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Development of a Watercourse-based Model to Assess the Canal Water Supply at the Farm Level

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CHAPTER 1

Irrigation Systems in Pakistan

PRESENTATION

Importance of Irrigation in Pakistan

Pakistan's climate is arid to semiarid. The temperature in most cultivable areas allows for year-round cultivafion. Annual precipitation over much of the Indus Plain ranges from 150 mm to 500 mm, whereas evaporation varies from 1,250 mm to 2,800 mm. Uneven rainfall and evaporation make crop production impossible withoul irrigation.

The total area of Pakistan is 79.6 million hectares (Mha). In 1988-89, the total cultivable area was 21 Mha (26 percent of the total area), of which 16.2 Mha (77 percent of the cultivable area) were irrigated. Of the total cultivable area of 21 Mha, 10.4 Mha were single-cropped, 5.7 Mha were double-cropped and 4.9 Mha were left fallow. Currently, irrigated agriculture accounts for about 90 percent of Pakistan's agricultural output. Irrigation is also essential for meeting demands for food, processing and exports (World Bank 1993).

The Indus Basin Irrigation System

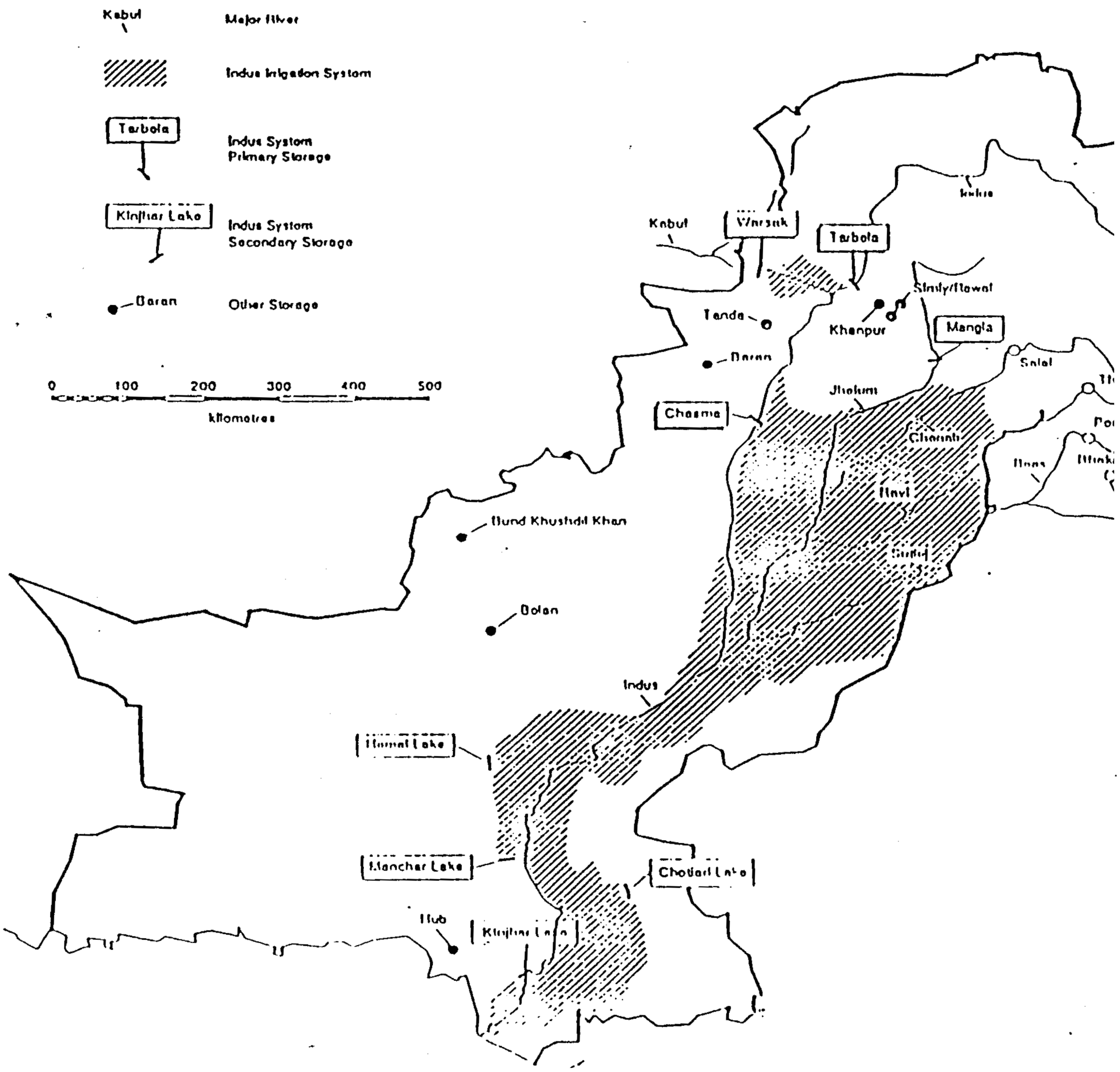
Pakistan is divided into three hydrological units: The Indus Plain covering an area of over 566,000 km^2 (70 percent of the area of the country); the Kharan Desert (15 percent) in the West Baluchistan with its inland drainage; and the arid Makran Coast (15 percent) along the Arabian Sea in the Southern part of Baluchistan.

The Indus and its tributaries have their sources in the Himalayan mountains and the Hindu Kush, with a total catchment area of 400,000 km^2, larger than that of the Ganges and the Brahamaputra. The inflow to these rivers is mainly derived from snow and glacier melt and rainfall in the catchment areas. The tributaries of the Indus, originating in India but flowing into Pakistan are the Jhelum, Chenab, Ravi and Sutlej (with a major tributary, Beas) rivers. Originating from Afghanistan, the other major tributary of the Indus is the Kabul River.

Under the Indus Water Treaty of 1960, the flows of the three so-called Eastern Rivers (Sutlej, Beas, and Ravi) have been allocated to India, whereas, with minor exceptions, Pakistan is entitled to all the water of the Western Rivers (Indus, Jhelum, and Chenab).

The Indus Basin Irrigation System (14.2 Mha) covers the world's largest contiguous irrigated area (Figure 1.1). It comprises two storage reservoirs, 16 barrages, 12 inter-river link canals, 2 syphons, and 43 main canals. The total design diversion capacity of the main canals is 128 billions cubic meters in a whole year. The total length of the link canals, main canal branches and distributaries is about 57,000 km.

Figure 1.1. Indus Irrigation ***system.***



The system has 88,600 outlets for the irrigation of service areas by tertiary canals or watercourses. The length of the farm channels and watercourses *is* about 1.6 million km.

Large-scale irrigation development was initiated by the British. The first controlled all-year irrigation began in 1859 with the completion of the Upper Bari **Doab** Canal offtaking from Madhopur Headwork on the Ravi River. The irrigation system has been developed on the run-off-the river flows; it has been improved since independence with the construction of two storage reservoirs. Still, supplies diverted to irrigation canals are subject to variation depending on the flow conditions. Out of the total irrigated area, 60 percent is perennial (supplying water all year round) and the remaining, nonperennial, is entitled to irrigation supplies only during the summer or the *kharif* season (from **15** April to 15 October).

A basic feature of the irrigation system has been to spread the irrigation water over as large an area as possible to expand settlement opportunities. In a context of scarce water, the objective of the Provincial Irrigation Departments, operating the system in each province, is to supply the canal water *on an* ***equitable*** basis.

In most of the major perennial systems, the entitlement of water from the irrigation outlet was fixed at 200 l/s for 1,000 ha of Culturable Command Area (CCA) (equivalent to **1.8** mm/day over the irrigable area) (Badruddin 1993). As this water would be insufficient to irrigate all of the irrigable areas, cropping intensities were, restricted at the design stage to around 25 percent for the winter or the *rabi* season (from 15 October to 15 April) and 50 percent for the kharif season (Badruddin 1993).

Groundwater Resources Development

Groundwater resources started to be an essential part of this irrigation system in the 1960s, with the installation of about 12,000 (Strosser 1994) public tubewells under several Salinity Control and Reclamation Programs (SCARPs). The main objectives of these programs were to reduce waterlogging problems and increase irrigation water supplies in areas with good quality groundwater.

Farmers have increased the low cropping intensities by investing in tubewells to tap ground-water resources, thus augmenting their water supply and enhancing the flexibility of their irrigation water supply. Private tubewell densities in canal command areas of the Punjab vary from 2 tubewells per 100 hectares of Culturable Command Area (CCA) to more than 13 tubewells per 100 hectares of CCA. with an average of 6-8 tubewells per 100 hectares of CCA (Strosser and Kuper 1993). Conservative estimates indicate that **40** percent *of* the total irrigation water supply at the farm gate in Punjab is derived from private tubewell supplies (Vander Velde and Johnson 1992).

A group of few small farmers have invested commonly in tubewells, sharing the operation and maintenance costs and managing their tubewell jointly. However, tubewells have mainly remained an attribute of larger farms (WAPDA 1980). Farmers who do not own tubewells also have access to the groundwater resources via the operation of a market for groundwater.

PROBLEMS FACED BY THE SYSTEM

Yields of the main crops in Pakistan are among the lowest in the world (World Bank 1993). Moreover, while the recorded increase in productivity has been satisfactory during the 1960s and 1970s (partly boosted by groundwater development) there is now a general recognition that the productivity of the main crops (apart for cotlon) is stagnant or even follow a decreasing trend (Bandaragoda 1993). The main reasons for this poor performance of irrigated agriculture in Pakistan are summarized below.

Poor Performance of the Public Sector

The low productivity of irrigated agriculture in Pakistan is partly explained by a poor performance of the public sector in supplying water; average delivery efficiency is estimated to range from 35 percent to 40 percent from the canal head to the root zone with maximum losses occurring in the watercourses (farm channels). (World Bank 1993). The Provincial Irrigation Departments are not able to attain their operational objective of supplying water equitably and they supply water in an unreliable manner (Kuper and Kijne 1992).

The main causes of this poor performance are:

- The funds allocated to the Operation and Maintenance (O&M) are inadequate (World Bank 1993). The system being more than a hundred years old, the irrigation facilities have been deteriorating.
- The rules of the system, defined by the British more than a century ago are outdated. In addition, general breakdown in the discipline and illegal pumping from the canals are major factors causing inequity in the distribution of water.

The direct effect on crop yields of such a low performance of the public sector is accompanied by an indirect effect on the sustainability of the agricultural sector (Strosser and Rieu 1993). Three environmental aspects are detailed below.

Waterlogging

The increase in the diversion of river flows for irrigation, seepage from the canals, watercourses, and irrigated areas, together with a flat topography and the lack of well-defined natural drainage in the Indus Plain have resulted in a gradual rise of the groundwater table. Heavy investments have been made to tackle this problem, first via the installation of public tubewells in the 1960s and the 1970s, and more recently with the installation of subsurface tile-drainage systems. At present, about 30 percent of the gross command area (GCA) is waterlogged and about 13 percent is considered highly waterlogged (World Bank 1993).

Salinity

Even if irrigated water is relatively free of salts, repeated irrigation and the rise in the water table will dissolve salts in the soil and bring them towards the surface. It is estimated that about 8 percent of the GCA is severely salt affected and another 6 percent is moderately affected (World Bank 1993).

Mining of the Aquifer

Even in the canal command areas, groundwater is the only source of water supply for many users because of the inequitable distribution of surface water. Tubewell pumpages recycles salts and the losses from the surface system. Due to the explosive development of groundwater by the private sector there is an increasing danger of excessive lowering of water tables and intrusion of saline water into the fresh aquifer.

SEARCH FOR NEW SOLUTIONS

While technological interventions still gather most of the financial resources allocated to the search for solutions (as shown by a recent lining program named Water Conservation project, to be implemented by the Punjab Irrigation Department and financed by the Asian Development Bank), there ***is*** an increasing interest in more software-oriented solutions. Alternative water allocation mechanisms, particularly watermarkets, are increasingly seen (the World Bank strongly advocates this solution (World Bank 1993)) as a means to increase irrigation efficiency and provide incentives for improved resource' management.

Water markets already exist *in* Pakistan: involving an important part of the farming community (Strosser and Kuper 1994), water markets do not relate only *to* tubewell water but also to canal water, even though the Canal and Drainage **Act** of 1873 forbids farmers to trade their canal water turns.

As little knowledge exists on how these markets operate and their relation to water use efficiency, the Pakistan Division of the International Irrigation Management Institute (IIMI) has launched a research program on this subject, as part of the IIMI-CEMAGREF[1] joint research program. The main objective of this research is to evaluate the feasibility and related consequences of water market development in Pakistan. The analysis of the supply, the **demand** and the **allocation** of irrigation water at the watercourse level forms the basis of the proposed research (Strosser and Rieu 1993). The analysis of the demand will be derived from the analysis of the farming systems and the modelling of representative farms. Comparing the demand to the supply of water at the farm level will help to understand the reasons for emerging water markets, and their possible further development.

[1] **CEMAGREF: Centre National du Machinisme Agricole, du Genie Rural, *des* Eaux et des Forets.**

CHAPTER 2

Objectives and Methodology

OBJECTIVES

The present *report* focuses on *the* supply component. The main objective is to estimate, with accuracy, the characteristics of the irrigation canal water supply at ***any*** point *(farm* gate) within *a* selected watercourse.

The following highlights some of the problems encountered in estimating canal water supply at the farm gate.

The present water allocation system within a watercourse command area is known as warabandi (wara means turn and bandi means fixation). The warabandi is basically a continuous rotation system in which one complete cycle of rotation usually lasts seven days, i.e., each farm receives its water once a week for a fixed duration. During his turn, a farmer ***is*** entitled to all the water flowing in the watercourse (Malhotra 1982). The duration of a water turn is based on the area operated by each farmer.

In theory, it allocates the canal water equitably among the farmers within a watercourse command area. However, the watercourse level is quoted by Malhotra (1982) as the level having the most important conveyance losses, and the lowest efficiency. As he states in his survey of the warabandi, the watercourse is expected to lose more water then the distributary as the wetted perimeter per cubic liter per second is ten times higher in the watercourse then in the distributary (on the assumption that the absorption loss is a direct function of the wetted perimeter).

Therefore, assessing the volume of water at the farm level induces the need for a study of watercourse features and, more particularly, of seepage losses. During the 1970s, such studies were carried out at the watercourse level by the Water and Power Development Authority (WAPDA) to assess the conveyance efficiency at the watercourse level:

- Several of these studies focused on the losses under steady-state conditions; losses were taken as the decrease in the rate of flow along the channels. They are summarized in Ashraf et al. (1977)

- As the previous studies failed to measure the additional water losses that occur during the operation of a watercourse system such as transient infiltration, dead storage, and water leaking through short-term watercourse breaches, more comprehensive studies were undertaken by Ali et al. (1978). following Trout (1977) and Bowers et al. (1978).

Two different measurement methodologies were carried out during these **studies:**

(i) The ponding method where seepage losses are deduced from the decrease of water in a section closed at both ends.

(ii) The fluming method, where two discharge measurement devices assess losses by an inflow outflow difference.

As some differences between these two measurement methodologies were highlighted by the previous studies, a complete survey of watercourse losses with the ponding method was undertaken by Trout et al (1981) In Ali et al. (1978), the comparison between ponding and fluming is also carried out.

The main purpose of these studies was to assess the possible benefits of lining watercourses, in On-Farm Water Management programs

In order to assess conveyance efficiency and losses along the watercourse, these studies monitored irrigation water distribution within the watercourse. It involved heavy field work, as staled by Ali and Al (1978), who used 5 engineers per watercourse during a three-week measurement program

However, these studies did not focus on the volume of water effectively received by farmer nor did they fully integrated the complete weekly rotation of turns in their analysis.

More recently, Sarwar (1991) developed a model for equitable water allocation along the watercourses in order to take into account the seepage losses in the conception of the warabandi.

The supply component of the research program launched by IIMI-CEMAGREF differs from the previous works by its objective which involves

- Testing the impact of water allocation scenarios (current practices versus strict warabandi versus water markets fully developed).
- Estimating the characteristics of the water supply at the farm gale during the whole year.

As it is irrelevant to think of a measurement survey as the one carried out by WAPDA in the 1970s, for example, for the assessment of the volume at farm level during the whole year , and as the objective involves the simulation of new allocations of canal water, the development *of* a *simulation* tool at the *watercourse* level *is* seen *as* the *most* appropriate approach and *is* the first step of ***the*** supply component *of* the research program.

PROPOSEDRESEARCHMETHODOLOGY

The *aim of* the present study-is the development *of a* hydraulic model *to estimate the volume of* canal water supplied Io farmers.

The approach chosen is described below.

In a first step,the characteristics of the canal water supply system at the watercourse level is described, in order to choose an appropriate description for the model, including:

1. The physical descriptiori of the watercourse

2. A brief analysis of the water allocation mechanisms (theory versus practice) at the watercourse level.

In a second step, a hydraulic **model** adapted to the watercourse **level** is developed. This includes:

1 A review of the different models used in furrow and border irrigation, along with their data requirements.

2. The analysis of the specific features of canal water allocation at the watercourse level, and the expected impact on the characteristics of the model.

3. The development of the model itself (programming).

In a third step, the *model* is calibrated for four sample watercourses with a specific focus on:

1. The assessment of seepage losses. with Ihe comparison of two measurements methods (ponding method versus fluming method)

2. The assessment of the advance phase characteristics.

3. An evaluation of the validify of the model and of its limitations

The fourth and last step is a first application of the model, **and** the use of its output for the analysis of the canal water supply performance at the farm level.

DESCRIPTION OF THE RESEARCH LOCALE

The field research activities required for the present study have taken place at an IIMI field site located in the Fordwah/Eastern Sadiqia irrigation system (Southeast Punjab) which is representative of the cotton-wheat agro-ecological zone (Figure 2 1)

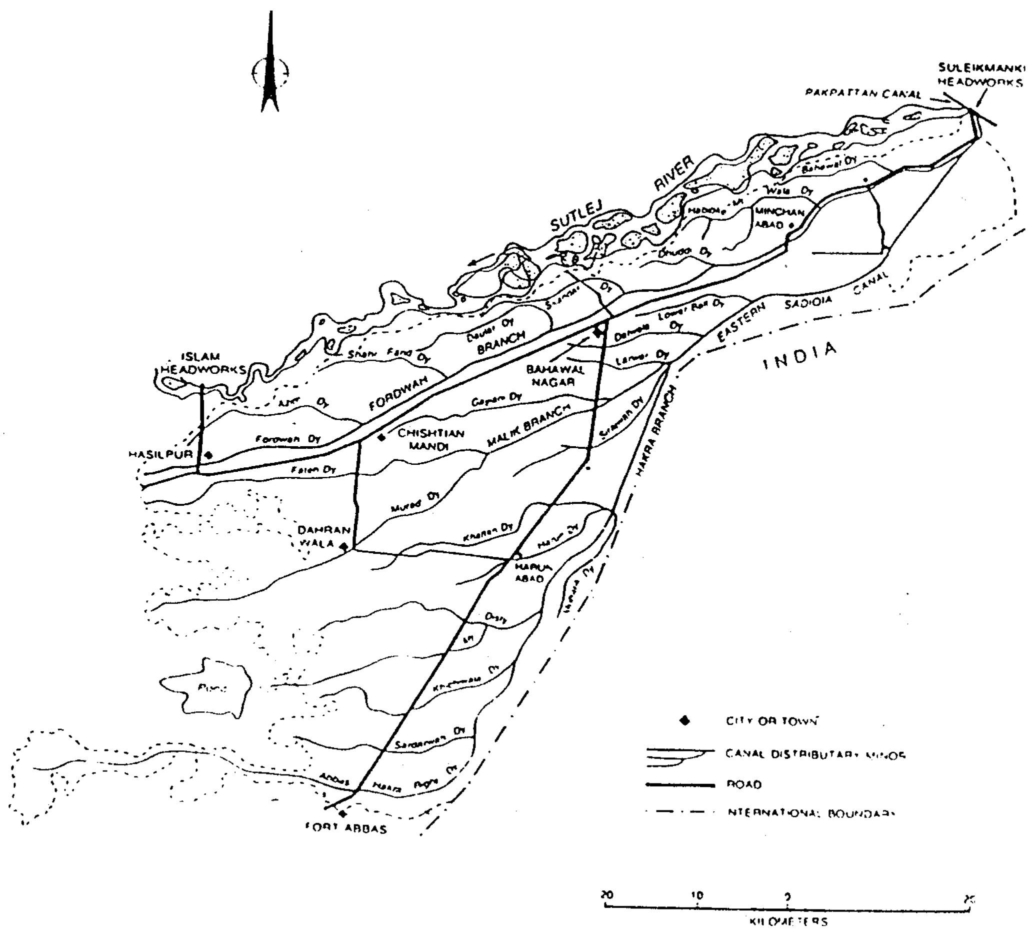
The eight watercourse command areas selected for the study have been monitored for more than a year Weekly effective warabandi and the discharge at the watercourse outlet have been recorded Four sample watercourses are located along the Fordwah Distributary (perennial canal) and the other four are located along the Azim Distributary (non-perennial canal) scattered from the head to the tail of these two distributaries

The study has mainly focused on the four watercourses located along the Fordwah Distributary However, some of the data, the ponding method results for example, have been collected on both distributaries.

In Table 2.1, the salient characteristics of the Fordwah Distributary are presented (Kuper and Kijne 1992).

Length (km)	**41**
Design discharge (m^3/s)	**4.5**
Culturable Command Area (ha)	**14,941**
Water allocation (l/s/ha)	**0.25**

Figure 2.1. The Fordwah/Eastern Sadiquia area in Southeast Punjab



Land-holdings in the Punjab canal colonies were laid out on a grid system of 25 acres (or 5'5 acre square); location of a field within a watercourse command area referred to the grid system. The area served by a particular watercourse may range from **40** lo **400** ha but the average is around 180 ha The flow in the watercourse command area is governed by an open outlet or ***mogha*** design to pass a discharge that self-adjusts in proportion to the ***flow*** in the parent canal. Design discharges for moghas can be less than 30 l/s and does not exceed 120 l/s to enable an efficient handling by individual farmers

A watercourse command area is named after its location *on* the distributary : Fordwah 14320-R, for example, refers to the distance from the head of the distributary in feet **(14320),** and the right side of the distributary (R). This watercourse, **4** kilometers from the head, will be named in the study as FD14.

