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FACTORS AFFECTING LOSSES FROM INDUS BASIN IRRIGATION CHANNELS

By Thomas J. Trout

Water Management Research Project
Colorado State University
Fort Collins, Colorado

July 1979

WATER MANAGEMENT
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FACTORS AFFECTING LOSSES FROM INDUS BASIN IRRIGATION CHANNELS

Water Management Technical Report No. 50

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Prepared by

Thomas J. Trout

Water Management Research Project
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ABSTRACT

FACTORS AFFECTING LOSSES FROM INDUS BASIN IRRIGATION CHANNELS

Tertiary irrigation conveyance systems (watercourses) in the Indus Basin lose 30 to 50 percent of their flow. Watercourse systems were studied in depth by ponding and inflow-outflow methods to determine functional relationships between several measurable parameters and the loss rates. The objective was to determine simple design changes that are low cost and can lead to increased conveyance efficiencies in the earthen channels.

Statistical analysis of the collected data indicated that:

1. watercourse loss rates (lps/100m) increase with, but slightly less than proportional to, the usual flow rate in the channel;
2. loss rates are lower in more often used channels;
3. loss rates are higher in elevated channels;
4. loss rates are very sensitive to changes in flow depths, and thus increase with upward fluctuations in flow rates or roughness coefficients; and
5. intake rates into upper bank soils are very high and are apparently caused by extensive rodent and insect burrows inside the banks.

A watercourse loss model was constructed based on the derived relationships, and was applied to several practical watercourse design alternatives.

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Thanks is also due to the many farmers who allowed us to use their watercourses as a laboratory in which to conduct the measurements, often providing valuable water for the ponding loss studies, and allowing us to install flumes in their channels, even though many were convinced that the flumes "ate" some of their water.

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LIST OF ABBREVIATIONS

a	coefficient defined by equation 21
b	coefficient defined by equation 11
c	coefficient defined by equation 21
d	water depth in a channel (cm)
fsl	full supply level
Δh	increased flow depth as a result of an obstruction or high field (cm)
ha	hectare (10,000m^2)
hm	hectometer (100m)
lps	liters per second
lps/hm	liters per second per 100 meters
n	Manning's roughness coefficient
n_o	original roughness coefficient value
osl	operational supply level
r^2	coefficient of determination
t	irrigation turn time
A	cross-sectional flow area (m^2)
BW	bank width at osl (cm)
CL	recently cleaned channel
D	distance from the mogha (hm)
D_{SK}	length of sarkari khal section (hm)
D_{FB}	length of farmer's branch section (hm)
E	conveyance efficiency
E_A	conveyance efficiency adjusted for flume effects
ΔE	osl elevation with respect to the adjacent land surface (cm)
EL	water surface elevation drop from mogha to the field (m)

F	F statistic
FB	farmer's branch
Gr	grassy, uncleaned channel
I	recently improved (rebuilt) channel
K	coefficient defined by equation 14
K_1	coefficient defined by equation 5
K_2	coefficient defined by equation 16
L	total losses (%)
L_D	length of channel drained (hm)
L_W	length of channel wetted (hm)
L_{DW}	length of channel wetted and drained (hm)
N	number of data samples
NS	non-SCARP
P	exponent defined by equation 14
Q	flow rate (lps)
Q_I	flow rate at sarkari khal outlet to the farmer's branch (lps)
Q_L	loss rate (lps/hm)
Q_{LO}	loss rate at osl (lps/hm)
Q_{L-A}	loss rate adjusted for flume effects (lps/hm)
Q_{L-M}	measured loss rates with flumes (lps/hm)
Q_{LSK}	initial sarkari khal loss rate (lps/hm)
Q_{LFB}	initial farmer's branch loss rate (lps/hm)
Q_{Li}	initial loss rate at the head of a channel section (lps/hm)
Q_M	watercourse inflow rate (lps)
Q_o	original watercourse flow rate (lps)
R	hydraulic radius (A/WP) (m)

S	channel slope (m/m or m/km)
Sc	SCARP
SK	sarkari khal channel
T	channel usage time (%)
TW	water surface top width (cm)
U	unimproved channel
WP	wetted perimeter length (cm)
Z	channel side slope at osl (horizontal to vertical)

LIST OF TERMS

bund	the dike or soil mound built around level basins to retain the water
farmer's branch	irrigation channel leading from the sarkari khal outlet to the individual fields
full supply level	the water surface elevation in a water channel when flowing normally under designed inflow rate and roughness conditions
mogha	the constant flow outlet structure from the irrigation canal into the watercourse
nucca	outlets from the main to branch watercourse channels or from channels to fields, usually composed of cuts through earthen banks
operational supply level	the observed water surface level in a watercourse channel which appeared to be flowing under normal operational conditions
sarkari khal	the primary portion of the watercourse conveyance channels originally laid out and constructed by the government on a right-of-way easement
SCARP	Salinity Control and Reclamation Project--refers to a watercourse served by a public tubewell
square	a nearly square land division containing about 10 ha
warabundi	the irrigation turn rotation system, usually based on land holding size and location on the watercourse, and often repeating weekly

Chapter 1

INTRODUCTION

Our world must continue to increase food production if the growing population is to be adequately fed. This necessity is especially acute in areas where the food supply is already critically short. In the past, a large part of the agricultural production increase has resulted from the development of basic resources--additional land has been brought under cultivation and more rivers have been dammed. The potential for further land and water resource development is dwindling, and the marginal costs of such development are soaring. The primary potential for increasing food production in the near future is to utilize available resources more efficiently.

In the arid and semiarid regions which comprise 50 percent of the world's arable land and is inhabited by 25-30 percent of the world's population (Biswas and Biswas, 1979), the most limiting agricultural input is usually water. In many arid regions the potential for development of significantly larger amounts of this resource no longer exists, or at least is very costly. If food production is to be increased, the existing water must be used more efficiently. More calories of food must be produced with the existing water supplies.

There are many facets of improved water use efficiency, from plant breeding to increase food output per unit of water uptake, to reducing losses from the irrigation system. This study is concerned with improving the efficiency of the tertiary conveyance system which carries the irrigation water from the canals to the fields.

In the past, especially in the technologically advanced societies, when it was realized that the efficiency of earthen channels was poor, the commonly adopted solution had been to line the channels with some type of impermeable material. Although the solution is a costly one, it was deemed the most practicable and could often be shown to have a benefit-cost ratio greater than one. Attempts to reduce the costs of saving water usually led to studies of lower cost linings rather than other alternative solutions.

For most of the lesser developed countries, a shortage of capital and a plentiful labor supply increase the desirability of alternatives other than channel lining to increase conveyance efficiencies. Even in capital rich areas, a lower cost alternative to channel lining, even if it only saves a portion of the water losses, is economically preferable in many situations, such as for temporary field ditches.

Although the costs and potential of channel lining have been considered in this project, the intent of this study was to develop alternative means of improving small irrigation channel conveyance efficiencies utilizing the most readily available and cheapest building material, soil. The emphasis was not on eliminating conveyance losses, but on developing simple design techniques, some of which involve no or little extra costs, which lead to a hydraulic system which conveys water more efficiently. Such design alternatives include system layout and operation, channel elevation, cross-sectional shape, maintenance methodologies, management, and bed and bank material preparation.

The nature of such practical techniques demands that they be adapted to a specific socioeconomic, agronomic, topographic, cultural, and historical situation; although this does not imply that many of the problem identification methodologies or resulting design techniques are not generalizable and adaptable to other environments. This study was conducted in the Indus Basin in Pakistan and the findings will apply primarily to the 78,000 small conveyance systems which irrigate 12 million hectares of land in that Basin. It is expected that many of the techniques will be applicable to many other irrigated areas, especially in lesser developed countries where channel lining is often too expensive an alternative.

Objective

The primary objective of this study is to develop techniques to convey irrigation water from the canal to the field in earthen channels more efficiently, in the context of the Indus Basin conditions.

Chapter 2

DESCRIPTION OF THE INDUS BASIN CONDITIONS

The Indus Basin Irrigation System

The Indus Basin is a large flat alluvial basin formed by sediments washed from the Himalayan mountains. The thickness of the alluvium in most areas is very great with the depth to bedrock usually being greater than 300 meters. The Indus River and its five principal tributaries originate in the mountain ranges where they gather monsoon rains and snowmelt and carry them across the plain to the Arabian sea. The three western rivers whose flow was allocated to Pakistan in the Indus Basin Treaty of 1960 (Michel, 1967) have a mean annual flow of about 171 billion cubic meters (Planning Division, GOP, 1977) (1 billion cubic meters ≈ 0.81 million acre-feet). The flow is highly seasonal with 75 percent coming in the months of June, July, August, and September. With the assistance of two storage reservoirs with capacities of 6.5 and 11.5 billion cubic meters, about 135 billion cubic meters is presently diverted into the irrigation canal system (Planning Division, GOP, 1977).

The present canal system was initiated by the British in the late 1800's. Significant further developments were added to the system in the 1960's as a result of the Indus Basin Treaty. Barrages divert the river flow into major canals which subdivide into successively smaller canals and eventually lead to distributaries or occasionally minors. From these small canals, outlets feed watercourses whose branches in turn lead to the individual fields. The canal outlet, or "mogha" is the traditional dividing line between the portion of the irrigation system controlled and maintained by the government and that portion operated by the farmers.

The heavy sediment load in the diverted water, the large areas commanded by individual canal systems, and the flat topography, demanded that the canal system be designed to flow at a nearly constant rate. Essentially the only flexibility is to turn the canals on or off. There are few return drains for the canals, so the water which is diverted must flow out through the moghas and onto the fields. These factors result in irrigation water being delivered to the watercourses at a relatively constant rate rather than in response to crop demand. The mogha has no gate. It is simply a device to equitably divide the water which is flowing in the canals between the various command areas.

The Indus Basin is underlain by an extensive groundwater aquifer. The water table was originally 6-30 m below the land surface in most areas, but with the advent of extensive canal irrigation and its accompanying seepage and deep percolation losses, the water table has risen in recent years to near the soil surface in lower lying areas. A result of this process has been waterlogging and salinization of some previously cultivatable lands.

In response to the rising water table and resulting land degradation, and the inherent inadequacy of the available canal water to irrigate the cultivatable land, a massive program of groundwater utilization through the use of wells (tubewells) was begun in the early 1960's in both the public and private sector.

The public "Salinity Control and Reclamation Project" (SCARP) utilizes 50 to 150 liter per second (lps) wells distributed over large land areas. The pumped water is usually added to the mogha inflow near the head of the water-courses to allow distribution of the pumped water to all farmers served by the watercourse. The combined water supply is usually about 250 percent of the canal supply. Mixing of the water supplies also allows dilution of the salts which are usually present in the tubewell water in greater quantities than in the canal water.

Private tubewells are installed by farmers or groups of farmers on or near their fields. Their capacities are smaller (usually less than 30 lps) and their water is seldom mixed with canal water. About 32 billion cubic meters of water is pumped from the groundwater each year, of which 60 percent is pumped from public wells (Lieftinck, 1969).

The response of the groundwater table to this extensive pumping has not been as large as expected, and in many areas, no dropping of water table levels has been recorded. This is indicative of the large amount of seepage and deep percolation losses from the irrigation system.

The primary limitation to further groundwater exploitation in many locations is the salinity levels in the groundwater. Thirty to forty percent of the cultivated basin is underlain with saline groundwater containing more than 2,000 parts per million of salts in the upper layers (less than 120 m depth) (Hussain and Ahmed, 1969). Deeper groundwater in most areas is highly saline, but most of the basin is underlain with at least a thin top layer of non-saline water.

The large groundwater aquifer provides potential for adding flexibility to an inflexible surface water delivery system if it is managed properly. Such a system would require sufficient annual recharge in nonsaline groundwater areas to allow pumpage during peak demand periods. The amount of deep percolation from most irrigated areas presently far exceeds this recharge requirement. However, an extensive irrigation efficiency improvement program, including canal and watercourse lining, could reduce groundwater recharge to below levels required for maintaining groundwater storage for peak demand pumping (Trout and Reuss, 1978).

The Watercourse

There are approximately 78,000 watercourses in the Indus Basin. Each watercourse serves an area of land which usually varies between 80 to 350 hectares and averages about 160 hectares. Water is allocated to the areas based on the crop water requirements of a desired cropping intensity. Many canal commands were originally designed to supply water to irrigate 75 percent of the land for one crop per year even though the climate allows two cropping seasons. The system was designed with water as a limiting factor. Mogha inflows generally vary from 20 to 80 liters per second (lps), or about 1 lps per 5 hectares of commanded cultivated area (i.e., 1 cubic foot per second per 350 acres).

On the majority of the commanded land, which had not been irrigated before building the canals, the watercourses were originally laid out and built at the time of the canal construction. The guidelines followed are given in the Canal and Drainage Act of 1873 (Jahania, 1973). The land was generally divided into approximately 10 hectare (25 acre) "squares" (about 330 meters on a side) which were to be farmed by one owner. Primary watercourse channels, called "sarkari khal" (official channel), were laid out usually on square boundaries or along diagonals to reach the highest corner of each square from the mogha. The farmer was then to build his own branches from this authorized outlet or "nucca" from the sarkari khal to his individual fields. The sarkari khal was originally built by government employees. The government reserved the legal right to enforce maintenance of the sarkari khal by the farmers who use the water, but it has seldom utilized that right.

As time passed and land passed from fathers to sons, land was subdivided and transferred such that many farmers no longer had direct access to a sarkari khal outlet. They have the option of petitioning the irrigation department to authorize an addition to the sarkari khal channel through another farmer's land to their fields, or to request the neighboring farmer to allow them to use his branches to reach their land. Both cases are found, but the latter is more common.

Many farmers have violated the general rule of one nucca per holding and have cut outlets from the sarkari khal in many places besides the authorized outlets.

Figure 1 shows a typical watercourse layout. Commonly, a 160 hectare watercourse command area will be served by about 4,000 meters (m) of sarkari khal and 18,000 m of farmer's branches, or about 140 meters of water channels per hectare irrigated. This large amount of channel is the result of both small holding sizes and small field sizes. Although there is much more total length of farmers' branches than sarkari khal on a watercourse, the average farmer will use about 80 percent sarkari khal and 20 percent farmers' branch to get the water to his fields.

Five meter right-of-ways were usually designated in the original watercourse layouts for construction and maintenance of the sarkari khal sections. Actual uncultivated widths consumed in the sarkari khals depend upon channel inflow rate, but watercourse averages tend to range between 2.2 and 2.6 m. Average uncultivated widths associated with farmers' branch channels vary from 1.7 to 2.2 m. This implies that, on a 160 hectare watercourse command area, 4 1/2 hectares, or about 3 percent of the land, is utilized for water conveyance channels.

As stated previously, water flows through the mogha constantly when the canal is in operation. The constant supply is allocated to the cultivators on a watercourse by

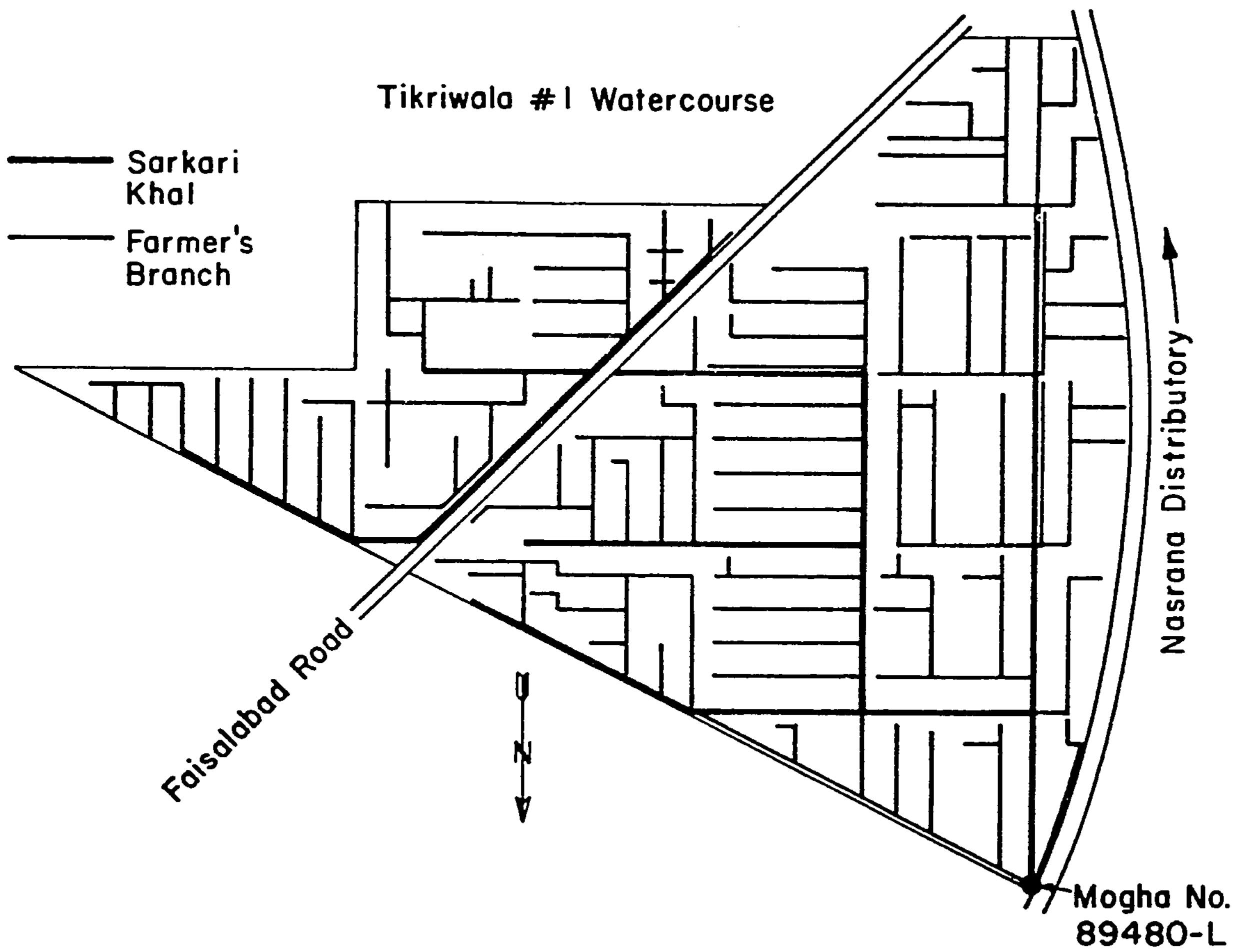


Figure 1. Layout of a sample watercourse.

a turn rotation system, called a "warabundi." Each cultivator's weekly turn time is based on the percentage of the total commanded land which he owns, with minor adjustments for filling and draining lengths of the sarkari khal. The water turn rotates from farmer to farmer beginning at the head near the mogha and moving progressively down each branch and from branch to branch until the turn of the last authorized outlet at the tail is finished. The rotation usually requires one week (although 10-day and 2-week rotations are also found). The water flows through each section of the sarkari khal each week. All of the sarkari khal and about 40 to 60 percent of the total length of farmers' branches are used each week in irrigating about 20 percent of the commanded land.

Although a weekly turn rotation requires that most of the channels are used each week, the regular movement of water progressively down the channels minimizes channel wetting within the rotation. Changing the rotation through the trading of water, which is common, increases the length of channels wetted and drained and causes additional water loss. Allowing the water to be utilized purely on a demand basis would require the filling and draining of much more length of channel each week and reduce delivery efficiencies.

The number of cultivators on a watercourse varies widely, but averages about 40, giving an average holding size of four hectares. The cultivator divides his land into small rectangular fractional hectare plots of usually 0.1 to 0.2 hectares in size. This allows more even distribution of water over the slightly irregular land surface irrigated almost exclusively in level basins, allows farmers with a short turn time to complete the irrigation of a portion of their small plots, and enables small farmers to raise a variety of crops. It also results in the large number of farmers' branches required to reach each small plot.

The climate is such that crops are grown year round. The year is roughly divided into a summer and a winter cropping season. Some nonperennial canal commands receive water only during the summer season when river flows are higher. All canal systems are closed during about a month of the winter season, when the consumptive use requirements are lowest, for repair and maintenance.

Farmers generally live in villages surrounded by their fields. Although labor is in short supply during critical planting and harvest periods, there are also periods when labor is plentiful. Manual labor receives \$0.10 to \$0.15 per hour (locally equal in cost to about 0.004 m^3 of concrete or five bricks). Farmers with larger holdings commonly hire both permanent and temporary labor; small farmers occasionally hire during planting and harvesting. Capital is scarce among the smaller farmers. Many bills are paid in kind. Bullocks are the predominant source of power, although farmers with larger holdings (> 10 hectares) often own a tractor, which they also rent to farmers with smaller holdings.

Cooperation among farmers is primarily within family groups and secondarily within "biradari" (brotherhood) groups. Cooperative activities on a watercourse or village level is usually a difficult undertaking. A common form of cooperative action witnessed is the cleaning and maintenance of the sarkari khal. When there is a problem with or lack of upkeep of the watercourse channels, the lack of cooperation and difficulty in organizing cooperative activities is the reason commonly given by farmers.

Watercourse Conveyance Losses

Measurements were initiated in 1973 by the personnel and cooperators of the Water Management Research Project in Pakistan (a U.S. Agency for International Development funded project carried out by personnel from Colorado State University) to determine the magnitude of watercourse conveyance losses. To date, thousands of inflow-outflow loss measurements with Cutthroat flumes (Skogerboe et al., 1973) and ponding loss measurements have been made on over 200 watercourse systems. Although measured watercourse conveyance losses have varied widely, the overwhelming conclusion is that 30 to 50 percent of the water which enters most watercourses at the head does not reach the fields. In an agricultural system where water is usually the most limiting input to additional crop output, this inefficiency is critical. It is also exacerbating the formerly mentioned waterlogging and salinity problems caused by excessive groundwater recharge and the resulting rise in water table levels.

Pakistan has put a high priority on providing more irrigation water for the farmer. This has usually involved the development of additional water resources through dams and water storage construction. This increasingly expensive program is being reevaluated in the light of the high watercourse conveyance loss findings. Reducing watercourse losses by five percentage points would provide the same amount of additional water to the field as the construction of an 11 billion cubic meter storage reservoir.

In-Depth Description of Watercourse Channels

Figure 2 shows a typical watercourse channel cross section. Wetted perimeter and bank shapes and elevations relative to the surrounding fields vary widely but can usually be described within certain bounds.

The wetted perimeter shape varies from circular segments to trapezoids with as steep as 0.5:1 (horizontal to vertical) side slopes. The shapes can usually be approximated by a power curve, such as is shown in Figure 3, with:

$$d = w(TW/2)^u \, , \tag{1}$$

where:

d = depth (m),

TW = water top width (m), and

w, u = empirical constants.

The coefficient w will generally be smaller in larger channels.

The wetted perimeter length of a watercourse cross section can generally be related to the flow rate, Q (lps), the roughness coefficient, n, and the channel slope, s (m/m) as shown in Eq. 2.

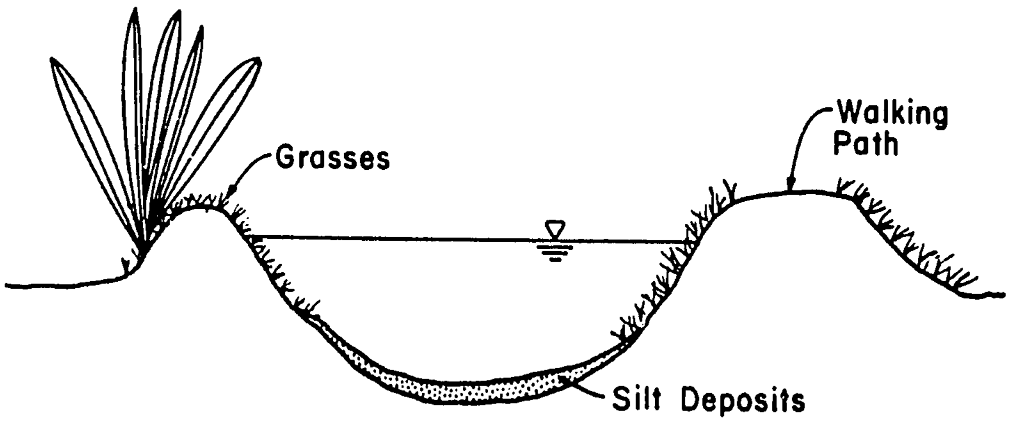


Figure 2. Cross-sectional view of a typical watercourse channel.

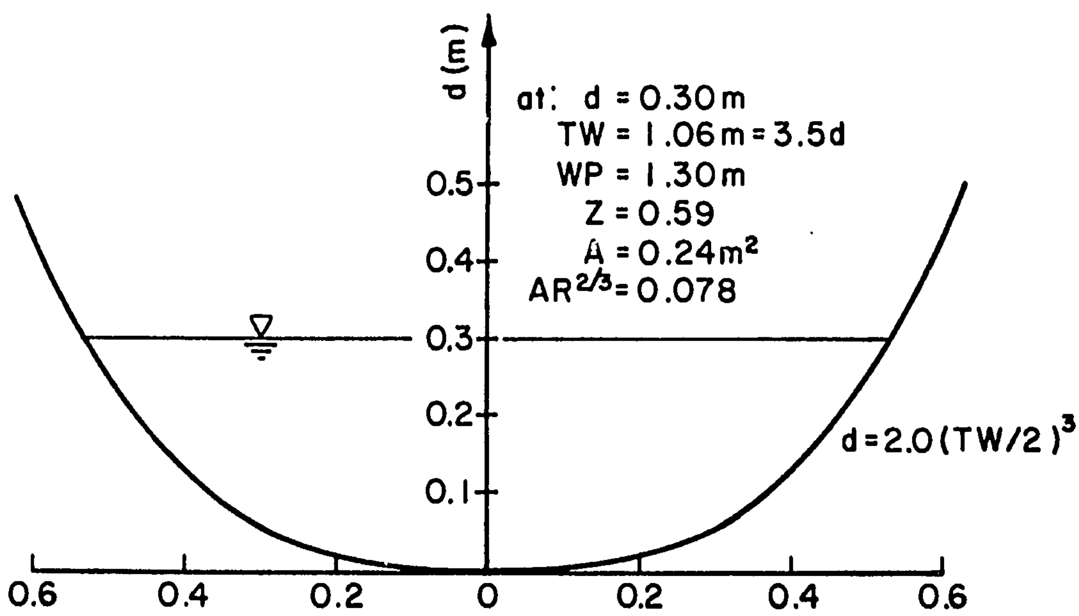


Figure 3. Watercourse cross section described by Equation 1 with w = 2.0 and u = 3.0.

$$WP = 3.9 \left(\frac{Qn}{\sqrt{S}}\right)^{0.4} \tag{2}$$

The coefficients given in the equation are those which provided the best fit for data collected on 10 watercourses. The coefficient of determination (r^2) of the relationship is 0.75.

The ratio of the top width to the depth usually is in the range of three to four. In 10 sample channels, this top width:depth ratio averaged 3.4 with a standard deviation of 1.1. The inner bank side slope near the full supply level varies widely, but tends to average about 1.0. In 50 channels where ponding losses were measured, the side slope, z (horizontal to vertical) averaged 1.0 with a standard deviation of 0.7. The wetted perimeter shape shown in Figure 3 has a top width:depth ratio of 3.5 and a side slope of Z = 0.6 at a depth of 0.30 m when its hydraulic section ($AR^{2/3}$, where A = cross-sectional flow area (m^2), and R = hydraulic radius (m) (this is equivalent to $Qn/\sqrt{S}$ in Eq. 2)) is equal to 0.078.

Bank shapes and widths are highly variable. Bank widths are often thick in the sarkari khal near the mogha where deposited silt has been placed during watercourse cleaning, and are often very thin in farmer's branches where farmers trim the banks to a minimum thickness while attempting to expand the sizes of their fields. In 50 channel sections, the bank width at the water surface level average and standard deviation values were 0.75 and 0.29 m respectively, while in just farmers' branches, the average bank width at full supply level (fsl) was 0.63 m.

Bank freeboard heights above the water surface level vary widely. In some channels, especially farmers' branches, the freeboard is so small that overtopping occurs when higher fields are being irrigated or when channel vegetation is heavy. Channels with less than 6 cm freeboard are not uncommon. Vegetative growth on the wetted perimeter and consequent changes in roughness coefficient and flow depth have a large effect on the freeboard, especially in small gradient watercourses.

Watercourse channels are usually built into the ground with the water surface just high enough to serve the downstream fields with between 3 and 10 cm of head. This, of course, requires elevated channels if downstream fields are at relatively higher elevations, and allows low sections if downstream fields are low. In the same 50 channels previously mentioned, the observed water surface level averaged 15 cm above the immediately surrounding fields, with a 15 cm standard deviation of the data.

The depth of water in flowing channels is commonly 0.2 to 0.4 meters, which means that there is often a portion of the water which will not drain into the last field irrigated. This dead storage water averaged 0.09 m^3 per meter of channel drained on four watercourses measured.

The topography of the Indus Plain is very flat. The average surface slope from Lahore, near the head of the plain, to the sea, a distance of about 1100 kilometers, is about 0.2 m/km (or 0.0002 m/m). Local topography is more variable, and watercourse water surface slopes commonly range from 0.15 m/km to 1.5 m/km. The mean water surface slope from 61 sample watercourse sections was 0.6 m/km. Experience with watercourse design has lead the author to consider watercourse slopes less than 0.4 m/km to be relatively flat, from 0.4 to 0.8 m/km to be moderate, and greater than 0.8 m/km to be relatively steep, in the context of the Indus Basin.

With small slopes, the flowing water velocities also tend to be small, usually in the range of 0.1 to 0.3 m/sec, and cross-sectional areas tend to be large for the amount of flow carried.

Watercourses are generally neither straight nor have regular cross sections. Most watercourses were originally laid out along surveyed land boundaries and were consequently quite straight. Over time, with cleaning and maintenance activities and gradual field boundary adjustments, often in favor of the more powerful neighbor, the channels have evolved to a more meandering alignment.

Watercourses generally do not have uniform cross sections. Wider sections tend to form at major junctions where soil has been continually borrowed to make earthen dams, then partially eroded away during dam opening. Near villages and other locations where there is continual human, animal, and cart traffic crossing the watercourse and no culvert has been installed, and in sections where water buffalo go to drink near where they are kept, the channel widening evolves into small ponds.

Outlets from watercourses (nuccas) are normally made by cutting holes in the channel banks. One or two nuccas will be cut for each field irrigated. Fields are usually about 15 to 30 meters wide, so one or both banks of farmers' branches often have 3 to 12 points where nucca cuts are regularly made per 100 meters length. Lowdermilk et al. (1978) measured an average of 771 outlet cuts per watercourse, or about one outlet per 20 m of channel. On most watercourses, farmers also cut nuccas directly into sarkari khal channels to irrigate their adjoining fields although such outlets are illegal. Refilled nucca cuts are usually thinner and have less freeboard than the rest of the banks, and more than half of the visible leakage water which goes completely through the banks, passes through at old outlet cuts. This leakage is a major problem in heavier soils that tend to crack upon drying. Washouts of watercourse banks, a problem especially in sandy soils, also occur almost exclusively at poorly blocked outlet cuts.

Field outlets are usually utilized less than 10 times a year, but at junctions, where water is also controlled by earthen dams, the outlets are used every week. Part of the soil utilized to build the check dams at junctions is washed downstream each time a check is built. This leads, over time, to a deficiency of soil in the area. As a consequence, banks get thinner and weaker and freeboards smaller in junction areas. As stated, enlarged sections also result from continually borrowing wet soil from inside the channel. Watercourse junctions are usually a problem area where water loss is higher than in other portions of the channels.

Indus Basin rivers emerge into the plains with heavy sediment loads washed from the Himalayan mountains and foothills. The sediment load diverted into canals, although small from October through February, increases to around 2000 parts per million (ppm) during the rest of the year and reaches peaks up to 10,000 ppm following periods of heavy precipitation (Mahmood, 1973). Figure 2 (p. 8) in Mahmood (1973) indicates that about 10 to 20 percent of the sediment diverted into one Indus Basin canal was bed load (dia. > 0.062 mm). Sediment loads have been reduced recently by the construction of water storage reservoirs.

The major canals, designed by regime theory concepts, pass most of the sediment on to watercourses through the moghas. The smaller capacity watercourses usually cannot carry the heavy sediment load and deposition occurs, especially in the initial reaches. During the rainy season, watercourses have been observed to aggrade 5 to 10 cm per month in the initial reaches. This aggradation will raise the channel bed, which in turn eventually raises the water level sufficient to submerge the mogha and reduce the inflow to many watercourses. The cultivators must then manually clean the sediment from the bed and throw it on the banks.

After several years of such cleaning activity, the banks become large walls of sediment spilling over onto the adjoining fields. The sediment piles often are covered with dense vegetation and make cleaning activities very difficult.

Finer sediments are carried further down the watercourse where a large portion pass out of nuccas onto the fields. Any sediment remaining in the stagnant dead storage water will be deposited on the watercourse beds and banks. Most watercourse channels have sediment covered beds. The beds in most major channels are underlain by a 5 to 15 cm thick layer of dense, predominately silty soils, thought to derive from sediment deposits. Because of the steepness and higher elevation of the upper banks, less sediment is deposited on the upper bank sides.

The soils in the Indus Plain, and consequently those from which watercourses were originally constructed, are alluvial deposits and vary widely from location to location. The majority are medium to heavy textured loams. The infiltration rates into these soils are usually less than 1 cm/hr and sometimes as low as 0.25 cm/hr. The low infiltration rates are largely a result of poor soil structure due to low organic matter content (most organic matter is generally removed from the fields for use as fuel or fodder) and poor tillage practices.

Bank soils have evolved over time to include both sediment and organic matter from vegetation cleaned from the channel and from decaying roots. They are perhaps the most organic soils in the Plains. The decaying organic matter also creates higher porosity, although banks which are used as foot paths, as are perhaps 20 percent of all sarkari khal banks, are more compacted. Since nearly all of the Indus Plains are frost free, there is no frost action to loosen soils.

Trees, bushes, and grasses grow profusely on watercourse banks. The trees are normally planted by the farmers as encouraged by the Forest Deparment. They are used as wood for constructing small equipment, for fuel, and for shade during the hot summers. Most of the trees found in cultivated regions are located along watercourse banks. The density of trees ranges from a nearly constant wall to an occasional tree every 30 to 100 meters.

Near watercourse heads, where silt piles are large, extensive phreatophytic bushes give watercourses a "jungly" look. They make monitoring and maintenance of the watercourse especially difficult.

Other areas are generally covered with extensive short grasses which grow profusely in the tropical climate. The coarse grass seldom grows taller than 15 cm, but propagates quickly from runners. In often used channels, the grass extends down to, and often into, the water surface. In seldom used channels the complete wetted perimeter is grass covered. Often only the banks are grass covered since dead storage water lays in the bed after the banks are dry.

Anchored, floating aquatic weeds grow quickly in channels which are usually full or contain dead storage water during the winter months when most of the storage derived irrigation water has low sediment loads and is clearer.

Manning's roughness coefficient, n, was measured on 16 watercourse channels in various conditions of vegetative growth and irregularity. Table 1 summarizes the results of those measurements. The study concluded that the roughness values derived for Indus Basin watercourses tended to be somewhat higher than those given in the Western literature for canals under similar conditions. It was proposed that the larger values were the result of the small channel sizes and the nature of the tropical vegetation.

Table 1. Measured values of Manning's roughness coefficient for Indus Basin watercourses in various conditions.

Earthen Channel Condition	Measured Manning's "n" Values
newly built, uniform, clean	.017-.032
older, winding, with no vegetation	.030-.035
uniform with short grasses	.026
winding with moderate grasses and weeds	.035-.055
with dense grasses and aquatic weeds	.042-.18

The measurements indicate that clean, uniform, straight channels have n values of about 0.005 less than irregular clean sections, that moderate grasses and weeds can increase n values by 0.01 to 0.02, and that extensive grasses and aquatic weeds can increase roughness coefficients in watercourses to well over 0.10. These results indicate the extreme variability in watercourse roughness coefficients and the large potential effects on flow depths of removing vegetation.

Cleaning is usually accomplished by scraping away the grasses and 1 to 2 cm of soil from the watercourse inner banks and removing the excess silt from the bed manually with a "kussie," which is a shovel bent like a hoe. Shaving the banks removes the surface silt layer and often exposes more porous soils underneath, but the grass roots usually remain and the grasses grow back quickly. Cleaning often also tends to make channel cross sections wider and shallower and sometimes leads to steeper side slopes, although the fact that this widening does not progress indefinitely implies that silt also accumulates in the grasses on the banks. Removed silt and vegetation is piled on the banks.

The frequency of cleaning varies widely from watercourse to watercourse and in different sections of a watercourse, but is seldom less than once a year or more than once a month. Head sections of the sarkari khal, especially if silt deposition is a problem, are cleaned most regularly. About six man-hours are required to clean the grasses from 100 m of watercourse channel.

Examples of watercourse cross-sectional shapes and vegetation are shown in Figures 4, 5 and 6. Figure 4 shows an irregular and winding section near the head with dense vegetation and piles of excavated silt. Figure 5 illustrates the degree of vegetative growth on the banks of some watercourse sections. Figure 6 shows a channel section widened into a pond by continual human and animal traffic.

Inflow through the mogha is usually fairly constant over time. Fluctuations result from changing water levels in the canals or from submergence of the mogha because of vegetation, silt deposition, or irrigating excessively high fields downstream. Assuming no submergence, inflows seldom vary from an average by more than 20 percent. About one third of the watercourses are nonperennial, meaning they receive canal water only for the summer season, from May through October. Most canals are closed for about one month during January and February when canal maintenance is undertaken. Canals are also occasionally closed during the monsoon season for a few days following heavy rains when there are no crop water needs, and water and sediment levels in the rivers are very high.

About one-fifth of the watercourses are presently supplemented with water from Salinity Control and Reclamation Project (SCARP) tubewells. This public tubewell water is mixed with the canal water near the mogha. The pumps run on a regular daily schedule and are commonly turned off for several hours during peak electricity use periods. They are

Figure 4. Watercourse section near the head with excavated silt piles and dense vegetation

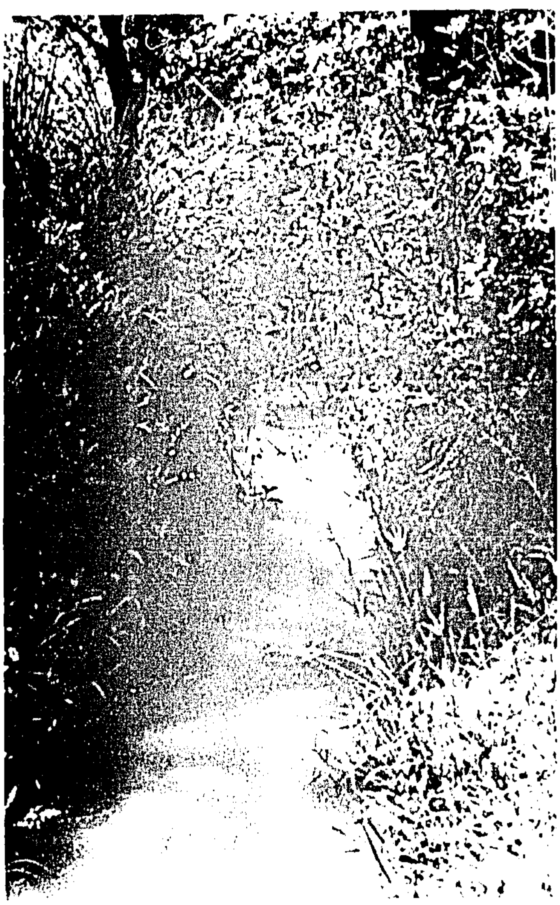


Figure 5. Dense vegetative growth on a watercourse section.

Figure 6. A channel section widened by animal and human traffic.

also turned off because of mechanical or electrical failures or by farmer request. SCARP watercourse flows consequently fluctuate depending on the tubewell scheduling. SCARP watercourses have less sediment deposition because of the dilution of the sediment laden canal water and larger flows which provide higher sediment carrying capacity.

Private tubewell water is normally not mixed with canal water, but is conveyed separately to the fields. On private tubewell supplemented watercourses, the channels are used more often and intermittently by clear and sediment carrying water.

The initial reaches of the sarkari khal are used to convey water most of the time. As water is rotated through the branches, channels farther from the mogha are used progressively less and less. The tail ends of farmers' branches are used only a few hours each year. On five studied watercourses, sarkari khal sections were filled an average of 36 percent of the time, while farmer's branch channels were filled only about 2 percent of the time. While nearly all sarkari khal sections are filled and drained each week, only about half of the farmer's branch channels are used during a weekly rotation period. Of the total length of channel utilized to convey water to the fields, 80 percent is sarkari khal, on the average.

Besides vegetation, there are insects, snakes, and rodents who also inhabit watercourse banks. Ants, mole crickets, worms, and termites; snakes; and primarily two species of rats, the nesokia and bandicoot, are commonly encountered when digging in Indus Basin watercourse banks.

There are several reasons why these inhabitants live in channel banks. Ditch banks provide a soil medium which is undisturbed for long periods of time, unlike the regularly cultivated fields. In a level basin irrigated area, the banks constitute the major location of sufficient soil mass which is permanently above water. Bank soils probably contain the largest amount of decaying organic matter, and support the lushest vegetation (a source of both food and cover) of any soils on the Indus Plain. The moisture derived from the watercourse flow not only supports the vegetation but also keeps the soil moist for easy digging. This combination of factors make watercourse banks an ideal habitat for worms, insects, snakes, and rodents.

