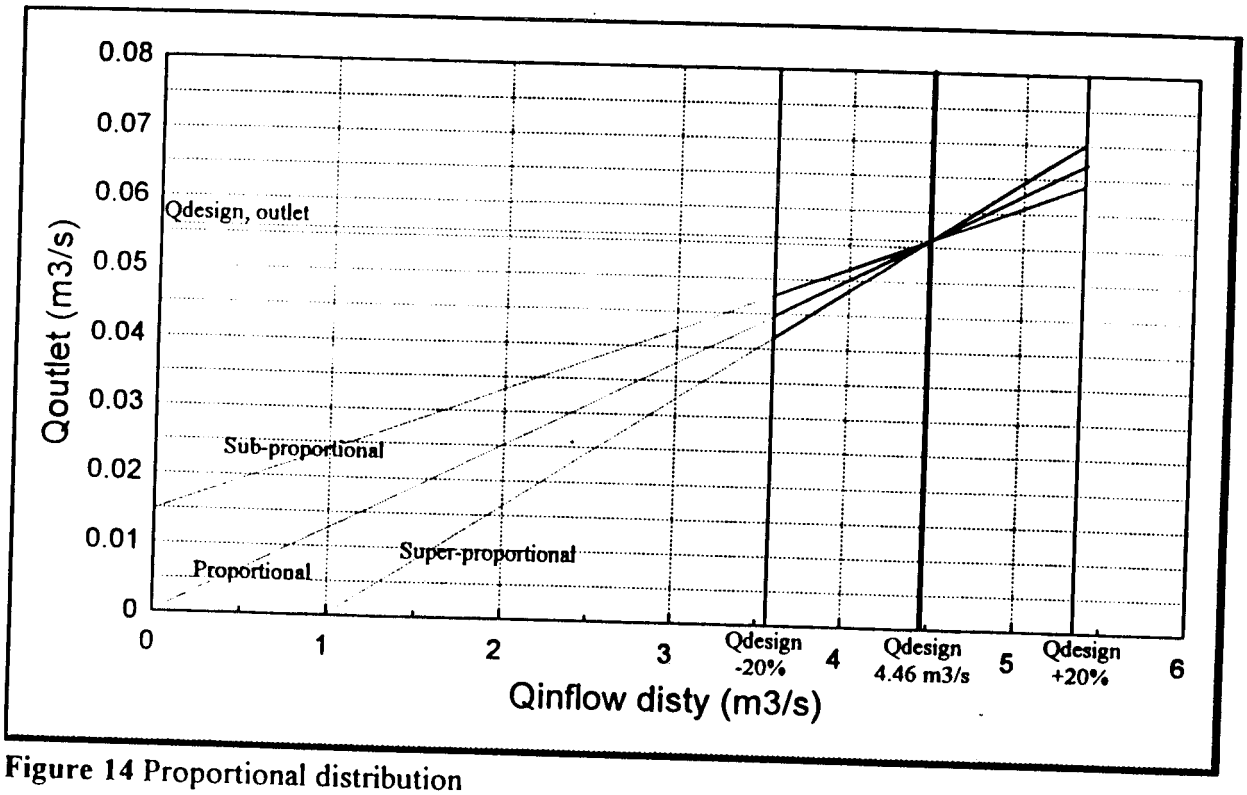


Figure 14 Proportional distribution

Originally distributaries in Pakistan have been designed to distribute water proportionally over a limited range around Q_{design}. Distributaries had a certain design discharge, and when the inflow was within a certain range around this discharge, all outlets should receive a discharge proportional to their command area.

Distributaries are designed so that, when running at Full Supply Discharge, all outlets receive their authorized discharge. Since there are no means of control in the distributaries themselves, it is extremely important that the hydraulic dimensions of all the hardware (channel, cross-structures and outlets) correspond to their design dimensions. Any variation in these dimensions will affect the hydraulic behavior of the channel, and consequently the equitable distribution of water.

Distributaries have originally been designed to divide the flow not only equitably, but also proportionally. This means that besides dividing the full supply discharge by the ratio of the area served, also variations in the discharge are equitably divided. Simply put, a 1% increase in discharge at the head should lead to a 1% increase in discharge at all the outlets.
This would be the correct definition of ideal performance for any distributary. With the present existing hardware of distributaries this kind of performance definition is not relevant however. The problems arise when this ideal performance has to be translated in dimensions of channels, structures and outlets. With the free outlet as a standard, it becomes difficult to distribute a range of discharges at the head in a predescribed manner.

In order to accomplish proportional behavior, the Q(h) relationship of the distributary must be related to the Q(h) relationship of the outlet. For the distributary a Q(h) relationship can either be defined by a structure or by the channel itself (see figure 15).

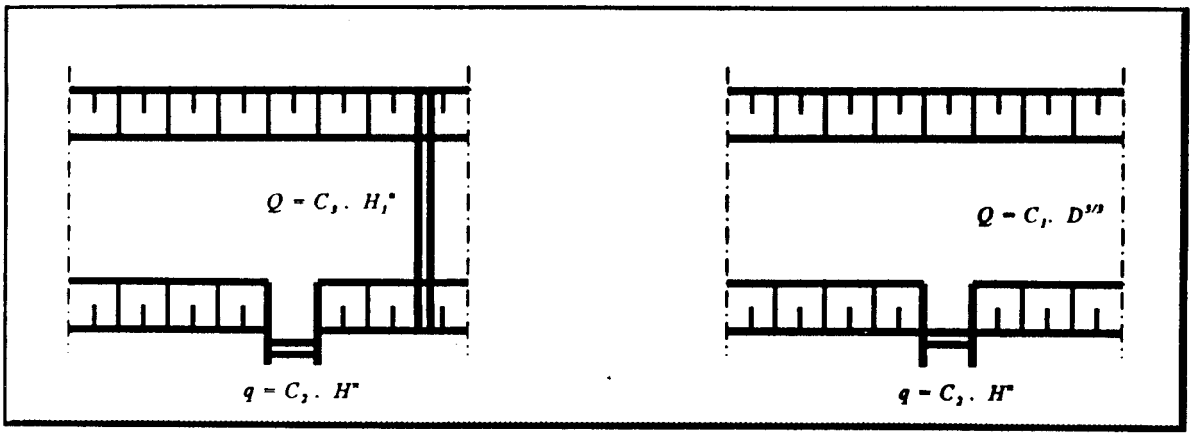


Figure 15 Top view of outlet with and without cross structure

In Pakistan, the distributaries are used to establish a relation between the discharge and the water level.

The hydraulic dimensions required to attain this performance, were determined as seen below. The definition of proportionality is that for each outlet the sensitivity S = 1

$$S = \frac{dq_i/q_i}{dQ/Q}$$

With Manning-Strickler's formula written as:

$Q = k.B.D.D^{2/3}.i^{1/2} \rightarrow Q = C_1.D^{5/3}$

$dQ = C_1.5/3.D^{2/3}.dD$

$\rightarrow dQ/Q = 5.dD/3.D$ (1)

Q: Canal discharge	(m^3/s)
k: roughness coefficient	($m^{1/3}/s$)
B: width of channel	(m)
D: water depth	(m)
i: slope	(-)

Two assumptions have been made to come to this result. First, the hydraulic radius is taken as equal to the depth (infinite width). Second, the area of flow is assumed to be linear with the depth (vertical side slopes).

In the same way, the discharge formula for an outlet is written as:

$q = C_2.H^n$

$dq = C_2.n.H^{n-1}.dH$

$\rightarrow dq/q = n.dH/H$ (2)

q: discharge through outlet (m^3/s)
H: head over crest (m)

Combining 1 and 2, and taking dD=dH, the following result is attained:

$$\frac{dq/q}{dQ/Q} = 1 \rightarrow \frac{n.dH/H}{5.dD/3.D} \rightarrow \frac{3.n.D}{5.H} = 1 \rightarrow H = \frac{3.n.D}{5}$$

n is different for weirs and orifices. For weirs such as the Open Flume, n =1.5, which leads to the design criterium for Open Flumes:

H = 9/10.D

This means that the crest of the flume should be placed at 1/10th of the depth above the bed level of the distributary.

For orifices such as the APM, n = ½. This leads to the design criterium for APM's:

H = 3/10.D

This means that the bottom of the roof block should be at 0.7*D above bed level. The last two equations are the basis of design of distributaries in Pakistan.

3.2 Practical aspects

However, both in the case of the OFRB and the APM, adjustments in the design have led to loss of proportionality. This will be explained in the following section. Because of this proportional distribution with variable flows can no longer be the target. It has been the experience that with the current setup of the hardware, the only achievable target is the proportional distribution when the channel is running at design discharge. Variations in inflow are no longer incorporated in the targets.

3.2.1 OFRB

First, the OFRB (figure 8). This outlet is designed to function as an open flume. The roof block is basically added to prevent excessive discharge in case of high water levels in the distributary. The discharge formula is of the weir type ($Q = C.B.H^{1.5}$).

The crests of the OFRB are not set at 0.1 D, but vary between 0.3 and 0.6 D In the case of Fordwah Distributary both the setting of the crest levels and the implementation of the roof blocks result in the non-proportionality of the channel.

The roof block is set at 0.1 to 0.2 m below F.S.L. That's why this outlet, originally designed as a flume, always works as an orifice. The relation between dq/q and dQ/Q is unknown and so is the discharge coefficient. This makes it impossible for anybody, including the Irrigation Department, to determine the discharge by measuring the head over crest. Consequently, any adjustments made on the dimensions to change the discharge, are based on trial-and-error and not on any formula.

3.2.2 APM

In the case of the APM, the situation is a little different. In Crump's original design H_s was set at 0.3 D. Y was kept equal to H_s, thus fixing the crest at 0.4 of the depth above the bed level. In this setting, the outlet would function proportionally.

This design led to serious problems with siltation, however. As soon as outlets on a channel were remodeled from pipes to APM's, the silt equilibrium of the channel was disturbed and the channel gradually silted up.This because the intake opening of a pipe was at bed level, whereas the APM has it's crest at 0.4 of the depth of the channel.

As experience was gained with the APM, the crest was lowered to 0.2 D, 0.1 D or even bed level, to improve the silt draw. In this setting, the APM is no longer proportional.

This is the case with Fordwah Distributary. The APM's do not function proportionally. Most of the APM's have their crests 0.1 D above bed level.

From all this the conclusion can be drawn that outlets, originally designed to achieve proportional division of flow, are used in such a way that they no longer have these characteristics. Although various phenomena hava been understood, some things still remain uncertain.

3.3 Required performance

The topic of this study is the relationship between canal maintenance and water distribution and not the effect of inflow variation on the distribution pattern. Therefore the choice has been made to look only at the situation of Full Supply Discharge and not for instance a range of 80% to 120% of Q_{design}.

The parameters which will be used to determine performance of a distributary are both parameters for a single outlet and a parameter for the distributary as a whole.

1. Parameter for each outlet

Of the outlets the ratio of the actual discharge to the authorized discharge will be used.

2. Performance for the total channel

The effectively supplied discharge of the incoming discharge is used as the parameter to measure the performance of the whole distributary. The effective discharge per outlet is

defined as the total discharge running through it when the discharge is equal or lower than authorized discharge. If the discharge is greater than authorized, the effective discharge is equal to authorized discharge. In other words, the surplus is considered to be ineffective. Because the discharge coming into the channel is equal to design discharge, a shortage at one outlet automatically means that there will be a surplus at other outlets, resulting in an ineffective discharge at this location.

In the study, the ratio of effectively supplied discharge to the sum of the authorized discharges of the outlets is used. Per definition, this ratio has a maximum of one. This maximum of one is ideal performance. Hence, performance is defined as:

If $q_i < q_{i,authorized}$ then $q_{i,effective} = q_i$ else $q_{i,effective} = q_{i,authorized}$

$$Performance = \sum_{i=1}^{n} \frac{q_{i,effective}}{q_{i,authorised}} * 100\%$$

4 Maintenance

4.1 Maintenance objective

All distributaries should be maintained in such a way that, when the channel is running at Full Supply Discharge, all outlets receive their authorized discharge. De facto this means that at each location in the distributary a fixed relationship between design discharge and corresponding water level is to be maintained. This function, which is often considered to be done by a structure, is to be achieved by the channel itself in the considered system.

4.2 Maintenance components

The maintenance which is done on distributaries can be subdivided in several components. First of all a subdivision can be made between maintenance which affects the hydraulics and maintenance which does not directly affect the hydraulic situation. These two categories can be further subdivided in:

4.2.1 Hydraulic maintenance:

1 works of desiltation

Works of desiltation are usually done by hand. Contractors, hired by the Irrigation Department do the work. In some cases, for instance in the desiltation campaign of 1992, communities were asked by the Irrigation Department to assist in the work. Even schoolchildren helped in this campaign. The channel must be completely empty for desiltation to take place. This work is planned for the month of January, which is the annual closing period. In this period, the entire system is closed.

2 kila bushing

Kila bushing is the restricting of the width of the channel by creating a berm. Bushes and twigs are planted along the sides of the channel so that in the course of time, they will hold the silt carried by the water and slowly a berm will develop.

3 berm cutting

Berm cutting involves the straightening of the banks. This work is done by hand. Grass and other organic material is removed and irregularities are cut away with shovels. The channel must be empty before the berms can be cut

4 adjusting and repairing outlets

This work is done by the Irrigation Department and comprises all changes and repairs made to outlets as described previously.

4.2.2 Non-hydraulic maintenance:

1 strengthening of banks

In the cases where a breach is expected to occur or has occurred before, banks can be strengthened. The bank will be broadened or increased in height.

2 closing of breaches

Whenever a breach has occurred, it will be closed as quickly as possible. First however, the channel must closed. For this the inlet gates of the distributary have to be closed. Then the breach can be repaired. Sometimes, in the case of a small breach, the farmers adjacent to the location will repair the breach themselves. Usually the Irrigation Department is involved.

3 miscellaneous works on outlets

If bricks come loose ore other decay takes place, this must be repaired. In this case the hydraulic properties are not affected. This work is normally done in January.

In this study only maintenance with a hydraulic impact is of interest. Non-hydraulic maintenance is necessary but does not immediately affect the performance. Bank strengthening for instance decreases the risk of a breach but does not affect the internal distribution of water.

In figure 16 the effect of maintenance on the water level is shown qualitatively. Maintenance results in lower water levels upstream and higher water levels downstream.

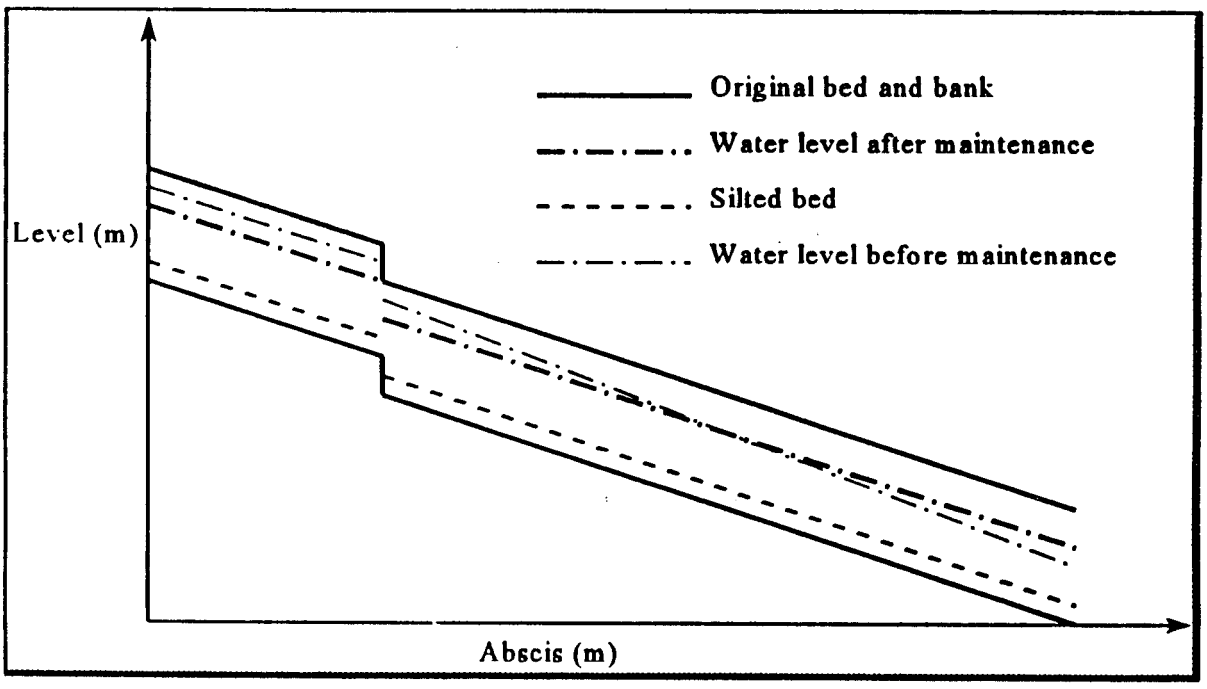


Figure 16 Development of water level due to maintenance

4.3 Silt distribution strategies

In the current situation, the distribution of silt is designed to be in proportion with the distribution of water. However, because the silt draw of an outlet is a complex phenomenon, this concept is hard to realize in the field.

Factors which determine the silt draw of outlets are:

1 The height of the crest above bed level

2 Local geometry of the intake opening of the outlet.

The silt draw of an outlet can be modified within certain limits independently of the Q(h) function of the outlet by varying the intake height. This is done with an extra tank and has been explained in chapter 2.2.3

Next to equitable distribution of water, also the silt is distributed amongst the outlets in the ratio to their CCA. One could question this approach. If the silt draw of outlets were to be maximized, the need for maintenance at the distributary level could be reduced.

5 Modeling the channel with SIC

5.1 SIC (Simulation of Irrigation Canals)

SIC is a tool with which existing irrigation systems can be analyzed. With it a representation of a physical system is made. The characteristics of the system can be simulated and analyzed. Both steady and unsteady flow can be simulated. In this paragraph the different equations which are used in SIC are given.

5.1.1 Steady flow

The steady flow computation of the program is based on the Manning-Strickler equation which is given below:

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

With:

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

and:

H :	energy height	[m]
x:	abscis	[m]
S_f :	slope	[-]
k:	constant	[-]
q :	incoming discharge	[s^{-1}]
Q:	discharge	[m]
g = 9.81		[$m.s^{-2}$]
A :	wetted area	[m]
n :	Manning coefficient	[$m^{-1/3}.s$]
R :	hydraulic radius	[m]

5.1.2 Unsteady flow

The unsteady flow computation is based on the St. Venant equations. They are approximated with the Preissman scheme given in figure 17.

Continuity:

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

Dynamic equation:

$$\frac{\delta Q}{\delta t} + \frac{\delta Q^2/A}{\delta t} + gA\frac{\delta z}{\delta x} = -gAS_f + kqV$$

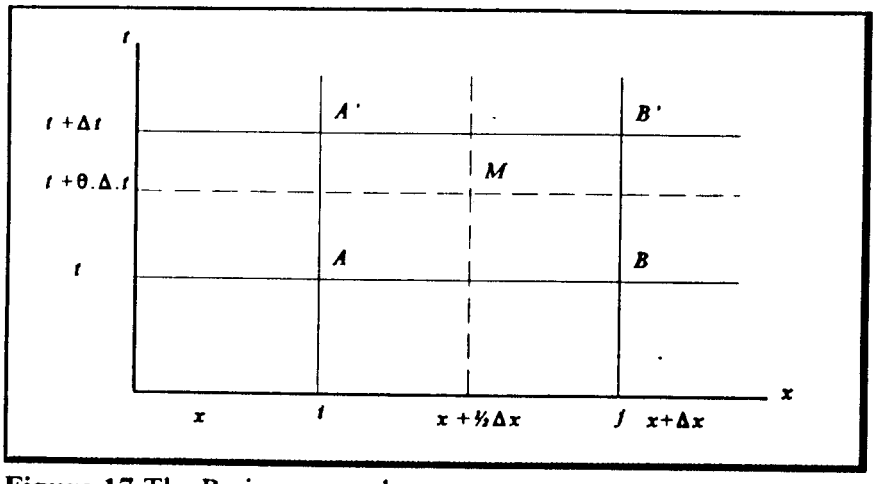


Figure 17 The Preissmann scheme

5.1.3 Structure formulas

The structure formulas in SIC are simplified to a certain extent. The formulas used differ from the classical formulas. Especially in the submerged region variances occur. In figures 18 and 19 the variables and the regions for the different formulas are given.

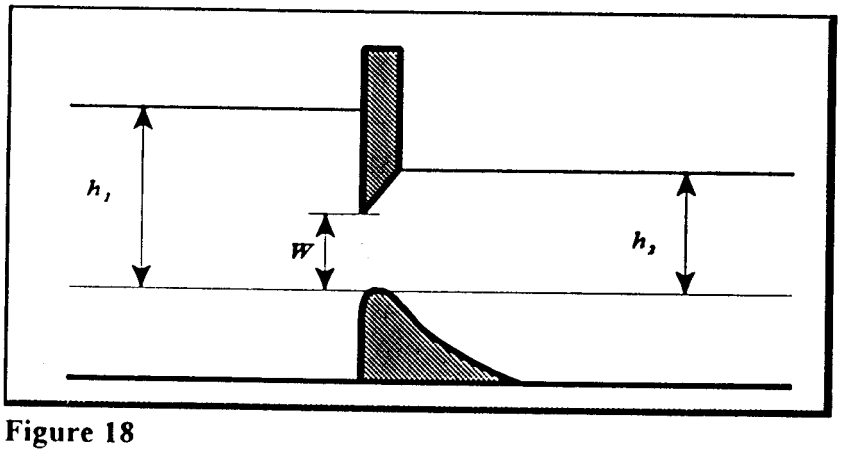


Figure 18

The ranges where the different SIC formulas are used, are presented in figure 19. They are characterized by:

1) Open, free flow
2) Orifice, free flow
3) Open, submerged flow
4) Orifice flow, partially submerged
5) Orifice flow, completely submerged
6) As 2, with low sill
7) As 3, with low sill
8) As 4, with low sill
9) As 5, with low sill

Note that the transition between open flow and orifice flow occurs for h_1=w.
In Appendix B the different formulas are given.

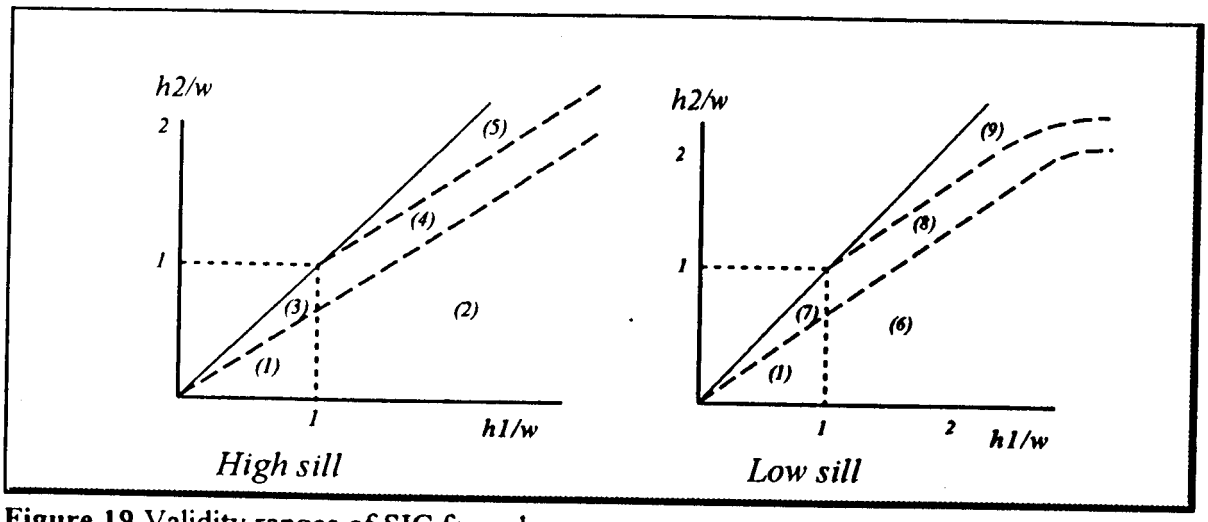


Figure 19 Validity ranges of SIC formulas

5.2 Input

The complete list of all desired input data comprises the following items.

5.2.1 Data regarding the channel:

For the data entry of a channel SIC requires cross sections at every node. The cross sections have been collected in the field. At 61 locations the cross section was measured. An example is given in figure 20. With these data a longitudinal profile has been made as well. The longitudinal profile is shown in figure 21

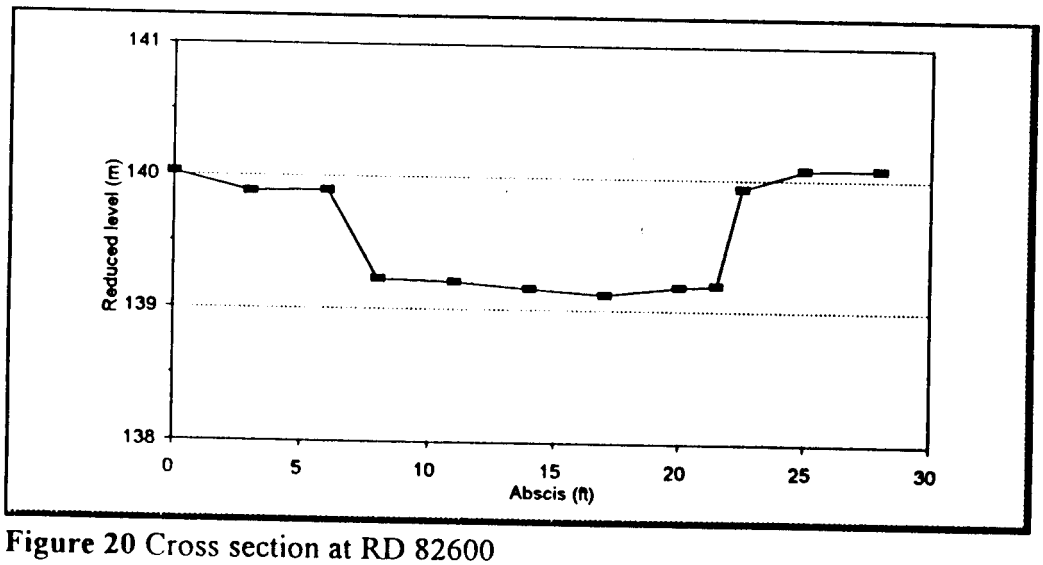


Figure 20 Cross section at RD 82600

5.2.2 Data regarding the cross-structures

These have also been collected in the field. Of each broad crested weir the width and crest level were collected. In table 2 the dimensions are given.