The furthest watercourse, Fordwah 130100-R is located two kilometers before the end of the distributary.

Table 2.2. *The* main characteristics *of the four* sample watercourses from *the Fordwah* Distributary.'

	Area (ha)	Number of land owners	Number *of* turns	Number of private tubewells	Design **Q** (l/s)	Actual Q (l/s)
FDI4	226	50	85	**16**	50	60
FD46	182	**19**	**53**	**7**	**43**	**93**
FD62	140	**36**	**53**	**15**	33	62
FD130	205	**49**	73	**16**	**68**	**75**

The characteristics of the four sample watercourses on the Azim Distributary are not quoted here, because they are not useful for the study.

[?] The actual Q is the average discharge at the outlet when the distributary is at full supply, and design Q is the design value

CHAPTER 3

Description of the Canal Water Supply System

The aim of this part is to select:

(i) A representation of the watercourse for the model.

(ii) A representation of the canal water allocation system for the model.

The spatial organization of the watercourse is first described together with a literature review of the works on the conveyance efficiency, that helps to foresee the seepage losses in the different kinds of channels.

As the different systems of canal water allocation are closely linked with the spatial organization of the watercourse, they are studied in the second part of this chapter.

Given the characteristics of the different channels on the one hand, and their use by a canal water allocation system on the other, the choice is made of an optimum spatial representation for the hydraulic model.

THE WATERCOURSE

The water conveyance system below the outlet (or mogha) can be basically divided into two portions:

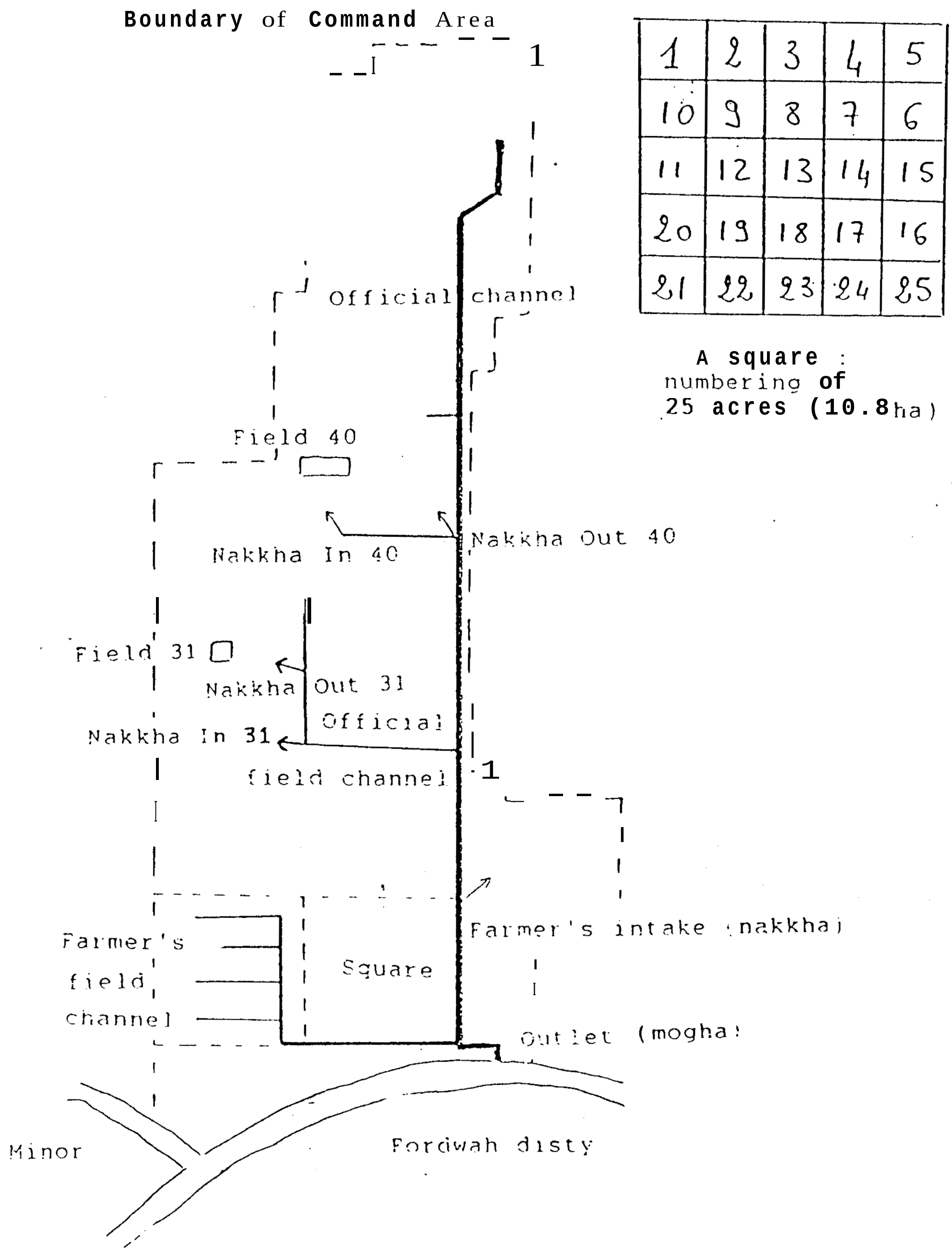
(i) The *sarkhari khal* or official channel laid out and constructed by the government.

(ii) The field channel which lead from the sarkhari khal to the individual fields. This portion can be further subdivided into two components:

- The official field channels, recorded in the farmer's warabandi, which belongs to the community.
- The farmer's fieldchannel offlaking from the official field channel or from the sarkhari khal and leading to the individual farmer's fields.

Figure 3.1 represents the schematic watercourse physical organization.

Figure 3 1 Schematic map of a Watercourse Command Area.



The Official Channels

The original intent of the official channels was to deliver water on a corner of each square of the watercourse command area The locations of these farm outlets (or *nakkha*) are taken into account by the canal officer (also called *patwari*) to lay out the official warabandi. In reality, a few squares of the sample Watercourses command area have no official nakkha (an average of 20% for the four sample watercourses)

The official channel can be lined under the lining program undertaken by the On-Farm Water Management Program, usually as the portion of channel located at the head of the watercourse which has a higher percentage of use In the four sample watercourses, only the head channel (300 meter long) of FD14 is lined

The official channel is located between the boundaries of farmers' landholdings and runs along the borders of a square This official channel is the basis of the so-called official warabandi, designed by the canal officer

This type of channel is usually better maintained than farmers' field channels; its maintenance is the responsibility of the farmers who have a common right to use it, and cleaning activities are regularly undertaken (a minimum of twice a year). However, in some areas, it is sometimes quite difficult to distinguish between official channels and other channels

The width of the bed of the official channel can vary from 0 5 meter lo 0.8 meter, and the slope of its banks from 1/3 to 1

The Field Channels

These channels are built by the farmers themselves and offtake from the official channel In the farmers' warabandi recorded on the sample watercourses, some of these channels are taken into account following an agreement among farmers They are named in the rest of the report as official field channels; beyond these channels i e beyond the nakkha stated in the farmers' warabandi, and reaching the farmers' fields are the farmers' field channels

The Official Field Channels

These channels have the same physical characteristics as the official channel described above. The joint system of the official channel and official field channels supplies canal water to each square of the command area In some sample watercourses, like FD62, the official field channel system does not supply water beyond the square level In FD14, many squares on the border of the main branch are supplied with canal water by an official field channel of an average length of 200 meters

The Farmers' Field Channels

In a watercourse where only one nakkha supplies each square, these channels may be as long as 500 meters Their average length is around 300 meters Holes have been dug every 30 m for the irrigation of one field and are open or closed by farmers before and afler the irrigation of the specific field The banks of these reaches are weaker and as it is used only once a week; rat holes may have damaged them.

The Nakkhas

A nakkha can either be a concrete structure (it *is* then called pacca nakkha) or a hole dug in a reach bank (kaccha *nakkha).*

A pacca nakkha eases the operation, but is often found to leak. In *a* command area where part of the watercourse has been lined (as in FD14), pacca nakkhas have also been built. However, the tail end has not been provided with these pacca nakkhas. In other command areas, the installation of pacca nakkha depends on farmers' will. For the same reasons as for the lining of the watercourse, nakkhas will be improved starting from the head of the watercourse.

The summary of the physical characteristics of the four sample watercourses of Fordwah Distributary is presented in Table 3 1.

Table 3 1 Physical characteristics of *the* four *sample* watercourses.

	FD14	FD46	FD62	FD130
Total length (m)	5810	3680	3251	8565
Farm intake	29	14	19	35
Main branches	3	1	1	2
Length (m)	4100	2500	2000	6100

Maps of these watercourses, along with a more comprehensive set of data for each watercourse, and the result of the calibration work *of* each watercourse, are given in Annex 3.

ASSESSMENTOF THE LOSSES AT THE WATERCOURSE LEVEL

In order to have a first assessment of the seepage in the different channels, we will refer to the extensive survey carried out by Ali et al. (1978) on three watercourses; the results presented by Ali et al. were consistent with similar studies carried out in the 1970s by WAPOA. In Table 3.2, are some of the final results on the conveyance losses highlighted in the survey of Ali at al. According to these results, 30 to 50 percent of the total conveyance losses occurred in the field channels.

Table 3.2. Conveyance losses in three different watercourses *(Ali et al. 1978).*

Watercourse	#6	#35	#52
Description			
Size (CCA) (ha)	148	120	128
Inflow rate (l/s)	60	37	36
Conveyance losses			
In official channels	19%	20%	34%
In field channels	21%	19%	17%
Total operational	40%	39%	51%

In the quoted study, no difference was made between the field official channels and the farmer's field channels. The field channels could be up to 600 meters long.

The study stated that average steady state **loss** rates per unit length in field channels were twice as much as those measured in the sarkhari khal channels. One of the primary reasons is that they are used much less often than the sarkhari khal channels, and the percent **of** usage correlates inversely with **loss** rates (Trout and Al ***1981).***

As the field channels are used only an average **of** one fifth of the total time of the warabandi, they contribute to 1/3 to 1/2 of the total seepage in the watercourse.

In the present study, the loss rate of the field official channel can be expected to be an intermediary between these two categories. These official field channels reduce the importance of the farmer's field channel.

Hence, the taking into account **of** the official field channels and the official channels in order to measure water supply **at** the farm's nakkha **will** neglect an estimated **25** percent to 35 percent of the total losses, depending on the extension of the official field channels.

THE CANAL WATER ALLOCATION SYSTEM

The warabandi and the water turns effectively used by each farmer is an important factor influencing the quantity of water received by each farmer. **At** present, within a watercourse area, three different kinds of warabandi can be identified.

After a presentation of each type, we will select one as the input describing the roster **of** turns in **the** model.

The Official Warabandi

This warabandi is a list in which the patwari registers for each farmer:

- His water turn (starling time and ending time).
- The in-nakkha and out-nakkha (both official).
- The area irrigable during the given water turn.

Originally, as described above, one official nakkha only was supposed to supply one square. In some watercourses, following farmers' requests, new nakkhas have been added on the official channel and have become official nakkhas.

The in-nakkha is the location where the farmer receives his water from the previous turn, the out-nakkha is the location where he is due to give the water to the next farmer. As shown on the schematic map, two different cases can occur :

- The in-nakkha is upstream of the out-nakkha. The advance phase takes place at the beginning of the farmer's water turn. The out-nakkha is the closest to the farmer's field.
- The in-nakkha is upstream of the out-nakkha. The water turn finishes with a drainage phase. The in-nakkha is closest to the farm's field.

The total time available (one week or **168** hours) divided by the Cultivable Command Area of the watercourse gives the irrigation time per acre.

The water turn of each farmer is made on this basis, with a duration proportional to the farmer's operated area and taking into account the advance time and the drainage time that may influence the quantity of canal water received during a water turn. The advance time or *Khal Barai* is added to the total irrigation time while the drainage time or *Nikhal* is subtracted from the total irrigation time.

The Khal Barai for each farmer is assessed by the patwari on the basis of **3** minutes per **67** meters of watercourse (the side of one acre). The Nikhal is assessed on the same basis of 3 minutes per 67 meters. It is the same for all farmers and does not incorporate differences due to **soil** type, watercourse characteristics, or seepage loss rate. An example of calculation of a water turn duration in **FD14** is shown below.

Water turn duration as calculated *by* the *patwari*

Watercourse: **FD14**
Culturable command area: 226 ha.
Average time calculated per hectare: **45** minutes. Farmer:
Area: 3 hectares.
Advance length : **268** meters i.e., **4*67** meters.
Drainage length : **670** meters i.e., 10*67 meters.
Total Water turn : **3*45** + **4*3** - **10*3** = **117** minutes.

The warabandi starts at the nakkha closest to the watercourse outlet, and goes downstream, one branch of the watercourse after the other.

Afler the annual canal closure in January for desiltation and maintenance of the conveyance system, the start of the warabandi will change from morning to night (or the contrary), *so* that farmers irrigating by night before the closure period will irrigate during day time after the closure period.

As a farmer's operated land area in a command area can vary, by shifting some land from one command area to the other, or by dividing it between brothers, the official warabandi can be regularly updated when requested by the farmers. However this is not a current practice as described in the following section.

The Farmers' Warabandi

In FD14, the last official warabandi was made three years ago. On Azim Distributary. 40 years have passed since the last change in the official warabandi has been implemented.

In FD130 , following a quarrel between farmers on the location of the main watercourse branch, two branches irrigate the command area. Instead of a natural sequence of water turns from head lo tail of each branch, the water turns alternate from one branch to the other lwlce per week because of the influence of some farmers and the lack of agreement.

In fact, most of the time, the warabandi in practice is different from the official warabandi, designed by the patwari. The main reason is that the official warabandi is not flexible enough: changes occur quickly in the land allocation or in the organization of the watercourse (implementation of a new nakkha). and the official warabandi is not updated often enough to take these new changes into consideration.

The farmers' warabandi, however, takes into account unofficial nakkhas. not recognized by the canal officer as point, of delivery of water to the farmers. These unofficial nakkhas can be located either on an official or on an official field channel

The way this warabandi is worked out is the same as the official warabandi. It is usually valid for one season as changes occur at the end of a season: contracts of lessee or tenants may change after the season, and farmers choose to implement changes in their landholding at this time of the year.

In order to improve the equity of the canal water supply within the watercourse command area, some agreements between farmers are implemented during a whole season or a year, Some of these agreements are:

- Rotations between 2,3 or more farmers often occur. Each week, a different farmer wilt either have the drawback of the advance phase or the benefit of the drainage phase.

- Water is sometimes used by 2 or 3 farmers during the same water turn or combined turn. Those farmers usually belong to the same family.

- When farmers have a small surface with less then half an hour of water turn, they may lease their turn to other farmers. The duration of the water turn is not long enough to irrigate the whole area of farmer's land, Thus, to give the turn to the next larger farmer and to get the turns back every month or every three weeks from this large farmer is seen as a better option that enables small farmers to use enough water in order to irrigate their total land area.

These three agreements can be combined together and some local agreements are somehow unique.

The Weekly Warabandi

Within their warabandi, farmers may still sell part of their water turn, or borrow some water that they will give back the following week in order to irrigate their fields in a proper manner.

Research has shown that farmers trade tubewell water more often than canal water. However, the importance of transactions with canal water is far from being negllgible. On average, **15** percent of the water turns of the rigid warabandi system are transacted by the irrigators (Strosser and Kuper **1994).**

CONCLUSION

For calculation of volume of water, losses within channels and the warabandi times are basic information that will be used in the following part of the report as an input of the hydraulic model. However, choices are required at this stage, as shown by the complexity of the situation (different kinds of channels, different types of warabandi).

The choices made are:

- The farmers' warabandi is used to describe the roster of turn.

- The *official* channels and the official *field* channels are considered, but not the farmers tield channels.

The farmers' warabandi is more updated than the official warabandi, and relatively close to reality, since the changes implemented by the farmers In the weekly warabandi are limited. Hence, in this report, the weekly canal water transactions are not included.

In the different warabandi, the location of the farms is stated by a nakkha. In the farmers' warabandi, beyond these nakkhas are the farmers' field channel which, in this presenl study, could contribute to 25 percent to **35** percent of the global conveyance losses. Assessing water at the nakkha level neglects such losses, but considerably eases Ihe field work as the mapping of the two klnds of official channels is simple.

The improvement of the physical representation of the watercourse and the assessment of the volume at the farm level involves:

- A survey of the farmer's field locations.

- The mapping of the different farmer's field branches irrigating these **fields.**

- The calibration of these branches.

Even a model with such a precise representation will not reach the level of accuracy of the monitoring of the whole watercourse command area, because it will not know which field is irrigated by a given farmer during his water turn

Figure 3.2 and Figure 3.3 show an example in FD62, comparing the exact location of the channels (official and official fields) with the farmer's field location.

Figure 3.2 Fanner's field location in Fordwah 62085-R

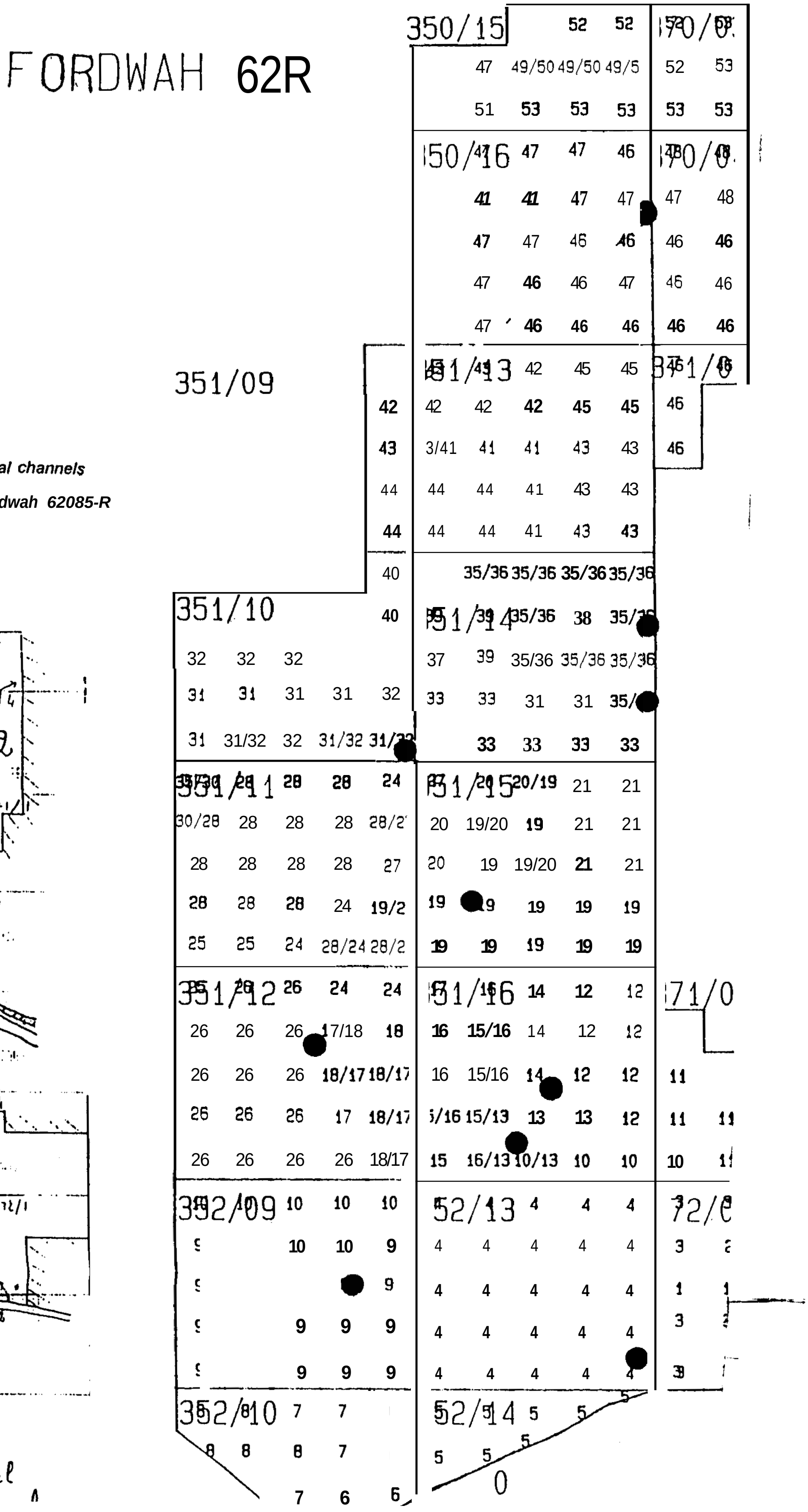
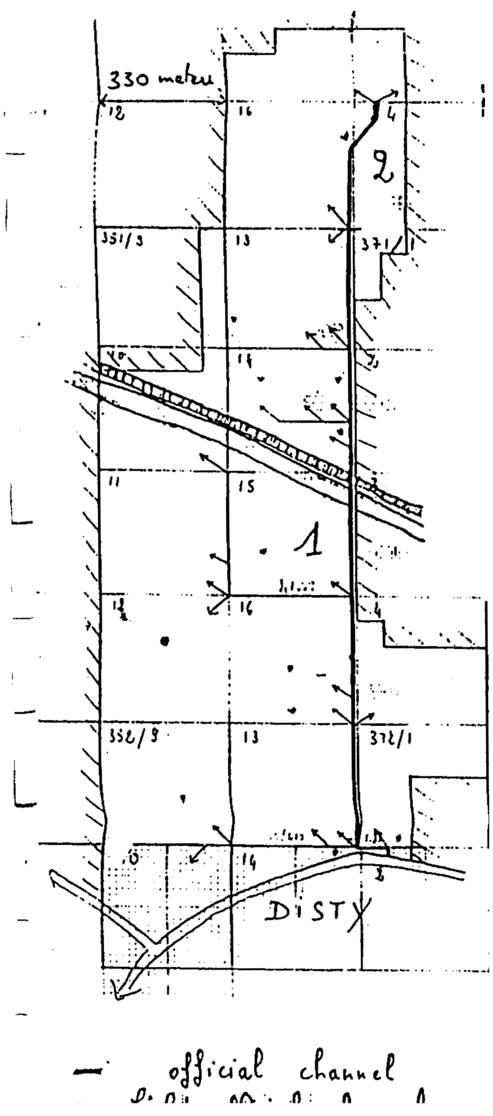


Figure 3.3: Location of the official channels and the official field channels in Fordwah 62085-R



CHAPTER 4

Development of a Watercourse-Based Hydraulic Model

As specified in the introductory section, the objective of this study is to assess the volume of canal water supplied lo the farmers by the canal water allocation system. Thus, there is a need for a model that predicts discharges and takes into account losses and others hydraulic features **of** the watercourse.