There creatures burrow in the banks to construct homes and search for food. Shaving away watercourse banks exposes an inner bank honeycombed with passages varying in size from 0.3 to 10 cm in diameter. Figures 7 and 8 show examples of insect and rodent holes in watercourse banks. Piles of soil left inside watercourses by burrowing rats indicate the porosity of many banks. Occasionally, the burrows will be near enough to the surface that the bank top will collapse under the weight of pedestrian traffic.

The most commonly encountered insects are ants. A cut with a shovel into a bank will often produce many ants scurrying for cover. Further excavation will reveal a complex

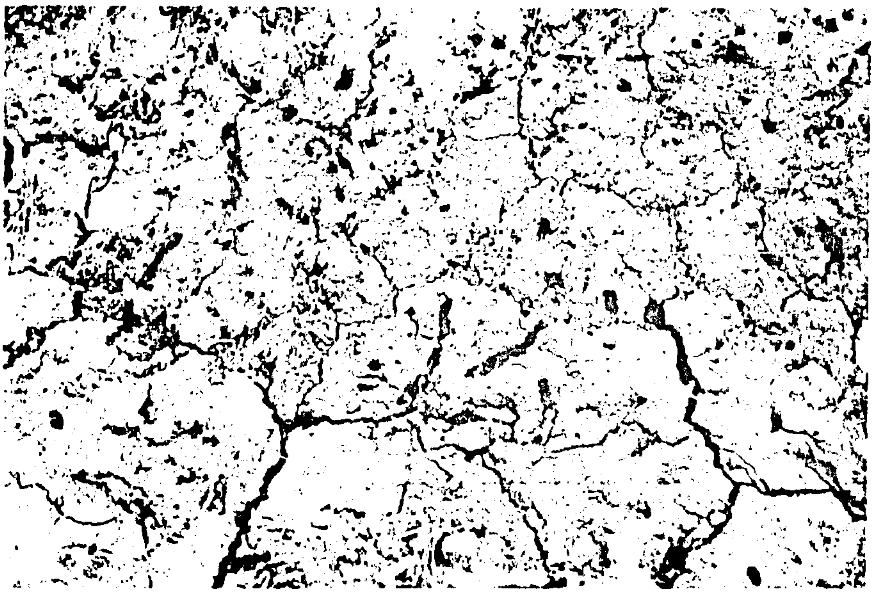


Figure 7. Insect holes in a watercourse bank exposed by scraping away the inner bank surface.

Figure 8. Rat burrows inside a watercourse bank exposed by excavating the inner portion of the bank.

myriad of tunnels, often as large as 1 cm diameter and larger caverns where eggs are kept. Bank cutting studies have found a density of ant holes greater than one per 100 cm^2 in various planes and locations in the banks. Most tend to concentrate in the upper and outside portions. They seldom extend down below the land surface. That ants concentrate in the upper bank is evidenced by the hundreds of ants sometimes flooded out of their holes when the water level in a channel is dammed up higher than the normal flow level. The ant holes often interconnect with rat burrows.

Snakes are observed less often than rats in watercourse banks. Several snakes are usually unearthed in the process of destroying old banks and rebuilding a watercourse. They are promptly killed by farmers who believe all snakes are dangerous.

The two types of rats found in watercourse banks have very different life styles. The most common bank rat is the nesokia (nesokia indica) or Indian Mole Rat, often referred to as the blind rat. It lives and feeds almost entirely underground. Nesokia feed on the succulent roots of grasses which grow on watercourse banks and occasionally burrow out under fields to feed on sugarcane, wheat, and rice roots. It burrows primarily to feed and is believed to cause little crop damage.

The bandicota bengalensis lives primarily on and under fields and surface feeds on grains and sugarcane. It will occasionally store quantities of grain in its underground burrows, which extend up to 1 meter below the field surface. The bandicoot burrows also lead to watercourse banks where they stay when the field is covered with water or the crop has been harvested and their cover is gone. In one 30 m long section of watercourse bank, adjoining a recently irrigated sugarcane field, 12 bandicoots were unearthed. Being surface feeders, they usually leave their holes open and surface evidence of their activity is obvious in harvested fields.

The bandicoot reproduces quickly and populations fluctuate greatly through cropping seasons, while nesokia populations tend to be more constant. Both types live solitarily except during breeding, and each rat maintains its own burrow system which sometimes extends more than 12 meters in length.

Primary rat burrows in watercourse banks extend longitudinally in the upper bank, usually near or above the normal operational supply level. Branches extend from the main burrow for feeding on roots or pushing out dirt to the inside or outside bank surface, and down and out under the fields for reaching field crops.

A flooded burrow is, of course, of no use to the rat, so the burrows are normally sealed towards the inside surface of the bank and are often dug above the normal operating level. But, leaks do occur, especially in abandoned burrows and from interconnecting ant holes, especially when the water level in the channel is higher than usual. Such breaks sometimes allow water to leak directly through the banks, and sometimes into the banks and under the field where it bubbles up to the field surface. But most commonly, the leaking is not visible on the surface although it can continue to enter the holes for long periods of time. It is assumed that the interconnected burrow systems are sufficiently large that their effective wetted perimeter can absorb large quantities of water.

Although the theory of flow in porous media is often used to better understand seepage from large canals, it is evident from this description of watercourses that leakage from these small channels cannot be completely described in the same terms. Silt deposition varies greatly with location along the wetted perimeter and distance from the source, the channels are regularly wetted and dried, vegetation is constantly changing, and the banks are riddled with "macropores" (rodent, snake, and insect holes) through which flow does not follow Darcy's Law. The importance of these macropores is indicated by the excessive infiltration rates measured into watercourse banks.

Though it is not possible to completely describe watercourse leakage by theoretical considerations, empirical studies can point out factors which are related to higher or lower loss rates. The factors which directly affect losses, such as sediment deposition and macropores, often are difficult to measure or quantify in a field study, so secondary parameters which affect these factors, are quantifiable, and are related to viable design alternatives, are measured instead. This discussion has indicated some of the measurable parameters, which include:

1. the percentage of time a channel is used, which would influence microbiological activity, silt deposition, vegetative growth, and insect and rodent activity in the lower banks,
2. the elevation of the watercourse relative to the surrounding fields, which would affect the head exerted on flow through macropores, the soil mass available for rodents and insects, and dead storage volumes,
3. the bank widths, which would influence the chances of leakage passing completely through the banks and the soil mass available to insects and rodents for burrowing,
4. side slope of the inner bank which would influence both the silt deposition on the upper banks and the fraction of the wetted perimeter which is on the banks,
5. distance of the section from the mogha, which will influence sediment deposition and will be correlated with usage time,
6. the depth of flow in the section relative to the usual flow depth, which will indicate the relative seepage rate into the upper banks compared to the bed, which in turn relates to macropore and sediment placement,

7. whether the channel has been recently cleaned or is covered with vegetation, which will influence both the relative flow depth and the surface sediment layer, especially on the banks,
8. whether the channel is aged or newly built, which should influence the amount of macropores in the banks and the amount of sediment and microbiological sealing on the wetted perimeter,
9. whether the section is sarkari khal or farmer's branch, which will influence the regularity of the channel usage and the occurrence of cleaning activities,
10. whether the channel carries canal, public tubewell and/or private tubewell water, which will influence regularity of use and sediment deposition,
11. the normal flow rate in the watercourse channel, which will indicate how losses vary with flow rate and channel size, and
12. the layout and temporal operation of the watercourse system, which will influence the continuity of use of the channels and the length of channels which must be filled and drained in the process of irrigating the fields.

Chapter 3

REVIEW OF PAST WORK

In the second quarter of this century, irrigation engineers in many countries began to realize that water would be a limiting input to irrigated agriculture in many areas and that more efficient utilization of the available water supply was important. It was also recognized by many researchers that losses in the conveyance system were significant, although actual estimates were wide ranging. Consequently, studies were undertaken to better understand canal seepage and to formulate solutions to the problem. Although some studies were aimed at reducing seepage losses from earthen canals, most were oriented towards evaluating the benefits of various types of canal linings. Nearly all dealt only with large canals - those with design capacities larger than 3 m^3/sec. Most studies also assumed that the only significant losses were those resulting from percolation through the interstices of the wetted perimeter (seepage).

Some of the earliest comprehensive work undertaken in California, and later Colorado, was reported by Rohwer and Stout (1948). Their studies over several years of loss measurement techniques, canal losses, and infiltration from seepage rings lead them to propose several factors which affect seepage rates, including (Robinson and Rohwer (1957)):

1. soil type and permeability of the bed material,
2. depth of water in the canal,
3. length of time the canal has been in operation,

4. depth of the groundwater table,
5. temperature of the soil and water,
6. the volume of silt in the water,
7. the salt content of the soil and water,
8. the percentage of entrapped air in the soil, and
9. the soil moisture tension.

Although it was recognized by all researchers that the bed and bank soils affected seepage rates, it was also reported that within soil types, there was extreme variability and soil type alone could not be used to predict seepage rates (Rohwer and Stout, 1948, and Worstell, 1976).

Silt deposits from conveyed water have been recognized to decrease seepage rates. Papfalvy (1969) reported the seepage from a large canal in the USSR decreased over time to very low rates due to siltation, and that when the normal flow depth was increased only 20 cm into a bank region with no silt accumulation, the seepage rate initially drastically increased, then decreased rapidly with time. His calculations determined that the ratio of the silt deposit permeability to that of the underlying soils was about 1:400. Kraaty (1977) reported that seepage losses from some canals in India and France decreased 20 percent and 80 percent respectively over time due to silt accumulation. Brockway (1973) claimed that the predominant cause of long-term decreases in seepage rates of canals in southern Idaho was sediment buildup on the canal bottom.

Dadayev (1957) reported that compaction of the in-place bed and bank soils can drastically reduce seepage losses.

He describes both tamping and rolling compactors which increase the soil bulk densities by 12 to 25 percent and reduced seepage rates to less than 5 percent of their initial value. A study over time revealed that the low seepage rates did rebound slightly over a two-year period, but remained a small fraction of the pre-compacted value.

Several seepage meter studies in large canals to determine the relative seepage rates of bed and bank materials led to inconclusive and conflicting results (Rohwer and Stout, 1948, Robinson and Rohwer, 1952, and Worstell, 1976). One ponding loss study, which found a direct curvilinear relationship between ponded depth and seepage rate, concluded that the upper bank materials must be more porous than bed materials (Robinson and Rohwer, 1957).

Hanson (1966) undertook many ponding loss measurements in small farmers' ditches in New Mexico and concluded that seepage rates through the upper ditch banks were much higher than through the channel bed. An analysis of Hanson's data by the author of this thesis indicated a curvilinear loss rate vs. depth relationship of:

$$Q_L = Q_{LO} e^{0.068 \Delta d} , \tag{3}$$

where:

Q_L = loss rate (liters per second per hectometer (100 meters),

Q_{LO} = loss rate at a datum flow depth where $\Delta d = 0$ (lps/hm),

e = base of the natural logrithm, and

Δd = depth change above or below a datum (cm).

For Hanson's data Q_{LO} varied between 1.0 and 2.1 lps/hm. He concluded from this data, and from data he collected which indicated that the Manning's roughness coefficient, n, for these earthen channels varied from 0.02 to 0.10, depending upon vegetation; that the amount of vegetation in a ditch had an extreme affect upon loss rates. His calculations indicated that a doubling of the roughness coefficient would result in an increase in the loss rate by three to four times.

Similar unpublished ponding loss data collected by Akram and Kemper in small irrigation channels in Colorado consistently exhibited the same curvilinear relationship between loss rate and depth of flow, also indicating that upper channel banks are much more pervious than the beds (personal communication from W. D. Kemper).

Seepage ring studies have consistently indicated a direct linear relationship between ponded depth and seepage rate, as would be predicted by Darcy's equation (Rohwer and Stout, 1948, Robinson and Rohwer, 1957, and Muckel, 1951). The relationship is not proportional, since a finite seepage rate is projected at zero depth. The slope of the relationship increases with increasing loss rates at a given depth. Data reported in Robinson and Rohwer (1952) indicated a 20 to 30 percent decrease in seepage rate with a 30 cm drop in depth of water in seepage rings.

Depth to the water table was also found to affect seepage rates in a limited study reported by Robinson and Rohwer (1957). It was concluded, as would be predicted from theory, that seepage rates increase with a dropping water table until hydraulic contact with the water table is broken. The level at which this contact is broken depends upon the permeability of the soil materials involved, but can be relatively shallow if soils with low permeabilities are present.

From the theory of flow in porous media, it is predicted that increasing temperature would increase seepage rates through a decrease of the viscosity of water. Studies reported by Robinson and Rohwer (1957) found the opposite was true. They explained the unexpected finding as result of a vapor pressure change and its affect on the size of entrapped air bubbles in the soil, which in turn affects the effective porosity and permeability.

Many researchers have recognized that the seepage rate from seepage rings, into canal wetted perimeters, and into water spreading sites, was strongly dependent upon the length of time water had been ponded on the surface. Allison (1947) found an S-shaped curve described the seepage rate over time. He reasoned that an initial seepage rate decrease was the result of soil dispersion and swelling, the following increase was the result of increasing dissolution of entrapped air particles, and the ensuing and continual decrease was the result of microbial activity. The average infiltration rates in his laboratory cylinders decreased 50 percent every 12-20 days after the peak rate had been reached. He supported his belief in the importance of microbial activity by testing sterile soil and water which did not exhibit the final decrease. Worstell (1976) found that Xylene treatments to remove moss from canals also tended to increase the seepage rates. Robinson and Rohwer (1952) also found drastically decreasing seepage rates over two seasons and explained the finding as microbial activity and the breaking down of soil particles. Brockway (1973) found that the herbicide treatments in a large canal increased the seepage rates, purportedly by its affect on microbiological activity. He attributes seasonal changes in canal seepage rates to microbiological activity in the impeding layer.

Muckel (1951), in a water spreading study, found the same results as Allison. He also found that short drying periods would allow the intake rates to rebound somewhat, but not to their initial values, and that plowing of the dried soil surface further increased seepage rates, although simply scratching the soil surface seemed to have no affect. He also found the decrease over time was not as great if the soil was covered with sod.

Worstell (1976) and Kraaty (1977) reported that seepage losses from seasonally used canals decrease as the season progresses.

Hanson (1966), in his study of farm ditch losses, found that leakage through closed turnouts were a major source of water loss. His ponding measurements indicated that regular earthen channels were losing an average of 4.3 percent of their flow per 100 m, while after securely sealing all turnouts, the mean loss was reduced to 2.3 percent/100 m. About

50 percent of the water loss was through leaking turnouts. Nearly all the leakage from tested concrete lined sections was from the turnouts.

Recent published work in canal seepage has been oriented primarily toward predicting and describing seepage from canals of various geometries into soils of various permeability configurations utilizing solutions to equations describing flow in porous media. These analytical and numerical techniques necessarily require relatively simple geometrical configurations and cannot be directly applied to the extremely heterogeneous watercourse channels with their macropores and spatially varied silt deposition patterns.

Summary of Recent Studies in Pakistan

In the past seven years, extensive studies of small irrigation channel water losses have been carried out in Pakistan by personnel and cooperators of the Water Management Research Project. The present study is under the auspices of the same project.

Initial studies to determine the problems in the on-farm water management system in Pakistan found that the water losses from the small farmer managed watercourses were much higher than previously supposed (Clyma et al., 1975, Ashraf et al., 1978). Subsequent efforts were aimed at determining the causes of the low conveyance efficiencies and formulating and testing solutions to the problems.

It was quickly realized that the losses from watercourses were much greater than could be explained by normal seepage into homogeneous bed and bank materials, since infiltration rates of the banks were much higher than infiltration rates into surrounding fields (Kemper et al., 1975 and Akram et al., 1976). Further study revealed that the silt covered beds had quite low seepage rates but water seeped and leaked into the upper banks at very high rates. Kemper et al. (1975) describes the watercourse banks as "the most populated (with insects and rodents) and most permeable soils in Pakistan." He reported that the intake capacity of such macropores in some sections was very high, and in one section, plugging the visible holes reduced losses by 75 percent.

Clyma, et al. (1975) undertook a survey of watercourse losses and found that losses are higher in larger watercourses and that losses are especially high in watercourses augmented by public tubewell water which were usually undersized for the augmented flow. Those channels visually assessed as "well maintained" had relatively low losses.

These findings led to the initiation of a watercourse renovation program where the old channels were destroyed and a new ditch constructed of proper size and design to carry the flow. Thus, the new channels, without the rodent and insect holes and of proper design and adequate capacity, were expected to reduce the avoidable losses - those beyond that expected from normal infiltration. Cheema et al. (1976) and Bowers et al. (1976) reported that losses were initially reduced by 20 to 50 percent in such rebuilt channels. Further unpublished data has indicated that such loss reduction is at least partially a temporary phenomena with the channels acquiring, over time, many of the characteristics which led to the previous high loss rates.

Additional studies were undertaken to better understand the factors affecting loss rates. Cheema et al. (1976) and Bowers et al. (1976) collected data which indicated that the loss rates in often used main channels were significantly lower than loss rates in occasionally utilized branches. They attributed this result to the hindrance to insect and rodent activity below the normal water level in channels usually full of water.

Bowers et al. (1976) successfully fitted loss rate vs. inflow rate data to the empirical formula:

$$\text{Loss} = a_1 e^{a_2 Q_M} \quad , \qquad (4)$$

where:

Loss = total water loss in a given section of channel (lps),

Q_M = inflow rate to the section (lps), and

a_1, a_2 = empirically derived constants.

Their data indicated that water loss was very sensitive to changes in inflow rates, and that this sensitivity was less in newly reconstructed watercourses (Bowers and Wahla, 1978).

Bowers also proposed that loss rate per unit distance was a linear function of the flow rate in the section:

$$\frac{dQ}{dD} = -K_1 Q \qquad (5)$$

where:

$\frac{dQ}{dD}$ = the change in flow rate per unit distance (termed "loss rate"),

Q = flow rate at any point, and

K_1 = an empirical constant with units of percent loss per unit length.

Integration of the above relationship produces an exponential relationship between percent flow remaining and distance:

$$Q/Q_M = e^{-K_1 D} \quad . \tag{6}$$

This differs from the traditional assumption that losses (assumed to result primarily from seepage) are a function of length of wetted perimeter (Rohwer and Stout, 1948), or the often utilized assumption that watercourse losses are a function only of length (resulting in constant loss rates per unit distance). Bowers data were not sufficient to indicate whether his assumption was better than the other mentioned assumptions. Bowers recognized the fact that flow rate as such, does not materially affect loss rates, but that incident factors, such as water flow depths, were the controlling variables.

Akram et al. (1976) reported that compaction of bank soils of reconstructed watercourses, or compaction of soil cores in the banks of existing watercourses, initially reduced loss rates to 20 to 35 percent of their previous values, although it was reported that a major portion of the reduction resulting from core compaction lasted only about a year.

A study reported in Akram et al. (1976) found that, during operation of a watercourse branch, up to 15 percent of the inflow is lost to operational conditions in addition to the steady state type of losses previously reported. A separate study summarized in the same report found that on one branch, 45 percent of the losses occurred in the areas of junctions where earthen dams were regularly built and broken and the channel banks were in a state of extreme deterioration.

Ahmed and Bowers (1978) reported that extensive cleaning and maintenance work on several watercourse branches led to significantly reduced water losses. Akram and Kemper (1978) conducted ponding loss studies before and after cleaning a channel section and concluded that the infiltration rates into cleaned channel banks and beds were often somewhat higher than before cleaning, but the decrease in operating level due to a reduction in vegetation and the roughness coefficient was sufficient in all cases to significantly reduce the loss rates.

Most of the data presented in this study has been previously reported by the author in Pakistan in the form of mimeographed reports of the Colorado State University Water Management Research Project, or as publications of the Water and Power Development Authority of Pakistan. It was felt that, because of the nature of the work and the dependence of other decisions on the findings, timely presentation was important. Data and analyses presented in this study which were thusly previously reported where this writer was first author, will not be referenced.

Chapter 4

METHODOLOGY

Watercourses were studied to determine the functional relationships between water losses and other measurable descriptive parameters such as those listed in Chapter 2. Once these relationships were determined, design choices which would reduce losses could be predicted.

Three types of field data collection methods were used: ponding loss measurements, steady state inflow-outflow flume measurements, and operational condition inflow-outflow flume measurements. The basic measurement methods are not new and all are discussed in the literature (Brockwell and Worstell, 1969, and Robinson and Rohwer, 1957), although some novel techniques are utilized.

Ponding Loss Measurements

Ponding loss measurements were used to accurately determine water loss rates at various water depths from short channel sections in which most conditions were constant. Measuring water channel losses by the ponding method involves filling a section of channel 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. To make the results comparable to other measurements from different lengths of sections and other types of measurements, the loss rate is expressed per unit of distance (liters per second per 100 meters (hectometer) or lps/hm).

The test sections were selected to be representative of existing watercourse channels. Often two or more adjoining sections were studied to determine the variability in the results. No changes were made to the natural state of the channel, such as sealing leaking insect holes or disturbing the vegetation.

The section lengths varied from 20 to 40 meters. Short lengths were studied so that the ponded water depth at each end of the section would not vary more than one centimeter from the operational flow depth due to the slope of the section.

Staff gauges were firmly inserted into the bottom of the channel for measuring the water depth changes. The gauges were normally installed in the channel before the study began, while water was flowing normally in the section, to determine the operational water surface level. Attempts were always made to determine the operational supply level (osl) as accurately as possible.

Once the channel had ceased being used, compacted earthen dams were built at the ends of the selected sections. The downstream dam was constructed first while the test section was being filled to 4 to 6 cm above the measured osl, then the section was closed at the upper end. The water level was usually maintained at the increased depth for a short time (1/2 to 1 hr) to saturate the banks, but this was not always possible because of water supply problems.

Five evenly spaced water surface width measurements were made in each section during the initial water surface level gauge readings and at least twice later during the water level recession. The average of the five readings was used as a measure of the water top width of the section at the water level indicated by the staff gauge.

Staff gauge and time readings were taken after each one-half centimeter of drop of the water surface. Readings were usually continued until the water level had dropped about 10 cm below the recorded osl. Occasionally, recessions were recorded until the channel was nearly empty.

Any visible leakage which passed through the bank and appeared outside the watercourse was noted and often volumetrically measured by catching the leaking water in a container for a measured period of time.

During or after the recession data collection, other parameters which describe the condition of the test section were measured. The elevation of the water surface at osl relative to the elevation of the surrounding field surfaces was measured with a surveyor's level. The width of each bank at the osl depth was estimated at five places with a measuring tape sighted across the bank top. A visual assessment was made of the condition of the channel wetted perimeter to determine whether it was clean or grassy, whether insect and rat holes were visible, and the general cross-sectional shape. An estimate or flume measurement was made of the normal flow rate in the section. The time during which water flowed in the channel section each week, and whether the channel was part of the sarkari khal or was a farmer's branch, was determined from a warabundi (irrigation turn rotation) schedule, or by questioning the farmers. The distance of the section from the mogha was estimated. Soil type was estimated by visual assessment on some of the sections. All data were recorded on a data collection sheet such as is shown in the Appendix.

Loss rates were determined from a graphical analysis of the water surface recession and top width data. Depth gauge readings were plotted versus time as is shown in Figure 9. The measured osl gauge reading is also marked on the graph and gauge readings are noted relative to the osl reading (Δd). The recession rate at any depth relative to the osl is determined by measuring the slope of the curve at that depth.

Average water surface widths were also plotted on the same graph versus gauge readings, with widths listed along the top of the graph (see Figure 9). The average width at any gauge reading was interpolated from the graph.

Multiplying the water surface recession rate times the interpolated average water surface width, with the correct unit conversions, results in a water loss rate per unit of length:

$$(Q_L)_{d_i} = \left(\frac{dd}{dt}\right)_{d_i} \times (TW_A)_{d_i} \times C \quad , \tag{7}$$

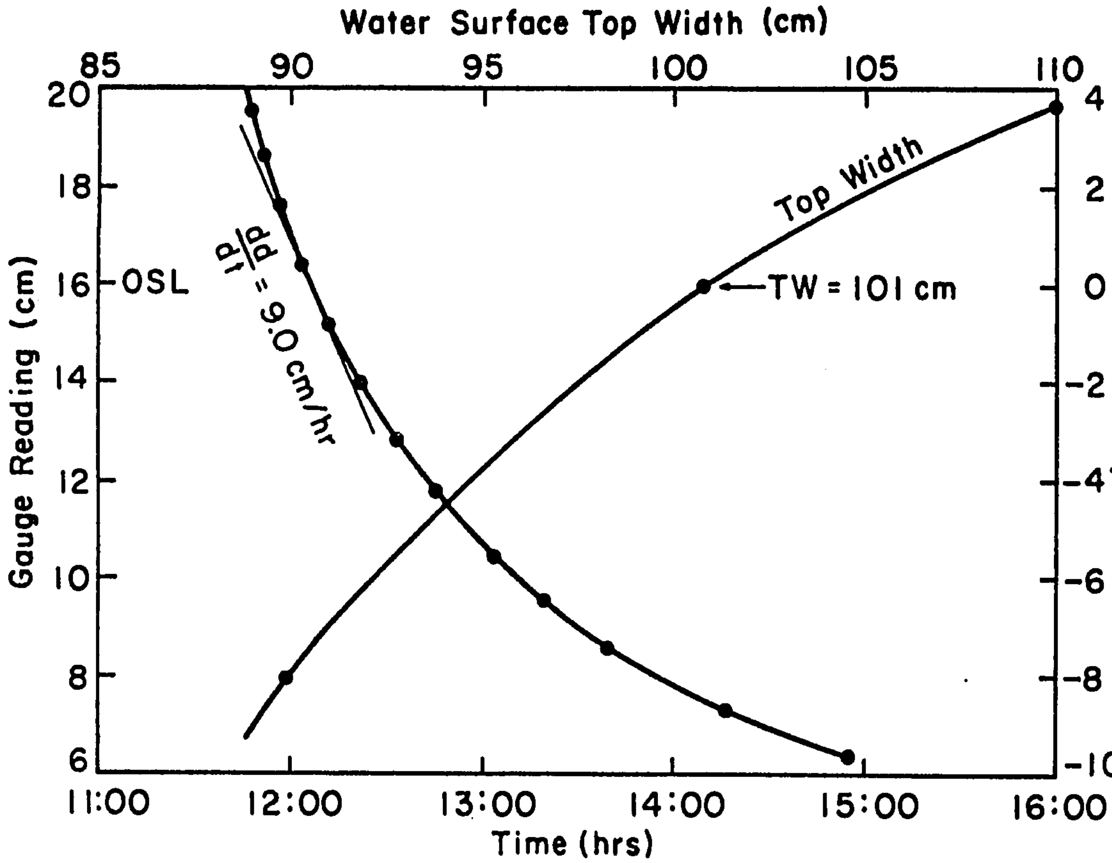


Figure 9. Gauge reading vs. time and water surface top width.

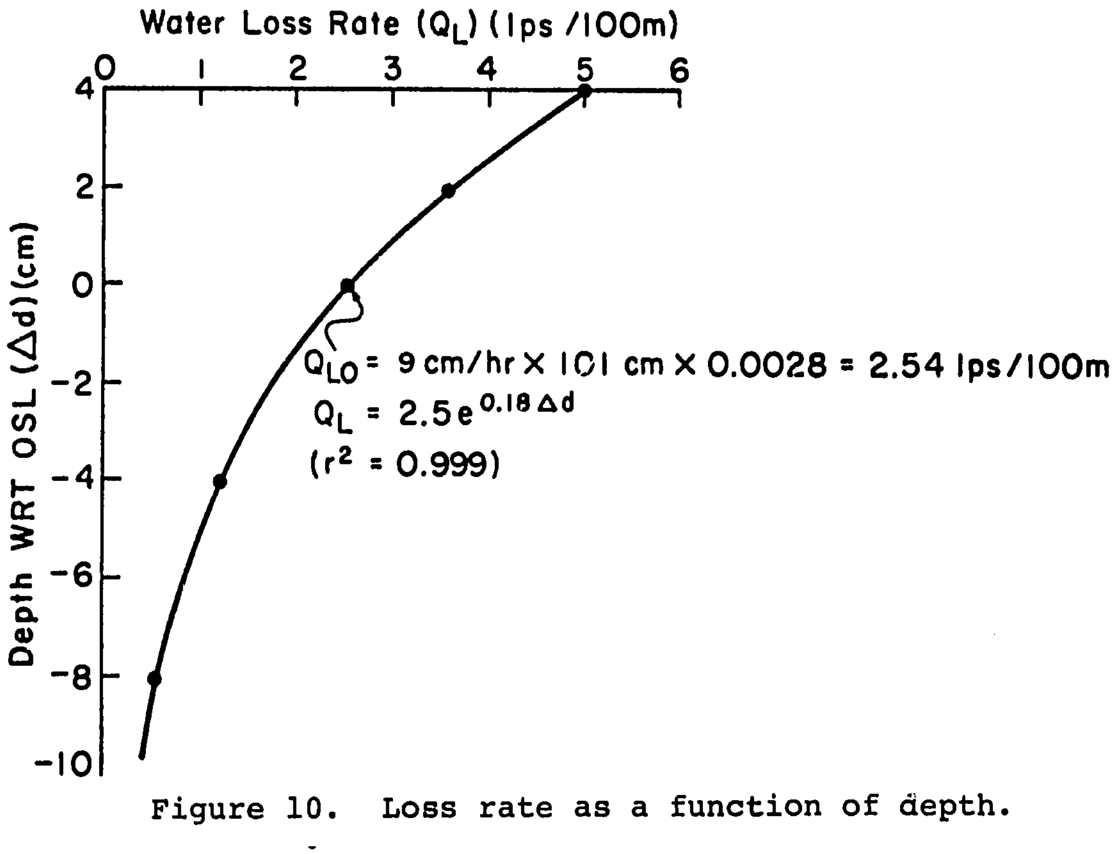


Figure 10. Loss rate as a function of depth.

where:

$(Q_L)_{d_i}$ = water loss rate at depth d_i (lps/hm*),

$(\frac{dd}{dt})_{d_i}$ = water surface recession rate at depth d_i (cm/hr),

$(TW_A)_{d_i}$ = average water surface top width at depth d_i (cm), and

C = conversion factor = $\frac{1}{1000}$ (ℓ/cm^3) x $\frac{1}{3600}$ (hr/sec) x 100 (cm/m) x 100 m length = .0028.

This calculation was repeated at several depths. The overall results were plotted on a graph of water loss rates versus depth relative to osl, as is shown in Figure 10.

A factor which would affect the water surface recession rate is the water surface evaporation during the test. Water surface evaporation will vary with the local climatic conditions. Gibb (1966) lists annual evapotranspiration rates for various areas in the irrigated Indus Basin between five and seven feet per year (150-215 cm/yr). If we convert an intermediate value of 180 cm/yr to the equivalent loss rate from a channel 100 cm wide and 100 m long, the result is 0.006 lps/hm. If this average value is increased four times to allow for tests during daylight hours on hot days, the equivalent loss rate is still only 0.024 lps/hm. Since this value is only about 1 percent of the average measured loss rate, the surface evaporation factor was considered negligible.

*hm = hectometer or 100 meters.

Steady State Inflow-Outflow Flume Measurements

Water conveyance loss rates were measured in longer channel sections (500-2500 m) under normal flowing water conditions by the inflow-outflow method. Although the loss measurement by inflow-outflow with flumes is not usually as accurate as ponding type measurements, it allows measurements in long sections which, by nature of the larger sample size, tends to average out some of the variability found in short ponding loss sections, and it is collected while the channels are in normal operation.

Flow measurements were made with Cutthroat flumes (Skogerboe et al., 1973) installed at each end of a channel section. The losses were taken as the difference in the measured flow rates in the two flumes, and the loss rate per unit length of channel (lps/hm) as the flow loss divided by the distance between the flumes. Flume readings were not made until the flow rates measured in both flumes had reached a constant value so that there was no change in storage in the section and flow conditions were steady state.

Any flow measuring device which creates a head loss in an open channel will raise the water level in the channel upstream from the device. Figure 10, and later analysis in this report, indicate that loss rates in watercourse channels are sensitive to changes in flow depth. A flume installed in a watercourse channel would thus be expected to increase the water losses above the value which normally occurs in the channel without the flume. The amount of this effect depends upon the head loss in the flume (or other flow measuring device); the extension of the raised water level upstream, termed the backwater curve, which is a function of the channel hydraulic characteristics of flow rate, cross-sectional shape, roughness, and especially slope; and the sensitivity of loss rates to depth changes.

To better understand the effect of flume head loss on water losses in a section, a theoretical analysis of flume-induced losses was made utilizing iterative backwater calculations (Chow, 1959, p. 262) and an empirically derived exponential relationship, to be presented later, between loss rate and changes in flow depth. Thus, in a channel with specific hydraulic characteristics which determine a certain backwater curve, the ratio of the measured to normal loss rate can be predicted in a channel section of a given length and a flume setting with a given head loss. The results of such an analysis for the channel shown in Figure 2 with a normal depth, d, of 0.30 m, Q = 45 lps, and three slope and head loss values, are given in Table 2. It can be seen from the table that watercourse loss measurements can be greatly affected by head loss in the flow measurement device, and this factor must be considered when making inflow-outflow measurements.

Cutthroat flumes were chosen as the flow measurement device because they have been calibrated under submerged flow conditions and flow measurements can be made with less head loss than in most other devices. Engineers and field technicians were trained to set the 0.91 m (3 feet long)

Table 2. Ratio of the measured (including induced losses resulting from the head loss through the flume) water losses to normal losses for a channel.

Head Loss in Flume (cm)	Channel Slope (m/km)	Section Length (m)					
		200	500	1000	1500	2000	3000
3.3	.2	1.50	1.36	1.24	1.17	1.14	1.09
	.5	1.36	1.19	1.10	1.07	1.05	1.03
	.8	1.26	1.12	1.06	1.04	1.03	1.02
6.7	.2	2.36	1.98	1.63	1.34	1.23	1.16
	.5	1.99	1.53	1.28	1.18	1.14	1.09
	.8	1.71	1.35	1.18	1.12	1.09	1.06
10.0	.2	3.73	2.96	2.26	1.90	1.68	1.46
	.5	2.94	2.04	1.54	1.36	1.27	1.18
	.8	2.45	1.66	1.33	1.22	1.17	1.11

*For normal depth = 0.3 m, flow rate = 45 lps, roughness coefficients varied with slope to maintain a hydraulic section of 0.08, and exponential coefficient value (b in Equation 11) = 0.15.

Cutthroat flumes used primarily in collecting the data according to the channel conditions. If slopes and freeboards were small and channel conditions were poor, the flumes were set to minimize the head loss and the flume-induced excess losses. If channel conditions were good and slopes were larger, then the flume could be set with larger head loss to obtain a more accurate flow measurement. Flume head losses usually varied from 3 to 10 cm.

For analyses of inflow-outflow data which were sensitive to the flume-induced losses, these effects were calculated and subtracted out by the same theoretical methods as were used to generate Table 2.

A graph was generated of the ratio of measured (including flume-induced) to normal loss rates versus section length for a flume head loss of 6 cm and a slope of .0005 and the same hydraulic characteristics and cross-sectional shape as was used to generate Table 2. After testing several equations, it was decided that an exponential curve with an added constant factor best fit the data, so through combined regression and trial and error techniques, a curve which minimizes the squares of the deviations in the commonly encountered section length range was generated. The best fit curve was:

$$\frac{Q_{L-M}}{Q_{L-N}} = 1.11 + 1.044e^{-0.209D} \qquad (8)$$

where:

Q_{L-M} = measured loss rate (lps/hm)

Q_{L-N} = normal loss rate (lps/hm)

D = channel section length (hm (hectometer) = 100 m)

The measured loss rates were thus adjusted for flume effects by the formula:

$$Q_{L-A} = Q_{L-M} \left(\frac{1}{1.11 + 1.044e^{-.209D}}\right), \qquad (9)$$

where:

Q_{L-A} = loss rates adjusted for flume-induced losses (lps/hm).

Conveyance efficiencies were also adjusted by:

$$E_A = 1 - (1 - E_M) \left(\frac{1}{1.11 + 1.044e^{-.209D}}\right) \qquad (10)$$

where:

E_M = measured conveyance efficiency or the ratio of the measured outflow to the measured inflow, and

E_A = conveyance efficiency adjusted for flume-induced losses.

Although the actual adjustment required in each case will depend upon the channel conditions and flume setting, the adjustment indicated above will at least allow an evaluation of the importance of the flume effects on the tested relationships.

Regression analyses were made to relate the steady state inflow-outflow loss measurements to the normal inflow rate, fluctuations in the flow rate in the upstream flume, the length of the measured section, and the relative slope of the section as determined by the average channel slope from the watercourse head to the field.

Operational Inflow-Outflow Flume Measurements

Realizing that there are water losses associated with the operation of a watercourse system which are not measured in steady state inflow-outflow or ponding type measurements, five watercourse systems were studied while operating during a complete warabundi turn rotation. Flow measurements were made continually with Cutthroat flumes installed at the watercourse head, at the outlet where the water flowed from the sarkari khal to the farmer's branch, and just above the field outlet. Consequently, all water which entered the watercourse system and which flowed into the fields being irrigated during the complete rotation passes through three flumes. In this way volumes of water, rather than just flow rates, flowing through each portion of the watercourse could be determined by integrating the area under the flow rate versus time hydrograph for each flume as shown in Figure 11.

The measured losses could then be compared to the losses which would have occurred had the system been functioning constantly under steady state conditions. This comparison is shown graphically in Figure 12. The difference between the operational and steady state losses is termed transient losses since they change with time.

Transient losses include dead storage water left lying in the bottom of channels after drainage is complete, the excess water which initially infiltrates into dry beds and banks, water losses while water is being moved from one field to another, and water loss resulting from short term outlet breaks and breaches in watercourse banks.

The first three types of transient losses depend upon the amount of channel which is filled and drained in the process of irrigating a field. Consequently, channel lengths wetted and drained were measured and related to transient losses. Dead storage losses were directly measured by estimating the cross-sectional area of dead storage water with top width and depth measurements made each 30 m in recently drained sections. Water volumes lost from short-term nucca or bank breaks were estimated from the drop in the hydrograph (Figure 11) from the steady state flow rate during the time of the break. Infiltration tests were made

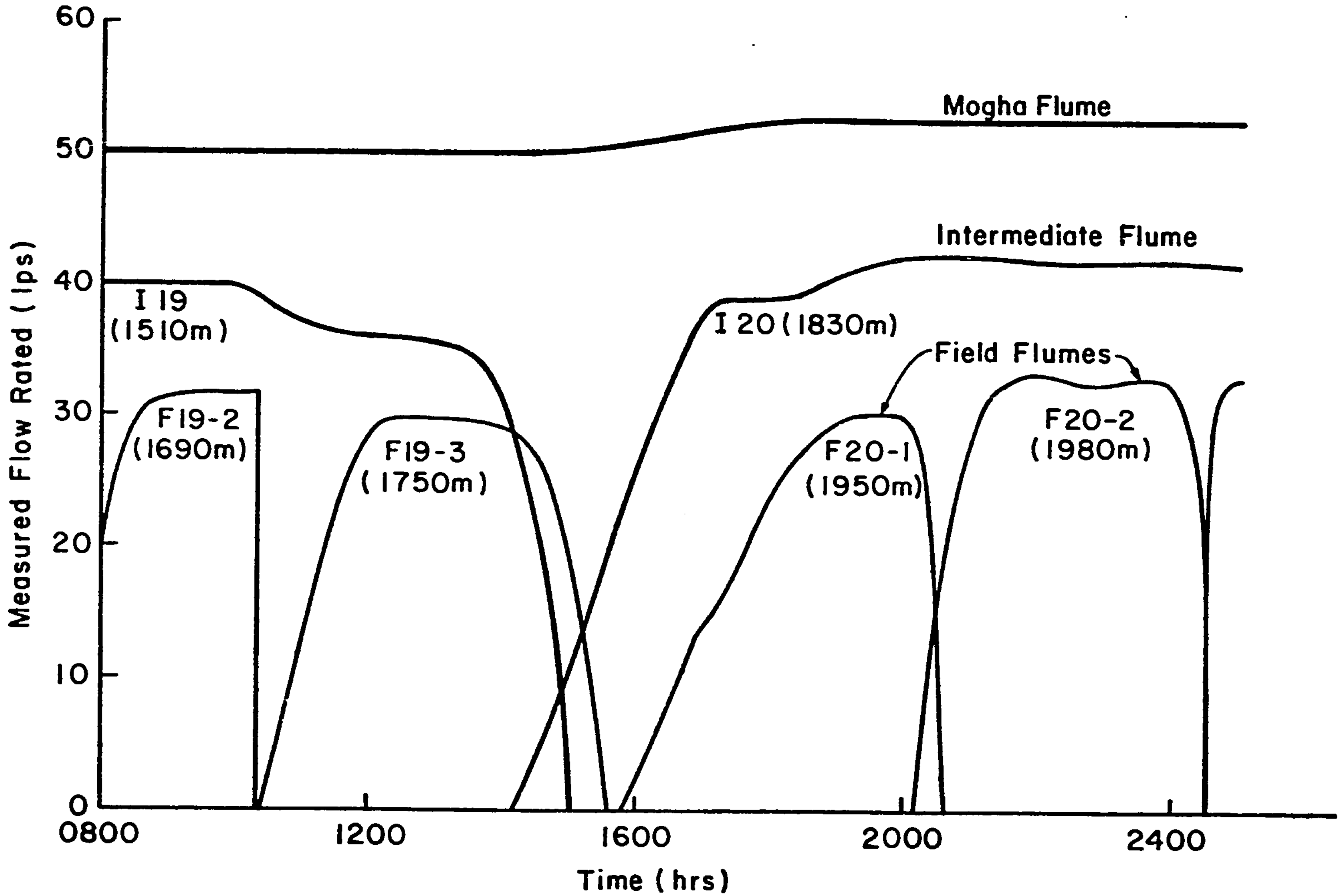
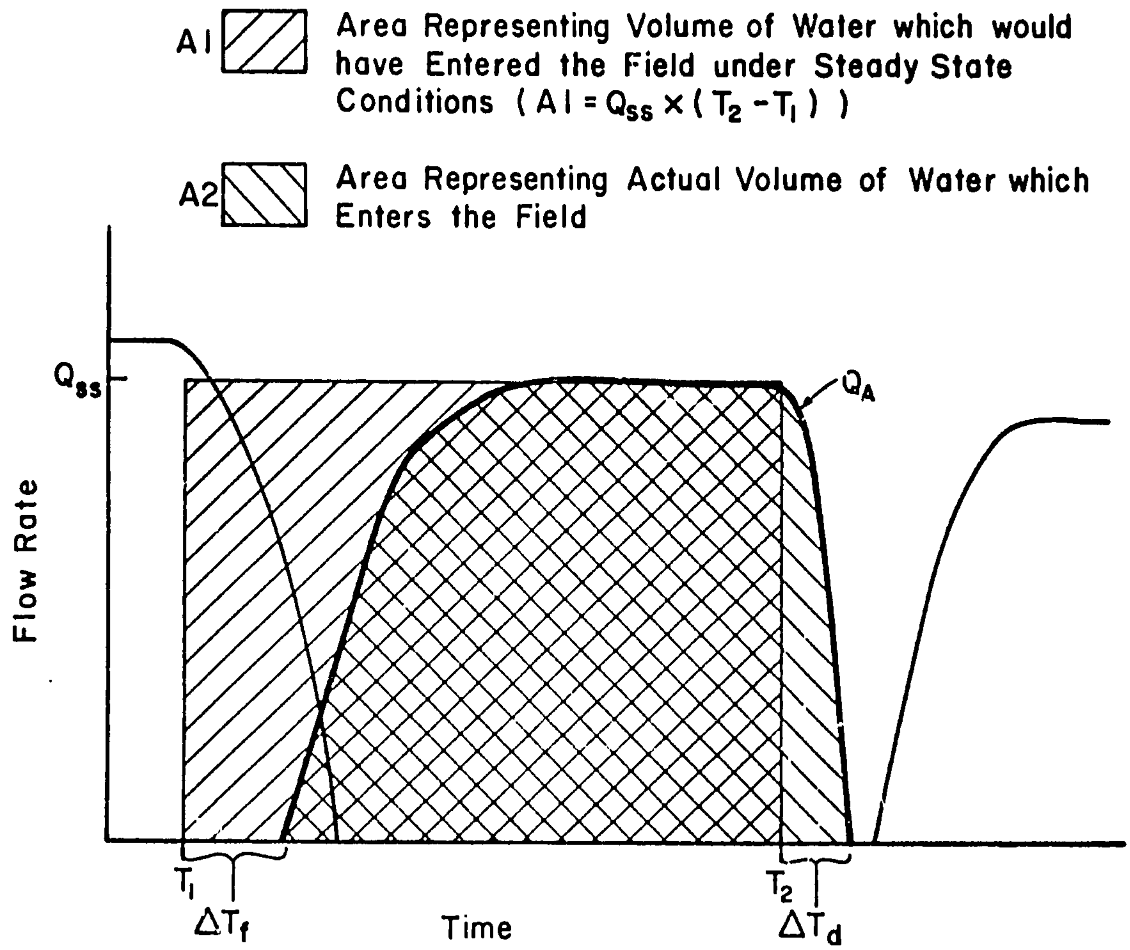


Figure 11. Flow hydrograph showing discharge through the mogha, intermediate, and field flumes over time during an operational loss study.



