BIBLIOGRAPHIC DATA SHEET

1. CONTROL NUMBER	2. SUBJECT CLASSIFICATION (695)
PN-AAJ-012	AP10-0000-0000

3. TITLE AND SUBTITLE (240)

Watercourse improvement manual

4. PERSONAL AUTHORS (100)

Trout, T. J.; Kemper, W. D.

5. CORPORATE AUTHORS (101)

Colo. State Univ. Engineering Research Center

6. DOCUMENT DATE (110)	7. NUMBER OF PAGES (120)	8. ARC NUMBER (170)
1980	266p.	631.7Ø2Ø2.T861

9. REFERENCE ORGANIZATION (130)

Colo. State

10. SUPPLEMENTARY NOTES (500)

(In Water management technical rpt. no. 58)

11. ABSTRACT (950)

The purpose of this manual is to assist both national and donor agency planners to determine the need for and carry out a program to improve the tertiary irrigation conveyance systems. Topics covered include evaluating and diagnosing problems in the present channel systems, proposing and testing solutions to the diagnosed problems, combining the solution techniques into improvement strategies, evaluating the improvement strategies and developing the institutions necessary to carry out the improvement programs. The manual deals both with processes, which will be of primary interest to the planners; and techniques, which would be useful to the engineers, economists and sociologists.

12. DESCRIPTORS (920)

Watercourses
Irrigation
Channel flow
Design
Strategy
Models
Manuals
Irrigation canals
Channels
Program management

13. PROJECT NUMBER (150)

931048900

14. CONTRACT NO.(140)	15. CONTRACT TYPE (140)
AID/ta-C-1411	Res.

16. TYPE OF DOCUMENT (160)

AID 590-7 (10-79)

C 31.70202
T 60-1

CONSORTIUM FOR
INTERNATIONAL
DEVELOPMENT

PN-AAJ-012

Colorado State University
University of Arizona
University of California at Davis
and at Riverside
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WATERCOURSE IMPROVEMENT MANUAL

Thomas J. Trout
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Water Management Research Project
Colorado State University
Fort Collins, Colorado
February 1980

WATER MANAGEMENT
TECHNICAL REPORT NO. 58

WATERCOURSE IMPROVEMENT MANUAL

WATER MANAGEMENT TECHNICAL REPORT 58

Prepared under support of
United States Agency for International Development
Contract AID/ta-C-1411.
All reported opinions, conclusions or
recommendations are those of the
authors and not those of the funding
agency or the United States Government.

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WATERCOURSE IMPROVEMENT MANUAL

SUMMARY

The purpose of this manual is to assist both national and donor agency planners to determine the need for and carry out a program to improve the tertiary irrigation conveyance systems. Topics covered include evaluating and diagnosing problems in the present channel systems, proposing and testing solutions to the diagnosed problems, combining the solution techniques into improvement strategies, evaluating the improvement strategies and developing the institutions necessary to carry out the improvement programs. The manual deals both with processes, which will be of primary interest to the planners; and techniques, which would be useful to the engineers, economists and sociologists.

ACKNOWLEDGMENTS

Many of the techniques and processes described in this manual were developed and tested in Pakistan as part of the Water Management Research Project. To name all of the people and institutions which supported and assisted this work would be impossible, but some of the more important participants should be acknowledged. Much of the early channel improvement work was carried out at the Mona Reclamation Experimental Project under the leadership of Chief Engineer Mian Moh'd Ashraf and Project Director Moh'd Munir. Much of the actual field work was carried out by Pakistani agricultural engineers, such as Waryam Ali Mohsin, Moh'd Akram, Mushtaq Gill, Shafiq Siddique and Zahid Saeed Khan. Several Colorado State University researchers in addition to the authors also contributed to the development of the techniques presented here including Alan Early, Wayne Clyma, John Reuss and Gaylord Skogerboe. Edward Sparling and Douglas Merrey assisted in the preparation of Sections 3 and 9, respectively.

Finally, none of this would have been possible without the patient and long term support and financial backing of both the Development Support Bureau and Mission to Pakistan of the U.S. Agency for International Development.

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NOMENCLATURE

A	cross-sectional flow area (m^2)
A_s	structure cross-sectional flow area (m^2)
a_1&a_2	empirical channel shape coefficients (Eq. 6)
B	channel bottom width (m)
b	empirical coefficient equivalent to the fractional change in loss rate per unit change in depth (m^{-1})
C	Chezy roughness coefficient
C_w	weir coefficient
D	channel water depth (m)
D_n	normal flow depth (m)
ΔD	depth fluctuation from the normal depth, $D-D_o$ (m)
ΔD_i	initial depth fluctuation at a point in a channel (m)
$\frac{dd}{dt}$	change in water level per unit time (cm/hr)
EL	water surface elevation (m)
ET	evapotranspiration
FB	freeboard allowance (m)
g	acceleration of gravity (9.81 m/sec^2)
H	head on a weir (m)
h_v	velocity head in a channel (m)
Δh	head loss through a structure (m)
L	channel length (100m)
L_s	bank side length (m)
L_w	weir length (m)
n	Manning's roughness coefficient
n_o	initial or base roughness coefficient
OL	operational water surface level (m)

Q	flow rate (lps)
Q_1	upstream flow rate (lps)
Q_2	downstream flow rate (lps)
Q_o	inflow rate (lps)
Q_D	design flow rate (lps)
Q_{DH}	design inflow rate at the head (lps)
Q_L	loss rate (lps/100m)
Q_{LP}	loss rate (%/100m)
Q_{LI}	initial instantaneous loss rate (%/100m)
Q_{LO}	loss rate at the normal flow depth, D_o (lps/100m)
R	hydraulic radius (m)
S	channel slope (m/m)
SR	seepage rate into a channel wetted perimeter (cm/hr)
TW	channel water surface top width (m)
V	flow velocity (m/sec)
V_s	flow velocity through a structure (m/sec)
WP	channel wetted perimeter length (m)
Z	channel side slope (horizontal/vertical)

SECTION 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 and population growth is rapid. 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 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 are 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, 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 handbook will discuss methods to improve the efficiency of the tertiary irrigation conveyance system, or the small channels (ditches, laterals, watercourses) which convey the water from the larger, often government operated, canal to the individual fields. These channels are often constructed, maintained, and/or operated cooperatively by groups of irrigators. They generally carry less than 150 liters per second (lps) of flow.

The intent of this manual is to assist country or regional officials to:

1. determine the quantity and causes of conveyance channel losses;

2. evaluate the benefits of reducing irrigation channel water losses;

3. develop techniques to reduce channel losses,

4. combine these techniques into conveyance channel improvement strategies based both on maximizing economic return and utilizing available inputs, and

5. develop the organizational and administrative structures which will be required to apply the improvement strategies.

The nature of an irrigation conveyance system demands that any improvement procedures, techniques, or structures must be adapted to a specific socioeconomic, agronomic, cultural, historical, and physical situation. But this specificity does not imply that techniques developed or procedures tested under different situations cannot be adapted to the desired conditions.

The procedures outlined in this handbook were primarily developed in Pakistan by the Water Management Research Project, carried out by personnel from Colorado State University and funded by the U.S. Agency for International Development. The project evolved into the pilot On-Farm Water Management (OFWM) Development Project in which 1500 Pakistan watercourses are being improved, and will soon be expanded to cover the entire country.

The proposed procedures and techniques were developed in the large, relatively flat, semiarid Indus Basin where a major portion of the water requirements for year-round cropping are met by supplemental irrigation. The 89,000 watercourse systems in the Basin each serve from 80 to 350 hectares of land, divided into less than one-fourth hectare level basins and cultivated by usually 15 to 100 cultivators. The similarities and differences between the environment in which these ideas were developed and the environment in which they are to be applied, must be kept in mind.

This manual is directed towards two audiences. The first are the planners, administrators, and donor agencies who set priorities and plan projects. Section 2 is intended to guide them as to the critical factors to consider and to assist them to develop a viable irrigation channel improvement program.

The second target audience is the engineers, economists, sociologists, and technicians who would carry out such a development program. The remainder of the handbook (Sections 3 through 10) is designed to assist them in evaluating the needs, devising a solution, and carrying out an improvement program. It is intended that sections of this handbook can be adapted to be used as a field and/or training manual by engineers and technicians.

Figure 1.1. An improved earthen channel section beside the original channel it replaced.

SECTION 2

GENERAL PROCEDURE FOR DEVELOPING AN IRRIGATION CHANNEL IMPROVEMENT PROGRAM

The procedure to develop and institute an irrigation channel improvement program is not unlike that used to solve any problem. After determining that a problem does exist and isolating the causes, solutions must be proposed and evaluated both under controlled and field conditions, and a program institutionalized to carry out the desired work. The initial stages will involve primarily research; the later stages, primarily institution building.

The first step in this procedure is to determine if irrigation conveyance channel water losses are a problem, and involves the actual physical measurements of loss. Techniques to evaluate the efficiency of existing irrigation channels are described in Section 3. It is likely that losses will be highly variable, so a large number of measurements will be required to confidently establish the average level. Losses may vary with water supply source, soil type, topography, geographic area, and channel layout and operation, so samples should be selected from each representative area. Private consultants, public agencies, or educational institutions could carry out the required measurements. Agencies which might have vested interests in, or preconceived opinions of the results should be avoided.

Once the amount (total volume or percent) or channel water losses is determined, the cost of that loss should be established. Irrigation water losses can be evaluated based on:

1. the cost of alternative supplies;

2. the opportunity cost (value for alternative uses), or

3. the marginal value to crop production.