S. No	Location (ft)	Width B (m)	C.R.L. (m)
1	15000	3.94	143.57
2	33300	3.94	142.16
3	65300	2.45	140.26

Table 2 Input data of cross structures

5.2.3 Data regarding the outlets

IIMI had previously collected the dimensions of all the outlets. This was done because the Irrigation Department's outlet register was not very reliable on the dimensions. The authorized discharges used to check the performance were taken from the Irrigation Departments outlet register.

Input data for outlets are:

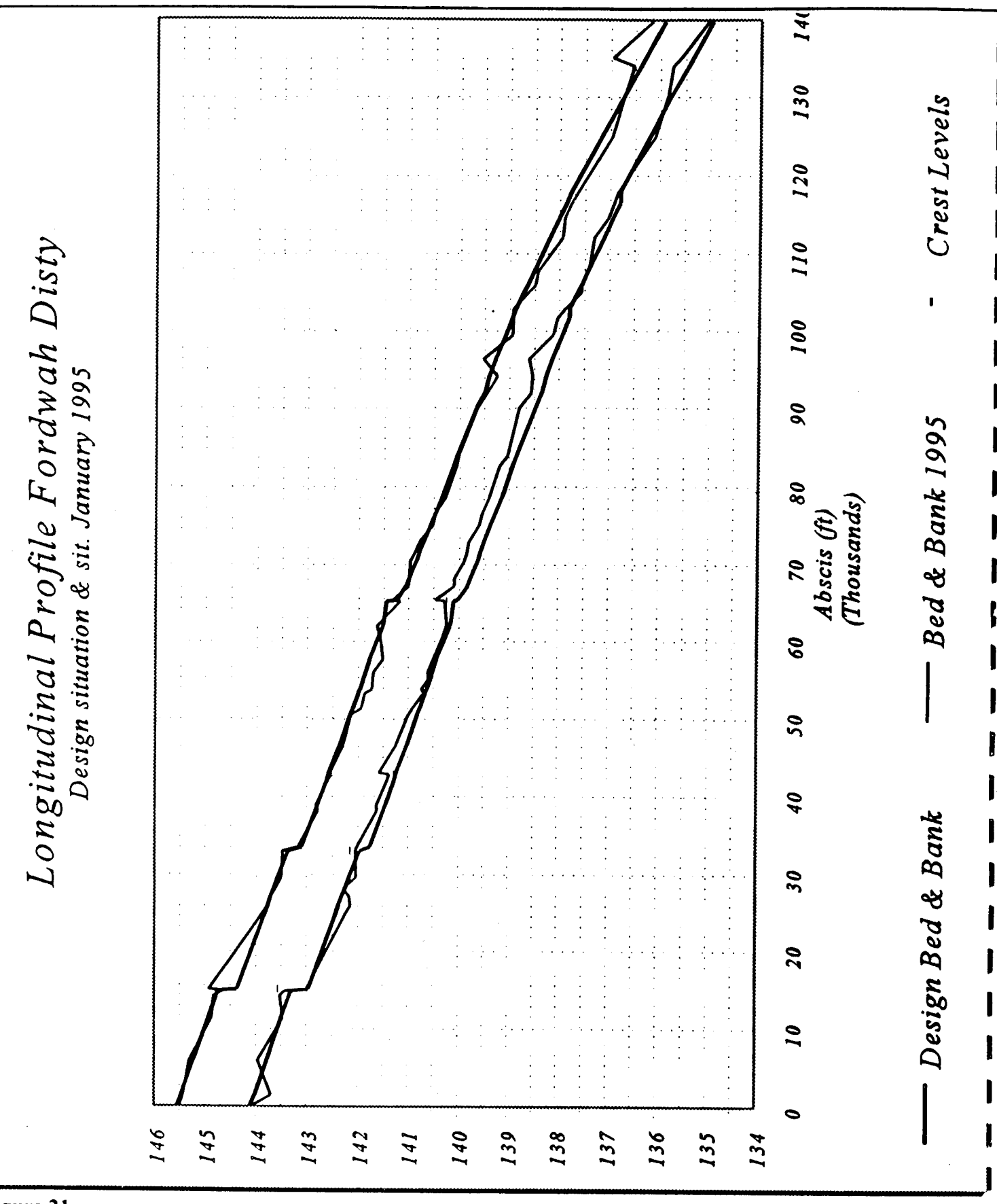
Type: APM, PCOFRB, or Pipe
width (m)
height (m)
Location (m)
Authorized discharge (m^3/s)

In SIC there is no separate option for modeling OFRB's. This is why they have been modeled as APM's. The problems related to this will be separately discussed in chapter 5.6
In table 3 an example for an outlet is given.

S.NO	Name	Type	Location (m)	width (m)	height (m)	crest (m)	authorized discharge (m^3/s)
4	14320-R	OFRB	4365	0.12	0.29	143.84	0.049

Table 3 Input data of outlets

The modeling of the minor that takes off at RD 65, Jiwan Minor, needs special attention. The minor is modeled as an outlet (broad crested weir). The discharge through the outlet is calculated in the model. The downstream boundary condition is taken just downstream of the outlet. Thus the distribution within the minor is not further taken into account. In the figure below the upstream and downstream water levels are given. In this figure it can be seen that the minor is always free flow.
For the complete list of input data see Appendix 3


Longitudinal Profile Fordwah Disty
Design situation & sit. January 1995
146
145
144
143
142
141
140
139
138
137
136
135
134
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
Abscis (ft)
(Thousands)
Design Bed & Bank
Bed & Bank 1995
Crest Levels

gure 21

5.3 Calibration

5.3.1 Variables

Calibrating hydrodynamic models is a difficult task. There are a large number of unknown variables which need to be determined. These variables cannot be measured directly in the field. The unknown variables are:

1 Inflow
2 Discharge coefficients of the outlets
3 Manning-Strickler coefficient
4 Seepage
5 Discharge coefficients of the cross structures

These values are to be determined by field measurements. In the field discharges and water levels can be measured. By doing this in a smart way, the unknown variables above can be determined. The question is how to do this as accurate as possible with limited resources

In the calibration some simplifications were made. These were:

1 The value for the Manning-Strickler coefficient was taken as uniform over the whole channel
2 The value for the seepage was taken as uniform over the whole channel. This induces an inaccuracy in the model, but the seepage was found to be relatively small, so that errors in measurements also affect the accuracy of the value for the seepage.

Discharges are in the end the main thing that counts. It is preferable to have a high accuracy on the discharges and a lower accuracy on the water levels than the other way around. But of course the two depend on each other.

The seepage can be determined by two methods: ponding tests and inflow outflow methods. The available resources did not permit ponding tests. This is why the seepage was determined by doing inflow outflow balances. One has to realize that this is already difficult in canals with just a few off takes such as branches. In the case of distributaries this becomes even more difficult, due to the large number of outlets for which the discharge needs to be determined.

A different way of doing inflow outflow tests would be to close the outlets and conduct an inflow outflow method over a long reach. This however is more difficult than it seems. The capacity of a distributary decreases in the direction of the flow. Therefore during such a test just a limited discharge can be passed. This will create low water levels at the head of the distributary. The corresponding result for the seepage will not be accurate. The errors made in this are of the same order as the seepage itself.

5.3.2 First approach

The first approach was the following

1 Calibrate the gate at the head of the channel
2 Calibrate the drop structures +Jiwan Minor

3 Calibrate 8 outlets and extrapolate the results to all other outlets
4 Determine Manning-Strickler coefficient by taking a reach without outlets, determine the discharge and measure the water levels upstream and downstream. In this way a value for the Manning-Strickler coefficient could be obtained.
5 Determine the seepage by inflow outflow over a long reach

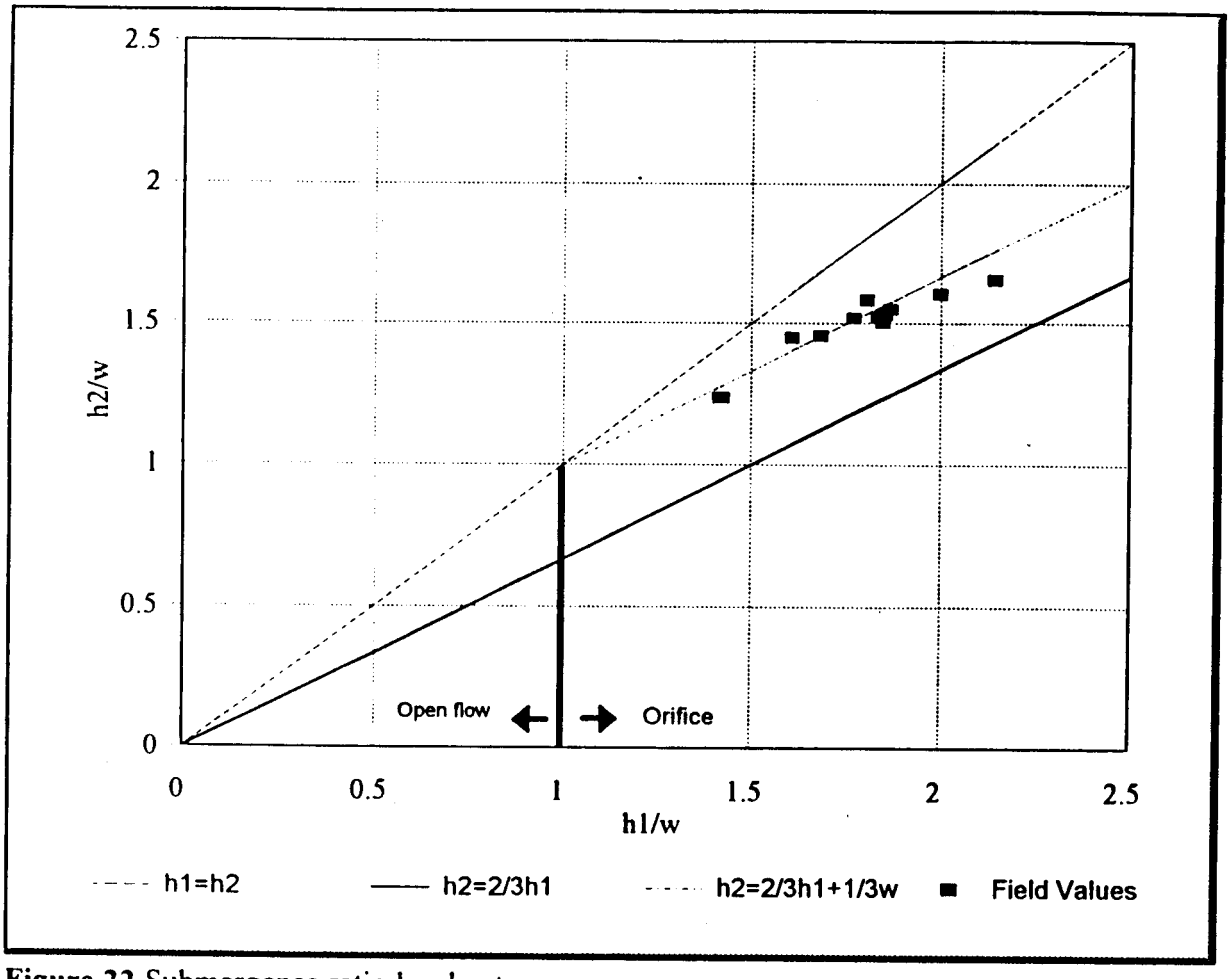


Figure 22 Submergence ratio head gate

This approach led to some insurmountable problems as will be explained below. First of all the gate at the head of the distributary is operating on the edge of free flow and submergence. (see figure 22)
This means that calibrating it will cost a lot of effort and the results might be unsatisfactory. There is a solution to this problem. At RD 15 of Fordwah Distributary there is a drop structure which operates under free flow conditions. This structure is easier to calibrate. Therefore the upstream boundary condition of the model was moved from RD 371 of Fordwah Branch to RD 15 of Fordwah Distributary.

The next thing to do was the calibration of the cross structures. As explained the drop at RD 15 was now the upstream boundary condition. Also the drop at RD 33 and RD 65 needed to be calibrated. For this five discharge measurements were taken at each location.
The results of these measurements are given in the figures below

RD 15

The drop structure at RD 15 proved easy to calibrate since it was free flow. The results of the discharge measurements can be plotted with the upstream water level (h1) on the x-axis or with $h1^{1.5}$ on the x-axis. In the latter case straight lines can be drawn to get the right discharge coefficient (see figures 23 and 24). The discharge coefficient which was found is equal to the theoretical value of 0.38.

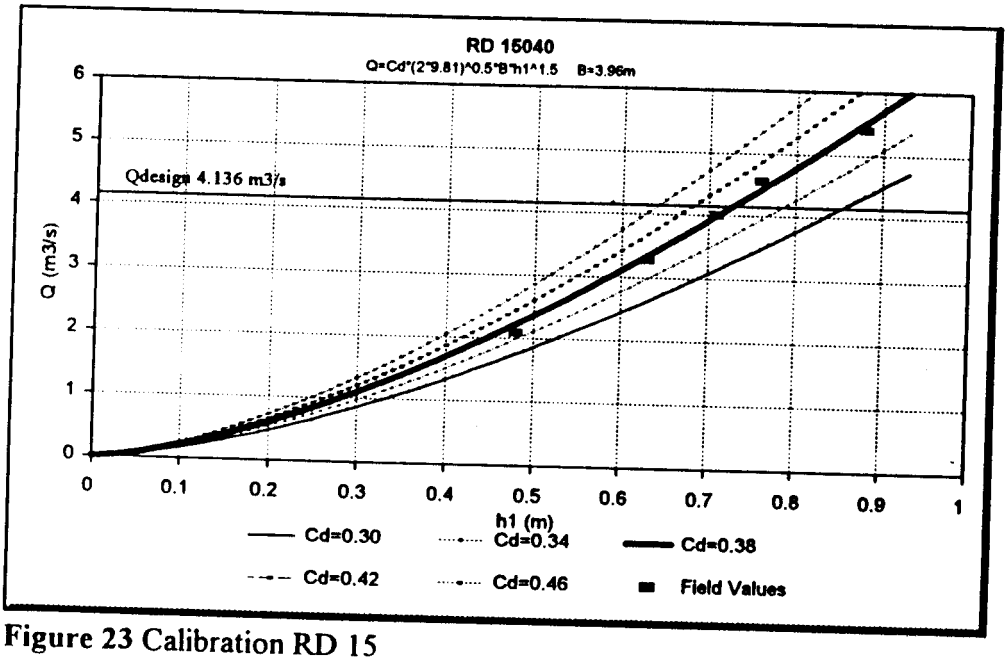


Figure 23 Calibration RD 15

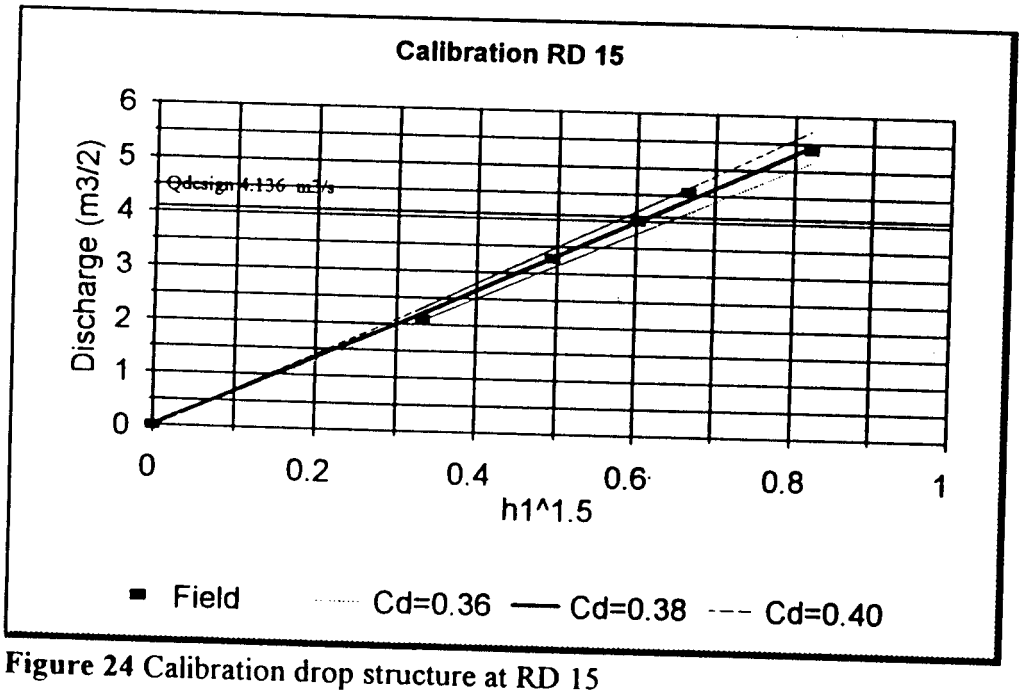


Figure 24 Calibration drop structure at RD 15

RD 33

The head loss over the structure at RD 33 was only 9 cm, which makes an accurate calibration difficult. Nevertheless four discharge measurements were taken. With a Cd value of 0.39 a proper curve could be fitted through these values (see figure 25).

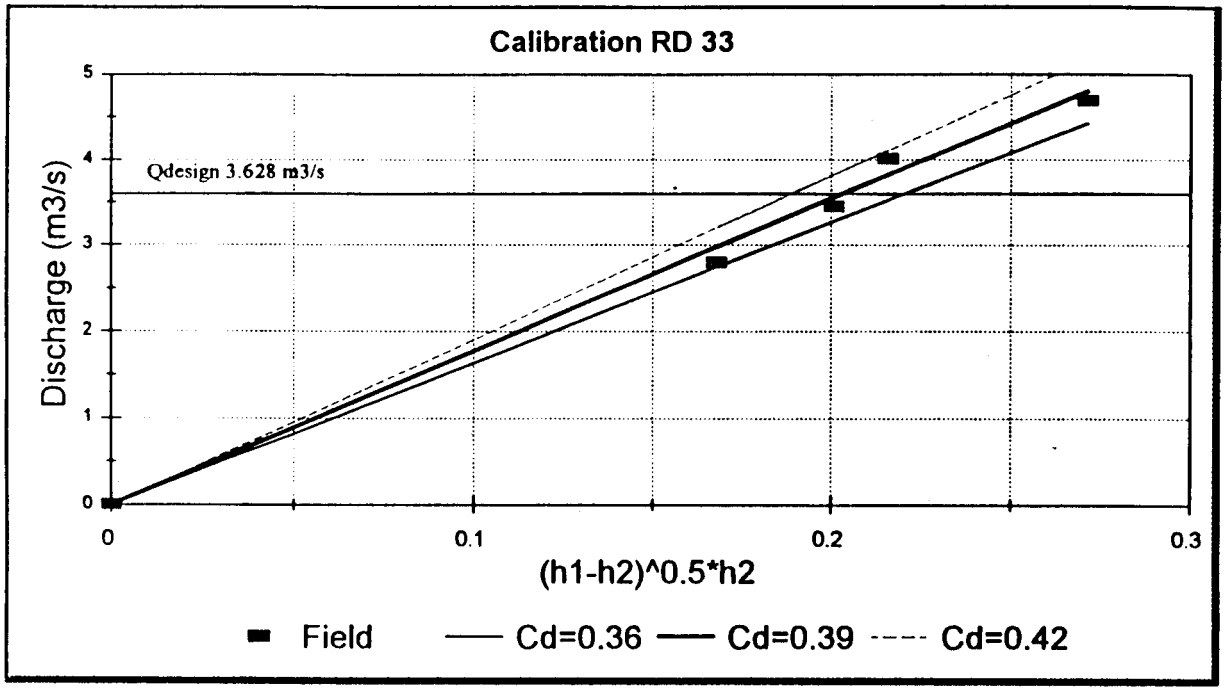


Figure 25 calibration drop structure at RD 33

RD 65

The head loss over the drop structure at RD 65 amounted to only 2 cm which makes a calibration virtually impossible. Therefore the discharge coefficient for this structure was determined by taking the average value of the discharge coefficients at RD 15 and RD 33. Since the structures are alike and the results of the two former were almost the same, this seems a plausible solution.

Jiwan Minor

The offtake of Jiwan Minor was calibrated by taking five discharge measurements. Observation in the field showed that for one of the measurements (with the lowest discharge) the structure was not running under free flow conditions. The other four were taken under free flow conditions. The discharge coefficient corresponded with the theoretical discharge coefficient of 0.38 (see figure 26).

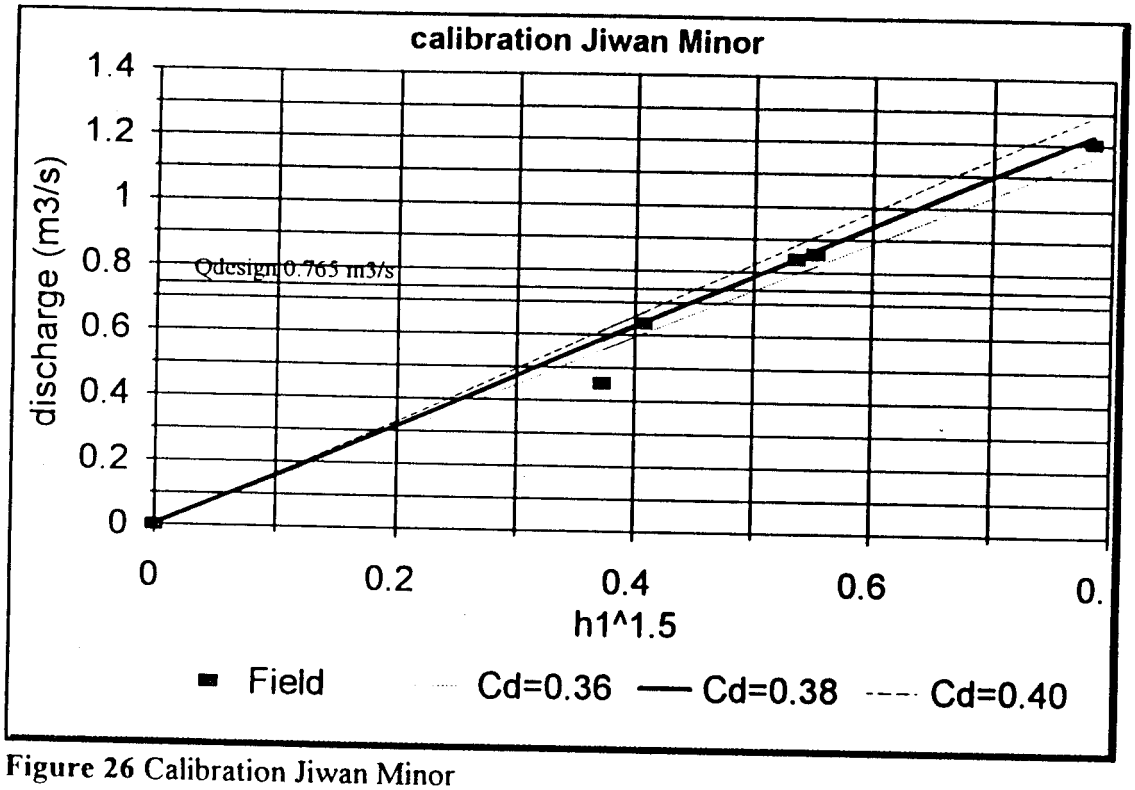


Figure 26 Calibration Jiwan Minor

Outlets

The next step was to calibrate a number of outlets and to extrapolate the results to all other outlets. The outlets that were measured and the resulting μ values (for the SIC - equation) were:

Outlet	μ value
14320-R	0.60
29690-R	0.45 - 0.60
57640-L	0.60
60000-L	0.57
68260-R	0.60
78850-R	0.68

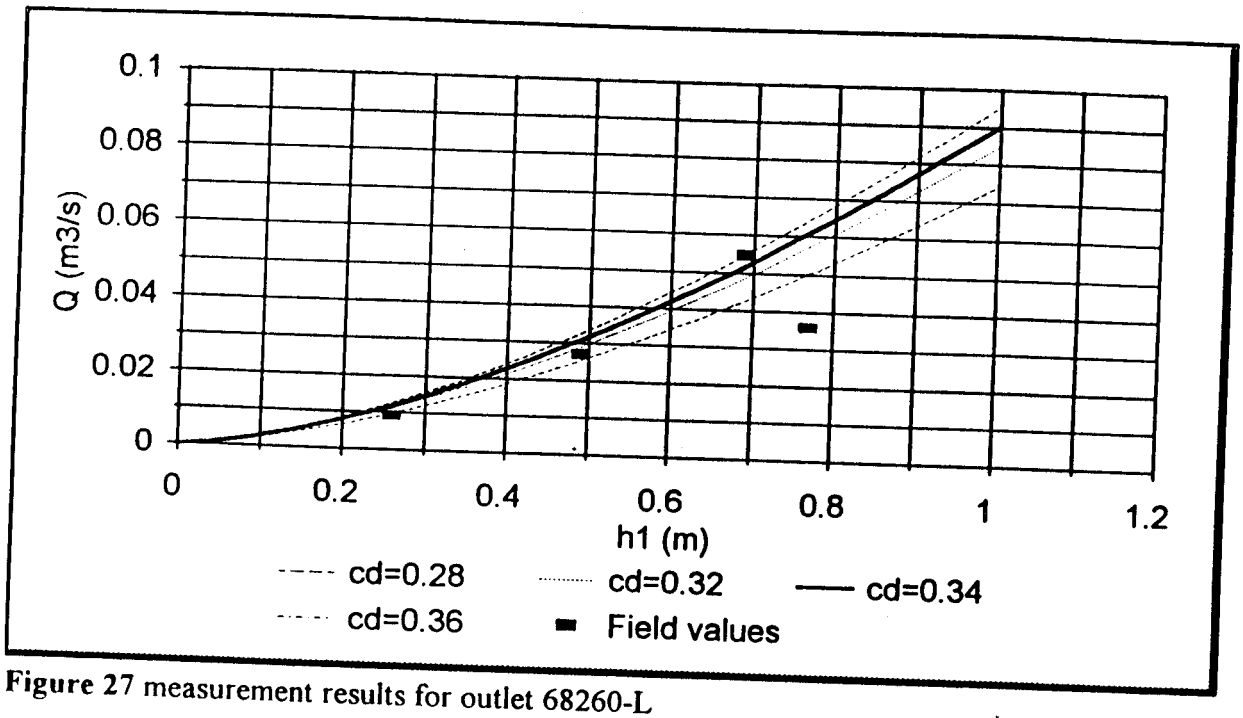


Figure 27 measurement results for outlet 68260-L

Table 4 Results of outlet calibrationIn figure 27 an example of measurements results is given for outlet 68260-L. One discharge measurement is clearly out of range.