T_1 = Time Water Flow is Turned from Previous Field

ΔT_f = Time Consumed in Filling the Channel Leading to the Present Field

T_2 = Time Water Flow is Turned to the Following Field

ΔT_d = Drainage Time of Channel into the Field

Q_{ss} = Steady State Flow Rate

Q_A = Actual Flow Rate

Transient Loss (V_{TL}) = A1 − A2

Figure 12. Graphical depiction of the transient loss calculation.

in short channel sections to attempt to define the intake versus time curve for watercourse channels, but the variability in the loss rates and antecedent moisture conditions was so great that no conclusions could be drawn about volumes of water initially absorbed into the wetted perimeters of dry channels.

Data Analysis

The purpose of the field data collection was to enable determination of functional relationships between water loss rates and design parameters. Although attempts were made to hold constant as many of the unmeasured factors as possible, the scatter in the collected field data was such that statistical techniques were required to determine the relationships between the factors. Initially, linear regressions were used to relate the variables. If this linear regression analysis, graphical investigation, or theoretical considerations indicated a curvilinear relationship, one or both of the variables were transformed logarithmically and the data was again linearly regressed so that exponential and power curve models could be evaluated. The coefficient of determination (r^2) was used to compare the fit of the models to the data.

Figure 10 indicates that loss rate in any channel section changes with the water depth. Plotting of the data from many channels indicated the relationship was curvilinear with a positive first derivative and could usually be represented by an exponential relationship of the form:

$$Q_L = Q_{LO}e^{b(\Delta d)} \tag{11}$$

where:

Q_L = water loss rate (lps/hm),

Q_{LO} = water loss rate at the osl ($\Delta d = 0$) (lps/hm),

b = an empirically derived exponent (cm^{-1}), and

Δd = water depth relative to the osl depth (cm).

The values of Q_{LO} and b were determined by running an exponential regression analysis of the values of Q_L and Δd taken from the graphs, such as Figure 10. The coefficient of determination (r^2) of the regression analysis was taken as the measure of the accuracy of the derived exponential relationship in describing the data. Normally the relationship was very good with r^2 values above 0.95 in two thirds of the samples, and the derived Q_{LO} values were nearly always within 5 percent of the actual measured water loss rate at osl.

In the ponding loss measurement, the derived coefficient and exponent of the loss versus depth relationship, Q_{LO} and b, were accepted as parameters which describe the water loss rate in the channel. The value Q_{LO} gives the normal loss rate in the test section at a reference depth (osl), so that the depth factor can be held constant, and b is a measure of the sensitivity of that loss rate to fluctuations in flow depth. The exponent can be interpreted mathematically as the fractional change in loss rate with a unit change in depth, since:

$$\frac{\frac{dQ_L}{dd}}{Q_L} = \frac{bQ_{LO}e^{b\Delta d}}{Q_{LO}e^{b\Delta d}} = b \tag{12}$$

where:

$\frac{dQ_L}{dd}$ = rate of change of loss rate with depth.

The exponent was utilized as a factor only for those samples with r^2 values above 0.95.

An attempt was then made to relate the other descriptive parameters measured during the ponding loss experiments to these Q_{LO} and b values. Multilinear step-wise regression analyses were made between Q_{LO} and b and the measured quantifiable factors of:

1. operational supply level elevation with respect to the elevation of the adjoining field surfaces (ΔE) (cm),
2. average bank width at osl, BW (cm),
3. normal flow rate in the channel, Q (lps),
4. the percentage of time the section is full, T (%),
5. the distance of the section from the mogha inlet, D (m), and
6. the side slope of the inner bank at the osl, Z (cm/cm).

In addition to these six quantifiable factors, there were several nonquantitative characteristics which were also noted. Relationships could not be derived between these factors and the loss rates, but the samples were broken into subgroups depending on the different characteristics, and analysis of variance tests were run between the data subsets to determine whether the factor did affect the loss rates. If they did have a significant affect, the data were often broken into the same subsets for regression analyses of the quantifiable factors. The six nonquantifiable factors were:

1. whether or not the watercourse was augmented by a public tubewell supply (SCARP/non-SCARP (Sc/NS)),
2. whether the section was sarkari khal or farmer's branch (SK/FB),
3. whether or not the section had been recently rebuilt (Improved/Unimproved (I/U)),
4. whether the section's wetted perimeter was generally clean or full of grass and vegetation (Cl/GR),
5. whether or not the banks had been thoroughly wet up before the data collection began (Wet/Dry (W/D)), and
6. soil type: Sandy (Sa), Sandy Loam (SaL), Loam (L), Silt Loam (SiL), and Clay Loam (CL).

In a few cases, attempts were made to select sections where most of the above listed factors could be controlled or held constant. This allowed one of the factors to be compared directly with the loss rate parameters. This was ore in the case of ΔE, BW, and Cl/GR factors.

Because it was realized that there were still important unmeasured factors which affect the loss rates, a large sample was collected so that derived relationships would be significant. Ponding loss measurements were conducted on over 120 test sections, but only about half of these samples were complete in every factor. This total sample size is adequate for such analysis, but when broken into subsets by the nonquantifiable factors, sample size was often too small to derive significant relationships from the more poorly correlated factors.

Two criteria were used to determine whether a relationship is meaningful. The first is the significance of the relationship. A relationship will be considered significant if the probability of the null hypothesis is less than 0.05 (termed as being significant at the 95 percent probability level). In some cases, the probability level will be given. The second is consistency. Because of the nonquantitative factors, the data is broken into several subsamples in attempt to eliminate the effects of these factors. A relationship will be considered more meaningful and dependable if it is consistently of the same sign and the coefficient is in the same range for the various subsamples.

For data collected with flumes during steady state inflow-outflow measurements, it was assumed that the channel was flowing at the usual depth and the measured loss rate was taken to be equivalent to the Q_{LO} derived in the ponding tests. That head losses in the flumes actually raise the water depth somewhat is discussed in the previous section, and Eqs. 9 and 10 were used to adjust inflow-outflow data for flume effects.

Since steady state losses were measured in long channel sections, two descriptive parameters, flow rate and distance from the inlet, varied within the test section. Flow rate will decrease by the amount of losses that have previously occurred. Distance, of course, increases linearly along the section. To allow consideration of these variations, the loss rate relationships were integrated over distance and the data was fitted to the resulting conveyance efficiency relationships.

In the operational loss studies, in addition to the multiple linear regression analysis used to determine the relationship between transient losses and distance of channel filled or drained, water balance techniques were used on the watercourse system to estimate values which could not be measured directly.

Theoretical Analysis

Design factors which describe the hydraulic characteristics of an open channel have previously been studied by many authors and both theoretical and empirical relationships have been proposed. One of the most commonly utilized design equations for uniform flow in open channels, and the one which will be used here, is Manning's equation (Manning, 1891).

Manning's equation is used to interrelate the functional relationships determined in the regression analyses with other hydraulic factors. As previously mentioned, loss rates vary with changes in the flow depth. With Manning's equation, variations in flow rate (Q), roughness coefficient (n), and slope (S), can also be related to depth changes and thus to loss rate for given channel cross-sectional shapes. This analysis allows the derived functional relationships to be extended to other factors, as well as provides a theoretical model for the flow rate factor which can then be verified with the data.

Application to Practical Design Alternatives

Once functional relationships between watercourse descriptive parameters and loss rates have been established allowing prediction of the consequences of various changes in those parameters on conveyance losses, it is possible to construct a mathematical model between the significant factors and conveyance losses. This model was then used to predict the effect of various watercourse design alternatives on water conveyance losses. Since Indus Basin watercourses are complex systems with nontechnical constraints (e.g., legal, cultural, and traditional) design alternatives will be considered in the light of these constraints. The constraints were not taken as strict limits to design alternatives, but alternatives were evaluated considering these nontechnical factors. Several alternative practical system designs were evaluated.

Loss Measurement Units

Several different units have been used to describe water losses from conveyance systems, ranging from seepage rates into wetted perimeters (m/day) to percent water loss from a given system. Each system of units has inherent weaknesses. The seepage rate unit assumes that the infiltration rate into all portions of the wetted perimeter is uniform and that water losses can be calculated knowing the total wetted perimeter area involved. It is usually applied to large canals where conditions and seepage rates are fairly uniform. The percent water loss unit makes no such assumptions, but is system specific and is not generalizable to other conveyance systems. Percent water loss is most useful for describing the efficiency a conveyance system and least useful for understanding water losses.

In a research effort to understand channel water losses, the most generalizable unit is best. However, seepage rate per unit area is not a good descriptive term because, as will be shown, seepage rates into different portions of watercourse channel wetted perimeters are highly variable and such a term would hide this variability, and because loss rates are not proportional to wetted perimeter lengths. In addition, use of this unit requires field measurement of wetted perimeter lengths, which is not otherwise done.

Seepage rates are much less variable longitudinally along a water channel than along the wetted perimeter. Therefore, water losses on a per unit length basis is a more representative unit. This unit has the disadvantage of inability to compare losses from channels of different sizes and capacities. Losses vary with both design capacity (channel size) and with fluctuations in flow rates in a given channel, but since neither relationship is proportional, loss rates per unit length as a percent of flow (%/hm) obscures recognized factors contributing to the variability.

The unit chosen as the best alternative for describing watercourse channel losses is the water loss rate per unit length (lps/hm). This unit has the additional advantage of being the actually measured parameter in most cases and does allow a comparison between the measurement techniques utilized. It can be generalized to total water losses for a given conveyance system by knowing the system length and time utilized. Once a relationship is established between this unit and other descriptive parameters, such as flow rates, it can be more accurately generalized to apply to other systems.

Chapter 5

RESULTS

Ponding Loss Measurements

Table 3 lists the data collected in 122 ponding loss measurements. The first digit of the data set identification number (I.D. No.) indicates whether the watercourse is non-SCARP (1) or SCARP (2). The second digit is the same within each subset for all tests made on the same watercourse. The third digit is the same within each watercourse group for tests made at the same time on adjoining sections. The final digit identifies the section. Sixty-five measurements are complete in all quantitative data. Because of inadequate number of samples with measured soil types, no analysis was made between soil type and loss rate.

The loss rates at operational supply level, Q_{LO}, and exponential coefficient, b, were determined by the curve fitting procedure described in Chapter 4. The coefficient of determination (r^2) for each derived curve is given.

The loss rates at osl exhibit a wide variability with a mean of 2.32 lps/hm*, a standard deviation of 1.95 lps/hm, and a range of 0.01 to 12.93 lps/hm for the 122 data sets. About half of this variability can be related to the measured parameters.

The data strongly support the hypothesis that the loss rate is exponentially related to the depth of the water in the watercourse. For two-thirds of the data sets, the

*hm (hectometer) = 100 m

Table 3. Ponding loss data including qualitative and quantitative variables and loss rates.

I.D. NO.	QUALITATIVE VARIABLES						QUANTITATIVE VARIABLES Q (LPS)	D (M)	I (0/0)	Z	HW (CM)	DF (CM)	LOSS RATE QLO (LPS/100M)	B (1/CM)	RSQ
1111.	NS	U	SK	CL	D	SAL	31	448	50	1.35	63	28	.10	.341	.962
1112.	NS	U	SK	CL	D	SAL	31	448	50	1.25	54	27	.09	.341	.980
1113.	NS	U	SK	CL	D	SAL	31	512	45	.82	104	18	.42	.331	.970
1121.	NS	U	FR	GR	W	SAL	28	384	2	.99	66	11	1.45	.211	.972
1122.	NS	U	FR	GR	W	SAL	28	384	2	.45	60	11	.83	.180	.995
1123.	NS	U	FR	CL	W	SAL	28	448	1	1.40	66	10	.49	.130	.960
1124.	NS	U	FR	CL	W	SAL	28	448	1	.54	71	15	.74	.128	.986
1211.	NS	U	FR	CL	D	SA	34	1600	4	.76	50	20	3.93	.057	.943
1212.	NS	U	FR	CL	D	SA	34	1600	4	.90	50	20	4.83	.051	.965
1221.	NS	U	FR	GR	D	SAL	57	320	10	1.40	72	30	2.38	.189	.998
1311.	NS	U	SK	GR	D	L	65	64	70	.22	68	8	.09	.089	.941
1312.	NS	U	SK	GR	D	L	65	64	70	.90	77	4	.01	.245	.997
1321.	NS	U	SK	GR	D	L	65	64	14	2.00	72	11	.14	.224	.997
1411.	NS	I	SK	CL	D		113	3840	7	.45	127	24	.14	.249	.980
1412.	NS	I	SK	CL	D		113	3840	7	1.50	101	26	.23	.168	.999
1413.	NS	I	SK	CL	D		113	3840	7	.71	107	31	.30	.019	.956
1421.	NS	U	SK	GR	D		113	4160	18	.53	64	21	.67	.056	.961
1422.	NS	U	SK	GR	D		113	4160	18	1.21	63	19	.58	.069	.968
1431.	NS	U	FR	GR	W		113	1408	4	1.47	49	25	1.03	.466	.819
1511.	NS	U	FR	GR	D		34	64	21	.75	35	17	.19	.605	.411
1512.	NS	U	FR	GR	D		34	64	5	1.60	28	11	.58	.195	.973
1521.	NS	U	FR	GR	W	SAL	23	256	4	1.20	34	11	.39	.237	.875
1522.	NS	U	FR	GR	D	SAL	20	192	3	2.40	41	15	1.28	.286	.977
1531.	NS	U	FR	GR	D		23	64	3	.45	28	10	1.04	.127	.991
1532.	NS	U	FR	GR	D		20	64	6	.57	35	20	.45	.159	.989
1611.	NS	U	SK	CL	W		48	640	13	2.34	82	20	.49	.128	.596
1612.	NS	U	SK	GR	W		48	704	10	1.20	88	7	.68	.149	.941
1613.	NS	U	SK	GR	W		48	768	10	1.25	112	9	.71	.208	.907
1711.	NS	U	SK	GR	W		76	384	7	1.16	143	30	1.03	.177	.974
1712.	NS	U	SK	CL	W		76	384	7	1.95	144	17	1.42	.187	.989
1811.	NS	I	SK	CL	D	SIL	40	2240	8	1.50	97		.55	.215	.995
1821.	NS	U	FR	GR	W	SIL	40	1728	2	1.33	50		.28	.174	.450
1831.	NS	I	SK	CL	D	SIL	48	320	36	.55			.22	.172	.827
2711.	S	I	SK	CL	W	CL	141	2880	25	.86	88	49	2.51	.188	.940
2721.	S	U	FR	CL	W	CL	170	320	8	.60	64	13	4.64	.182	.990
2211.	S	I	SK	GR	W	CL	85	1408	7	1.04	88	34	1.11	.156	.997
2221.	S	U	SK	GR	W	CL	85	1280	5	.65	76	14	3.43	.152	.998
2811.	S	U	FR	GR	W	L	85	64	14	.92	49	21	2.69	.098	.930
2611.	S	I	SK	CL	W	CL	57	1600	7	.63	61	6	1.86	.199	.980
2511.	S	U	FR	CL	W	SIL	99	448	5	.30	71	19	.56	.203	.950
2512.	S	U	FR	CL	W	SIL	99	448	5	.45	75	25	1.02	.192	.950
2513.	S	U	FR	CL	W	SIL	99	512	5	.24	78	23	1.30	.146	.990
2411.	S	U	FR	CL	W	SA	79	64	7	3.30	122	4	3.25	.050	.910
2412.	S	U	FR	CL	W	SA	79	64	7	.49	105	3	1.86	.021	.460
2413.	S	U	FR	CL	W	SA	79	128	7	.92	82	-5	4.55	.275	.930
2421.	S	U	SK	GR	D	SAL	79	64	5	.70	107	-42	.46	.170	.950
2422.	S	U	SK	GR	D	SAL	79	64	5	.50	95	-33	.37	.160	.920
2423.	S	U	SK	GR	D	SAL	79	128	5	1.00	100	-20	.28	.171	.960
2431.	S	U	SK	GR	D	SAL	79	128	11	1.60	134	24	3.06	.115	.960
2432.	S	U	SK	GR	D	SAL	79	128	11	2.00	130	26	3.43	.281	.970
2311.	S	I	SK	GR		CL	127	64	10	.68	96	30	3.46	.203	.940
2312.	S	I	SK	GR		CL	127	64	10	.98	96	36	1.79	.161	.984
2321.	S	I	SK		W		127	64	10	.56		32	5.57	.140	.990
2322.	S	I	SK		W		127	64	10	1.20		35	4.83	.165	.990
2323.	S	I	SK		W		127	128	10	1.11		35	3.43	.127	.990
2324.	S	I	SK		W		127	128	10	.93		49	2.32	.169	.950
2331.	S	I	SK		W		113	768	5	1.11		14	1.79	.037	.884
2332.	S	I	SK		W		113	768	5	.80		6	1.80	.109	.803
2333.	S	I	SK		W		113	832	5	1.00		0	1.12	.098	.926
2334.	S	I	SK		W		113	832	5	1.43		15	3.04	.167	.905
2111.	S	I	SK		W		113	1920	12	1.00		11	1.98	.103	.990
2112.	S	I	SK		W		113	1920	12	.83		16	3.25	.113	.990
2113.	S	I	SK		W		113	1984	12	.55		18	2.69	.110	.950
2114.	S	I	SK		W		113	1984	12	.71		39	2.60	.044	.950
2115.	S	I	SK		W		113	2048	12	.81		18	2.13	.084	.920
2121.	S	I	SK	CL	D		113	1856	18	.94	127	10	1.26	.121	.984
2131.	S	I	SK	CL	D		113	2368	12	.96	101	25	2.23	.134	.994
2132.	S	I	SK	CL	D		113	2368	8	1.03	116	22	3.26	.119	.973
2231.	S	U	FR	CL	D	SIL	99	256	1	.60	71	21	2.35	.084	.987
2232.	S	U	FR	CL	D	SIL	99	256	1	.50	60	28	4.19	.074	.997

Table 3 (continued).

I.D. NO.	QUALITATIVE VARIABLES						QUANTITATIVE VARIABLES Q (LPS)	D (M)	T (%)	Z	BW (CM)	DE (CM)	LOSS RATE QLO (LPS/100M)	B (1/CM)	RSQ
2233.	S	U	FB	CL	D	SIL	99	320	1	.30	57	26	2.83	.062	.936
2141.	S	U	FB	CL		SIL	99	256	1	.50	60	28	3.62	.056	.995
2142.	S	U	FB	CL		SIL	99	256	1	.50	50	28	5.15	.064	.997
2151.	S	I	SK				113	2560	9	.75	24	86	.69	.123	.953
2161.	S	U	FB	GR	W		113	1536	2	.83	68	23	12.93	.163	.989
2162.	S	U	FB	GR	W		113	1536	2	.85	72	23	5.52	.059	.986
2163.	S	U	FB	GR	W		113	1600	2	1.25	99	23	7.40	.073	.991
2621.	S	I	SK	CL	D		57	1024	36	1.35	125	16	1.14	.150	.984
2622.	S	I	SK	CL	D		57	1088	38	.82	118	19	1.17	.118	.999
2623.	S	I	SK	CL	D		57	1152	38	1.30	125	34	1.14	.127	.989
2624.	S	I	SK	CL	D		57	1216	38	1.32	112	39	.98	.121	.998
2171.	S	I	SK		D					1.25			2.69	.124	.957
2172.	S	I	SK		D					1.25			2.90	.066	.985
2173.	S	I	SK		W					1.10			1.11	.084	.906
2174.	S	I	SK		W					1.20			1.04	.103	.926
2181.	S	U	FB		D					.90			2.12	.170	.945
2182.	S	U	FB		D					.40			1.45	.113	.945
2183.	S	U	FB		W					1.15			1.45	.113	.960
2184.	S	U	FB		W					.61			1.20	.110	.980
2191.	S	U	FB	GR									3.78	.050	.940
2195.	S	U	FB	CL									2.78	.066	.961
2192.	S	U	FB	GR									3.85	.085	.994
2196.	S	U	FB	CL									3.11	.081	.907
2193.	S	U	FB	GR									.54	.156	.980
2197.	S	U	FB	CL									1.44	.153	.962
2341.	S	I	SK	GR									3.00	.163	.988
2345.	S	I	SK	CL									2.42	.127	.993
2342.	S	I	SK	GR									2.52	.147	.996
2346.	S	I	SK	CL									3.18	.131	.995
2343.	S	I	SK	GR									1.62	.226	.987
2347.	S	I	SK	CL									1.96	.186	.989
2911.	S	I	SK										5.20	.096	.910
2921.	S	I	SK								30		5.48	.125	.900
2931.	S	I	SK								60		6.40	.188	.980
2912.	S	I	SK								90		6.87	.144	.990
2922.	S	I	SK								30		6.22	.140	.930
2932.	S	I	SK								60		4.83	.130	.885
2913.	S	I	SK								90		3.53	.055	.890
2923.	S	I	SK								30		3.43	.071	.970
2933.	S	I	SK								60		2.41	.065	.980
2941.	S	I	FB								90		2.05	.303	.985
2942.	S	I	FB										2.51	.170	.787
2943.	S	I	FB										2.01	.244	.913
2944.	S	I	FB										5.24	.100	.970
2945.	S	I	FB										5.20	.116	.940
2631.	S	U	SK				23	1152	15				1.32	.190	.996
2641.	S	U	FB				40	320	8				.80	.055	.910
2951.	S	I	SK										1.35	.089	.968
2961.	S	I	SK										2.23	.134	
2962.	S	I	SK										3.25	.119	
2971.	S	I	SK										4.73	.127	.940
2351.	S	I	SK										1.95	.076	.900

Symbols

NS = Non-SCARP
S = SCARP
U = Unimproved
I = Improved
SK = Sarkari Khal
FB = Farmer's Branch
CL = Cleaned
GR = Grassy
D = Dry

W = Wet
SA = Sand
SAL = Sandy Loam
L = Loam
SIL = Silty Loam
CL = Clay Loam
Q = Inflow
D = Distance from mogha

T = Time used (%)
Z = Bank Side Slope
BW = Bank Width
DE = Elevation above Field (ΔE)
QLO = Derived Loss Rate at osl (Q_{LO})
B = Derived Exponent (b)
RSQ = r^2 (coefficient of determination) of derived equation

relationship between loss rate and depth fits the exponential Eq. 11 with a coefficient of determination above 0.95. In only 11 of the 122 total data sets does the relationship have an r^2 value below 0.90.

The exponent coefficient, b, which was considered only for those cases where the r^2 value of the derived equation was greater than 0.95, is also widely variable. The mean for 81 cases is 0.150 with a standard deviation of 0.069. Values varied from 0.019 to 0.605. Two-thirds of this variability is related later in this chapter to the measured parameters.

A tendency for the b value to be lower in watercourses with higher loss rates was noticed, so b was regressed with Q_{LO}. It was found that such an inverse relationship does exist, and it is highly significant, but the coefficient of determination is low and the equation coefficient is small relative to the intercept. The regression equation for the total data (for $r^2 \geq 0.95$) is:

$$b = 0.172 - 0.0097\ Q_{LO} \qquad r^2 = .088 \tag{13}$$

Subsets of the total data display the same general relationship, although the equation coefficient is much higher for non-SCARP than for SCARP channels. The absolute change in loss rate with depth increases with increasing operational supply level loss rate (Q_{LO}), but less than proportionally, while the fractional change decreases slightly (12 percent with one standard deviation change in Q_{LO}) with increasing osl loss rate.

This finding would indicate that watercourse sections with higher loss rates have relatively higher intake rates in the lower portions of the wetted perimeter relative to higher portions, as compared to sections with low loss rates. This could be a result of more rodent and insect holes being lower in the channel in high loss rate sections.

Analysis of variance tests were run on the loss rate and b values of the five data sets divided according to the nonquantitative factors. The most significant finding was that loss rates are higher in SCARP (Sc) augmented watercourse channels than in non-SCARP (NS) channels. This was true both on the total sample and from just the farmers' branches. It should be noted that all SCARP channels were from the Mona Project area in SCARP II. Means, standard deviations, and significance levels for the various subsamples are given in Table 4.

This finding can be partially attributed to a biasing in the sample. SCARP watercourses carried significantly more flow on the average than non-SCARP channels (94.9 lps versus 55.5 lps). Regression analyses determined that loss rate is directly related to flow rate (Q) with a regression slope coefficient of 0.023 for the total data set. This coefficient, when multiplied by the difference between the mean flow rates for the two data sets, would predict a difference in the loss rates, attributable to Q, of 0.9 lps/hm, or a little less than half of the difference in the averages.

Table 4. Summary of analysis of variance results.

Variable	Data Subset 1	Data Subset 2	Data Set	Data Subset 1 N	Data Subset 1 Mean	Data Subset 1 SD*	Data Subset 2 N	Data Subset 2 Mean	Data Subset 2 SD*	F**	Significance
Q_{LO} (lps/hm)	Sc	NS	Total	89	2.87	1.93	33	0.84	1.04	32.56	99%
	Sc	NS	FB	32	3.23	2.44	15	1.33	1.37	7.90	99%
	SK	FB	Total	75	2.13	1.67	47	2.62	2.32	1.85	82%
	SK	FB	NS,U	13	0.50	0.42	15	1.33	1.37	4.41	95%
	I	U	Total	60	2.63	1.65	62	2.02	2.18	3.06	92%
	I	U	Sc,SK	50	2.79	1.55	7	1.77	1.49	2.70	89%
b (cm^{-1})	Sc	NS	Total	57	.135	.053	24	.186	.087	10.51	99%
	Sc	NS	FB	19	.125	.065	11	.167	.060	3.11	91%
	SK	FB	Total	51	.155	.071	30	.141	.065	0.87	65%
	SK	FB	NS,U	9	.219	.109	11	.167	.060	1.82	81%
	I	U	Total	38	.139	.055	43	.160	.078	1.87	82%
	I	U	Sc,SK	32	.132	.040	6	.180	.056	6.29	98%

*Standard Deviation
**F Statistic

Symbols:

SK = Sarkari Khal
FB = Farmer's Branch
Sc = SCARP
NS = Non-SCARP

I = Improved
U = Unimproved
Cl = Cleaned
Gr = Grassy

The usage time parameter, T, tended to be lower for SCARP data sets, and would, because of an inverse regression relationship between T and Q_{LO}, predict 0.2 lps/hm higher average loss rates in SCARP channels. Both factors combined can explain about half the measured difference between loss rates in SCARP and non-SCARP watercourses. The remainder must be attributed to other aspects of the two types of watercourses.

A second consistent finding that is significant for a non-SCARP, unimproved (NS, U) channel sub-sample, is that farmers' branches (FB) have higher loss rates than sarkari khal (SK) sections. As would be expected, the sarkari khal sections are used much more than FB sections and thus the T variable is much higher in SK sections. The regression analysis found T to be inversely related to the loss rate, so the difference of the means times the regression coefficient would predict a difference in loss rates of about 0.5 lps/hm for both the total sample and the non-SCARP, unimproved (NS, U) sub-sample. This could explain all the difference in loss rates for the total sample, but only about half of the difference for the NS, U sub-sample.

The analysis of variance also indicates that losses in improved channels (I) are higher than in unimproved sections (U). This can be interpreted as the result of a biased sample since most (92 percent) of the improved channel data were from SCARP watercourses which were large channels that had higher loss rates. A sub-sample of SCARP, SK data produced the same result, although the level of significance was only 89 percent. This unexpected result could be a result of small sample size since it represented sections from only three or four watercourses.

Neither of the last two qualitative factors--whether the channel was wet (W) or dry (D) previous to conducting the ponding loss, or whether it was recently cleaned (CL) or grassy (GR)--showed any consistent affect on loss rates.

The variation in the exponential coefficient, or b value of Eq. 11, was also analyzed for the nonquantitative variable subsets. The b value of SCARP watercourses was consistently and significantly lower than the b value determined for non-SCARP watercourses. This could be partially the result of the reverse finding already presented of higher loss rates for SCARP than for non-SCARP sections, and the regression finding (Eq. 13) that the b value is generally inversely related to the loss rates. The effects of this interrelationship could not be separated out.

Improved (I) watercourses also have lower b values than unimproved (U) ones. The result is significant in a SCARP-sarkari khal data sub-sample and consistent with the total data set. This result would be expected since rebuilt watercourse banks have a more uniform permeability because of the absence of the holes present in older banks, but because of the previously mentioned bias in the improved/unimproved sample, additional data are required to confidently establish the relationship.

Whether the test section was sarkari khal or farmer's branch, wet or dry, or grassy or recently cleaned seemed to have no consistent affect on the b value. The significant analysis of variance results are summarized in Table 4.

Loss rate (Q_{LO}) and the exponent coefficient, b, of the loss rate versus depth relationship were linearly regressed with six quantitative variables:

1. usual flow rate, Q (lps),
2. distance from the inlet, D (m),
3. operational supply level elevation with respect to the field surface, ΔE (cm),
4. bank width at the osl, BW (cm),
5. channel bank side slope, Z, and
6. percent of time of channel section usage, T (%).

The regressions were run on the complete data plus eight sub-samples which the analysis of variance results indicated were important and for which sufficient data exist. A step-wise selection procedure was used to select the important variables. The process was stopped when no further reduction in the deviation of the data from the regression relationship was obtained by adding additional variables. The variables are listed in the order of their inclusion in the regression equation in Table 5.

A consistent result of the regression analyses is that the coefficients of determination are generally low. With six measured parameters, about one-fourth of the variability in the loss rates can be explained in the larger data sets,

Table 5. Regression analysis results with loss rate at osl (Q_{LO}) as dependent variable.

Data Set	N	r^2	F	Level of Signifi- cance	SD** of resi- duals	Derived Linear Regression Equations With Parameters Listed in Order of their Insertion into the Regression
Total	65	.22	5.85	99%	1.93	$Q_{LO} = 0.87 + 0.023Q^* - 0.034T^* - 0.00032D$
Sc	35	.22	2.85	95%	2.25	$Q_{LO} = -0.05 + 0.028Q^* - 0.059T + 0.903Z$
Sc,U	22	.56	5.40	99%	2.10	$Q_{LO} = -1.92 + 0.003D^* + 0.023\Delta E + 1.00Z + 0.030Q$
Sc,I	13	.53	5.74	98%	0.66	$Q_{LO} = 0.53 + 0.020Q^* - 0.021\Delta E^*$
NS	30	.22	2.37	91%	1.01	$Q_{LO} = 1.17 - 0.019T^* - 0.009Q + 0.031\Delta E$
NS,SK, U	13	.60	16.24	99%	0.28	$Q_{LO} = 0.89 - 0.013T^*$
NS,FB	14	.84	17.53	99%	0.63	$Q_{LO} = -0.03 + 0.0021D^* - 0.039Q^* + 0.107\Delta E$
U	49	.34	7.64	99%	2.00	$Q_{LO} = 0.05 + 0.027Q^* - 0.040T^* + 0.038\Delta E$
FB	30	.49	12.74	99%	2.00	$Q_{LO} = -0.40 + 0.0024D^* + 0.028Q^*$

*F to enter regression equation larger than 4.0
**Standard Deviation

Symbols:

Sc = SCARP
NS = Non-SCARP
U = Unimproved
I = Improved

SK = Sarkari Khal
FB = Farmer's Branch
Q = usual flow rate (lps)
T = usage time (%)

D = distance from mogha (m)
ΔE = osl elevation with respect to the field surface (cm)
Z = side slope (cm/cm)
Q_{LO} = loss rate at osl (lps/hm)

and about 50 percent of the variability can be explained in the subsets after accounting for the effects of the non-quantitative factors. There are obviously important parameters affecting loss rates which were not measured (some of which are not measurable), and loss rates cannot be predicted with accuracy on the basis of the six measured parameters alone. But the derived coefficients can be used to indicate the sign and degree of the relationship between the various parameters and the loss rates.

The usual flow rate in the channel, Q, which is an indicator of channel size, has the most consistent and significant affect on loss rate. In four of the nine data sets it is the most significant factor, and is, with one exception, included in the regression equation. Except for two cases, the coefficient values vary only from 0.019 to 0.030. These coefficients predict that a 100 percent increase from the mean Q value would lead to a 75 to 100 percent increase in the loss rate from its mean, or that a channel which normally carries twice as much water will probably lose 1.75 to 2 times as much water as the smaller channel. Both cases where the relationship between Q_{LO} and Q is negative involved non-SCARP channels (NS; and NS, FB sub-samples). This inconsistency reduces the confidence in the relationship.

The time factor, T, is inversely related to the loss rate for all regressions. In two cases it is the most important factor and in three cases it is second in importance to Q. Its coefficient varies from -0.013 to -0.059, and loss rates decreased about 2 percent for each unit increase in T. The relationship would predict that a channel which is full 50 percent of the time would have a loss rate 50 percent less than one used 25 percent of the time, other factors being constant. A complication with the determination of the affects of time is that time also varies between sarkari khal and farmer's branch sections and SCARP and non-SCARP sections, creating an intercorrelation with these non-quantitative variables, although in one sub-sample where these factors are cancelled out (NS, SK, U), the relationship is still significant.

The relationship between distance from the mogha, D, and loss rate, Q_{LO}, is positive in all cases except the total data set where SCARP/non-SCARP interactions probably caused the reversal. It is the most important parameter for three sub-samples where data is primarily from farmer's branch channels.

Elevation of the operational supply level with respect to the field level, ΔE, is significantly related to loss rates in five of the nine analyses, but the sign of the coefficient is negative in one of the sub-samples. The positive coefficients vary between 0.023 and 0.107, with one value 2½ times larger than the average of the other three of 0.030.

The last two factors, bank side slope and bank width, were not consistently correlated with loss rates.

The most important factor affecting the b value is channel usage time, T. In six of the eight data sets, it is the most important factor, and the sign of its coefficient, which varies widely from 0.0018 to 0.018, is positive in all cases.

The bank inner side slope, Z, is important for all but one data subset, and is directly related with its coefficient varying from 0.036 to 0.042 in all except one case. The coefficient would predict an increase from the mean b value of 25 percent with a 100 percent increase in Z, or, for example, with a flattening of the sides from a 45° angle to a 26° angle with horizontal.

The distance factor, D, is contained in the regression equation for all except two subsets. It is always inversely related to b.

Flow rate, Q, and ΔE both have consistently negative coefficients with b but neither are important in more than half the data sets. The relationship between BW and b is erratic and in only one case does it contribute significantly to the regression equation.

A summary of the regression analysis with b as the dependent variable is given in Table 6.

Four sets of ponding tests were conducted to test the influence of particular factors on the loss rates. Test sections were selected such that most other factors would remain relatively constant. Adjoining sections were usually utilized.

One experiment to test the affect of bank width on loss rate involved trimming the bank of an existing channel to successively thinner widths while monitoring the loss rates.

Table 6. Regression analysis results with "b" as dependent variable.

Data Set	N	r^2	F	Level of Significance	SD** of residuals	Derived Linear Regression Equation with Parameters Listed in Order of Insertion into the Regression.
Total	49	.38	9.30	99%	.061	b = .115 + 0.0018T* - 0.0002D* + 0.042Z*
Sc	27	.14	1.66	79%	.050	b = .120 + 0.036Z - 0.0006ΔE
Sc,U	16	.59	5.60	99%	.047	b = .148 + 0.018T* - 0.0008ΔE - 0.0009BW
Sc,I	11	.82	6.84	98%	.014	b = .282 - 0.013Z - 0.00002D + 0.0020T - 0.0007Q*
NS	22	.56	7.67	99%	.066	b = .128 + 0.0026T* - 0.00002D* + 0.040Z
NS, U	19	.84	13.38	99%	.042	b = .110 + 0.0027T* - 0.00002D* + 0.051Z* - 0.0010Q
U	35	.66	14.70	99%	.051	b = .135 + 0.0028T* - 0.00003D* + 0.041Z* - 0.00028Q
FB	21	.61	6.23	99%	.045	b = .153 + 0.011T* - 0.0032ΔE* + 0.039Z - 0.00003D

*F to enter the regression equation greater than 4.0
**Standard Deviation

Symbols:

Sc = SCARP
NS = non-SCARP
U = Unimproved
I = Improved

FB = Farmer's Branch
T = Usage time (%)
D = Distance from the mogha (m)
Z = Side slope (cm/cm)

ΔE = osl elevation with respect to the field surface (cm)
Q = Usual flow rate (lps)
BW = Bank width (cm)

The loss rates did not increase until after the last trimming, when the bank width at the osl elevation was 17 cm and the loss rate doubled. Because of bank weakness, further trimming was not possible.

In a complementary experiment, three replications of new channel sections were constructed with bank widths at full supply level of 30, 60 and 90 cm. Analysis of the ponding measurements in the sections (No. 2911-2933 in Table 3) indicated that the probability of a relationship existing between bank width and loss rate in the newly built sections was very low.

Two sets of the ponding loss test sections (Table 3, Nos. 2321-2334 and 2411-2432) were chosen primarily to measure the affects of the height of the osl above the adjacent land surface (ΔE) on loss rate. Both studies, whose site selections tended to hold most other factors constant, indicated positive, significant relationships between ΔE and Q_{LO}. Loss rates tended to increase about 25 percent for each 10 cm increase in ΔE. This is consistent with the coefficient determined on three of the five regression analyses subsets.

Ponding losses were measured in two sets of test sections both before and after the grasses were cleaned from the inner banks and part of the deposited silt was removed from the bed. The Q_{LO} and b values for each section before and after cleaning are given in Table 3 (Nos. 2191-2197 and 2341-2347). The before cleaning operational supply level is used in both measurements, although the actual water surface level decreased about 12 cm as a result of the reduced roughness in the cleaned channels. Half of the sections have increased loss rates after cleaning, while in the other half they decrease. There is only a small and insignificant difference between the means of the two measurement sets. The increasing loss rate with cleaning is probably associated with increased infiltration into banks from which the surface silt layer has been shaved away. Decreasing loss rates could be the result of the plugging of holes, which become visible during the cleaning process, by the farmer. The loss rates at the decreased osl level (a result of the lower roughness coefficient) were reduced to about one-fourth of their previous values.

The b values decreased in five of the six cleaned sections. Mean b values decreased with cleaning from 0.138 cm^{-1} to 0.124 cm^{-1}. The difference of the two sets of values is significant at the 90 percent probability level. The decrease could be the combined result of increasing infiltration rates into the lower banks resulting from removing the surface silt layer and uncovering macropores, and decreasing seepage rates into the upper banks due to occasional plugging of the larger holes in the upper banks.