In the first part of this chapter, different models used in hydraulics of surface irrigation are presented; most of them have been developed for furrow irrigation or border irrigation. **As** the hydraulic data ***of*** a watercourse can be highly localized, and such values ***as*** the Strickler are difficult to measure, the data collection process is the major constraint on the choice of the model. Hence these models are classified with respect to the data needed; this classification is compared to the actual data collection which can be completed during a field survey of a reasonable duration. From this comparison, a type of model is chosen: the volume-balance model.

This choice still needs to be confirmed: this is done in a study of the salient hydraulic features of the water allocation system. This study also enables the development of the different parts of the model.

A first test of the validity of the model is then made: it is compared with a kinematic-wave model which was developed for the watercourse, but was not calibraled.

HYDRAULIC MODELS **IN** SURFACE IRRIGATION

The Equations **of** Hydrodynamics

Basically, two equations describe the flow of water over a soil surface; these well-known equations are named as the Saint-Venant equations.

The first equation [Equation (1)] is the mass conservation equation.

$$(1) \qquad \frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} + I_x = 0$$

The second equation [Equation (2)] is the equation of motion.

$$\text{(2)} \qquad \frac{1}{g}\frac{\partial V}{\partial t} + \frac{V}{g}\frac{\partial V}{\partial x} + \frac{\partial y}{\partial x} = S_0 - S_f + \frac{I_x V}{2gA}$$

x is the distance; t is the time; Q is the flow rate; A is the cross-section area of flow; I_x is the volume rate of infiltration per unit length of channel; g is the ratio of weight to mass; $V=Q/A$ is the average velocity in the flow cross section: y is the flow depth; S_0 is the channel bottom slope; and S_f is the channel friction slope.

Equation (1) is in fact the combination of two equations describing:

(i) the change of stored volume within a small slice of water during a period dt (Equation (3)]:

$$\text{(3)} \qquad dV = -\left(\frac{\partial Q}{\partial x} + I\right) dx\, dt$$

and (ii) the change in storage during the same period [Equation (4)]:

$$\text{(4)} \qquad dV = \frac{\partial A}{at} dx\, dt$$

In the equation of motion, the depth gradient (dyldx) represents the unbalanced hydrostatic pressure forces on the slice of water, the bottom slope (S_0) is the component in the direction of flow of the gravitational force on the same element, and S_f is the friction forces on this slice. The remaining terms represent the inertial reactions to those terms: local acceleration dv/dt, convective acceleration VdV/dx and the last term ($I_xV/2gA$) the net acceleration stemming from the removal of zero-velocity components of the surface-stream at the bed by infiltration.

The major assumptions made at this stage are:

(i) The channel angle is small enough *so* that the sinus may be approximated by the angle

(ii) The pressure distribution normal to the channel bed is hydrostatic.

(iii) The unsteady shear forces are equal to steady shear forces

(iv) Velocity is uniform in a cross section.

The flow described with these assumptions is known as the gradually varied flow. Discontinuity caused by cross-structures or by the hydraulic jump can not be described by these assumptions.

The friction slope S_f is given by the Manning-Strickler equation (5)

(5) $$S_f = \frac{Q^2 n^2}{A^2 R_H^{4/3}}$$

n is Manning's number; in the following, we will use the Strickler K, K=1/n; A is cross-area, and R_H is hydraulic radius.

In order to represent the infiltration phenomenon, which is the main issue in furrow or border irrigation, an equation which represents infiltration is also required; several laws are available in the literature, For infiltration in the case of furrow irrigation. the most common infiltration law is the Kosliakov's modified law as described in the equation 6 below.

(6) $$I = K * t^a + Ct;$$

For the specification and the calibration of this law, three parameters K, a, C need to be calibrated; such a complex law represents transient infiltration and steady infiltration.

Classification of Models

All mathematical models of the surface irrigation process use the volume-balance or continuity equation (1), and differ from one another primarily in the choice of the motion equation.

After a first look at the models that have been developed and the main assumptions behind these models, we will compare them in respect of data needed.

The Hydrodynamic Model

This model solves the Saint-Venant equations in full.

Different methods have been used for this purpose : Katopodes and Slrelkoff (1977) have presented the method of characteristics to compute a solution, while the Eulerian approach is based on a deformable control volume (Souza 1981 quoted in Walker 1987).

This exhaustive model is computational expensive and delicate. But its accuracy under a wide range of slope, roughness, inflow rate and infiltration characteristics makes it possible to regard it as a standard with which the more simplified models can be compared.

The main assumptions of this model are the same ones as Saint-Venanl equations.

The Zero-Inertia Model

In lhe case of the Zero-Inertia Model, the motion equation [Equation (2)] is approximated by:

(6) $$\frac{dy}{dx} = S_0 - S_f$$

The Zero-lnerlia Model neglects the inertial and the acceleration lerms of the momentum equation.

Basset et al (1980) show that this approximation is valid in the case of a Froude number smaller than 0 2 (which happens most of the time in furrow irrigation with small discharges).

In the case of the reaches in a command area, with:

$S=0.3\ m^2$ (wetted surface);
$v=0.2\ m/s$ (velocity of water);
$L=1\ m$ (width at the surface);
we can calculate a Froude number equal to 0.1.

Thus the Zero-Inertia Model would give accurate results.

Souza (quoted in Walker et al. 1987) compares the advance phase of this model with the full hydrodynamic model; he shows that the difference between these two models are negligible. However, the computation time of the Zero-Inertia Model is much shorter.

Walker el al. (1987)have verified the accuracy of the model, which matches with field experiments,

The Kinematic-Wave Model

If the bottom slope is steep enough, the depth gradient of the equation is much smaller than either of right hand terms

The motion equation becomes:

$$(7) \qquad S_0 = S_f$$

The relation between depth and discharge is given by the Manning equation based on normal depth. The result is the so-called Normal-Depth Model, the most common of the Kinematic-Wave Models.

As for the computation of the hydraulic flow, no downstream condition is needed as the normal-depth assumption is used everywhere. This type of model cannot handle downstream boundary conditions which affect the flow upstream.

The advance front of water in the channel is nearly vertical, as no depth gradient is taken into account The discharge at the farmer's offtake will therefore become quickly the stabilized discharge.

The dimensionless equation indicates that the factor P=So*X/Yo (So is the bottom slope, Yo the water depth, and X the advance length) is relevant for the accuracy of this model.

Tinney and Basset (1961)show that for P>2, errors in the computation of the advance phase is less than 10 percent. In the watercourse case where X is most of the time 300 meters, and Yo can be averaged at 0.3 meters, the slope should be higher than 2/1000 for this assumption to be valid.

However, the analysis of the slopes in the watercourse of the studied area shows that this is not the case (slope is between 1/10000 and 1/1000). Moreover, some fields are high with respect to the normal depth in the watercourse reach. Thus a downstream boundary condition is needed in order to represent the change in the flow surface upstream when water enters these field.

The Volume-Balance Model

The Volume-Balance model is the most simplified one. The momentum equation is no longer taken into account.

The mass conservation is written during the advance phase as:

$$(9) \qquad Q_0 t = V_y(t) + V_z(t)$$

Equation (9) shows that $Q_0 t$ (assuming discharge at the inlet is constant), the volume delivered at the inlet is equal to the volume on the soil surface at time t $[V_y(t)]$ plus the volume infiltrated $[V_z(t)]$.

In order to assess the volume of water, it is assumed that the volume on the soil surface can be expressed in terms of the upstream flow area A_0 the advance of (he stream X_a and a constant shape factor r_y. The average cross-section area A is considered constant, equal to $A_0 * r_y$.

In the more sophisticated model described above, the shape of the surface profile comes out as part of the solution. In this assumed average surface depth approach, the shape factor is set prior to the computations, and replaces the motion equation.

The new equation is then:

$$(10) \qquad Q_0 t = A_0 * r_y * X_a + V_z(t)$$

As the infiltration depth is a known function of submersion time alone, $V_z(t)$ is a known function of the advance function and the equation (10) can be solved.

In order to simplify the computation, a shape infiltration factor r_z can also be specified. Then only the infiltration depth at the intake is required for the model.

The same restrictions as for the Kinematic-Wave Model apply for the application of the Volume-Balance Model, In addition. some inaccuracy is added in the assumption that the shape factor is constant. A comparison survey (Holzapfel et al. 1984) shows that the volume-balance approach would yield a good prediction of the advance phase of the waler when the surface shape factor, and the infillration shape factor are well established.

Data Requirements

Table 4.1 sums up the characteristics of the models and displays a first classifications of these models.

The three models using an equation of motion (Hydrodynamic, ***Zero-Inertia,*** Kinematic-Wave) all need the same data. Assumptions made in Zero-Inertia and Kinematic-Wave models do not lead to less input parameters. they only reduce computation costs.

Data needed by the Volume-Balance Model highly depends on the way this computation method is used.

Table 4.1 Data requirements of hydraulic models used in furrow irrigation.

Hydrodynamic Model Zero-Inertia Model Kinematic-Wave Model	* Inflow discharge * Cross-section shape * Slope * Strickler * Soil infiltration law * Time of irrigation cut-off
Volume-Balance Model (Advance phase)	* Inflow discharge * Surface shape factor * Soil infiltration law * Inlet wet-cross section (through Strickler or measurement)
Volume-Balance Model (Drainage phase)	* Slope * Strickler at the inlet and the outlet (for the wet-cross section) * Soil infiltration law

Conclusion

In order to have an idea of the type of model which is needed, the data collection in the fields, with its constraints and its inaccuracies is presented first. Reports on the parameters (Table 4.1) are listed below

- The Strickler. In small reaches, not more then one meter large and with a water depth changing quickly with the shape of the bottom (from 20 cm to 40 cm), it is quite impossible to assess it.

- The infiltration law. Measurements of seepage are very difficult; a flume is not so accurate, and the law cannot be calibrated on advance phase (Renault 1989) as steady state will last much longer then the advance phase

- The cross-section. field survey yields a high variability in cross section, especially when the reach is in an open-zone and cattle will bath or cross it. In general, accuracy is low on such measurements, as bank-slope or bed width changes from one place to another.

- The slope: a topographic survey was made for the bottom of the reach.

Conclusion : First Assumptions Stemming from the Data Collection Process

As the Strickler is difficult lo calibrale, a simple volume-balance model is better adapted.

An infiltration law taking into account unsteady seepage loss is difficult to calibrate for the watercourse reaches.

Hence, a simple Volume-Balance Model is advocated on the basis of the data collected in a watercourse.

- In order to confirm this assumption, and *to* develop the Volume-Balance Model, a study of the hydraulic features of *the* canal water allocation system is *further* needed.

HYDRAULIC FEATURES OF THE CANAL WATER ALLOCATION SYSTEM

Comparison of a Watercourse Reach with a Furrow

In order to understand better the differences between furrow irrigation and the water allocation through the watercourse command area, a Froude similitude is made.

The scale ratio stems from the comparison of a furrow bed width with a watercourse bed widlh. As in open channel flow, the Froude number (v/sqrt(gh)) is dominant; other scale ratios (for length, discharge, Strickler) are based on the constancy of Froude number. They stem from the similitude.

The results are shown in Table 4.2; the last row gives the ratio of advance time on total runoff time. Different watercourse reaches are presented in Annex 3.

Table 4.2. Froude similitude: Comparison between a furrow and a watercourse.[3]

	Furrow	Similitude	Watercourse
Bed width	10 cm	60 cm (L=6)	60 cm
Lenglh	160 m	960 m (*L)	330 m
Discharge	.8 l/s	71 l/s (*$L^{2.5}$)	70 l/s
Strickler	25	21 (*$L^{-1/6}$)	
Slope	.0006	0.0006	0.0001 to 0.001
Adv/total	1/3	1/3	1/25

[3] The example of furrow infiltration is taken from Maillol (1992).

The similitude yields a common discharge in the watercourse. Differences are in the length of the reach; a 1 kilometer-long reach for an advance phase is never fulfilled in the usual water allocation schedule.

Another striking feature is the ratio of advance time to total time. During a water supply schedule, the advance time will usually be 15 lo 20 minutes and the total time the reach is used can range from one hour to one week, depending on the location of the reach (usually more than **6** hours of use).

Table A1.1 in Annex 1 assesses the relative importance of advance and drainage phases during a warabandi, in the four watercourses located on Fordwah Distributary. Less then 10 percent of the farmers are concerned (it is less on the point of view of area) by a relative important advance phase (more lhen 30% of total warabandi time). An accurate model on.the advance or drainage phase will be relevant only to assess accurately the volume received by such farmers.

The comparison between the different figures given in Table 4.2 highlights the theoretical difference between the aim of modelling furrow irrigalion and lhe present situation (water supply at the nakkha level).

In furrow irrigation, the main focus is on the infiltrated amount of water. We need therefore to know the exact location of water during the advance phase and the drainage phase related to the timing of these processes.

In the present study, the aim of the model is to know the volume of water that a farmer receives during his turn. The importance of the advance phase is therefore reduced, and losses in the canal need to be measured during the stationary phase, which can last from 6 hours to 1 week. Concerning the drainage phase, relevant information is the volume of water farmers will get during drainage. The timing of the process or lhe location of the recession front, for example, are not necessary outputs.

Analysis of Transient Phases

A Volume-Balance Model simplifies the various unsteady phenomena that can occur in the canal water allocation system. These phenomena are:

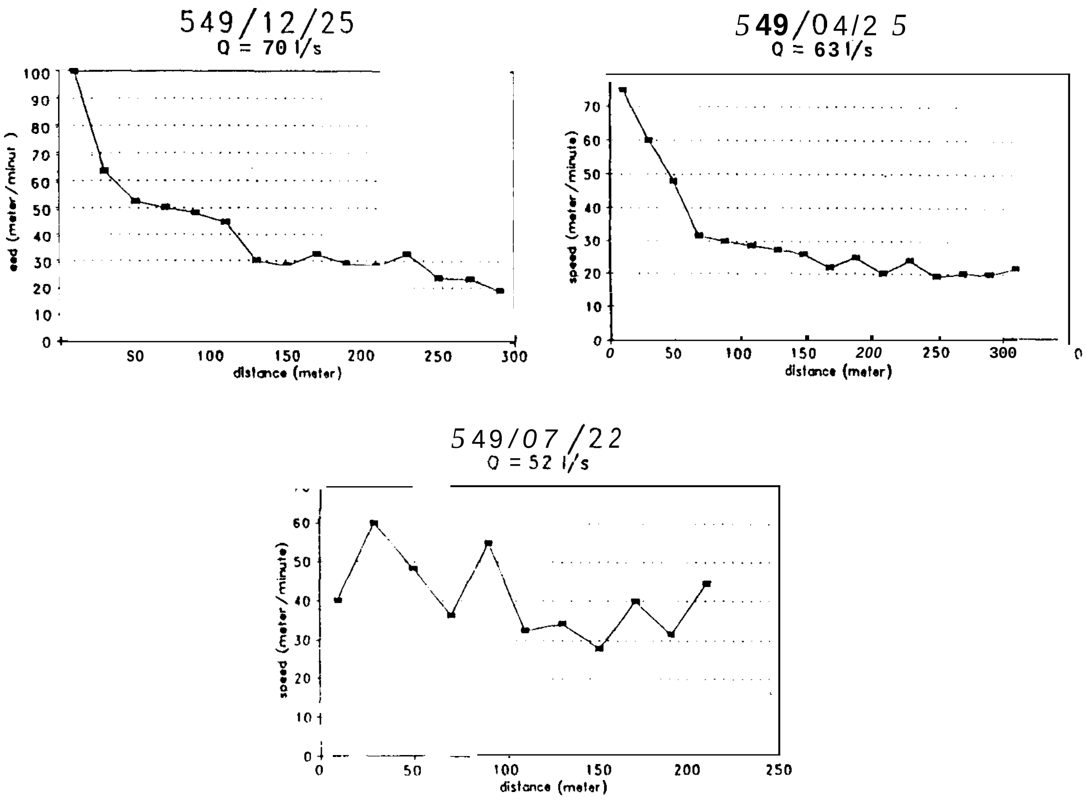
- Advance phase
- Drainage phase
- Lagtimes

Study of these phenomenas enables one to state whether such a model is reliable; moreover, this study enables lo determine how the model can deal with the computation of the advance and the drainage phase.

Advance Phase

The advance phase measurements made on FD14 are studied in the following section, using graphics of speed with respect to distance as Renault (1989) recommends. Out of eight advance phase measurements. three have been selected, as good examples of the phenomenas occurring in a watercourse.

Figure 4.1. Three Selecfed Advance Phase Measurements in FD14 (the title above each graph gives the location of the beginning of the advance phase and the discharge measured at the outlet).



The graphs show that the advance speed is high at the beginning, due to the storage and dam effect afler the opening of a nakkha. Then the advance speed is reduced and becomes quite stable, with some special points, due to highly localized features of the reaches.

The decrease of the advance speed afler a certain distance is due to the transient infiltration rate, which, for sandy soils as it is our case, is high.

In a reach like 549/07/22, the changes in the advance speed are hectic, owing to a cross section with highly localized features.

Comparison between different speeds for different reaches yield the same results, except for two reaches: farmers' interviews and a watercourse survey show that one reach is an area where there is level problems, slope being quasi-null, and the other reach is in an open area with a very wide and irregular cross section due to cattle crossing.

In order lo assess properly the advance phase in these reaches,a full hydrodynamic model (for the dam effect), and transient infiltration would be required. In some reaches, the variability of the cross section is *so* high that results yielded by such a model would not be reliable anyway.

Such a model seems to be rather sophisticated for a process which is important for 5 to 10 percent of farmers. Moreover, assessing the volume delivered to the farmers does not imply a complete study of the advance phase. Measuring the hydrogramme at the farm's nakkha, or simply working on the volume of water stored in a reach on a volume-balance basis could be enough to appraise the volume that is not delivered to the farmer because of advance phase.

Drainage Phase

The volume of canal water received during the drainage phase is difficult to assess, as farmers will not agree for flume measurements, since fluming slightly increases the level of water upstream. and thus decreases the volume supplied during the drainage phase.

Experiments show that it is quite impossible to follow the recession front owing to the watercourse irregularities Water will pond in certain places or flow quickly in others...

An attempt has been made to assess the volume by inserting a gauge in the watercourse bed at the nakkha of the farmer located at the end of the main branch of FD14, which receives an important drainage volume. The level of water was measured every 15 minutes, during three and a half hours of drainage time for a 2 km long reach. Assuming the Strickler coefficient, the discharge can be computed using the Manning's formula. Measurements are taken till the water does not flow anymore in the fields.

Such a method, however, has led to results highly dependent on the choice of the Strickler coefficient. For an assumed Strickler of 25, for example, the volume computed was more than the total volume that could be stored in the reach upstream!

Given these difficulties, measurements of drainage phase were not continued. Drainage phase in actual fields conditions is impossible to calibrate accurately.

Lagtime

The lagtime between the head and the tail-end of the watercourse depends on various parameters:

(1) The varialion of discharge at the outlet

(2) The initial discharge at the outlet, which yields the initial speed of flow

(3) The speed of the mass transfer in a reach.

(4) The distance from head to tail

Below are stated some salient values for the assessment of lagtimes for F046 (located at the first third of the Fordwah distributary):

* The quickest variation of discharge at the outlet (measured after gate closure at the head of the distributary) is 25 l/s (14 cm less in the distributary) in 3 hours.

* The average discharge at the outlet is 80 l/s.

* Speed of the mass transfer in the reaches: in a first approximation, the Saint-Venant equations are simplified to Hayami equation, which represents a transport-diffusion phenomenon (Schuurmans, 91) The diffusion celerity is roughly v = 1.5*V, where V is the speed of the flow
In FD46, v = 1,5*0 08/0.32 = 0.4 m/s

* The maximum distance from head to tail is 2 kilometer.

Given a step Function at the outlet, the lagtime until it reaches the farmer at the tail is:

$$\delta T = \frac{length}{mass\ speed} - \frac{2000}{0.4} = 5000\ seconds = 83\ minutes.$$

Given the quickest variation rate at the head, maximum difference of discharge between tail and head will be

$$\delta q = (variation\ rate) * (duration) = \frac{25}{(3*60)} * 83 = 12\ l/s.$$

In this watercourse, with 80 l/s at the head, the tail receives 65 l/s. With 68 l/s at the head, the tail receives 55 l/s. Hence the quickest variation of discharge at the head induces a difference of 15% between the real discharge and the discharge computed by the model, when the water is at the end of the watercourse.
Given the scarcity of such transient phases from full supply discharge to low supply discharge at the mogha (fig 6 1 and 6.2), the lagtimes are negligible.

Importance of a Downstream Boundary Condition

In some areas supplied by a given watercourse, Farmers' fields can be high with respect to the watercourse bed The level of water rises before water starts to flow into the field, increasing storage and seepage losses upstream and thus decreasing the water available to the crop in the field. The use of a Volume-Balance Model does not take into account a downstream boundary condition on the flow, and the field level is thus not taken into account.

The extent of high level with respect to normal flow is therefore studied as follows.