As can be seen from the table and the figure the results are scattered. The reason for this was twofold: First of all the choice of the outlets was not the most appropriate. This became clear only after the field measurements had begun. The second reason was the irregular inflow at the head. This resulted in many days in which the channel was completely empty and in the days when there was water, the levels were fluctuating very fast, which resulted in some useless measurements. This is why eventually it was decided to take a different approach.

5.3.3 Second approach

In the second approach things were done differently. Instead of looking at the outgoing discharges, the ongoing discharges were measured.

For two days the water level upstream of the cross structure at RD 15 was measured every half hour. Because the discharge coefficient of this structure had been determined in the first approach, the discharge over these two days was known. These measurements were converted in an hydro graph for SIC. This hydro graph has been given in figure 13 (chapter 2). Because the inflow was not constant in time, the dynamic module in SIC had to be used.

On the second day discharge measurements and water level measurements were done on various locations along the channel. These measurements were compared with the results that the model gave with the inflow file over these two days. The different variables were adjusted in order to match the results that the model produced at these locations with the field measurements.

The following measurements were done:

- ❑ RD 33 — At this location the up- and down stream water levels were measured .. times. Because the discharge coefficient was determined earlier, the discharges were also known
- ❑ RD 65 — At this location the up- and downstream water levels were measured. Next to that four discharge measurements were done just downstream of RD 65
- ❑ RD 107 — At this location the discharge was measured four times. The water level was measured four times as well. There is no cross structure here so there is only one water level to measure.

Two reaches of the channel were now analyzed separately. For reach RD 15 to RD 65 the locations RD 33 and RD 65 were used as checkpoint. For the reach RD 65 to RD 107 the measurements at RD 107 were used as checkpoint.

For the reaches RD 15 - RD 65 and RD 65 to RD 107 the two steps given below were repeated. Because step two will affect the discharges, the two steps had to be repeated until both the discharges and the water levels were correct.

1) Adjust the discharge coefficients of the outlets and the seepage until the discharges at the checkpoint(s) are correct.
2) Adjust the value for the Manning-Strickler coefficient until the water levels are correct.

For each reach separately there are many combinations of the discharge coefficients of the outlets and the seepage which will produce the correct discharge at the downstream checkpoint. Next to that the value of the Manning-Strickler coefficient downstream of the analyzed reach also affects the water levels at the checkpoint. In order to overcome these problems both the seepage and the Manning-Strickler coefficient were taken as uniform over the whole channel.

The final result was obtained after many iterations. To give an idea of the accuracy that was obtained in this way, the following graphs (28 to 33) are presented which give the water levels and the discharges at the checkpoints for the uncalibrated model as well as the final, calibrated version.

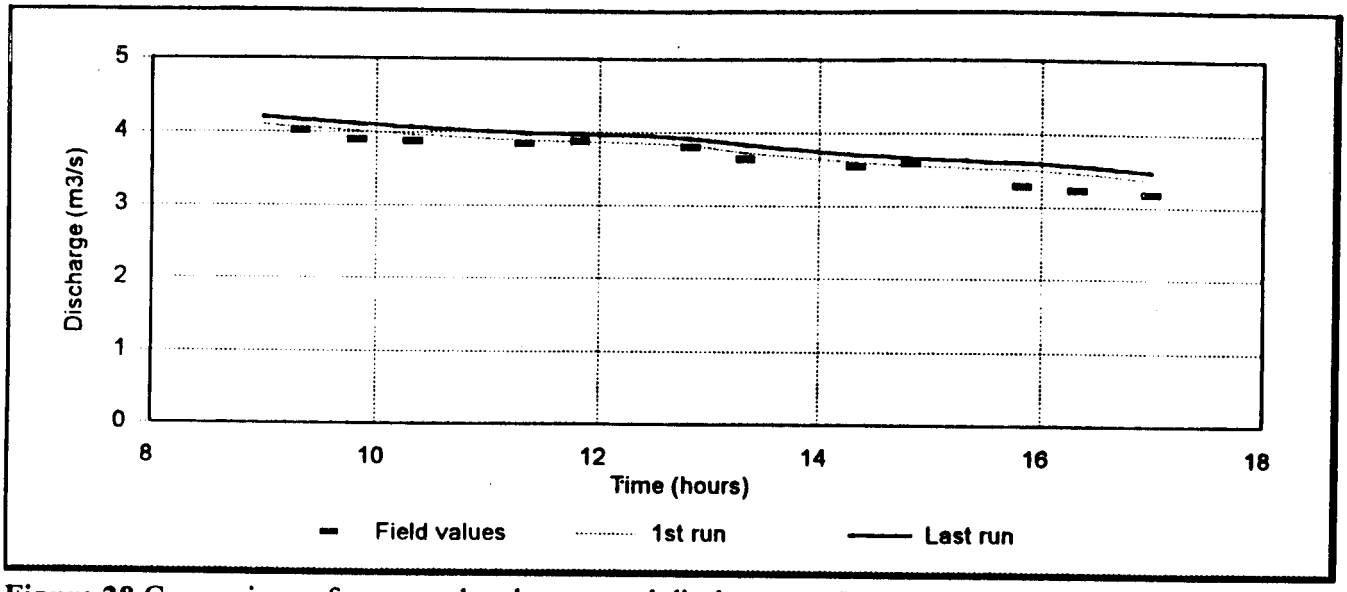


Figure 28 Comparison of measured and computed discharges at RD 33

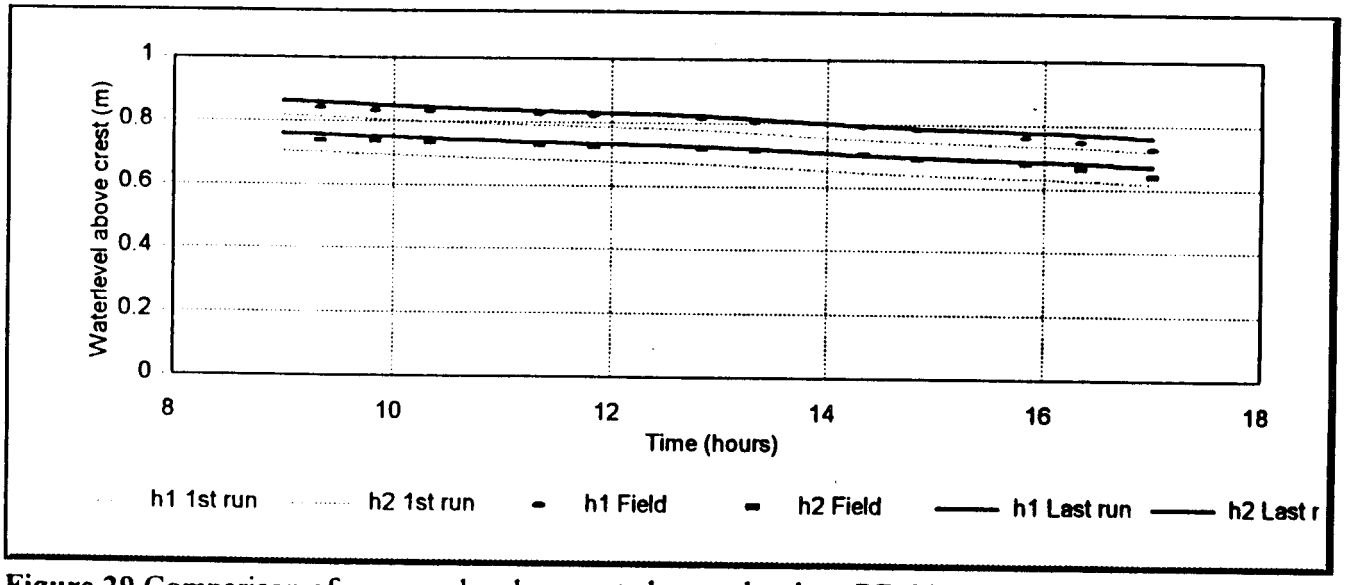


Figure 29 Comparison of measured and computed water levels at RD 33

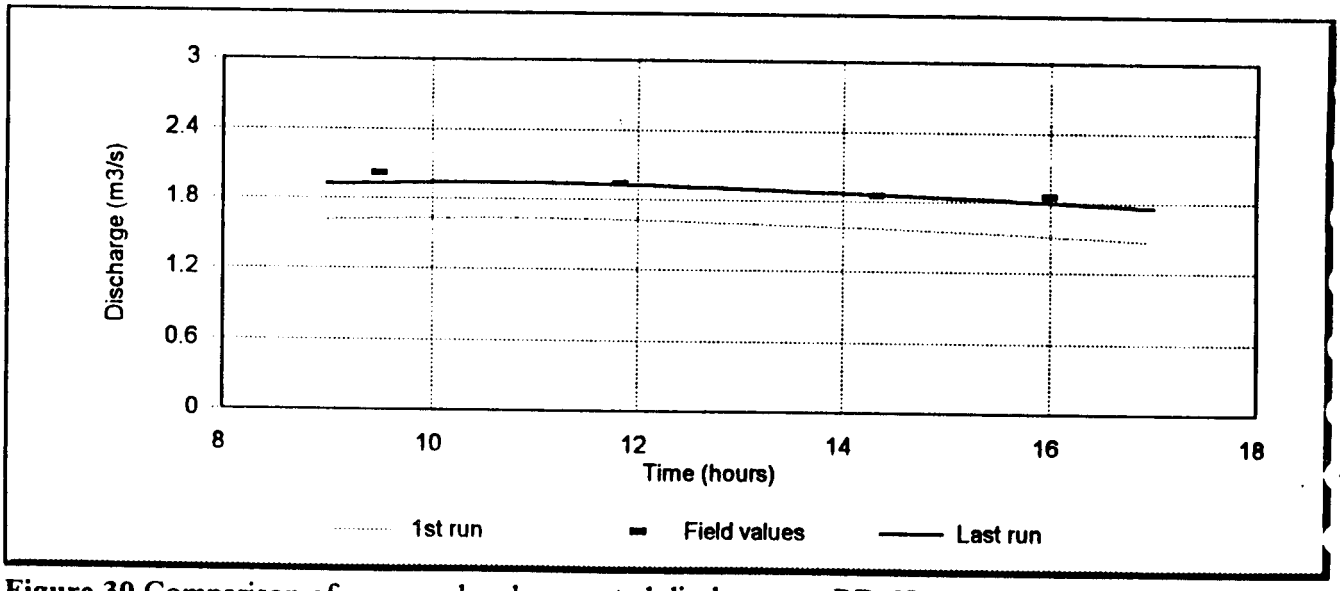


Figure 30 Comparison of measured and computed discharges at RD 65

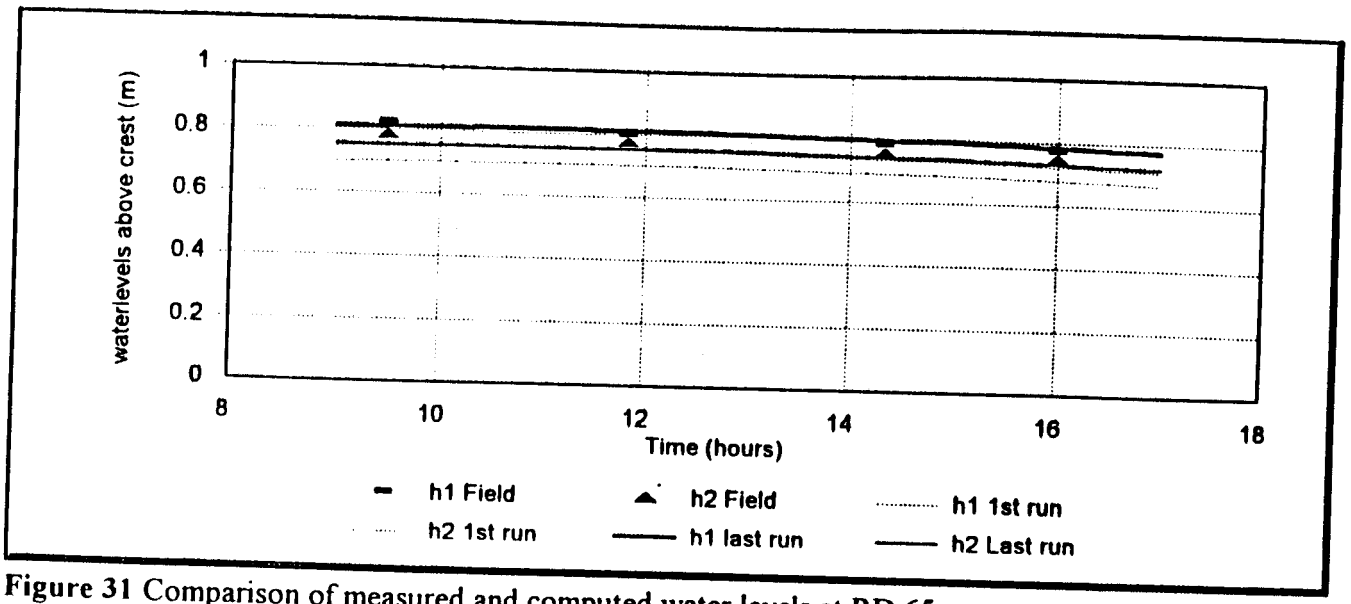


Figure 31 Comparison of measured and computed water levels at RD 65

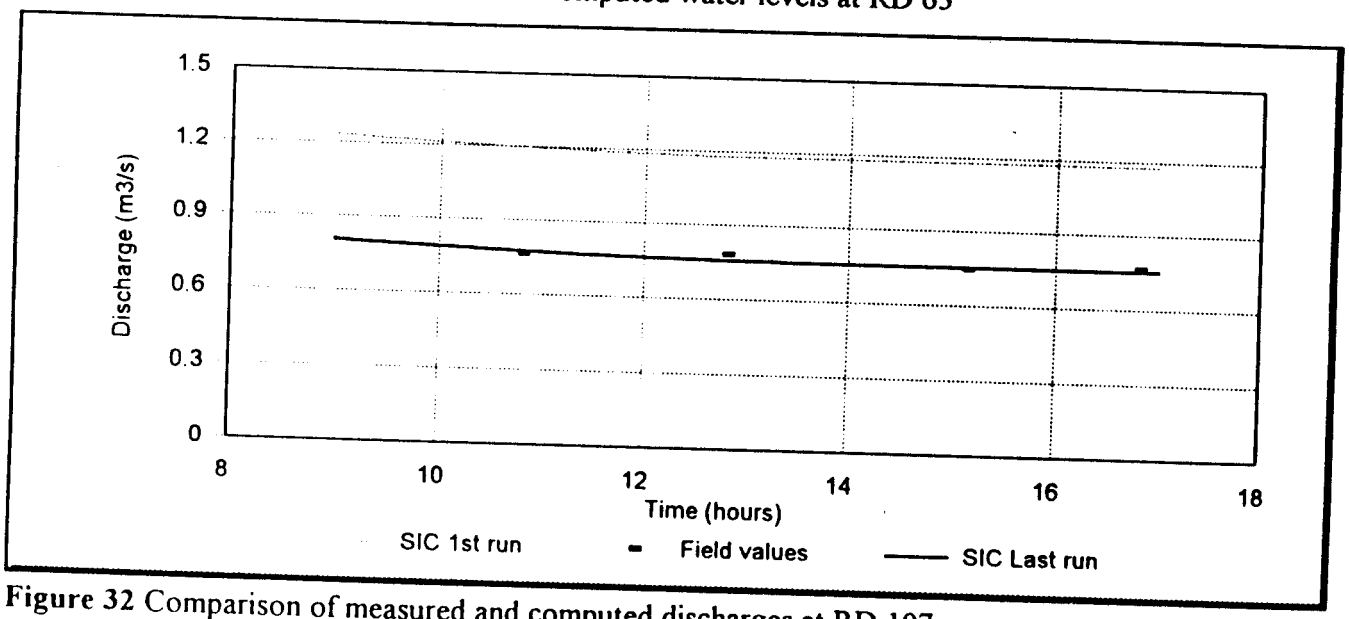


Figure 32 Comparison of measured and computed discharges at RD 107

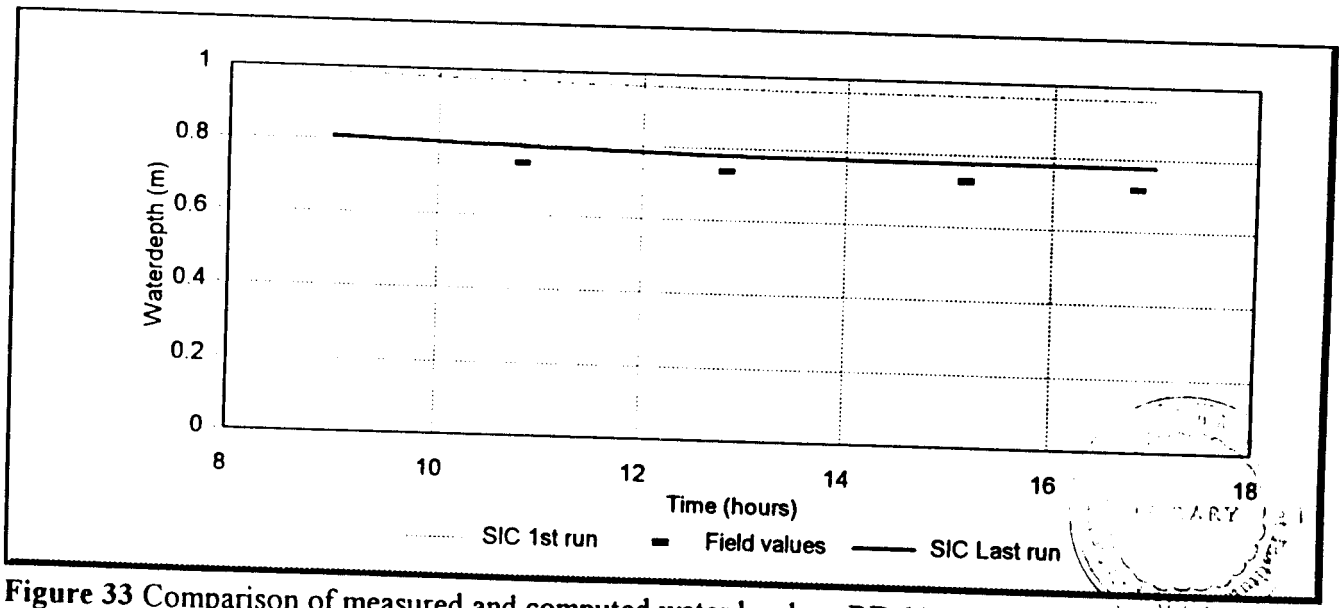


Figure 33 Comparison of measured and computed water levels at RD 107

5.4 Results

The calibration of the structures with the first approach together with the second approach gives satisfactory results for the values of the variables of the model. This model can now be used to simulate various kinds of maintenance on the channel.

The results obtained with the calibrated model will be shown in this paragraph. The results for the seepage are that 4.9% of the incoming discharge is lost to seepage. This is lower than the figure currently used by the Irrigation Department. It means that when the discharge coming from Fordwah Branch is equal to design, there is more water to distribute than assumed by the Irrigation Department.

The value for the Manning-Strickler coefficient turned out to be 0.026. This seems a normal value.

The tail of the distributary is reached when the incoming discharge is 3.8 m^3/s. This is 85% of design discharge. It is difficult to say wether this is good or bad because there is no comparison.

The water distribution at design discharge is given in figure 34. In this graph the situation regarding the equity is given. The distribution is **not** equitable. Although the results are quite scattered, it can be said that the middle reach gets too much water while the head and tail reach do not receive enough.

It is quite difficult to get a clear picture of the situation from this graph. In figure 35, 36 and 37 the same results are presented in a different way. In the graph 35 the absolute values of the differences between actual and design discharges are presented. In the graphs 36 and 37 the discharges running through the distributary are shown and compared with the design discharges. In figure 36 the absolute values are given, in figure 37 the percentages are shown. From these graphs it becomes clear that up to RD 65 the discharge in the distributary is higher than intended, and downstream from RD 65 it is lower than designed. This jump is predominantly caused by the outlet at 60000-L and Jiwan Minor, which receive more water than they should.

The relative shortage increases in the direction of the flow. As expected Jiwan Minor gets more water than its fair share.