Steady State Inflow-Outflow Measurements

Steady state inflow-outflow flume data were gathered primarily from four sources. Loss measurements were measured on long sections of 62 watercourses in the Sahiwal Tehsil by the Government of the Punjab On-Farm Water

Management Development Project. The data set, listed in Punjab On-Farm Water Management Project (1978), will be referred to as OFWM data. Forty watercourses were studied by a comprehensive watercourse survey by Lowdermilk et al. (1978). Data from this study, referred to as "Survey" data, is summarized in Lowdermilk et al. (1978). Steady state loss measurements were made on TW 56-L, TW 51 and TW 78 watercourses in the Mona Reclamation Experimental Project area and is listed in Cheema et al. (1976), Bowers et al. (1976), and Clyma et al. (1975) respectively. Additional unpublished data was collected on several other Mona project area watercourses by the Mona staff. The watercourse tube-well identification number is used to label the Mona data.

Two additional sets of steady state inflow-outflow data listed in Clyma et al. (1975) were analyzed. The first set was collected on several watercourses in the Lyallpur (Faisalabad) area and the second on one watercourse located near Multan.

Steady state flume inflow-outflow loss measurements were also analyzed from the five watercourses where operational loss studies were made. The data, which includes over 1500 steady state flume measurements will be labeled TW 81-R, for the right watercourse served by Tubewell #81 of the Mona Project area; Tik 1, for the first mogha serving Tikriwala Village near Faisalabad; MP 6, for the outlet D21000-L, Mianwali District; MP 35, for the watercourse serving Chak 31/BC in Bahawalpur District; and MP 52, for the watercourse served by outlet 8AR/52 near the city of Moro in central Sind

Province. The steady state data will not be listed because of the large amount involved. Only the results of statistical analyses will be given.

Loss Rate as a Function of Normal Inflow Rate

Three sets of available data involved sufficient numbers of watercourses that analysis of the relationship between the normally occurring inflow rate and steady state loss rates could be made. In the Survey and Mona data, several measurements were made on each watercourse, and the loss rate and normal inflow rate for each watercourse was taken as the arithmetic average of the measured inflow and loss rates. The Survey data included measurements on 31 watercourses with 5 to 12 measurements per watercourse. The Mona data included 10 watercourses.

The OFWM data involved only one measurement on each of 61 watercourses. It was assumed that the inflow rate during the measurement was the normal inflow rate. Fluctuations from the normal rate, which could not be determined, would tend to cause the intercept to be smaller in the linear regression and the exponent to be larger in the power curve regression.

The derived linear regression equations, listed in Table 7, of both the OFWM and Survey data indicate direct relationships between loss rate and normal inflow rate with a positive intercept. The Mona data exhibited no relationship between the two factors. Flume effect adjustments tended to improve the coefficient of determination (r^2) and decrease both the intercept and the slope.

Table 7. Derived linear and power curve regression equations relating steady state loss rates (Q_L) to the normal inflow rate (Q_M).

	Linear			Power Curve		
Watercourse	Regression Equation	r^2	Sig.*	Regression Equation	r^2	Sig.*
OFWM[1]	$Q_L = .134 + .010\ Q_M$	.491	> 99%	$Q_L = .034\ Q_M^{.742}$	.453	> 99%
OFWM-ADJ**	$Q_{L-A} = .119 + .009\ Q_M$	.503	> 99%	$Q_{L-A} = .029\ Q_M^{.753}$	.475	> 99%
Survey[2]-Means***	$Q_L = 1.10 + .046\ Q_M$	.463	> 99%	$Q_L = .256\ Q_M^{.644}$	.403	> 99%
Survey-ADJ-Means	$Q_{L-A} = .50 + .037\ Q_M$	.501	> 99%	$Q_{L-A} = .064\ Q_M^{.911}$	.474	> 99%
Mona[3]-Means		.003				

*Level of Significance.
**ADJ refers to data adjusted for flume effects by Equation 9.
***The mean loss rates, adjusted loss rates and inflow rates were used from each watercourse.
1/Data from On-Farm Water Management Project, 1978.
2/Data from Lowdermilk et al., 1978.
3/Primarily unpublished data.

It is reasonable to assume from knowledge of the physical system that the true relationship should tend to zero loss rate at zero normal flow rate. This would indicate the relationship really is curvilinear with a negative second derivative and a zero intercept.

A curve which fits this criteria is a curve described by an equation of the form:

$$Q_L = KQ^P \tag{14}$$

where K and P are a derived coefficient and exponent and P is less than 1.0. By logarithmically transforming both variables, the data was fit by linear regression techniques to such a power curve. The resulting equations are listed along with the linear regression equations in Table 7. The relationships are highly significant, although the r^2 values are slightly lower than the linear regression values. (Derived coefficients of determination cannot be compared directly between linear and transformed data.) The equations do exhibit exponents less than one, although adjusting the data tends to increase the exponent value. Figure 13 depicts the Survey data and the derived regression curves.

Loss Rate as a Function of Inflow Rate Fluctuations

Inflow rates fluctuated sufficiently during the measurement period on three operationally studied watercourses (MP6, MP35, and MP52), three Mona watercourses (TW51, TW56-L, and TW78) and several of the Survey watercourses, that the relationship between average loss rates over the

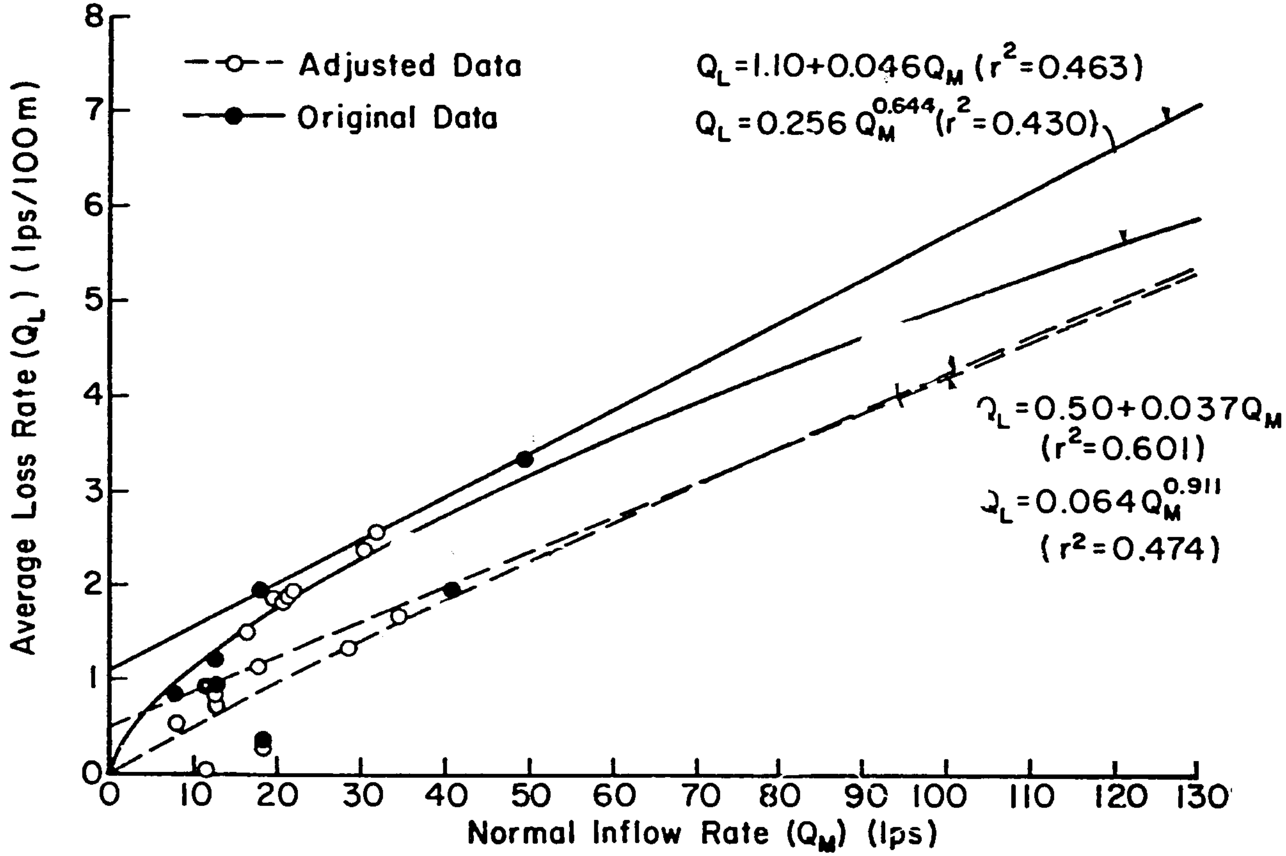


Figure 13. Average steady state loss rate and adjusted loss rate vs. average inflow rate for the Survey watercourses.

measured section and inflow rates could be analyzed. Linear regressions of all sets of data except the TW78 data indicated that the relationship is direct, with a negative intercept. The TW78 regression relationship was direct, but had a positive intercept. Table 8 lists the results of the analyses for the seven data sets.

The intercepts and coefficients listed in Table 8 for the Survey data are arithmetic means of the values from the watercourse relationships which had significance probabilities above 95 or 90 percent. Twenty percent and 38 percent respectively of the analyzed watercourses were significant at the two levels. The individual watercourse results are listed in the appendix. The TW51 and TW56 data involved measurements of the same channel section and length, which is why the r^2 values are high.

The consistently negative intercepts and positive slopes indicate that, since loss rates must approach zero at zero inflow rates, the true relationship is curvilinear with a positive second derivative (concave upwards). Hydraulic concepts to be presented in Chapter 6 indicate that the relationship should conform to an adjusted exponential in the range of flow rates near the normal inflow rate, so the loss rate data was logarithmically transformed and linearly regressed in order to fit it to an exponential model. For all three operationally studied and Mona watercourses and about 60 percent of the studied Survey watercourses, regression of the transformed data resulted in

Table 8. Derived linear and exponential regression equations relating steady state loss rates (Q_L) to changes in inflow rate (ΔQ_M).

Watercourse	Linear: Regression equation	Linear: r^2	Linear: Sig.*	Exponential: Regression equation	Exponential: r^2	Exponential: Sig.*
MP 6	$Q_L = -2.13 + .061(\Delta Q_M)$	.269	>99%	$Q_L = .0042e^{.037(\Delta Q_M)}$	.149	>99%
MP 6-ADJ**	$Q_{L-A} = -1.13 + .033(\Delta Q_M)$	.304	>99%	$Q_{L-A} = .0033e^{.083(\Delta Q_M)}$	.157	>99%
MP35	$Q_L = -6.75 + .220(\Delta Q_M)$	.480	>99%	$Q_L = .037e^{.091(\Delta Q_M)}$	.559	>99%
MP35-ADJ	$Q_{L-A} = -3.64 + .130(\Delta Q_M)$	.618	>99%	$Q_{L-A} = .017e^{.098(\Delta Q_M)}$	.622	>99%
MP52	$Q_L = -0.70 + .057(\Delta Q_M)$	.120	>99%	$Q_L = .400e^{.029(\Delta Q_M)}$	.140	>99%
MP52-ADJ	$Q_{L-A} = -0.58 + .045(\Delta Q_M)$	.214	>99%	$Q_L = .306e^{.031(\Delta Q_M)}$	.229	>99%
TW78[1]	$Q_L = 0.85 + .020(\Delta Q_M)$	.189	>90%	$Q_L = .82e^{.012(\Delta Q_M)}$	.153	>90%
TW78-ADJ	$Q_{L-A} = 0.74 + .014(\Delta Q_M)$	.184	>90%	$Q_{L-A} = .70e^{.011(\Delta Q_M)}$	.142	
TW51[2]	$Q_L = -0.35 + .013(\Delta Q_M)$	.855	>99%	$Q_L = .243e^{.013(\Delta Q_M)}$	.789	>99%
TW56[3]	$Q_L = -2.13 + .030(\Delta Q_M)$	.784	>99%	$Q_L = .022e^{.030(\Delta Q_M)}$	.967	>99%
Survey[4]-Ave.***	$Q_L = -1.46 + .117(\Delta Q_M)$		>95%	$Q_L = .46e^{.041(\Delta Q_M)}$		>95%
Survey-ADJ-Ave.	$Q_{L-A} = -1.00 + .081(\Delta Q_M)$		>95%	$Q_{L-A} = .39e^{.040(\Delta Q_M)}$		>95%
Survey-Ave.	$Q_L = -2.26 + .136(\Delta Q_M)$		>90%	$Q_L = .47e^{.040(\Delta Q_M)}$		>90%
Survey-ADJ-Ave.	$Q_{L-A} = -1.38 + .113(\Delta Q_M)$		>90%	$Q_L = .43e^{.041(\Delta Q_M)}$		>90%

*Level of significance.

**ADJ refers to data adjusted for flume effects by Equation 9.

***Average refers to the arithmetic average of the coefficients for all individual watercourse regressions with levels of significance above 95 percent and 90 percent respectively.

1/Data from Clyma et al., 1975.

2/Data from Cheema et al., 1976.

3/Data from Bowers et al., 1976.

4/Data from Lowdermilk et al., 1978.

higher coefficients of determination (r^2) than the linear regressions. The coefficient of the derived exponential equations have no physical meaning since it is known that Q_L must approach 0 at $Q_M = 0$. However, the exponent coefficient does not depend on the chosen origin and should indicate the fractional change in loss rate with a unit change in inflow rate. Derived exponential coefficients vary from .012 to .091 and average 0.04. This indicates an average 4 percent increase in loss rate with each liter per second increase in inflow rate.

By taking the ratio of the rate of change of both loss rate and inflow rate divided by their mean values for each watercourse, the ratio of the percent change in loss rates with a percent change in inflow rate from the mean can be calculated for both linear and exponential models. The average calculated values for the linear and exponential models are 1.65 and 1.92, respectively, indicating that loss rates increase or decrease 1.6 to 1.9 times as fast as inflow rates around the mean inflow rates.

Although the flume effect adjustment factor is not a function of flow rate, the adjustment when made in the data did consistently increase the coefficients of determination and probability of significance of the loss rate versus inflow rate fluctuation relationship for most watercourses. The adjustment however had no large nor consistent effect on the derived equation coefficients.

Figure 14 shows the data and derived linear and exponential regression lines for MP6 watercourse.

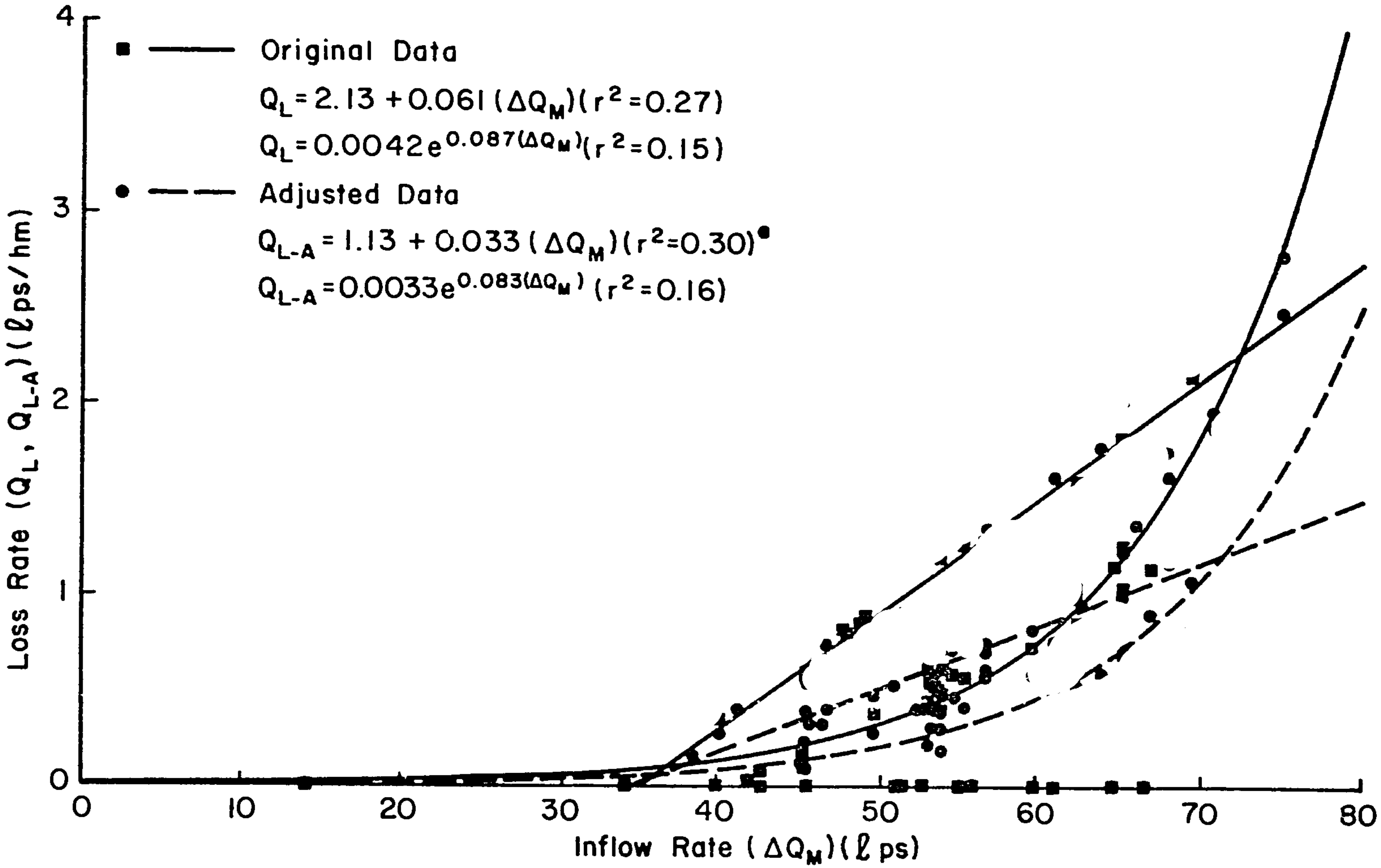


Figure 14. Steady state loss rate (Q_L) vs. inflow rate changes (ΔQ_M) for MP 6 watercourse including derived linear and exponential regression curves.

Loss Rate as a Function of Distance from the Mogha

The relationship between average loss rate per unit distance (lps/hm) and the length of the section was determined for the five operationally studied watercourses, TW78, Multan watercourse, the Lyallpur data, the OFWM data, and the Survey watercourses. Since each inflow-outflow measured channel section began at the mogha, the section length is also a measure of the average distance of the section from the watercourse head.

The analyses consistently indicate that loss rates decrease as distance from the mogha increases. The derived linear regression equations are given in Table 9. All of the equations are significant at the 95 percent level and indicate that loss rates decrease from 1 to 30 percent from the mean with each hectometer increase in section length. The Survey "average" listings in Table 9, as in the previous table, are arithmetic averages of the intercepts and coefficients of all regression equations significant at the 95 percent and 90 percent levels, respectively. The individual Survey watercourse regressions (listed in the Appendix) were significant at the 95 percent level on 16 percent of the measured watercourses, and significant at the 90 percent level on 38 percent of the watercourses.

The measured inverse relationship could result from:

1. flume effects which cause more losses in shorter sections,

Table 9. Average steady state loss rate (Q_L) as a function section length from the mogha (D).

Watercourse	Regression Equation Q_L(lps/100 m) D(100 m)	r^2	Sig.* (%)
TW81-R	Q_L=5.68-.184D	.658	> 99%
TW81-R-ADJ**	Q_{L-A}=4.07-.114D	.505	> 99%
TW81-R-SK	Q_L=3.37-.102D_{SK}	.274	> 99%
TW81-R-SK-ADJ	Q_{L-A}=2.04-.033D_{SK}	.053	
TIK 1	Q_L=2.32-.093D	.280	> 99%
TIK 1-ADJ	Q_{L-A}=1.53-.054D	.244	> 99%
TIK 1-SK	Q_L=1.69-.076D_{SK}	.276	> 99%
TIK 1-SK-ADJ	Q_{L-A}=1.06-.041D_{SK}	.207	> 99%
MP 6	Q_L=1.61-.030D	.029	> 95%
MP 6-ADJ	Q_{L-A}=0.65+.007D	.006	
MP 6-SK	Q_L=1.48-.050D_{SK}	.016	
MP 6-SK-ADJ	Q_{L-A}=0.83-.015D_{SK}	.006	
MP 35	Q_L=4.51-.304D	.150	> 99%
MP 35-ADJ	Q_{L-A}=2.61-.151D	.133	> 99%
MP 52	Q_L=2.60-.079D	.295	> 99%
MP 52-ADJ	Q_{L-A}=1.70+.041D	.232	> 99%
MP 52-SK	Q_L=8.74-.441D_{SK}	.229	> 99%
MP 52-SK-ADJ	Q_{L-A}=4.48-.209D_{SK}	.221	> 99%
TW 78[1]	Q_L=3.59-.134D	.531	> 99%
TW 78-ADJ	Q_{L-A}=2.63-.091D	.481	> 99%
Multan W/C[1]	Q_L=1.24-.030D	.163	
Multan W/C-ADJ	Q_{L-A}=0.79-.010D	.035	
Lyallpur Composite[1]	Q_L=2.09-.073D	.063	
Lyallpur Composite-ADJ	Q_{L-A}=1.35-.034D	.029	
OFWM-Composite[2]	Q_L=1.01-.009D	.090	> 95%
OFWM-Composite-ADJ	Q_{L-A}=0.86-.007D	.067	> 95%
Survey Composite[3]	Q_L=3.95-.122D	.018	> 95%
Survey Average***	Q_L=5.14-.233D		> 95%
Survey ADJ Average	Q_{L-A}=2.50-.079D		> 95%
Survey Average	Q_L=5.59-.446D		> 90%
Survey ADJ Average	Q_{L-A}=3.00-.136D		> 90%

*Level of significance.
**ADJ refers to data adjusted for flume effects by Equation 9.
***Average refers to the arithmetic average of coefficients for all individual watercourse regressions with levels of significance above 95 percent and 90 percent respective.
1/Data from Clyma et al., 1975.
2/Data from Punjab On-Farm Water Management Development Project, 1978.
3/Data from Lowdermilk et al., 1978.

2. the biasing effect of shorter sections tending to include a higher proportion of farmers' branches which tend to have higher loss rates;
3. decreasing flow rates with distance from the mogha and the direct relationship between flow rates and loss rates; and/or
4. a tendency for reduced seepage rates into channels which lie farther from the mogha.

In order to determine whether seepage rates really decreased with distance from the mogha, the effects of the first three factors must be cancelled out. First, the flume adjustment factor (Eq. 9) was applied to the data. This adjustment reduced the r^2 values and the coefficients of all the loss rate versus distance from the mogha (Q_L versus D) relationships. The flume adjustment generally reduced the coefficients relating Q_L to D by at least 40 percent, indicating that the relatively greater losses caused by flumes on short sections was a major factor in the derived Q_L versus D relationship.

Second, the loss rates in different lengths of only sarkari khal sections were linearly regressed with section length on four watercourses: TW81-R, Tik 1, MP 6 and MP 52. In all four cases, the r^2 value was reduced from the value derived with both sarkari khal and farmers' branch sections included. The coefficient relating Q_L to D was reduced by one-third to one-half compared to values derived utilizing flow in both types of sections on TW81-R and Tik 1 watercourses, but on MP 52 and MP 6 watercourses, very high loss rates measured in short sarkari khal sections caused the coefficient to increase. When both measurement biases are eliminated the slope coefficients on three of the water-courses are reduced to less than 50 percent of their initial value, but a significant inverse relationship still exists between Q_L and D.

In order to analyze the effect of decreasing flow rates with distance on loss rates, two relationships were assumed: one where loss rate is proportional to flow rate, and the other where flow rate has no effect on loss rate. Since the data were collected on fairly long sections which began at the mogha, the two relationships were integrated so that measured conveyance efficiencies could be correlated with the section length.

If loss rate (Q_L = dQ/dD) is proportional to flow rate (Q), then:

$$\frac{dQ}{dD} = -K_1 Q. \qquad (15)$$

Integrating:

$$\int_{Q_O}^{Q_F} \frac{dQ}{Q} = \int_{O}^{D} -K_1 dD,$$

$$\ln Q_F - \ln Q_O = -K_1 D,$$

$$Q_F/Q_O = e^{-K_1 D}, \qquad (16)$$

where:

$\frac{dQ}{dD}$ = change of flow rate with distance, or loss rate,

K_1 = instantaneous fractional loss rate (hm^{-1}),

Q_o = initial flow rate at D = 0 (lps),

Q_F = final flow rate at distance D (lps), and

Q_F/Q_o = steady state conveyance efficiency (E).

If flow rate is assumed not to affect loss rates, then:

$$\frac{dQ}{dD} = -K_2. \tag{17}$$

Integrating:

$$\int_{Q_o}^{Q_F} dQ = \int_0^D -K_2 dD,$$

$$Q_F - Q_o = -K_2 D,$$

$$(Q_F - Q_o)/Q_o = -(K_2/Q_o)D,$$

$$Q_F/Q_o = 1-(K_2/Q_o)D, \tag{18}$$

where:

K_2 = loss rate (lps/hm), and

K_2/Q_o = loss rate as a fraction of initial flow rate (hm^{-1}).

Both conveyance efficiency and conveyance efficiency adjusted for flume effects data were regressed both linearly and exponentially with distance (D) to determine which of Eqs. 15 or 17 best described the data. The results of the regression analyses are given in Table 10. The two models described the data equally well with each giving higher r^2

Table 10. Derived linear and exponential regression equations relating conveyance efficiency (E) to distance from the mogha (D).

Watercourse	Linear Regression Equation	r^2	Sig.*	Exponential Regression Equation	r^2	Sig.*
TW 81-R	$E=.58-.005D$	.027		$E=.62e^{-.020D}$	.046	
TW 81-R-ADJ**	$E_A=.70-.009D$	.075		$E_A=.73e^{-.019D}$	.078	
TW 81-R-SK	$E=.91-.018D_{SK}$	.194	95%	$E=1.02e^{-.035D_{SK}}$	.154	> 95%
TW 81-R-SK-ADJ	$E_A=.96-.018D_{SK}$	.242	95%	$E_A=1.03e^{-.029D_{SK}}$	.190	> 95%
Tik 1	$E=.81-.005D$	.044	99%	$E=.80e^{-.006D}$	.035	> 95%
Tik 1-ADJ	$E_A=.88-.007D$	.124	99%	$E_A=.88e^{-.009D}$	.122	> 99%
Tik 1-SK	$E=.87-.003D_{SK}$	.026	95%	$E=.86e^{-.003D_{SK}}$	.018	
Tik 1-SK-ADJ	$E_A=.92-.004D_{SK}$	.096	99%	$E_A=.92e^{-.005D_{SK}}$	.085	> 99%
MP 6	$E=.93-.013D$	.242	99%	$E=.98e^{-.023D}$	.231	> 99%
MP 6-ADJ	$E_A=1.01-.013D$	.431	99%	$E_A=1.05e^{-.019D}$	.396	> 99%
MP 6-SK	$E=.99-.012D_{SK}$	.269	99%	$E=1.02e^{-.017D_{SK}}$	.325	> 99%
MP 6-SK-ADJ	$E_A=1.01-.011D_{SK}$	.325	99%	$E_A=1.04e^{-.016D_{SK}}$	.301	> 99%
MP 35	$E=.92-.011D$	.168	99%	$E=.92e^{-.015D}$	.178	> 99%
MP 35-ADJ	$E_A=.98-.012D$	.314	99%	$E_A=.99e^{-.015D}$	.329	> 99%
MP 52	$E=.77-.012D$	.516	99%	$E=.80e^{-.023D}$	.490	> 99%
MP 52-ADJ	$E_A=.86-.013D$	.665	99%	$E_A=.89e^{-.023D}$	.637	> 99%
MP 52-SK	$E=.81-.012D_{SK}$	.398	99%	$E=.82e^{-.022D_{SK}}$	.377	> 99%
MP 52-SK-ADJ	$E_A=.90-.015D_{SK}$	.619	99%	$E_A=.93e^{-.023D_{SK}}$	.583	> 99%
TW 78[1]	$E=.75-.007D$	.085		$E=.74e^{-.010D}$	.090	
TW 78-ADJ	$E_A=.84-.009D$	.208	95%	$E_A=.84e^{-.012D}$	.231	> 99%
Multan W/C[1]	$E=.95-.011D$	.381	99%	$E=.95e^{-.014D}$	.344	> 99%
Multan W/C-ADJ	$E_A=.97-.011D$	.449	99%	$E_A=.98e^{-.013D}$	.417	> 99%
Lyallpur Composite[1]	$E=.87-.012D$	.165	99%	$E=.87e^{-.019D}$	.150	> 99%
Lyallpur Composite-ADJ	$E_A=.94-.014D$	.276	99%	$E_A=.95e^{-.019D}$	.259	> 99%
OFWM-Composite[2]	$E=.80-.004D$	.216	99%	$E=.80e^{-.006D}$	.214	> 99%
OFWM-Comp.-ADJ	$E_A=.83-.004D$	.242	99%	$E_A=.83e^{-.005D}$	.239	> 99%
Survey[3] Average	$E=.75-.029D$		95%	$E=.84e^{-.071D}$		> 95%
Survey ADJ Ave.	$E_A=.87-.034D$		95%	$E_A=.88e^{-.048D}$		> 95%
Survey Average	$E=.76-.034D$		90%	$E=.80e^{-.079D}$		> 90%
Survey ADJ Ave.	$E_A=.85-.030D$		90%	$E_A=.89e^{-.050D}$		> 90%

*Level of significance.
**ADJ refers to data adjusted for flume effects by Equation 10.
***Average refers to the arithmetic average of the coefficients for all individual watercourse regressions with levels of significance above 95 percent and 90 percent respectively.
1/Data from Clyma et al., 1975.
2/Data from Punjab On-Farm Water Management Project, 1978.
3/Data from Lowdermilk et al., 1978.

values for some of the cases. This implies either that loss rate is between being constant and proportional to flow rate, or that there is some other factor which tends to increase seepage rates at longer distances from the mogha and counters the effect of decreasing flow rates. Both ponding and inflow-outflow data analysis indicated that loss rate does in fact vary with inflow rate but less than proportionally. Figure 15 depicting the data and derived equations for watercourse MP 52 illustrate that the variability between values predicted by the two models is not great in the applicable distance ranges.

This finding indicates that decreasing flow rates could explain a portion of the tendency for loss rates to decrease with distance but does not prove the effect is significant. If loss rates were related to flow rates by the power curve given in Eq. 14, with the exponent, P, varying between 0.7 and 0.9, loss rates would decrease about 0.02 to 0.04 lps/hm for each 100 m decrease in D.

One unexpected but consistent finding is that both the derived linear and exponential equations relating conveyance efficiency to distance predict efficiencies significantly below one at zero distance. The average derived intercept value from the linear correlation is 0.81, while the intercept of the exponential model averages slightly higher (as would be expected) at 0.83. The fact that the intercepts are significantly below one indicates that the loss rates would decrease with distance even if conveyance efficiencies

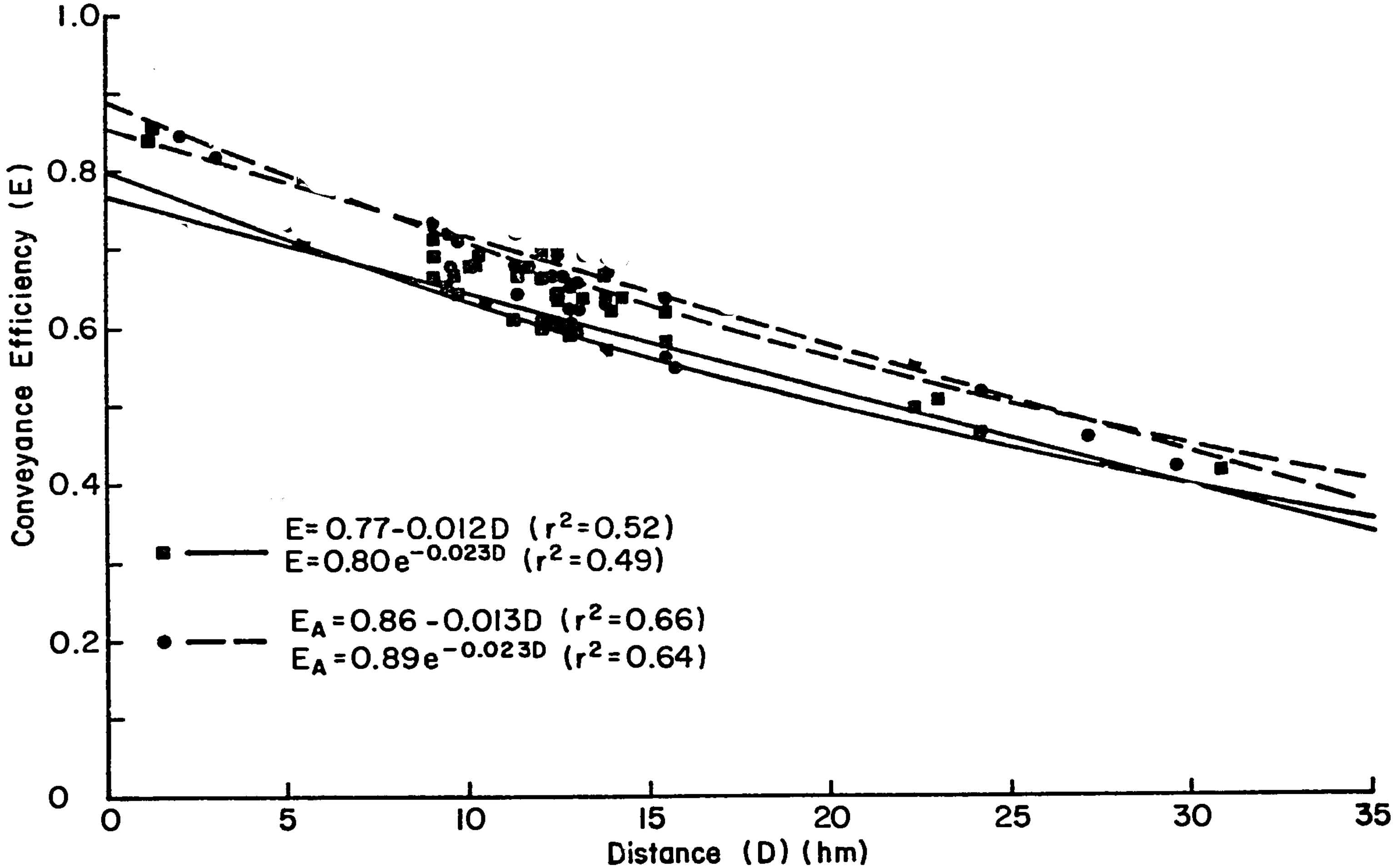


Figure 15. Conveyance efficiency (E) and conveyance efficiency adjusted for flume effects (E_A) vs. conveyance distance (D) for MP 52 watercourse.

do decrease linearly with distance. The slope of a line connecting the value 1.0 at the intercept and a point at a distance, D, on the line depicting conveyance efficiency versus distance will give the negative of the average fractional loss rate per hectometer to that distance. Flume effects would tend to cause the intercepts to be less than one, and when the data is adjusted for flume induced losses, the mean intercepts are raised to 0.87 and 0.90 for the linear and exponential models, respectively. When the third measurement biasing, that of shorter section lengths containing a higher proportion of farmer's branch, is also eliminated by considering loss rates only on sarkari khal sections (as was done for TW81-R, Tik 1, MP 6, and MP 52 watercourses) the intercepts again increase and average 0.95 and 0.98 for the linear and exponential cases.

In summary, the indication of the inflow-outflow data that loss rates decrease with distance is concluded to be primarily an artifact caused by the projected conveyance efficiencies at 0 distance being less than 1.0. Flume effects and channel type (sarkari khal versus farmer's branch) biases in the measurement process is seen to explain much of this result and also to reduce to degree and significance of the loss rate-distance relationship. The decreasing flow rates with distance which should, according to the finding in the previous section, tend to cause loss rates to decrease with distance, can explain a portion of the remaining decrease. The portion of the inverse relationship left unexplained on some watercourses could be the result of a high loss rate in the initial sections below the mogha. In the light of the derived relationships shown in Table 10, it is improbable that seepage rates into watercourse wetted perimeters decrease with distance of the channel section from the mogha.

Loss Rate as a Function of Channel Slope and Elevation Drop

An additional factor which could tend to affect steady state loss rates which was monitored during the operational studies, is the elevation drop from the mogha to the field. The factor will reflect both average slope of the water channel to the field, and flow depth changes in a channel resulting from the backwater effects of a relatively high or low field being irrigated. Table 11 lists linear regression equations derived for the five operationally studied watercourses relating loss rate to average slope from the mogha to the field, S (m/km), and elevation drop from the mogha to the field, EL (m). Since distance from the mogha to the field, D, is, as would be expected, highly correlated with El, and loss rate is inversely related to D, the distance factor was added to the correlations to separate out the indirect effects of the distance factor.

Although loss rates consistently decrease as the elevation drop to the field increases, most of this result on three of the five watercourses is seen to primarily result from the intercorrelation with distance, since adding the EL factor to the relationship between Q_L and D tends to increase the r^2 value very little.

Table 11. Derived linear regression equations describing the relationship between loss rate (Q_L) and slope (S), elevation drop (EL), and distance (D), for the five operationally studied watercourses.

Watercourse	Independent Variable	Regression Equation	r^2	Sig.*
TW 81-R	S (m/km)	Q_L=4.27-2.88S	.35	> 99%
	EL (m)	Q_L=4.17-0.113EL	.66	> 99%
	D (hm)	Q_L=4.18-0.116D	.49	> 99%
	S and D	Q_L=6.40-2.46S-.125D	.69	> 99%
	EL and D	Q_L=4.17-0.133EL+.003D	.66	> 99%
Tik 1	S (m/km)	Q_L=0.91+0.245S	.003	
	EL (m)	Q_L=4.73-0.731EL	.186	> 99%
	D (hm)	Q_L=2.51-0.135D	.275	> 99%
	S and D	Q_L=2.01+0.770S+0.106D	.307	> 99%
	EL and D	Q_L=7.68-1.796EL-0.161D	.294	> 99%
MP 6	S (m/km)	Q_L=1.20+0.280S	.008	> 90%
	EL (m)	Q_L=1.78-0.670EL	.051	> 99%
	D (hm)	Q_L=2.19-0.061D	.102	> 99%
	S and D	Q_L=2.13+0.093S-0.061D	.103	> 99%
	EL and D	Q_L=2.25-0.274EL-0.061D	.109	> 99%
MP 35	S (m/km)	Q_L=1.37+0.984S	.037	> 99%
	EL (m)	Q_L=6.32-8.037EL	.173	> 99%
	D (hm)	Q_L=4.51-0.304D	.150	> 99%
	S and D	Q_L=5.66-0.696S-0.365D	.160	> 99%
	EL and D	Q_L=6.93-6.089EL-0.213D	.236	> 99%
MP 52	S (m/km)	Q_L=0.37+1.438S	.639	> 99%
	EL (m)	Q_L=2.11-0.578EL	.017	> 95%
	D (hm)	Q_L=3.16-0.091D	.290	> 99%
	S and D	Q_L=0.68+1.355S-0.030D	.643	> 99%
	EL and D	Q_L=3.04+0.187EL-0.121D	.292	> 99%

*Level of significance.

The slope factor is directly related to loss rates for four of the five watercourses, although the one inverse relationship is highly significant. Again, only in two of the five cases does slope add significantly to the predictability of Q_L after D has already been added.

It is concluded that local relatively high fields tend to increase loss rates, probably the result of heading up the water in the channel, but that the overall effects of the topography on loss rates are more complicated than can be described by slope or elevation drop.

Loss Rates in Sarkari Khal and Farmers' Branches

Steady state loss rates were measured in both sarkari khals and farmers' branches in all five operational studies. Table 12 lists the time weighted average loss rates in both sections for each watercourse. In four of the five watercourses, farmers' branch loss rates are larger than sarkari khal loss rates. The mean loss rates in the sarkari khal channels and farmers' branches were 1.2 and 2.4 lps/hm, respectively, although the difference is strongly influenced by the large difference on MP 6 watercourse which was very sandy. On the other four watercourses, farmers' branch loss rates averaged 50 percent larger than sarkari khal loss rates. Since inflow rates in farmers' branches are less because of the losses in sarkari khal sections, the difference in percent losses per unit length in the sarkari khal and farmers' branches are even larger. Flume-induced losses,

Table 12. Time weighted average steady state loss rates (Q_L) on sarkari khal and farmers' branch sections of the five operationally studied watercourses.

Watercourse	Steady State Loss Rates (lps/hm) Sarkari Khal	Farmer's Branch
TW81-R	1.9	2.9
Tik 1	0.6	1.6
MP 6	0.8	4.9
MP 35	1.7	1.1
MP 52	1.1	1.4
Average	1.2	2.4

which would have increased this difference because of the relatively shorter farmers' branch sections measured, were not adjusted for.

Operational Measurements of Transient Losses

Operational conveyance loss studies were undertaken on five watercourse systems. Table 13 summarizes the transient losses and dead storage on each watercourse. An average of 6.8 percent of the total inflow was lost to transient conditions, with a range of only 5.7 to 8.4 percent.

Much of the variability in the transient losses is explained by the total lengths of channels filled and drained and the average inflow rate to each watercourse. Table 14 lists regression equations for each watercourse between transient loss volumes, V_{TL} (m^3), for each field irrigated and the length of channel filled, L_W (m), and drained, L_D (m), in the process of irrigating a field. Between half and three-fourths of the variability in transient losses on a watercourse can be explained in terms of these lengths.