Table 4 3 was made after a survey of fields higher then the watercourse bed in the four sample watercourses along Fordwah Distributary.

the four sample watercourses

	% of the CCA
FD14	5
FD46	6
FD62	13
FD130	4

The problem of high elevation of fields affects small areas scattered in a watercourse, each one less than 4 hectares (10 acres) in area. Its extent is not large enough so that all the area supplied by a nakkha could be affected.

Farmers can have fields affected by level problems among other fields. In order to take this effecl into account, it is necessary to (i) idenlify fields with level problem, (ii) lo monitor the water allocation to specific fields and (iii) lo assess discharge at the field level. To calculate the average amount of water received by a farmer, the ratio of fields with level problem lo the total farm area may be used. However, it is seen as a rather inaccurale approach as farmers can be expected to grow crops with a lower water requirement in fields wilh level problems. Moreover, our concern is at the nakkha level. As this field level problem usually affects less than 10 percent of Ihe CCA. it is not taken into account in the development of the Hydraulic Model.

Comparison of a Volume-Balance Model with a Kinematic-Wave Model

A Volume-Balance Model seems well adapted to the characteristics of the watercourse and lo the objective of assessing volumes of water. Such a model can be compared to a more sophisticated model, a Kinematic-Wave Model developed by Barreteau (1993) in the same research locale, which was not calibrated. Results of this comparison will provide information on the limitation of a volume-balance approach.,

The two models were compared for three different outlet discharges, 20 l/s, 40 l/s and 5011s on FD14

The same infiltration laws were used. The data needed for the Volume-Balance Model such as advance speed were yielded by Ihe Kinematic-Wave Model.

The comparison between Ihe outputs of the Iwo models shows that the criteria for a significant difference between these model at these discharge is the imporlance of the advance or Ihe drainage phase.

If the ratio, advance lime or drainage time to total warabandi turn is more than 30 percent, a difference in calculated volumes of more than 10 percent is obtained.

Table AI.I in Annex 1 shows that for about 10 percent of the farmers, there will be a significanl difference between these two models. Out of these 10 percent, 5 percent improve allocation through local deals such as rotation turns; the effecl of the advance or drainage phase occurs only once in three to five weeks, and is thus negligible. It can therefore be concluded that a significant difference between the two models will occur for 5 percent of the farmers only.

As the Kinematic-Wave Model was not calibrated, we do not know which model is closer to the real world. But advance speed given by the Kinematic-Wave Model, 12 meters per minute, is far less then the advance speed measured during field activities of the present study.

In fact, a main assumption of the Kinematic-Wave Model related to the slope value is not always fulfilled on the watercourse. Moreover it has been seen that the dam-effect at the opening of a nakkha increases the advance speed and this makes this aspect rather complex,

The comparison confirms some of the conclusions of the section, "Hydraulic Fealures of the Canal Water Allocation System" in Chapter 4; when the advance phase or the drainage phase is small with respect to the total water turn, it can be roughly assessed, and a sophisticated model is not needed.

As little difference exists between the ***output of the*** two models, ***the*** simplest ***model*** can be used ***for*** estimating volumes of water received by farmers.

DEVELOPMENT OF AN HYDRAULIC MODEL

Preliminary

In the first section of Chapter 4, it was shown that given the low slope of the Indus Basin, the Zero-Inertia Model would be required for the modelling of water flows in the watercourse. But data collection and calibration of such a model would be a heavy task, time consuming and based on several not very reliable measurements.

The aim of this study and the specific aspects stressed in the second section of Chapter 4 show that the need of a sophisticated model is not obvious.

- The advance phase in a reach is somehow very complex, and representing it accurately does not improve information on the volume supplied to the farmers.
- The drainage phase is hectic, and difficult to calibrate.
- The length of watercourse and speed of variation of discharge at the watercourse outlet are not *so* large, and the processes of mass propagation in a watercourse can be neglected.
- Taking level of the fields into account would be complex and not adapted to the level (nakkha level) of the present study.

The Volume Calculation in the Model

Following the comments given above under "Preliminary", we now focus our work on a model based on the following:

- *To* select *a* simple volume-balance approach.
- To assess the advance phase by measurements.
- To assume that the drainage volume of water supplied during the drainage phase is equal lo Ihe volume stored in the reach.
- Within a walercourse command area, classify reaches according to their characteristics collected through a walercourse survey and physical measurements.

Basic equations used for the computation of the volume supplied to each farmers:

(11) $$V = \int_{t_b}^{t_{ew}} Q_f(t)dt + V_d$$

where

(12) $$Q_f = Q_o - l(Q_o) * D;$$

(13) $$t_b = t_{bw} + \frac{u}{V(Q_r)};$$

(14) $$V_d = Q_f(t_{ew}) \frac{D_d}{V(Q_r(t_{ew}))};$$

I„ and t_{ew} are, respectively, the beginning and the end of the turn of the farmer **f**; **Q, is** the discharge at his nakkha. taking into accounl losses per meter, function of the discharge **I(Q.)**, and the total length of the reach upstream D; t_b takes into account the advance phase; V_d *is* the drainage volume received by Ihe farmer

Calculation of t_b involves Ihe lenglh of the reach which will be filled **(D.)**, divided by the speed **of** advance front V(Q,) (C?, is the discharge at the head of Ihis reach)

Calculation of V_d involves Ihe duration of drainage phase (calculated with the discharge at the head of this reach, at the time of the closure of the reach), multiplied by discharge at Ihe farm outlet; in fact, it is Ihe volume stored in the reach, minus seepage losses occurring during drainage phase.

This is simply an integration of the discharge at the farm nakkhas during the farmer's water turn. The discharge at the nakkha is computed as the discharge at the outlet minus losses (lagtimes are small). The farmer receives full discharge as soon as the advance phase proceeding at constant speed is finished.

After the end of his turn. he may receive the drainage volume from a reach during a time equal to the advance phase time in the same reach

The Data Requirements of the Model

The information required *by the* model *for* calculating the volume *of* water at *the* farm nakkha will *be*

The Warabandi in Function

The warabandi collected is one of the main inputs of the model. A warabandi for the whole season. or a weekly collected warabandi can be an input into the model. Both types of nakkha (official and unofficial) are slated in this warabandi. So the model will take into account the official reaches and the farmers' official reaches.

Each square has at least one nakkha to provide water. In a watercourse like FD130. where two squares had no nakkha. the warabandi was improved through the addition of nakkhas; hence the maximum length of farmer's reach, not taken into account in the model, is 400 meters.

Each farmer is recorded by the nakkha from which he irrigates his field (several farmers can use the same nakkha). The volume of water computed for each farmer is based on the discharge at the farmer's nakkha. and on the computed advance and drainage phase until this nakkha.

Discharge at the Watercourse Outlet

A main input is the discharge at Ihe watercourse outlet. The level of water with respect to a White Mark, upstream and downstream of the watercourse outlet is collected every day by IIMI field staff. During the data collection phase, these outlets were calibrated (Annex 2), to transform the water level into corresponding discharge. As only one discharge a day is available at the outlet, the model linearly extrapolates discharge between the discharge of two following days.

Watercourse Features

Collection of data of the hydraulic features of a watercourse *is* further explained in Chapter *5.*

To summarize, in a single watercourse, the different reaches will be classified into classes homogenous from the hydraulic point of view (same advance speed and similar losses).

The Programming of the Model

This model has been developed in Turbo-Pascal object oriented 7.0; apart from a better organization of the program, the main asset of this language is to enable further improvements. As the number of farmers and the number of reaches highly depend on the watercourse characteristics. pointers allowing memory to be dynamic were extensively used

After the end of his turn, he may receive the drainage volume from a reach during a time equal to the advance phase time in the same reach.

The Data Requirements of the Model

The information required by the model for calculating the volume of water at the farm nakkha will be:

The Warabandi in Function

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The Description ***of the*** Geometry

The watercourse is a tree, which is oriented from upstream to downstream. We define:

- **A** node *is* a point where the watercourse splits into several branches going downstream; two branches cannot join.

- A branch is a watercourse between two following nodes.

For the user, the basic unit is a reach: it is a branch which is homogenous with respect to hydraulic features (seepage, advance time, etc.). The user needs to number the different reaches and enter them with their characteristics:

- Length

- Number of the reach upstream

- Hydraulic class

On these reaches, the nakkha where the farmer receives water, **is** located. The user needs to enter:

- The number of the reach.

- The distance of the nakkha from the upstream end.

A first step of the program is **to** insert the nakkhas on the previous geometry. **A** new node is inserted at the nakkha location. It means that each reach is split into secondary reaches, between two following new nodes. These nodes *are* the only points ***of*** the system where the discharge needs ***to*** be calculated.

A reach is linked to:

- **A** node upstream.

- A node downstream.

- **A** node father (the downstream node **of** the reach upstream, or the outlet for a reach starting **at** the outlet).

A node is linked to

- **A** reach upstream (in the case of the outlet, first node of the system, the reach upstream is pointer nil)

A nakkha is linked to:

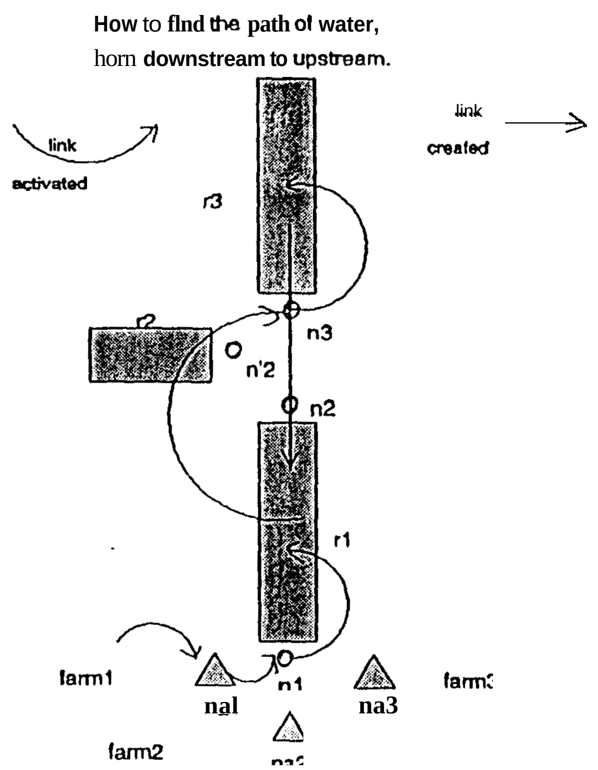
- A node located at the same place.

A farmer is linked to:

- A nakkha where he receives water.

The relations between these different objects are further explained **in** Figure 4.2, where the a path of water is identified through the localization of a nakkha. Different links are activated, going upstream. Reaches are linked going downstream for the further computation of discharge.

Figure 4.2. Identification of the path of water.



The Description of the Warabandi

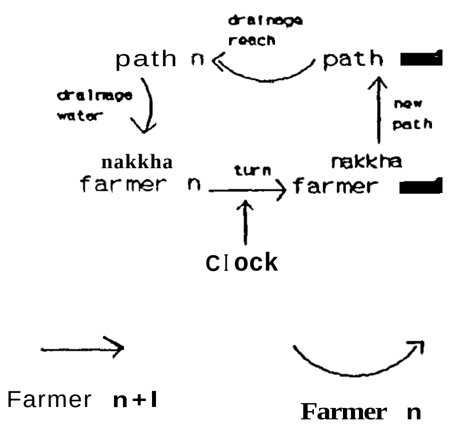
The farmer characteristics, stemming from the warabandi file are:

- The nakkha where he receives water (nakkha-in).
- The nakkha where the following farmer takes the water (nakkha-out).
- The duration of his turn.
- The following farmer in the roster of turn.

The model selects the nakkha closest to the farmers' field (nakkha-in if there is a drainage phase, nakkha-out otherwise) for the hydraulic computation.

At the end of a turn, the model shifts to the next farmer; the new nakkha location changes the path of the water, identified in a first step. In a second step, comparison of this path with the previous path yields the place where drainage phase starts, if there is one; the drainage volume is added to the previous farmer, and then the hydrograrn for the new farmer is computed (Figure 4.3).

Figure 4.3. Main ***routine of the Model.***



The Hydraulic Computation

Every node and every nakkha is linked to an object called **hydro.** This object stores the variations of discharge with respect to time.

As input data, the model needs the hydrogram for the outlet. After the identification of the path of water, this path is memorized **by** a link between two following reaches, starting upstream (Figure 4.2).

The hydrogram at the upstream node is known; a function attached to the object reach transform this hydrogram for the node downstream, by computing the advance phase or the discharge every minute. The link between reaches allows the transfer of the downstream hydrogram to the upstream node; if the reach is not linked, that means that the nakkha is at the same location as the node: the hydrogram is transferred to the farmer's nakkha.

CHAPTER 5

Data Collection and Calibration of the Model

Three main factors will interfere with the volume of water provided by the irrigation system:

- Seepage loss and leakage lhrough nakkhas or rat holes. This will affect all the farmers, mostly the ones located at the tail.
- Advance phase.
- Drainage phase.

The aim of this part is to set up a procedure to assess in the fields the different inputs for the model. These include:

- Measurements.
- Data analysis.
- Choice of a representation for the model.
- Choice of a measurement methodology.

THE ASSESSMENTOF LOSSES

Two main experiments can be run in the watercourses in order to assess losses.

- The ponding method yields a seepage rate as a function of the height of water in a 20 meter reach.
- The fluming method yields a loss rate along the watercourse between two points, with respect to discharge.

In a first step, flurning and ponding will be compared, on the basis of methodology and of results. Then the data collection for losses in a command area shall be presented.

The Ponding Method

Methodology

Measuring reach losses by the ponding method involves filling a section of the reach closed at both ends and determining the decrease in the volume of water in the section over time. This volume decrease is determined by measuring the area of the surface of the ponded water (top **width** times the section length) and the rate of recession of the water surface.

The test sections were scattered along the watercourse on different branches in order **to** be representative of existing watercourse reaches. In each watercourse, three ponding method have been carried out.

The section length is 20 meters long.

Two gauges are inserted in the watercourse bed, **5** meters from each end **of** the test section

The height of water at normal flow (the usual discharge in this part of the watercourse) can be seen on the watercourse bank; the test section is filled to ***6* cm** above this level. The measurement starts afler 3 cm (one inch) water level recession in order to thoroughly wet the banks and assess the steady infiltration rate; **if** the banks were dry before the test, the test section is filled **to 9 cm** above the normal flow level, and measurements start after 6 cm water level recession.

Four evenly spaced water surface, width measurements are made during the water level recession; the average of the four readings along with the gauge reading at the time of the measurement are used as a measure of top width at various depths.

Gauge reading are noted every 10 minutes in the beginning and every **6** mm (0.02 feet); one can expect an accuracy of 0.01 feet on the level measured.

Data collected besides these measurements:

- Whether the section is straight or not.
- Maintenance of the reach : well maintained, maintained, maintained with vegetation on the top of the banks, poorly maintained (vegetation in the bed).
- Soil texture.
- Are any insect or rat holes visible
- Whether the section was wet or not.
- Water used for the test (canal or tubewell water).

Data Analysis

Ponding was carried out in the eight sampled watercourses, four on the Fordwah Distributary and four op the Azim Distributary.

The analysis of the results is presented below.

1) Seepage rate as a function of water level.

Following the study of Trout (1981), data analysis on each ponding test were run in order to yield **a** seepage rate in the following form [Equation (1)]:

$$(1) \qquad Q_S = Q_{S0}\exp^{(c(y-y_0))}$$

Where Q_s is the seepage rate (in liter per second per 100 meters) and **y** the level of water (in centimeter), Q_{s0}, y_0 is the seepage rate and the level of water at the "usual" discharge, **c** is the exponential coefficient of variation.

On 22 ponding tests, a regression was run in order to find c;

- For 11 tests, R squared is higher than 0.8.
- For 7 tests, R squared is below 0.8 but higher than 0.5.
- R squared is lower than 0.5 for **4** tests.

[The results of these regressions are not as good as in the Trout **(1981)** study.]

- The average seepage rate at normal depth (Q_{s0}) is 0.67 with a high variability from 0.07 to **1.71;**
- The average value of c is **0.18**, with a high variability from 0 to **0.48.**

This relative high value of **c** may be explained by the fact that rodents and other insects build galleries in the watercourse banks (Trout **1981).** With an increase in the level of water, water will flow in these galleries, the seepage rate therefore increasing highly.

Unlike furrow irrigation, where infiltration can be estimated in first approximation as proportional to wetted perimeter, this approximation is not valid in the watercourse.

2) Seepage rate as a function of the reach features.

Values of Q_{s0} were analyzed in order to find which of the following parameter was relevant and the most significant in influencing the level of losses for a given reach:

- Height of water level at normal flow.
- Top width at normal flow.
- Maintenance of the reach (values between 0 and 1 were given to the different degrees of maintenance).

- Soil type : three columns, clay, sand, and loam were used to describe the soil texture.

  * Sandy-loam soil: 0.5 in the sand and the loam column.

  * Clay soil: 1 in the clay column.

  As these three columns of soil types are dependent (sum is always 1), each set of two columns was tested; the one with the best results was chosen.

- Time ratio on a week, during which water is running in the reach.

The regression analysis showed that three parameters were significant, with R squared equal to 0.63 [Equation (2)].

$$Q_{SO}=0.04H+0.07Q-0.33C \quad (2)$$

where H is the height in feet, W is width in feet and C is clay.

Height and width at normal flow can be related to the usual discharge in the watercourse; therefore, this correlation shows that seepage increases with discharge in the reach.

Measurements made in watercourses of Azim Distributary where clay is more common yielded low seepage rates. This will account for the importance of clay in the final equation. Other types of soil, loam or sand, common in Fordwah watercourses, were not relevant.

Use of the Results as Input for the Model

Two major problems occur when the seepage rates yielded by the ponding method are used as input for the model.

1) Spatial Variability

In FD14 main branch (2 kilometers long), starting from the end of the lined head reach to the tail, three ponding tests have been made. They yielded similar seepage rates (0.3 l/s) at the normal flow height of 30 cm, but the coefficient c is highly variable:

* c = - 0.05, 100 meters from the head.
* c = 0.17, 400 meters from the head.
* c = 0.41, 1000 meters from the head.

The first test seems not valid, as seepage increases when water level decreases. Results of the two ponding tests further down the main branch are very different with respect to c. The extrapolation of these two tests on the whole reach is rather problematic: on which part of the watercourse is the second test valid?

2) Calculation of Seepage Loss Rate: Sensitivity to the Data Collected

The coefficient c was found to be very high, so seepage increases quickly with water level. As the calculation of water level as a function of discharge involves Manning equation [III-A-1 (5)], seepage rate is sensitive to the cross section (cross slope and bed width), the slope and the Manning.

The graphs presented below show the sensitivity of seepage losses to the Strickler and the slope on one watercourse reach, the main branch of FD46 (Slope=0.0005; bed width=0.9 m; cross slope=1/2; Strickler=25).

For high discharge, ± 10%, variation of the Strickler value will yield more than 50 percent difference on the loss rate; ± 20% variation of the slope yields more than 60 percent difference on the loss rate.

As the accuracy on the Strickler or the bed-slope is not more than that, the accuracy on the losses for high discharge is quite enormous !

Figure 5.1. Variability of the seepage rate with the Strickler coefficient.

$$seepage\ rate = 0.44\,e^{0.14(h-30.1)}$$

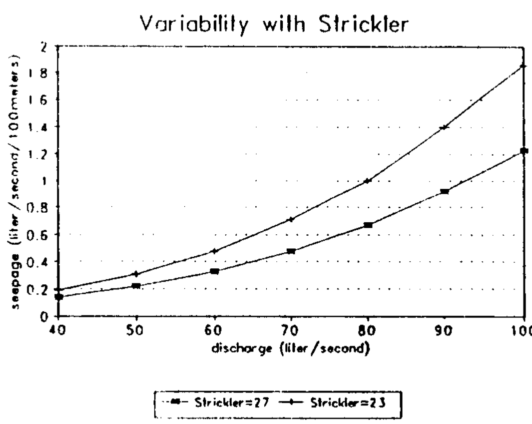
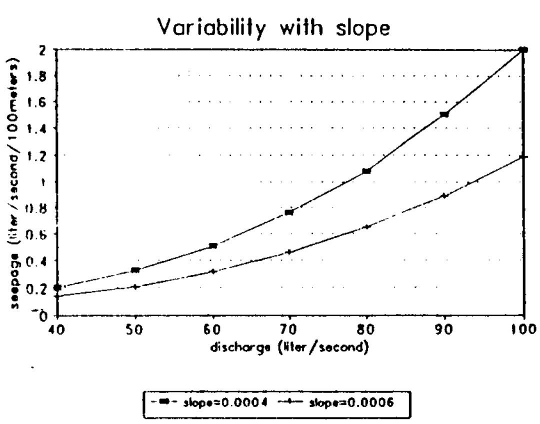


Figure 5.2. Variability of the seepage with slope.



The Fluming Method

When the distributary supplies water, discharge is measured on the official watercourse with a cut-throat flume, or a current meter if the reach is lined. This method will calculate losses as a difference of two discharges.

Methodology

The following sums up an article of Skogerboe (1973).

The cut-throat flume can be used in both free-flow and submerged-flow conditions. However, in order to have a good accuracy, one should aim at having free-flow conditions. Submerged flow is accurate enough only with a ratio between Hb and Ha of less than 90 percent.

In order to install a flume correctly, one should select a reach part where water is not too high so it will not get over the banks, and where velocity is high enough, in order to manage to get as close as possible to free-flow conditions.

If the first attempt was not successful to get a valid submerged flow, the bed of the watercourse should be filled with earth in order to increase the flume height.

A level is used to have good flow conditions and good gage reading; leakage through the sides or beneath the flume should also be checked.

Readings should be taken until they stabilize, as the flume induces change of the level of the water upstream and thus change the storage of the water. Before those new stable conditions are fulfilled, enough time should elapse, and readings will get stable.

On the whole, it can take from three fourths of an hour to one and a half hours to have good fluming results.