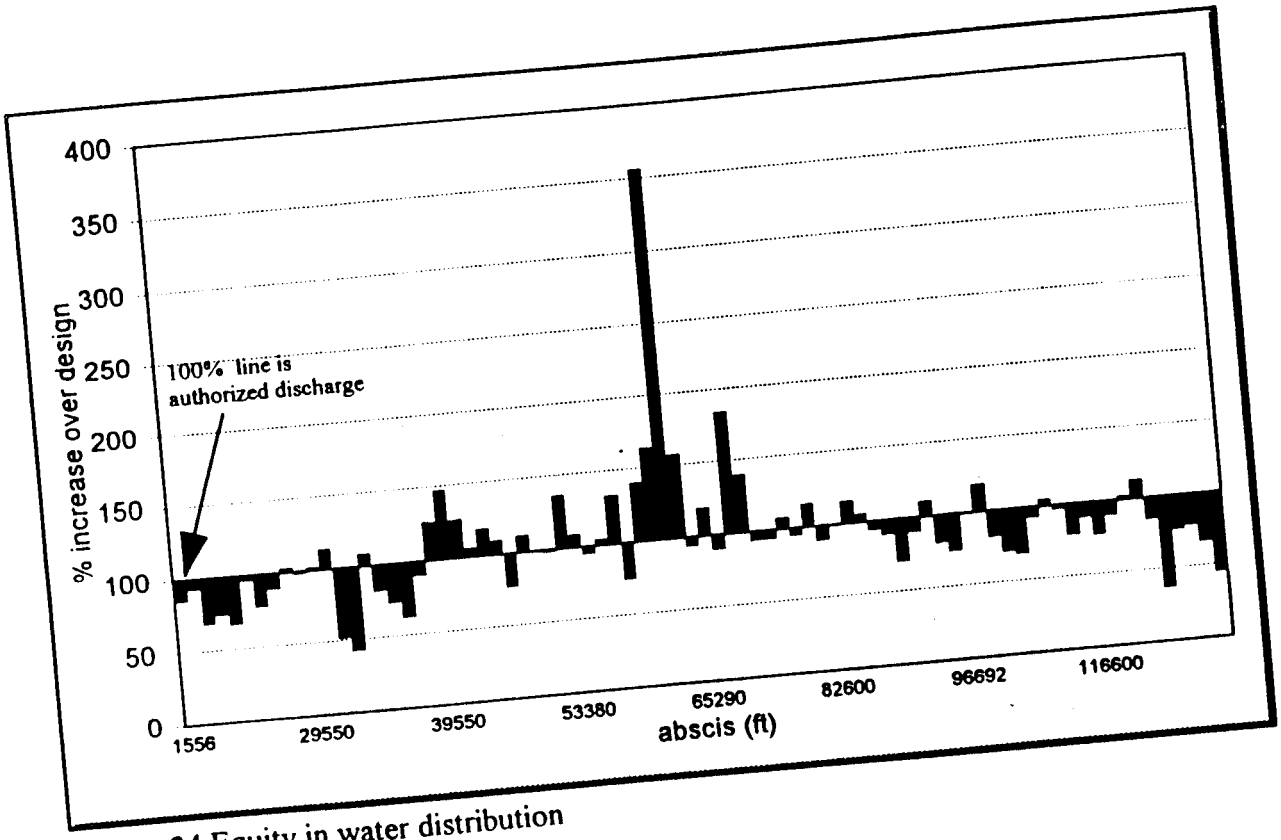


Figure 34 Equity in water distribution

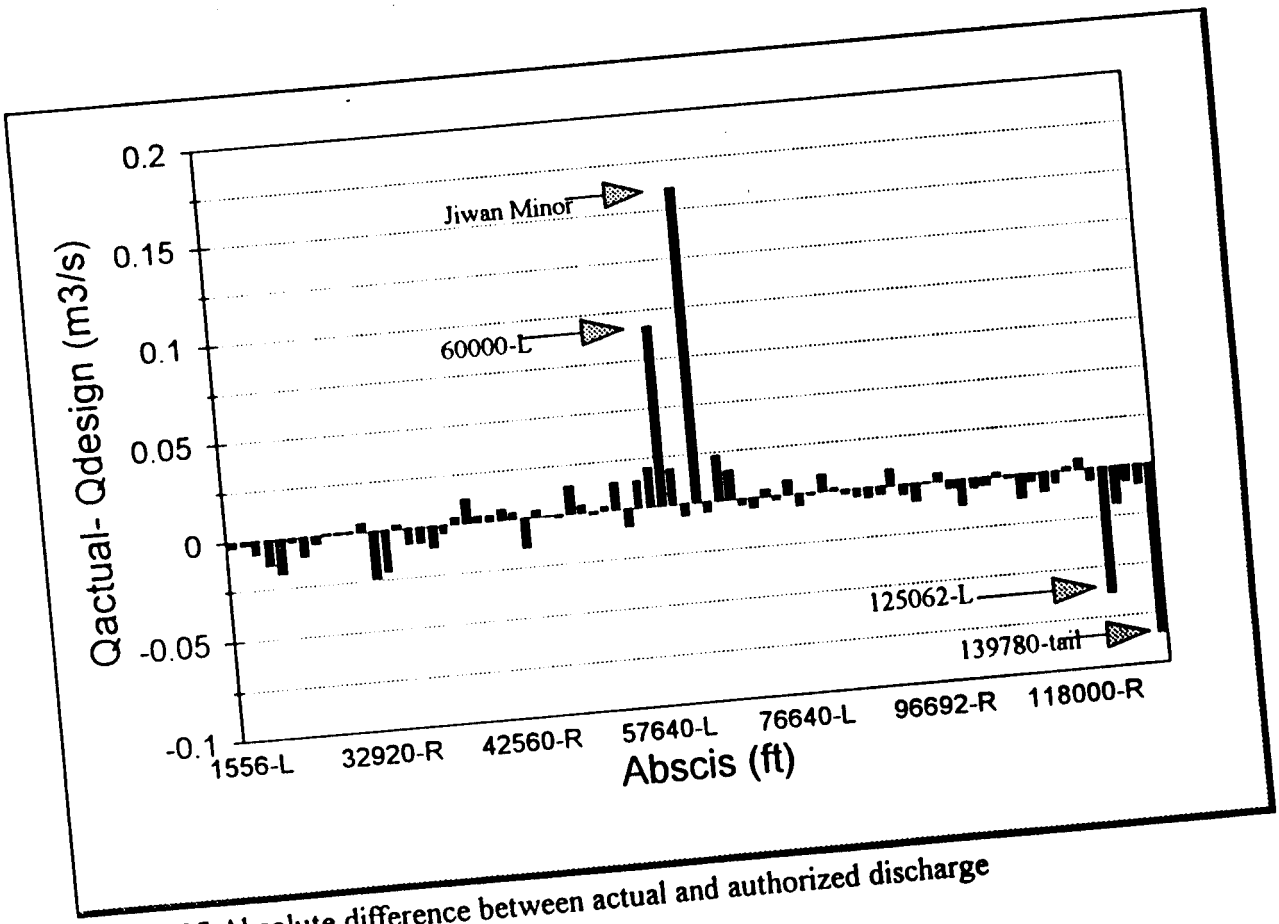


Figure 35 Absolute difference between actual and authorized discharge

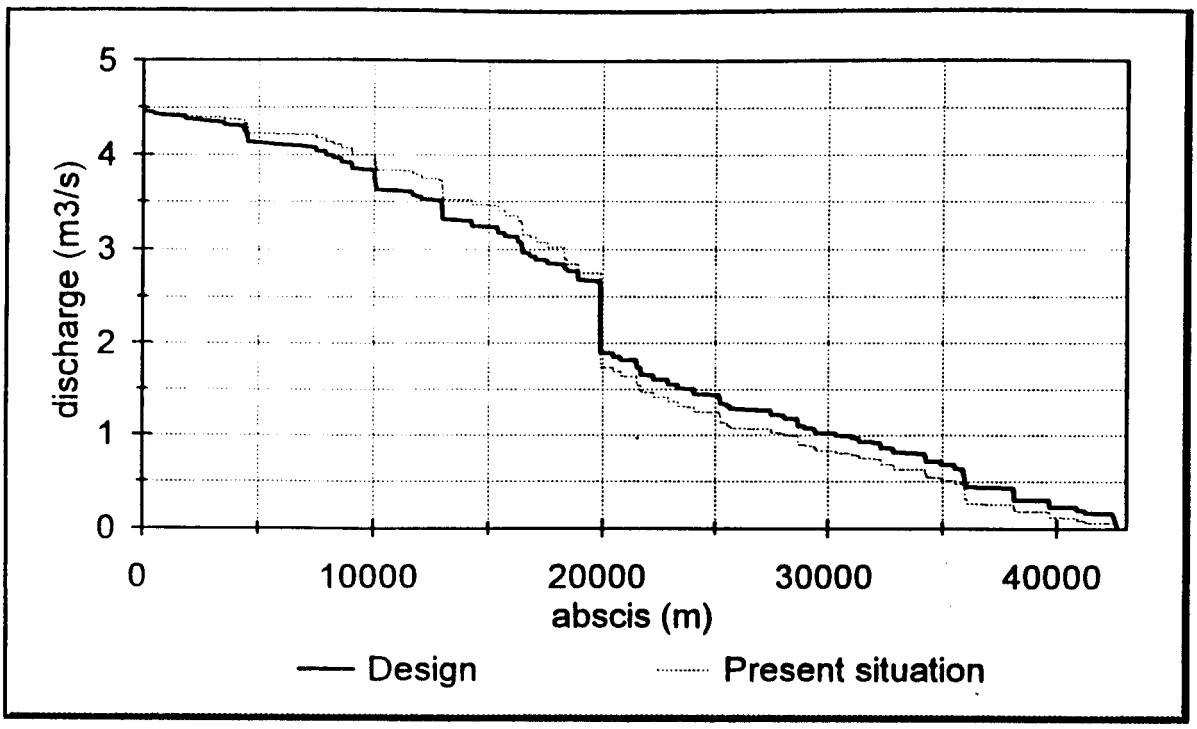


Figure 36 Total discharge in the distributary design and actual

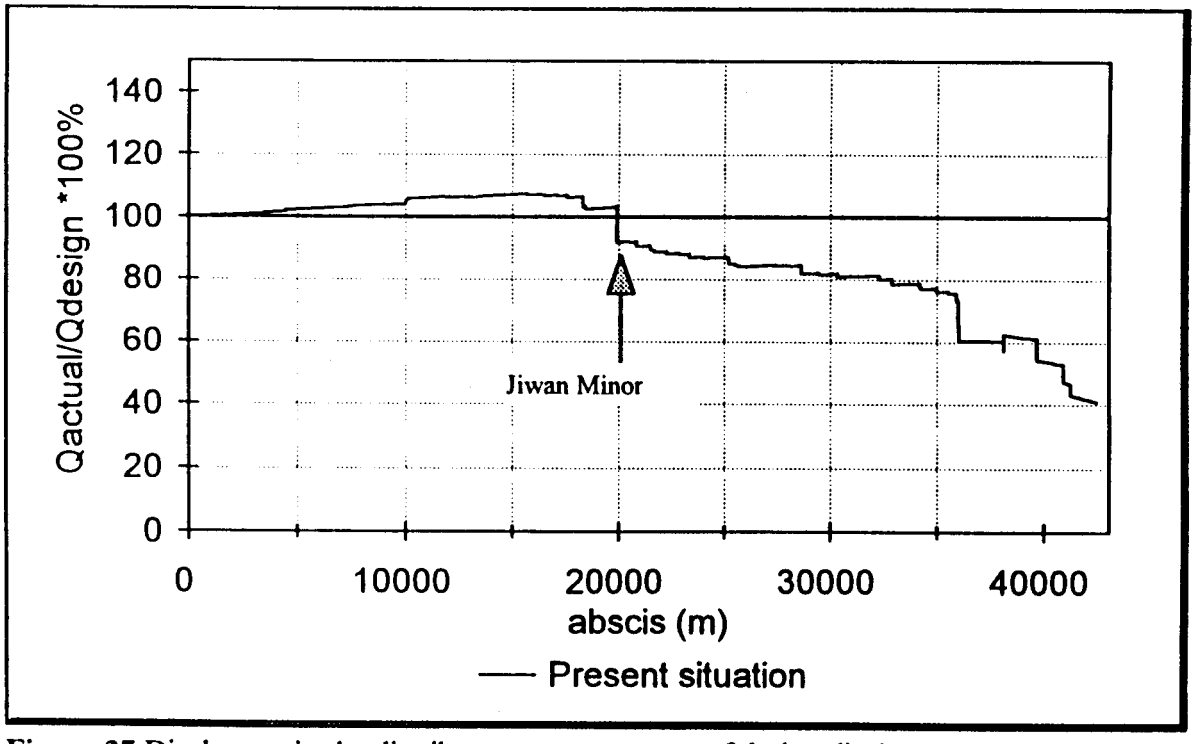


Figure 37 Discharges in the distributary as percentage of design discharge

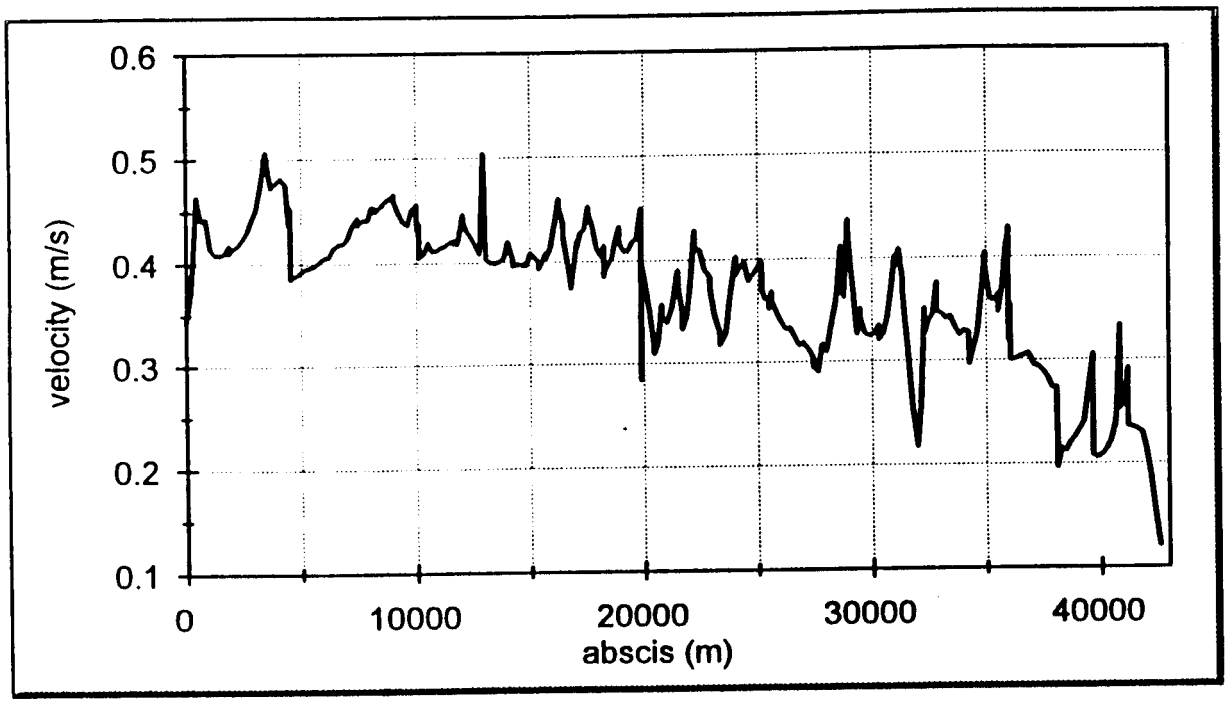


Figure 38 Velocities at Full Supply Discharge

The velocities are given in figure 38. The average velocity decreases in the direction of flow. There are sharp fluctuations which are difficult to explain from a physical point of view. The figure does lead to some understanding why the current maintenance of this distributary is focused on the tail reach. Here the velocities are the lowest, and more siltation should occur.

Another result is figure 39. This shows the available freeboard when running at FSD. It shows that in the region between RD 54 And RD 58 there is just 2 cm on average.

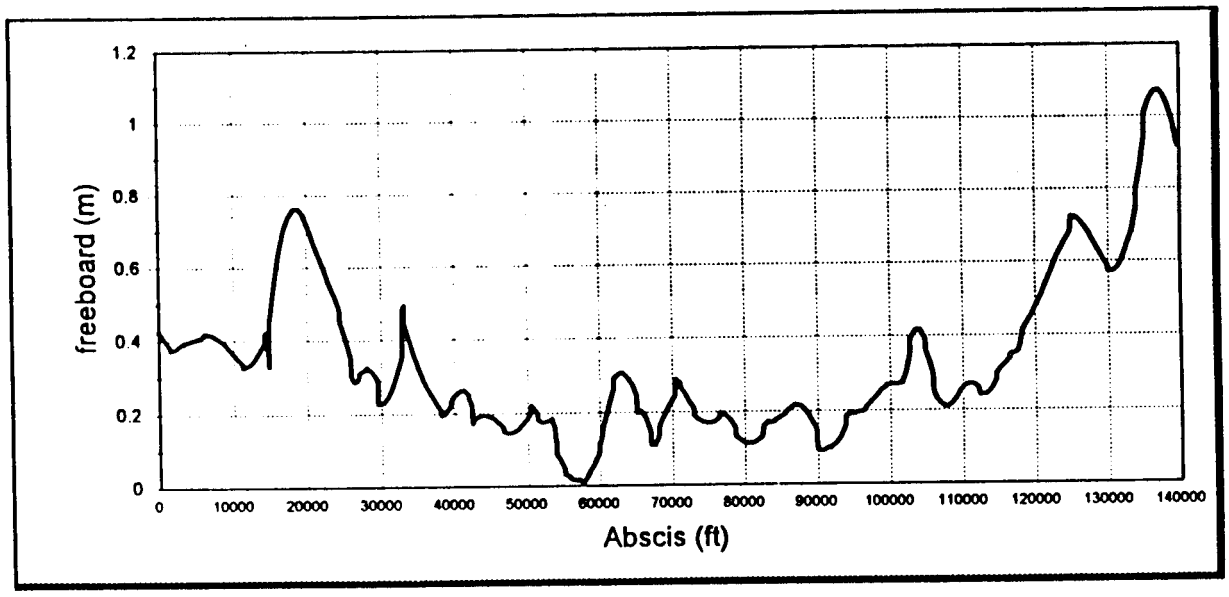


Figure 39 Freeboard

5.5 Limitations of SIC

5.5.1 Number of nodes

One of the limitations which had to be dealt with was the limited amount of nodes that SIC can handle. The maximum in this version of SIC was 80. Fordwah Distributary has 87 outlets. In order to overcome this problem three small outlets were left out of the model. Eight other outlets were combined to four. The outlets that were combined had the same crest levels and the same heights. The widths were added up.

5.5.2 Structure formulas

The formulas which SIC uses for cross structures led to some problems.

a) Weir-orifice transition

First of all the transition between weir flow and orifice flow has to be discussed. In SIC this transition takes place for $h_1 = W$ (see figure 18). However, in reality critical flow occurs above the crest. In a free flow situation the transition will not take place for $h_1 = W$ but for $h_1 = 1.5$ W. This means that when in the field an outlet is operating as a weir, in SIC it can be calculated to be operating as an orifice.

b) Crumps module

Another problem is the formula that SIC uses for APM's. It is given below

$$Q = B * \sqrt{2g} * (\mu * h_1^{3/2} - \mu_1 * (h_1 - W)^{3/2}) \qquad h1 > w,\ h2 < \frac{h1}{1 + 0.14 * \frac{h1}{w}}$$

with: $$\mu = \mu_F + 0.08 * (1 - \frac{1}{\frac{h_1}{W}})$$

and: $$\mu_1 = \mu_F + 0.08 * (1 - \frac{1}{\frac{h_1}{W} - 1})$$

The problem with this formula is that the discharge coefficient in this formula is not constant for variable upstream water levels. The APM however, was specifically designed to have a constant discharge coefficient. It has a rounded roof to prevent the jet from contracting.

The classical formula is : $Q = Cd * B * W * \sqrt{2g} * \sqrt{(h_1 - w)}$

There is a discrepancy between these two formulas. The result is that the calibration of the different outlets becomes very difficult. In fact the calibration can only be correct for one specific upstream water level. In figure 40 the two functions are shown. One is the formula used by SIC, the other is the classical formula for an APM. The two functions can only match in one point.

c) OFRB

Another problem is that there is a difference between the hydraulic behavior of an APM and an OFRB. This difference cannot be simulated in SIC. For an OFRB the transition between open flow and orifice flow is not continuous. The equation for an OFRB is given below:

In SIC the transition between weir flow and open flow is always continuous. For the calibration this results in the same problem as with the APM. In figure 40 the equation for the OFRB is also shown. The calibration can only be correct for one upstream water level.

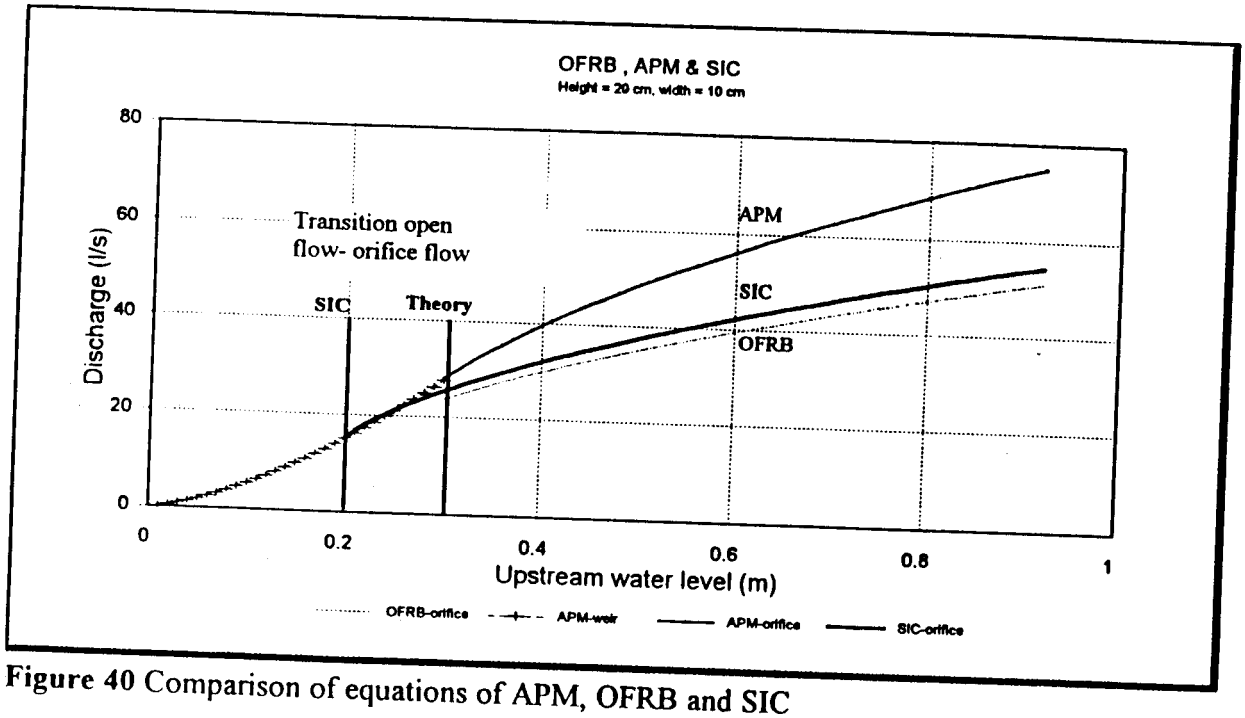


Figure 40 Comparison of equations of APM, OFRB and SIC

All the above mentioned problems have in common that reality is more complex than computer simulations. With the present modelling SIC can not completely cover all the characteristics of outlets. The above mentioned improvements could be made, though.

5.5.3 Tail dry problems

In Fordwah Branch there is a structural shortage of water. As in all upstream controlled systems this leads to deficiencies at the tail. This applies to Fordwah Distributary as well. Due to reduced inflow and increased outflow the tail of Fordwah Distributary often runs dry. SIC cannot handle this problem. If the tail runs dry the computation stops, both in steady and unsteady flow.

If distributaries like this are to be analyzed completely, a solution has to be found for this problem. Only if this is done a complete picture of the performance of the channel can be obtained.

It would be interesting to have the possibility of being able to simulate how far the water will go for a certain incoming discharge.

6 Simulations of proposed measures

6.1 Modeling maintenance

When maintenance is modeled in a hydraulic model there are only a limited number of parameters which can be changed. It is important to realize what the options are. At the same time the final goal must be kept in mind: we are looking for maintenance scenarios which increase the equity in water distribution. A completely equitable distribution is the situation in which the Effectivity is equal to 100%, as has been defined in section 3.3. In other words, each outlet receives it's authorized discharge.

The parameters which can be changed are the same as has been mentioned several times earlier, namely:

1 Channel	Widths, depths, bed slope and slopes of banks can be changed. Next to that the Manning-Strickler coefficient can be changed.
2 Outlets:	Although maintenance strictly spoken does not comprise adjustments of outlets, they do belong to the available options for the Irrigation Department in controlling the water distribution. The outlets can be changed in size or setting. In the present situation this is frequently done by the ID. With this model it is possible to analyze the effects of changes to outlets on the water distribution downstream of the outlet.
3 Cross structures	Changes to cross structures are as easy to simulate as modifications of outlets. Basically only the width and the crest level can be changed. In the present situation changes to cross structures do not occur very often, as has been mentioned earlier.

Following the results of the previous chapter, observations in the field and a global idea of what kinds of maintenance could improve the equity the following scenarios have been simulated:

1 Desilting of the entire channel to design bed level:
In this simulation the bed level of the entire channel is restored to the design bed level. The outlets are left as they are and the other dimensions of the channel such as the widths are also left untouched.

2 Bringing the bedlevel and the width back to design:
In this simulation all the dimensions of the channel are restored to the design situation. The outlets are not changed.

3 Desilting of the entire channel by a certain amount: cm

4 Silting up of the channel by 5 cm
This simulation is mainly done to see the effects of absence of maintenance.

5 Reducing the size of outlets that receive more than 30% extra to design discharge.

6 Desilting of the channel from RD 65 to tail by such an amount that the free flow conditions at RD 65 are restored.

7 Changing of the Manning-Strickler coefficient from 0.026 to 0.023
Although there is no determined physical equivalent of maintenance which could effect this change, it is interesting to have an idea of the effects of changes in roughness on the distribution.

In order to draw conclusions the graphical representation of the results of simulations is a practical but important aspect. Because the possibilities of the graphical representations are numerous a choice has to be made. On the x-axis the position along the distributary is used. Below the possibilities for the y-axis are given .

Kind of information	Y-axis	
Water levels in distributary	$h_{new}(x) - h_{actual}(x)$	[m],
	$h_{new}(x) - h_{design}(x)$	[m],
	$h_{new}(x)$ together with $h_{design}(x)$ and $h_{actual}(x)$	[m]
Discharges in distributary	$Q_{new}(x)$ together with $Q_{design}(x)$ and $Q_{actual}(x)$	[m³/s]
	$Q_{new}(x) - Q_{design}(x)$	[m³/s]
Discharges through outlets	$Q_{new}(x)$ together with $Q_{design}(x)$	[m³/s]
	$Q_{new}(x) - Q_{design}(x)$	[m³/s]
	$(Q_{new}(x) - Q_{design}(x))/Q_{design}(x) * 100\%$	[%]
	$(Q_{new}(x) - Q_{old}(x))/Q_{old}(x) * 100\%$	[%]

In this report the last two options are used. This because the discharges through the outlets are the most important output.

If the results of the simulations are to be compared it is necessary to have one value for each simulation which represents the performance of the total distributary . This value is the Effectivity as has been explained in chapter 3.3.

6.2 The effect of current maintenance

In the current situation the focus of the maintenance is on the tail portion of the channel, i.e. from RD 65 to the tail. Several simulations have been performed to check the effects of this maintenance on the water distribution.

One of these is displayed in the figures 41 and 42. Here the desilting of Fordwah Distributary

from RD 65 to the tail is given. The discharges running through the outlets at the tail of the channel clearly increase but the discharges through the outlets between RD 65 and RD 116 decrease.
The Effectivity with this kind of maintenance decreases from 89.3% to 87.3%.