Wasted irrigation water often causes drainage, salinity, or waterlogging problems, and the cost of these problems should also be considered. Conversely, deep percolated irrigation water can sometimes be repumped from the groundwater, or eventually drains back into surface supplies where it can be reused. The cost of irrigation losses under such conditions is often just the cost of repumping or diverting the water back into the irrigation system. Section 3 further explains factors to consider in placing a value on the irrigation channel water losses. Once the value is established, improvement techniques which reduce losses can be evaluated in terms of their economic desirability.

Before appropriate improvement techniques can be intelligently proposed, the causes of the losses must be established. If the initial water loss measurements were carefully documented, they should at least give an initial indication of the reasons for any excessive water losses. Additional studies should be carried out, as needed, to further isolate the specific problems. Examples of problems include:

1. highly permeable soils,
2. insect or rodent holes in the banks,
3. weedy and grassy channels,
4. small field sizes requiring extensive channel networks,
5. poor maintenance,
6. poor operating procedures, or
7. an excess supply of water.

Suggested techniques to isolate and evaluate these and other problems are given in Section 4.

Once the specific problems are determined, techniques to reduce losses should be proposed and tested. Generally the specific physical techniques should be tested under relatively controlled conditions before they are used in full scale field testing. Test sections should be installed at the earliest opportunity because many of the benefits of channel improvements will vary over time, requiring long-term monitoring. The improvement techniques may include both physical changes in the channels, such as channel lining or the installation of outlets; and institutional changes in the way the conveyance systems are managed and operated. Sections 5 and 8 describe some techniques for the reduction of irrigation channel losses.

An effort should be made to establish a realistic cost and associated water savings of each technique. Individual improvement techniques will often be overlapping or conflicting, and must consequently be evaluated in the form of packages. Packages should be evaluated in terms of farmer acceptance, economic feasibility, capital requirements, personnel availability, rapidity of implementation, institutional requirements, and simplicity. Section 8 outlines procedures to develop an overall channel improvement strategy and methods to evaluate the strategy.

Once desirable channel improvement packages are proposed, they must be field tested on a limited scale under the conditions in which they are expected to apply. It might be

desirable to test several packages at this stage, especially if conditions are variable. During this phase, practical constraints to implementation should be determined and the package adjusted to alleviate such constraints. The costs of the improvements, measured water savings and productivity changes should also be monitored closely.

During this initial research stage, existing village social structures and organizational institutions should be studied so that organizational factors which facilitate or constrain efficient water use can be determined and understood. Such information can often be used to help the farmers develop or modify the organizations and formulate the guidelines and policies which will facilitate both the improvement process and maintenance of the improvements and efficient utilization of the water. These organizations, guidelines, and policies can then be tested during the initial field testing of the improvement techniques and evolved to where they become an integral part of the improvement process. Both the technical and the organizational aspects of the program must be sound and adapted to local conditions for the program to succeed.

The final testing phase should involve a pilot project. The pilot project should be large enough to indicate potential problems in organization and institutional support, and to determine acceptance of the program by the water users. Section 9 discusses some of the organizational and institutional aspects of a channel improvement program. The capabilities of both the public and private sectors, and of the irrigators to provide the required inputs should also be established. These inputs might include:

1. capital,
2. labor,
3. raw materials,
4. fabricated structures,
5. machinery,
6. engineers and technicians,
7. training for engineers and technicians,
8. ongoing maintenance,
9. legal support and enforcement, and
10. project administration.

Monitoring and evaluation must be an integral part of the pilot project. The program should remain flexible at this stage so that it can be constantly adjusted in response to the needs indicated in the evaluation. As problems are encountered, specific techniques will probably need revision. These changes may need to be evaluated on a limited level before integration into the pilot project. Consequently several research phases may be working simultaneously.

By the end of the pilot project successful improvement strategies and the needed inputs and means to supply the inputs, as well as the total costs and resulting water savings should be fully established. Training institutions should be established and training and field manuals written. Implementation agencies should be in place and experienced. Private sector support entrepreneurs should be established. And the organizational requirements among the water users should be determined and tested, and given the required political, legal, and financial support.

The final phase of an irrigation channel improvement program will involve the application of the program on a large scale. If the pilot stage was successful, the large scale project should involve primarily instituting the same procedures on an expanded scale, and the only adjustments required should pertain to scale.

With full government support, the time required to go from initial evaluation to large scale project will vary from 1 to 5 years, depending on the complexity of the technology applied, the institutional changes required, and the availability of trained personnel.

This general process covering problem identification, development of solutions, and implementation is discussed in detail in "The Research-Development Process" by the Water Management Research Project staff (1980) and individuals and organizations contemplating improvement programs will find much information therein which will facilitate the improvement.

SECTION 3

EVALUATING THE PROBLEM

The fact that you are reading this manual indicates that you believe that problems may exist with the conveyance channels of an irrigation system. The purpose of this section is to outline procedures which will allow you to define and evaluate the problem and to determine if a channel improvement effort is desirable. Whether irrigation channel improvements are needed depend primarily on two factors:

1. how much irrigation water is being lost from the conveyance channels, and

2. the value of saving part of the lost water.

The first factor can be evaluated with conveyance loss measurements. Several techniques are available to the economist to put a value on irrigation water.

3.1 Measuring Irrigation Channel Conveyance Losses

Measurement of conveyance losses is the only way to determine how much water is being lost from that portion of the irrigation system. Two methods are used to measure losses from small channels: inflow-outflow, and ponding.

3.1.1 Inflow-outflow loss measurements

The inflow-outflow method involves measuring the amount of water which flows into a channel section and the amount which flows out at the tail of the section when no water is being usefully diverted between the two measuring points. The loss is the difference between these two measured amounts. The measurements can be either of total volumes of water, or if the channel is flowing steadily with little change in the measured flow rates at either end, directly of flow rates.

To measure steady state (constant flow) conveyance losses in a channel section, the flow measurement devices should be installed at the beginning and end of the channel section. The same type and size of device should be used if possible, so that any biased errors in the devices are cancelled out. Flows should be monitored in both devices until they are steady. Because the flow measurement device will generally change the depth of the flow and thus channel storage upstream from the device, from 5 minutes to an hour may be required depending on the slope of the channel to reach constant measurements in a channel flowing under steady state conditions. If flow rates into the channel are fluctuating, they will affect the measurements at the head of the section earlier than the downstream measurement. If measurements during fluctuating

flow must be made, the time for flow changes to pass between measurement points must be estimated and the measurements timed accordingly. Appendix 1 gives a sample data collection sheet for inflow-outflow loss measurements.

The losses can be represented either in the form of a rate of decrease in flow rate per unit length of channel:

$$Q_L = \frac{Q_1 - Q_2}{L} \qquad (1)$$

where: Q_L = loss rate (liters per second per 100 meters length (lps/100m)),

Q_1 = flow rate in the upstream device (lps),

Q_2 = flow rate in the downstream device (lps), and

L = the length of channel between the measurements (100m)

or in terms of percent loss in flow per unit length of channel:

$$Q_{LP} = \frac{Q_1 - Q_2}{Q_1 \times L} \times 100, \qquad (2)$$

where Q_{LP} = loss rate (%/100m).

These "normalized" steady state loss measurements can be compared between different channels to determine average loss rates for a region or type of channel. Channel losses are not calculated on a wetted area basis (wetted perimeter x length) as are losses for large canals, because infiltration rates in small channels commonly vary by a factor larger than 10 along the wetted perimeter at a particular cross section and losses are thus not proportional to wetted perimeter length.

Loss rates per unit length can be converted back to total flow losses by multiplying by the total length of the channel. For example, if the loss rate for a certain type of channel was determined to be 1.0 lps/100m, the flow loss in a 3000 meter channel would be expected to be 30 lps. If the channel were used 5 days per week and 40 weeks per year, the total losses from the channel would be:

$$30 \text{ lps} \times 86{,}400 \text{ sec/day} \times 5 \text{ days/wk} \times 40 \text{ wk/yr} \div 1000 \ \ell/m^3 = 518{,}400 \ m^3/yr.$$

Likewise, if the loss rate were 1.0%/100m, 30 percent of the total flow would be lost in a 3000m channel.

These loss rate designations assume a steady decrease in flow along a channel. Losses in some channels appear to be proportional to the usual amount of water in the channel. This implies that, as the flow along a channel decreases as a result of losses upstream, the loss rate will also decrease. If the loss rate is proportional to the flow rate:

$$Q_L = Q_{LI}Q \tag{3}$$

where: Q_{LI} = the initial instantaneous loss rate (%/100m), and

Q = the flow rate at distance along a water channel (lps).

This relationship leads to the equation:

$$Q = Q_o e^{-Q_{LI}L} \tag{4}$$

where: Q_o = the inflow rate at the head of the channel, and

e = the base of the natural logarithm.

Figure 3.1 shows the measured flow rate along two long channels. The first channel has a constant loss rate while the loss rate in the second is proportional to the usual flow rate. In Pakistan, the measurements indicated the true situation lay between these two (constant and proportional loss rate) extremes. Plotting flow rates measured at several points along a channel (as in Fig. 3.1) will indicate whether the loss rates tend to be constant (a straight line relationship) or proportional to the flow rates (curvilinear relationship).

In most small irrigation conveyance systems, most of the water losses are a result of steady state infiltration and leakage, and thus can be measured by steady state flow measurements. However, additional "transient" type losses, those which are constantly changing, also occur. Transient losses include:

1. the water lost while wetting and filling empty channels;

2. "dead storage" water left lying in the bottom of channels after the drainage into the field is complete; and

3. water loss through short term breaks in the banks or leaks in the outlets.