Data Analysis

Fluming was carried out on the four watercourses of the Fordwah Distributary; in order to work out a calibration method on flumes, the first two watercourses, Fordwah-14 and Fordwah-46, had fluming done thoroughly, and for the other two, the method was applied.

1) Data Collected in Fordwah 14

A lot of fluming have been carried out on the main branch of this watercourse. This branch is homogenous with respect to cross section, slope and maintenance. We will therefore assume that it is also homogenous with respect to losses.

Discharge at the outlet is measured with a current meter, because the reach at the head is lined.

Discharge at other points of the reach are measured with a flume; for the calculation of losses, we will assume that the line reach has no losses.

Some of the results collected were strange:

- One set of result yields greater losses for lower discharge on the first reach.

- On two sets of measurements, discharge downstream was higher than the 600 meters upstream discharge.

As the former reports on conveyance losses stated (Ali et al. 1978), the losses mainly depend on the distance from the outlet (D) and the discharge at the head (Q). A regression analysis of the loss rate was therefore undertaken, as a function of two independent variables Q and D. Results are displayed in Table 5.1.

For the whole reach, R square is very low, and the losses depend mainly on the distance from the outlet.

Two different cases stem from the data analysis made.

For the first 700 meters seepage has high value, and the variability of the data is high; these measurements were carried out with the same discharge at the head. Nothing can be concluded for seepage loss values in this first section. We can only try to figure out the reason for such a high variability:

Table 5.1. Regression analysis of loss rate in FD14.

Independent variables		Q, D	Q	D
The whole reach:	R squared	0.28	0.0025	0.26
	Q coeff	- 0.016	- 0.009	
	D coeff	- 0.00085		- 0.00083
Last 1,000 meters:	R squared	0.74	0.70	0.37
	Q coeff	0.035	0.039	
	D coeff	- 0.00017		- 0.00038

- As stated by Ali et al. (1978), the flume effect (increase of the water level upstream of the flume which induces an increase in the seepage rate), tends to increase measured loss rates over those which normally occur by a larger percentage in shorter sections than in longer sections. It explains the correlation (easy to see on the graph) of losses with the length of the reach.

- The number of pacca (concrete) nakkhas, most of them leaking from time to time. Some of them are located on the line portion which was never taken into account for the calculations.

- Accuracy on fluming is plus or minus 5 percent (see Ali et al. 1978); on two flumes in order to measure losses, it is plus or minus 8 percent.

  So accuracy is decreasing when losses are decreasing, i.e., when the reach is shorter.

  Example : Q1=60 l/s (+/- 5%) : 57 to 63 l/s
  Q2=50 l/s (+/- 5%): 47.5 to 52.5 l/s;

  Loss in the watercourse is 16.6 percent of the initial discharge +/- 8% because of the inaccuracy of fluming: seepage calculation can yield either 4.5 or 15.5 l/s losses!

- High variability was shown by the ponding methods on this main branch.

Owing to leakage at nakkhas, which is difficult to assess, and the problem of flume effect, improvement in the loss rate measurement is unlikely with the fluming method. Further measurements were not carried out on this section.

The whole reach: Measurements on this reach after at least a 1,000 meter distance are easier to comment on. R square value is higher, and losses depend mainly on discharge, which is consistent with the former studies, most probably, because the length of the reach balances the variability due to nakkha leakage and the lack of accuracy. The assumption of the flume effect playing a large role for the first 700 meters is also confirmed by the lower seepage for these big reaches. Moreover, those measurements were made with two different discharges at the outlet, which ease the calibration work (by easily assessing the importance of discharge in the seepage).

The low range of discharge (50 to 65 liters per second) could be surprising. It should be stressed that this branch has water only for half a week, and discharges at the outlet are not very different because, most of the time, the distributary is either at full supply or has no water (refer to outlet discharge throughout the season in Chapter 5).

2) Data Collected in Fordwah 46

Several fluming measurements have been made on the main branch of FD46, which is 3,000 meters long. In order to avoid too much inaccuracy on leakage assessment with the fluming method, measurements were carried out every 1,000 m on this 3,000 m long watercourse.

Parameters such as the widening of the watercourse at some places due to cattle crossing, the low level of maintenance of the last 1,000 meters were not taken into account for the location of the flume.

Table 5.2. Regression analysis of loss rate in FD46.

Independent variables		Q, D	Q	D
The whole reach	R squared	0.79	0.78	0.0057
	Q coeff	0.0078	0.0077	
	D coeff	-8.9E-6		-3.3E-5
The first 1,000 meters	R squared		0.74	
	Q coeff		0.0076	

The same regression analysis done in the case of FD14 was undertaken for this watercourse, as a function of two independent variables, the discharge Q, and the distance from the head D.

This regression mainly yields dependence on the discharge at the outlet (Q). The range of discharge was high (from 40 l/s to 100 l/s). The first regression was done on the first 1,000 meters; the result of the regression done on the whole reach shows that the last three points measured at 2,000 meter distance and at 3,000 meter distance are homogenous with the first reach. It is quite surprising to see that there is no difference between measurements taken for the first two sections and measurements taken for the last section, as the last section is badly maintained.

These measurements are plotted in Figure 5.4.

Calibration of a Seepage Law

As this data analysis process needs a large collection of data, an attempt was made to reduce this data analysis using an assumption tested in literature [Ali et al. (1978)].

The loss law is assumed to be a linear function, the same at every point of an homogenous reach [Equation (4)].

$$S = aQ + b \tag{4}$$

so

$$\frac{dQ}{dx} = -(aQ + b) \tag{4'}$$

where S is the loss rate per second and per hundred meter.

This equation yields for the discharge, after transformation through equation (4'), an exponential form for Q [Equation (5)].

$$Q = (Q_0 + \frac{b}{a})\, e^{-ax} - \frac{b}{a} \tag{5}$$

In a first approximation, a development of this formula (a is very small) is possible [Equation (6)].

$$S = aQ_0 - \frac{ba}{2}x + \frac{b}{a} \qquad (6)$$

which can be compared to the linear regression done before.
A major asset of this method is that it needs less mèasurements.

With two or three measurements, a linear law can be tested, following these simple rules:

- If the loss rate varies highly with respect to discharge (or to distance), a is rather big; the instantaneous loss rate at the measured outlet discharge is much bigger than the average loss rate measured on the reach (Fordwah 14).
- If the loss rate varies slowly with respect to distance, than a is small; the coefficients a and b are nearly similar to the a and b coefficients yielded by the linear seepage law (Fordwah 46).

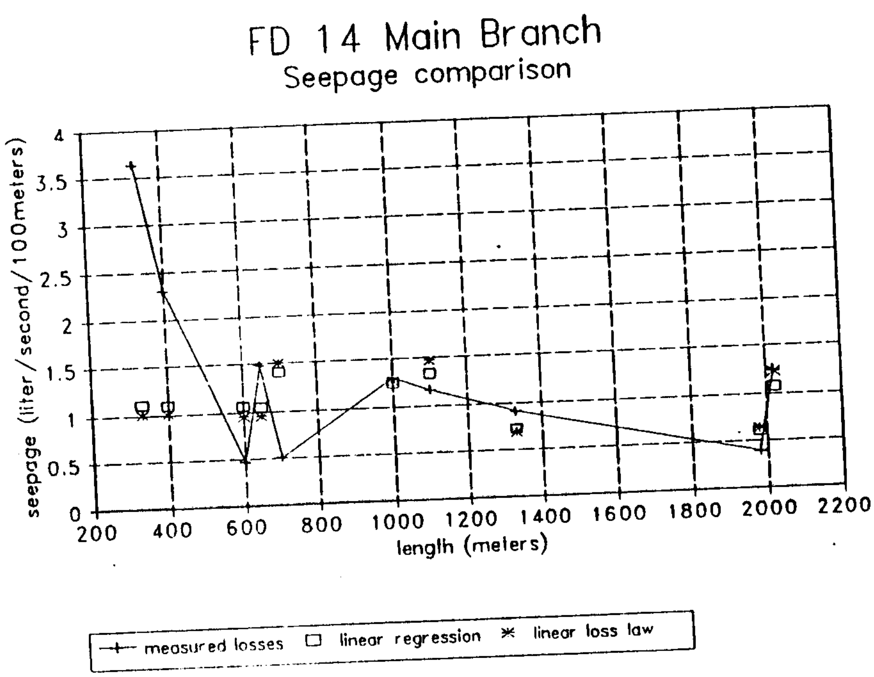
The coefficients of the loss linear law are tested by a trial and error method using the values calculated by the model.

In the following two graphs, the two methods, the linear regression and the linear instant seepage law are compared.

Losses for FD14 are plotted in Figure 5.3; they are calculated between the head and the fluming point. As the regression result for the whole reach is mainly dependent on the distance from the head (losses increase as the distance gets shorter), it is plotted as a function the distance from the beginning of the main branch (the end of the line reach).

Results for FD46 are shown in Figure 5.4: losses are plotted with respect to discharge at the outlet as the range of discharges was high. The flumings made 2 km and 3 km from the head are also plotted in the same graph (label 2,3).

Figure 5.3. Seepage measurements and calibration in FD14 with the fluming method.



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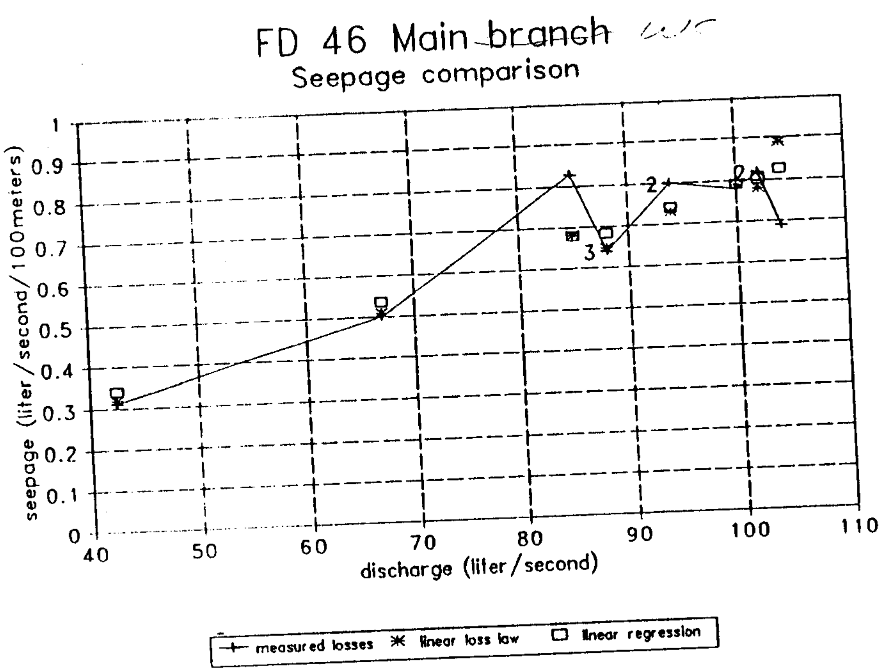


Conclusion

During the analysis of the results given by the two measurement methods, drawbacks and limitations of these method were highlighted. The present section shows the different drawbacks for each methods, trying to assess their importance: on this basis, the choice of a calibration method is made.

Weak Points of the Ponding Method

- Water level at normal flow:
  The way the normal water level is assessed is not accurate at all. Trout (1981) recommends the insertion of a gauge in the bed, while there is the usual discharge and leaving the gauge with the level noted on it. But ponding can often be made only two or three days after the insertion of the gauge, and the gauge can be removed by passers by.

- Leakage:
  Ponding method does not take into account leakage through nakkhas, or at particular places such as those where roads cross, or cattle bath, or banks leak. The aim of this study is to assess the volume of water farmers get. Therefore, all these losses should be taken into account, not only the seepage loss that the ponding method can evaluate.

- Hydraulic conditions:
  Trout et al. (1981) state that during a ponding test, sediments will deposit and decrease seepage losses. This method will underestimate seepage as compared to flow conditions.

- Extrapolation:
  Loss rate often needs to be measured on reaches longer then one kilometer. In order to extrapolate ponding measurements, it should be assumed that:
  * A branch is homogenous with respect to losses.
  * The ponding test is significant for the whole branch.

  This is rather difficult to prove.

- Sensitivity :
  The previous example in FD46 was carried out with c equal to 0.14; the average c on the eight watercourses is 0.18. Hence, the calculated seepage rate stemming from the ponding method is highly sensitive to the reach features. Moreover, as the ponding test yields losses with respect to height, the Manning, difficult to assess, is required to calculate the height as a function of discharge; and seepage rate was found to be very sensitive to Strickler's value.

Weak Points of the Fluming Method

- Increase in the upstream level:
  As the flume induces a rise of the level of the water upstream by a few centimeters, seepage losses will increase upstream. Measurements made during fluming yielded an increase in level of water upstream of five centimeters, which will change the surface curve up to 200 meters upstream (for a slope of 0.0005).

  If it is close to a nakkha, leakage can increase in an important way with only this little rise of the level. It should therefore be checked. But one cannot help the increase in seepage loss.

  An analysis of this effect in Ali et al.(1978) indicates that this flume-induced depth increase will often cause flume loss measurements to overestimate actual loss rates by 5 to 30 percent of the loss rate.

- Field constraint:
  For high-level land, or low-slope reaches, since water has a low velocity and can spill over the banks, farmers will not allow the carrying out of such an experiment. Some non-cooperative farmers will not let you do it even next to their land. So flume experiments are usually done in the official watercourse, 200 meters upstream of the irrigated field.

- Accuracy:
  Accuracy on a flume is ± 5 percent; so, in order to assess leakage and seepage properly, the two measurement points should be far enough from each other so that this accuracy problem is not too disturbing. Stemming from the data collection in FD 14, two flumes should be at least 1,000 meters apart (for reaches losing a lot of water, this distance can be reduced). And, it is assumed that on the whole length, the reach is homogenous with respect to losses.

- Losses as a function of discharge:
  Two flume measurements provide a loss rate for a given discharge; measurements at different discharge should be carried out in order to calibrate the loss law as a function of discharge. But the distributary is running either at full supply or with no water, so chance will be the key factor for the discharge range.

Calibration Method

As the ponding method has given results less homogenous than the fluming method, as these results are more difficult to analyze, and as a loss rate calculated with this method is very sensitive to difficult to assess coefficients such as the Strickler, the fluming method is advocated as the measurement method.

Different steps are followed for the calibration of the loss rates:

1. Survey of the Watercourse

   - Quick appraisal by walking along the watercourse, if possible when it is at full discharge, in order to assess slope problems. Comparison of cross sections. Detection of highly particular places.

   - Farmers can be interviewed on the watercourse status; they will help in locating slope problem and will confirm the previous appraisal.

2. Identification of Proper Location of the Measurements

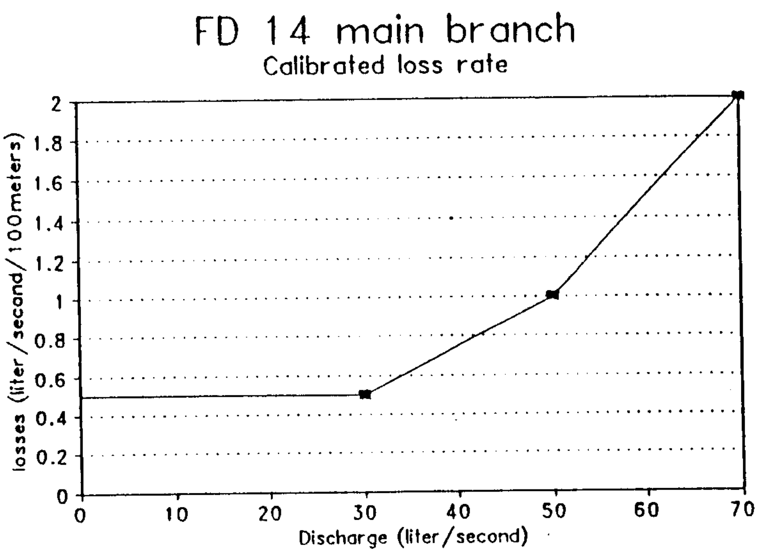
   - Using the previous results, measuring points should be planned.

3. Measurements
   - In order to correctly assess losses, sets of measurements shall be done with different discharges at the watercourse head. This is sometimes difficult, as the distributary is either full or empty and we do not control the water flows.

4. Calculation of Losses
   - Losses are calculated and further measurements planned, for example, to correctly assess where reaches with high losses start. The watercourse shall be divided into homogenous reaches in respect of these losses.

5. Calibration of the Watercourse Losses
   - The loss rate calculated with a given discharge at the watercourse head is an average for the range of discharges between the watercourse head and the measurement point. The model needs the instant seepage rate, at a given discharge. These instant seepage rates are higher than the average rate. The higher the average seepage rate is, the bigger the difference between instant seepage rate and average seepage rate. One should proceed by trial and error: a seepage rate file is entered; the model gives as an output the discharge at all the nakkhas: results of the model are checked with the measured discharge in the field. If results at low discharge are available, two linear pieces of loss rate function can be stuck together to better describe the phenomenon. It is assumed that with low discharge the loss rate remains constant (only the bed of the channel is wetted). Results were satisfactory except on FD14 main branch.

Figure 5.5. The calibrated loss rate as an input for the model in FD14.



6. Verification
   - Another set of measurements shall be made, at some nakkhas far from the previous fluming, in order to assess whether the model was calibrated properly, and whether the losses are consistent with the calibration.

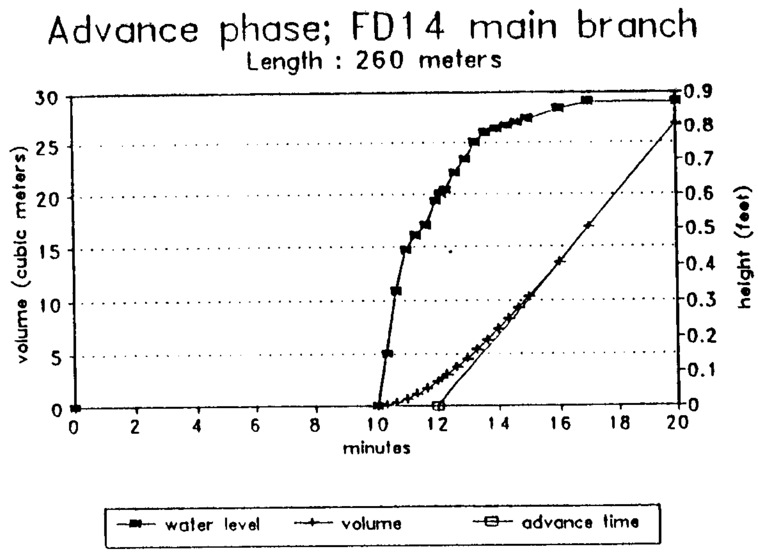
CALIBRATION OF THE ADVANCE PHASE

As stated before, the advance process of water in the reach per se is not of specific interest in the present study. The only point of interest is the volume of water supplied to a farm. The model is working with a constant advance speed for a given reach; the farm receives full supply after the duration of the advance phase (length over advance speed). Two calibration methods of the advance phase are presented below.

Measurements of the Nakkha Inflow

As we are working on the nakkha level, it is in fact the volume of water going through the nakkha during a water turn. Hence, similar to the method described in an attempt to assess drainage volume, to measure the volume delivered to the farmer during advance phase a gauge was inserted at his nakkha. The height of water with respect to time was recorded until the permanent state was reached. Using Strickler's coefficient, volume received by the farm can be computed. Figure 5.6 shows the data for a reach of FD14 watercourse.

Figure 5.6. Assessment of the advance phase duration.



(Features of the reach: Strickler=33; bed width=0.6;bank slope=0.5; slope=0.001).

A simple analysis of the data can be made in order to simplify the advance phase:

- The volume received by the farm during the unsteady phase is integrated.
- Using the steady state discharge, a new beginning time is calculated assuming full discharge is instantly delivered to the farmer.

From the volume point of view, this advance phase can be summarized in two periods: the first period without any discharge (12 minutes, the advance time in our case); and the second period with a full discharge. Therefore, advance time value for the model for FD14 is 12 minutes. The advance speed is thus deduced.

It has to be noted that the influence of the Strickler on such results is very low, because the transient phase at the outlet is short with respect to the advance time in the reach. The transient seepage losses are taken into account in such a measurement.

Assessment of Reach Storage

Another way to assess the advance speed is to follow the volume-balance approach. Considering the volume supplied to the farmer, he receives the volume delivered during his whole water turn, minus the volume stored in the reach which was empty before his turn. Measuring the height of the water in the reach at a given discharge enables the calculation of the volume stored and the "speed" of advance phase.

For the previous example, as the head of the reach supplied a discharge of 60 l/s, we have:

V=length * cross section=50 m^3.
Advance time=volume/discharge=13 mn 50 s.

This value is more sensitive to the reach cross section, and to the measurement of the water level. Moreover, in this case, transient seepage loss is not taken into account.

In FD 14 and FD 46, difference of advance time between these two methods were less then 20 percent; it can be explained by transient seepage, and by the sensitivity of the storage method.

In the previous example, for a reach which is 260 meters long, a two-minute inaccuracy compared to the duration of a water turn (usually more then half an hour long), represents less than 7 percent of the duration of the water turn.

Conclusion

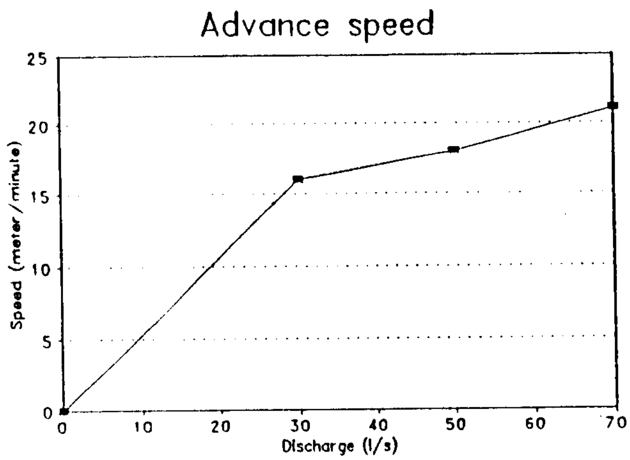
Given the small difference between these two methods (which also means that transient losses during the advance phase is negligible with respect to the volume supplied to a farmer), it was decided to work out the advance speed using the height measurement and the cross section at a given discharge and calculating the volume of water stored. These measurements were carried out before the flumes used to calibrate seepage losses were installed. The discharge is yielded by the flume.