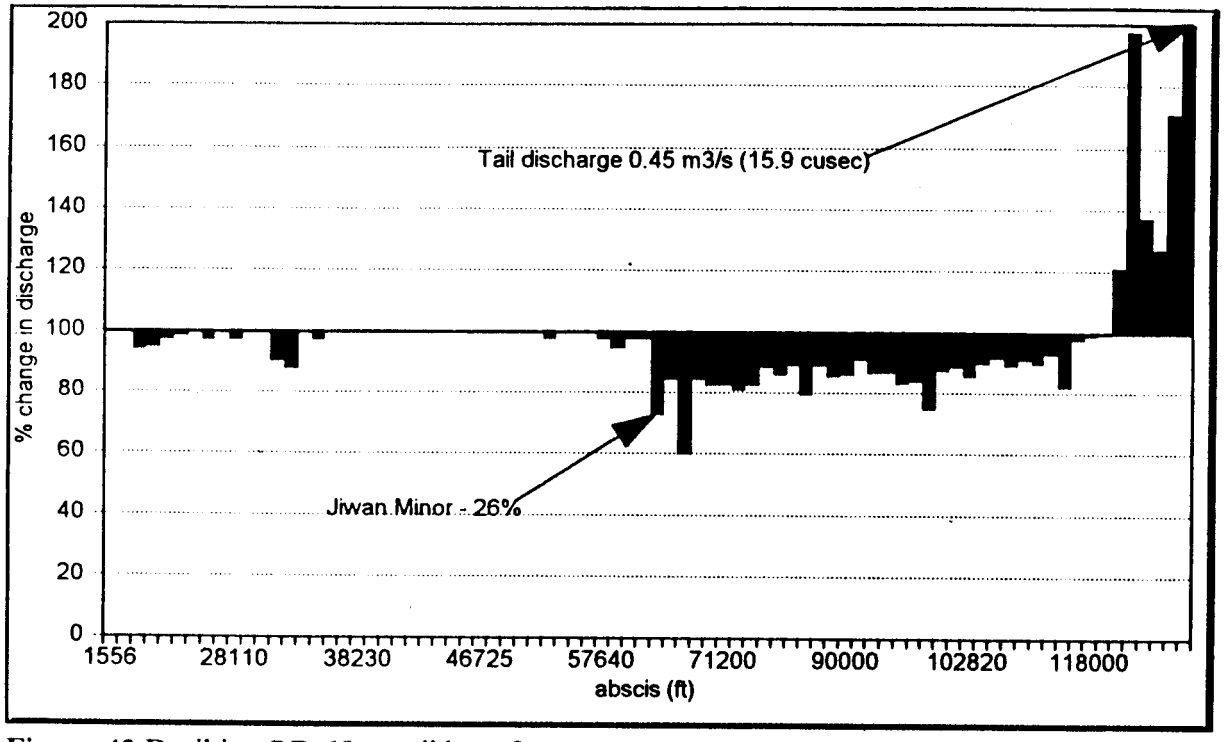


Figure 42 Desilting RD 65 to tail by 1 foot

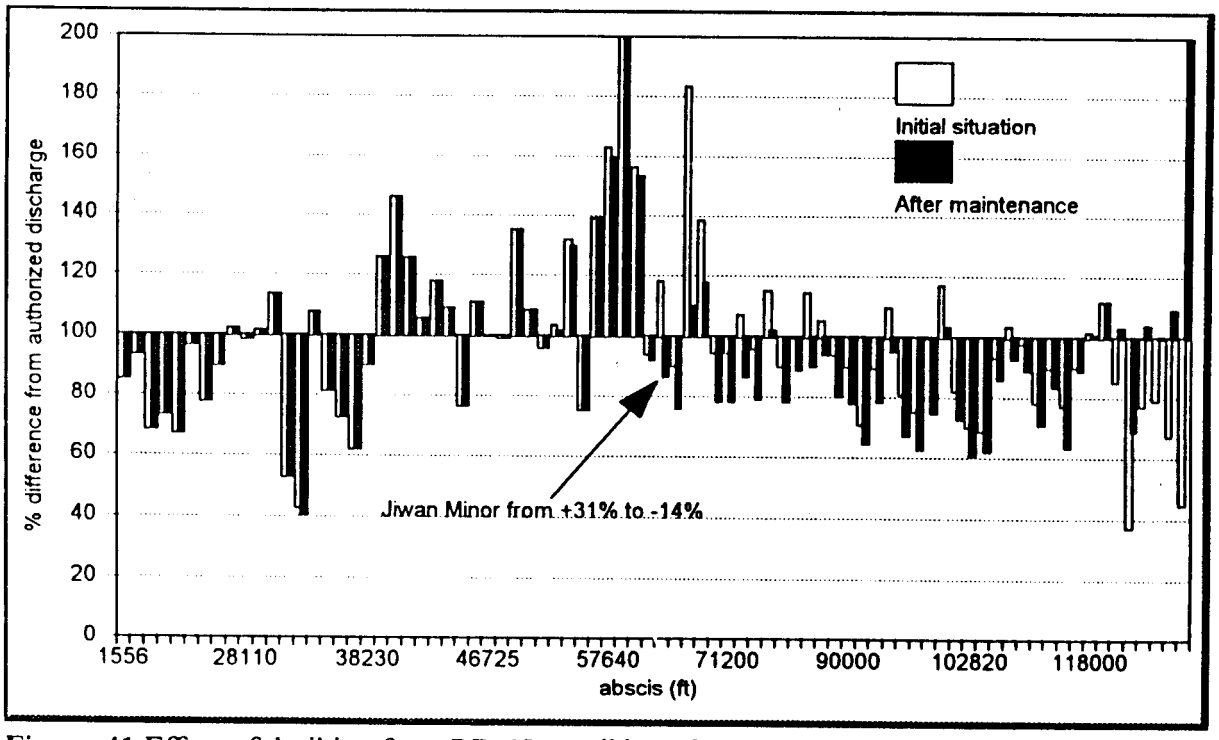


Figure 41 Effect of desilting from RD 65 to tail by 1 foot on the equity

Because of the desiltation downstream of RD 65 the water level at RD 65 drops. The structure at RD 65 is submerged, however. The discharge through Jiwan Minor reduces considerably.

6.3 The effect of measures of siltation and desiltation

In order to get an idea of what siltation does to the distribution a simulation has been carried out to see in what way 5 cm of siltation changes the distribution. This is shown in figure 43.

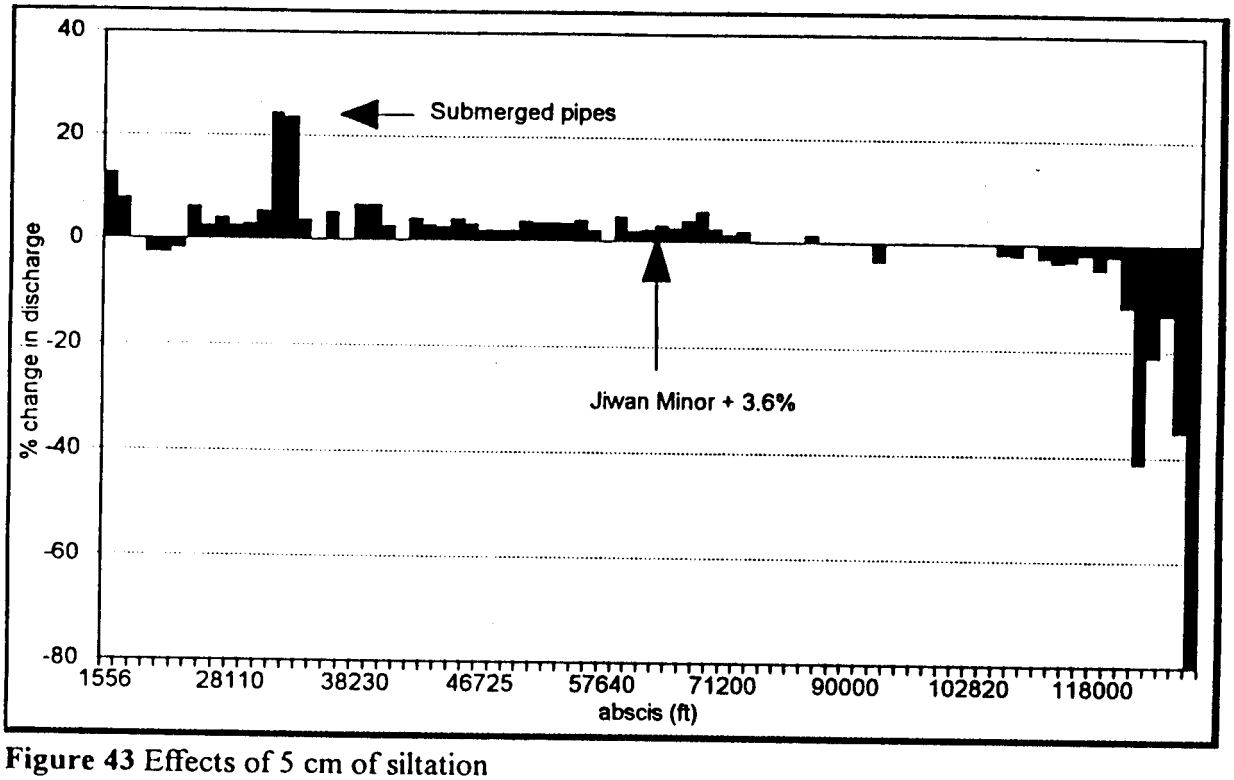


Figure 43 Effects of 5 cm of siltation

Because of the raising of the water level, outlets which are located on the upstream portion receive more water and outlets which are located on the downstream side receive less. At the tail of the distributary there is hardly any water left. The total effective supply becomes 87.6% which is a marginal decrease.

In the same way the effects of desiltation can be shown. This can be seen in figure 44.

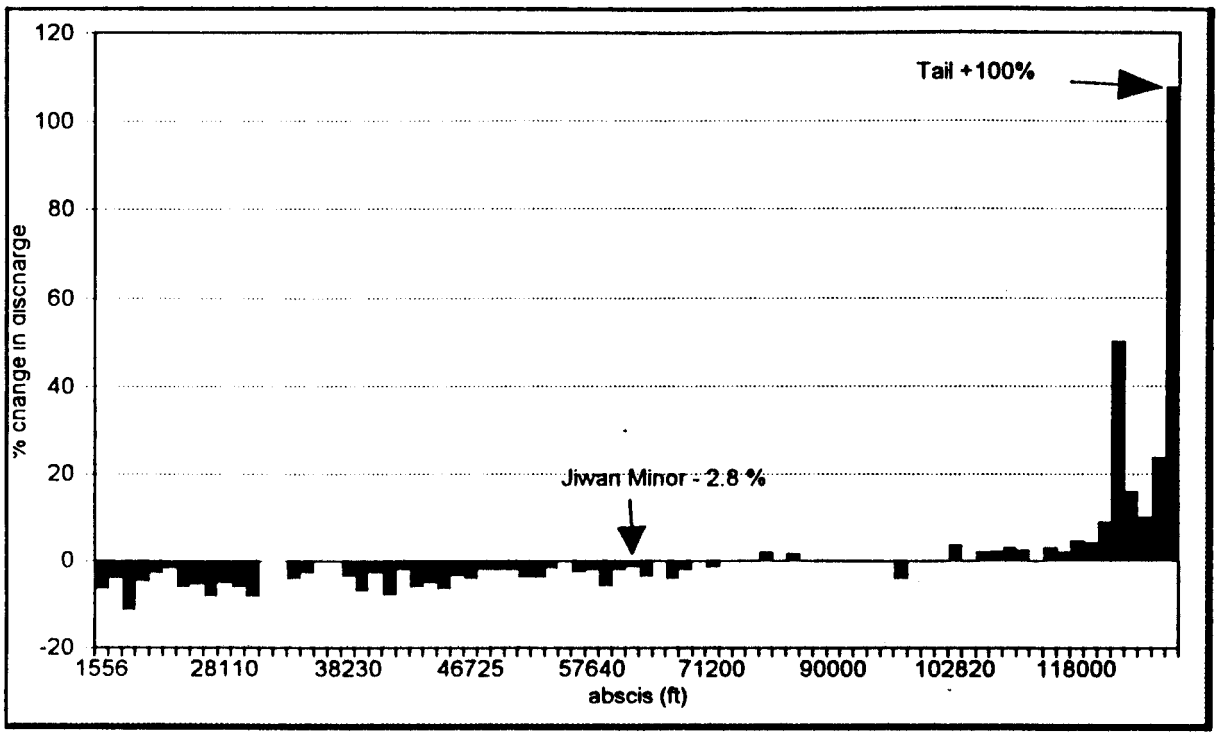


Figure 44 Effects of desilting Fordwah Distributary by 5 cm

It can be seen that a small amount of desilting increases the discharges at the tail sharply. The total effective supplied discharge increases from 89.3 to 90.4 which is only a marginal improvement.

6.4 The influence of reducing outlets

One possibility of improving the current situation would be to reduce outlets which receive too much. There are 9 outlets which receive more than 30 % over their authorized discharge. If we just reduce these outlets and not do anything else it could be possible to improve the effectivity.

The outlets which are reduced are: 39550-R, 51500-L, 54080-R, 56000-L, 57640-L, 60000-L, 60410-L, 68260-L and 70530-R. In figure 45 the impact of such a change is shown. In figure 46 the effects on the equity are given.

The effectivity of the entire channel increases from 89.3% to 91.5%. Again, this is only a marginal increase.

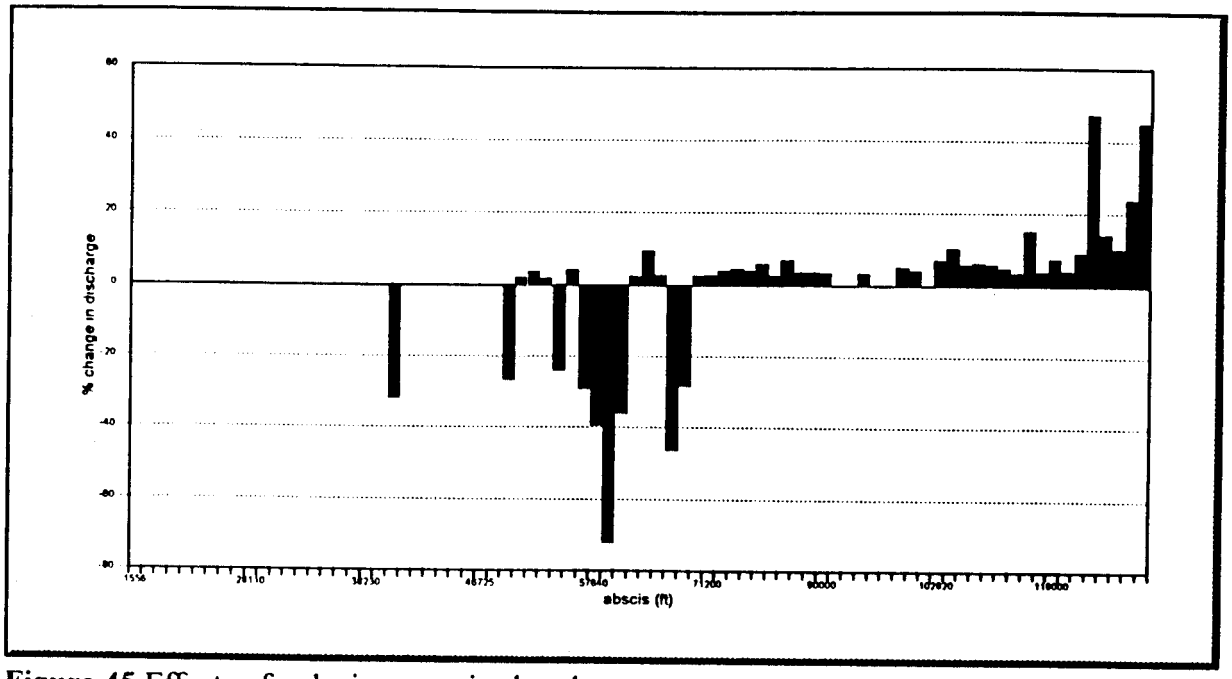


Figure 45 Effects of reducing oversized outlets

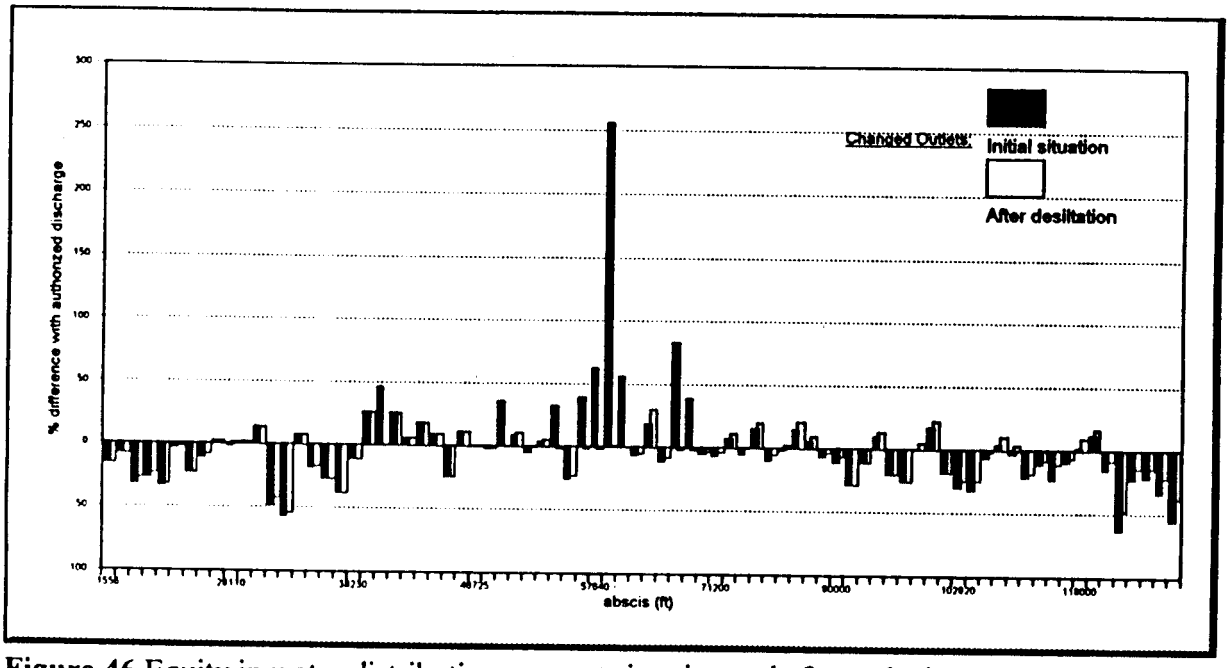


Figure 46 Equity in water distribution: present situation and after reducing 9 outlets

6.5 Result of raising crest at RD 65 and decreasing size of outlets

In this simulation two objectives have been combined: restoring the free flow situation at RD 65 in order to restore the proportional behavior of the structure and the reduction of outlets which draw an excessive discharge (see figure 47).
The result of this simulation is that the effective supply increases from 89.3% to 94.0%.
One result of the raising of the crest at RD 65 is that overtopping takes place immediately upstream of RD 65. The banks at this location are not high enough.

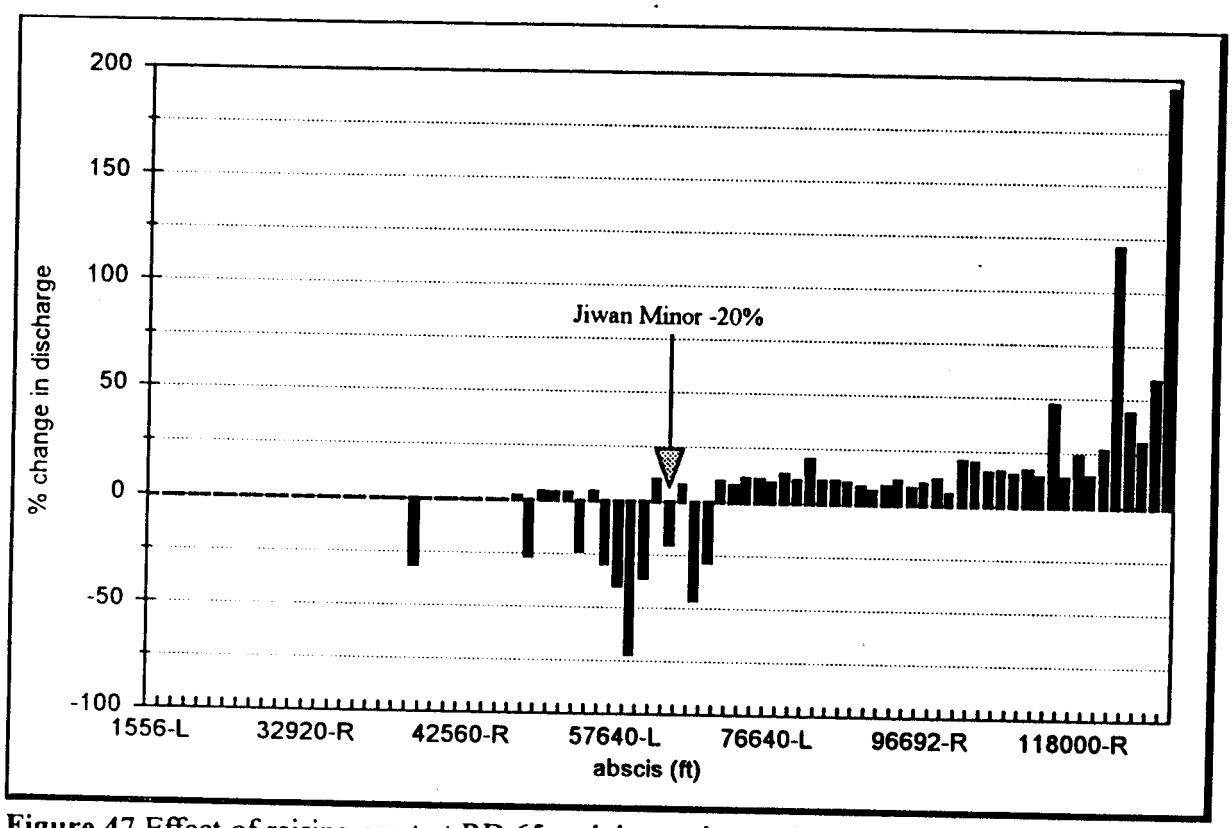


Figure 47 Effect of raising crest at RD 65 and decreasing outlets

6.6 The performance of the original design

An attempt has also been made to model the original design of the distributary in order to check if this design did perform according to the specifications. The input data of the channel were based on the longitudinal profile of the channel as given in the most recent longitudinal profile. The input data of the outlets were based on the Outlet Register. Several difficulties were encountered with this model:

1. The discharge coefficients used for the different outlets were not known
2. The formula used in SIC for APM's is different than the classical formula used by the Irrigation Department
3. Inconsistencies between the two sources. For the data of the outlets it was not known whether they stem from the same date as the longitudinal profile.

Because of these difficulties the original design was no longer studied.

6.7 Global results

In order to compare the results of the different simulations one parameter, the ratio of effective discharge to total discharge, has been calculated for all simulations. They are presented in the table below:

		Effective discharge (%)
Current situation		89.3
Simulations:		
1	Siltation over whole channel by cm	87.6
2	Desiltation over whole channel by 5 cm	90.4
3	Changing of Manning's coefficient from 0.026 to 0.023	90.1
4	Desilting from RD 65 to tail by 30 cm	87.3
5	Desilting from RD 65 to tail by 50 cm	81.3
6	Desilting the whole channel by 10 cm	78.6
7	Desilting of the whole channel to bed level	88.9
8	Restoring bed level and width to design dimensions	91.7
9	Reducing oversized outlets	91.5
10	Raising crest at RD 65	90.9
11	Raising crest of RD 65 and reducing oversized outlets	**94.0**

Table 6 Resulting effectiveness of the performed simulations

It can be seen that of all the performed simulations the last one gives the best results. The analysis of these results will be given in chapter 7.

6.8 Effects of variable inflow

In order to obtain some insight in the distribution of variations in the inflow, the incoming discharge has been varied. This is not the topic of this study but it has been done to check at which discharge the tail falls dry. The lowest discharge at which water reaches the tail of this distributary is 3.693 m^3/s. This is 83% of design discharge. The distribution pattern at this discharge has been presented in figure 48. IIMI will conduct further study into the response of distributaries on varying inflows.
The reliability of this figure is questionable because of the APM formula which SIC uses.

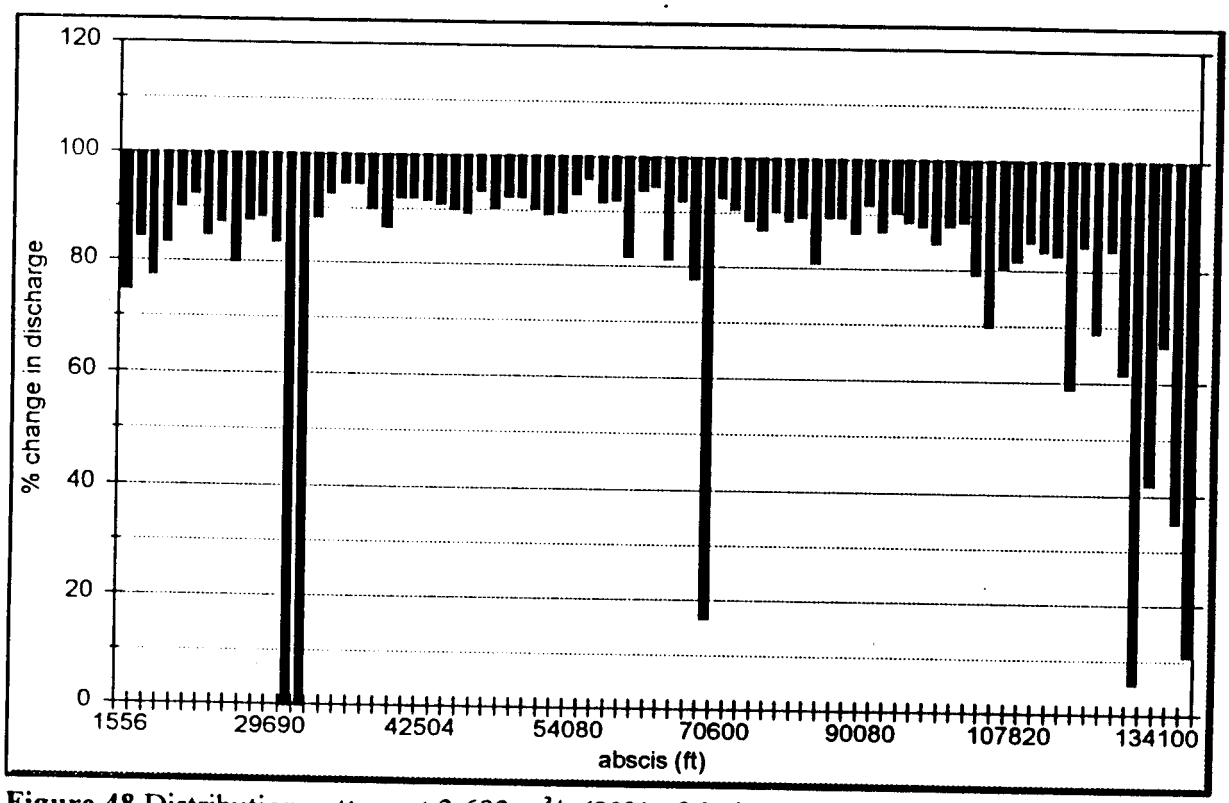


Figure 48 Distribution pattern at 3.693 m^3/s (83% of design discharge)

7 Theoretical approach of maintenance

7.1 Background

In the previous chapters of this study an attempt was made to analyze effects of maintenance based on measurements on an existing situation in the field. It was found that there are a large number of variables which play a role in the Performance of the distributary. Variations in dimensions of structures as well as variations in dimensions of the channel have effect on the Performance. In this chapter the number of variables is reduced to only one. Variations in dimensions of structures are no longer taken into account. The variations in dimensions of the channel, which are a three-dimensional phenomenon, are reduced to one parameter only. In this way a more clear insight in the relation between maintenance and water distribution is obtained.

If the formula of Manning-Strickler is taken, maintenance can be considered as the actions which have effect on the factor $A.R^{2/3}$. The roughness and slope are considered to be independent from the maintenance.

In this paragraph the effects of changes of this factor on the effectivity of supply are determined.