The amount of water lost to these types of losses will vary greatly with the operation, layout, and maintenance of the conveyance system. If water is frequently changed from one channel to another, transient losses will be large. In Pakistan watercourses, transient losses amounted to about

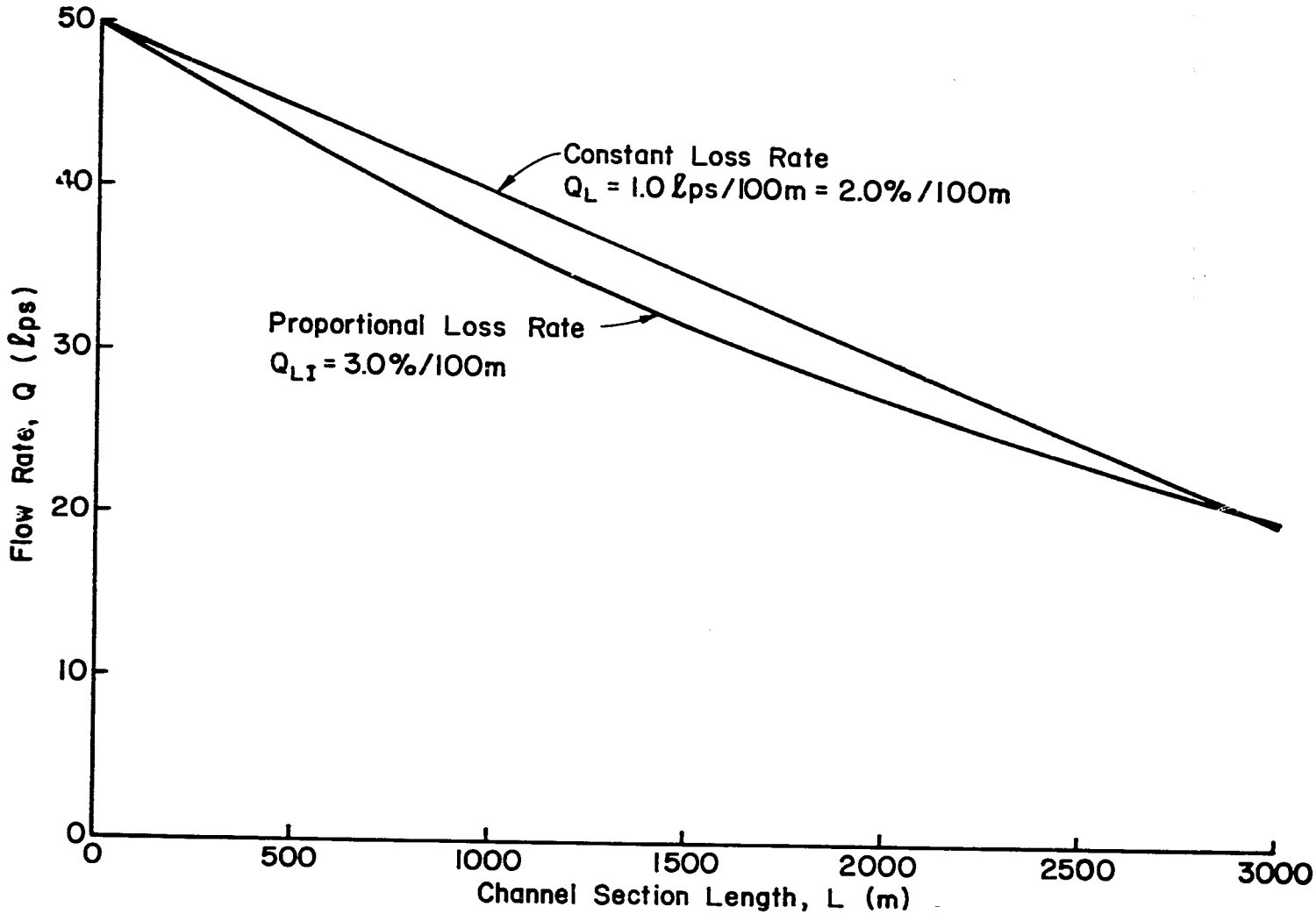


Figure 3.1. A comparison in flow rate decrease along a channel with a constant loss rate, Q_L (Eq. 1), and a loss rate proportional to the flow rate, $Q_{LI} \times Q$ (Eq. 3).

0.2 m^3 per meter length of channel filled and drained, or about 7 percent of the total inflow to the watercourses.

Total operational conveyance losses, including transient losses, can be measured with flow measurement devices if measurements are made over time of all the flow entering and leaving the conveyance system. The flow rates entering the system at the head and leaving it at the tail can then be plotted on a "flow hydrograph" as shown in Figure 3.2. These flow rate curves can be integrated graphically (the area under the curve), or numerically (Simpson's Rule or Trapezoidal Rule integration) to determine the total volume of water which flowed through each flume. The difference between steady state flow rate losses and these volumetric operational loss measurements will be the amount of transient losses.

Although valuable information is gained, operational loss measurements are fairly difficult and time consuming to carry out and should be used only to indicate the degree of transient losses for a type (operational and physical layout) of conveyance system. Further explanation of the procedures and analysis for operational loss measurements are given in Trout and Bowers (1979).

Two factors can affect the accuracy of inflow-outflow conveyance loss measurements. The first involves the accuracy of the measuring devices and the second, the affect of the measuring devices on the flow conditions in the channel.

Most open channel flow measuring devices are accurate to only within $\pm$ 3 to $\pm$10 percent under field conditions. Consequently long channel sections must be measured so that the losses (or the difference between the measured flow rates) are significantly larger than the possible inaccuracies of the measurements. For example, if measurement flumes are used which are accurate to $\pm$ 5 percent in a channel which loses 1 percent of its flow per 100 meters, a channel at least 1000m long must be measured before the losses are larger than the maximum possible error. Generally, accuracy of the measurement increases as the length of channel increases.

This error factor is the biggest disadvantage to inflow-outflow loss measurements. It requires that extreme care be taken to properly use accurate measurement devices, and that several measurements be taken so that random errors can be accounted for.

Another problem with inflow-outflow loss measurements in small channels is that the head loss (drop in the water as it passes through the device) of the measurement device causes the water level to increase up-channel from the device. In many earthen channels, this causes the loss rate to increase above the rate occurring with no measuring device in the channel,

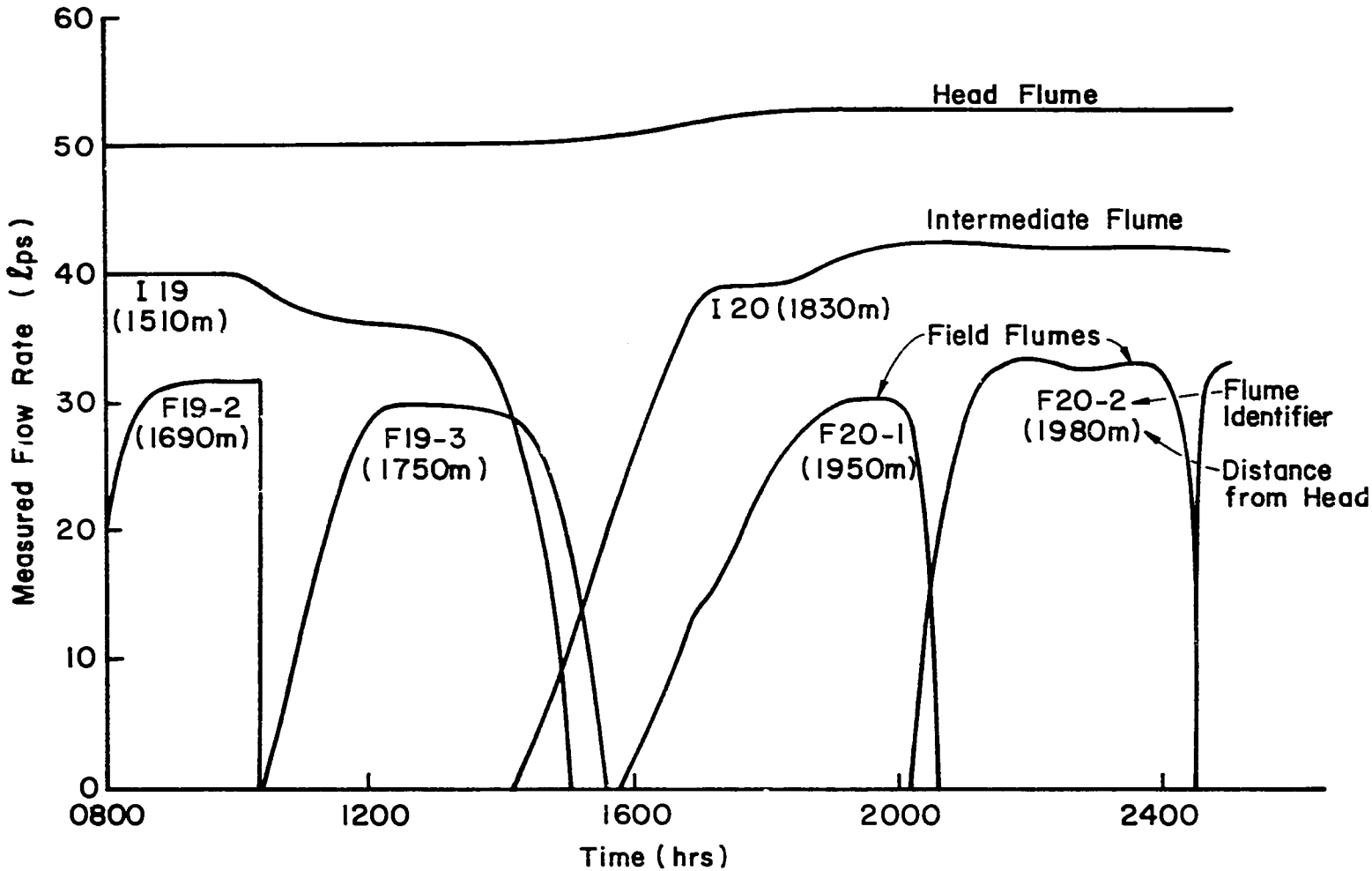


Figure 3.2. Flow hydrograph showing discharge through the head, intermediate, and field flow measurement devices over time during an operational loss study.

because loss rates usually increase as the depth of flow in the channel increases. Average loss rates measured in channels in Pakistan and Colorado, doubled with each 5 cm increase in the flow depth (Akram et al., 1980; and Trout, 1979a). The relationship between loss rate and flow depth can be easily determined with ponding loss measurements, as will be explained in the next subsection.