As stated in the previous section, the main problem is that a low range of discharge is available in the four sample watercourses of the research locale, the distributary running either at full supply or with no water.

The advance speed, stemming from computation with the Kinematic-Wave Model, does not vary quickly with the discharge, apart from small discharges. This is quite understandable because the cross section is a function increasing with the discharge; the ratio of the volume of water stored over discharge is a ratio of two increasing functions, which increase slowly.

The input data for the model are given at different discharges. The model will extrapolate as shown in the following graph (Figure 5.7).

Figure 5.7. The calibrated advance speed as an input for the model.



CALIBRATION OF THE DRAINAGE PHASE

As stated before, methods to assess the drainage volume did not yield any good results. So the drainage phase speed is assumed to be the same as the advance phase speed. This means that the volume of water stored in the reach during the transient phase is assumed to be totally supplied to the last farmer. The same infiltration laws as during steady flow are used.

ORGANIZATION OF THE DATA COLLECTION

The main problem is related to the scheduling of the water supply (level of water in the distributary) and the warabandi. The White Mark measurements at the outlets were recorded before and after the fluming (most of the time, with a one-hour difference). If the distributary is in a steady phase, no difference will be measured; a difference of less than 0.05 feet (less than 0.1 cusec) is negligible; with a higher difference, measurements are not reliable.

Most of the time, the Fordwah Distributary is either full or empty; the intermediate discharges do not occur very often. Moreover, when they occur, the supply can change quickly at the head, and measurements are not reliable, as stated in the previous paragraph.

Thus it is hard to have good loss measurements for different outlet discharges.

The planning of the measurements are made according to the warabandi; for the head reach, measurement can be made five or six days a week, versus only one day a week for the tail end of the watercourse. Measuring the advance phase or the drainage phase must be carefully planned according to the warabandi schedule and taking into account specific arrangements made by farmers for modifying the warabandi.

Given these two main field constraints, collecting the data is a time-consuming activity.

Two watercourse or more should be calibrate at the same time in order to improve the data collection efficiency.

VALIDITY OF THE MODEL

Inaccuracy Related to the Model Assumptions

Neglecting Farmers' Reaches

In the present study, losses in the official field channels and in the official channel have been assessed. Thus, part of the total number of field channels have been taken into account; this proportion depends on the watercourse but it is never more then 30 percent (see map for FD 62). It can be considered that about 25 percent to 35 percent of the losses in the water coming to the field not been taken into account as shown by the study of Ali et al. (1978) quoted in Chapter 3.

Classification of the reaches

An important part of the calibration is the classification of reaches of a single watercourse with respect to losses and to advance speed. This work, stemming from a previous survey and from the measurements themselves, will have a major influence on the results given by the model: the watercourse can be divided into two main classes, and, more precisely into ten different classes.

Advance Phase

As stated in Chapter 3, the accuracy of the model will be low for the farmers having a long advance or drainage time with respect to their total water turn. It was found that such farmers are less then 10 percent, and half of them are involved in rotation schemes or in lease turns, thus reducing the inaccuracy of the results calculated by the model.

Inaccuracy Related to the Data Collection Methodology

Calibration of the Advance/Loss Law

The range of discharge available to calibrate the laws is rather small. This drawback will result in high inaccuracy of the volume of water delivered when discharges will be out of the range of the discharge used for the calibration of the model. Hopefully, such phases are quite scarce. During the rabi season, couples of two following days with high differences in the measured discharge (full supply and a supply less than 25 l/s) were recorded: 12 such couples for FD46, 15 couples for FD14. The inaccuracy on the computed volume related to these couples lasts less then one day.

The Outlet Discharge

As only one level is recorded a day, little is known on the fluctuations of the distributary discharge during a day. Moreover, during the transient phases related to ID operations at the head of the distributary, the timing of the decrease or the increase of water is unknown. The model extrapolates linearly between two daily following discharges, a probably valid approximation when these two discharges are similar, but less appropriate when the discharge at the watercourse head soars from 0 to 60 l/s in less than one day. Such transient phases have been quoted in the calibration of the advance/loss law.

The implementation of the SIC hydraulic model (Simulation of Irrigation Canals, developed by the CEMAGREF) on the Fordwah Distributary, to predict the water level of the distributary at the sample outlets, or the installation of a stage-recorder at the head of each outlet should improve this input data.

Transient Losses

Ali et al. (1978) state as a conclusion of their survey that an average of 15 percent of the losses in a watercourse are due to transient losses. As a comparison between the two advance phase measurements, these transient losses occurring during advance phase were found to be negligible. However, more than half of these losses (Ali et al. 1978), nearly 10 percent of the total losses, occur during the drainage phase as a consequence of dead storage. In this study, no measurements were carried out for the drainage phase; it has been assumed that the drainage volume is the volume of water stored in the reach.

Conclusion

Table 5.3. Conveyance efficiency computed by the model in the four sample watercourses.

	FD 14	FD 46	FD 62	FD 130
conveyance efficiency	76%	88%	94%	78%

One of the main conclusion that Ali et al. (1978) draw from the global conveyance study quoted in Chapter 3, is that the average conveyance efficiency of the watercourses in Pakistan is 60 percent.

In the present study, Table 3.1 displays the conveyance efficiency as an output of the model. This conveyance efficiency is much higher than the efficiency measured by Ali et al., especially for the watercourses, FD46 and FD62.

According to the work of Ali et al., between 30 and 40 percent of the total losses (not taking into account the farmers' field channels and the transient phases), are neglected by such a model. Nevertheless, the previous table highlights the main differences between the four sample watercourses. These differences are further studied in the following Chapter.

In order to valid the model, the volume of water at the nakkha level should have been measured and compared to the volume yielded by the model. During this short work (four months in Pakistan), no time was left for such a validation.

As the work of the calibration phase has been done on the assessment of volume, either through the estimation of advance speed or of the losses, and the model simply integrates these results, such a validation is less important then that with a more sophisticated model.

CHAPTER 6

A First Application of the Model

A first application of the model is presented in this chapter. For the rabi 93/94 season,the canal water supplied to farmers in the four sample watercourse command area of Fordwah Distributary have been computed, using:

- The farmer's warabandi, taking into account rotation schedules.
- The daily water level recorded at each outlet.

As the weekly warabandi changes slightly from the farmer's warabandi, this is not the real amount of water supplied to the farmer. Nevertheless, it can be interpreted as the amount they received before reacting by trading canal water.

As the canal water supply should be proportional to the area, the following results are presented in millimeter per week. A bias is therefore introduced, as the volume is not quoted. High differences for very small farmers arise from a negligible volume as compared to the amount supplied weekly at the head. But these small farmers, being poorer, should be highlighted in a study of equity, as they are more sensitive to the supply of water. In this chapter, no weighing factor is introduced in the analysis; the area of the farm is ignored.

In the first section of this chapter, the discharge at each outlet is presented: it is the result of the daily measurements of water level, and of the calibration of the outlets further detailed in Annex 2.

The raw result of the model is then presented. As it is difficult to analyze, a performance study is undertaken in order to understand better the different phenomena. this study particularly focuses on equity.

ANALYSIS OF THE OUTLET DISCHARGE

Figures 6.1 and 6.2 show the discharge at the head of each outlet with respect to the design discharge. The actual discharge of these watercourses are higher then the design discharge, particularly for FD46 and FD62, which outlets have been severely tampered with. The crest of the two latter mogha is lower than the distributary bed, so during the transient phases in distributary supply when water is not flowing anymore in FD14, there is still a significant discharge in FD46. Similar effects can be seen between FD62 and FD130, increased by the fact that FD130 is located at the tail end of the Fordwah Distributary.

Figure 6.1. FD14 and FD46: Outlet discharge during the rabi 93/94 season.

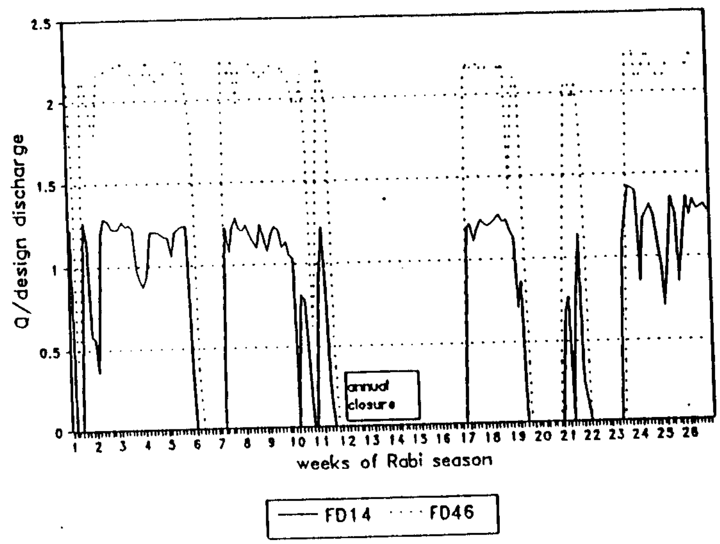
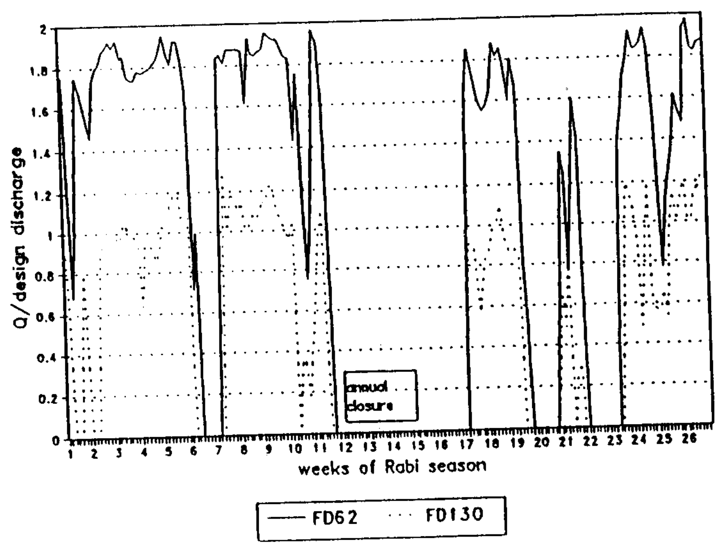


Figure 6.2. FD62 and FD130: Outlet discharge during the rabi 93/94 season.

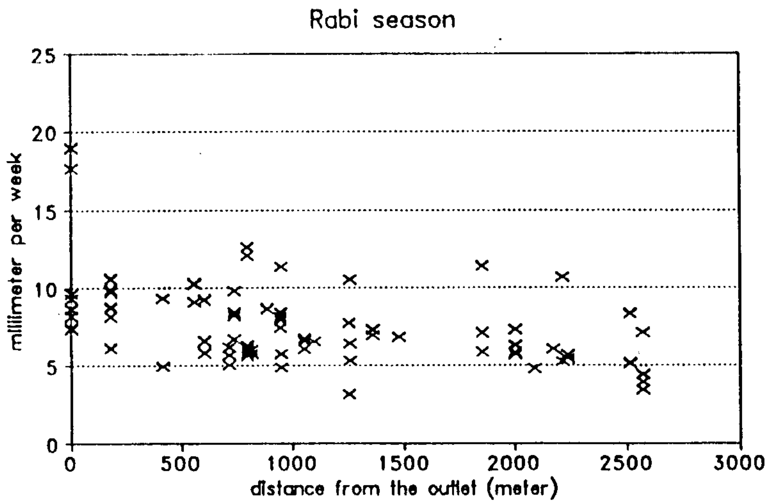


VOLUMES OF CANAL WATER SUPPLIED AT THE NAKKHA LEVEL

For each watercourse command area, the average amount of canal water supplied to a farmer during a water turn is plotted with respect to the distance from the head of the watercourse. (A farmer can have different water turns if he has fields located at different locations of the command area.) In order to check any aberration in the model output before the performance assessment, the analysis of the peculiar points that can be seen on the graph is undertaken. Whether this peculiar point is accurate, whether the model assumptions explain the peculiarity of this water turn, and whether it is a highly peculiar case can be worked out through this analysis.

Fordwah 14320-R

Figure 6.3. Quantity of canal water delivered to each farmer in FD14.



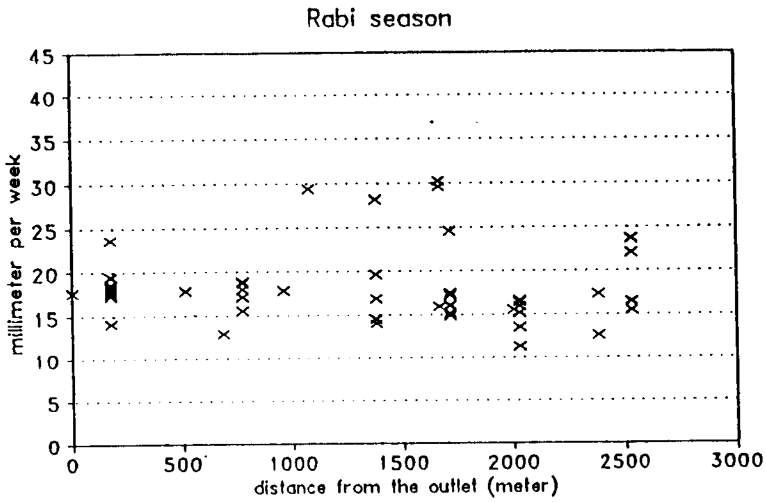
In FD 14 (Figure 6.3), two water turns located at the outlet were found to have more then 15 mm a week; they do not have advance or drainage time, they have very small areas (less then one acre) which can explain this long water turn. Volume computed is reliable in these cases.

One water turn 1,200 meters from the outlet receives less then 4 mm a week. It is located at the end of the first branch, where seepage is very high, and advance speed is low due to some slope problem; hence, such a volume is not surprising.

A decreasing trend with respect to distance from the outlet due to losses along the watercourse can be seen. This watercourse was found to have a high loss rate. Farmers located at the same nakkha (on the same vertical), can receive different volumes, owing to the warabandi.

Fordwah 46725-R

Figure 6.4. Quantity of canal water delivered to each farmer in FD46.



In FD 46 (Figure 6.4) height[4] of water supplied by the distributary is much higher due to modifications in the outlet structure. Four water turns receiving nearly 30 mm a week had a mango orchard. According to the ID rule, an orchard is to receive twice as much water as the other crops. These orchards do not exist anymore, but their lands still have the same double water turn; hence these four water turns are taken into account in the following indicator calculation. Seepage rate in this watercourse is not high, as the watercourse was improved to receive a higher discharge (twice the design discharge).

Fordwah 62085-R

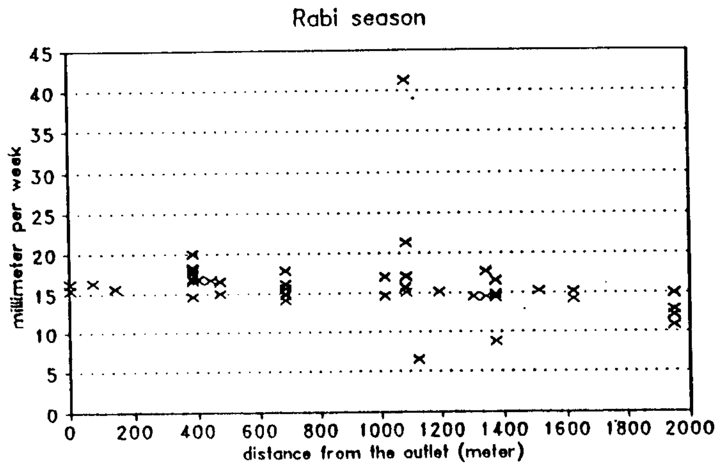
In this watercourse (Figure 6.5), one farmer receives more then 40 mm a week compared to an average of only 15 mm for the watercourse. A big drain passing through the middle of the watercourse takes away more then two thirds of this farmer's land. The water turn, however, was not changed. It was decided not to take into account this water turn in the calculation of the various indicators.[5]

[4] The Y-axis scale is different to that in the previous figure.

[5] It increases the equity indicator by 0.1.

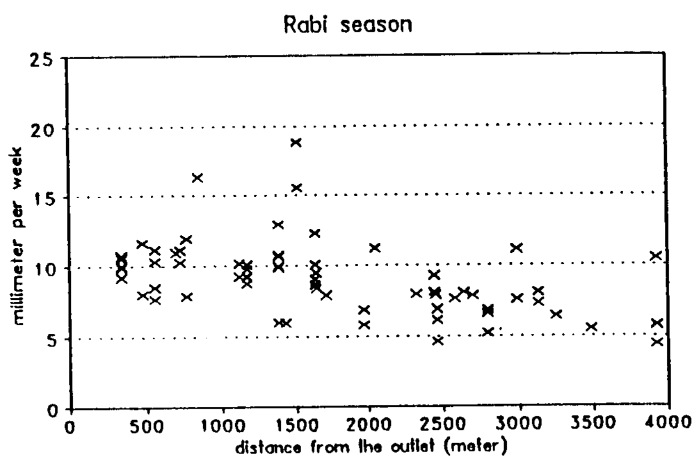
This watercourse has very low seepage losses, except the last two hundred meters which are located in an open area, i.e., barren lands are on each side of the watercourse and hence are damaged by cattle crossing.

Figure 6.5. Quantity of canal water delivered to each farmer in FD62.



Fordwah 130100-R

Figure 6.6. Quantity of canal water delivered to each farmer in FD130.



In this watercourse (Figure 6.6), three water turns receiving more then 15 mm a week were checked. They had an area close to the average area for the watercourse and their water turns did not include any transient phase. So these values were kept as stemming from inequity in the canal water allocation system.

Seepage losses in this watercourse are less than those for FD 14. However as its length is longer, the decreasing trend from the head to the tail is more significant.

Conclusion

These graphs provide a first idea on the water supply at the farm level and highlight inequity. However, it is quite difficult to sort out reasons for this inequity. In order to assess causes of inequity more clearly as well as appraise the overall performance of this system, performance indicators are calculated in the following section.

PRESENTATION OF PERFORMANCE INDICATORS

Until now, performance analysis was undertaken at the distributary level (Kuper and Kijne 1993). This model enables to better understand the impacts of seepage, outlet discharge, warabandi, and the ID operations on the farmers' supply of canal water.

Following Molden and Gates (1990), four performance indicators were calculated for these three sample watercourses. They were calculated both at intake level, thus comparing the three outlets, and at the farm level. We will compare the delivered amount to the design amount, as information on crop water requirements are not yet available. Hence, these performance indicators will assess the water delivery with respect to the goal of the Irrigation Department (ID).

Adequacy

The main goal of the irrigation system is to deliver the amount of water scheduled. In the case of the Indus Basin, this quantity does not match the amount to adequately irrigate crops; the goal of the Irrigation Department, in a scarce water context, is to spread the water equitably among the users.

The performance parameter for adequacy for a region R served by the system over a time period T is:

$$P_a=\frac{1}{T}\sum_T\left(\frac{1}{R}\sum_R p_a\right) \tag{1}$$

where

$$p_a=\frac{Q_d}{Q_s}\ \ if\ Q_d \leq Q_s \tag{2}$$

$$p_a=1\ \ otherwise$$

Q_d is the delivered amount.

Q_s is the amount scheduled by the ID.

When Q_d exceeds Q_s, the delivery is considered adequate regardless of the magnitude of the excess. Molden and Gates (1990) present a tentative performance standard in which a P_a value of more than 0.9 is assumed to be good, a value between 0.8 and 0.9 to be fair, and that below 0.8 to be poor.

Efficiency

This indicator will assess the waste of water from the Irrigation Department point of view. A poor efficiency indicator does not mean that water is wasted; it means that the system allows farmers to have more water than its scheduled amount. It is expressed as:

$$P_f=\frac{1}{T}\sum_T\left(\frac{1}{R}\sum_R p_f\right) \tag{3}$$

where

$$p_f=\frac{Q_s}{Q_d}\ \ if\ Q_s \leq Q_d \tag{4}$$

$$p_f=1\ \ otherwise$$

As the value of P_f approaches unity, this measure indicates an increasing compatibility with the goal of efficient water delivery for the region. The condition P_f=1 reveals that system is efficient, but provides no information regarding the extent of under delivery. This information is provided by P_a<=1. Similarly, when P_a=1, we have P_f<=1.

Molden and Gates (1990) suggest values of 0.85 and 0.7 as boundaries between good, fair and poor performance in terms of efficiency.

Dependability

Dependability is defined as the uniformity over time of the ratio of the delivered amount, Q_d, to the scheduled amount Q_s. A system that performs in a consistent manner is considered dependable. The value of this parameter is based on the hypothesis that farmers prefer a system that delivers water in a predictable way, even though it may be inadequate in amount, over a system that is unpredictable but delivers adequate quantity of irrigation water over the entire season. It is expressed as:

$$(5) \qquad P_d = \frac{1}{R}\sum_R CV_T(P_d)$$

where

$$(6) \qquad CV_T = \frac{1}{\bar{X}}\sqrt{\frac{1}{n}\sum_{i=1}^{n}(x_i - \bar{x})^2}$$

The closer the value of P_d is to zero, the greater the degree of dependability of water delivery. Molden and Gates (1990) suggest that performance is good if P_d falls between 0 and 0.1, fair for values between 0.11 and 0.2, and poor for P_d values above 0.2.

Equity

Equity of water distribution can be defined as the delivery of a fair share of water to users throughout the system. The stated objective in Pakistan's system is the delivery of a fixed proportion of a water supply based on the area of land owned by each farmer. Molden and Gates indicator for equity is the spatial uniformity of the ratio of the delivered amount to the scheduled amount (Qd/Qs):

$$(7) \qquad P_e = \frac{1}{T}\sum_T CV_R\left(\frac{Q_d}{Q_s}\right)$$

The closer the value of Pê is to zero, the greater the degree of equity is. Molden and Gates (1990) suggest as boundaries between good, fair and poor performance in terms of equity, P_e values of 0.1 and 0.25.