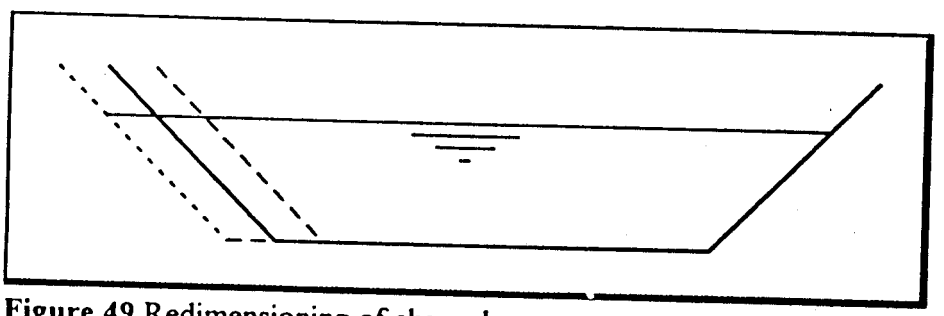


Figure 49 Redimensioning of channel

Over the whole length of the channel, the variable $AR^{2/3}$ of the cross-sections is scaled by a constant factor. This was attained by varying the width of the channel (see figure 49). The range over which the variable was changed was from 0.4 to 1.3 of the design value.

Before these computations were done the systems characteristics were fixed by changing the settings of the outlets in such a way that both the sensitivity S=1 was obtained at design discharge as well as authorized discharge. In this case the simplifications described in chapter 3 (infinite width and vertical sideslopes) were **not** applied. The real hydraulic radius R and the real and the real area of flow A were taken.

In order to get a sensitivity of S=1 for all outlets, the height of the outlet was altered. The effect on the discharge was compensated by a subsequent change in the width of each outlet. The resulting effect is that for an inflow equal to design discharge, each outlet receives it's authorized discharge and also has a sensitivity of S=1

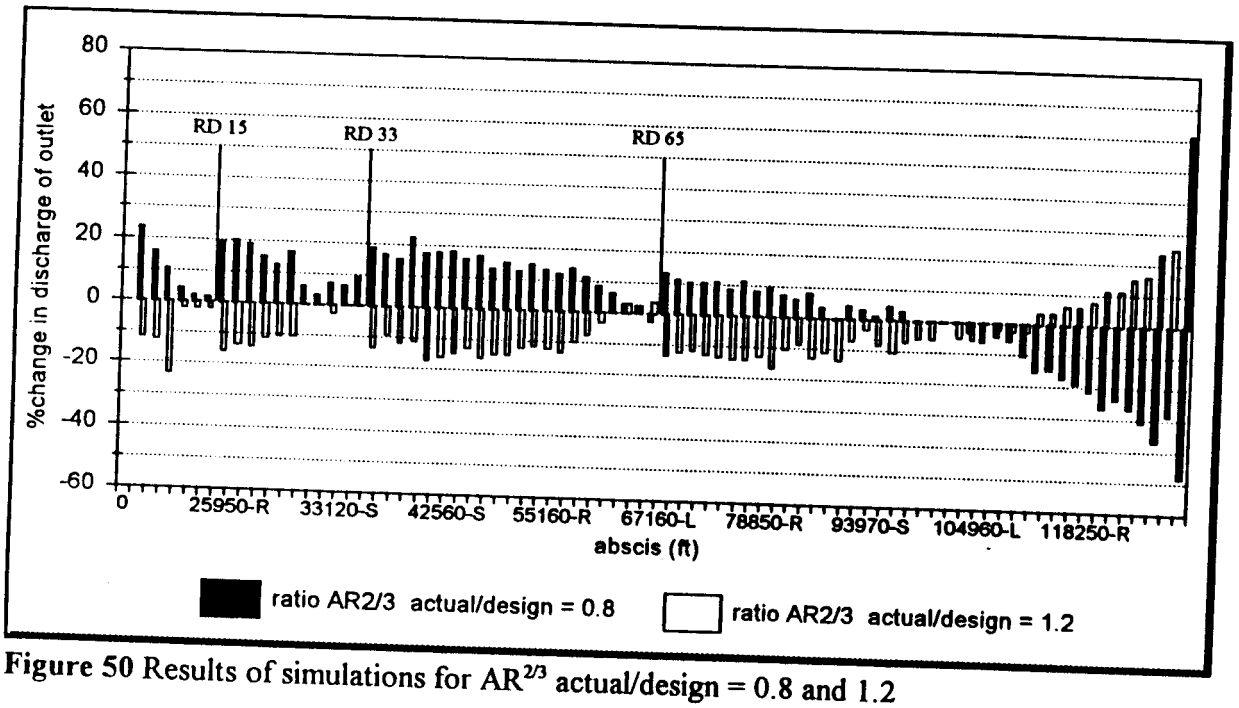


Figure 50 Results of simulations for $AR^{2/3}$ actual/design = 0.8 and 1.2

The following effects can be seen in figure 50 (the series with a ratio $AR^{2/3}$ actual/design is 0.8 will be used for explanation):

1 The tail of the distributary responds the strongest to change of $AR^{2/3}$ ratio to 0.8. The level in the first part of the channel is higher than design. This leads to higher discharges through the outlets in this section. At the end of the channel there is not enough water left and the canal water level drops below design level and consequently the Outlet discharges are far below design discharges.

2 A drop structure clearly makes the portion of the channel directly upstream insensitive to any changes in $AR^{2/3}$. The length of the upstream portion of the channel which is influenced by the drop structure is dependant of the backwater curve. This has not been studied further.
The positive effect of drop structures is very clear. Adding more drop structures in the channel would be beneficial to the sensitivity of the distributary to changes in the cross profile but requires more head loss over the length of the channel. In reality however, this head loss is not available.

3 The two graphs in figure 50 are complementary except for the portions immediately upstream of the drop structures. An increase in $AR^{2/3}$ of the channel cannot affect the Q(h) relationship determined by a structure, but a decrease can affect the Q(h) relation This is clearly visible at RD 33.

4 The Effectivity of the supply as computed for the whole channel can be seen in the figure. The portion of the discharge which is not effective is visible as the summation of the part of the graph underneath the x-axis.

5 The irregularities in the graph are caused by rounding errors. All discharges are computed with an accuracy of 0.001 m^3/s. The range of discharges of outlets is from 0.010 to 0.100 m^3/s. this can cause errors of up to 10%. No further action has been taken to improve the accuracy.

7.2 Effectivity and maintenance of canal cross section

The results are presented in figure 51.
This is the most important outcome of this study. The conclusion that can be drawn from this figure is that the performance of this distributary is relatively insensitive to maintenance of the canal cross section. Measures to control the factor $A.R^{2/3}$ do not have a considerable effect on

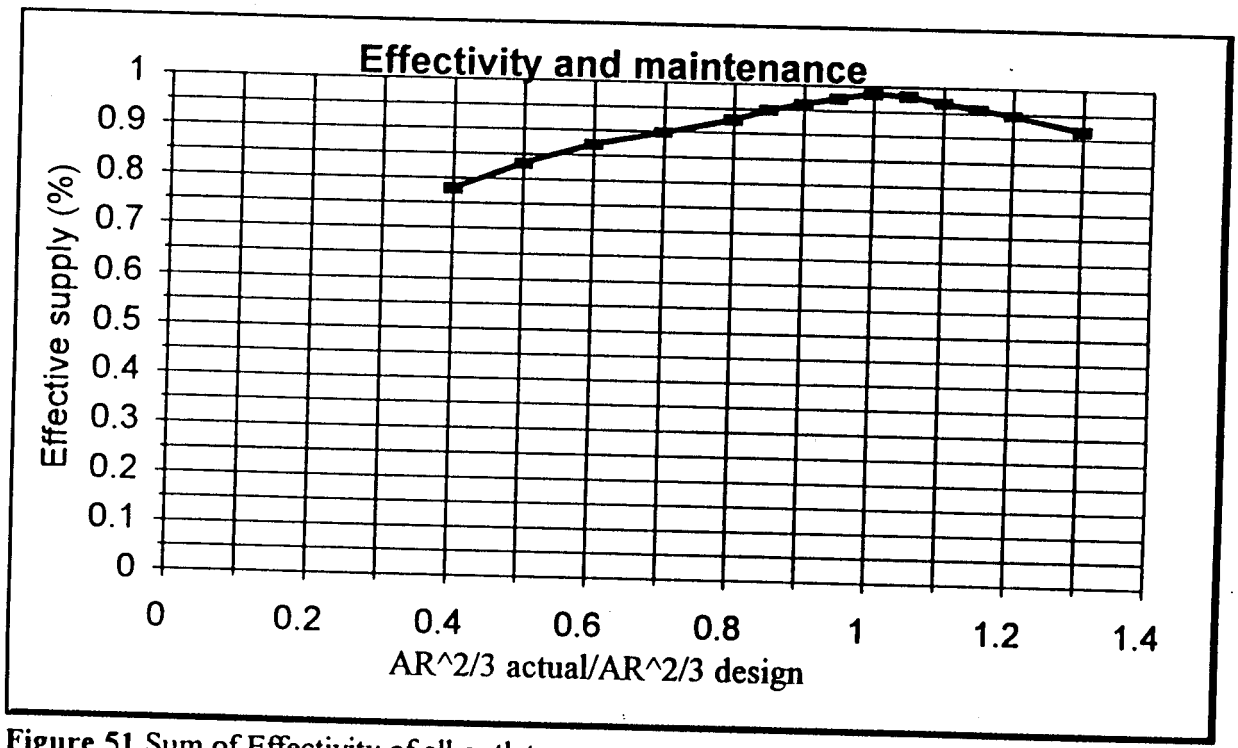


Figure 51 Sum of Effectivity of all outlets

the performance. Since $A.R^{2/3}$ is a factor which is difficult to visualize the results are plotted for the ratio of Aactual/Adesign, with A being the surface of flow in m^2 (figure 52).

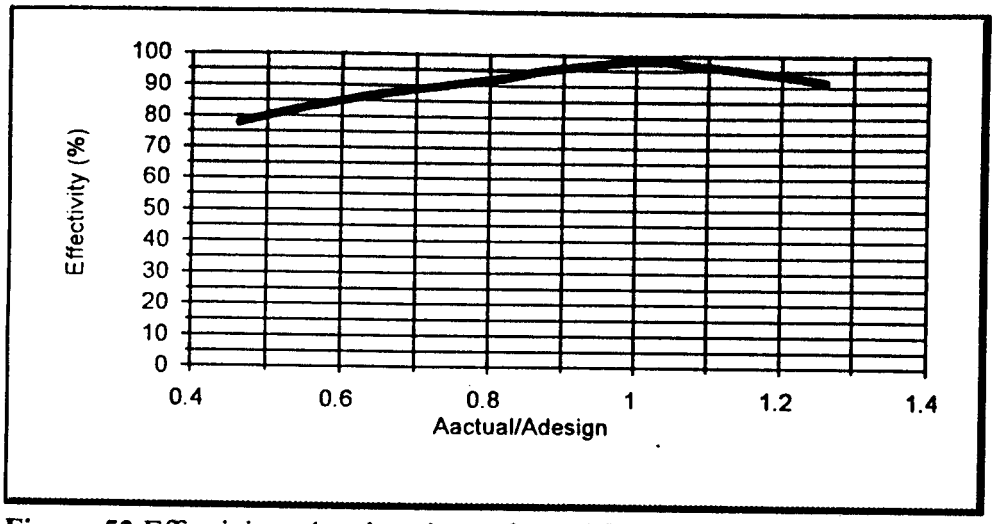


Figure 52 Effectivity related to the surface of flow

After observing these results one could question the definition of performance as posed in this study. Lower boundaries could be set below the authorized discharges of outlets. If the calculated discharge falls below a boundary of, for example -20% of the authorized discharge for a specific outlet the total discharge could be counted as ineffective. This would lead to a result which would be more sensitive to changes in $AR^{2/3}$. The purpose of this exercise however is not to open a discussion on performance but to show the sensitivity of the distributary for changes in the cross-sections of the channel.

The behavior of outlets depends on the location along the distributary and the possible influence of a downstream dropstructure.
In figure 53, three different outlets have been compared. The three have different responses to changes in $A.R^{2/3}$:

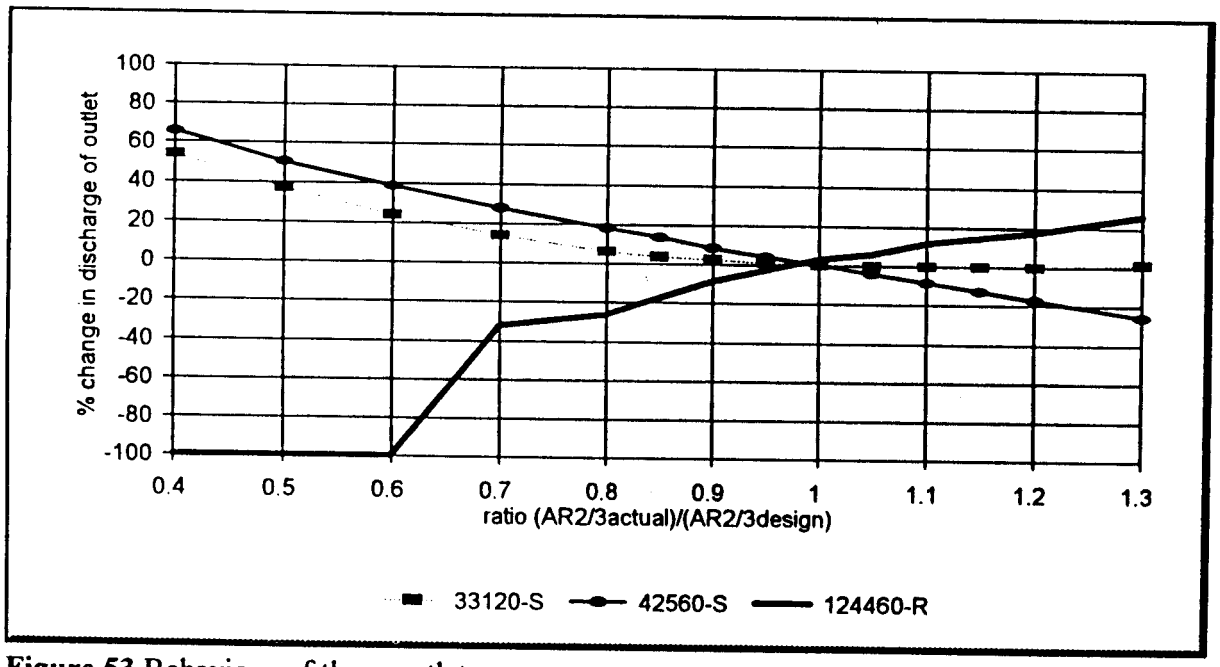


Figure 53 Behaviour of three outlets

Outlet 33120: This outlets lies immediately upstream of the dropstructure at RD 33.3. When the cross section of the channel is greater than design, the cross structure maintains the relationship between discharge and water level. When the channel decreases in size, the channel itself will determine this relationship, causing a higher discharge

Outlet 42560: This outlet is not under the influence of a drop structure and lies in the region which benefits from a decrease in $A.R^{2/3}$

Outlet 124460: This outlet is situated near the tail of the distributary and suffers from decreases in $A.R^{2/3}$

8 Conclusions and recommendations

In this chapter the final conclusions of this study are given, together with some recommendations for future developments and study.

8.1 Conclusions of the study

a) In the description of the current situation it became clear that the first order problem for Fordwah Distributary is the irregular inflow. This problem is caused by operations upstream and is not further studied. In order to study the relationship between maintenance and distribution, the inflow is assumed to be correct in this study.

b) It was found that maintenance of the canal cross-section plays an important role in the distribution of water, because maintenance determines the relationship between discharge and water level in a distributary and the discharge of the outlets is dependant on these water levels.

c) Next, the present performance of the channel was determined. For this a hydraulic model of the channel was made. Extensive field measurements were executed to calibrate this model. This resulted in a fairly accurate description of the existing channel. The inaccuracy of the discharges improved from 55% to 5.4%.
With the model the distribution pattern in the current situation was determined. It was found that, although some outlets draw an excessive discharge, the overall performance was not bad. 85% of the incoming discharge was distributed correctly. Shortages occur near the tail of the distributary.

d) Another conclusion that can be drawn from this study is that determining the performance of a distributary is very time consuming and labor intensive. A more rapid approach of performance assessment is desirable.

e) Possible measures to improve the performance were suggested and analyzed. It was found that, although marginal improvements could be made, 100% effectivity was not possible.

f) Furthermore it became clear that measures of desiltation do not give better results than simple adjustments of outlets. The best performance was obtained by the decreasing in size of a limited number of outlets together with the raising of the crest of one cross structure. This resulted in an effectivity of supply of 94 %.

g) Apart from this, an attempt was made to establish a theoretical relationship between the overall performance of this distributary and the variance of the factor which determines the canal Q-h relationship, i.e. the factor $A.R^{2/3}$ of the Manning-Strickler equation. The effective supply was determined for variations in $A.R^{2/3}$ from 0.4 to 1.3 of design values. It was found that the performance of the channel decreased only from an initial value of 100% to 77%. Thus the distributary is relatively insensitive to variations in this factor. This confirms the same conclusion found in the previous chapter, namely that maintenance of the channel does not improve it's performance

significantly and can certainly be questioned from a cost-effective point of view.

8.2 Recommendations

8.2.1 Rapid assessment of performance of distributaries

Objectives of a rapid assessment

In this report extensive research has been done to gain insight in the present distribution of water in Fordwah Distributary, the causes of deficiencies and ways to improve the distribution. Use has been made of a hydrodynamic software package. By and large this took about 6 months. However, if the Irrigation Department would like to examine any distributary in the future without having so much time available, a shorter procedure is necessary.
Objective of this procedure is to gain insight in the water distribution of a secondary channel with a limited amount of time and manpower. Causes and locations of possible deficiencies are to be determined rapidly.
Previously in this report it has been shown that measures of desiltation or other changes to the distributary are not effective when the distribution needs to be improved. By correcting outlets which do not take the required discharge a much better result can be obtained.
In the procedure the effectively distributed discharge is used as the parameter with which a distributary is evaluated. With a limited number of measurements this parameter is determined for a distributary. With the result the choice whether or not to intervene can better be justified. It is important to realize that the result of this approach is that the physical properties of a distributary are now regarded independent from the water distribution. For the physical state of the channel a separate procedure is needed.

Procedure

The procedure itself is based upon the idea to divide a channel in a limited amount of sections, each having about the same size in Culturable Command Area. When the channel is running at Full Supply Discharge, discharge measurements are to be taken in the field at the head of the channel and at the boundaries of the sections. With the measurements the total effective and ineffective supply of the distributary can be determined.
The procedure is more elaborately described in Appendix E

8.2.2 Upgrading of SIC

In this study it was found that several properties of SIC need to be improved. Tail-dry problems, discharge formulas of outlets and the transition between open flow and orifice flow are features which need to be improved.

8.3 Need further research

Following this study, the next subjects are recommended for further study

1 Silt strategy

The distribution of silt plays a very important role in the maintenance of irrigation systems in Pakistan. On this subject a lot of research is necessary. Several different possibilities can be

mentioned

a. Relationship between operations and maintenance
This study has contributed to initializing a study into the relationship between operations at the primary level and sediment transport. Operations influence the sediment transport capacity of a channel and thus the siltation. CEMAGREF, IIMI and ISRI, Pakistan will conduct joint research into this subject in the future.

b. Silt draw of outlets
In the current setup the aim is to distribute silt equitably. It might be possible to reduce maintenance costs if the silt draw of outlets were maximized. The aim would then be to extract the silt from the water as early in the system as possible. This strategy could be thought at the primary level as well as at the secondary level. At the primary level this would mean that the silt has to be directed into the distributaries which are located on the upstream side of the primary canal. At the secondary level this implies that the silt draw of all outlets is to be maximized, thereby reducing the amount of silt in the direction of the flow.
The consequences of such an approach would be that the workload of desilting will be reallocated. The Irrigation Department will have less work to do on maintenance. Upstream farmers will have more work in maintaining their watercourses.

c. Advantages of lining
The discussion on lining needs to be focussed on the effects on the siltation. If by implementation of lining the roughness can be decreased, the velocities can be increased. This could lead to a decrease in siltation. A look at possible advantages of vertical side slopes in lined canals is also necessary. We consider it very well possible that vertical side slopes lead to improvements of the sustainability of Pakistans irrigation systems in the long run.

2 Research on other types of outlets

For those distributaries for which irregular inflow is not a problem, it might be advisable to install outlets with the property of dq/dh=0, or S=0. In this way the distribution can be made independent from maintenance. Baffle distributors have the characteristic that they can keep the discharge relatively constant over a certain range of the upstream water level. These outlets might be preferable over the outlets currently used. Research could prove this. The cost aspect of these outlets is something which needs to be investigated also.

3 Installation of a new communication system along Fordwah Branch.

The author has found that the absence of a communication system along Fordwah Branch is the most urgent problem which needs to be tackled. A communication system is a prerequisite for proper water management. Research needs to be conducted into the setup of such system. Amongst others, the following subjects need attention:

a. The locations of the system which need to be connected to the communication system.
b. The way in which the flows of information are to be directed (bottom-up)
c. The command structure (top-down)
d. Technical issues such as the what kind of information is to be gathered
e. Available hardware. One important choice which has to be made is whether a communication system should be wired or wireless.

f. The costs need to be made clear. The author has the firm belief that whatever the costs are, a communication system is an absolute necessity

4 Research into legal changes

Research should be done on the issue whether the responsibility of the Irrigation Department could be expanded from distributing water only to distribution of both water and information. It should become the responsibility of the Irrigation Department to make information on both target discharges and actual achieved discharges available to the public. Then it is possible for the water users to judge the performance of the Irrigation Department. This implies a study into the legal aspects of such an increase in responsibility of the Irrigation Department. Transparency of irrigation management is also considered to be a prerequisite for a good performance of this irrigation system.

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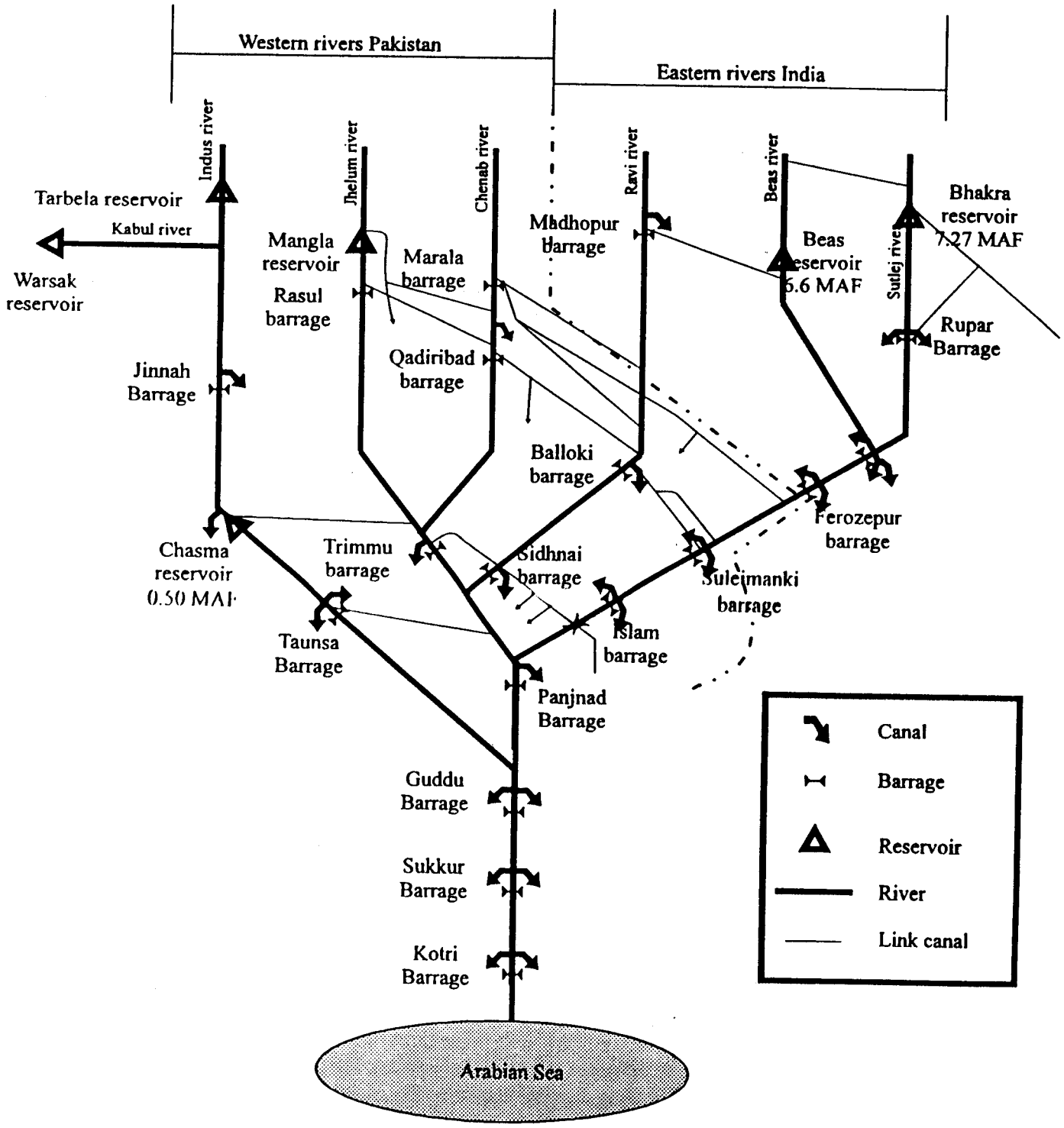
Appendices

Appendix A: Indus Basin irrigation system



Appendix B: Structure formulas in SIC

The structure formulas in SIC are simplified to a certain extent. The formulas used differ from the classical formulas. Especially in the submerged region variances occur. In figures xx and xx the different regions and formulas used are given.