When the flow level in a channel is raised by the installation of a measurement device, the depth increase extends upstream of the installation since the flow depth gradually decreases to the normal depth. The distance of this depth increase is primarily dependent on the channel cross-sectional shape and normal depth, the flow rate, the channel slope, and the amount the depth is increased at the device, and can be calculated using iterative backwater curve techniques (see Chow, (1959), Chapter 10).

An analysis was made of the effect of flow depth increases on loss rates for small channels where the cross-sectional shape is defined by a power curve (which can be used to describe most aged small channels) of the form:

$$TW = a_1 D^{a_2}, \tag{5}$$

where: TW = channel water surface top width (m),

D = channel water depth (m), and

a_1 and a_2 = constants,

and the loss rate is exponentially related to the flow depth:

$$Q_L = Q_{LO} e^{b\Delta D} \tag{6}$$

where: Q_{LO} = the loss rate at the normal flow depth, D_o (lps/100m),

ΔD = depth fluctuation from the normal depth $(D-D_o)$(m) and

b = an empirical coefficient equivalent to the fractional change in loss rate per unit change in depth (m^{-1}).

The average b value in Pakistan watercourses is $15m^{-1}$.

The results of the analysis indicated that the relative increase in loss rate as a result of a flow depth increase is a complex function of the channel slope S (m/m), the empirical exponential coefficient, b(m^{-1}), the increased depth

of flow at the measurement device, ΔD_i (m), the channel normal flow depth, D_n (m), and the length of the measured section, L(m).

$$\frac{Q_L}{Q_{Lo}} = (0.445S^{-0.195}b^{1.56}\Delta D_i^{1.62}D_n^{0.159})\exp(-0.149S^{0.306}D_n^{-0.260}L)$$

$$+ (0.000414S^{-0.918}b^{1.39}\Delta D_i^{1.57}D_n^{0.750}) + 1 \qquad (7)$$

The cross-sectional shape, channel width, and the channel roughness coefficient did not have a major affect on the relative loss rates. This "best fit" equation tends to overestimate the loss rate increase in long sections. Figures 3.3, 3.4 and 3.5 give an indication of the importance of these induced head losses on the measured loss rates. A measurement device which creates a head loss of 8 cm in a 1000 m long channel which normally flows 0.45 m deep on a slope of 0.0006 will increase the losses between 25 and 70 percent for b values between 10 and 20. This emphasizes the importance of selecting and installing measurement devices with minimum head loss and, again, the importance of making inflow-outflow measurements in long channel sections, especially in channels with small slopes.

This biasing head loss factor can be accounted for by dividing the measured loss rate by the factor given in Eq. 7 to convert back to the probable loss rate in an undisturbed channel.

Several devices are available for flow measurement in small open channels, including circular or rectangular submerged orifices; V-notch, Cipoletti and rectangular (contracted or suppressed, sharp or broad crested) weirs; Parshall, trapezoidal, H, and Cutthroat flumes; and propeller meters. Each device has certain inherent advantages and disadvantages under various conditions. A complete description of each device and instruction for proper use will not be given here, but can be found in various flow measurement manuals (USBR, 1967; Bos, 1976; Skogerboe, et al., 1973, SCS, 1962). However, listing some important characteristics of each device will be helpful in the selection of the most appropriate type for the given conditions.

Submerged orifices are accurate measurement devices if properly installed and are relatively easy to construct and install, but each size is usable over a relatively narrow flow range without creating excessive head loss (the drop in the water surface across the orifice). Their accuracy can be affected by upstream and downstream flow conditions if they are not properly installed and clogging of the orifice with debris can be a problem, especially in permanent installations.

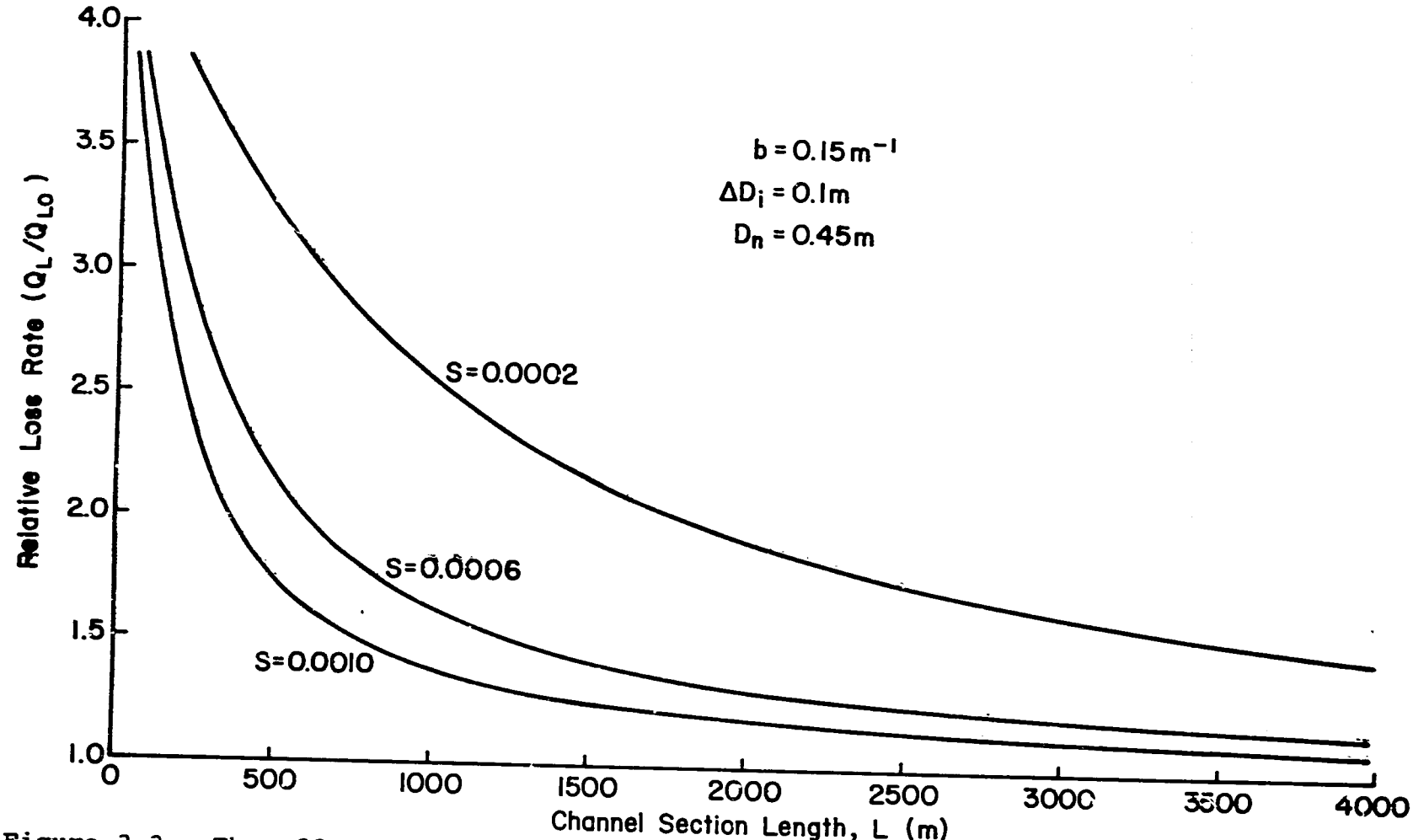


Figure 3.3. The effect of an induced flow depth increase, ΔD_i, on water loss rates, Q_L, relative to the normal loss rates, Q_{LO}, for various channel section lengths, L, and slopes, S.