During the rabi season, the distributary has often been with no water or with little water (Figures 6.1 and 6.2), as highlighted by the low adequacy indicator. Higher values for FD46 and FD62 result from the greater discharges at the outlets, twice as much as the design discharge. The values calculated at the farm level are slightly lower because of the impact of seepage losses. The outlet design explains that the losses along the watercourse will have no influence on the adequacy at the farm level as compared to the outlet level, for FD46 and FD62, contrary to what happens for FD14 and FD130.

The efficiency indicator also stresses the important differences in outlet discharges between the four watercourses. The efficiency is high for FD14 and FD130, with well-designed outlets, because no excess water is delivered to these command areas through the canal system. In a way,there is no "waste of water." This indicator increases at farm level due to the losses along the watercourse.

The dependability is very low, the distributary has been run at full supply or has been empty. This has an important impact on the performance of FD130 and FD14, as they receive no water for a greater number of days during the season.

Study of Inequity

If equity is assessed on the volume of water received by farmers each week [Equation (7)], in a few weeks during which a break in the supply of the distributary happened, provide a very bad, and thus very high, equity indicator. The effect of such a period is important for the average on the whole season, and all the watercourses have similar indicators, equal to 0.5. But other factors influencing the equity of distribution are not expressed and are difficult to be detected.

Hence the equity indicator has been computed on the total amount of water received during the whole season, according to the following equation:

$$(8) \qquad P_e = CV_R\left(\frac{1}{T}\sum_T \frac{Q_d}{Q_s}\right)$$

Inequity can occur due to four main reasons:

a) **Seepage losses**: Farmers located at the tail end of the watercourse will receive less water than the ones located at the head of the watercourse.

b) **Warabandi design**: The actual warabandi was recorded. Every week, some exchanges of turns may occur. This is not taken into account. Because of the influence of some farmers, or bakshishing the Patwari, the warabandi may change from the official rules.

c) **ID operation**: Gates operations at the distributary level or at the main system level, performed by the Irrigation Department, can induce a shortage of water in the distributary at the same moment during a week, for several weeks. Thus the shortage of water would occur at the same time for many weeks, and farmers warabandi's turns are before these operation may have more water, inducing an inequity between farmers.

d) **Outlet design**: As already stated, the actual dimensions of the outlet can be different from the design and lead to inequity in canal water supply.

Assessing inequity related to these points:

a) To appraise the inequity stemming from the features of the watercourse, a warabandi following theID rules was developed. (It assumes that the watercourse has standard features: no losses, a constant drainage/advance speed equal to 22 meters per minute.) If the watercourse has these assumed features, the equity should be maximum. Using the calibrated model to calculate the amount of water farmers get through this warabandi enables the assessment of the marginal inequity related to the hydraulic features of the watercourses themselves. The discharge at the head is the average discharge for the distributary at full supply. (It is around this discharge that most measurements were done.) Results are given in Table 6.3, loss column. The inequity is higher for long watercourses, like FD130, or in watercourses where high seepage losses were measured (FD14). In FD62 and FD46, where seepage loss rates were low, inequity indicator is also low.

b) The inequity of the warabandi schedule used is measured assuming that the watercourse has standard features. The discharge at the mogha is also constant, as defined in the previous simulation.

This inequity in most of the watercourses is much higher than the one due to seepage losses as highlighted by the comparison between the loss column and the warabandi column in Table 6.3. Only in FD130, a particularly long and badly designed watercourse, are the two indicators equivalent.

Both factors are taken into account in the fourth column, where equity inside the watercourse is assessed with a constant discharge at the head. It can be concluded from these figures that the duration of the warabandi water turns does not balance the effect of seepage losses. The calculation of the duration of the water turns of the farmer's warabandi is, as far as these four watercourses are concerned, independent of the seepage losses.

c) In order to assess the inequity induced by the ID operation, a warabandi, taking into account the features of the reach, has been developed for the four watercourses as specified by Sharwar (1991). Such a warabandi yields, at constant discharge, an equity indicator approximately equal to 0.03. This warabandi was used for the whole rabi season data set and the equity indicator was calculated on the output data (Table 6.3, column iD).

Table 6.3 shows that this equity indicator is rather low and negligible compared to the inequity caused by seepage or warabandi. Hence the equity computed for the whole season is nearly the same as during a week at constant discharge.

Overall Performance Study of the Four Sample Watercourses on Fordwah Distributary

As equity is a special issue in the Indus irrigation system, it has been studied more precisely in the following section and will not be detailed here.

Table 6.1 shows the different indicators for the outlets of the four sample watercourses of Fordwah Distributary, calculated for the rabi season (16 october to 16 april). Q_s is the design discharge at the outlet, proportional to the culturable command area.

Table 6.1. Indicators calculated at the outlet level for the four sample watercourses on Fordwah Distributary.

	P_a	P_d	P_f	Q_s (l/s)
Target	1	0	1	
FD14	0.61	0.78	0.91	50
FD46	0.71	0.66	0.65	43
FD62	0.72	0.64	0.73	33
FD130	0.55	0.83	0.97	68

In Table 6.2, the indicators are calculated at the nakkha level of each sample watercourse, using the results computed by the model. Q_s which is the scheduled amount based on the design of the ID in millimeters during one week; different values for each watercourse account for the increase of the culturable command area (FD14), or the decrease of this area because part of the land is barren (FD130).

Table 6.2. Indicators calculated at the nakkha level for the four sample watercourses on Fordwah Distributary.

	P_a	P_d	P_f	Q_s (mm)
Target	1	0	1	13.7
FD14	0.51	0.77	0.97	13.54
FD46	0.70	0.66	0.72	14.56
FD62	0.71	0.65	0.77	14.34
FD130	0.48	0.82	0.93	17.64

Table 6.3. *Equity indicators for the four sample watercourses on Fordwah Distributary.*

EQUITY	Loss	Warabandi	Both	ID	whole season	Mogha	General
FD14	0.19	0.28	0.36	0.08	0.35		
FD46	0.08	0.23	0.22	0.04	0.22		
FD62	0.08	0.15	0.15	0.04	0.16		
FD130	0.2	0.2	0.28	0.06	0.28		
Disty						0.36	0.49

d) The inequity caused by the outlet specification is the spatial variability for these four mogha of the average over the whole season of the ratio Q_d over Q_t. Results (Table 6.3, mogha column) are comparable to the inequity in the command area of FD14, and much bigger than the inequity inside the other three watercourses.

Conclusion

On the whole, between all the farmers (Table 6.3, general column) of these three sample watercourse inequity is 0.49 mainly because of (in order of importance):

1) The dimensions of the mogha.
2) The warabandi schedule implemented.
3) The seepage losses.
4) The operation of the distributary and of the main canal by the ID.

The actual dimensions of the mogha comprise the main factor explaining the inequity of water supply at the farm level. However, four calibrated watercourses on this distributary are not sufficient to make this conclusion. The fact that seepage losses take third place in respect to the impact on equity is however quite surprising.

As stated in Chapter 3 the model does not take into account the farmers' field channels, and thus neglects about 30 percent of the losses occurring in a watercourse. Neglecting these losses improves the equity indicator.

Watercourses with discharges higher than the design (FD46,FD62) seep less. This seems to be in contradiction with Trout et al. (1981), which found that SCARP watercourses (with a public tubewell, thus discharge at the head is increased) have a higher loss rate .

Farmer improvements of the watercourse itself, as a response to a higher discharge at the watercourse head, may explain the low loss rates for FD46 and FD62. Another reason for the low loss rate is that these two watercourse have a single main branch, as compared to FD14 and FD130, both longer with at least two main branches (see Trout et al. 1981).

The other indicators show that the main problem is the poor operation by the Irrigation Department in supplying water to the distributary. This can be explained by the location of the distributary, at the tail of Fordwah Branch, and more than 100 km downstream of the Suleikmanki Headworks.

However, the analysis of these results and particularly the equity analysis suffer from the assumptions of the model, as an average of 30 percent of the total losses are not taken into account. But the main differences between FD14, FD130 on the one hand, and FD62 and FD130 on the other remain, as well as the importance of inequity stemming from the farmer's warabandi and the mogha design.

CHAPTER 7

Conclusion

The present model is a first step in the assessment of the supply of water at the farm level. It enables giving prominence to the institutional deficiencies of the warabandi system. Far from taking into account losses for an improved equity, the warabandi implemented in the fields induces an inequity more important than the one stemming from the losses. It also shows that tampering of the outlets has a major effect on the equity of the supply of canal water.

However, some major assumptions reduce the scope of the model. A first step in improving the model is to take into account the farmers' field channels. It involves, necessarily, a mapping of each location of the farmers' fields. The length of the farmers' field channels leading to a particular field can be approximately deduced from this mapping. Once the calibration of a sample of the farmers' field channels is made, they could be included in the model. For such a method, referred to Menenti et al. (1992), who give some indications on the appraisal of agricultural water in large schemes using sampling methods. On the hydraulic point of view, the major inaccuracy stems from the assessment of the drainage phase, which concerns only a few farmers.

The loss rate was found to be highly variable with the localization of the channel, and also with time, as the loss rate after a cleaning session is less than before. Therefore, modelling the supply of water in a watercourse command area is a rather difficult and questionable objective.

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Annex 1

Importance of the Advance/Drainage Phase

Assuming a constant advance speed and a drainage speed of 12 meters per minute, the ratio advance/drainage time over total water turn duration has been computed. Farmers whose farms have a ratio bigger then 30 percent only are reported. (When no value is given, it means that the water turn duration is null: The farmer only receives drainage water.) The advance/drainage speed used is far less than the reality (20 m/minute), but it is the value yielded by the Kinematic-Wave Model presented in Barreteau (1993).

The third column of Table A1.1 reports whether the volume is affected by the advance phase (kha for Khal Baraì) or drainage phase (nik for Nikhal). The last column gives the area of the plot attached to the given water turn. As farmers who reported are likely to have a short water turn, the area concerned is smaller than the average area in the watercourse. Percentage values are quoted in the summary.

As a reaction to the inequity induced by an unpredictable advance or drainage phase, which particularly affects the farmers reported in Table A1.1, some farmers have implemented a rotation schedule. Such a rotation balances the effect of the advance or drainage phase on 2, 3, 4 or 5 farm.

Another agreement can be a lease turn, as very small farmers prefer to have enough water to irrigate their whole land once a month, than little water once a week. This information is reported in column 5.

The difference between FD14 and FD130 on one side and FD62 and FD46 on the other side is already highlighted by the table. As FD46 and FD62 watercourses have a single main branch, few farmers of these watercourses are reported in the table. As FD14 and FD130 watercourses have at least two branches, total length is increased, and several farmers of these two watercourses are reported. On the whole, 6 percent of the total area and less than 10 percent of the total number of water turns, more than half of which are following a rotation schedule, are reported in the table.

Table A1.1. Ratio of advance/drainage time over total water turn duration for the four sample watercourses.

WC	farm #	kha/nik	ratio	rotation	area
fd14	10	k	30	n	3
fd14	20	k	43	y	2
fd14	22	k	70	y	2
fd14	24	n	46	n	0.625
fd14	29	k	33	n	2.25
fd14	31	n	77	n	2
fd14	43	k	38	y	4.5
fd14	64	k	83	y	0.5
fd14	89	k	125	y	0.5
fd14	76	n		n	4.25
fd46	16	k	55	y	1
fd46	28	k	33	y	3.5
fd46	47	k	62	y	2.25
fd46	55	n	80	y	10.25
fd62	34	k	96	n	1
fd62	53	n	44	n	9
fd130	17	k	30	y	7.5
fd130	30	k	44	n	4.25
fd130	33	n		n	0.5
fd130	77	n	173	y	1
fd130	56	n	33	y	15
fd130	59	k	75	y	8
fd130	60	k	75	y	1
fd130	64	n	50	n	4.5
fd130	73	n	100	n	12
WC	total farms	with rota	no rota	% area	
fd14	84	5	5	4.2	
fd46	54	4	0	4.2	
fd62	53	0	2	3.1	
fd130	74	5	4	11.5	
sum	265	14	11	6	

Annex 2

Calibration of the Outlet

Each outlet of the sample watercourse was improved with measurement devices: Two White Marks, one upstream of the outlet (measuring the water level in the distributary), and one downstream of the outlet were inserted on concrete structures. During the annual closure, the level of the crest of the outlet was recorded with respect to the upstream White Mark. A similar work was done at the downstream side, for the level of the bed. IIMI has been recording White Mark measurements every day for the last four seasons at least, which enable, once the outlet is calibrated, the estimation of the discharge at the head of the watercourse.

The measurements required for the calibration of the outlet can be carried out during the seepage loss measurements.

All the outlets that were seen are of the AOSM (Adjustable Orifice Semi Module) type (Mahbub and Gulhati 1951). In fact, it is an Open Flume with a Roof Block, slightly adapted to the silt conditions, so that each watercourse gets an even share of silt, and that the distributary will not be heavily silted. The slope of the overture enables a supercritical flow, with a hydraulic jump a few centimeters downstream. Therefore, Hb, which is the measurement downstream, was not relevant for all the calibrations developed.

Two different regimes were calibrated for each outlet:

Free Flow: The water level is too low in the distributary to reach the Roof Block; it is a simple flume, for which discharge is given by:

(1) $$Q = K B_t H_a^{3/2}$$

(Mahbub and Gulhati 1951)

where

- H_a is the height of water with respect to the crest in feet (yielded by White Mark measurement upstream).
- B_t is the width of the opening of the flume in feet.
- K is a design coefficient adjustable with B_t.

Orifice Modular: The water level is higher than the Roof Block. The discharge is given by:

(2) $$Q = K' B_t Y \sqrt{H_s}$$

where

- Y is the height of the Roof Block in respect to the crest in feet.
- H_s is the level of water with respect to the bottom of the Roof Block in feet.
- K' is a design coefficient, from 7.3 to 8.3.

The features of the outlets Y and B, have been recorded during the previous annual closure, and they were checked again during the study (no change occurred for these four watercourse).

Those formulas are the design formulas; for the calibration, three steps are followed:

(1) To match the design formula with field measurements.

(2) If the results are not satisfactory, because the outlet has been tampered with (for example, the slope of the flume is not the designed one), a second attempt is made changing only the coefficient in the formula.

(3) If the results are still unsatisfactory, a regression is run comparing ln(Q) with ln(Ha) (free flow) or ln(Q) and ln(Ha-Y) (Orifice Modular).

For the calibration, continuity of the discharge between Free-Flow and Orifice Modular is assumed.

H-Q Rating Curves:

FD14:

$$F\ F:\ Q=1.17\,H_a$$
$$O\ M:\ Q=2.4\,(H_a-0.95)^{1/2}$$

FD46:

$$F\ F:\ Q=H_a^{3/2}$$
$$O\ M:\ Q=3.06\,(H_a-1.4)^{0.24}$$

FD62:

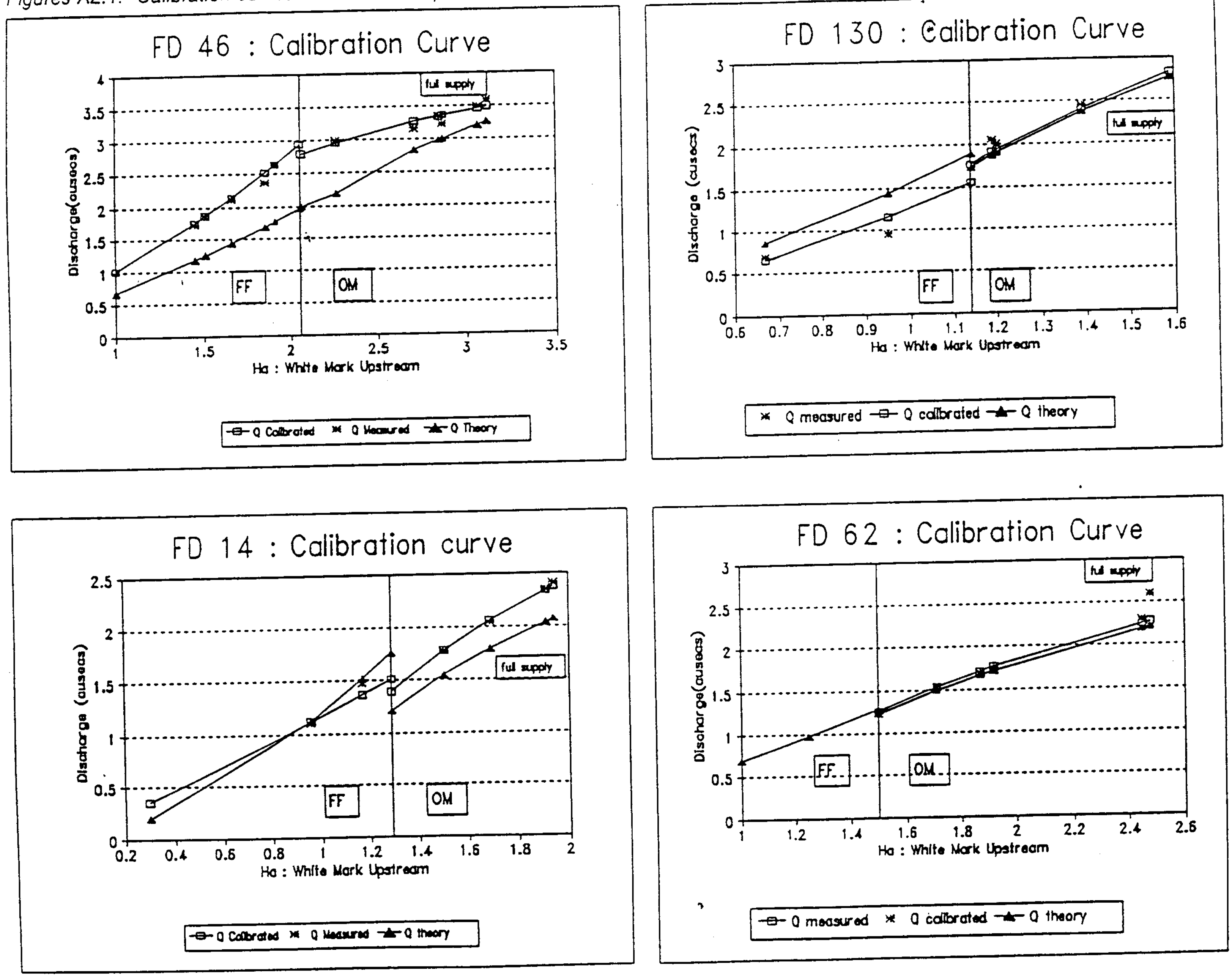
$$F\ F:\ Q=0.7\,H_a^{3/2}$$
$$O\ M:\ Q=1.9\,(H_a-1.06)^{1/2}$$

FD130:

$$F\ F:\ Q=1.26\,H_a^{1.64}$$
$$O\ M:\ Q=3.29\,(H_a-0.85)^{1/2}$$

On the following page, the four figures showing the measured, calibrated and theoretical discharges for the four watercourses are displayed.

Figures A2.1. Calibration curves of the four sample watercourses.



Annex 3

Maps and Calibration Data of the Four Sample Watercourses

Figure A3.1. Map of the watercourse command area: Fordwah 14320-R.