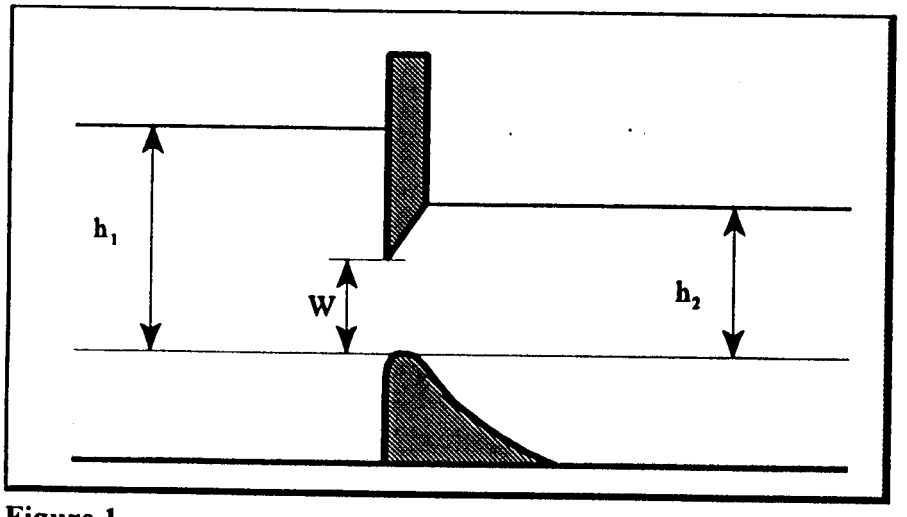


Figure 1

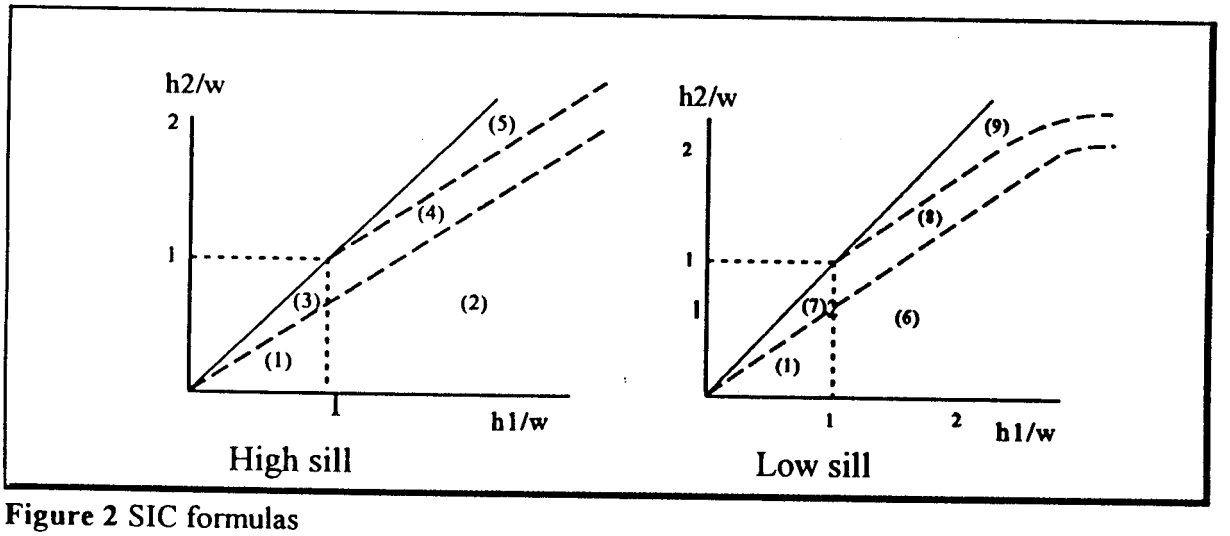


Figure 2 SIC formulas

The ranges where the different SIC formulas are used, are presented in figure 19.
They are characterized by:

1) Open, free flow
2) Orifice, free flow
3) Open, submerged flow

4) Orifice flow, partially submerged
5) Orifice flow, completely submerged
6) As 2, with low sill
7) As 3, with low sill
8) As 4, with low sill
9) As 5, with low sill

Structure formulas:

1) $Q=\mu_F*L*\sqrt{2}g*h_1^{3/2}$ $\quad$ $0 < h_1 < w,\ h_2 < 2/3\ h1$

2) $Q=\mu*L*\sqrt{2}g*(h_1^{3/2}-(h_1-W)^{3/2})$ $\quad$ $h_1 > w,\ h_2 < 2/3\ h1$

with: $\mu = \mu_F$

3) $Q=\mu_S*L*\sqrt{2}g*(h_1-h_2)^{1/2}*h_2$ $\quad$ $0 < h_1 < w,\ 2/3h_1 < h_2 < h_1$

with : $\mu_S=\frac{3\sqrt{3}}{2}*\mu_F$

4) $Q=\mu_F*L*\sqrt{2}g*[\frac{3\sqrt{3}}{2}*((h_1-h_2)^{1/2}*h_2)-(h_1-W)^{3/2}]$ $\quad$ $h_1 > w,\ 2/3h_1 < h_2 < 2/3h_1+W/3$

5) $Q=\mu'*L*\sqrt{2}g*(h_1-h_2)^{1/2}*W$ $\quad$ $h_1>w,\ h_2 > 2/3\ h_1 +1/3W$

with: $\mu'=\mu_S=\frac{3\sqrt{3}}{2}*\mu_F$

6) $Q=L*\sqrt{2}g*(\mu*h_1^{3/2}-\mu_1*(h_1-W)^{3/2})$ $\quad$ $h1>w,\ h2<\frac{h1}{1+0.14*\frac{h1}{w}}$

with: $\mu=\mu_F+0.08*(1-\frac{1}{\frac{h_1}{W}})$

and: $\mu_1=\mu_F+0.08*(1-\frac{1}{\frac{h_1}{W}-1})$

7) $Q=k_F*\mu_F*L*\sqrt{2}g*h_1^{3/2}$ $\quad$ $h1<w,\ \frac{h1}{1+0.14*\frac{h1}{w}}<h2<h1$

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

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

x < 0.2: $k_F = 5x*(1-(1-\frac{0.2}{\sqrt{1-\alpha}})^{\beta})$

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

8) $Q = L*\sqrt{2g}*[k_F*\mu*h_1^{3/2} - \mu_1*(h_1 - W)^{3/2}]$ h1>w, $\frac{h1}{1+0.14*\frac{h1}{w}} < h2 < \frac{h_1 + W*(0.14*\frac{h_1}{W}+0.14)}{1.14+0.14*\frac{h_1}{w}}$

For k_F, μ and μ_1 see equations 6 and 7

9) $Q = L*\sqrt{2g}*[k_F*\mu*h_1^{3/2} - k_{F1}*\mu_1*(h_1 - W)^{3/2}]$ h1>w, $\frac{h_1 + W*(0.14*\frac{h_1}{W}+0.14)}{1.14+0.14*\frac{h_1}{w}} < h_2 < h_1$

For k_F, μ and μ_1 see equations 6 and 7

For k_{F1}: $x = \sqrt{1 - \frac{h_2 - W}{h_1 - W}}$

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

x < 0.2: $k_F = 5x*(1-(1-\frac{0.2}{\sqrt{1-\alpha}})^{\beta})$

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

Appendix C: Structures Data Entry, actual situation

Date: January 1995
Made by :Walter Hart & Anwar Iqbal

1) Cross Device Description

S No	Name	Location (ft)	Location (m)	Width B (ft)	Width B (m)	C.R.L. (m)	Remarks
1	15000	15000	4572	12.92	3.94	143.57	
2	33300	33300	10150	12.92	3.94	142.16	
3	65300	65300	19903	8.05	2.45	140.26	

2) Nodes Description

S No	Name	Location (ft)	Location (m)	Width B (ft)	Width B (m)	Height H (ft)	Height H (m)	C.R.L. (m)	Remarks
1	1556-L	1556	474	0.21	0.06	0.86	0.26	144.625	
2	6100-L	6100	1859	0.24	0.07	1.00	0.30	144.291	
3	11450-L	11450	3490	0.22	0.07	0.75	0.23	144.132	
4	14320-R	14320	4365	0.40	0.12	0.95	0.29	143.843	
5	14710-R	14710	4484	0.20	0.06	1.55	0.47	143.559	
6	14910-R	14910	4545	0.39	0.12	1.16	0.35	143.533	
7	24500-R	24500	7468	0.21	0.06	1.39	0.42	142.793	
8	25950-R	25950	7910	0.20	0.06	1.37	0.42	142.592	
9	27050-L	27050	8245	0.29	0.09	0.72	0.22	142.838	
10	28110-R	28110	8568	0.19	0.06	1.41	0.43	142.459	
11	29550-R	29550	9007	0.20	0.06	1.27	0.39	142.456	
12	29690-R	29690	9050	0.26	0.08	1.21	0.37	142.581	
13	32920-R	32920	10034	0.88	0.27	0.00	0.00	142.326	Pipe Diam.
14	32940-R	32940	10040	0.62	0.19	0.00	0.00	142.201	Pipe Diam.
15	32940-L	32940	10040	0.26	0.08	0.80	0.24	142.410	
16	33000-R	33000	10058	0.26	0.08	0.81	0.25	141.910	
17	33120-R	33120	10095	0.25	0.08	0.42	0.13	142.130	
18	33160-R	33160	10107	0.22	0.07	0.45	0.14	142.130	
19	38230-R	38230	11653	0.27	0.08	0.87	0.27	141.999	
20	38830-L	38830	11835	0.17	0.05	0.69	0.21	141.976	
21	39550-R	39550	12055	0.26	0.08	0.90	0.27	141.656	
22	42040-L	42040	12814	0.14	0.04	0.70	0.21	141.727	
23	42504-L	42504	12955	0.22	0.07	1.40	0.43	141.461	
24	42510-L	42510	12957	0.21	0.06	1.15	0.35	141.535	
25	42560-R	42560	12972	0.26	0.08	1.08	0.33	141.618	
26	42580-R	42580	12978	0.20	0.06	1.80	0.55	141.502	
27	42600-R	42600	12984	0.28	0.09	0.68	0.21	141.596	
28	46725-R	46725	14242	0.23	0.07	1.43	0.44	141.179	
29	50575-R	50575	15415	0.21	0.06	1.56	0.48	140.833	
30	51500-L	51500	15697	0.26	0.08	1.12	0.34	140.780	
31	53380-R	53380	16270	0.22	0.07	1.43	0.44	140.813	

32	53920-R	53920	16435	0.20	0.06	1.00	0.30	140.965	
33	54060-R	54060	16477	0.35	0.11	1.05	0.32	140.892	
34	54080-R	54080	16484	0.24	0.07	1.46	0.45	140.612	
35	55160-R	55160	16813	0.25	0.08	0.48	0.15	140.572	
36	56000-L	56000	17069	0.22	0.07	1.14	0.35	140.508	
37	57640-L	57640	17569	0.22	0.07	1.14	0.35	140.315	
38	60000-L	60000	18288	0.35	0.11	3.00	0.91	140.437	Outlet broken
39	60410-L	60410	18413	0.23	0.07	1.24	0.38	140.281	
40	62085-Cm	62085	18924	0.46	0.14	1.10	0.33	140.160	62085-R, 62225-L
41	65300-R	65290	19900	3.05	0.93	0	0	141.260	Jiwan Minor
42	67160-L	67160	20470	0.20	0.06	0.74	0.23	140.036	
43	68260-L	68260	20806	0.20	0.06	0.46	0.14	140.077	
44	70530-R	70530	21498	0.20	0.06	0.97	0.30	139.775	
45	70600-L	70600	21519	0.20	0.06	0.81	0.25	139.876	
46	71200-Cm	71200	21702	0.41	0.12	0.75	0.23	139.854	71200-R, 71697-L
47	73008-R	73008	22253	0.22	0.07	0.99	0.30	139.787	
48	75140-R	75140	22903	0.20	0.06	1.00	0.30	139.627	
49	76640-L	76640	23360	0.22	0.07	0.85	0.26	139.406	
50	78850-R	78850	24033	0.23	0.07	0.94	0.29	139.393	
51	82600-L	82600	25176	0.20	0.06	0.71	0.22	139.051	
52	82700-R	82700	25207	0.27	0.08	1.44	0.44	139.205	
53	83700-R	83700	25512	0.22	0.07	0.46	0.14	139.035	
54	84140-L	84140	25646	0.20	0.06	0.60	0.18	139.102	
55	90000-R	90000	27432	0.20	0.06	0.60	0.18	138.705	
56	90080-L	90080	27456	0.20	0.06	0.24	0.07	138.721	
57	91960-R	91960	28029	0.20	0.06	0.60	0.18	138.579	
58	93970-Cm	93970	28642	0.40	0.12	0.88	0.27	138.356	93970-R, 94350-R
59	95000-R	95000	28956	0.19	0.06	0.35	0.11	138.413	
60	96300-L	96300	29352	0.23	0.07	0.46	0.14	138.343	
61	96692-R	96692	29472	0.21	0.06	0.40	0.12	138.196	
62	99500-R	99500	30328	0.22	0.07	0.43	0.13	138.020	
63	101800-R	101800	31029	0.20	0.06	0.36	0.11	137.892	
64	102820-R	102820	31340	0.24	0.07	0.67	0.20	137.943	
65	104960-L	104960	31992	0.20	0.06	0.27	0.08	137.907	
66	106000-R	106000	32309	0.32	0.10	0.70	0.21	137.664	
67	107820-R	107820	32864	0.34	0.10	0.68	0.21	137.562	
68	112250-R	112250	34214	0.21	0.06	0.76	0.23	137.158	
69	112400-L	112400	34260	0.30	0.09	0.65	0.20	137.219	
70	114700-R	114700	34961	0.20	0.06	0.66	0.20	137.000	Broken, B not unifor
71	116600-L	116600	35540	0.20	0.06	0.00	0.00	136.962	roof broken, open flu
72	117775-R	117775	35898	0.40	0.12	0.60	0.18	136.663	
73	118000-R	118000	35966	0.80	0.24	0.82	0.25	136.777	
74	118250-R	118250	36043	0.34	0.10	0.67	0.20	136.560	
75	125000-R	125000	38100	0.33	0.10	0.56	0.17	136.129	
76	125062-L	125062	38119	0.51	0.16	0.81	0.25	136.318	
77	130100-R	130100	39654	0.51	0.16	0.76	0.23	135.933	
78	134100-R	134100	40874	0.34	0.10	0.57	0.17	135.616	
79	135180-R	135180	41203	0.23	0.07	0.67	0.20	135.556	
80	139780-TF	139780	42605	1.59	0.48	0.00	0.00	135.234	Open flume

3) Composed Nodes

S No	Name	Location (ft)	Location (m)	Width B (ft)	Width B (m)	Height H (ft)	Height H (m)	C.R.L. (m)	Remarks
40	62085-S	62085	18924	0.46	0.14	1.10	0.33	140.160	62085-R, 62225-L
46	71200-S	71200	21702	0.41	0.12	0.75	0.23	139.854	71200-R, 71697-L
59	93970-S	93970	28642	0.40	0.12	0.88	0.27	138.356	93970-R, 94350-R

The nodes above were composed of the nodes below

1	62085-R	62085	18924	0.24	0.07	1.06	0.32	140.181
2	62225-L	62225	18966	0.22	0.07	1.13	0.34	140.140
3	71200-R	71200	21702	0.21	0.06	0.76	0.23	139.873
4	71697-L	71697	21853	0.20	0.06	0.73	0.22	139.864
5	93970-R	93970	28642	0.20	0.06	0.95	0.29	138.361
6	94350-R	94350	28758	0.20	0.06	0.80	0.24	138.352

Cross sections of Fordwah Distributary, actual situation

Survey done by Anwar Iqbal & Walter Hart
Date: January 1995
X-sections started from left bank

Explanation:
RD. X-section: Exact location of measurement
Validity: The X-section will be used to describe these nodes in SIC
Reference: The reduced level of this place is used to determine the reduced levels of the X-section

RD. X-section	(ft)	100								
Validity		Head Fordwah Distributary								
Reference		IIMI B.M. RD 371			Ref. level (ft)		479.50			
Hor. distance	(ft)	0	3	5	7	9	11	13	15	18
Red. level	(m)	145.57	145.39	145.30	144.13	144.50	144.30	144.15	144.03	143.99
Hor. distance	(ft)	21	24	27	30	33	36	39	42	45
Red. level	(m)	144.05	144.02	144.07	144.09	144.12	144.17	144.20	144.24	144.28
Hor. distance	(ft)	48	51	53	55	58	61			
Red. level	(m)	144.28	144.28	144.55	145.27	145.40	145.54			

RD. X-section	(ft)	1550								
Validity		1556-L								
Reference		Crest of outlet 1556-L			Ref. level (ft)		474.49			
Hor. distance	(ft)	0	3	5	7	9	12	15	18	21
Red. level	(m)	145.4475	145.3454	144.5026	144.0637	143.7483	143.6065	143.6431	143.6949	143.7864
Hor. distance	(ft)	24	27	29	31	33	36	39		
Red. level	(m)	143.861	144.0454	144.1674	144.3807	145.1961	145.2463	145.4048		

RD. X-section	(ft)	6020								
Validity		6100-L								
Reference		Crest of outlet 6100-L			Ref. level (ft)		473.395			
Hor. distance	(ft)	0	3	5	7	10	13	16	19	22
Red. level	(m)	145.2662	144.9918	144.1841	143.9967	143.9357	143.9007	143.922	143.9311	143.9708
Hor. distance	(ft)	25	28	31	34	37	40	42	44	46
Red. level	(m)	143.9799	143.9616	143.9708	143.9647	143.9799	143.9799	144.0409	144.2756	145.0833
Hor. distance	(ft)	49								
Red. level	(m)	145.3728								

RD. X-section	(ft)	11470								
Validity		11450-L								
Reference		Crest of outlet 11450-L			Ref. level (ft)		472.875			
Hor. distance	(ft)	0	3	5	7	10	13	16	19	22
Red. level	(m)	144.8562	144.6032	144.6093	143.7345	143.5974	143.6126	143.5242	143.5212	143.4968
Hor. distance	(ft)	25	28	31	33	35	38			
Red. level	(m)	143.4846	143.4435	143.448	143.4145	144.6047	144.8897			

RD. X-section	(ft)	14310								
Validity		14320-R								
Reference		Crest of 14320-R			Ref. level (ft)		471.925			
Hor. distance	(ft)	0	3	6	8	10	13	16	19	22
Red. level	(m)	144.9278	144.8486	144.6367	144.1399	143.5273	143.573	143.5181	143.5395	143.5242
Hor. distance	(ft)	25	28	31	34	37	40	42	44	46
Red. level	(m)	143.5379	143.5334	143.5669	143.5029	143.4785	143.4572	143.448	143.9327	144.8257
Hor. distance	(ft)	49								
Red. level	(m)	144.7404								

RD. X-section	(ft)	14900								
Validity		14710-R, 14910-R, 15030 Drop U/S								
Reference		From IIMI BM RD 15030 Bridge			Ref. level (ft)		476.75			
Hor. distance	(ft)	0	3	6	8	10	13	16	19	22
Red. level	(m)	144.6169	144.4904	144.4295	143.8747	143.4816	143.4755	143.4663	143.4633	143.4511
Hor. distance	(ft)	25	28	31	34	37	40	42	44	46.5
Red. level	(m)	143.4419	143.4297	143.4297	143.4267	143.4023	143.4114	143.4998	143.8961	144.4417

Hor. distance	(ft)	49	52
Red. level	(m)	144.6459	144.6124

RD. X-section (ft) 24490
Validity 24500-R
Reference Crest of outlet 24500-R Ref. level (ft) 468.48

Hor. distance	(ft)	0	3	6	8	11	14	17	20	23
Red. level	(m)	143.8626	143.765	143.4861	142.5976	142.4026	142.302	142.2715	142.2837	142.2776
Hor. distance	(ft)	26	29	32	34	36	39			
Red. level	(m)	142.2791	142.2852	142.3812	142.5428	143.64	144.0302			

RD. X-section (ft) 25970
Validity 25950-R
Reference Crest of outlet 25950-R Ref. level (ft) 467.82

Hor. distance	(ft)	0	3	6	8	10	13	16	19	22
Red. level	(m)	143.576	143.4282	143.4084	142.9634	142.7805	142.5854	142.433	142.305	142.1679
Hor. distance	(ft)	25	28	31	34	36	37	39		
Red. level	(m)	142.0886	142.0398	142.0551	142.1709	142.619	143.7178	143.9997		

RD. X-section (ft) 27060
Validity 27050-L
Reference Crest of outlet 27050-L Ref. level (ft) 468.63

Hor. distance	(ft)	0	3	6	8	11	14	17	20	23
Red. level	(m)	143.7117	143.6324	143.3642	142.2944	142.1694	142.0901	142.1267	142.1755	142.2303
Hor. distance	(ft)	26	29	32	35	37	39	42		
Red. level	(m)	142.337	142.4376	142.5336	142.6296	142.6845	143.4191	143.7361		

RD. X-section (ft) 28100
Validity 28110-R
Reference Crest of outlet 28110-R Ref. level (ft) 467.385

Hor. distance	(ft)	0	3	6	8	11	14	17	20	23
Red. level	(m)	143.5898	143.5257	143.2971	142.398	142.3096	142.3248	142.3279	142.2944	142.2791
Hor. distance	(ft)	26	29	32	35	36	37	40		
Red. level	(m)	142.2913	142.2578	142.2182	142.2364	142.2303	143.3352	143.6995		

RD. X-section (ft) 29600
Validity 29550-R, 29690-R
Reference Crest of outlet 29550-R Ref. level (ft) 467.375

Hor. distance	(ft)	0	3	5	7	10	13	16	19	22
Red. level	(m)	143.3764	143.224	142.5595	142.3858	142.2944	142.2364	142.1968	142.1313	142.0429
Hor. distance	(ft)	25	28	30	32	35	37			
Red. level	(m)	141.9499	141.8341	141.9057	143.1417	143.4587	143.6446			

RD. X-section (ft) 33050
Validity 32920-R, 32940-R, 32940-L, 33000-R, 33120-R, 33160-R, 33300 Drop U/S
Reference From IIMI BM RD33300 Bridge Ref. level (ft) 470.1

Hor. distance	(ft)	0	3	6	8	10	13	16	19	22
Red. level	(m)	143.4724	143.2865	142.9786	142.4605	142.2349	142.0673	142.1069	142.1069	142.1008
Hor. distance	(ft)	25	28	31	35	38	40	41	44	
Red. level	(m)	142.0795	142.052	142.0307	142.0398	142.0002	142.5062	143.2042	143.5029	

RD. X-section (ft) 38250
Validity 38230-R
Reference From crest of o/l 38230-R Ref. level (ft) 465.875

Hor. distance	(ft)	0	3	5	7	9	12	15	18	21
Red. level	(m)	142.8064	142.6479	142.1023	141.8966	141.7594	141.6482	141.6147	141.6025	141.6299
Hor. distance	(ft)	24	27	30	33	36	38	40	42	
Red. level	(m)	141.6573	141.6543	141.6878	141.7366	141.8737	142.1694	142.6235	142.7577	

RD. X-section (ft) 38810
Validity 38830-L, 39550-R
Reference From crest of O/L 38830-L Ref. level (ft) 465.8

Hor. distance	(ft)	0	3	5	8	11	14	17	20	23
Red. level	(m)	142.8095	142.6296	141.9469	141.7914	141.7396	141.7	141.6939	141.6543	141.6451
Hor. distance	(ft)	26	29	32	35	37	39	41	43	
Red. level	(m)	141.639	141.6299	141.6238	141.6543	141.7579	142.5809	142.6784	142.8765	

RD. X-section (ft) 42050
Validity 42040-L
Reference Crest of O/L 42040-L Ref. level (ft) 464.985

Hor. distance	(ft)	0	2	4	6	9	12	15	18	21

Red level (m)	142.6403	142.4849	141.9682	141.5964	141.5049	141.4699	141.4257	141.38	141.4074	
Hor distance (ft)	24	27	30	33	36	38	40	42		
Red level (m)	141.4348	141.4744	141.5537	141.6299	141.7488	141.7823	142.4041	142.5809		

RD X-section (ft) 42835
Validity
Reference 42504-L, 42510-L, 42560-R, 42580-R, 42600-R, 42850 Bridge U/S
From IIMI BM at RD 42850 Bridge Ref. level (ft) 468.626

Hor distance (ft)	0	3	6	7	10	13	16	19	22
Red level (m)	142.5138	142.398	141.6969	141.6781	141.6049	141.4348	141.3464	141.3159	141.2489
Hor distance (ft)	25	27	29	31	33	36			
Red level (m)	141.1696	141.5293	141.7427	142.3005	142.3614	142.5504			

RD X-section (ft) 46700
Validity
Reference 46725-R
From crest of O/L 46725-R Ref. level (ft) 463.166

Hor distance (ft)	0	3	5	7	10	13	16	19	22
Red level (m)	142.2273	142.0414	141.4775	141.3434	141.3038	141.3739	141.3464	141.2702	141.2672
Hor distance (ft)	25	28	31	33	35	37	39	41	43
Red level (m)	141.2458	141.2291	141.1788	141.1727	141.194	141.3586	141.7366	142.1176	142.3462

RD X-section (ft) 50600
Validity
Reference 50575-R
From crest of O/L 50575-R Ref. level (ft) 462.05

Hor distance (ft)	0	3	4	6	9	12	15	18	21
Red level (m)	142.1511	141.9255	141.447	141.0172	141.0081	141.0401	141.0569	141.0569	141.0416
Hor distance (ft)	24	27	30	32	34	36	39		
Red level (m)	140.9868	140.8862	140.9045	140.9289	141.1453	141.985	142.1572		

RD X-section (ft) 51495
Validity
Reference 51500-L
From crest of O/L 51500-L Ref. level (ft) 461.875

Hor distance (ft)	0	3	6	8	11	14	17	20	23
Red level (m)	141.9195	141.9621	141.9012	140.9441	140.8466	140.877	140.9167	140.9441	140.9471
Hor distance (ft)	26	29	32	35	37	39	42	45	
Red level (m)	140.9624	140.9685	141.0142	141.1087	141.2337	142.1541	142.0658	141.9713	

RD X-section (ft) 53370
Validity
Reference 53380-R
From crest of outlet 53380-R Ref. level (ft) 461.985

Hor distance (ft)	0	3	6	8	10	13	16	19	22
Red level (m)	142.0002	141.9636	141.7381	141.2687	141.0828	140.8999	140.7932	140.7673	140.6926
Hor distance (ft)	25	28	31	33	36	39			
Red level (m)	140.6012	140.586	140.7109	141.8143	141.9515	141.7533			