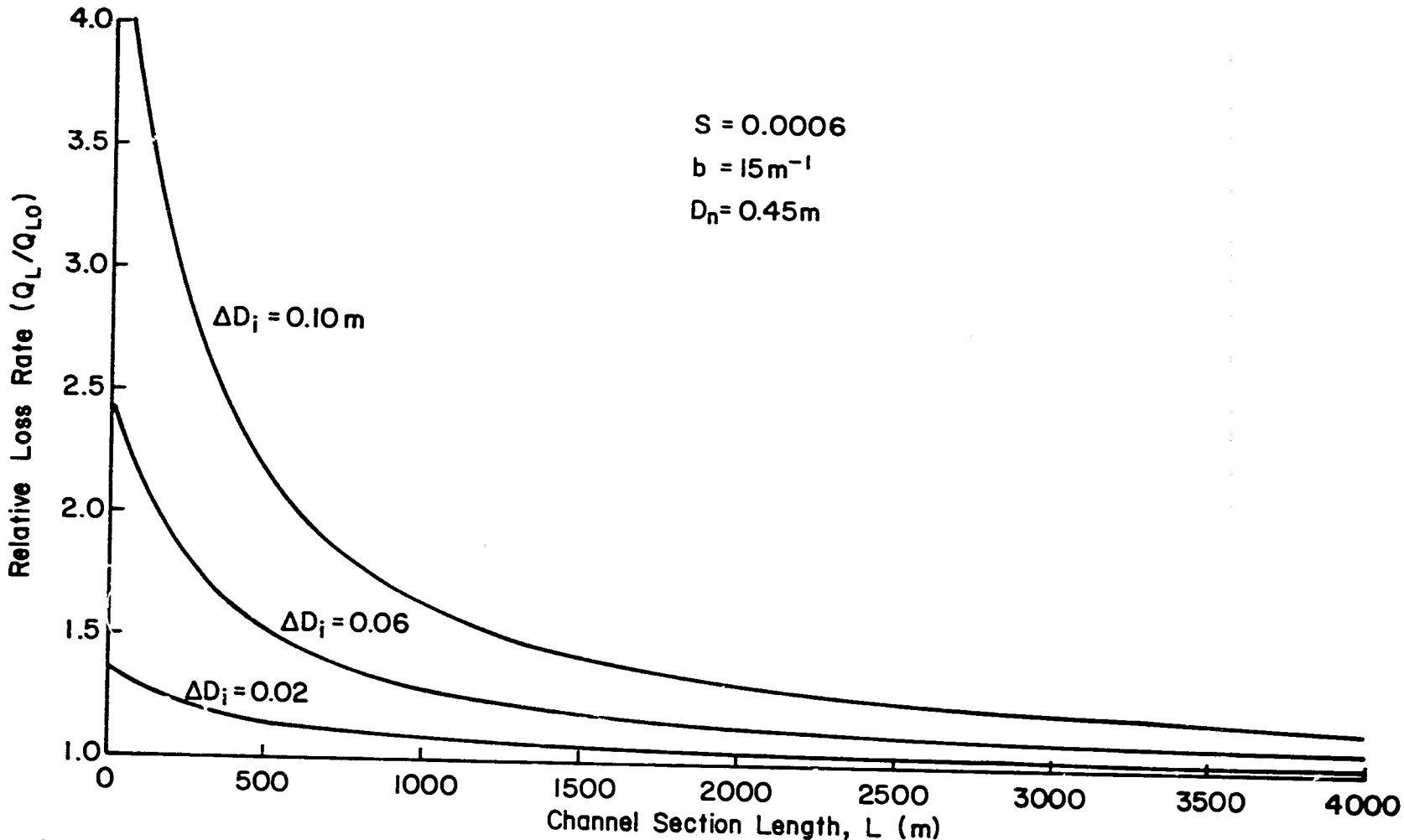


Figure 3.4. The effect of an induced flow depth increase, ΔD_i, on water loss rates, Q_L, relative to the normal loss rates, Q_{LO}, for various channel section lengths, L.

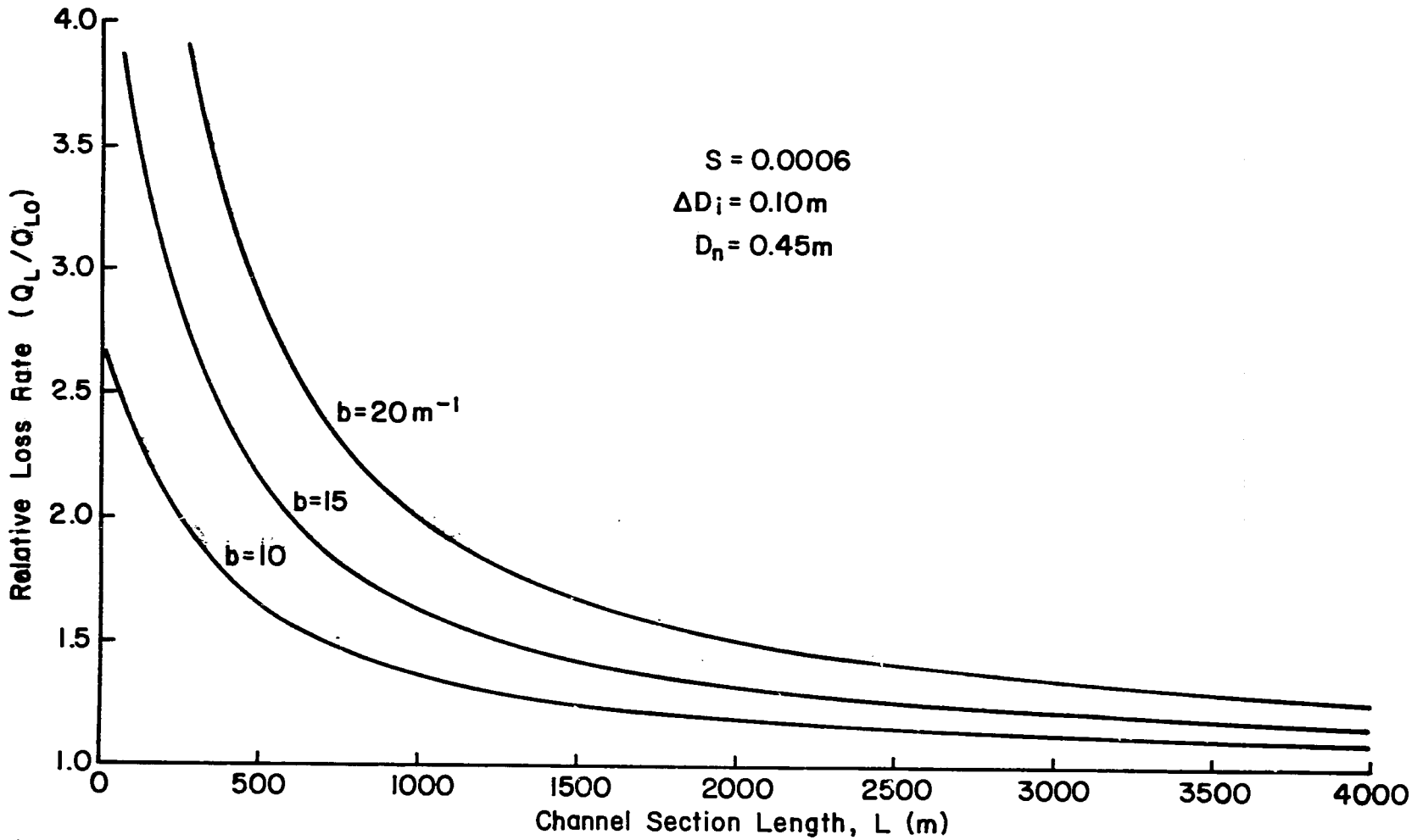


Figure 3.5. The effect of an induced flow depth increase, ΔD_i, on water loss rates, Q_L, relative to the normal loss rates, Q_{LO}, for various section lengths, L, and exponential coefficient, (b), values.

Weirs are more flexible to flow fluctuations than orifices, but are accurate only under free flow conditions and thus require a relatively large head loss. Weirs are fairly easy to fabricate but are also sensitive to upstream channel conditions and thus can be difficult to install correctly in small channels. Weirs are commonly used for permanent installations when head loss is not a problem.

Most flumes are constructed so as to eliminate most of the effects of upstream or downstream channel conditions, and thus can be installed under a wider range of conditions. They are often desirable for temporary installations where less attention can be given to channel conditions. Flumes can measure a fairly wide range of flows, and if calibrated for submerged flows, can get fairly accurate readings at low head-loss values. Construction difficulty varies but is generally more difficult than orifices or weirs. Although installation of a well designed portable flume is relatively easy, it must be carefully leveled to obtain accurate readings. Because of their size, flumes are more difficult to transport and usually more costly to fabricate. Cutthroat flumes (Skogerboe, et al., 1973), because they are relatively easy to fabricate and have been calibrated under submerged conditions, are recommended for temporary installations, especially if low head loss is desirable. Figure 3.6 shows a Cutthroat flume being used to measure flows in a small channel.

Figure 3.6. Measuring water flow in a small channel with a Cutthroat flume.

Propeller meters measure flow velocity, which can be converted to flow rate if the velocity distribution in the channel is known or can be determined. Because of the difficulty in determining the distribution in small, irregularly shaped channels, propeller meters are less accurate than the other listed devices, and are often too inaccurate for loss measurements in small channels. The main advantage of these velocity meters is that they create negligible head loss and thus do not disturb the normal flow. Good quality small propeller meters (pygmy meters) are expensive but velocity measurements can be made quickly and easily.

An adaptation of the propeller meter which increases its accuracy is to permanently fix the propeller in a cross-sectional shape through which all the water is channeled. The device can then be calibrated to read flow rates or volumes rather than velocity. A disadvantage of this device is that channeling of water into the device creates appreciable head loss. A popular commercially available meter of this type is the Sparling meter which is commonly used to measure outflows from tubewells.

3.1.2 Ponding loss measurements

The ponding method of measuring channel losses involves filling a channel section which is closed at both ends and measuring the rate at which the ponded water seeps and leaks out of the section. The method is simple and yields precise results, but can only measure the losses in short sections. Since loss rates are highly variable, several measurements must be made if an accurate estimate of the average loss rate in a channel is desired. Conversely, the ponding method can effectively determine which sections of a conveyance system are losing the most water and thus isolate problems. Drawbacks of the ponding method are that the channel cannot be used while the measurement is being made, and a source of water must be available to fill the section.

Ponding loss measurements do not include transient losses, and should therefore be comparable to steady state inflow-outflow measurements. Ponding measurements will sometimes give a conservative estimate of true steady state losses because more sediment will settle from the still water and tend to seal the channel wetted perimeter. This is especially true if additional sediment is added to the ponded water during the construction of the end dams.

Two factors are critical in making ponding loss measurements. The first is that the channel should be wet for sufficient time before the measurements are made to insure that the seepage rate is not higher than the normal rate due to rapid infiltration into dry banks. This is most easily accomplished by making the measurements immediately following a period when the channel was in use. Alternately, the ponded water can be maintained in the section to the desired depth for a period of time (one hour should usually be sufficient) before readings are taken.

The second critical factor is to make the measurement when the water surface is at the same level as when it is flowing normally. This level is most easily determined by installing a staff gauge in the channel section while the channel is being used normally and recording the operational water surface level. It should be noted that this operational level (OL) will fluctuate somewhat with the inflow rate, the elevation and proximity of the field being irrigated, and the amount of vegetation in the channel (channel roughness), so the level should be determined when these conditions are roughly average.