Table 13. Transient losses on five watercourse systems during one warabundi turn rotation.

	Watercourse					
	TW 81-R	Tik 1	MP 6*	MP 35*	MP 52*	Average
Inflow volume (m^3)	43,000	25,000	34,100	23,300	20,000	29,100
Channel length utilized (filled and drained) (m)	12,500	21,650	11,650	9,600	10,000	12,800
Steady state losses (%)	48	28	33	33	45	37
<u>Transient Losses</u>:						
Total volume (m^3)	2975	2086	2296	1271	1271	1980
Percent of inflow (%)	6.9	8.4	6.8	5.7	6.3	6.8
Percent of total losses (%)	12.4	22.8	17.0	17.4	12.6	16.4
Per channel length (m^3/m)	0.238	0.096	0.197	0.132	0.127	0.163
Per channel length per unit inflow ($m^3/m/(m^3/sec)$)	3.35	2.32	3.50	3.43	3.78	3.35
<u>Dead Storage</u>:						
Total volume (m^3)	1308	1283	494	827	1073	997
Percent of inflow (%)	3.0	5.1	1.4	3.6	5.3	3.7
Percent of total losses (%)	5.5	14.0	6.8	10.9	9.8	9.4
Percent of transient losses (%)	44.0	61.5	17.0	65.0	88.1	55.1
Per channel length (m^3/m)	0.116	0.059	0.040	0.086	0.101	0.080
Per channel length per unit inflow ($m^3/m/(m^3/sec)$)	1.63	1.43	0.71	2.23	3.01	1.80

*Values are the average of three weeks (three turn rotations) of data collection.

Table 14. Regression equations describing the relationship between transient losses (V_{TL}) and the length of channel filled (L_W) and drained (L_D) to irrigate each field.

Watercourse	Regression equation	Coefficient of determination (r^2)
TW81-R	$V_{TL} = -15.0 + 0.31\ L_W - 0.10\ L_D$	0.58
Tik 1	$V_{TL} = 3.1 + 0.14\ L_W - 0.06\ L_D$	0.52
MP 6	$V_{TL} = 0.3 + 0.21\ L_W - 0.04\ L_D$	0.59
MP 35	$V_{TL} = 2.6 + 0.16\ L_W - 0.07\ L_D$	0.77
MP 52	$V_{TL} = 21.0 + 0.15\ L_W - 0.08\ L_D$	0.70

Each regression coefficient and the difference between the two coefficient values were then regressed with the time weighted average inflow rate, Q_M (lps), to the watercourse. The resulting predicted relationships were:

$$b_{LW} = -.03 + .0047\ Q_M \qquad (r^2 = .94),$$

$$b_{LD} = -.05 - .0005\ Q_M \qquad (r^2 = .10),$$

$$\Delta b_L = -.08 + .0042\ Q_M \qquad (r^2 = .96),$$

where:

b_{LW} = the L_W regression slope coefficient (from Table 14) which is the volume of loss per unit length of channel filled (m^3/m),

b_{LD} = the L_D regression slope coefficient (from Table 14) which is the volume of water regained per unit length of channel drained (m^3/m), and

Δb_L = $b_{LW} + b_{LD}$ = the volume of water loss per length of channel filled and drained (m^3/m).

The intercept of the first equation, which has a very high r^2 value, is relatively small compared to the second term, thereby indicating that the transient loss per unit length of channel filled is nearly proportional to the flow rate. The intercept value of the poorly correlated second equation is relatively large compared to the second term, indicating that the water regained by draining a certain length of channel is strongly affected by factors other than the inflow rate. For example, it will be affected by the local topography which influences how much of the channel storage water drains into the field and what portion remains in the channel as dead storage.

Since during a complete rotation turn on a watercourse the length of channel filled and drained will be equal, the third equation will be most useful in predicting watercourse transient losses. According to the highly correlated regression equation, watercourse transient losses can be estimated by:

$$V_{TL} = (-.08 + .0042\ Q_M)L_{DW} \qquad (19)$$

where:

V_{TL} = transient losses (m^3),

Q_M = normal inflow rate (lps), and

L_{DW} = length of channel filled and drained.

Because the constants of the regression equations listed in Table 14 were ignored in developing this equation, it underestimates the true transient losses, listed in Table 13, by from 0 to 40 percent with an average of 20 percent. The linear equation is not valid at low flow rates (< 20 lps) where no transient losses are predicted.

Additional variation between values given in Table 13 for transient loss per meter length per lps inflow for each watercourse can be explained by the difference in steady state loss rates for each watercourse. As the steady state loss rate increases by one percentage point, the average slope coefficient value of Eq. 19 increases by 1½ percent of its value.

An average of 55 percent of the transient losses were dead storage on the five watercourses. Watercourse MP 6 was a sandy watercourse with high intake rates in the farmers' branches, allowing much of the dead storage water to seep away before measurements could be made, so the MP 6 value is an underestimation of the true dead storage. Generally, less than 5 percent of the watercourse inflow is lost to dead storage, which would usually amount to less than 7 percent of the water delivered to the farmer's branch. Dead storage cannot be explained as well as transient losses in terms of lengths of channel utilized, inflow rate, or loss rate. It is highly dependent upon the topography of the watercourse command area. If slopes are high and fairly uniform, most water will drain from the channels into the fields.

Chapter 6

HYDRAULIC ANALYSIS OF WATERCOURSE FLOW

Several water channel design factors can be interrelated through hydraulic open channel flow design equations. Manning's equation (Manning, 1891) given below, which has been used satisfactorily in designing Indus Basin watercourses, will be used.

$$Q = \frac{1}{n} AR^{2/3}S^{1/2} \tag{19}$$

where:

Q = flow rate (m^3/sec),

n = roughness coefficient,

A = cross-sectional flow area (m^2),

R = hydraulic radius = A/WP(m),

WP = wetted perimeter (m), and

S = slope of the energy line (water surface)(m/m).

Manning's equation can be rearranged so that:

$$\frac{Qn}{\sqrt{S}} = AR^{2/3} \tag{19a}$$

The parameters on the right depend only on the cross-sectional shape and size.

If a channel's cross-sectional shape is known, the flow depth, d, can be related to the cross-sectional parameters, $AR^{2/3}$, and thus to the flow rate (Q), roughness coefficient (n), and slope (S). For several optimum (minimum wetted perimeter) cross-sectional shapes, an explicit relationship between depth and $AR^{2/3}$ can be developed, and is of the form:

$$d = a(AR^{2/3})^{3/8} , \tag{20}$$

where a is a coefficient dependent on the cross-sectional shape. As an example, for a triangular channel with 1:1 side slopes, a = 1.30.

In these channels, the width increases with the depth. For cross-sectional shapes where depth is not related explicitly to the other shape factors, which is the situation if flow depth is increased in an existing channel, the relationship between d and $AR^{2/3}$ cannot be explicitly determined, but can be implicitly estimated. Such estimations were made using regression techniques to fit calculated data to a power curve (as suggested by Eq. 20) of the form:

$$d = a(AR^{2/3})^{c} = a\left(\frac{Qn}{\sqrt{S}}\right)^{c} \tag{21}$$

Several cross-sectional shapes were studied including circular segments, trapezoids, shapes described by the power curve given in Eq. 1 with various coefficient values, and actual cross-sectional channel shapes measured in the field. In all the cases tested, the coefficient, a, varied between 1.15 and 1.40 and the exponent, c, varied from 0.44 to 0.55. For the cross-sectional shape shown in Figure 3, the a and c values are 1.16 and 0.53, respectively. The equation with these coefficients describes the actual calculated data with an r^2 of 0.9999.

Thus, through manipulation of Manning's equation, an accurate relationship can be developed for typical watercourse cross-sectional shapes, which relates the flow depth to flow rate, roughness coefficient, and slope. Such a relationship is shown in Figure 16 for $a = 1.2$ and $c = 0.5$ in Equation 21.

Equation 21 can then be combined with Eq. 11 to develop a relationship between the change in loss rate with changes in Q or n.

$$\frac{Q_L}{Q_{LO}} = e^{b(\Delta d)} = e^{b(d-d_o)} = \exp\left\{ba\left[\left(\frac{Qn}{\sqrt{S}}\right)^c - \left(\frac{Q_o n_o}{\sqrt{S}}\right)^c\right]\right\} \quad (22)$$

where:

d_o = original depth (cm) with $Q = Q_o$ and $n = n_o$,

Q_o = original flow rate (m^3/sec),

n_o = original roughness coefficient, and

b is defined in Eq. 11.

Equation 22 with three slope values and four values for $Q_o \times n_o$ is shown graphically in Figure 17. The extreme sensitivity of loss rates to fluctuations in flow rates and channel roughness is evident from the graph which indicates that loss rates can double with a 30 percent increase in Q or n.

Figure 18 illustrates the sensitivity of Eq. 22 to variations in the coefficients a, c, and b. The small deviation of the dashed lines representing $c = 0.45$ and $c = 0.55$ from the median value of 0.50 shows that the equation is not very sensitive to the c value through its range. Lines representing b x a values of 0.10 to 0.30 however are widely spread. Since the common range of a is between 1.15 and 1.4,

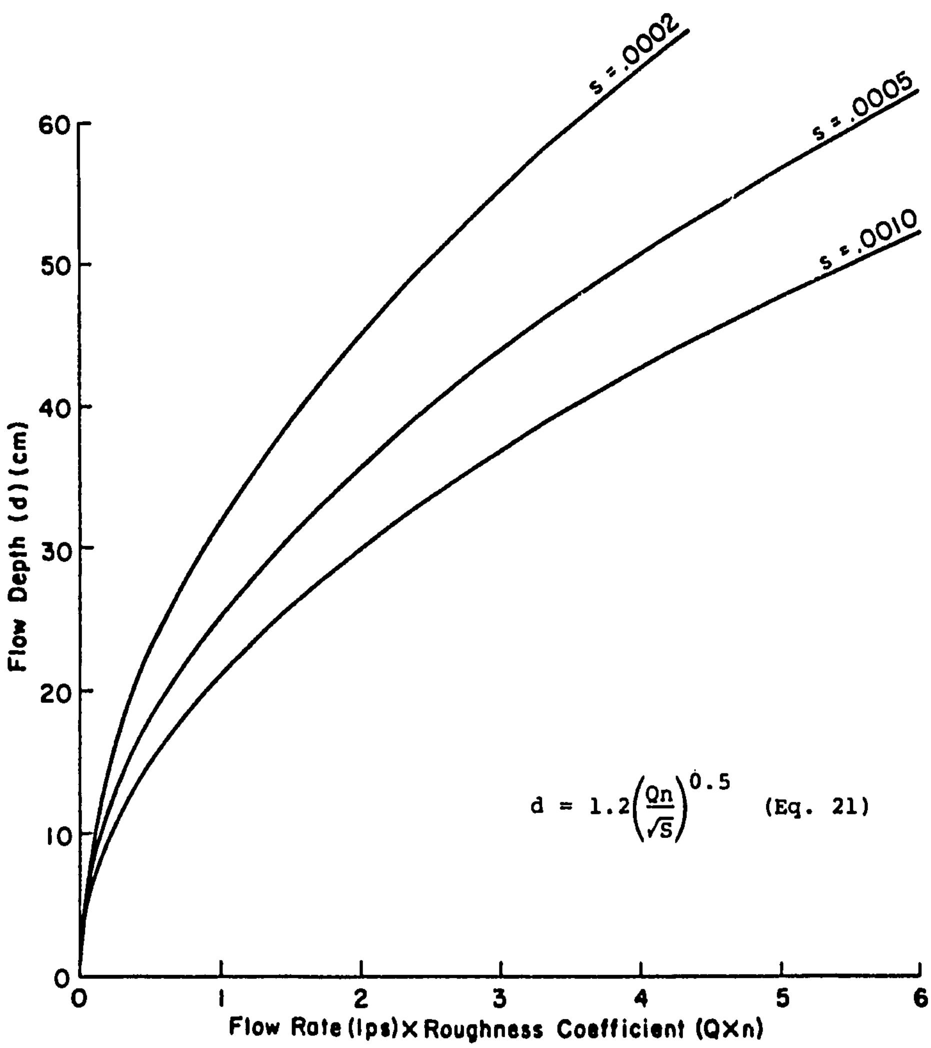


Figure 16. Flow depth (d) as a function of flow rate times roughness coefficient (Qxn).

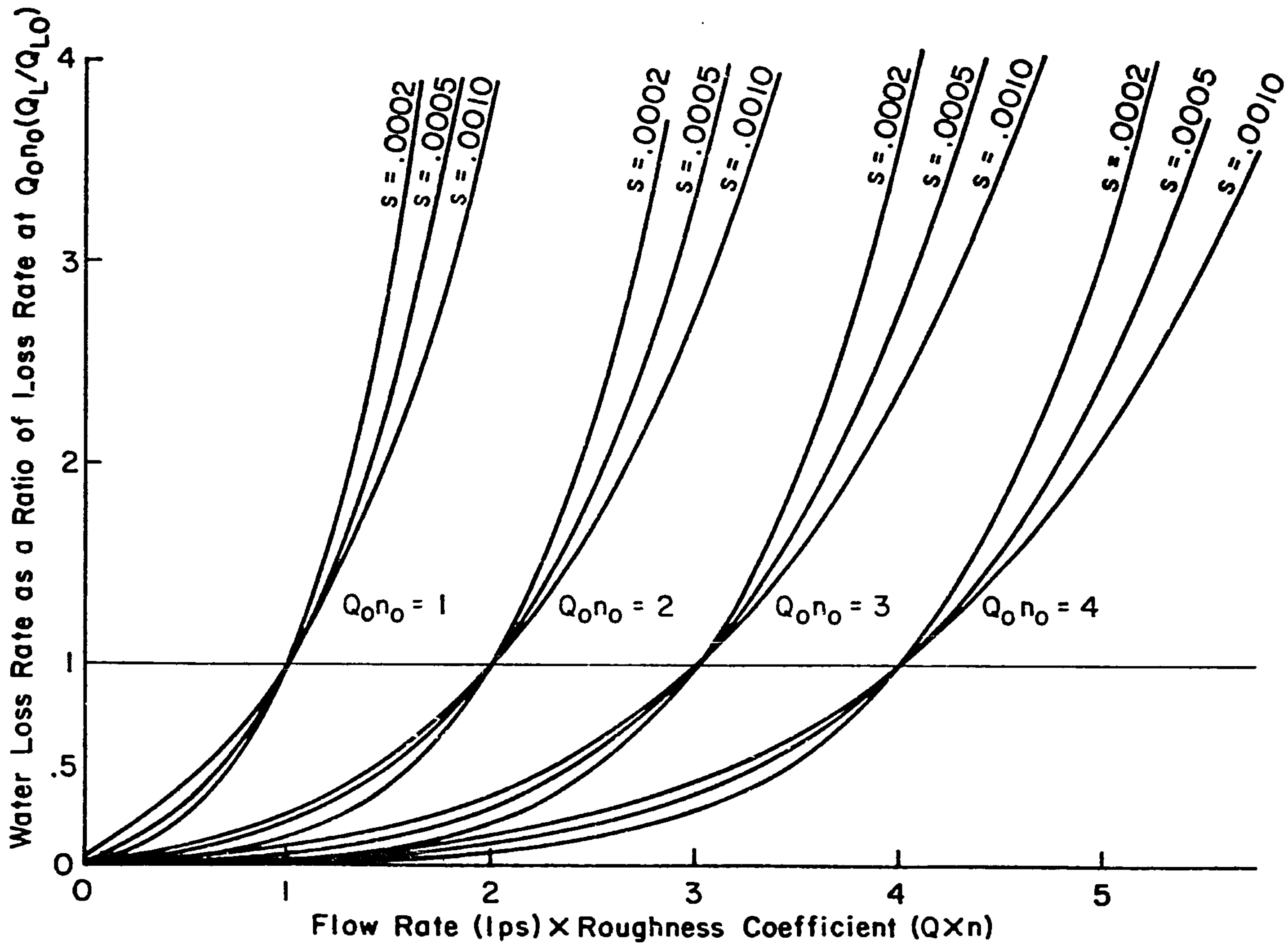


Figure 17. Relative change in loss rates with a short-term relative change in the product of flow rate and roughness coefficient (from Equation 22 with a = 1.2, b = 15, and c = 0.5).

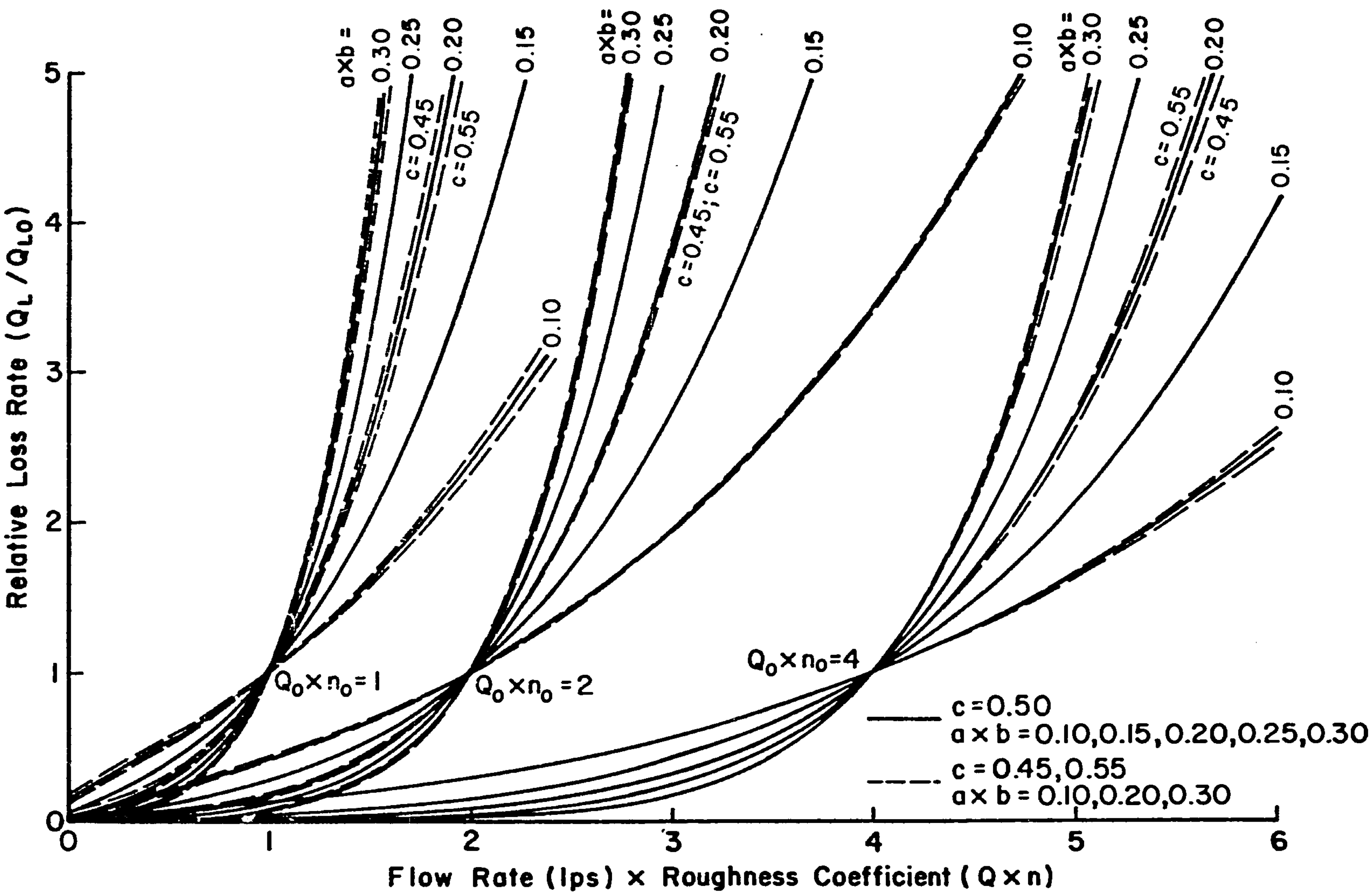


Figure 18. Sensitivity of Equation 22 to variations in a, b, and c.

if a misestimation half its range were made, the resulting error in b x a would be from 0.01 to 0.04 which could result in an error in the predicted loss rate by as much as 30 percent.

The b value, which averaged 0.15, had a standard deviation of .07 in the ponding tests. Misestimation of b by one standard deviation will usually lead to more than a 50 percent error in the estimated loss rate and can cause up to a 100 percent error. Equation 22 is sensitive to b value variations, but relatively insensitive to the cross-sectional shape parameters.

A variation in slope, S, from .0005 to .0002 or .0010 depicted in Figure 17 is equivalent to a 25 percent change in b x a and will cause up to a 30 percent error in the loss rate estimate.

The changes in roughness and flow rates referred to here are fluctuations over short periods of time. It would be assumed that a watercourse channel would evolve over time to long-term changes. For example, if a decision is made to permanently split a water supply into two channels, the steady state loss rates would initially be drastically reduced (more than proportionally) as depicted in Figure 17. However, this new flow rate would, over time, establish a new normal flow depth to which the watercourse would evolve, and the eventual loss rate decrease would probably be governed primarily by the relationship between loss rate and normal flow rate described by Eq. 14, which is less than proportional.

Chapter 7

ANALYSIS OF THE FINDINGS

Several of the factors which might affect the amount of water which is lost from an earthen watercourse in the process of conveying it from the inlet mogha to the field have been measured and analyzed. The analyses indicated some of the parameters which do affect losses, tell the direction and sometimes the degree (or slope) of the interaction, and occasionally indicate the shape of a nonlinear relationship. In this chapter, physical explanations will be given for the derived relationships and an initial attempt will be made to construct a model to estimate watercourse losses based on the findings. The scatter in the data and the relatively low coefficients of determination of the derived relationships indicate that factors in addition to those quantified affect loss rates.

Factors which Affect Loss Rates at the Usual Flow Depth

Both the ponding and inflow-outflow data indicate that loss rates are higher in watercourses which normally carry larger quantities of water. The linearly regressed ponding data indicated the relationship is slightly less than proportional with loss rates increasing between 75 percent and 100 percent as fast as normal inflow rates. Inflow-outflow measured loss rates increased 75 percent to 90 percent as fast as normal inflow rates in the range of the mean inflow rate.

A power curve relationship has two advantages over the linear model. It approaches zero at 0 flow rate as the physical system must do, and its exponent is equal to the percent change of the dependent variable with a percent change of the independent variable over a complete range. Because of these advantages, the relationship will be modeled by a power curve of the form:

$$Q_L = KQ^P , \tag{14}$$

where:

K = a constant which can be derived given Q_L, Q_M, and P values

P = the percent change in Q_L with a one percent change in Q_M

Figure 19 depicts conveyance efficiency vs. distance for a channel where loss rates are described by Eq. 14 when the initial loss rate - 1.0 lps/hm, inflow - 40 lps, and the P value is varied from 0 to 1.0.

Most water channel loss studies in the past have used the wetted perimeter length to represent channel size and have assumed intake rates to be constant across the wetted perimeter (see Chapter 3). Equation 2 indicates that water-course wetted perimeters only increase about half as fast as loss rates increase with flow rates, since the exponent of that derived power curve relationship is 0.4. This means that intake rates into the wetted perimeters of larger channels must be higher than into the wetted perimeters of small capacity channels. Specific reasons for this observed

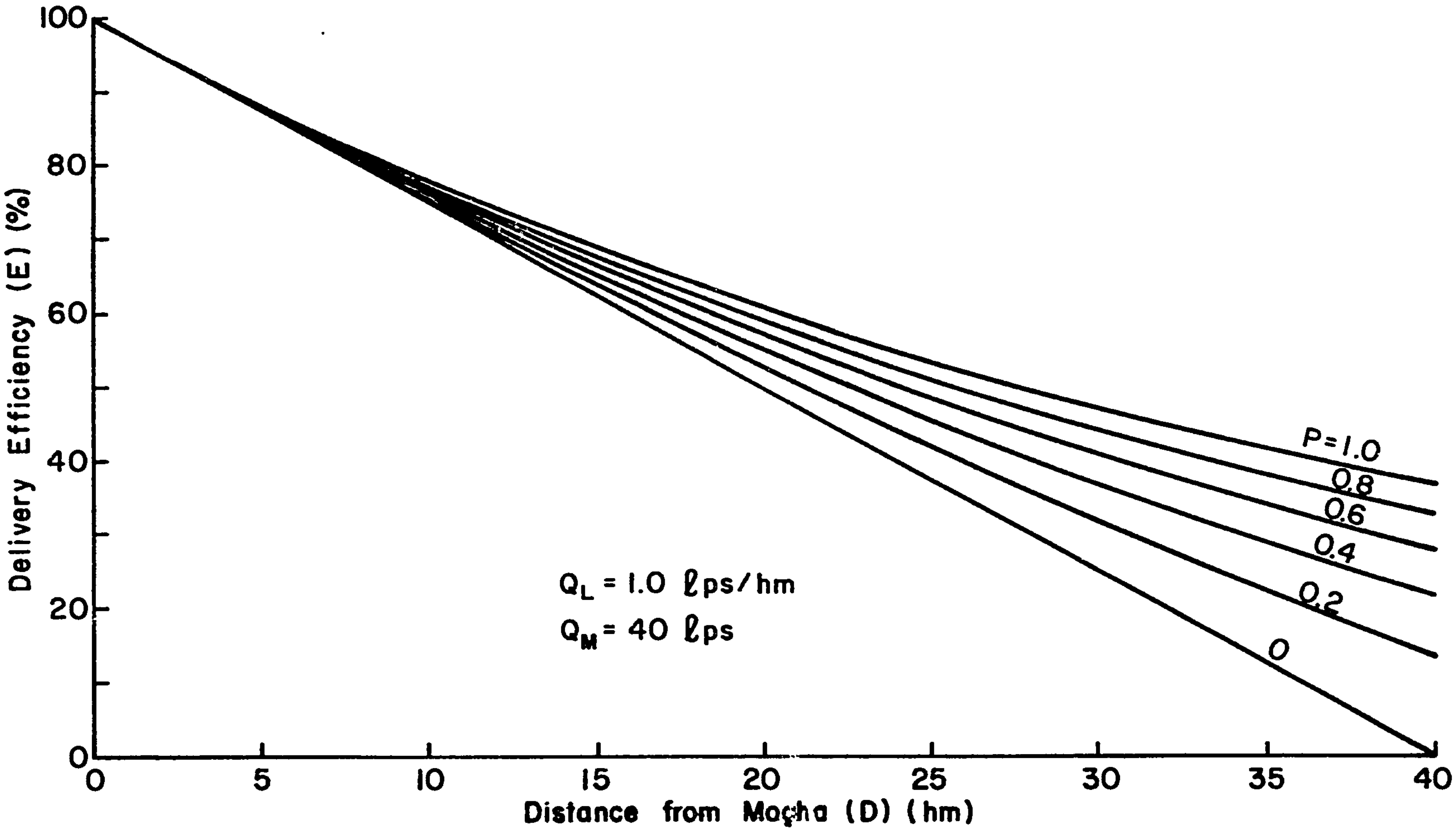


Figure 19. Delivery efficiency (E) vs. conveyance distance (D) for different P values where $\frac{dQ}{dD} = KQ^P$ (Equation 14).

relation were not determined, but possible explanations include:

1. farmers on larger watercourses do not maintain their watercourses as well, and
2. larger flow rates result in less silt deposition.

The inflow-outflow data presented in Table 10 indicate that the relationship between conveyance efficiency and distance is explain d equally well by a linear (P = 0) or exponential (P = 1) model. The true relationship probably lies somewhere between these two extremes. The relationships described by Eq. 14 with P values between 0 and 1 and depicted in Figure 19 do fall between these two models.

Two other factors which were found to be related to loss rates--usage time, T, and distance, D--both tend to increase loss rates with distance and straighten curves of the type depicted in Figure 19. Percent of time used consistently correlated inversely with loss rates in the ponding loss studies and generally decreases in sections at greater distance from the mogha. The regression equations consistently indicated that in the range where most ponding data was collected (1 percent < T < 30 percent), a decrease in T by 10 (say from 30 percent to 20 percent) will result in an increase in loss rates by about 20 percent.

Channel usage time decreases with distance from the mogha. The shape of this relationship will depend on the watercourse layout and will generally vary between linear (for a long rectangular area served by one sarkari khal channel) and exponential. The combination of these two relationships causes loss rates as influenced by T to increase with distance.

There are several possible physical reasons for the affect of T on Q_L. One is the finding reported by many researchers (referenced in Chapter 3) that infiltration rates into flooded soils decrease over time, and although the rate will rebound after drying and rewetting, it does not rebound to its original value. The more a soil is flooded, the lower its infiltration rate. Reasons for this phenomena were given in Chapter 3.

A second explanation for the inverse relationship between Q_L and T is that channels filled with water more of the time allow less vegetative growth on the wetted perimeter (especially in the generally very turbid canal water). With less vegetative growth, the roughness coefficient and therefore operational supply level is more stable and does not rise into the highly porous upper bank sections as often. Frequently used sections also have stable operational supply levels because they usually lie farther from the irrigated fields where field surface elevation differences cause water surface backwater curves and osl fluctuations.

An often filled channel with a stable osl will also tend to inhibit the burrowing of insects and rodents into the wetted portions of the banks, and thus the number of macropores and the resulting leakage rate will be decreased.

The percent time used factor is probably the most important cause for farmers' branches having higher loss rates than sarkari khal sections. In the five operational studies, the average sarkari khal section was used 36 percent of the time while the average farmer's branch was used only about 2 percent of the rotation period. The mean T values for sarkari khal and farmer's branch sections in the ponding study were 20 percent and 4.6 percent, respectively. This lower sarkari khal T value in the ponding studies is the result of the difficulty of scheduling ponding loss measurements in the most often used channel sections, while the higher farmer's branch T value is the result of choosing sections near the sarkari khal outlet where water is more readily available. These time differences can explain most of the measured Q_L differences between sarkari khal and farmer's branch sections (Tables 4 and 12).

Other reasons for the higher farmer's branch loss rates include undersized farmer's branch channels due to no legal right-of-way allowance and the perpetual attempt to enlarge fields by shaving away watercourse banks, and more operational supply level fluctuation due to the proximity of most farmer's branch sections to the irrigated fields to which water must be delivered at different heights. The unaccounted for measurement bias of more flume induced losses in the shorter branches would also increase measured branch loss rates.

The ponding loss analyses indicated that loss rates increase with distance from the mogha, especially in farmers' branches, although it is sometimes difficult to separate the effect of this factor from the decreasing usage time and flow rate effects previously mentioned. A possible physical explanation for this relationship is that most silt deposition takes place near the head and deposition decreases in sections farther from the mogha. Less silt deposition can lead to higher water seepage rates into the wetted perimeter. The distance factor could be a reason for the noted linear tendency of the delivery efficiency vs. distance relationship (Table 10).

Both the distance and usage time effects on loss rates would predict that loss rates will increase with distance from the mogha. The relationship with the decreasing flow rates (Eq. 14) would predict decreasing loss rates with distance. The available data is insufficient to determine the relative strengths of these relationships. The inflow-outflow data (Table 9), after adjustment for flume effects and channel type biases indicated that the loss rates decreased slightly with distance.

The data clearly indicate that SCARP watercourses have higher loss rates than non-SCARP watercourses. This finding supports previous similar conclusions of inflow-outflow loss studies reported in Lowdermilk et al. (1978) and Clyma et al. (1975). Lowdermilk's survey of 40 watercourses found that median loss rates on SCARP systems were about 80 percent higher than for non-SCARP watercourses (2.97 lps/hm vs. 1.62 lps/hm). Clyma's data indicate that SCARP watercourse loss rates are at least double those found in non-SCARP watercourse systems. These ponding loss data indicate the SCARP loss rates are about three times larger than loss rates in non-SCARP channels.

Part of this difference can be attributed to the finding that loss rates are greater in watercourses with higher inflow rates. The flow rate in the measured SCARP watercourses averaged 70 percent larger than in non-SCARP channels. Equation 14 with P = 0.8 would predict a loss rate difference of about 60 percent between the data sets.

Another reason for the higher SCARP loss rates is probably the fact that the SCARP watercourses were never redesigned and rebuilt to carry the increased flow from the combined canal and public tubewell supply, with the resulting increase in loss rates that was discussed previously when the flow and consequently the flow depth in a watercourse is increased. It would be expected that the watercourse channels would evolve over time as a result of cleaning and maintenance activities to carry the increased flow more efficiently. However, this apparently has not completely taken place yet in the 15 years since the SCARP II public tubewells, where most of the SCARP data were collected, have been installed. Since the farmer's flow at the field has been increased in spite of the higher losses, and consequently his water deficit is less severe; perhaps he has not been as willing to upgrade his channels to more efficiently carry the increased flow. Mirza et al. (1975) found that there is in fact relatively less cleaning and maintenance activity observed on public tubewell augmented watercourses.

A third possible reason for SCARP watercourses having higher loss rates than non-SCARP ones is that, with the added clear tubewell water, the silt carrying capacity of the flow is increased and the tendency for the silt in the canal water to deposit in the channel is reduced. The decreased sedimentation in the channels could allow higher seepage rates into the wetted perimeter.

Bank width did not appear to have any affect on loss rates. Obviously, there must be a point where sufficiently thin banks must lead to breaks and leaks, and this situation has been observed in the field; but such visible leakage is a small enough percentage of the total losses that, for the tested sections, it was not a significant factor. Visible leakage which passes through the macropores and appears on the outside of the watercourse test sections was noted and often measured. About 20 percent of the ponding loss sections had visible leakage. The measured leakage usually amounted to less than 20 percent of the total loss rate at the osl and was very sensitive to depth changes in most sections. Most visible leakage stopped flowing when the water level dropped to near or slightly below the operational supply level, indicating the importance of the macropores at or above osl. Visible leakage did not seem to occur more often in watercourse sections with thinner banks. Visible leakage never amounted to more than 5 percent of the total losses on the operationally studied watercourses, and usually was less than 2 percent of the total.

A possible reason that channels with thicker banks do not have lower loss rates is that they have more soil mass and consequently might provide a more inviting habitat to burrowing insects and rodents than thin watercourse banks. It was not verified whether or not thicker banks actually have more macropores. Banks which are thick enough to securely support the conveyed water appear to have no further significant effect upon water loss rates.

The side slope factor, Z, displayed no consistent influence on loss rate. It is reasonable to expect that steeper bank slopes (lower Z values) would allow less silt deposition on the sides and lead to higher loss rates. This expectation was not supported by the data analysis. Shorter wetted perimeters, especially on the bank sides, which would be expected from channels with larger Z values might counter the silt deposition effect. From the parameters measured and the data analyzed, no relationship can be established between the cross-sectional shapes of channels and loss rates.

The elevation difference between the channel water surface level and the surface of the surrounding fields (ΔE) had a direct affect on loss rates in four out of five of the significant ponding study data sets analyzed. The measurements designed specifically to test ΔE effects indicated a direct relationship, with Q_{LO} increasing about

25 percent with a 10 cm increase in ΔE. Physically, this relationship could be caused by the increasing hydraulic gradient pushing water out through macropores as ΔE increases. It is also possible that burrowing insects and rodents would be more attracted to the larger soil mass higher above the fields formed by banks of higher watercourse channels, which could in turn result in more macropores in the higher watercourse banks.

Factors which Influence Loss Rates through Water Level Fluctuations

The previous discussion treats the loss rates which occur in channels flowing at their usual operating level. The ponding data shows that loss rates are very sensitive to depth fluctuations, and that the relationship between depth fluctuations and loss rates is best described by an exponential function (Eq. 11).

The ponding data analysis indicated some of the factors which affect the sensitivity of this relationship, or the exponential coefficient, b, in Eq. 11. Many of the relationships are difficult to explain physically, especially because of the intercorrelation between the b value and operational supply level loss rate, Q_{LO} (Eq. 13). The measured lower b values in recently rebuilt (I) watercourses would be expected because the new banks have fewer macropores, especially in the upper regions, which are believed to greatly influence loss rates at deeper flow depths. The finding that usage time directly affects b values could be the result of the previously mentioned more abrupt change in bank porosity at full supply level resulting from a stabler fsl and less burrowing and thus fewer macropores below the water surface level of the often full channel.

The analysis given in Table 8 relating loss rates to inflow rate fluctuations predicts that loss rate will increase by 1.5 to 4.0 percent for each 1 percent increase in flow rate from its mean value. For the same average flow rates and a b value of 0.15 cm^{-1}, Eq. 22 predicts loss rate increases of 2 to 5 percent per 1 percent increase in flow rates. The rate of increase predicted by the model averaged 35 percent higher than that predicted by the regression equations derived from the data, indicating that the model might overestimate the effects of flow rate fluctuations and perhaps flow depth changes.

The measured change in loss rate with depth is the secondary result of two other factors which are changing with flow depth. The first is the changing length of the wetted perimeter through which seepage is taking place, and the second is the changing average rate of seepage into the wetted perimeter. Since the change in loss rate is occurring much more rapidly than both the change in wetted perimeter length and an expected change in seepage rate resulting from the change in pressure head, it must be assumed that water seeps into watercourse bank soils at a much higher rate than into the bed soils.

The loss rate is made up of the sum of the seepage rates, s, into each section of the wetted perimeter. If both seepage rate and wetted perimeter length are taken relative to depth, and the effects of the varying pressure are ignored, then:

$$Q_L = s(d_n) \times (WP(d_n) - WP(d_{n-1})) + s(d_{n-1}) \times (WP(d_{n-1}) - WP(d_{n-2})) + \ldots + s(d_1) \times (WP(d_1) - WP(d_o)) \quad (23)$$

where:

$s(d_n)$ = the seepage rate into the banks at height d_n above the channel bottom per unit of channel length, and

$(WP(d_n) - WP(d_{n-1}))$ = the length of wetted perimeter from depth d_n to depth d_{n-1}.

As this incremental equation is taken to its limit, it reduces to:

$$Q_L = \int_o^{d_n} s(d) \frac{dWP}{dd} dd . \quad (24)$$

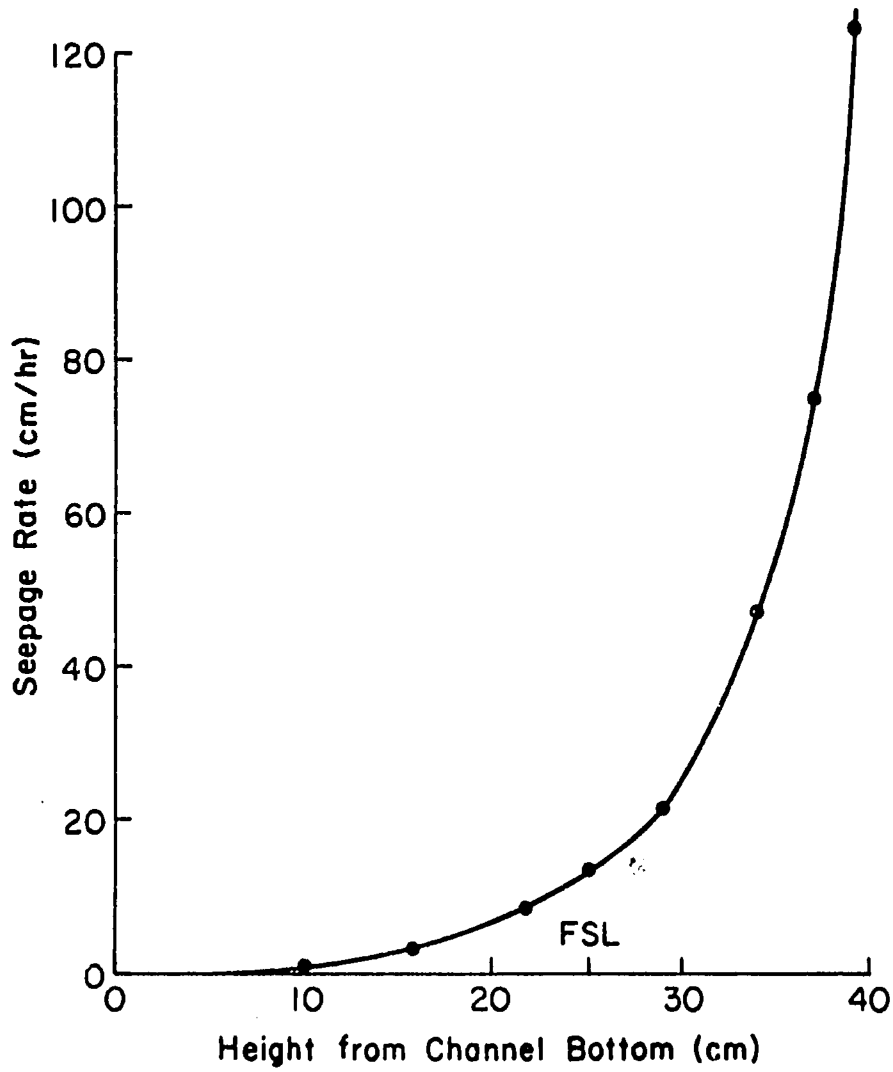
If the derivative with respect to depth is taken of both sides, the seepage rate at any depth, s(d), can be determined by:

$$s(d) = \frac{\frac{dQ_L}{dd}}{\frac{dWP}{dd}} . \quad (25)$$

The numerator of this equation can be calculated from the slope of the line of the loss rate vs. depth relationship shown in Figure 10 at any depth, or from the derivative of Eq. 11. The denominator can also be determined graphically by measuring the slope of a line depicting the relationship between the length of wetted perimeter and depth, at any depth, or, if the relationship can be mathematically modeled, by the derivative of the equation.