RD X-section (ft) 53900
Validity
Reference 53920-R
From crest of O/L 53920-R Ref. level (ft) 462.485

Hor distance (ft)	0	3	6	8	10	13	16	19	22
Red level (m)	141.7884	141.7168	141.7274	141.1727	141.0051	140.877	140.81	140.7765	140.7521
Hor distance (ft)	25	28	31	33	35	36	39	42	
Red level (m)	140.7368	140.7124	140.6911	140.7734	141.0873	141.8052	141.889	141.6909	

RD X-section (ft) 55220
Validity
Reference 54060-R, 54080-R, 55160-L
From crest of O/L 55160-L Ref. level (ft) 461.195

Hor distance (ft)	0	3	6	8	11	14	17	20	23
Red level (m)	141.6238	141.6314	141.6147	140.5646	140.5479	140.6241	140.7886	140.8923	140.8892
Hor distance (ft)	26	29	32	35	38	39	42	45	
Red level (m)	140.9228	140.9532	140.9837	140.9532	140.9807	141.5903	141.7427	141.825	

RD X-section (ft) 56050
Validity
Reference 56000-L
From crest of O/L 56000-L Ref. level (ft) 460.985

Hor distance (ft)	0	3	6	8	11	14	17	20	23
Red level (m)	141.8067	141.7244	141.575	140.621	140.6027	140.621	140.6119	140.5936	140.5783
Hor. distance (ft)	26	29	31	32	35	38			
Red level (m)	140.685	140.7124	140.9502	141.5537	141.5659	141.6238			

RD X-section (ft) 57700
Validity
Reference 57640-L
From crest of O/L 57640-L Ref. level (ft) 460.35

Hor distance	(ft)	0	3	6	8	10	13	16	19	22
Red level	(m)	141.4973	141.4638	141.4607	140.8725	140.6896	140.586	140.5707	140.5067	140.4884
Hor distance	(ft)	25	28	30	32	35	38			
Red level	(m)	140.5524	140.3726	140.65	141.4699	141.4485	141.543			

RD X-section (ft) 60460
Validity 60410-L
Reference From crest of O/L 60410-L Ref. level (ft) 460.24

Hor distance	(ft)	0	3	6	8	11	14	17	20	23
Red level	(m)	141.6406	141.6436	141.3358	140.3513	140.2994	140.3177	140.3452	140.3878	140.4275
Hor distance	(ft)	26	29	31	33	34	37	40	43	
Red level	(m)	140.4518	140.5311	140.6804	140.8603	141.2718	141.2108	141.1925	141.5095	

RD X-section (ft) 62200
Validity 62085-R, 62225-L
Reference From crest of O/L 62225-L Ref. level (ft) 459.785

Hor distance	(ft)	0	3	6	8	11	14	17	20	23
Red level	(m)	141.5811	141.3586	141.2337	140.3071	140.1638	140.2248	140.2583	140.2827	140.301
Hor distance	(ft)	26	28	30	33	36				
Red level	(m)	140.3406	140.5509	141.127	141.1879	141.7061				

RD X-section (ft) 65260
Validity Jiwan Minor, Drop at 65300 U/S
Reference From IIMI BM at RD 65300 Dro Ref. level (ft) 463.44

Hor distance	(ft)	0	3	6	8	11	14	17	20	23
Red level	(m)	141.0584	141.1956	141.1712	140.5219	140.4457	140.4366	140.3878	140.3634	140.3574
Hor distance	(ft)	26	29	32	35	37	38	41	44	
Red level	(m)	140.2781	140.2415	140.2263	140.2903	140.5738	141.1376	141.2961	141.2718	

RD X-section (ft) 67200
Validity RD 67160-L
Reference From crest of O/L 67160-L Ref. level (ft) 459.435

Hor distance	(ft)	0	3	6	8	10	12	14	17	20
Red level	(m)	140.7307	140.7277	140.8587	140.7155	140.4899	140.2004	140.0419	140.0541	140.1089
Hor distance	(ft)	23	26	29	32	35	37	39	41	44
Red level	(m)	140.1608	140.1973	140.2278	140.301	140.3558	140.496	140.6728	141.0233	141.1971
Hor distance	(ft)	47								
Red level	(m)	141.2794								

RD X-section (ft) 68300
Validity 68260-L
Reference Crest of O/L 68260-L Ref. level (ft) 459.57

Hor distance	(ft)	0	3	6	8	11	14	17	20	23
Red level	(m)	140.9212	141.0279	140.8344	140.1379	140.1013	140.0922	140.1044	140.1013	140.1013
Hor distance	(ft)	26	29	31	34	37	40	43		
Red level	(m)	140.0861	140.0586	140.845	140.7688	140.7292	140.8847	141.098		

RD X-section (ft) 70550
Validity 70530-R, 70600-L
Reference Crest of O/L 70530-R Ref. level (ft) 458.58

Hor distance	(ft)	0	3	6	8	10	13	16	19	22
Red level	(m)	141.0401	141.0919	140.8938	140.1836	139.952	139.8636	139.8422	139.8422	139.8727
Hor distance	(ft)	25	27	30	33					
Red level	(m)	139.9123	140.6896	140.7414	140.9395					

RD X-section (ft) 71400
Validity 71200-R, 71697-L
Reference crest of O/L 71200-R Ref. level (ft) 458.88

Hor distance	(ft)	0	3	6	8	10	13	16	19	22
Red level	(m)	140.845	140.5646	140.5707	140.2171	140.0465	139.9306	139.8819	139.8453	139.83
Hor distance	(ft)	25	27	29	32	35	38			
Red level	(m)	139.8514	139.8544	140.6469	140.9029	140.8816	140.97			

RD X-section (ft) 73000
Validity 73008-R
Reference Crest of O/L 73008-R Ref. level (ft) 458.62

Hor distance	(ft)	0	3	6	8	11	14	17	20	22
Red level	(m)	140.5981	140.6561	140.5799	139.827	139.7782	139.7843	139.8102	139.8331	139.8788
Hor distance	(ft)	24	25	28	31					
Red level	(m)	140.1836	140.5524	140.8908	140.9974					

RD. X-section	(ft)	75135								
Validity		75140-R								
Reference		Crest of O/L 75140-R			Ref. level (ft)		458.095			
Hor. distance	(ft)	0	3	6	8	11	14	17	20	23
Red. level	(m)	140.4412	140.5143	140.3955	139.7462	139.5969	139.5725	139.5816	139.6395	139.7828
Hor. distance	(ft)	25	28	31						
Red. level	(m)	140.4777	140.6484	140.6058						

RD. X-section	(ft)	76630								
Validity		76640-L								
Reference		Crest of O/L 76640-L			Ref. level (ft)		457.37			
Hor. distance	(ft)	0	3	6	8	11	14	17	20	23
Red. level	(m)	140.4214	140.4762	140.2659	139.5344	139.5192	139.5313	139.5435	139.5588	139.574
Hor. distance	(ft)	25	26.5	29	32					
Red. level	(m)	139.6563	140.2354	140.4915	140.5372					

RD. X-section	(ft)	78700								
Validity		78850-R								
Reference		Crest of O/L 78850-R			Ref. level (ft)		457.325			
Hor. distance	(ft)	0	3	6	8	9	12	15	18	21
Red. level	(m)	140.2034	140.2095	140.1364	139.6304	139.4353	139.3408	139.3652	139.3866	139.4719
Hor. distance	(ft)	23	26	29						
Red. level	(m)	140.112	140.1089	140.3497						

RD. X-section	(ft)	82650								
Validity		82600-L, 82700-R								
Reference		Crest of O/L 82600-L			Ref. level (ft)		456.205			
Hor. distance	(ft)	0	3	6	8	11	14	17	20	21.5
Red. level	(m)	140.0236	139.8803	139.8956	139.222	139.2037	139.1549	139.1122	139.1732	139.1884
Hor. distance	(ft)	22.5	25	28						
Red. level	(m)	139.9322	140.0754	140.0846						

RD. X-section	(ft)	83920								
Validity		83700-R, 84140-L								
Reference		Crest of O/L 84140-L			Ref. level (ft)		456.37			
Hor. distance	(ft)	0	3	6	8	11	14	17	20	22
Red. level	(m)	140.1105	139.8666	139.7599	139.2326	139.0711	139.0254	138.9888	139.0315	139.8605
Hor. distance	(ft)	25	28							
Red. level	(m)	139.8026	139.9398							

RD. X-section	(ft)	90060								
Validity		90000-R, 90080-L								
Reference		Crest of O/L 90080-L			Ref. level (ft)		455.12			
Hor. distance	(ft)	0	3	6	8	9	12	15	18	21
Red. level	(m)	139.51	139.4582	139.3972	138.8867	138.7572	138.7236	138.7663	138.8181	138.8852
Hor. distance	(ft)	24	26	29	32					
Red. level	(m)	138.9035	139.4155	139.5984	139.7752					

RD. X-section	(ft)	91980								
Validity		91960-R								
Reference		Crest of O/L 91960-R			Ref. level (ft)		454.655			
Hor. distance	(ft)	0	3	6	8	11	14	17	19	20.5
Red. level	(m)	139.4689	139.5054	139.2707	138.6124	138.5636	138.5057	138.5026	138.6489	139.1976
Hor. distance	(ft)	23	26	32						
Red. level	(m)	139.2037	139.3622	139.4445						

RD. X-section	(ft)	94170								
Validity		93970-R, 94350-R								
Reference		Crest of O/L 93970-R			Ref. level (ft)		453.94			
Hor. distance	(ft)	0	3	6	8	11	14	17	20	22.5
Red. level	(m)	139.2265	139.1564	139.2052	138.492	138.4432	138.5133	138.5529	138.6261	139.1412
Hor. distance	(ft)	25	28							
Red. level	(m)	139.318	139.2875							

RD. X-section	(ft)	95200								
Validity		95000-R								
Reference		Crest of O/L 95000-R			Ref. level (ft)		454.11			
Hor. distance	(ft)	0	3	6	8	10	13	16	19	22
Red. level	(m)	139.4673	139.2174	139.1351	139.1686	138.62	138.6048	138.5773	138.5377	138.6291
Hor. distance	(ft)	24	27	30	33					
Red. level	(m)	139.065	139.068	139.1839	139.2509					

RD. X-section (ft)	96450
Validity	96300-L, 96692-R
Reference	Crest of 96300-L Ref. level (ft) 453.88
Hor. distance (ft)	0 3 4 7 10 13 16 19 21
Red. level (m)	139.7493 139.2677 138.7312 138.6642 138.6368 138.6215 138.5849 138.6154 138.8257
Hor. distance (ft)	22 25 28
Red. level (m)	139.158 139.225 139.3408

RD. X-section (ft)	99505
Validity	99500-R
Reference	IIMI BM at RD 99500-R Ref. level (ft) 455.465
Hor. distance (ft)	0 3 6 8 11 14 17 20 22
Red. level (m)	138.9507 138.9842 138.8836 138.1735 138.0851 138.0485 138.1491 138.1933 138.7983
Hor. distance (ft)	25 28
Red. level (m)	139.0299 138.9309

RD. X-section (ft)	101805
Validity	101800-R
Reference	Crest of O/L 101800-R Ref. level (ft) 452.44
Hor. distance (ft)	0 3 6 8 11 14 17 19 21
Red. level (m)	138.8212 138.7328 138.6962 138.047 137.983 138.0134 138.0439 138.0835 138.6291
Hor. distance (ft)	22 25 28 30
Red. level (m)	138.6169 138.7175 138.9035 139.0406

RD. X-section (ft)	102850
Validity	102820-R
Reference	Crest of O/L 102820-R Ref. level (ft) 452.57
Hor. distance (ft)	0 3 6 8 11 14 17 19 21
Red. level (m)	138.9217 138.8303 138.6017 137.9799 137.8732 137.8732 137.8946 137.9433 138.4554
Hor. distance (ft)	24 27
Red. level (m)	138.6474 138.9461

RD. X-section (ft)	105010
Validity	104960-L
Reference	Crest of O/L 104960-L Ref. level (ft) 452.45
Hor. distance (ft)	0 3 5 6 9 12 15 17 19
Red. level (m)	138.7038 138.3106 137.7803 137.6096 137.4877 137.448 137.4297 137.6888 137.8443
Hor. distance (ft)	20.5 23
Red. level (m)	138.2679 138.5758

RD. X-section (ft)	105980
Validity	106000-R
Reference	Crest of O/L 106000-R Ref. level (ft) 451.655
Hor. distance (ft)	0 3 6 8 11 14 15 17.5 20
Red. level (m)	138.4478 138.3167 138.2344 137.5913 137.5608 137.5242 137.5608 138.1887 138.3259
Hor. distance (ft)	23
Red. level (m)	138.5301

RD. X-section (ft)	107900
Validity	107820-R
Reference	Crest of O/L 107820-R Ref. level (ft) 451.32
Hor. distance (ft)	0 3 6 8 11 14 16.5 19 22
Red. level (m)	138.3548 138.1293 138.0866 137.4953 137.3886 137.4252 138.0622 138.2847 138.5133

RD. X-section (ft)	112240
Validity	112250-R, 112400-L
Reference	Crest of O/L 112250-R Ref. level (ft) 449.995
Hor. distance (ft)	0 3 6 8 9 12 15 18 19
Red. level (m)	138.1704 138.2405 138.0607 137.4419 137.573 137.2225 137.2316 137.2347 137.317
Hor. distance (ft)	21 24 27
Red. level (m)	137.8321 138.015 137.7163

RD. X-section (ft)	114750
Validity	114700-R
Reference	From IIMI BM at RD 114700-R Ref. level (ft) 451.765
Hor. distance (ft)	0 3 6 8 11 14 16 19 22
Red. level (m)	138.0744 137.6751 137.6538 137.0442 136.9802 137.035 137.5837 137.6843 137.9342
Hor. distance (ft)	25
Red. level (m)	137.7574

RD. X-section (ft)	116630
Validity	116600-L
Reference	Crest of O/L 116600-L Ref. level (ft) 449.35
Hor. distance (ft)	0 3 6 8 11 14 16 18 21
Red. level (m)	138.0073 137.7544 137.4557 136.9771 136.9192 136.8918 136.9131 137.5532 137.7452
Hor. distance (ft)	24
Red. level (m)	137.5532

RD. X-section (ft)	118010
Validity	117775-R. 118000-R, 118250-R
Reference	Crest of O/L 118000-R Ref. level (ft) 448.743
Hor. distance (ft)	0 3 6 8 11 15 17.5 20 23
Red. level (m)	137.676 137.7004 137.359 136.8348 136.7982 136.8531 137.356 137.6151 137.609

RD. X-section (ft)	125030
Validity	125000-R, 125062-L
Reference	Crest of O/L 125000-R Ref. level (ft) 446.618
Hor. distance (ft)	0 2 4 6 7 9 11 13.5 14.5
Red. level (m)	137.1533 137.2722 137.3179 136.882 136.1414 136.0865 136.0834 136.1231 136.8881
Hor. distance (ft)	16.5 18.5 20.5 23
Red. level (m)	137.2813 137.2661 136.9765 136.7723

RD. X-section (ft)	130130
Validity	130100-R
Reference	From IIMI BM at O/L 130100-R Ref. level (ft) 448.988
Hor. distance (ft)	0 2 4 6 7 8 10 12 13
Red. level (m)	137.3636 137.4764 137.1777 136.7113 136.3761 135.8823 135.8488 135.867 136.3486
Hor. distance (ft)	15 16 18
Red. level (m)	136.7174 136.8211 136.7327

RD. X-section (ft)	134080
Validity	134100-R
Reference	Crest of O/L 134100-R Ref. level (ft) 444.933
Hor. distance (ft)	0 2 4 6 7 9 12 13 15
Red. level (m)	137.1426 137.2036 136.9902 136.4385 135.7832 135.7192 135.7741 136.2953 136.8104
Hor. distance (ft)	17 19 21
Red. level (m)	136.8896 136.7281 136.533

RD. X-section (ft)	135190
Validity	135180-R
Reference	Crest of O/L 135180-R Ref. level (ft) 444.738
Hor. distance (ft)	0 2 4 5 6 7.5 9.5 11.5 12.5
Red. level (m)	137.9458 137.641 137.3697 137.1167 136.1993 135.6171 135.5897 135.6019 136.1505
Hor. distance (ft)	14 16 18 20 23
Red. level (m)	136.501 136.8485 136.8089 136.8058 136.9674

RD. X-section (ft)	139770
Validity	139780-Tail
Reference	From IIMI BM at RD 139780 Ref. level (ft) 446.353
Hor. distance (ft)	0 2 4 6 7.25 9 10 11.5 12.5
Red. level (m)	136.2922 136.3105 136.0514 135.5973 135.0913 135.0487 135.0547 135.0944 135.5699
Hor. distance (ft)	14 16 19
Red. level (m)	135.6583 136.1581 136.0271

X-sections of singular points

RD. X-section (ft)	14900
Validity	15030 Drop U/S
Reference	From IIMI BM RD 15030 Bridg Ref. level (ft) 476.75
Hor. distance (ft)	0 3 6 8 10 13 16 19 22
Red. level (m)	144.6169 144.4904 144.4295 143.8747 143.4816 143.4755 143.4663 143.4633 143.4511
Hor. distance (ft)	25 28 31 34 37 40 42 44 46.5
Red. level (m)	143.4419 143.4297 143.4297 143.4267 143.4023 143.4114 143.4998 143.8961 144.4417
Hor. distance (ft)	49 52
Red. level (m)	144.6459 144.6124

RD. X-section (ft)	15100
Validity	15030 Drop D/S

Reference		From IIMI BM RD 15030 Bridg			Ref. level	(ft)	476.75			
Hor. distance	(ft)	0	3	5	7	10	13	16	19	22
Red. level	(m)	144.8074	144.5255	143.3139	143.128	143.1188	143.064	143.0792	143.1463	143.2194
Hor. distance	(ft)	25	28	31	34	37	40	43	45	47
Red. level	(m)	143.2209	143.1234	143.0122	142.7836	142.9329	142.9116	142.9299	142.9512	143.2987
Hor. distance	(ft)	49	52							
Red. level	(m)	144.8684	145.0513							

RD. X-section	(ft)	33050								
Validity		33300 Drop U/S								
Reference		From IIMI BM RD33300 Bridge			Ref. level	(ft)	470.1			
Hor. distance	(ft)	0	3	6	8	10	13	16	19	22
Red. level	(m)	143.4724	143.2865	142.9786	142.4605	142.2349	142.0673	142.1069	142.1069	142.1008
Hor. distance	(ft)	25	28	31	35	38	40	41	44	
Red. level	(m)	142.0795	142.052	142.0307	142.0398	142.0002	142.5062	143.2042	143.5029	

RD. X-section	(ft)	33500								
Validity		33300 Drop D/S								
Reference		From IIMI BM RD33300 Bridge			Ref. level	(ft)	470.1			
Hor. distance	(ft)	0	3	5	8	11	14	17	20	23
Red. level	(m)	143.1371	142.9451	142.1343	141.9911	142.0338	142.0673	142.0703	142.0795	142.1587
Hor. distance	(ft)	26	29	32	35	38	40	42	44	
Red. level	(m)	142.1374	142.1039	142.0185	141.9621	141.9179	141.9088	143.003	143.2255	

RD. X-section	(ft)	42835								
Validity		42850 Bridge U/S								
Reference		From IIMI BM at RD 42850 Bridge				Ref. level	(ft)	468.625		
Hor. distance	(ft)	0	3	5	7	10	13	16	19	22
Red. level	(m)	142.5138	142.398	141.6969	141.5781	141.5049	141.4348	141.3464	141.3159	141.2489
Hor. distance	(ft)	25	27	29	31	33	36			
Red. level	(m)	141.1696	141.5293	141.7427	142.3005	142.3614	142.5504			

RD. X-section	(ft)	43100								
Validity		42850 Bridge D/S								
Reference		From IIMI BM at RD 42850 Bridge				Ref. level	(ft)	468.625		
Hor. distance	(ft)	0	3	5	7	10	13	16	19	22
Red. level	(m)	142.5565	142.3568	141.6177	141.2641	141.319	141.4104	141.508	141.5872	141.6573
Hor. distance	(ft)	25	28	31	34	37	40	42	44	46
Red. level	(m)	141.7152	141.7061	141.6848	141.6665	141.636	141.6482	142.302	142.3919	142.5504

RD. X-section	(ft)	65260								
Validity		Drop at 65300 U/S								
Reference		From IIMI BM at RD 65300 Drop				Ref. level	(ft)	463.44		
Hor. distance	(ft)	0	3	6	8	11	14	17	20	23
Red. level	(m)	141.0584	141.1956	141.1712	140.5219	140.4457	140.4366	140.3878	140.3634	140.3574
Hor. distance	(ft)	26	29	32	35	37	38	41	44	
Red. level	(m)	140.2781	140.2415	140.2263	140.2903	140.5738	141.1376	141.2961	141.2718	

RD. X-section	(ft)	65450								
Validity		Drop at 65300 D/S								
Reference		From IIMI BM at RD 65300			Ref. level	(ft)	463.44			
Hor. distance	(ft)	0	3	6	8	11	14	17	20	23
Red. level	(m)	141.2413	141.1163	141.0675	140.4457	140.4579	140.4823	140.4732	140.4701	140.4732
Hor. distance	(ft)	26	29	32	34	37	40			
Red. level	(m)	140.4031	140.3177	140.3787	141.226	141.3571	141.3998			

Appendix E: Rapid assessment procedure

The start of this procedure is made by structuring the data of a distributary in a specific way. First a choice has to be made on how much time is available for field measurements. We suppose that a team of two persons can be made available for two days to take field measurements on one distributary. With a current meter this team can take five discharge measurement per day. A total of ten discharge measurements could then be made. The presumption of the procedure is that measurements on all outlets is too labor-intensive and that therefore measurements in the distributary should be taken.
The next step is to divide the distributary under investigation into the same number of sections as there will be measurements (in this example ten). The culturable command areas of these sections would preferably be of equal size or almost equal size. In figure 1 an example of the situation is given.

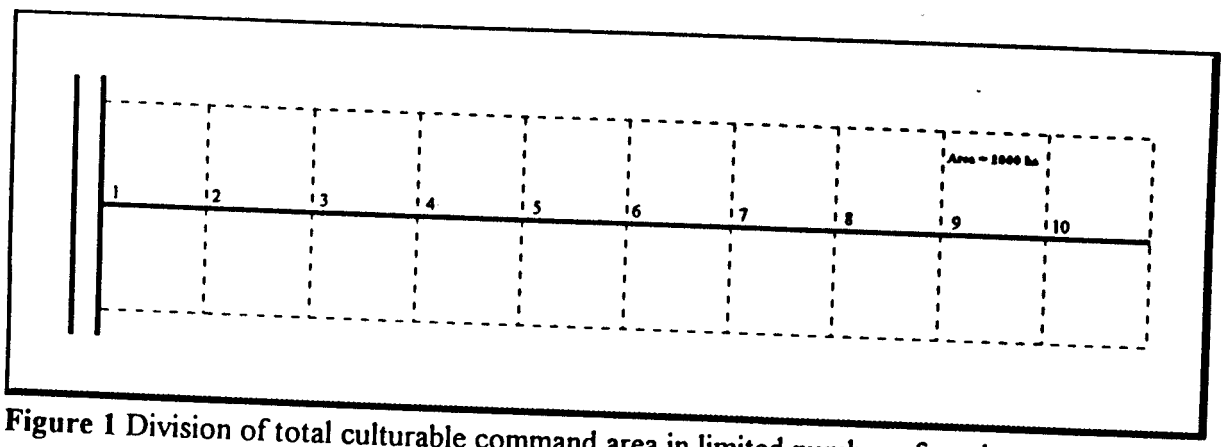


Figure 1 Division of total culturable command area in limited number of sections

The reaches between the numbers all have authorized outgoing discharges which are the totals of the authorized discharges of the outlets in that reach. These totals can be subtracted from the discharge in the distributary. With the Outlet Register and the known discharges deducted for the seepage a list should then be made. This list is given in table 1. The discharge measurements in the distributary can be done after this.
Before the field measurements can start it must be absolutely certain that the distributary is running at design discharge. Therefore the measurements must start on the upstream side. If the discharge is not correct, the gateopening must be adjusted until it is correct. This can take some time. After this is done, the measurement at the head must be repeated in order to check if the distributary is now running at the required discharge. Now the rest of the current meterings can be done.

Location	1	2	3	4	5	6	7	8	9	10
Qdesign	10.00	[illegible]	[illegible]	[illegible]	[illegible]	3.00	4.00	3.00	2.00	1.00
Qmeasured	10.00	[illegible]	7.50	[illegible]	[illegible]	4.00	3.50	2.50	1.50	0.50
Qout, design	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Q out, actual	2.00	0.50	1.00	1.50	1.00	0.50	1.00	1.00	1.00	0.50
Qout, effective	1.00	0.50	1.00	1.00	1.00	0.50	1.00	1.00	1.00	0.50
Qout, ineffective	1.00	0.00	0.00	0.50	0.00	0.00	0.00	0.00	0.00	0.00

Table 1 Example

Results

With these measurements, the total effective and ineffective distributed discharge can easily be calculated:

Total effective outflow = Σ Qout, effective = 8.500 m^3/s
Percentage effective discharge = (Σ Qout, effective)/ Qdesign =85 %

The figures in the table above can be given in a graphical way to show the results more clearly

With a limited number of measurements the effectiveness of the distribution within a distributary can thus be determined. Next to that, the sections in which the outflow is too high can be located very easily. With these figures, the manager then has to decide whether or not to intervene.

An important side result of the procedure is that the kD formula used by the gage reader for the determination of the discharge at the head of the distributary can also be checked.

The most important disadvantage of this procedure is that at the head of the distributary, the outgoing discharges are comparatively small compared to the ongoing discharges. Therefore two large, almost equal numbers are to be subtracted to find a third value. This can cause errors.

It is important to realize that the physical properties of the channel are not investigated. The procedure focussess solely on the waterdistribution. The procedure to assess the physical condition of the distributary is an independant procedure. It contains inspection of berms, freeboard, siltation, waterlevels, submergence of cross-structures and widths of the channel.