Since the goal is to determine true steady state loss rates, sections should be selected which are representative of the measured channel. Selecting unusually clean or straight sections, or sections with no leaks or outlets will not give representative results. Leakage should not be stopped if a true loss measurement is desired.

The length of section measured will depend on the slope of the channel. The variation in the ponded water level from the operational level should not be more than 2 cm. If channel slopes are small (less than .0005), sections as long as 100 meters can be measured, but if slopes are steep (greater than .001), section lengths should be less than 30 m. Staff gauges installed near the ends of proposed sections in running water can be used to determine the deviation of the ponded water level from the operational level in the section. Section lengths should be as long as possible within these limits so that a larger and more representative sample is measured, and so the end effects of seepage into or around the dams is minimized.

The easiest time to initiate ponding loss measurements is immediately after a channel section has been used. The downstream dam can be installed first while letting upstream drainage water fill the section to a few centimeters above the operating level. When the section is filled as desired, the upstream dam can be installed. Several adjacent sections can be measured concurrently by repeating this procedure. The dams can consist of closed permanent check structures, canvas dams, sheet metal firmly inserted into the channel bed and banks, or soil bunds constructed across the channel. The dams, of course, must be leak proof.

Once both dams are installed and are checked for leakage and the channel is thoroughly wet, readings on a staff gauge firmly inserted into the bed or bank of the channel near the center of the section should be made over time. Readings must be begun while the water level is above the previously determined operating level, so if the channels were initially dry, additional water might need to be added before measurement begins. Staff gauge readings should be made every 2 to 5

minutes initially while the water level is dropping quickly, but can be made every 15 to 30 minutes after the level drop slows. Readings should be continued until the water level is far enough below the operational level to determine the recession rate at the operational level. Figure 3.7 shows the layout for ponding loss measurements and Figure 3.8 shows a ponding loss measurement in progress. Appendix 2 shows a sample data collection sheet for ponding loss measurements.

When the water level is at the operational level, the average water surface top width must be determined. This can be done by measuring the top width with a tape or measuring rod several places along the section and taking the average of the measurements.

The loss rate is the rate at which water seeps and leaks out of the section. This is equivalent to the change in volume of the ponded water over an elapsed time, or the change in depth in a given time interval times the water surface area. The change of depth over time is best determined by drawing a curve of gauge reading vs. time and measuring the slope of the curve at the desired depth, usually the operating level. Figure 3.9 illustrates this calculation. When this value is multiplied by the average top width at the same depth, a loss rate per unit length results:

$$Q_L(\text{lps/100m}) = \frac{dd}{dt}(\text{cm/hr}) \times TW(\text{cm}) \times 1/3600 \text{ min/sec} \times 100 \text{ m}$$

$$\times\ 100 \text{ cm/m} \times 1/1000 \text{ liters/cm}^3, \text{ or}$$

$$Q_L = \frac{dd}{dt} \times TW \times .0028; \qquad (8)$$

where: $\frac{dd}{dt}$ = the change in water level depth per unit time (cm/hr), and

TW = average top width (cm).

The best estimate of the loss rate in a channel is the average of the loss rates measured in each ponded section of the channel.

As was mentioned earlier, the loss rate varies with the surface level of water in the channel. The relationship between loss rate and depth fluctuations (water surface elevation changes) can be determined by the ponding loss method if the channel is initially filled several centimeters above the operating level and readings are continued until the surface drops several centimeters below the operating level. Top width measurements also must be determined at the initial depth and periodically during the measurement. These data are then analyzed as described above, except the loss rate is calculated at several depths relative to the normal

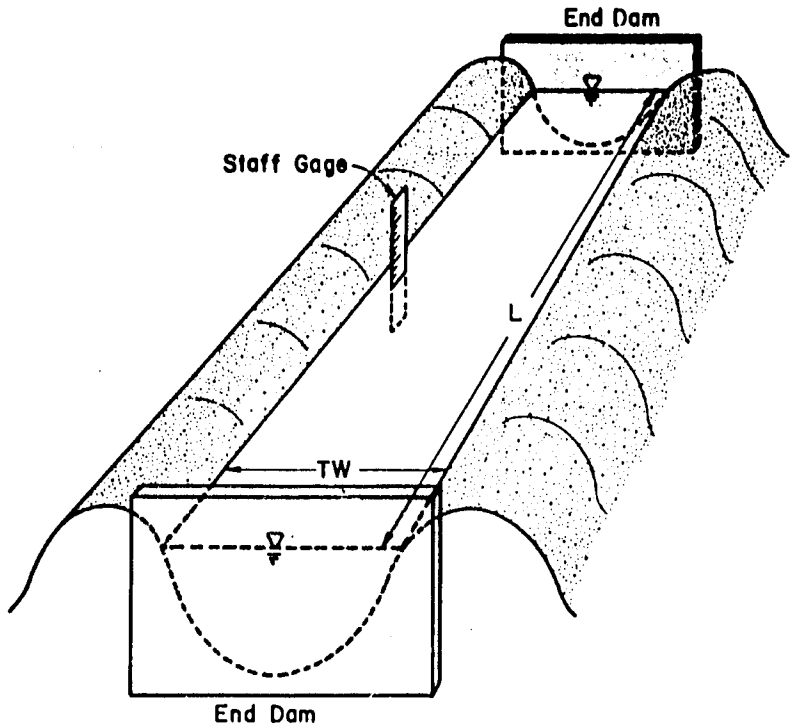


Figure 3.7. Setup for ponding loss measurements.

Figure 3.8. Ponding loss measurement in progress.

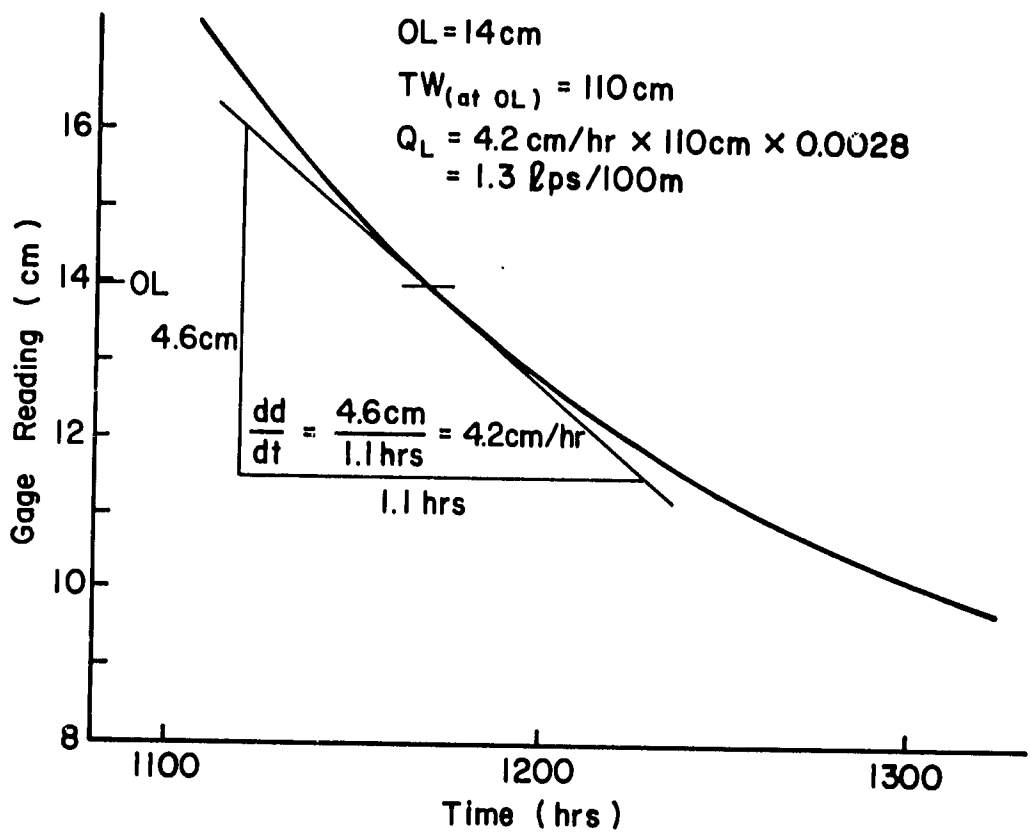


Figure 3.9. Ponding loss rate calculation - graphical determination of the water surface rate of recession.

operating levels as shown in Figure 3.10. These loss rates can then be graphed vs. depth as shown in Figure 3.11 and the relationship determined between the factors. The relationship in Pakistan and Colorado channels was strongly exponential (Trout, 1979a and Akram, et al., 1980), and regression analysis of the transformed variables was used to fit the results such as those shown in Figure 3.11 to an equation of the form of Eq. 6. The regression equation for the curve in Figure 3.11 is shown in the figure.

A variation of ponding method can also be used to determine the excess amount of water which is infiltrated into initially dry channel banks and beds. If a short section is dammed at both ends and filled to the operating level very quickly, the rate at which water must be added to the section over time to maintain the constant water level indicates the loss rate into dry sections. The difference between the volume of water added to the section initially while the banks are becoming wet and the rate of infiltration is decreasing, and the volume added during an equal period of time later while the loss rate is fairly steady, divided by the section length, is the excess volume lost to wet up the dry banks. This is a transient loss and is not measured in steady state measurements. The wetting up loss for a conveyance system is equal to this calculated loss per unit length, times the length of dry channels filled. The time required before the loss rate becomes steady also indicates the time a channel section must be filled before regular steady state measurements can be made.

3.2 Evaluating the Costs of Conveyance Losses

Determining the costs of water losses from irrigation conveyance systems or inversely, the value of reducing losses, will depend upon many local physical and economic conditions. It will depend first upon where the lost water eventually ends up and whether it can be beneficially reused, or whether it creates additional problems and costs; and second upon the value of that lost water for agricultural production.