Figure 20 depicts the seepage rate as a function of depth determined graphically from the data for a sample watercourse section. Figure 21 shows the seepage rate as a function of depth calculated mathematically for a hypothetical watercourse whose cross section is shown in Figure 3 and whose loss rate is represented by Eq. 11 with Q_{LO} = 2.0 lps/100 m and b = 0.15 cm^{-1}. Both figures indicate that the seepage rate into the higher banks is much greater than that into the bed and lower banks. Although the influence of the variation in pressure head on seepage rate has not been considered, for the ranges of pressure heads which apply, the effect on seepage rate should not normally be greater than 10 to 30 percent. Adjustment for this factor would have the effect of adjusting the seepage rate scale somewhat, but would not change the overall conclusion.

There are several possible reasons why the seepage rate into the highly permeable banks might be as much as 100 times greater than into the bed. The slope of the bank sides inhibit sediment deposition which can seal the bed. Often the finer sediments from stagnent dead storage water are deposited on the bed. Also, silt which does deposit on the bed is often left to build up over time, while that on the bank is periodically shaved away with grasses and



'igure 20. Seepage rate into watercourse bank soils at various heights up the banks from the channel bottom (determined graphically from ponding loss data for a sample section).

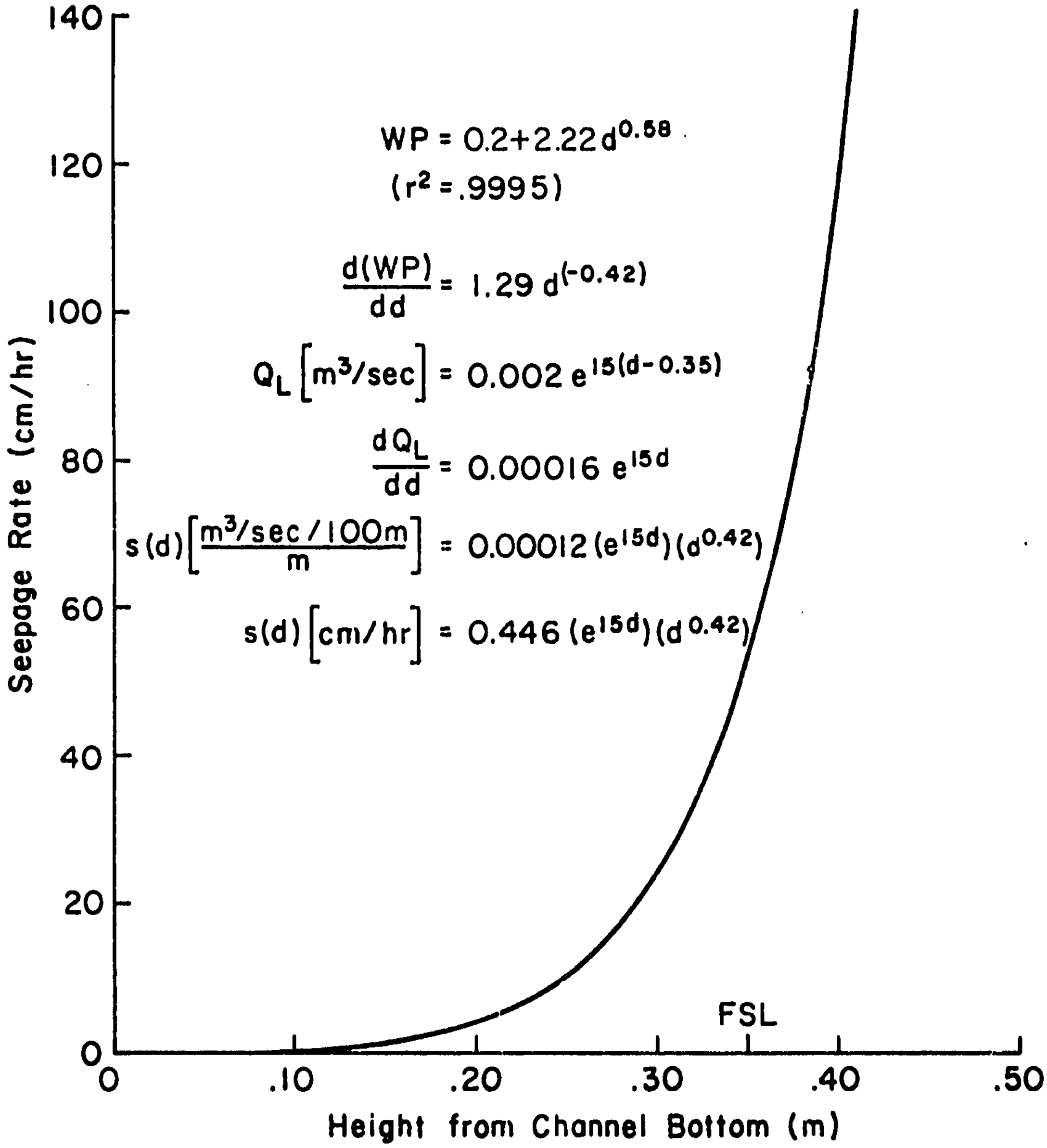


Figure 21. Seepage rate into watercourse bank soils at various heights up the banks from the channel bottom calculated from Equation 25 for the hypothetical channel shown in Figure 3.

vegetation during cleaning and maintenance activities. The bed often has dead storage water standing in it for a high percentage of time which will facilitate microbial activity which can tend to seal pores and lower seepage rates.

But these reasons do not explain why seepage rates into the upper banks are 10 to 100 times higher than infiltration rates into the surrounding fields. A phenomenon in addition to seepage resulting from normal soil permeability must be taking place.

One such phenomenon observed in nearly every watercourse bank and described more fully in Chapter 2 are the macropores resulting from the activity of burrowing insects and rodents. The burrows are usually found in the upper portions of the bank, often near and above the normal full supply level.

The extensive system of macropores could explain the high seepage rates into watercourse banks, and the extreme sensitivity of this seepage rate with height along the banks. It could also explain the extreme variability in measured ponding loss rates and the low percentage of the variability which can be explained by the measured parameters. Even when most parameters were held constant, the measured rates were still extremely variable. Table 15 lists the measured loss rates at osl (Q_{LO}) for various sets of ponding measurements where either the same section was measured several times over time or adjoining sections were measured at one point in time. In most cases, the standard deviation of the

Table 15. Comparison of measured loss rates in test sections where most conditions are constant.

A. Measurements in the Same Section over Time

Section	Time from first measurement	Loss Rates (Q_{LO})(lps/hm) Individual measurements	Mean	Standard Deviation
1	0,3,15 days	4.19 3.62 5.15	4.32	0.77
2	0,3,16 months	1.35 3.27 1.45	2.02	1.08
3	0,0,10,25 months	5.75 3.71 1.86 3.45	3.70	1.60
3(a)	0,0,10,25 months	8.35 6.31 2.78 1.79	4.81	3.06

B. Measurements in Adjoining Sections

Set	Measured Loss Rates (Q_{LO})(lps/hm) Sec 1	Sec 2	Sec 3	Sec 4	Mean	Standard Deviation
1	12.44	5.75	8.35		8.85	3.37
2	2.34	4.19	2.83		3.12	0.96
3	12.90	5.57	7.43		8.63	3.81
4	1.14	1.17	1.14	0.98	1.11	0.08
5	0.56	1.02	1.30		0.96	0.38
6	5.57	4.83	3.43	2.32	4.04	1.45
7	1.79	1.80	1.12	3.04	1.94	0.80
8	3.25	1.86	4.55		3.22	1.35
9	0.46	0.37	0.28		0.37	0.09
10	3.78	3.85	0.54		2.72	1.89
11	2.78	3.11	1.44		2.44	0.88
12	3.00	2.52	1.62		2.38	0.70
13	2.42	3.18	1.96		2.52	0.62
14	5.20	6.87	3.53		5.20	1.67
15	5.48	6.22	3.43		5.04	1.44
16	6.40	4.83	2.41		4.55	2.01
17	0.10	0.09	0.42		0.20	0.18
18	1.45	0.83	0.49	0.74	0.88	0.41
19	0.14	0.23	0.30		0.22	0.08
20	0.49	0.68	0.71		0.62	0.12

sets of three or four measurements is one-third to one-half of the mean. This variation, like the high seepage rates, cannot be explained by normal variations in permeability, but could be explained by point sources of seepage through macropores.

Watercourse Water Loss Model

The watercourse model will not be a predictive model in the usual sense, in that it cannot predict the water losses from a watercourse system purely from a set of parameter values. The reason such a model was not constructed is that the system is too complex and the number of parameters too great to make such a procedure practical. The amount of time and energy involved in measuring the required parameters would far exceed that needed to measure losses directly.

The analyses, however, were able to indicate significant functional relationships between loss rates and several parameters. The model will thus be constructed, based on these findings, to indicate relative changes in losses with changes in these parameters. The base loss rate value can be measured for a given channel or estimated.

The usefulness of such a model is in determining means to improve the conveyance efficiencies of existing systems, or in indicating to the watercourse designer alternatives which will lead to reduced losses. Thus the model will be capable of dealing with the types of questions most commonly encountered in existing irrigation systems.

The watercourse loss model will be constructed in terms of the percent conveyance losses, or volume of water lost from the watercourse system in the process of delivering it to the fields, divided by the volume which enters the watercourse at the head, times 100.

$$L(\%) = \frac{V_L}{V_M} \times 100 = \frac{V_M - V_F}{V_M} (100) , \tag{26}$$

where:

L = conveyance losses (%),

V_L = water volume lost from the watercourse system (m^3),

V_M = inflow volume at the watercourse head (m^3), and

V_F = volume of water delivered to the field (m^3).

The conveyance losses are equal to one hundred minus the delivery efficiency (E) in percent.

The total losses are the sum of the steady state (V_{LSS}) and transient (V_{LT}) losses.

$$V_L = V_{LSS} + V_{LT} \tag{27}$$

Transient losses were shown in Chapter 5 to be a function of the normal inflow rate (Q_M) and the length of channel wetted (L_W) and drained (L_D), and can be estimated by the equation:

$$V_{LT} = 0.0047\ Q_M L_W - (.05 + .0005\ Q_M) L_D \tag{28}$$

Steady state losses are equal to the change in the steady state flow rates between the head (Q_M) and field (Q_F) times the time (t).

$$V_{LSS} = (Q_M - Q_F)t \tag{29}$$

Since loss rates in the farmers' branches tend to be significantly higher than the loss rates in the sarkari khal, the steady state losses will be divided into that portion which occurs in the sarkari khal and that which occurs in the farmers' branches.

$$V_{LSS} = [(Q_M - Q_I) + (Q_I - Q_F)]t, \tag{30}$$

where:

Q_I = the intermediate steady state flow rate at the outlet from the sarkari khal to the farmers' branch (lps).

Intermediate and field flow rates can be determined by the integration of the loss rate (Q_L in lps/hm) in the upstream channel over the distance (D).

$$Q = Q_M - \int_0^D Q_L dD \tag{31}$$

Equation 14 describes a power curve relationship between steady state loss rate and normal flow rate.

$$Q_L = \frac{dQ}{dD} = KQ^P \tag{14}$$

The Q and D terms of Eq. 14 can be separated and integrated from Q_M to Q and along the channel from 0 to distance D, to give V_{LSS} in terms of D and Q.

$$\int_{Q_M}^{Q} Q^{-P} dQ = \int_0^D -KdD \tag{32}$$

$$Q_M - Q = Q_M - \left[Q_M^{(1-P)} - K(1-P)D\right]^{\frac{1}{1-P}} \tag{33}$$

Since $K = \dfrac{Q_{Li}}{Q_M^{\ P}}$, where Q_{Li} is the initial loss rate at the section head (lps/hm),

$$Q_M - Q = Q_M - \left[Q_M^{(1-P)} - \frac{Q_{Li}}{Q_M^{\ P}} (1-P)D\right]^{(\frac{1}{1-P})} \tag{34}$$

Combining Eqs. 27, 28, 30, and 34 gives:

$$L(\%) = \left[\left\{Q_M - \left[Q_M^{(1-P)} - \left(\frac{Q_{LSK}}{Q_M^{\ P}}\right)(1-P)D_{SK}\right]^{(\frac{1}{1-P})} + Q_I - \left[Q_I^{(1-P)} - \left(\frac{Q_{LFB}}{Q_I^{\ P}}\right)(1-P)D_{FB}\right]^{(\frac{1}{1-P})}\right\}t + 0.0047\ Q_M L_W - (.05 + 0.0005\ Q_M)L_D\right] \times \frac{100}{Q_M t}\ , \tag{35}$$

where:

Q_{LSK} = loss rate in the initial section of the sarkari khal (lps/hm),

Q_{LFB} = loss rate in the initial section of the farmer's branch (lps/hm),

Q_I = initial flow rate in the farmer's branch (lps) determined by the second term of Eq. 34 with $D = D_{SK}$,

D_{SK} = sarkari khal length (hectometers), and

D_{FB} = farmer's branch length (hectometers).

Since the second part of the first term of Eq. 35 (in brackets) is equal to Q_I, Eq. 35 can be simplified to:

$$L(\%) = \left\{ 1 - \left[Q_I^{(1-P)} - \left(\frac{Q_{LFB}}{Q_I^{P}}\right)(1-P)D_{FB} \right]^{\left(\frac{1}{1-P}\right)} \frac{1}{Q_M} + \left[(0.0047\ L_W - \left(\frac{.05\ L_D}{Q_M}\right) - 0.005\ \ L_D) \right] \frac{1}{t} \right\} \times 100, \tag{36}$$

where:

$$Q_I = \left[Q_M^{(1-P)} - \left(\frac{Q_{LSK}}{Q_M^{P}}\right)(1-P)D_{SK} \right]^{\left(\frac{1}{1-P}\right)} .$$

Factors which affect the loss rate value can be applied directly to Q_{LSK} and Q_{LFB} before they are inserted into Eq. 36. Loss rates are an inverse function of usage time (T). The data indicate that Q_L changes about 2 percent for each unit change in T. Since most farmers' branches are normally utilized less than 10 percent of the time, no time adjustment will usually be made in farmer's branch loss rates. Changes in sarkari khal loss rates will be treated as fluctuations due to changes in T.

$$Q_{LSK-T} = Q_{LSK} - .02\ Q_{LSK}(T-T_O), \tag{37}$$

where:

Q_{LSK-T} = Q_{LSK} adjusted for time fluctuations,

T_O = an initial usage time in the channel section (%), and

T = the new usage time (%).

For example, if a sarkari khal section which is normally used 20 percent of the time (T_O) and has an initial loss rate of 1.0 lps (Q_{LSK}), will be utilized 40 percent of the time in the future, the loss rate would be predicted to decrease to 0.6 lps/hm. The limits to this linear relationship are obvious.

A similar linear adjustment can be made for changes in ΔE. The special ponding loss studies of Q_L vs. ΔE indicated that a change in ΔE of 1 cm leads to a 2.5 percent change in Q_L. The adjustment can be calculated by:

$$Q_{L-\Delta E} = Q_L + .025\ Q_L(\Delta E - \Delta E_O)\ , \qquad (38)$$

where:

$Q_{L-\Delta E}$ = Q_L adjusted for ΔE fluctuations, and

$(\Delta E - \Delta E_O)$ = a change from the initial average elevation difference (cm).

Parameters which affect the water surface level in a channel (d) will affect the loss rate according to Eq. 11.

$$Q_L = Q_{LO}e^{b\Delta d} \qquad (11)$$

Two types of factors influence the flow depth. The first type, discussed in Chapter 6, affect the normal flow depth (i.e., ΔQ and n). Equation 22 relates these factors to changes in the loss rate. The effect of changes in flow rate (ΔQ) or roughness coefficient (n) on loss rates (Q_L) will be calculated by:

$$Q_{L-\Delta Qxn} = Q_L \exp\left\{1.2b\left[\left(\frac{Qn}{\sqrt{S}}\right)^{.5} - \left(\frac{Q_o n_o}{\sqrt{S}}\right)^{.5}\right]\right\} \qquad (39)$$

where:

$Q_{L-\Delta Qxn}$ is the loss rate adjusted for changes in flow rate and roughness coefficient,

Q_o and n_o are original values of flow rate (m^3/sec) and roughness coefficient,

Q and n are the new values of flow rate (m^3/sec) and roughness coefficient, and

b is the exponent of Eq. 11 (m^{-1}).

This equation assumes an a and c value in Eq. 22 of 1.2 and 0.5 respectively.

The second type of factor which affects flow depths creates a head loss (Δh) in the channel. Obstructions in the water channel, such as partially opened checks, water measurement flumes, or trash; or relatively high or low fields will cause gradually varied flow conditions. The effect on the full supply level can be calculated with backwater curves such as was done for flume installations in Chapter 4. Since backwater curve calculations are iterative and depend on several hydraulic parameters, it is not possible to form a general mathematical relationship between the induced head loss (Δh) and loss rates. A relationship for one "average" case is represented by the flume adjustment factor, Eq. 9. For more general conditions, it is easier to select the correct adjustment factor from a table, such as Table 2.

Once the loss rate (Q_L) is adjusted for projected changes in usage time, relative elevation, inflow rate, roughness coefficient, and/or in-channel head losses, the adjusted value can be inserted into Eq. 36 and projected percent losses (L) can be calculated.

Chapter 8

APPLICATIONS FOR WATERCOURSE DESIGN

Many design alternatives are available to the engineer designing a watercourse. Often the most important valuation in choosing between the alternatives is their resulting water conveyance losses. The model developed in the last chapter will be applied to some of the more common practical watercourse design choices to indicate which will result in reduced losses, or to predict the water savings in changing from the commonly encountered practice.

The watercourse model calculations were made on a Hewlett-Packard 9825A desk-top computer. The program is listed in the Appendix. Unless otherwise noted, values used in the model were:

Initial (spatially) sarkari khal loss rate, Q_{LSK} = 1.0 lps/hm;

Initial farmer's branch loss rate, Q_{LFB} = 2.0 lps/hm;

Sarkari khal length, D_{SK} = 12 hm (1200 m);

Farmer's branch length, D_{FB} = 3 hm (300 m);

Rate of change of loss rate with flow rate, P = 0.8;

Exponent of loss rate vs. depth relationship (Eq. 11), b = 15 m^{-1} (0.15 cm^{-1});

Channel slope, S = .0005 m/m;

Roughness coefficient, n = .035.

These values are in the middle range of the collected data.

Since losses (L) are given in percent there is ambiguity in expressing changes in the losses. An absolute change in losses, or a change as a percent of inflow, will be written as a change by a certain number of percentage points. A relative change of loss or any other value will be written as a percent change. For example, 10 percent increase in an initial 50 percent loss results in a 55 percent loss, while a change by 10 percentage points results in a 60 percent loss.

Reorganization of Field Shapes to Decrease the Number of Farmers' Branches

As indicated in Chapter 2, fields are usually small and rectangular in shape, requiring about 120 meters of farmer's branch per hectare of land irrigated. If fields were reorganized into long narrow borders, the field sizes could be maintained so that stream sizes or crop rotations on small holdings need not be changed, while the number of branches to irrigate them could be reduced. Present fields are normally about 60 m in length. If this were doubled or tripled, while reducing the width by the same proportion, the total length of branches could be reduced to 60 m/ha, or about 50 percent of the original length. An example of such a field reorganization is shown in Figures 22 and 23.

Conveyance losses would be reduced in three ways. First, the usage of the remaining branches could be doubled (from 2 percent to 4 percent on the average), which could reduce steady state farmers' branch loss rates (Q_{LFB}), according to the findings (Eq. 37), by 4 percent, and total losses, according to the model (Eq. 36), by 1/2 percentage point. Second, the length of channel wetted and drained

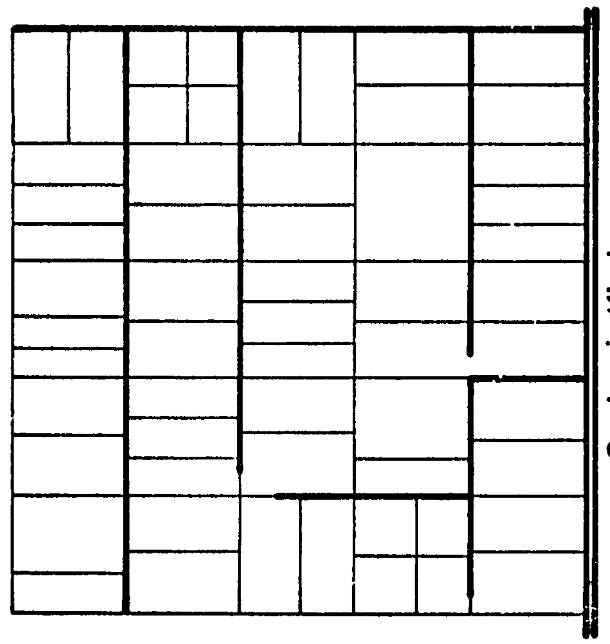


Figure 22. An example of present field and farmer's branch channel layout on a 10 hectare "square" of land showing 1300 m or 130 m/ha of channels.

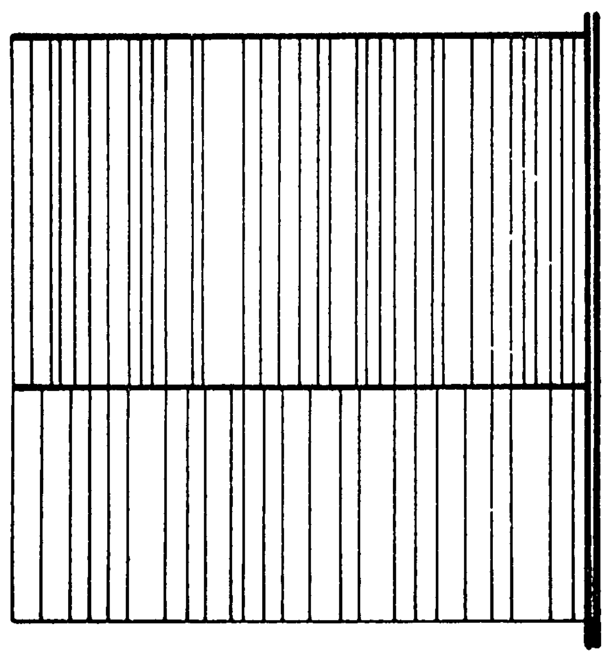


Figure 23. An example of field and farmer's branch channel layout after reorganization of fields into long narrow basins showing 640 m or 64 m/ha of channels.

per week would be reduced, although not by 50 percent since a higher percentage of the reduced number of branches will be used, but perhaps by 30 percent. The resulting 30 percent reduction in transient losses would lead to a reduction in total losses by about 2 percentage points. Third, the average length of farmer's branch used could be reduced by about 100 m, which would reduce total steady state losses by about 6 percentage points. The total water savings resulting from the field reorganization would be predicted to be 8 1/2 percent of the inflow. Saving 8 1/2 percent of the water in the watercourse is equivalent in extra water available for crop production to providing 20 billion cubic meters of extra water at the canal head, or an additional 15 percent of the present total diversion.

The land for the additional approximately 160 m/ha of "bunds" (border dikes) required for the long narrow fields would be available from the 60 m/ha of abandoned watercourse channels which are about three times as wide as bunds.

The cost of field reorganization will depend on the local topography, land holdings, soil type, and mechanization. Field reorganization will require some field releveling, which can be costly. This requirement could be used as a catalyst to promote precision land leveling of the often irregular surfaces of the fields. Efficient reorganization will require some land consolidation which will be time consuming and require strong legal support. Cultivating borders is more efficient with tractors, but more difficult with the single point bullock-pulled plow with which cross cultivation is important. Irrigation application uniformities will decrease on the longer fields from slightly on low intake rate soils to appreciably on sandy soils. The decreased application efficiencies will sometimes be greater than the increased conveyance efficiencies and must be considered in any overall efficiency calculations.

Subdivision of Watercourse Command Areas and Flows to Decrease Conveyance Distances and/or Increase Channel Usage Times

On most watercourse systems, sarkari khal channels are efficiently laid out so that significantly reducing the length of channel used or distance to the field is not usually possible, unless more canals are constructed and watercourses are subdivided lengthwise. Since the constantly flowing canals should have lower loss rates than watercourses, such a layout change should result in reduced losses, although the construction work would be costly, would disrupt established transportation and land holding patterns, and would take some land out of cultivation. Decreasing average sarkari khal distance from the mogha to the field by half would, as shown in Figure 24, reduce watercourse losses about 10 to 20 percentage points, depending on the loss rate. Perhaps more important, the tail farmers would be receiving about 30 percent more water from the shorter sarkari khals.

If distances cannot be shortened, warabundi turns, which proportion water based on magha flow rather than field

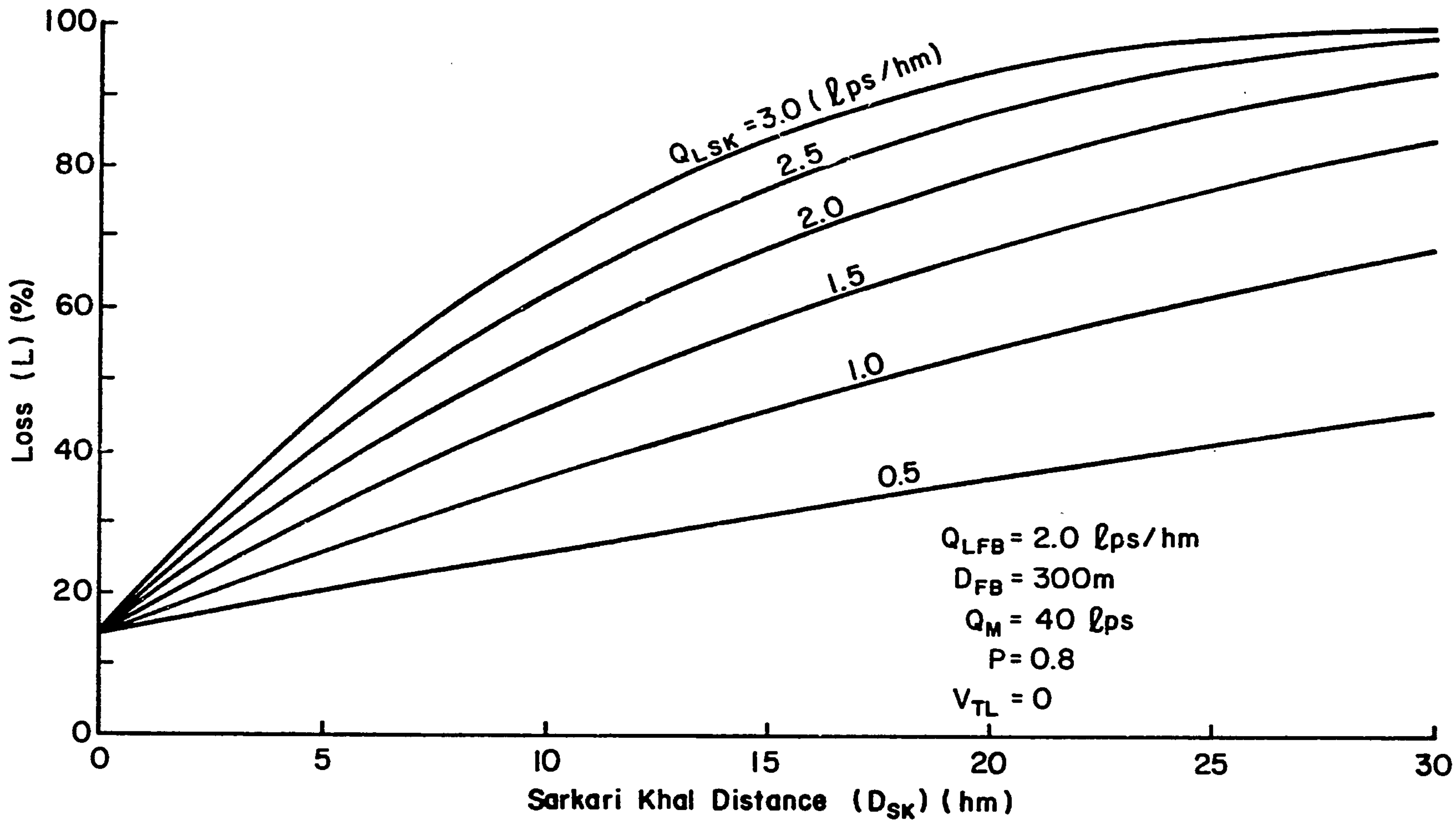


Figure 24. Percent conveyance losses (L) vs. sarkari khal length (D_{SK}) for different initial sarkari khal loss rates (Q_{LSK}).

delivery, could at least be readjusted so that tail farmers, who generally receive less than 50 percent as much flow as those whose land lies near the head, get a more proportionate supply of water. Any such reapportionment would, however, increase overall losses since time weighted average conveyance lengths would be increased. Equally redistributing field deliveries will increase the time weighted average conveyance distances by 100 to 500 m and total conveyance losses by 2 to 10 percentage points.

Figure 24 indicates the advantage in water savings of installing small tubewells spaced around the watercourse command area near the fields rather than constructing one large tubewell near the watercourse head. Two properly placed tubewells instead of one well at the head of a watercourse could reduce conveyance distance to most fields by 50 percent and result in 25 percent more of their water reaching the fields. Even though two 20 lps tubewells are more costly to construct than one 40 lps tubewell, the value of the pumped water is actually 25 percent higher, which will at least partially offset the added costs. The common practice in the Indian portions of the Basin is to drill and line several small (10-15 lps) tubewells near the irrigated fields and utilize a portable pump and power supply. This technique can reduce both costs and water losses.

An additional benefit of scattered small tubewells is that the pumped water need not be mixed with canal water, but can be utilized when the canal water is flowing in other portions of the watercourse. The implications of this alternative on losses will be discussed later. The alternative is of course a good one only if groundwater quality is sufficiently good that dilution is not necessary.

When watercourse command areas are divided, the channel inflows will be reduced and usage times of the channels increased, since fewer channels remain and total time for irrigation remains constant (168 hrs/wk). If a watercourse system is split in two pieces, and flow rates to each are cut by 50 percent, say from 50 lps to 25 lps, sarkari khal loss rates would decrease from the initial value, say from 1.5 lps/hm to 0.86 lps/hm (less than a 50 percent decrease), as a result of the decrease in flow rates; and further to 0.52 lps/hm as a result of doubling sarkari khal average usage from 20 percent to 40 percent of the time. The split would result in sarkari khal losses being reduced by 8.5 percentage points, and total steady state losses by about 7 percentage points without changing conveyance distances at all. Transient losses should be about the same since about the same total length of channel will be used in irrigating half as many fields on each section of the watercourse.

This implies that even subdividing a watercourse command area widthwise without reducing conveyance distances should reduce total losses. This could be accomplished by installing more moghas along a canal, or by dividing a watercourse flow between its major branches.

Reducing inflow rates will reduce application uniformities, since advance times on the level basins will be increased, unless field sizes are reduced. The resulting increase in application losses could be appreciable in soils with high intake rates, but for the most common silt and silty clay loam alluvial soils, irrigated in the Indus Basin, the intake rate is sufficiently low that the extra application losses would be low. The increase in application losses will also be lower for higher flow rates.

The reduced flow rates and resulting increased irrigation turn time per farmer would cause increased labor requirements for the farmers. Splitting a watercourse flow into two channels could nearly double the labor requirements since two different farmers' fields would be irrigated all the time. If the original flow was large (larger than 50 lps), as is the case in most SCARP (public tubewell augmented) watercourses, the reduced flows will be much easier for the farmer to control with his shovel and earthen dams, which will lead to less actual physical labor per irrigation turn, and will sometimes reduce the number of laborers required per turn from two to one man. The easier to handle flows will also leave the irrigator more free time to monitor his irrigation more closely, or to work among his neighboring fields during his turn if he chooses. The reduction from large flows should also lead to fewer large, short-term bund breaks and bank breaches.

If watercourse flows are greater than 40 lps in low intake rate soils, or 50 lps in sandy soils, significant overall water savings should be realizable by subdividing watercourse areas and flows, or spitting flows down two major branches of an existing watercourse. The value of the saved water (at $100/ha-m) would usually be greater than the cost of the extra labor requirement (at $0.15/hr).

Reduction of Transient Losses by Reducing the Length of Wetted Channels

Transient losses for total watercourses were consistently about 7 percent of the inflow. However, for an individual farmer they can vary greatly depending on the length of channel he fills or drains in the process of irrigating his fields. Figure 25 shows the sensitivity of total losses to these factors.

If a farmer can organize his fields and manage his irrigations such that he irrigates only neighboring or nearby fields during one turn, he can save a significant amount of his water. For example, assume a farmer whose fields lie on three branches, each extending about 200 m from the sarkari khal, has a turn time of 8 hours. If he irrigates fields lying only on one branch each turn, Figure 25 predicts his losses will be 43 percent. If he splits his time between fields on two branches, his losses will increase to 46 percent, and if he irrigates from all three branches his losses would be 49 percent. While irrigating a single field on a branch for only an hour, nearly 20 percent of the

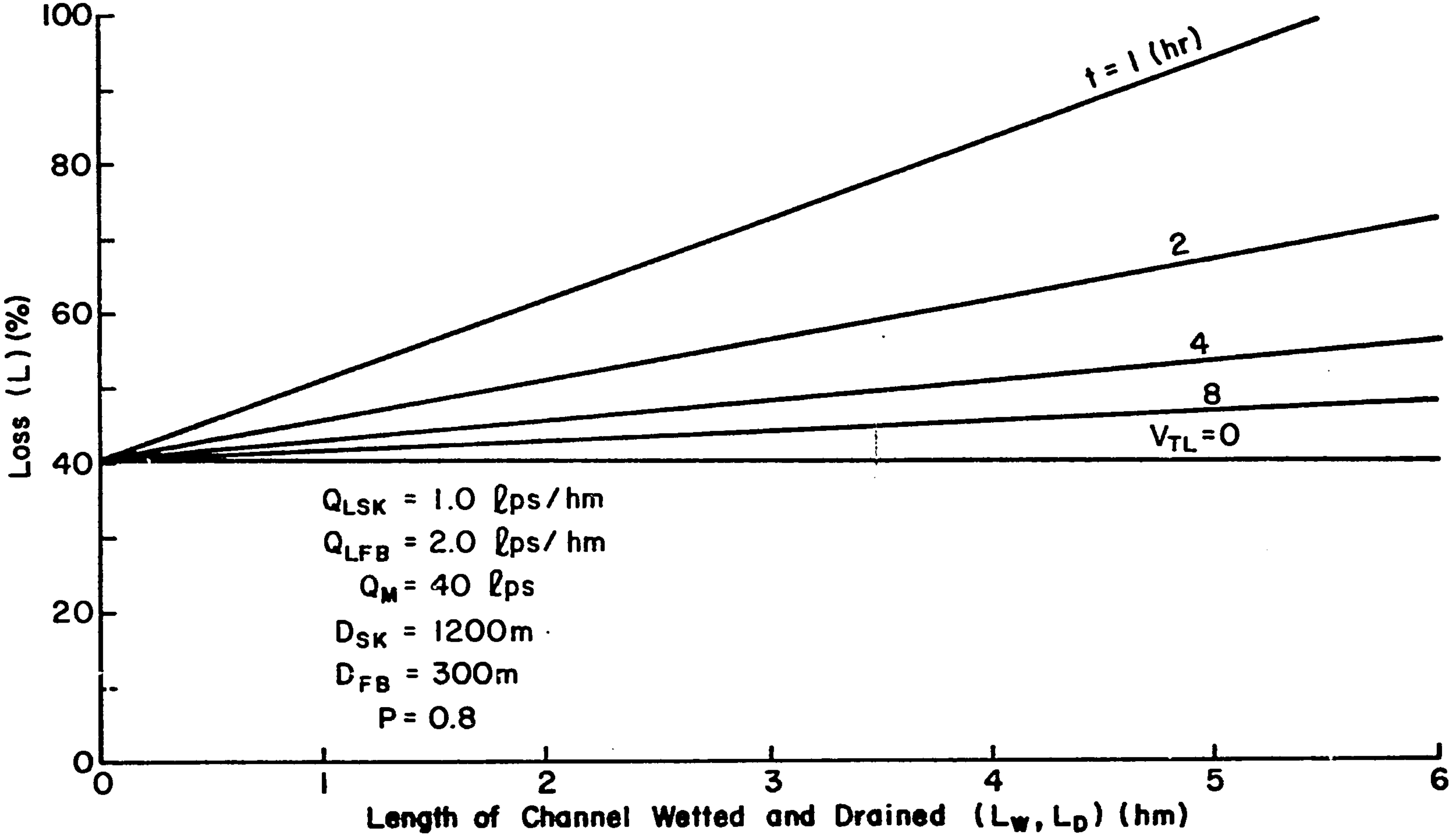


Figure 25. Percent conveyance losses (L) vs. length of channel wetted and drained (L_W and L_D) for different irrigation times (t).

hour's water would be lost to filling 200 m of previously empty channels.

Reshaping fields into long narrow borders would assist the farmer in reducing branch channel usage since more land is accessible from each branch. Field reorganization and, in some cases, additional outlets from the sarkari khal, could reduce the length of branch channels to some fields and would reduce losses if leak-free outlets are available. Land consolidation also would allow farmers longer turn times on compact blocks of land and would result in reduced transient losses.

The warabundi turn rotation system has been designed to minimize the amount of channel filled and drained and thus minimize the transient losses. Deviations from the warabundi increase transient losses. A demand system where water is moved randomly around the watercourse system could result in 10 to 40 percent of the inflow being lost to transient conditions alone, unless the channels were lined.

Some flexibility in farmer turn times, however, should decrease application losses and could lead to increased yields through watering fields more when required instead of strictly when the water turn comes. This could be accomplished without additional transient losses by allowing flexibility in turn times without changing turn orders, or with minimal transient loss increases if farmers whose land lies on the same branches or on the same sarkari khal outlet were allowed to trade irrigation turns. Such informal arrangements are common among farmers on some watercourses.

<u>Elevation of the Watercourse with Respect to the Surrounding Land</u>

A watercourse designer is faced with the choice of how high to construct a watercourse relative to the surrounding fields. In an area with adequate slope, the elevation question is not critical since the head is available in a short distance to convey water onto the fields. But in low gradient open conveyance systems where slopes of 0.2 to 0.8 m/km are common, such as are found in the Indus Basin, adequate head must be designed into the system. This can be done in a gravity system either by elevating the entire watercourse sufficiently to irrigate the highest fields, or by building the watercourse lower into the ground and checking the water up to irrigate the higher fields.

The findings indicate both techniques will influence loss rates. The model predicts a 2.5 percent increase in loss rates per cm increase in the normal full supply level elevation (ΔE). Table 2 indicates the induced loss rate resulting from temporarily checking up water above the normal fsl. Figure 26 graphically depicts both relationships. It is evident from the figure that checking up water results in higher losses than building higher watercourses, especially on low gradient channels. Because of the sensitivity of the backwater curve calculation to channel slope, the losses resulting from checking up the water level is also sensitive to slope, as indicated in Figure 26. But if only a few fields are significantly higher than the rest, total water can be saved with lower channels.

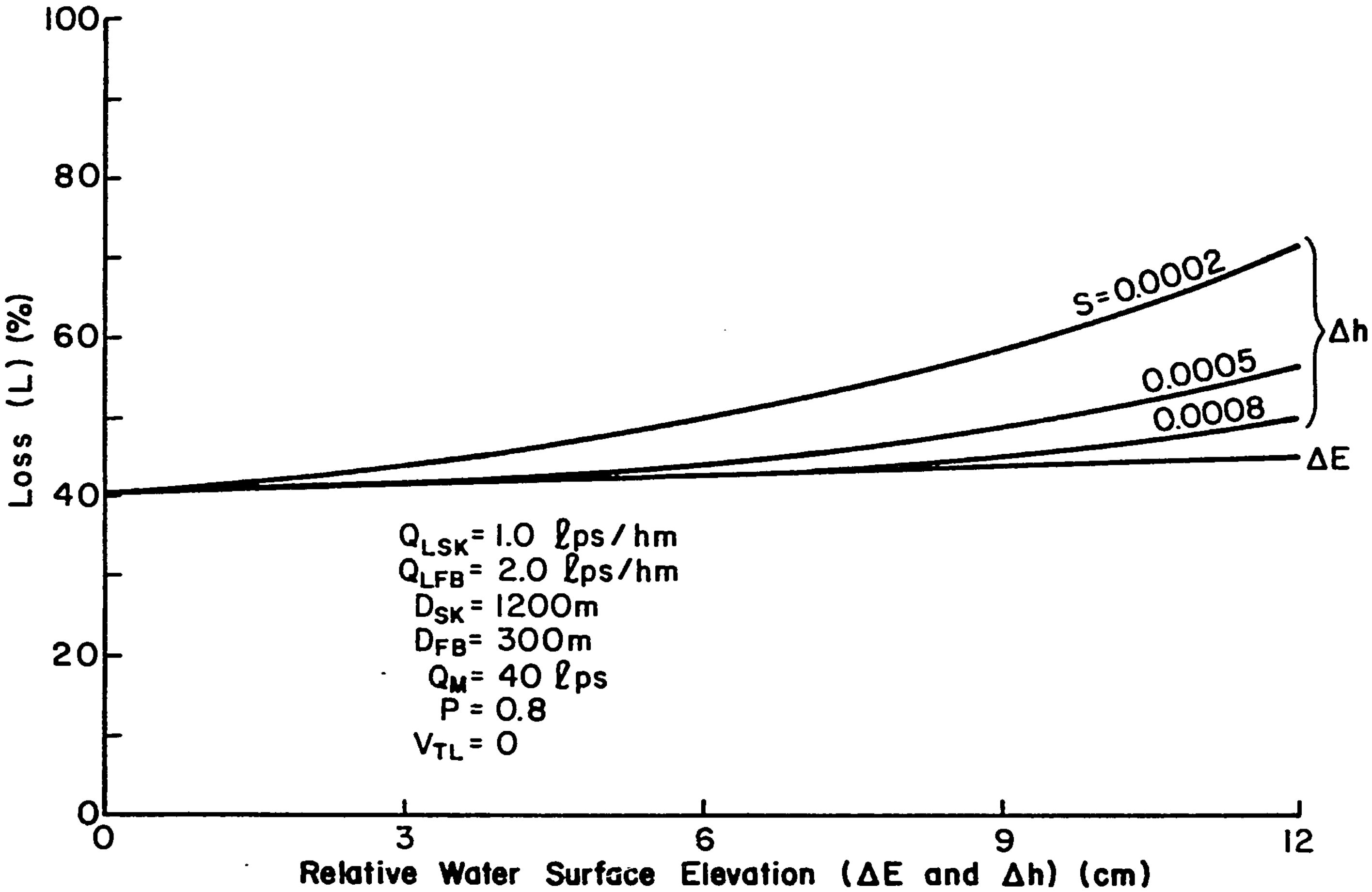


Figure 26. Percent conveyance losses vs. increasing design full supply level elevation (ΔE) and checked up flow depth (Δh).