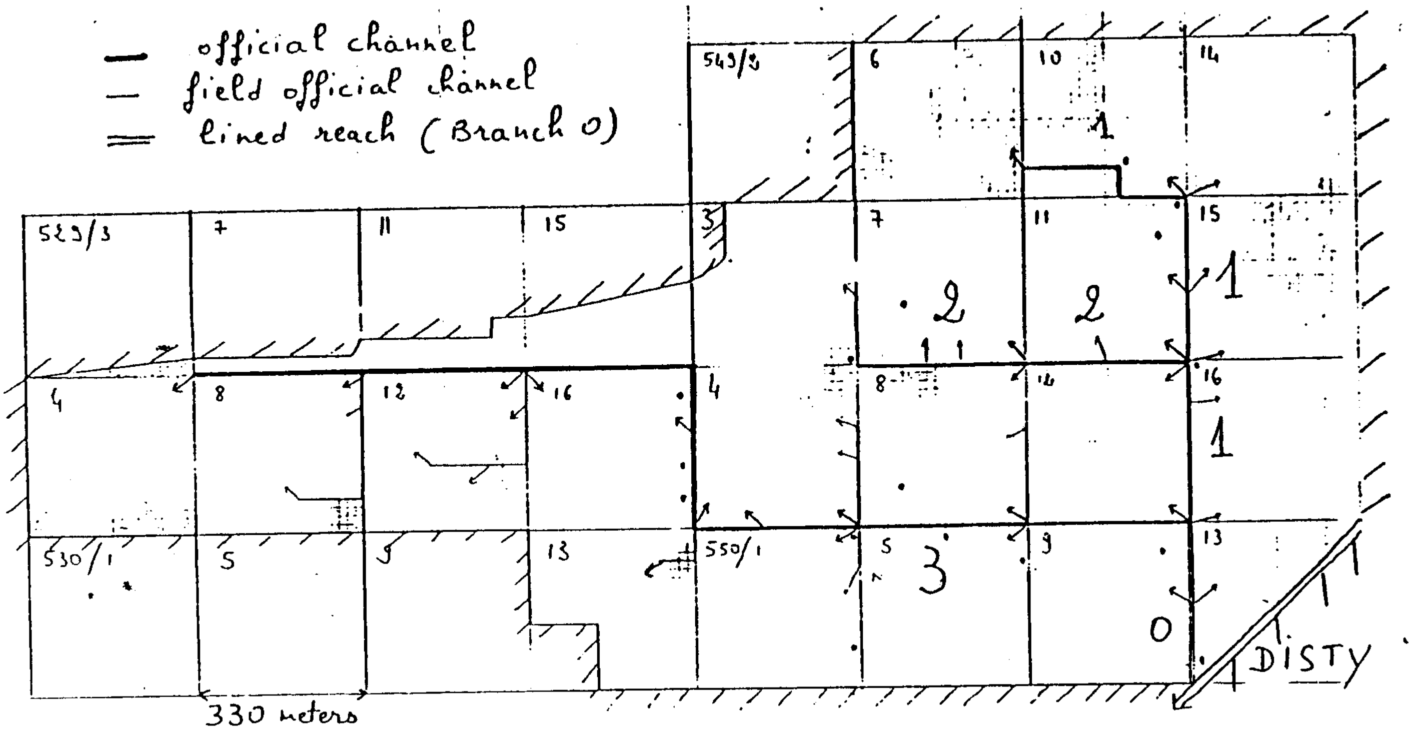


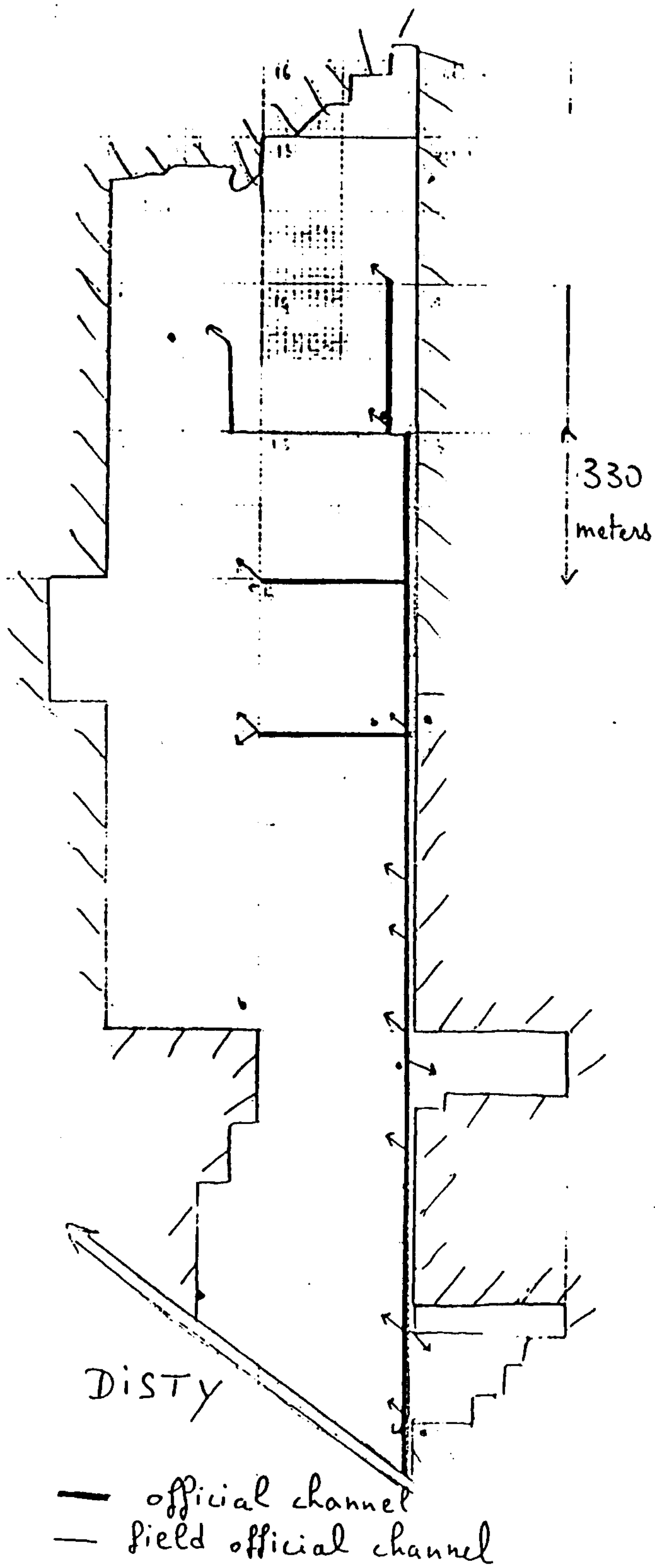
Table A3.1. Physical characteristics of FD14.

	Branches 2 and 3	Branch 1
Bed width (cm)	50 to 60	60
Slope of the banks	1/2	1
Water level (cm)	25	30 to 35
Width at the surface (cm)	85 to 90	120
Slope	6/10,000	2/10,000
Infiltration rate (l/s/100m) at 50 l/s	1	3
Infiltration rate (l/s/100m) at 70 l/s	2	4.5
Advance speed (m/mn)	18	13

The slope of Branch 1 is very low and quasi-null in the last 300 meters, which explains its high seepage rate.

As compared to its overall good maintenance, and the good conditions of the flow, seepage of Branches 2 and 3 is high.

Figure A3.2. Map of the watercourse command area: Fordwah 46725-R.



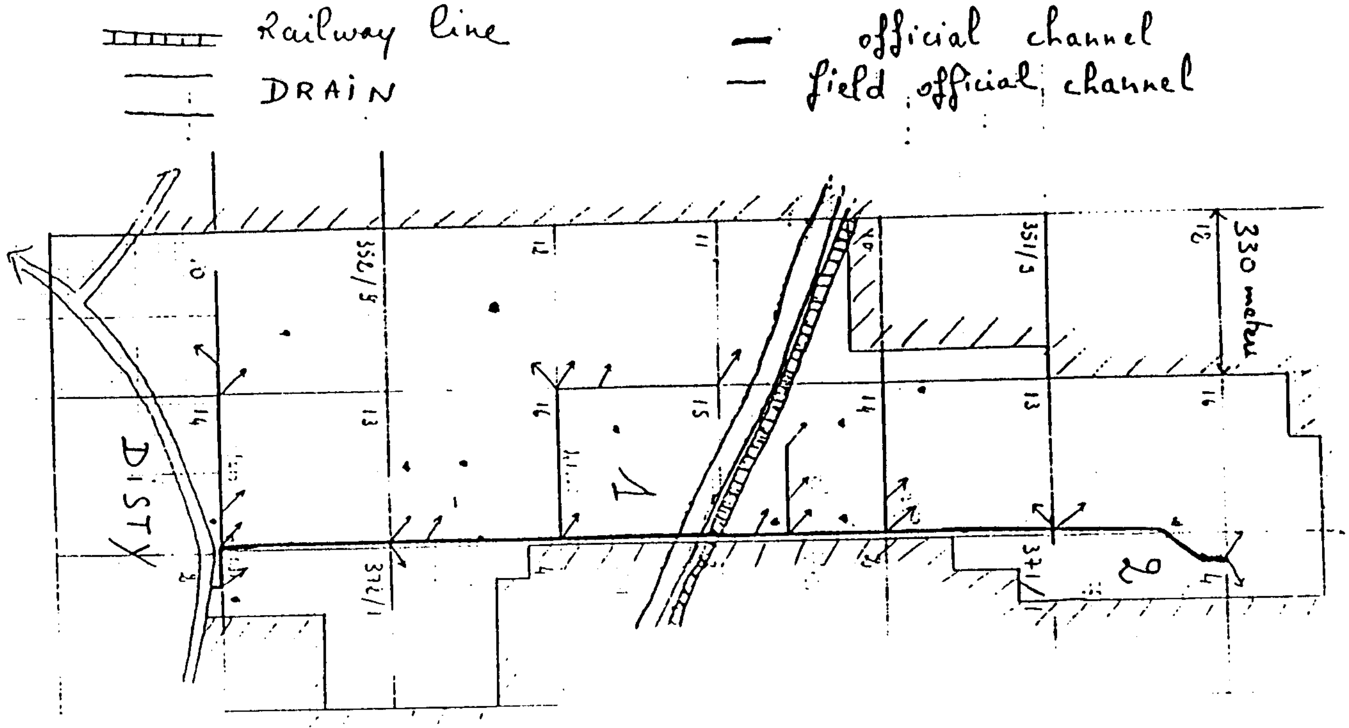
The main branch of this watercourse is homogenous with respect to seepage losses and advance speed, in spite of the last 300 meters of the first branch, which are badly maintained.

Table A3.2. Physical characteristics of FD46.

Bed width (cm)	60 to 70
Slope of the banks	1/3
Water level (cm)	40
Width at the surface (cm)	90 to 100
Slope	4/10,000
Infiltration rate (discharge=70l/s)	0.6 l/s/100m
Infiltration rate (discharge=100l/s)	0.9 l/s/100m
Advance speed (m/mn)	21

Obviously, this watercourse has been improved in order to receive a greater discharge.

Figure A3.3. Map of the watercourse command area: Fordwah 62085-R.



This watercourse was divided in two main portions after the measurements of seepage losses.

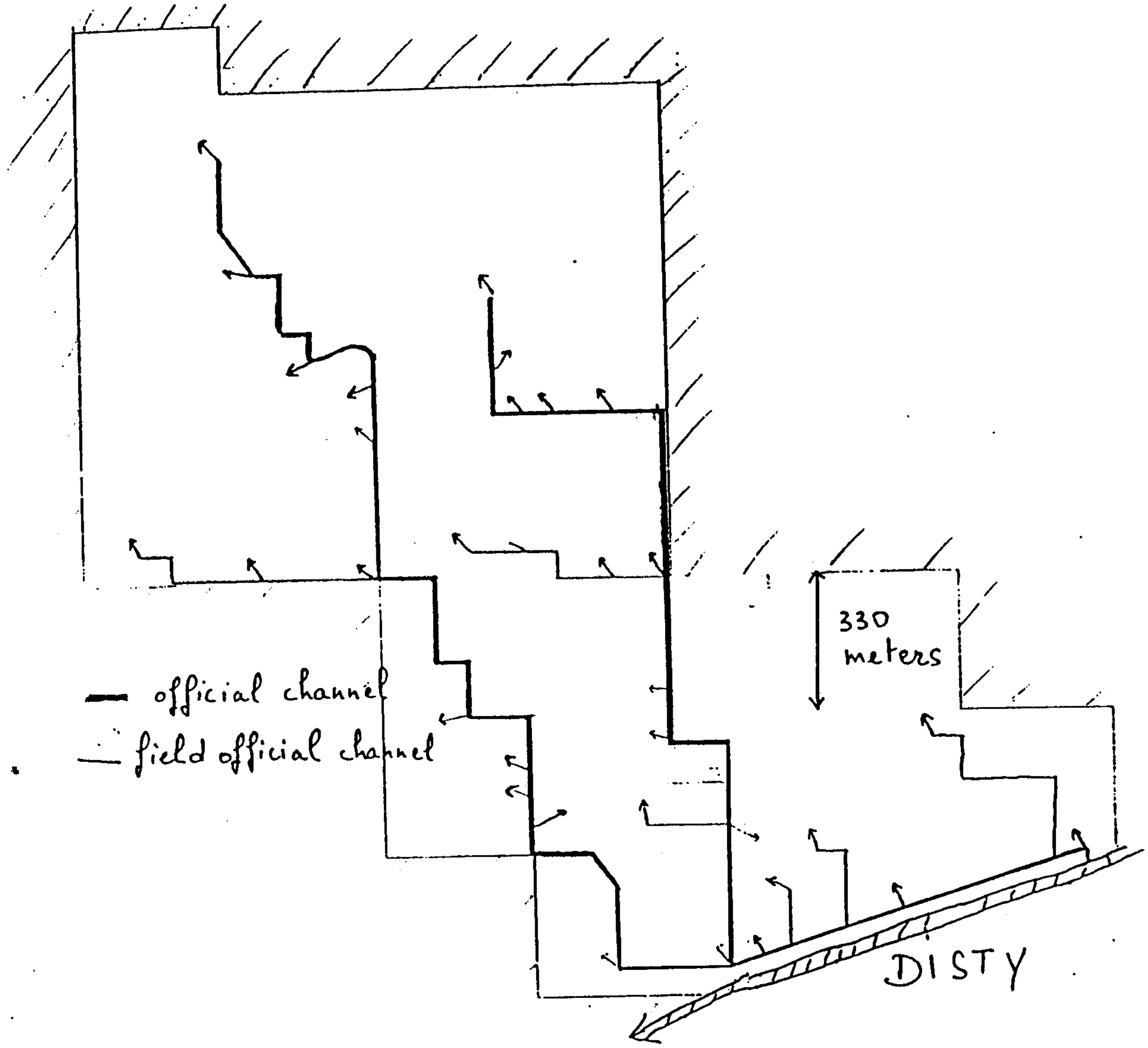
The first portion, which ends 300 meters before the end of the main branch, was found to seep very little. In the middle of this portion, a metal reach crosses a drain, 25 meters large.

The second portion starts in an open area, where cattle usually cross the watercourse. The slope is small, and the seepage rate is very high. As the cross section is highly variable, the advance speed was measured.

Table A3.3. Physical characteristics of FD62.

	Portion 1	Portion 2
Bed width (cm)	55 to 65	
Slope of the banks	1	
Water level (cm)	20	
Width at the surface (cm)	90 to 100	
Slope	8/10,000	2/10,000
Infiltration rate (l/s/100m) at 50 l/s	0.2	7
Infiltration rate (l/s/100 m) at 80 l/s	0.4	13
Advance speed (m/mn)	24	15

Figure A3.4. Map of the watercourse command area Fordwah 130100-R.



The two main branches are homogenous with respect to seepage. The soil is more sandy than in the other three watercourses.

Due to the length of the branches, losses can be very high.

Table A3.4. Physical characteristics of FD130.

Bed width (cm)	65 to 75
Slope of the bank	1
Water level (cm)	20
Width at the surface (cm)	110
Average slope	6/10,000
Infiltration rate (discharge=60l/s)	0.8 l/s/100m
Infiltration rate (discharge=80l/s)	1 l/s/100m
Advance speed (m/mn)	20

Annex 4

Model User's Guide

The Input Files

Four files are needed as an input by the model; two of them deal with the geometry of the watercourse, one is the warabandi file, and the other, the hydraulic file, stems from field measurements.

The Geometry File

Using the command IIMI Data/Geometry Setup on a new spreadsheet will display the standard formula to record the geometry of the watercourse.

Example: (FD46)

Geometry of the watercourse			
Reach #	Class	UPS Reach #	Length
=========	=========	============	=========
1	1	0	1512
2	1	1	288
3	1	1	336
4	1	2	333
5	1	2	360
6	1	5	500
7	1	5	350

The geometry of the watercourse takes into account the different branches of the watercourse network. In order to work out this geometry, the nodes (places where different reaches meet) should be pointed out on the map. A branch starts at a node, ends either at the following node or at the end of the reach. The numbering can start upstream, going downstream. Each branch is numerated, and is spatially localized by the reach upstream, from where the water is coming in order to get into this branch (UPS reach #). The first reach starting from the outlet has an upstream reach which number is 0.

The class refers to the hydraulic parameters (seepage, advance speed). So, after the measurements are carried out, the reaches can be classified according to their seepage rates. One "geometric" reach can be divided in two hydraulic reaches to take into account the seepage losses. The numbering should then be changed.

The last column records the length of the reach in meters from head to tail. This can be obtained either from a topographic survey, or in a first approximation from the ID map.

The Nakkha File

On the warabandi list used, the farmer's nakkha are recorded as a location on the ID map. This location needs to be translated in the reach codes used to describe the geometry.

Example: (FD46)

NAKKHA	REACH CODE	METERS
=========	===========	=========
412/13/15	1	132
412/13/16	1	132
411/16/25	1	306
432/01/01	1	306
431/03/10	1	813

All the nakkha used in the warabandi file shall be recorded in this nakkha spreadsheet; the program shall othetwise quote the nakkha which location it does not know.

The third column is the location on the reach, the distance (in meters) starting from the head. One should make sure that this distance is not greater than the total length of the reach (Table 1).

The Hydraulic File

In order to assess the volume of water a farmer gets, three main inputs are needed:

* Seepage rate

* Advance speed

* Drainage speed

As these values depend on the discharge value, measurements taken at different discharges enable the approximation of the seepage curve with a semi-linear function, same for the advance and drainage speed.

In this study, on account of the difficulty of assessing the drainage volume, we assume that drainage speed is the same as advance speed, which means that a farmer located at the end of a reach receives all the water stored in this reach.

A typology of the watercourse in each command area should be done in respect of the seepage losses. A branch (from a node to the following node) can be divided according to this typology. The geometry file refers to these classes.

Example:

Hydraulics of the watercourse.				
For each class, discharge should be put in growing order.				
Class	Q (l/s)	S (l/s/100m)	Va(m/mn)	Vd(m/mn)
=========	=========	=============	=========	=========
1	75	0	25	25
2	30	0.5	16	16
2	50	1	18	18
2	70	2	20	20
3	30	1.5	12	12
3	50	3	13	13
3	70	4.5	14	14

Class 1 is the line watercourse at the head; seepage is equal to 0. In a first approximation, advance speed is considered constant in respect to discharge.

Class 2 is a narrow watercourse with a good slope, mainly located on the main branch.

Class 3 is a wide watercourse with slope problem on the first branch.

Warabandi File

As compared to the formula used to record warabandi, some changes have been made:

* Combine turns are taken into account in the column, Combine.

In the order of the warabandi, combine turns are numerated; the same number is given to the farmers combining their turns.

* Rotation: In order to decrease inequity stemming from the advance or drainage phase, farmers are rotating.

  The column, Rotation numerates in the order of the warabandi the different rotation occurring during a week. The farmers involved in the same rotation get the same number.

Management of the Files

All these files are entered In Quattro in a standard form. They must be converted into print files in order to be read by the model. They should be written in proper size in order to be translated by the model.
A specific menu has been built in Quattro. This menu is loaded by retrieving the Quattro.wq1, file which is the main directory (JP).
The command used in order to manage the different files in a proper way are all in IIMI Data.

General Command (for hydraulic, geometry and nakkha file)

1-To enter data.

In order to properly write these four files, a setup macro shall be used (in IIMI Data), which retrieves the standard spreadsheet formula.

2-To save data.

When these files are entered, they should also be saved as a print file. The command IIMI Data/Print block/Select Prn is used.

- It selects all the spreadsheet.
- It asks for the output prn file (one ought to give the same name than the .WQ1 file).

The macro is finished.

The spreadsheet HAS STILL TO BE PRINTED, using Print/Spreadsheet print.

Model Command

Running the Model

- The model shall always be run in the directory of the concerned watercourse.

- Two models can be used

  * pakijpr : takes into account the rotations, giving an average volume to the farmers.

  * pakijpnr : does not take into account the rotations.

- The model asks the name of the four print files quoted ahead.

- For the discharge at the mogha, three options:

  * constant discharge (l/s)

  * discharge per day ; using the daily discharge recorded at the mogha.

The computer asks first for the time of the beginning of the warabandi. (6 a.m. during rabi, 6 p.m. during rabi).
The time of the measurements and the discharge (cusecs) should then be entered; if the discharge was not measured (Friday, or off-day), then type -1. The computer will calculate the discharge every minute a linear interpolation between two following discharge measured.

  * discharge at any time; type the time (min) and the discharge (l/s).

- The results displayed are the farmer turn:

  * beginning of his turn as stated in the warabandi.(mn)

  * end of his turn as stated in the warabandi.(mn)

  * the volume of water they get as a stair function;

when discharge is positive, the time stated is when they start to have water, that can be a few minutes after the schedule beginning if there is an advance phase. Each discharge related to time is the beginning of a new step. At the end, they can get water longer then the scheduled time, if they have a drainage phase.

- On the final file:

  * farmer's name with identification number.

  * total volume of water they get in cubic meter.

  * height of water they can put on their total area with this volume (volume divided by area displayed in mm).

Warabandi Command

1- Updating data.

In order to update the former warabandi files (entered by IIMI until now), setting the column at the right size, add the two new columns; the macro IIMI Data/Warabandi entry/Warabandi Update should be used.

2- Entering data.

Same way as previously explained using Warabandi Setup.

3- Working on data.

Combine water turns should be aggregated for the use of the model; as we do not know exactly how the share of water was made, and thus it is impossible to assess the volumes. The macro IIMI Data/Warabandi entry/Warabandi Combine aggregates the different combine turns into one, keeping the name and ID of the first farmer.

Rotation can occur between the farmers. In order to easily change the Warabandi lists, a macro IIMI Data/Warabandi entry/ Warabandi rotate asks you the number of the rotation you want to change, and the new ident of the farmer first.

4- Saving data.

- a) run Warabandi Align in order to center a few column.

- b) run Print Block/Select Warabandi. A few difference will occur in the spreadsheet

  * rotation will become second column.
  * data related to the owner are erased.
  * Nakkha out of the last farmer will be Nakkha in of the first one.

The blocks to be printed are selected, and the macro asks you for the PRN file name.
As with the other kind of file, the spreadsheet NEEDS TO BE PRINTED, using Print/Spreadsheet Print

CAREFUL: The spreadsheet in this new form shall not be saved! Warabandi previous to this print operation shall be kept.

5- Organization of files:

The file with the standard Warabandi shall be saved in two different files, because Macro manipulations can damage the file. One file is a security file, the other is used for the translation in a print file, and for various updating.

Abstract

Poor irrigation performance is the main concern of irrigation systems in Pakistan, as it jeopardizes the sustainability of the agricultural sector. At the tertiary level, assessing efficiency has been hitherto undertaken by the monitoring of watercourses. Hence, in order to better understand the allocation of canal water inside a watercourse command area, a hydraulic model is developed in the present study.

The aim of the present model is to calculate the volume of canal water delivered at the farm inlet, given the water allocation system (which is a roster of turn) and the tertiary canal inlet discharge. As the tertiary canal is short, and the flow is mostly steady, a volume-balance model is accurate enough to calculate the volume supplied to the farmers. A protocol is set up in order to collect the data for the calibration. For the assessment of the seepage losses which is the main input of the model, the fluming method is advocated. Measured advance speed is also an important input.

An application of this model on the winter season 93/94 enables to assess the main reason of inequity at the watercourse level. Inside a watercourse command area, the fixed roster of turn which should allocate the canal water in an equitable manner induces important inequity, more than the inequity induced by the losses in the main tertiary canal.