3.2.1 Determination of reuse potential and additional costs

Irrigation water which is lost from conveyance systems can initially go to one of three places:

1. evaporate from the water surface,
2. remain in the surface soils near the channel, or
3. seep to the groundwater.

That part which evaporates from the water surface is removed from the local hydrologic system, and thus has no potential for use and creates no secondary problems. The amount evaporated will depend on the length and surface width of the channels and the atmospheric potential evaporation rate.

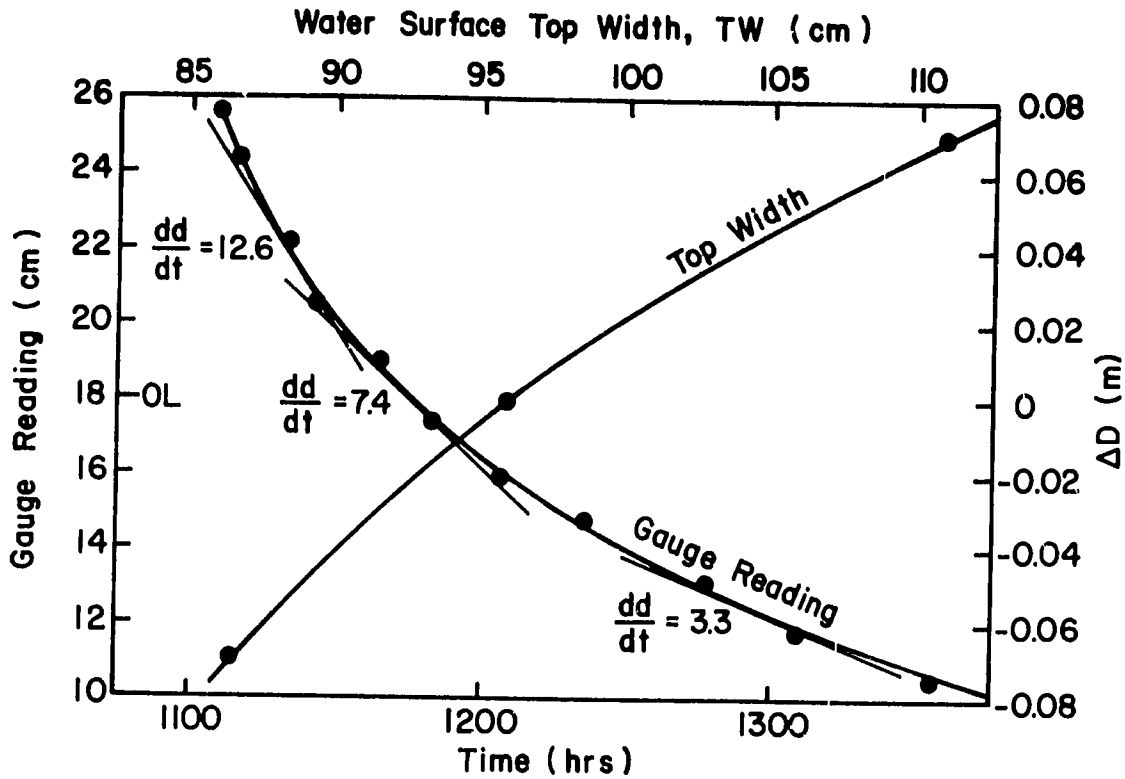


Figure 3.10. Loss rate calculation - graphical determination of recession rate and top width at various depths.

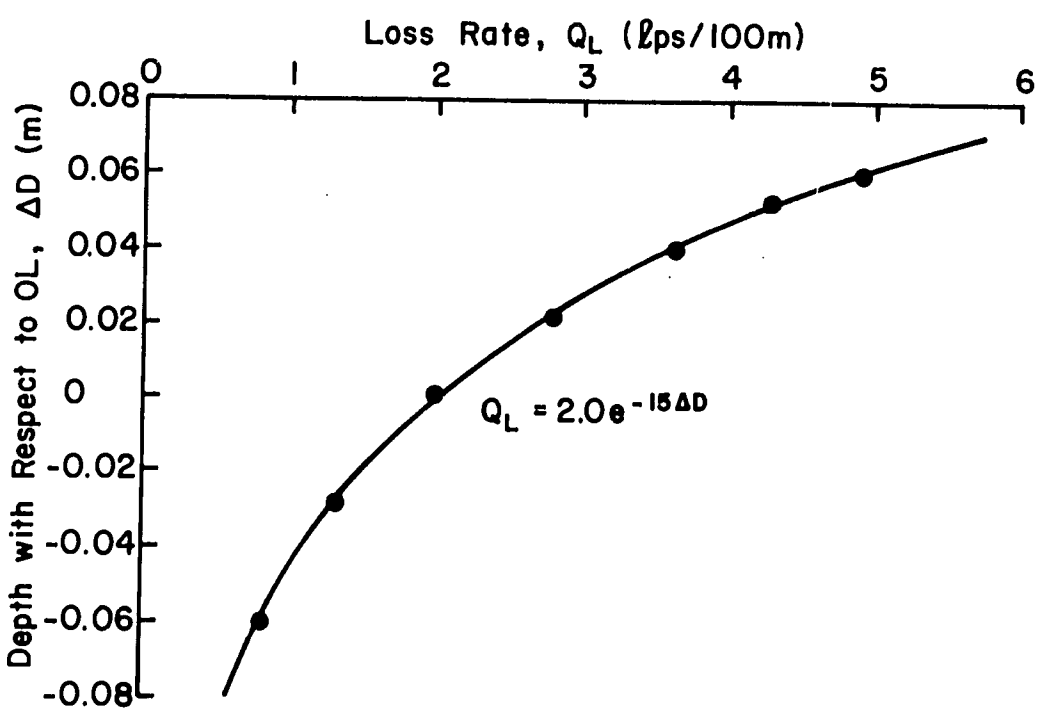


Figure 3.11. Loss rate as a function of depth.

In conveyance systems where losses are high, surface evaporation will be a very small percentage of the total loss. In Pakistan, surface evaporation amounts to about 1 percent of the total conveyance losses and about 0.3 percent of the inflows in the tertiary system. Methods to compute surface evaporation are given in the next section.

Initial infiltration and leakage into a channel wetted perimeter often wets up dry banks and resaturates the surrounding soil which has been dried by surface evaporation and plant evapotranspiration. Part of the seepage from a constantly flowing channel is used (evapotranspired) by the nearby plants. Whether or not this evapotranspired water can be considered usefully utilized depends upon the benefit derived from growing plants.

If the plants on the channel banks are cultivated and harvested, such as fruit trees or grazed grasses, part of the losses to soil moisture replenishment could be considered beneficially used. If the only growth is undesirable weeds, under-utilized trees, or seldom used grasses, the transpired seepage would be considered lost, and could even create an additional cost if labor is required to periodically clean the bank growth which inhibits the water flow or damages the channels.

Occasionally water will leak through or run over watercourse banks onto surrounding fields. This water is economically used only if it improves the crop yields in the wetted area. If the field is otherwise adequately irrigated, the leakage generally has negative value. It may leach nutrients out of the soil or waterlog and/or lead to salinization of the soil, and thus decrease the crop yields. The additional costs may be appreciable.

Even if all losses were eventually utilized by beneficial crops along the channels, a cost is still incurred because crops grown in overly moist soils utilize water inefficiently. Purposefully planting such crops along the banks can partially cause the seepage they are utilizing by drying the banks and increasing the initial infiltration rates.

The water which evaporates from the soil surface of the wetted perimeter of the channel after the channel is drained serves no useful purpose, but will, like water surface evaporation, be a small percentage of the total losses.

The value of seepage water which is stored in the bank soil can be considered equal to the economic value of the crops which utilize the water. For example, if the grasses growing on channel banks support one goat per 100 m, then the value of that food should be estimated and subtracted from the costs of the lost water. The additional costs due to leakage on adjoining fields are equal to the reduced value

of decreased crop yields. If leakage reduces crop yields by 50 percent on 1 hectare of land adjoining a long channel section, then the value of the lost production should be added to the value of the lost water. Since any earthen channel improvement will still allow the replenishment of water evapotranspired from the bank soils, this reduction in the costs of losses need not be considered when determining the value of reducing losses unless channel lining is being evaluated.

The remaining losses eventually percolate to the groundwater table. The resulting value or additional costs of this additional groundwater storage depend upon the local geology. If the groundwater is of good quality and can be repumped, the only cost of the amount which is repumped is the pumping costs. In fact, if the normal water supply is fairly constant and the crop water requirements vary, the groundwater supply can have a higher value than the canal supply because it is available when needed most and adds flexibility to an otherwise rigid supply system. Under such conditions, if managed properly, deep percolation resulting from channel losses can be a benefit rather than a cost.

In some irrigated basins, the groundwater eventually returns to rivers and streams. To the extent that these supplies are reused, and if they don't lead to degradation of the groundwater or rivers, no cost is associated with the losses. If the groundwater carries more salts to the river which harms downstream users, the value of such harm must be added to the value of the water lost from the channels.

If the lost conveyance water deep percolates to a groundwater which is not reusable because of high salinity levels or other water quality problems or because of low yield of the aquifer, the loss has no reuse value.

In some poorly drained irrigated areas, deep percolation from irrigation systems causes the water table to rise until it eventually comes near enough to the surface to degrade the surface soil and decrease crop production. Examples of such salinity and waterlogging problems are evident in many irrigated areas of the world. Under such circumstances, the deep percolated channel losses not only have no reuse value, but the portion of the land degradation and crop production decrease attributable to the conveyance losses or the cost of alleviating the problem must be charged to the losses in addition to the sacrificed value of the water.