For example, if 10 percent of the land area on a watercourse is about 10 cm higher than the rest of the commanded area, should the designer construct the watercourse high enough to serve the high fields, or only at an elevation to serve the lower 90 percent and design for extra bank to check up the water to irrigate the high fields. For the conditions given in Figure 26, the graph indicates that all farmers would suffer 6 percent additional losses as a result of elevating the watercourse, while the 10 percent with high fields would lose 21 percent additional water by checking up the water in a lower watercourse if the channel slope were flat (S = 0.2 m/km) or 7 percent additional water if the channel were steeper (S = 0.8 m/km). In both cases the total watercourse losses would be less with the lower watercourse.

In general, the elevation decision will depend on the channel gradient and the number of farmers with higher fields. With these factors known, alternatives can be tested with Table 2 and the model, or from Figure 26 to determine what elevation results in the lowest total steady state losses. One additional factor which must be considered is that in checking up water in a channel, a farmer could be sacrificing a significant portion of his water share to channel storage which the next farmer with a lower field will regain. Alterations in the warabundi schedule might be required to facilitate equitable distribution of the water.

The transient loss, dead storage, will also be affected by channel elevation since lower watercourses will have more dead storage, especially if slopes are small. Geometrical analyses of dead storage indicates that a moderately sized and sloped watercourse will lose 5 to 8 m^3 of dead storage water in each long drained channel for each cm that the channel is lowered. At this rate, if three sarkari khal branches were lowered 10 cm, less than 1 percent of the inflow would be lost to additional dead storage. Because of dead storage loss, it would be preferable, especially on low gradient watercourses, to build the farmers' branches, which have hundreds of tails, high enough to serve the commanded fields, and check up water only in the sarkari khal sections, unless a branch tail farmer has a low field into which he can drain the dead storage water.

Effect of Inflow Rate Fluctuations

Inflow rate fluctuations affect flow depths (Figure 16) and loss rates (Figure 17). The watercourse model can be used to convert these loss rates to total losses, as shown in Figure 27. Losses do not increase as loss rates shown in Figure 17 since increased loss rates cause flow rates to decrease more quickly along the channel, which in turn lead to decreasing loss rates with distance. Figure 27 shows that at lower flow rates, percent losses increase as flow rates decrease, since the loss rates decrease less quickly than flow rates in this range. At higher flow rates, the loss rates increase more quickly than the linearly increasing flow rates, and losses increase. The increase is less

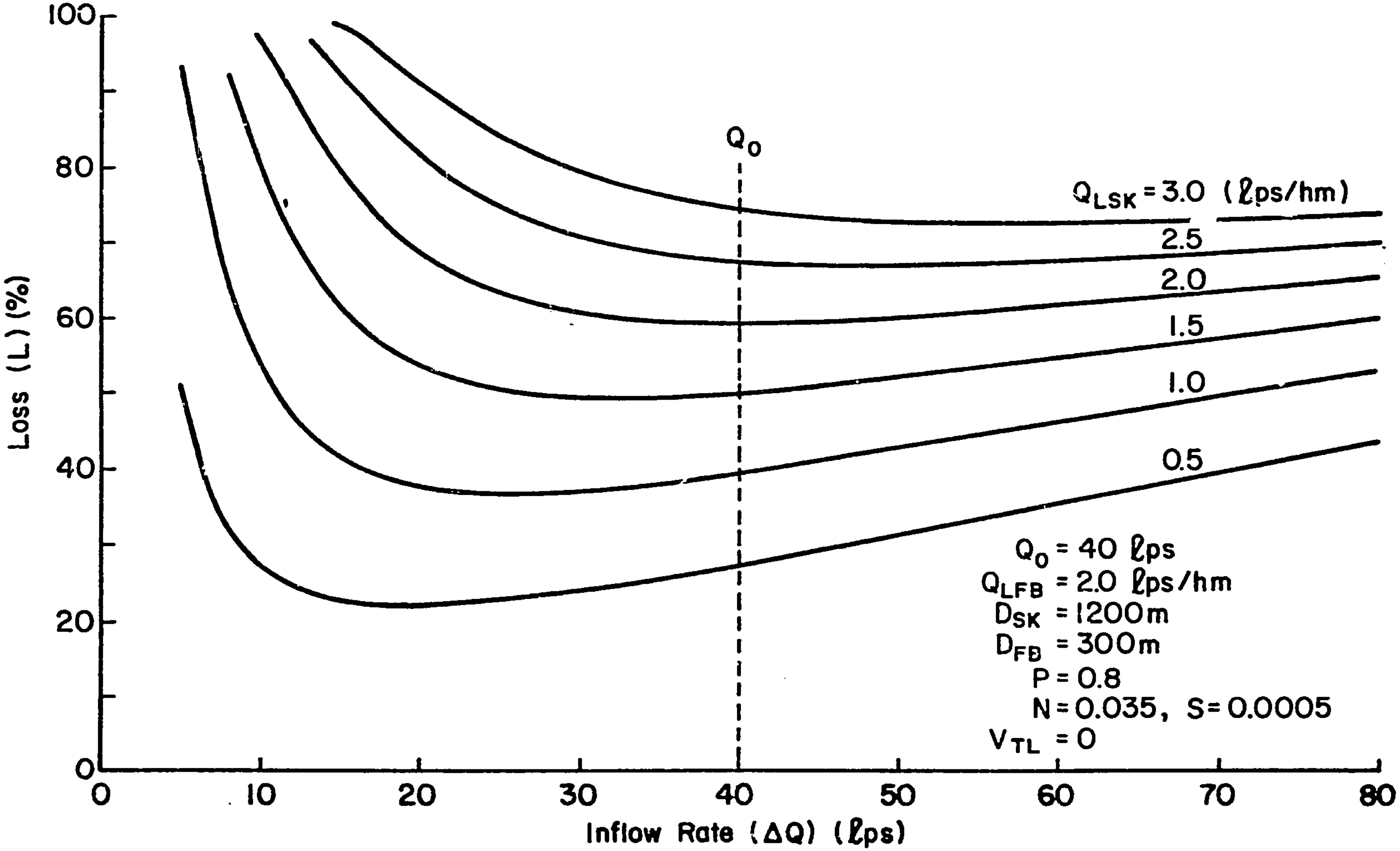


Figure 27. Percent conveyance losses vs. inflow rate fluctuations (ΔQ) from an original value of 40 lps (Q_o) for different initial sarkari khal loss rates (Q_{LSK}).

pronounced at high initial loss rates since the flows in the channel decrease more rapidly and the remaining flow in the channel at a given distance is not as much greater than the flow which would remain if the usual inflow rate were occurring.

Figure 28 more explicitly depicts the efficiency of the extra water added to a given channel. For example, the figure indicates that if 20 lps of flow is added to a watercourse which normally flows 40 lps and has a loss rate in the initial sarkari khal section of 1.0 lps/hm, only 39 percent or about 8 lps of the additional 20 lps will reach the field. Except when loss rates are low, only 30 to 50 percent of the extra added water reaches a field lying 1500 m from the mogha.

These figures predict that adding 40 lps of tubewell water to a watercourse which normally carries 40 lps or canal water will only result in an additional 12 to 14 lps reaching the field, or 30 to 35 percent of the tubewell water. They also estimate that increasing canal flows by 30 percent during periods of peak requirements will result in only about 20 percent more water at the field. Conversely, if canal water supplies are in short supply, instead of rotating the supplies at near the normal flow levels, which is presently required by the regime type design of the canals, if flows to all watercourses were reduced, more of the available water would reach the fields. For example, in a 40 lps watercourse which normally loses

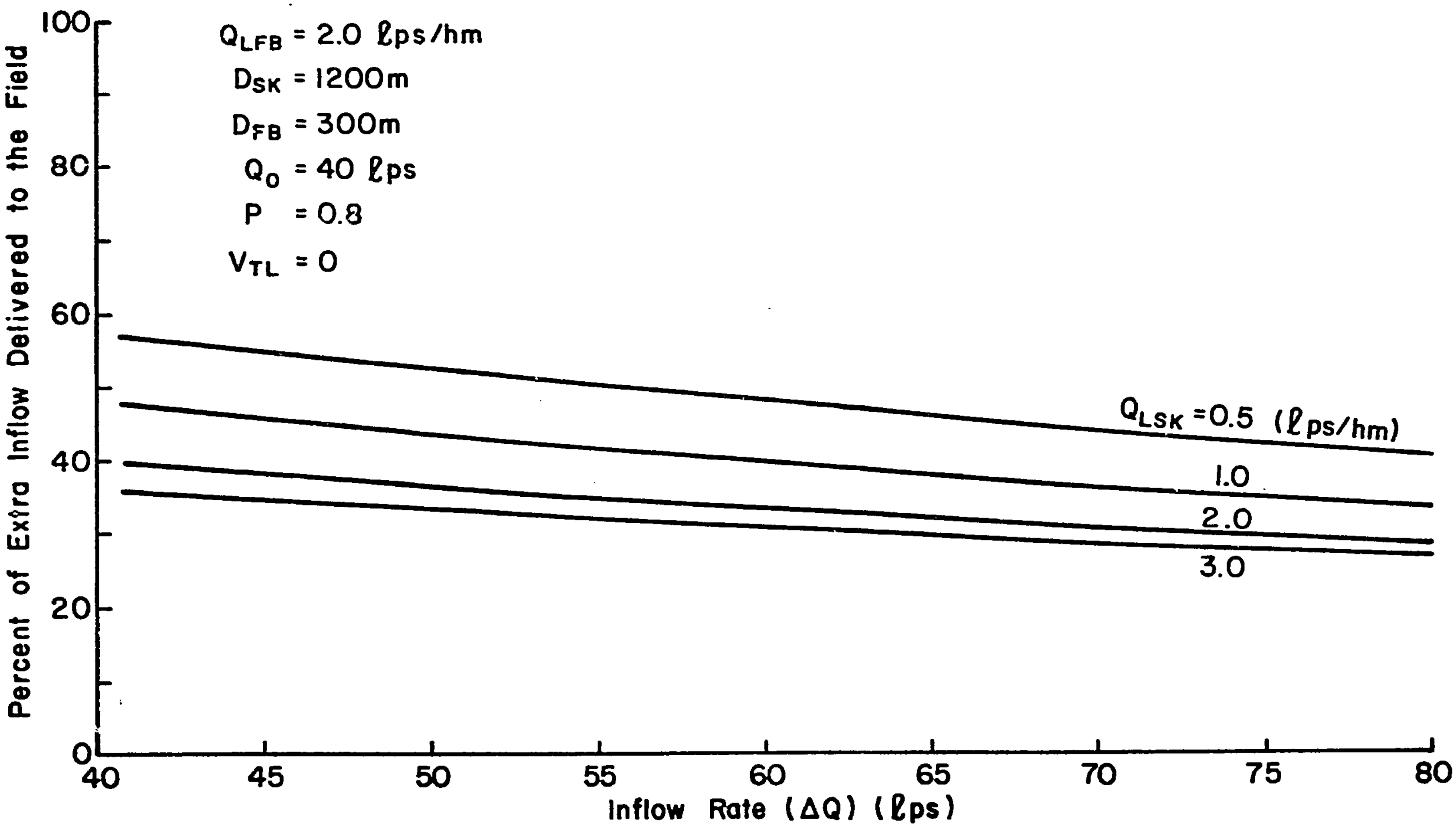


Figure 28. Percent of extra inflow to a watercourse above a normal 40 lps which is delivered to the field at different initial sarkari khal loss rates (Q_{LSK}).

about 1.0 lps/hm in the initial section, decreasing the flow 30 percent to 28 lps would decrease field deliveries by only 6 lps.

Increases in inflow rates will generally result in only a small portion of the additional water reaching the fields unless the channels are redesigned to carry the increased flows. Downward fluctuations can lead to improved overall delivery efficiencies during water short periods.

Flow rates into farmers' branches will be increased by any program which decreases the losses in the sarkari khal channels, such as earthen renovation or channel lining. Figure 29 illustrates the effect of decreasing sarkari khal loss rates (in the initial section) from an original 2.0 lps/hm on total losses when different lengths of farmers' branches are utilized. The bottom line ($D_{FB} = 0$) shows that although the losses in the sarkari khal decrease rapidly as loss rates decrease, the total losses, especially when the farmer branch channels are longer, decrease much less; demonstrating the rapidly increasing losses in the farmers' branches. The vertical distance between the bottom curve and the desired D_{FB} curve at the reduced Q_{LSK} rate represents the losses in the farmers' branch. At a Q_{LSK} decrease of 50 percent, from 2.0 to 1.0 lps/hm, total losses to a field lying 300 m from the sarkari khal decrease only 10 percentage points, although losses to the branch entrance decreased 20 percentage points. Nearly 50 percent of the total losses will occur in the 300 m branch.

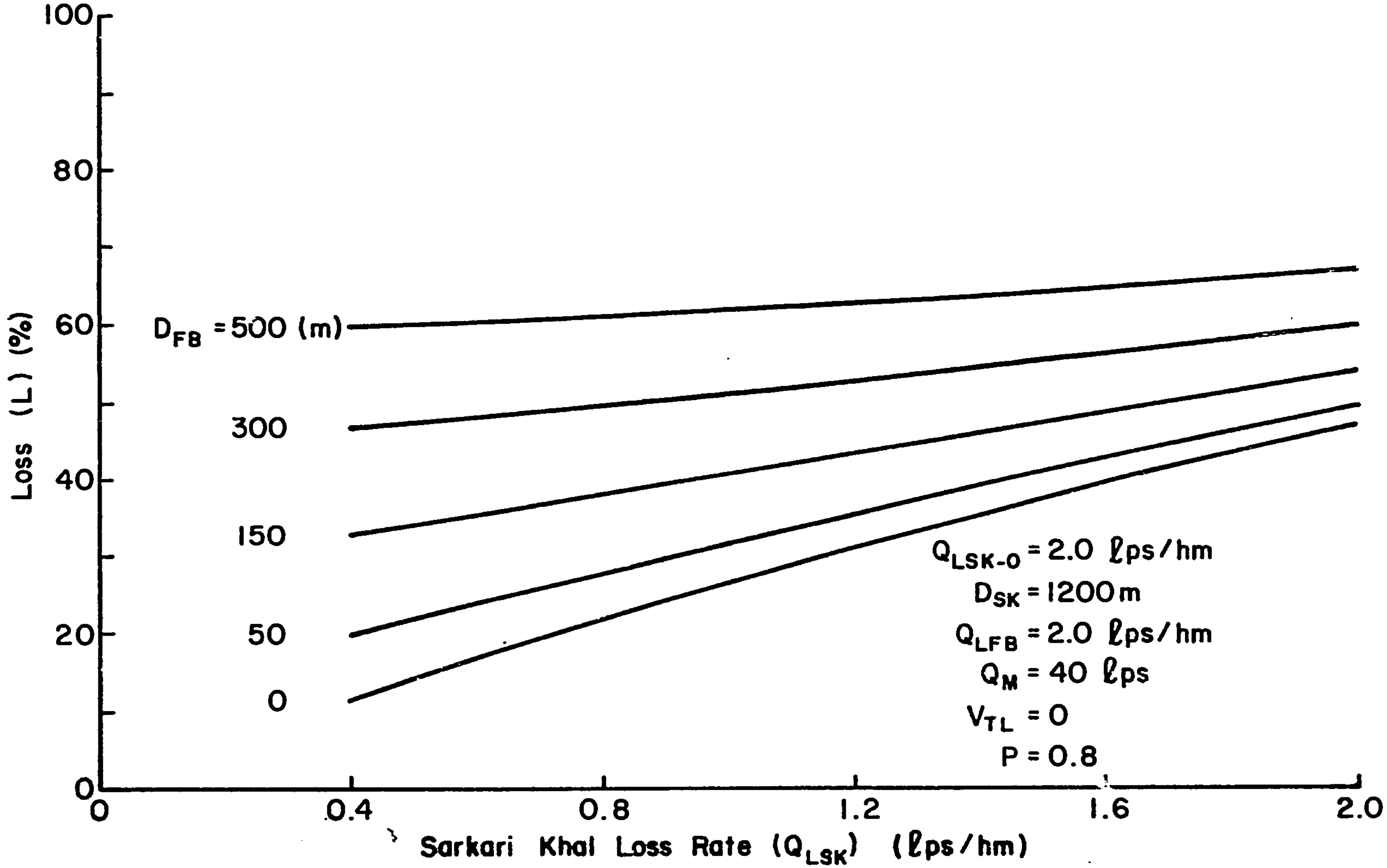


Figure 29. Percent conveyance losses (L) vs. initial sarkari khal loss rate (Q_{LSK}) for different farmer's branch lengths (D_{FB}).

If a sarkari khal improvement program does not include either enlargement of the farmers' branches or splitting the flow into two branches, much of the benefits of the program will not be realized at the field.

Cleaning Watercourse Channels to Reduce Roughness Coefficients

Changing vegetation and roughness coefficient in a channel also affects the flow depth and loss rate (Figures 16 and 17). Table 1 shows that Manning's roughness coefficient (n) commonly varies between 0.03 and 0.05 and occasionally is as high as 0.10 in watercourses with various amounts, types, and lengths of vegetation. Figure 30 shows the affect of such channel roughness changes on conveyance losses. An increase in roughness from 0.03 (a clean channel) to 0.05 (moderate to heavy grasses and vegetation), will result in an increase in losses of 10 to 20 percentage points or a decrease in field deliveries of 30 to 40 percent.

How often a channel is cleaned will thus also affect the losses. Figure 31 depicts the losses on a watercourse where the roughness coefficient varies from 0.03 after cleaning to 0.06 six months after cleaning, in an S-shaped curve. Three cleaning schedules are depicted: every six months, three months, and one month. The average losses for each schedule are 57 percent, 49 percent, and 41 percent respectively, compared to 38 percent in a permanently clean channel. Thirty-seven percent more water is predicted to reach the field under monthly cleaning as compared to semi-annual cleaning.

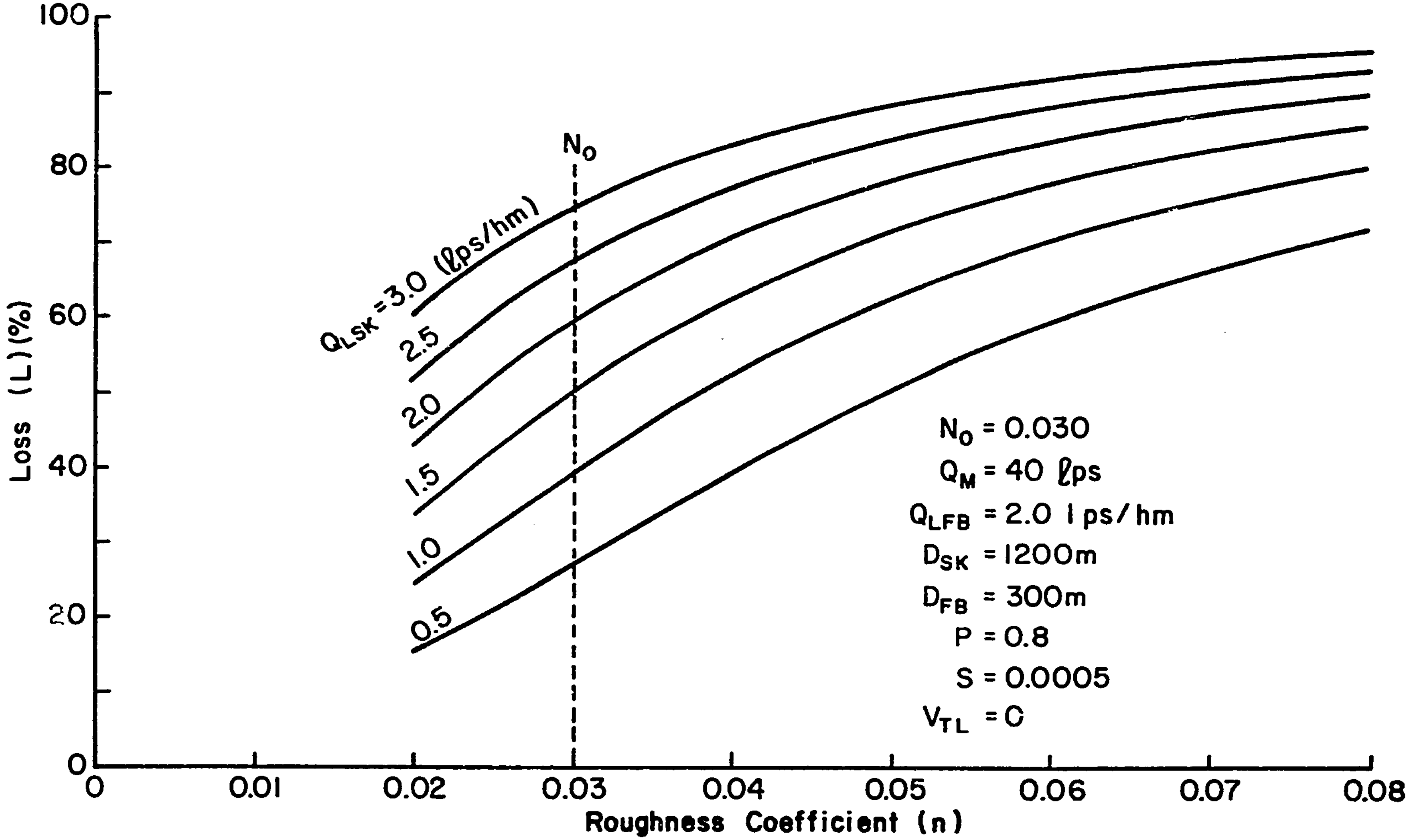


Figure 30. Percent conveyance losses vs. changes in Manning's roughness coefficient (n) from an original value of 0.03 (N_o) for different initial sarkari khal loss rates (Q_{LSK}).

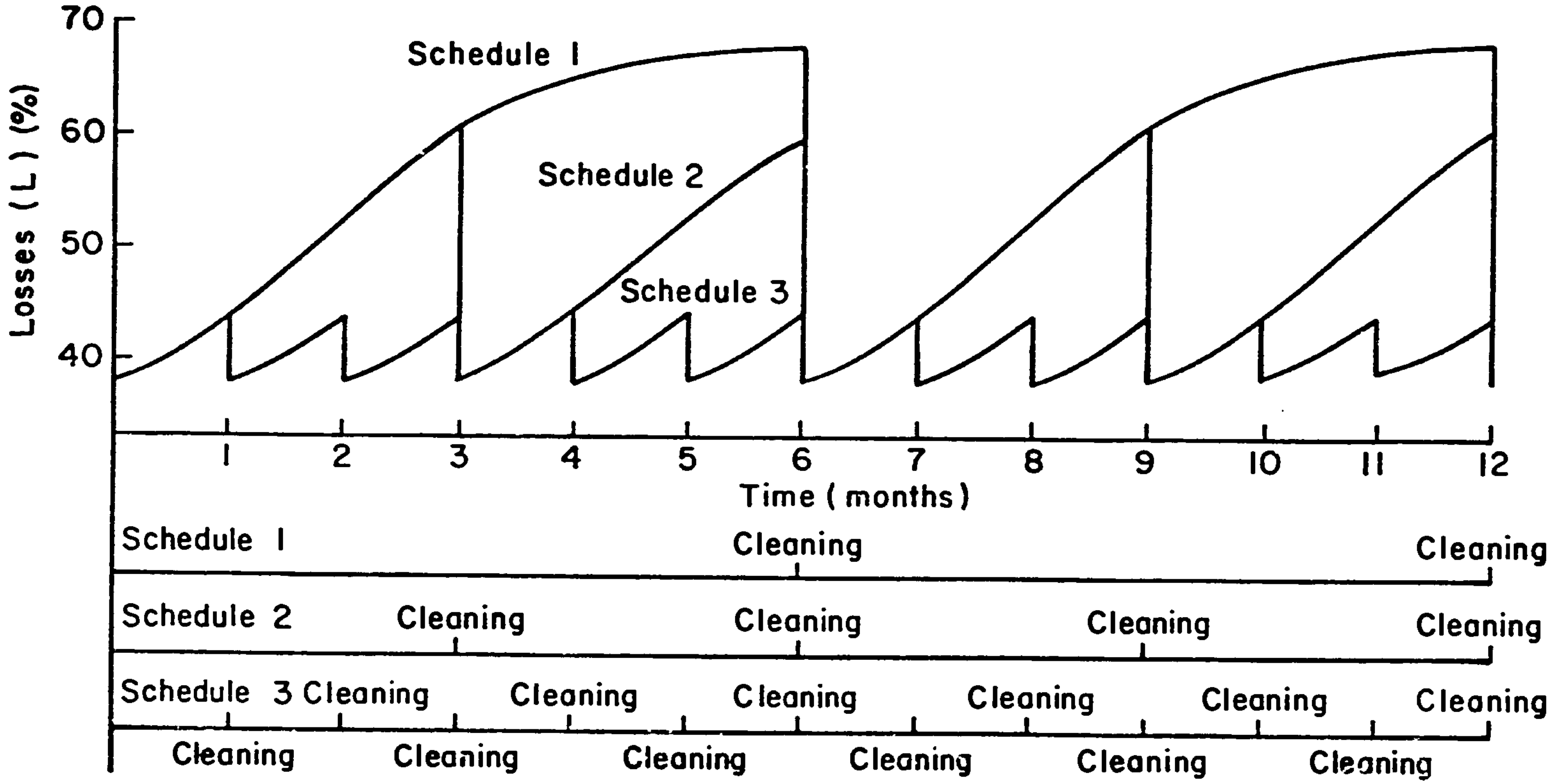


Figure 31. Percent conveyance losses over time on a hypothetical watercourse under a semi-annual, quarterly and monthly cleaning schedule, based on the watercourse model with the roughness coefficient (n) fluctuating between 0.03 and 0.06.

Figure 32, derived from Figure 31, shows the annual excess losses vs. number of cleanings. In Pakistan, the labor required to manually clean channels costs about $0.15 per hour and can clean about 15 m of channel per hour. The cost of cleaning 4000 m of sarkari khal channels of a watercourse would be about $40. Cleaning should be economically advantageous whenever the value of water saved is more than $40. The value of water in the Indus Basin varies widely seasonally, but tends to average about $100 per hectare-meter. Since an additional cleaning, which costs $40 will, according to Figure 32, save more than 0.4 ha-m/yr (or $40 worth) of water until cleanings total at least 12 per year, it would be a good investment for the farmers to clean their channels about monthly.

This calculation averages several annually variable factors such as vegetative growth rates, labor costs and scarcity, and the value of water to crop growth; and actual cleaning schedules should be adjusted for these factors. However, the results do indicate that under most conditions, more watercourse cleaning will be repaid in additional water at the field.

Since the real cause of the increased losses is the increased flow depth, a more direct indicator of the need to clean vegetation would be permanent staff gauges installed in the channel which would indicate flow depth increases resulting from vegetative growth. A 5 cm depth increase will result in higher losses by about 10 percentage points

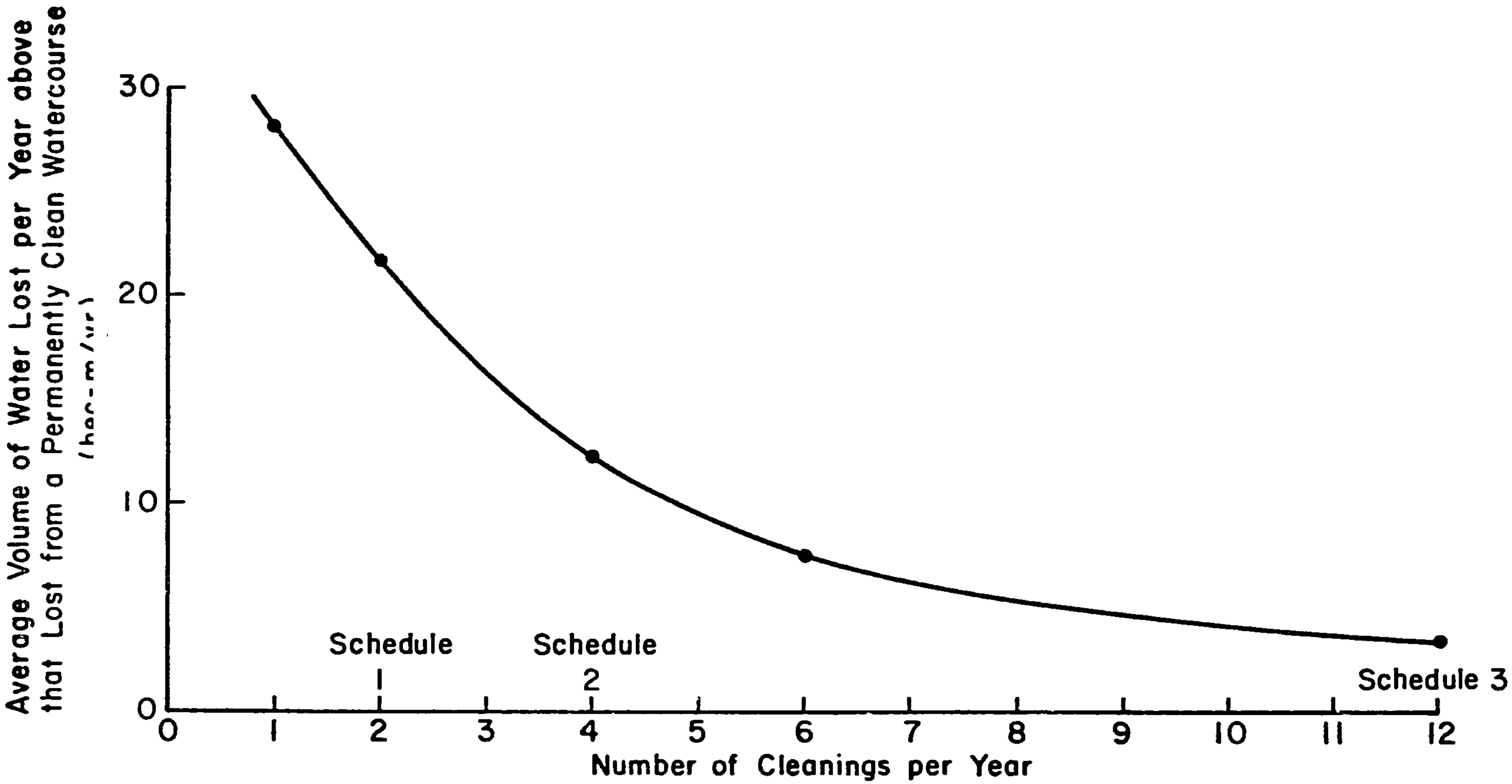


Figure 32. Excess conveyance water lost per year from a hypothetical watercourse vs. the number of cleanings.

while a 10 cm depth increase will result in 20 percentage points of additional losses.

Decreasing Seepage Rates into Watercourse Wetted Perimeters

The most potential for reducing watercourse conveyance losses lies in reducing the loss rates by reducing the seepage rates into the watercourse banks. Seepage rates into upper watercourse banks were measured to be as much as 100 times higher than field intake rates. The primary reason for this high rate was proposed to be leakage into macropores--the snake, insect and rodent burrows in the banks.

Seepage rates could be reduced either by eliminating the macropores, or by isolating them from the flowing water. Destroying and reconstructing the earthen banks will eliminate the macropores and has been shown to reduce loss rates to about 50 percent of the previous value, but it does not eliminate the burrowing insects and rodents. They soon return to the new banks and continue their activity. Only bank reconstruction at regular intervals can maintain a more homogeneous and less permeable bank medium. Although this is a feasible alternative in mechanized societies, and is in fact a recommended policy of some state extension personnel in the U.S., it is a fairly expensive alternative in a labor intensive society.

Although many alternatives for discouraging rats and ants from living in watercourse banks have been considered, because of the ideal ecosystem of the channel banks and the lack of other attractive environments, chasing them away seems unlikely. A method yet to be fully tested involves compacting the bank soils sufficiently hard that it is more difficult for the rodents or insects to dig, which may induce them to move to softer soils.

The other means to eliminate the burrowing activity would be to kill the rats and insects. Such a program would necessarily involve a widespread and continual concerted effort. The primary inhabitants of the banks are not normally harmful to the agricultural system in other ways (the burrowing bank rat is usually nekosia, or the "blind rat" which eats primarily grass roots, while the grain feeding and storing bandicoot rat lives primarily under fields). Consequently it will be difficult to motivate farmers to maintain such a control program.

The other alternative to reduce the macropore induced losses is to build the banks in such a way that rat and insect burrowing does not lead to higher loss rates. One obvious but expensive method is to line the channel wetted perimeter with some type of hard material suoh as brick masonry or concrete through which the rodents cannot burrow. However, as has been experienced in the field, if the lining cracks or if leaks form, they often interconnect with the extensive burrow system in the bank and can cause loss rates nearly as high as is experienced in unlined sections. Watercourse lining methods, costs, and benefits should continue to be studied to determine their feasibility.

Since the seepage rates are high only into the banks and especially in the upper portion of the banks, partial lining of the channel sides with earthen beds is a cheaper alternative which has shown promise in the field. Even impermeable bank cores buried near the inside of the upper bank would be expected to isolate most macropores from the flowing water. Core materials such as concrete, soil cement, and plastic are being field tested, but preliminary results are discouraging.

Perhaps the lowest cost engineering technique that could reduce the effects of rat and insect activity on loss rates would be to dig a trench in each bank near the inside and firmly compact a soil core in the bank. This shorter term solution would destroy the continuity of present macropores into the banks and perhaps discourage further burrowing into the compacted soil. Field tests of soil cores have given encouraging but mixed results. The technique, like bank reconstruction, is a temporary solution which must be repeated at intervals to be effective.

Chapter 9

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

1. Water losses from the tertiary irrigation conveyance systems (watercourses) in the Indus Basin are large and generally vary from 30 to 50 percent of the inflow.
2. A convenient unit for describing water loss from earthen channels is in terms of a flow loss rate with units of volume per unit time per unit length of channel (lps/hm).
3. Watercourse loss rates are directly related to flow rates and tend to increase about 80 percent as fast as flow rates.
4. Watercourse loss rates are inversely related to the percent usage time (T) of a channel and decrease about two percent for each unit increase in usage time in the range of 10 percent $<$ T $<$ 30 percent.
5. Watercourse loss rates are directly related to the elevation of the channel relative to the elevation of the surrounding land surface (ΔE), and increase about 2.5 percent for each centimeter increase in ΔE.
6. Farmer's branch loss rates are about double those in sarkari khal sections, primarily because of less usage time.
7. Public tubewell (SCARP) augmented watercourse loss rates are much higher than nonaugmented watercourse loss rates, partially because of the larger flow rates.

8. Earthen channel loss rates are very sensitive to flow depth changes and factors which affect flow depths such as fluctuations in inflow rates and roughness coefficients, and checking up of the water level.
9. Leakage rates into upper channel banks are often 100 times higher than into the channel beds and as much as 100 times higher than infiltration rates into surrounding field surfaces, and are thought to be the result of leakage into macropores (insect and rodent burrows).
10. Macropore induced losses cause large variability in measured loss rates, even in adjoining channel sections, and decrease the ability of models to predict loss rates.

Recommendations

1. Watercourse areas and flow rates should be subdivided when inflows are greater than 50 lps. The resulting increased channel usage time and/or reduced conveyance distance will lead to reduced losses.
2. Small capacity tubewells scattered around a watercourse command area are preferable to one large tubewell near the watercourse head since conveyance distances and thus losses will be reduced, and because the water need not be mixed with the canal flow.
3. Small rectangular fields should be reorganized into 120 to 180 m long narrow level basins so that many of the farmers' branches can be eliminated and conveyance losses can be reduced.

4. Farmers should organize their fields and crops such that they can, whenever possible, irrigate adjacent plots during one irrigation turn and thus minimize their transient losses.
5. Only deviations from the warabundi irrigation turn rotation system which result in few additional channels being filled and drained, such as trading between near-by farmers, should be allowed.
6. Since increased inflows lead to greatly increased losses, inflows to an earthen channel should be maintained as constant as possible.
7. Channels should be redesigned for the increased flows before tubewell water is added to canal water in a watercourse, or large portions of the tubewell water will be lost from the conveyance system.
8. Cleaning and maintenance of earthen watercourse channels should be done approximately monthly, depending on the vegetative growth, labor availability, and water requirements.
9. If a program is undertaken to reduce the losses in sarkari khal channels, the farmers' branches must be renovated and enlarged or the water split into two branches so that the additional water is not lost from the overfull branches.
10. If only a few fields exist higher than the other commanded fields, the watercourse level should be designed to serve the lower fields and extra bank added to allow checking water up to serve the high fields.

Chapter 10

NEED FOR FURTHER RESEARCH

This study of existing earthen watercourses in Pakistan has indicated several problems where further research is needed both to further understand Indus Basin watercourses and to confidently generalize the findings to other areas. The logical next stage of the research in Pakistan should involve carrying out experiments specifically designed to test the functional relationships that were indicated in this study, including studies to:

1. verify the effects of usage time on loss rates,
2. verify the relationship between channel elevation and loss rates,
3. determine the effects of silt deposition on loss rates, and
4. better determine and describe the effects of rodent and insect burrows (macropores) on water loss rates.

Also, the results predicted by the watercourse model should be tested in full-scale field applications. These include:

1. splitting watercourse command areas to reduce flow rates and conveyance distances and increase channel usage time,
2. splitting the flow and warabundi down two major branches of an existing watercourse,

3. building watercourses lower and checking up the water to serve higher fields,
4. installing several small capacity tubewells spaced around a watercourse command area rather than one large tubewell at the head,
5. testing several channel cleaning and maintenance schedules,
6. undertaking rat and/or insect control programs on total watercourses,
7. testing methods and degrees of bank soil compaction and its effect on insect and rodent burrowing and loss rates,
8. reorganizing the fields on a watercourse into long narrow borders and eliminating as many farmers' branches as possible,
9. testing flexible warabundi rotations where trading between neighboring farmers is allowed, and
10. further testing bank cores and partial lining techniques to reduce loss rates.

LIST OF REFERENCES

Ahmed, B. and Bowers, S. A. 1978. "Improving Watercourse Conveyance Efficiency through Cleaning and Maintenance." In Improving Irrigation Water Management on Farms, Annual Technical Report, Colorado State University, Fort Collins, pp. 427-438.

Akram, M. and Kemper, W. D. 1976. "Watercourse Improvement: Methods, Costs, and Loss Rates at Tubewell 78 Watercourse." In Water Management Research in Arid and Subhumid Lands of Less Developed Countries, Annual Technical Report, Colorado State University, Fort Collins, pp. 91-104.

Akram, M., Kemper, W. D., and Bowers, S. A. 1978. "Effects of Cleaning a Watercourse on Rates of Water Loss." In Improving Irrigation Water Management on Farms, Annual Technical Report, Colorado State University, Fort Collins, pp. 439-459.

Allison, L. E. 1947. "Effect of Microorganisms on Permeability of Soil under Prolonged Submergence." Soil Science, 63:439-449, June.

Ashraf, M., Kemper, W. D., Chowdhry, M. M., Ahmed, B., and Trout, T. 1978. "Review of Water Loss Measurement in Pakistan." In Improving Irrigation Water Management on Farms, Annual Technical Report, Colorado State University, Fort Collins, pp. 206-229.

Biwas, M. R. and Biswas, A. K. 1979. Food, Climate and Agriculture. Wiley-Interscience Publication, New York.

Bowers, S. A., Ahmed, B., Khan, Ch. M. H., and Azeem, M. 1976. "Water Losses from Watercourse 62394L (Tubewell MN56)." In Water Management Research in Arid and Subhumid Lands of Less Developed Countries, Annual Technical Report, Colorado State University, Fort Collins, pp. 105-116.

Bowers, S. A. and Wahla, M. 1978. "Water Conveyance Losses on Tubewell 81-R Watercourse after Improvement." In Improving Irrigation Water Management on Farms, Annual Technical Report, Colorado State University, Fort Collins, pp. 273-289.

Brockway, C. E. 1973. "Investigation of Natural Sealing Effects in Irrigation Channels." Water Resources Research Institute, University of Idaho, Moscow, June.

Brockway, D. E. and Worstell, R. V. 1969. "Field Evaluation of Seepage Measurement Methods." In Proceedings, Second Seepage Symposium, Phoenix, Arizona, March 25-27, 1968, ARS 41-147.

Cheema, M. A., Akram, M., Yasin, M., and Bowers, S. A. 1976. "Water Loss Measurements on Watercourse 50440L (Tubewell 51)." In Water Management Research in Arid and Sub-humid Lands of Less Developed Countries, Annual Technical Report, Colorado State University, Fort Collins, pp. 83-90.

Chow, V.T. 1959. Open-Channel Hydraulics. McGraw-Hill Book Company, New York.

Clyma, W., Ali, A., and Ashraf, M. 1975. "Watercourse Losses." In Water Management Research in Arid and Sub-humid Lands of Less Developed Countries, Annual Technical Report, Colorado State University, Fort Collins, pp. 170-191.

Dadayev, G. T. 1957. "Soil Stabilization in Irrigation Constructions." In Third Congress on Irrigation and Drainage Transactions, Vol. II, Question 7, R20, pp. 7.345-7.368, International Commission of Irrigation and Drainage.

Gibb, Sir Alexander and Partners, International Land Development Consultants N.V., and Huntington Technical Services, Ltd. 1966. Programme for the Development of Irrigation and Agriculture in West Pakistan, Metcalf and Cooper, Ltd., London.

Hanson, E. G. 1966. "The Seepage Problem Defined." Paper No. 66-728, presented at the American Society of Agricultural Engineers Winter Meeting, Chicago, Illinois, December 6-9.

Hussian, M. and Ahmed, N. 1969. "Salt Balance and Alkali Hazards in the Indus Basin," Directorate of Land Reclamation Research Publication No. 17, Volume II, Lahore.

Jahania, Ch. M. H. 1973. The Canal and Drainage Act, 1873 (as modified up to 1976). Mansoor Book House, 2 Katchery Road, Anarkali, Lahore, Pakistan.

Kemper, W. D., Clyma, W., Akram, M., Haif, M. and Parvez, I. 1975. "Watercourse Losses as Related to Composition and Condition of Banks." In Water Management Research in Arid and Sub-humid Lands of Less Developed Countries, Annual Report, Colorado State University, Fort Collins, pp. 137-153.

Kraatz, D. B. 1977. Irrigation Canal Lining. FAO Land and Water Development Series No. 1. Food and Agriculture Organization of the United Nations, Rome.

Lieftinck, P., Sadove, A. R., and Creyke, T. C. 1969. Water and Power Resources of West Pakistan. Johns Hopkins Press, Baltimore, Maryland.

Lowdermilk, M. K., Early, A. C., and Freeman, D. M. 1978. "Farm Irrigation Constraints and Farmers' Responses: Comprehensive Field Survey in Pakistan." Water Management Research Project, Technical Report No. 48, Colorado State University, Fort Collins, (6 volumes).

Mahmood, K. 1973. "Planning Sediment Distribution in Surface Irrigation Systems." Water Management Technical Report No. 26, Colorado State University, Fort Collins.

Manning, R. 1891. "On the Flow of Water in Open Channels and Pipes." Transactions, Institution of Civil Engineers of Ireland, Vol. 20, pp. 161-207, Dublin.

Michel, A. 1967. The Indus Rivers. Yale University Press, New Haven, Connecticut.

Muckel, D. C. 1951. "Research in Water Spreading." Proceedings ASCE Irrigation Division, Vol. 77, Separate No. 111, December.

Norse, D. 1979. "Natural Resources, Development Strategies, and the World Food Problem." In Food, Climate, and Man, Ed. by M.R. and A.K. Biswas, Wiley-Interscience Publication, New York.

Papfalvy, F., Starosolszky, E., and Szigyarta, E. Z. 1969. "Reduction of Seepage Losses from Irrigation Canals as a Result of Siltation." In Proceedings of the Thirteenth Congress of the International Association for Hydraulic Research, Vol. 4, Subject D, No. 33, pp. 299-309.

Planning Division, Water Resources Section, Government of Pakistan. 1977. "Information on Water Resources of Pakistan." Government of Pakistan, Islamabad.

Punjab On Farm Water Management Development Project. 1978. "Watercourse Losses in Sahiwal Tehsil." Punjab Department of Agriculture, Lahore.

Robinson, A. B. and Rohwer, C. 1952. "Study of Seepage Losses from Irrigation Channels." Progress Report for 1951. USDA, SCS, Fort Collins, Colorado, April 25, 1951.

Robinson, A. R., Jr. and Rohwer, C. 1957. "Measurement of Canal Seepage." ASCE Transactions 122:347-363.

Rohwer, C. and Stout, O. V. 1948. "Seepage Losses from Irrigation Channels." Colorado Agricultural Experiment Station, Technical Bulletin No. 38, Fort Collins.

Skogerboe, G. V., Bennett, R., and Walker, W. 1973. "Selection and Installation of Cutthroat Flumes for Measuring Irrigation and Drainage Water." Colorado Agricultural Experiment Station, Technical Bulletin No. 120, Fort Collins.

Trout, T. and Reuss, J. 1978. "Utilizing Irrigation Seepage for Groundwater Storage." In Improving Irrigation Water Management on Farms, Annual Technical Report, Colorado State University, Fort Collins, pp. 491-500.

Worstell, R. V. 1976. "Estimating Seepage Losses from Canal Systems." Journal of Irrigation and Drainage Division, ASCE, 102:IRI,137-147, March.

Worthington, E. B., ed. 1977. Arid Land Irrigation in Developing Countries. Pergamon Press, Elmsford, New York.

APPENDIX

Table A-1. Regression analyses of survey watercourse data.

Watercourse Code*	N	Data means	Regression equation	** r^2	*** F	**** Sig. %
101-2	7	$\bar{Q}_L = 1.11$	$Q_L =$			< 70
		$\bar{Q}_{L-A} = 0.91$	$Q_{L-A} = 0.76 + 0.012D$	.587	6.8	95
		$\bar{E} = .69$	$E = 1.03 - 0.021D$	.939	76.8	99
		$\bar{E}_A = .73$	$E_A = 1.05 - 0.020D$	.947	89.9	99
		$\bar{D} = 16.4$	$E = 1.11e^{-.031D}$	.913	52.8	99
		$\bar{Q}_M = 61.6$	$E_A = 1.12e^{-.027D}$	.921	62.6	99
			$Q_L =$			< 70
			$Q_{L-A} =$			< 70
			$Q_L =$			< 70
			$Q_{L-A} =$			< 70
104-1	8	$\bar{Q}_L = 2.97$	$Q_L = 6.03 - .307D$	.348	3.6	89
		$\bar{Q}_{L-A} = 2.32$	$Q_{L-A} = 4.23 - .192D$	.261	2.1	80
		$\bar{E} = .49$	$E =$			< 70
		$\bar{E}_a = .60$	$E_A =$			< 70
		$\bar{D} = 10.0$	$E =$			< 70
		$\bar{Q}_M = 53.6$	$E_A =$			< 70
			$Q_L =$			< 70
			$Q_{L-A} =$			< 70
			$Q_L =$			< 70
			$Q_{L-A} =$			< 70

*Code number used in Lowdermilk, et al. (1978)
**Coefficient of determination
***F statistic
****Level of significance in percent
Units: Q_L, Q_{L-A}: lps/hm
Q_M : lps
D : 100hm

Table A-1. Regression analyses of survey watercourse data (cont'd).

Watercourse Code*	N	Data means	Regression equation	r^2 **	F ***	Sig. % ****
104-2	14	$\bar{Q}_L = 2.60$	$Q_L = 4.20 - 0.259D$	.242	3.8	93
		$\bar{Q}_{L-A} = 1.95$	$Q_{L-A} = 2.34 - 0.121D$	.189	2.8	88
		$\bar{E} = .56$	$E =$			< 70
		$\bar{E}_A = .69$	$E_A = .81 - .019D$	.265	4.3	94
		$\bar{D} = 6.2$	$E =$			< 70
		$\bar{Q}_M = 21.9$	$E_A = .78e^{-.026D}$	.218	3.3	91
			$Q_L = -3.28 + 0.268Q_M$	.208	3.1	90
			$Q_{L-A} = -1.77 + 0.154Q_M$	.244	3.9	93
			$Q_L =$			< 70
			$Q_{L-A} =$			< 70
104-3	5	$\bar{Q}_L = 0.84$	$Q_L = 4.20 - 0.259D$	.242	3.8	93
		$\bar{Q}_{L-A} = 0.53$	$Q_{L-A} = 2.34 - 0.121D$	.189	2.8	88
		$\bar{E} = .61$	$E =$			< 70
		$\bar{E}_a = .75$	$E_A = .81 - .019D$	.265	4.3	94
		$\bar{D} = 3.5$	$E =$			< 70
		$\bar{Q}_M = 8.0$	$E_A = .78e^{-.026D}$	.218	3.3	91
			$Q_L =$			< 70
			$Q_{L-A} =$			< 70
			$Q_L =$			< 70
			$Q_{L-A} =$			< 70

*Code number used in Lowdermilk, et al. (1978)
**Coefficient of determination
***F statistic
****Level of significance in percent

Units: Q_L, Q_{L-A}: lps/hm
Q_M : lps
D : 100hm

Table A-1. Regression analyses of survey watercourse data (cont'd).

Watercourse Code*	N	Data means	Regression equation	r^2 **	F ***	Sig. % ****
105-1	7	$\overline{Q}_L = 5.94$	$Q_L = 12.60 - 0.560D$	.457	4.2	90
		$\overline{Q}_{L-A} = 3.53$	$Q_{L-A} = 6.87 - 0.282D$	.480	4.6	91
		$\overline{E} = .64$	$E = .78 - 0.012D$	.404	3.4	87
		$\overline{E}_A = .71$	$E_A = .88 - 0.014D$	.526	5.5	93
		$\overline{D} = 11.8$	$E = .77e^{-019D}$	.302	2.2	80
		$\overline{Q}_M = 76.7$	$E_A = .86e^{-.019D}$	.427	3.7	89
			$Q_L = -10.49 + 0.215Q_M$	.443	4.0	90
			$Q_{L-A} = -4.54 + 0.105Q_M$	.443	4.0	90
			$Q_L = .16e^{+.039Q_M}$	.733	13.8	99
			$Q_{L-A} = .23e^{+.030Q_M}$	.659	9.7	97
106-1	7	$\overline{Q}_L = 8.07$	$Q_L = 20.92 - 1.242D$	.496	4.9	92
		$\overline{Q}_{L-A} = 6.18$	$Q_{L-A} = 13.77 - 0.735D$	.388	3.2	86
		$\overline{E} = .47$	$E =$			< 70
		$\overline{E}_a = .58$	$E_A =$			< 70
		$\overline{D} = 10.3$	$E =$			< 70
		$\overline{Q}_M = 127.9$	$E_A =$			< 70
			$Q_L = -3.21 + 0.89Q_M$	.432	3.8	89
			$Q_{L-A} = -2.17 + 0.065Q_M$	.526	5.6	93
			$Q_L = .95e^{+.015Q_M}$	.654	9.5	97
			$Q_{L-A} = .79e^{+.014Q_M}$	.711	12.3	98

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**Coefficient of determination
***F statistic
****Level of significance in percent

Units: Q_L, Q_{L-A}: lps/hm
Q_M : lps
D : 100hm

Table A-1. Regression analyses of survey watercourse data (cont'd).

Watercourse Code*	N	Data means	Regression equation	** r^2	*** F	**** Sig. %
107-1	13	$\overline{Q}_L$ = 1.94	Q_L = 3.13 - 0.061D	.429	8.3	98
		$\overline{Q}_{L-A}$ = 1.52	Q_{L-A} = 2.34 - 0.041D	.431	8.4	98
		$\overline{E}$ = .51	E = .65 - 0.007D	.462	9.5	99
		$\overline{E}_A$ = .58	E_A = .74 - 0.008D	.643	20.0	99
		$\overline{D}$ = 19.8	E = $.65e^{-.014D}$	.599	16.4	99
		$\overline{Q}_M$ = 54.6	E_A = $.75e^{-.014D}$	.765	35.8	99
			Q_L =			< 70
			Q_{L-A} =			< 70
			Q_L =			< 70
			Q_{L-A} =			< 70
107-3	9	$\overline{Q}_L$ = 1.39	Q_L = 2.19 - 0.052D	.660	11.7	99
		$\overline{Q}_{L-A}$ = 1.05	Q_{L-A} = 1.45 - 0.028D	.511	6.3	95
		$\overline{E}$ = .52	E = .75 - 0.015D	.574	9.4	98
		$\overline{E}_a$ = .66	E_A = .83 - 0.012D	.815	26.6	99
		$\overline{D}$ = 15.9	E = $.86e^{-.045D}$	.329	3.4	89
		$\overline{Q}_M$ = 33.2	E_A = $.86e^{-.020D}$	.774	20.6	99
			Q_L =			< 70
			Q_{L-A} =			< 70
			Q_L =			< 70
			Q_{L-A} =			< 70

*Code number used in Lowdermilk, et al. (1978)
**Coefficient of determination
***F statistic
****Level of significance in percent

Units: Q_L, Q_{L-A}: lps/hm
Q_M : lps
D : 100hm

Table A-1. Regression analyses of survey watercourse data (cont'd).

Watercourse Code*	N	Data means	Regression equation	** r^2	*** F	**** Sig. %
119-1	6	$\overline{Q}_L = 4.73$	$Q_L = 6.97 - 0.149D$	.478	3.7	87
		$\overline{Q}_{L-A} = 3.52$	$Q_{L-A} = 4.92 - 0.093D$	.443	3.2	85
		$\overline{E} = .34$	$E = .47 - 0.008D$	.325	1.9	76
		$\overline{E}_A = .47$	$E_A = .62 - 0.010D$	.510	4.2	89
		$\overline{D} = 15.1$	$E = .47e^{-.031D}$	.499	4.0	88
		$\overline{Q}_M = 71.2$	$E_A = .64e^{-.027D}$	.660	7.8	95
			$Q_L =$			< 70
			$Q_{L-A} =$			< 70
			$Q_L =$			< 70
			$Q_{L-A} =$			< 70
119-2	5	$\overline{Q}_L = 2.69$	$Q_L =$			< 70
		$\overline{Q}_{L-A} = 2.26$	$Q_{L-A} =$			< 70
		$\overline{E} = .50$	$E =$			< 70
		$\overline{E}_a = .57$	$E_A =$			< 70
		$\overline{D} = 16.7$	$E =$			< 70
		$\overline{Q}_M = 76.1$	$E_A =$			< 70
			$Q_L = -1.09 + 0.050Q_M$	.682	6.4	91
			$Q_{L-A} = -1.28 + 0.047Q_M$	.812	12.9	96
			$Q_L = .30e^{+.027Q_M}$	.612	4.7	88
			$Q_{L-A} = .23e^{+.028Q_M}$	.725	7.9	93

*Code number used in Lowdermilk, et al. (1978)
**Coefficient of determination
***F statistic
****Level of significance in percent

Units: Q_L, Q_{L-A}: lps/hm
Q_M : lps
D : 100hm

Table A-1. Regression analyses of survey watercourse data (cont'd).

Watercourse Code*	N	Data means	Regression equation	** r^2	*** F	**** Sig. %
110-2	7	$\overline{Q}_L = 1.95$	$Q_L =$			70
		$\overline{Q}_{L-A} = 1.57$	$Q_{L-A} =$			70
		$\overline{E} = .41$	$E = .81 - 0.030D$	.670	10.2	98
		$\overline{E}_A = .50$	$E_A = .94 - 0.032D$	.811	21.4	99
		$\overline{D} = 13.5$	$E = 1.33e^{-.102D}$	.640	8.9	97
		$\overline{Q}_M = 40.9$	$E_A = 1.24e^{-.075D}$	.772	17.0	99
			$Q_L =$			< 70
			$Q_{L-A} =$			< 70
			$Q_L =$			< 70
			$Q_{L-A} =$			< 70
110-3	11	$\overline{Q}_L = 7.05$	$Q_L = 9.44 - 1.337D$	.273	3.4	90
		$\overline{Q}_{L-A} = 3.67$	$Q_{L-A} = 4.67 - 0.561D$	.212	2.4	85
		$\overline{E} = .70$	$E = .77 - 0.034D$	.269	3.3	90
		$\overline{E}_a = .83$	$E_A = .89 - 0.033D$	.521	9.8	99
		$\overline{D} = 1.8$	$E = .76e^{-.051D}$	.274	3.4	90
		$\overline{Q}_M = 26.9$	$E_A = .89e^{-.041D}$	.537	10.5	99
			$Q_L = 2.91 + 0.153Q_M$	.121	1.2	70
			$Q_{L-A} = 1.27 + 0.089Q_M$	.180	2.0	81
			$Q_L = 1.68e^{+.042Q_M}$	.235	2.8	87
			$Q_{L-A} = 0.78e^{+.048Q_M}$	.331	4.5	94

*Code number used in Lowdermilk, et al. (1978)
**Coefficient of determination
***F statistic
****Level of significance in percent
Units: Q_L, Q_{L-A}: lps/hm
Q_M : lps
D : 100hm

Table A-1. Regression analyses of survey watercourse data.(cont'd).

Watercourse Code*	N	Data means	Regression equation	r^2**	F***	Sig. %****
111-1	12	$\bar{Q}_L = 6.03$	$Q_L =$			< 70
		$\bar{Q}_{L-A} = 4.23$	$Q_{L-A} =$			< 70
		$\bar{E} = .44$	$E =$			< 70
		$\bar{E}_A = .59$	$E_A = .78 - 0.030D$	.192	2.4	85
		$\bar{D} = 6.1$	$E =$			< 70
		$\bar{Q}_M = 60.2$	$E_A = .73e^{-.042D}$	.123	1.4	74
			$Q_L = -1.35 + 0.122Q_M$	.479	9.2	99
			$Q_{L-A} = -1.00 + 0.087Q_M$	.517	10.7	99
			$Q_L = .89e^{+.039Q_M}$	.211	2.7	87
			$Q_{L-A} = .88e^{+.025Q_M}$	.695	22.9	99
111-2	10	$\bar{Q}_L = 3.43$	$Q_L = 7.87 - 0.569D$	.398	5.28	95
		$\bar{Q}_{L-A} = 2.38$	$Q_{L-A} = 5.15 - 0.352D$	.337	4.0	92
		$\bar{E} = .36$	$E = .12 + 0.031D$	.323	3.8	91
		$\bar{E}_a = .53$	$E_A =$			< 70
		$\bar{D} = 7.9$	$E = .11e^{+.113D}$	.151	1.4	73
		$\bar{Q}_M = 30.4$	$E_A =$			< 70
			$Q_L = -.84 + 0.139Q_M$	.558	10.1	99
			$Q_{L-A} = -.70 + 0.101Q_M$	.653	15.1	99
			$Q_L = .65e^{+.041Q_M}$	.528	8.9	98
			$Q_{L-A} = .50e^{+.040Q_M}$	.594	11.7	99

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Units: Q_L, Q_{L-A}: lps/hm
Q_M : lps
D : 100hm

Table A-1. Regression analyses of survey watercourse data (cont'd).

Watercourse Code*	N	Data means	Regression equation	r^2 **	F ***	Sig.% ****
112-1	14	$\bar{Q}_L = 2.60$	$Q_L = 5.65 - 0.189D$	.433	9.15	99
		$\bar{Q}_{L-A} = 2.08$	$Q_{L-A} = 4.08 - 0.125D$	.382	7.40	98
		$\bar{E} = .61$	$E =$			< 70
		$\bar{E}_A = .67$	$E_A =$			< 70
		$\bar{D} = 16.1$	$E =$			< 70
		$\bar{Q}_M = 74.3$	$E_A =$			< 70
			$Q_L = -1.62 + 0.057Q_M$	.319	5.6	96
			$Q_{L-A} = -1.64 + 0.050Q_M$	.494	11.7	99
			$Q_L = .25e^{+.026Q_M}$	.367	7.0	98
			$Q_{L-A} = .21e^{+.027Q_M}$	.418	8.6	99
112-2	7	$\bar{Q}_L = 0.93$	$Q_L = 1.68 - 0.110D$	.863	31.6	99
		$\bar{Q}_{L-A} = 0.57$	$Q_{L-A} = 0.97 - 0.055D$	.842	26.7	99
		$\bar{E} = .66$	$E =$			< 70
		$\bar{E}_a = .75$	$E_A =$			< 70
		$\bar{D} = 7.3$	$E =$			< 70
		$\bar{Q}_M = 11.7$	$E_A =$			< 70
			$Q_L = -.92 + 0.154Q_M$	.506	5.1	93
			$Q_{L-A} = -.52 + 0.093Q_M$	.694	11.3	98
			$Q_L = .02e^{+.309Q_M}$	.716	12.6	98
			$Q_{L-A} = .02e^{+.274Q_M}$	.779	17.6	99

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Units: Q_L, Q_{L-A}: lps/hm
Q_M : lps
D : 100hm

Table A-1. Regression analyses of survey watercourse data (cont'd).

Watercourse Code*	N	Data means	Regression equation	r^2 **	F ***	Sig. % ****
112-3	10	$\overline{Q}_L = 4.27$	$Q_L = 10.32 - 0.414D$	.474	7.2	97
		$\overline{Q}_{L-A} = 2.84$	$Q_{L-A} = 6.28 - 0.234D$	.491	7.7	98
		$\overline{E} = .43$	$E =$			< 70
		$\overline{E}_A = .55$	$E_A = .65 - 0.007D$	.340	4.1	92
		$\overline{D} = 14.7$	$E =$			< 70
		$\overline{Q}_M = 48.8$	$E_A = .65e^{-.013D}$	.304	3.5	90
			$Q_L = -4.73 + 0.184Q_M$	.318	3.7	91
			$Q_{L-A} = -2.72 + 0.114Q_M$	.394	5.2	95
			$Q_L = .39e^{+.037Q_M}$	.364	4.6	93
			$Q_{L-A} = .34e^{+.034Q_M}$	.435	6.1	96
113-1	8	$\overline{Q}_L = 1.86$	$Q_L =$			< 70
		$\overline{Q}_{L-A} = 1.35$	$Q_{L-A} =$			< 70
		$\overline{E} = .59$	$E =$			< 70
		$\overline{E}_a = .69$	$E_A =$			< 70
		$\overline{D} = 7.1$	$E =$			< 70
		$\overline{Q}_M = 28.7$	$E_A =$			< 70
			$Q_L = -0.02 + 0.066Q_M$	.476	5.46	94
			$Q_{L-A} = -0.10 + 0.051Q_M$	.495	5.87	95
			$Q_L = .52e^{+.033Q_M}$	.438	4.68	93
			$Q_{L-A} = .36e^{+.035Q_M}$	.504	6.11	95

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**Coefficient of determination
***F statistic
****Level of significance in percent

Units: Q_L, Q_{L-A}: lps/hm
Q_M : lps
D : 100hm

Table A-1. Regression analyses of survey watercourse data (cont'd).

Watercourse Code*	N	Data means	Regression equation	** r^2	*** F	**** Sig. %
113-2	5	$\overline{Q}_L = 3.34$	$Q_L =$			< 70
		$\overline{Q}_{L-A} = 2.65$	$Q_{L-A} =$			< 70
		$\overline{E} = .34$	$E = .71 - 0.037D$	.496	2.95	81
		$\overline{E}_A = .47$	$E_A = .88 - 0.041D$	.671	6.12	91
		$\overline{D} = 10.0$	$E = 1.00e^{-.112D}$	.619	4.88	89
		$\overline{Q}_M = 49.5$	$E_A = 1.14e^{-.091D}$	.754	9.19	94
			$Q_L =$			< 70
			$Q_{L-A} =$			< 70
			$Q_L =$			< 70
			$Q_{L-A} =$			< 70
113-3	12	$\overline{Q}_L = 2.41$	$Q_L = 8.93 - 0.572D$	.296	4.21	93
		$\overline{Q}_{L-A} = 1.75$	$Q_{L-A} = 5.75 - 0.352D$	.281	3.92	92
		$\overline{E} = .62$	$E =$			< 70
		$\overline{E}_a = .70$	$E_A =$			< 70
		$\overline{D} = 11.4$	$E =$			< 70
		$\overline{Q}_M = 41.8$	$E_A =$			< 70
			$Q_L = -5.02 + 0.178Q_M$	.950	191.1	99
			$Q_{L-A} = -2.97 + 0.113Q_M$	.948	183.2	99
			$Q_L = .24e^{+.037Q_M}$	.696	22.89	99
			$Q_{L-A} = .22e^{+.034Q_M}$	.667	20.06	99

*Code number used in Lowdermilk, et al. (1978)
**Coefficient of determination
***F statistic
****Level of significance in percent

Units: Q_L, Q_{L-A} : lps/hm
Q_M : lps
D : 100hm

Table A-1. Regression analyses of survey watercourse data (cont'd).

Watercourse Code*	N	Data means	Regression equation	r^2 **	F ***	Sig.% ****
114-1		$\overline{Q}_L = 4.46$	$Q_L =$			< 70
		$\overline{Q}_{L-A} = 2.59$	$Q_{L-A} =$			< 70
		$\overline{E} = .60$	$E = .74 - .028D$	.381	2.46	81
		$\overline{E}_A = .73$	$E_A = .87 - .031D$	.619	6.51	94
		$\overline{D} = 4.7$	$E = .71e^{-051D}$	.351	2.16	78
		$\overline{Q}_M = 31.9$	$E_A = .87e^{-046D}$	.628	6.77	94
			$Q_L = -0.56 + 0.157Q_M$	.460	3.41	86
			$Q_{L-A} = -0.49 + 0.096Q_M$	.650	7.42	95
			$Q_L = .41e^{+.054Q_M}$	.660	7.78	95
			$Q_{L-A} = .23e^{+.057Q_M}$	.765	12.99	98
114-2	9	$\overline{Q}_L = 2.41$	$Q_L =$			< 70
		$\overline{Q}_{L-A} = 1.68$	$Q_{L-A} =$			< 70
		$\overline{E} = .40$	$E = .57 - 0.014D$	.460	5.96	95
		$\overline{E}_a = .54$	$E_A = .75 - 0.017D$	.692	15.71	99
		$\overline{D} = 11.9$	$E = .47e^{-.038D}$	.364	4.02	91
		$\overline{Q}_M = 34.8$	$E_A = .74e^{-.037D}$	.778	24.53	99
			$Q_L =$			< 70
			$Q_{L-A} = 0.75 + 0.027Q_M$	.383	4.35	92
			$Q_L = .47e^{+.029Q_M}$	.407	4.80	93
			$Q_{L-A} = .26e^{+.035Q_M}$	.542	8.30	98

*Code number used in Lowdermilk, et al. (1978)
**Coefficient of determination
***F statistic
****Level of significance in percent

Units: Q_L, Q_{L-A}: lps/hm
Q_M : lps
D : 100hm

Table A-1. Regression analyses of survey watercourse data (cont'd).

Watercourse Code*	N	Data means	Regression equation	r^2 **	F ***	Sig.% ****
114-3	12	$\bar{Q}_L = 3.16$	$Q_L = 7.30 - 0.642D$	.263	3.57	91
		$\bar{Q}_{L-A} = 1.90$	$Q_{L-A} = 3.71 - 0.281D$	.210	2.65	86
		$\bar{E} = .46$	$E =$			< 70
		$\bar{E}_A = .62$	$E_A = .77 - 0.023D$	.284	3.98	93
		$\bar{D} = 6.4$	$E =$			< 70
		$\bar{Q}_M = 21.5$	$E_A = .82e^{-.049D}$	.306	4.41	94
			$Q_L =$			< 70
			$Q_{L-A} =$			< 70
			$Q_L = .56e^{+.049Q_M}$	.251	3.36	90
			$Q_{L-A} = .35e^{+.053Q_M}$	.333	4.99	95
114-4	6	$\bar{Q}_L = 0.93$	$Q_L = 0.10 + 0.140D$	.467	3.51	87
		$\bar{Q}_{L-A} = 0.70$	$Q_{L-A} = -0.02 + 0.118D$	.524	4.41	90
		$\bar{E} = .55$	$E = .84 - 0.049D$	.602	6.05	93
		$\bar{E}_a = .67$	$E_A = .95 - 0.047D$	.705	9.55	94
		$\bar{D} = 6.1$	$E = .90e^{-.090D}$	.506	4.09	89
		$\bar{Q}_M = 12.9$	$E_A = .99e^{-.068D}$	.633	6.92	94
			$Q_L = 0.13 + 0.063Q_M$	.766	13.08	98
			$Q_{L-A} = 0.34 + 0.051Q_M$	.784	14.47	98
			$Q_L = .28e^{+.080Q_M}$	.904	37.69	99
			$Q_{L-A} = .16e^{+.097Q_M}$	.919	45.60	99

*Code number used in Lowdermilk, et al. (1978)
**Coefficient of determination
***F statistic
****Level of significance in percent

Units: Q_L, Q_{L-A}: lps/hm
Q_M : lps
D : 100hm

Table A-1. Regression analyses of survey watercourse data (cont'd).

Watercourse Code*	N	Data means	Regression equation	r^2 **	F ***	Sig. % ****
115-1	5	$\bar{Q}_L$ = 5.66	Q_L =			< 70
		$\bar{Q}_{L-A}$ = 3.80	Q_{L-A} =			< 70
		$\bar{E}$ = .35	$E = .54 - 0.032D$	.649	5.53	90
		$\bar{E}_A$ = .54	$E_A = .78 - 0.040D$	.866	19.40	98
		$\bar{D}$ = 6.0	$E = .67e^{-.132D}$	.816	13.38	96
		$\bar{Q}_M$ = 42.8	$E_A = .90e^{-.097D}$	.924	36.38	99
			$Q_L = 1.11 + 0.107Q_M$	.345	1.58	70
			$Q_{L-A} = 0.23 + 0.083Q_M$	.557	3.77	85
			$Q_L = 2.26e^{+.019Q_M}$	.358	1.67	71
			$Q_{L-A} = 1.22e^{+.024Q_M}$	.643	5.40	90
115-2	8	$\bar{Q}_L$ = 2.60	$Q_L = 4.40 - 0.648D$	.235	1.85	78
		$\bar{Q}_{L-A}$ = 1.49	$Q_{L-A} = 2.29 - 0.283D$			< 70
		$\bar{E}$ = .61	E =			< 70
		$\bar{E}_a$ = .77	$E_A = .90 - 0.048D$	.276	2.29	82
		$\bar{D}$ = 2.8	E =			< 70
		$\bar{Q}_M$ = 16.5	$E_A = .90e^{-.060D}$	.270	2.21	81
			$Q_L = 0.22 + 0.142Q_M$	.222	1.71	76
			$Q_{L-A} = -0.01 + 0.091Q_M$	.287	2.42	83
			$Q_L = .53e^{+.082Q_M}$	.255	2.05	80
			$Q_{L-A} = .33e^{+.079Q_M}$	.260	2.11	80

*Code number used in Lowdermilk, et al. (1978)
**Coefficient of determination
***F statistic
****Level of significance in percent

Units: Q_L, Q_{L-A}: lps/hm
Q_M : lps
D : 100hm

Table A-1. Regression analyses of survey watercourse data (cont'd).

Watercourse Code*	N	Data means	Regression equation	** r^2	*** F	**** Sig. %
115-3	7	$\bar{Q}_L = 8.82$	$Q_L =$			<.70
		$\bar{Q}_{L-A} = 4.38$	$Q_{L-A} =$			<.70
		$\bar{E} = .61$	$E =$			<.70
		$\bar{E}_A = .71$	$E_A = .85 - 0.027D$	.551	6.13	.94
		$\bar{D} = 3.2$	$E =$			<.70
		$\bar{Q}_M = 15.2$	$E_A = .84e^{-.036D}$	.581	6.94	.95
			$Q_L = 9.65 + 1.210Q_M$	.488	4.77	.90
			$Q_{L-A} = -4.53 + 0.586Q_M$	.505	5.10	.93
			$Q_L =$			<.70
			$Q_{L-A} = 0.61e^{+.078Q_M}$	.263	1.79	.76
115-5	6	$\bar{Q}_L = 2.97$	$Q_L =$			<.70
		$\bar{Q}_{L-A} = 1.83$	$Q_{L-A} =$			<.70
		$\bar{E} = .62$	$E =$			<.70
		$\bar{E}_a = .76$	$E_A =$			<.70
		$\bar{D} = 4.0$	$E =$			<.70
		$\bar{Q}_M = 21.0$	$E_A =$			<.70
			$Q_L = 0.64 + 0.110Q_M$	.451	3.29	.86
			$Q_{L-A} = 0.16 + 0.079Q_M$	.600	6.00	.93
			$Q_L = .22e^{+.080Q_M}$	.487	3.80	.88
			$Q_{L-A} = .13e^{+.082Q_M}$	.534	4.58	.90

*Code number used in Lowdermilk, et al. (1978)
**Coefficient of determination
***F statistic
****Level of significance

Units: Q_L, Q_{L-A}: lps/hm
Q_M : lps
D : 100hm

Table A-1. Regression analyses of survey watercourse data.(cont'd).

Watercourse Code*	N	Data means	Regression equation	r^2 **	F ***	Sig. % ****
116-1	7	$\bar{Q}_L$ = 2.69	Q_L =			< 70
		$\bar{Q}_{L-A}$ = 1.86	Q_{L-A} =			< 70
		$\bar{E}$ = .38	$E = .73 - 0.076D$	.718	12.70	98
		$\bar{E}_A$ = .56	$E_A = .92 - 0.072D$	.833	24.92	99
		$\bar{D}$ = 5.2	$E = .82e^{-.204D}$	.482	4.65	92
		$\bar{Q}_M$ = 19.5	$E_A = .96e^{-.084D}$	.781	17.79	99
			$Q_L = -0.06 + 0.141Q_M$	.967	148.30	99
			$Q_{L-A} = -0.20 + 0.105Q_M$	.988	404.20	99
			$Q_L = .76e^{+.039Q_M}$	.748	14.81	99
			$Q_{L-A} = .48e^{+.041Q_M}$	.809	21.23	99
116-2	9	$\bar{Q}_L$ = 1.21	Q_L =			< 70
		$\bar{Q}_{L-A}$ = 0.83	Q_{L-A} =			< 70
		$\bar{E}$ = .34	$E = .66 - 0.035D$	.525	7.75	93
		$\bar{E}_a$ = .48	$E_A = .80 - 0.036D$	.729	18.87	99
		$\bar{D}$ = 9.1	$E = .53e^{-.088D}$	.535	8.06	97
		$\bar{Q}_M$ = 12.6	$E_A = .79e^{-.071D}$	.795	27.20	99
			Q_L =			< 70
			$Q_{L-A} = 0.30 + 0.042Q_M$	.271	2.60	85
			Q_L =			< 70
			Q_{L-A} =			70

*Code number used in Lowdermilk, et al. (1978)
**Coefficient of determination
***F statistic
****Level of significance in percent
Units: Q_L, Q_{L-A}: lps/hm
Q_M : lps
D : 100hm

PONDING LOSS MEASUREMENT

DATA SHEET

(Always note measurement units)

Date ____________________
Experimenter ______________

Purpose for measurement ________________________________
Location of Watercourse ________________________________
Location of test section (sq #/Ac#) ________________________________
Gauge reading at full supply level (FSL) ________________________
Time channel was full before testing began __________ hours
Channel section length ____________________

Water Surface Recession Data:

Time	Gauge Reading	Time	Gauge Reading	Time	Gauge Reading

Comments: ____

Channel Section Water Surface Width:

	Gauge Reading	Width	Ave. Width
Initial:	________	____, ____, ____, ____, ____,	________
	________	____, ____, ____, ____, ____,	________
Near FSL:	________	____, ____, ____, ____, ____,	________
	________	____, ____, ____, ____, ____,	________
Final:	________	____, ____, ____, ____, ____,	________

Additional Information:

1. Bank width at FSL
   Left Bank ______, ______, ______, ______, ______, Average __________
   Right Bank ______, ______, ______, ______, ______, Average __________

2. Elevation difference between FSL and surrounding fields:
   BM. rod reading __________ BM. description ________________
   FSL (rod reading) ________________________________
   Right side field rod reading ____, ____, ____, Ave ____, Δ Elev ___
   Left side field rod reading ____, ____, ____, Ave ____, Δ Elev ___
   Remarks (adjoining roads, etc.) ________________________

3. Cross-sections: Draw a typical cross-section giving top width and depth; include banks in drawing

Figure A-1. Sample ponding loss measurement data collection sheet.

4. How many days or hours per week does water flow in this section? ______________________

5. How far is the section from the mogha? ______________
Is the section in the Sarkari Khal or on a farmer's branch? ______________________

6. Flow rate: What is the measured or estimated flow at the section? ______________ cusecs.

7. Bank material: Estimate the soil type: ______________
Measure the bulk density: Sample Vol. __________; Sample dry wt. ___
Bulk Density __________.

8. Is there any visible leakage? _______ From what type of hole? ______________

Measure leakage volumetrically at different gauge readings and record gauge reading when leakage stopped. ______________

<u>type leak</u>	<u>gauge reading</u>	<u>loss rate (units)</u>

9. Adjoining crop losses:

Area of land adjoining the test section where crops have been damaged by seepage: ______________; crop: ______________

10. Describe the condition of the wetted perimeter; i.e., whether it is grassy or clean, the presence of rat or insect holts, etc: ______________
____
____
____

Figure A-1 (continued).

```
0: dim I[13],Q[50],O[50],D[50],K[50],R[20],P[20],C[20],E[20]
1: pclr
2: ent "X-Max",r4
3: ent "X Tick Interval",r5
4: ent "Decimal Fix",r6
5: fxd r6
6: scl 0,r4,0,100
7: xax 0,r5,0,r4,1
8: fxd 0
9: yax 0,10,0,100,1
10: csiz 2.5,2,1,0
11: plt r4/3,-10,1
12: dsp "Type X Axis Label,exc,cont";stp
13: csiz 2.5,2,1,90
14: plt -r4/17,35,1
15: lbl "LOSS (L) (%)"
16: csiz 2,2,1,0
17: enp "QM   (lps) (1)",I[1]
18: enp "QLSK(lps/100m(2)",I[2]
19: enp "QLFB(lps/100m(3)",I[3]
20: enp "DSK (100m) (4)",I[4]
21: enp "DFB (100m) (5)",I[5]
22: enp "LW  (100m) (6)",I[6]
23: enp "LD  (100m) (7)",I[7]
24: enp "P              (8)",I[8]
25: enp "QO   (LPS) (O)",O
26: enp "NO             (R)",R
27: enp "N              (9)",I[9]
28: enp "S   (m/m)  (10)",I[10]
29: enp "DEL E  (cm)(11)",I[11]
30: enp "DEL EO (cm)(G)",G
31: enp "T    (%)   (12)",I[12]
32: enp "TO   (%)   (U)",U
33: enp "Time  (hr) (13)",I[13]
34: enp "Indep. Variable",r3
35: ent "Initial Value",r0
36: ent "Final Value",r1
37: fxd 3;prt "I[]",r3,"From",r0,"To",r1
38: (r1-r0)/50)r2
39: "B":r0)I[r3]
40: "D":if I[1]=0 and I[9]=R;gto "A"
41: I[2]-.02I[2](I[12]-U))K
42: K+.025K(I[11]-G))K
43: .001I[1])Q[1];.001O)O[1];0)D[1];I[4]/50)Z;.001K/Q[1]^I[8])K
44: for I=1 to 49
45: Kexp(18(\(Q[I]I[9]/\I[10])-\(O[I]R/\I[10]))))K[I]
46: O[I]-ZKO[I]^I[8])O[I+1]
47: Q[I]-ZK[I]Q[I]^I[8])Q[I+1]
48: D[I]+Z)D[I+1]
49: next I
50: Q[50])R[1];O[50])P[1];0)E[1];I[5]/20)Y;.001I[3]/R[1]^I[8])C
51: for I=1 to 19
52: Cexp(18(\(R[I]I[9]/\I[10])-\(P[I]R/\I[10]))))C[I]
53: P[I]-YCP[I]^I[8])P[I+1]
54: R[I]-YC[I]R[I]^I[8])R[I+1]
55: E[I]+Y)E[I+1]
56: next I
57: 100(1-R[20]/Q[1]+(4.4I[6]-.5I[7])/36I[13]))L
58: gto "C"
59: "A":
60: I[1])Q;I[2])A;I[3])C;I[4])D;I[5])B;I[6])W;I[7])M;I[8])P;I[9])N;I[10])S
61: I[11])E;I[12])T;I[13])Z
62: A-.02A(T-U))K
63: K+.025K(E-G))K
64: ((-K/Q^P)(1-P)D+Q^(1-P))^(1/(1-P)))F
65: if F<=0;jmp 5
66: if I[r3]=r0;prt "QLSK=",K,"QLFB=",C
67: 100-100((-C/F^P)(1-P)B+F^(1-P))^(1/(1-P))/Q+(4.7W-.5M-.05/Q)/.36Z)L
68: "C":if L>100;jmp 2
69: plt I[r3],L
70: I[r3]+r2)I[r3]
71: if I[r3]<=r1;gto "D"
72: pen
73: prt "QLSK=",K,"QLFB=",C
74: ent "If new curve:0; if not 1",r8
75: if r8=0;dsp "Enter new parameter value 00)I[n]";stp ;gto "B"
76: end
*4894
```

Figure A-2. Hewlett-Packard 9825A minicomputer and 9872A plotter watercourse model program.

Section A (lines 1-16) setup and labels the plotter axis.

Section B (lines 17-39) allows input of the data in the units given (the independent variable is chosen using the number given in the enter statement).

Section C (lines 42-59) is used to iteratively calculate the loss with inflow rate (Q_M) or roughness coefficient (n) fluctuations.

1. Eqs. 37 and 38 are first used to adjust the initial loss rate for elevation (ΔE) and usage time (T) changes.
2. Eq. 39 is used to adjust loss rate for the flow rate and roughness changes.
3. Eq. 31 is used to calculate the flow rate after a distance iteration for both normal and changed Q and n values.
4. The two calculated Q values are reinserted into Eq. 39 to calculate the loss rate in the following distance iteration.
5. The procedure is repeated for the farmers' branch section.

Section D (lines 60-68) utilizes Eqs. 37, 38, and 36 to calculate loss when inflow rates and roughness coefficients are the normal values.

Section E (lines 72-80) plots the data and allows parameters to be changed to generate new curves.

Figure A-2 (continued).