In summary, when evaluating the costs of small conveyance system losses, reuse values which must be subtracted from the cost include:

1. the value of beneficial plants growing near the channels which derive their water from channel losses; and

2. the value of water pumped from the groundwater or diverted from return flow to the river minus the cost of repumping and any water quality decrease.

Costs which must be added to the sacrificed value of the lost water include:

1. the decreased value of crops near the channels due to leakage; and

2. the costs of land degradation and depressed crop yields caused by salinity and waterlogging (or the costs of drainage to alleviate such problems) attributable to channel deep percolation.

3.2.2 Determination of the marginal value of the lost water

The marginal value of irrigation water, which is equivalent to the benefit derivable from reducing the conveyance losses, can be determined by three methods:

1. the marginal value to crop production,

2. the opportunity cost (value for alternative uses), or

3. the market price.

Different methods will apply under different conditions. Only summaries of these three techniques will be given here. Young and Gray (1972) comprehensively treat the problem of establishing the economic value of water.

The marginal value of water for crop production is the value of additional output which can be produced with an additional unit of water. The value is difficult to determine because it varies from year to year and crop to crop, with other input levels and with the time of the year. Most crops exhibit a similarly shaped relationship between water inputs and crop yields, an example of which is shown in Figure 3.12. The curve demonstrates that additional increments of water generally result in progressively smaller increments of additional yield until the crop water requirements are fully met. Beyond this point, additional water can actually lead to a yield decrease. A complication not shown by such a relationship is the importance of the timing of the water. Such a relationship is valid only for a given (often the optimal) water application timing. A second complication is that, for a given crop, the yield with a given water input will depend upon other inputs such as nutrients, soil, and the weather during the growing season. Only relative yields for a given input level are shown. The relationship between water and

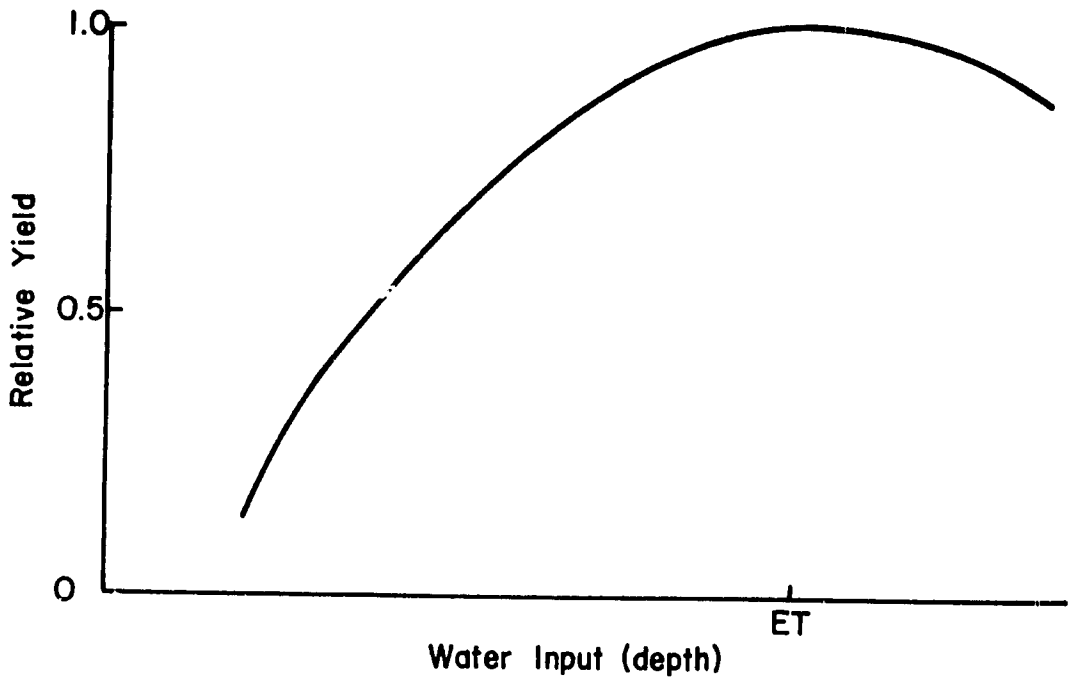


Figure 3.12. Hypothetical crop production function.

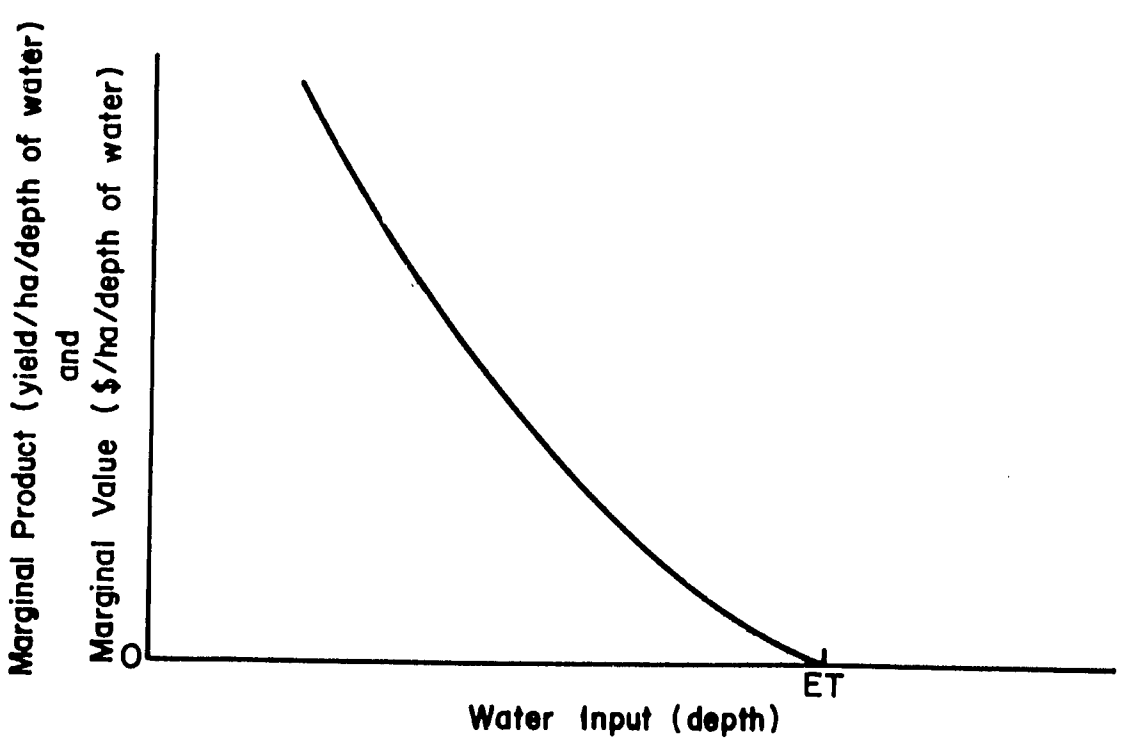


Figure 3.13. Hypothetical marginal products and values of a supplemental water supply for the production function shown above.

yields for a given crop, location, and input level can best be determined from controlled experiments, but can also be estimated by analyzing farmer production levels if water input levels can be determined. Heady and Hexem (1978) further discuss the methods of establishing crop production functions and present functions for several crops.

Once the water production function is determined, the present location on the curve must be established. Water input levels can be most accurately determined by a large sample of actual measurements at the field. This is very time consuming. The other alternative is to calculate the water applications from regional diversion and groundwater pumping data, and cropping pattern information, although the system efficiencies are often so inaccurate that such a method can provide only a rough estimate. The most practical method would be to collect limited field data to calibrate or verify the regional data.

Once potential production increases with additional water have been estimated, the value of that increase can be determined by multiplying by the revenue derived by the farmer per unit of production, minus any cost which might be associated with the additional water application and larger crop yields. A marginal product and value curve for the crop production function shown in Figure 3.12 is shown in Figure 3.13.

When the marginal values of water for individual crops have been established, the cropping mix must be determined so that the crop value can be converted to an overall marginal value for a region. The present cropping pattern can be used as an initial point, but it can't be assumed that the pattern will not change when the water supplies increase. For example, higher risk and more profitable crops might be grown if more water is available. A speculative cropping mix with increasing water supplies must be assumed.

If water supply is a limiting factor to production and additional land and labor is available, additional water supplies could lead simply to an increase in the cropping intensity with no changes in water application rates or yield levels. This would imply that the marginal value of additional water remains constant, and equal to the value of water as presently applied, until the land is fully utilized or another limiting factor is encountered. This will be true if the farmer perceives that he is optimally allocating his limited water supply, and makes establishing an initial value for supplemental water much simpler.

One analytical method of simultaneously taking account diminishing returns, timing, and possible cropping pattern changes is the construction of a linear programming model based on representative farm budgets. Solving the linear

program for various levels of water availability allows the generation of a marginal value relationship, such as that shown in Figure 3.13 for one crop and condition, for given present overall water use conditions. Anderson and Maass (1974) describe such a linear programming technique and Johnson (1978) demonstrates its application to Pakistani farmers.

A second method sometimes utilized for determining the value of water is the opportunity cost, or water's value to alternative uses. In agrarian or nonindustrial economies, there are generally few alternative uses for irrigation water. A possible example of an alternative use would be diverting the water to ponds for livestock bathing or fish rearing. Opportunity cost can seldom be used to establish the value of irrigation water.

If irrigation water is freely bought and sold, the market price might provide an accurate assessment of the farmer's perception of the value of the water to him. For example, if a farmer freely sells his well water to his neighbors, the price they are willing to pay should indicate the value they feel they can derive from that water. In many regions where water supply is controlled by the government, this technique has limited applicability, but even if the market for water is limited, any established current market price should at least give an indication whether the results of other methods of evaluating water are reasonable. The market price is usually a much more quickly and easily determinable value than those previously described.

The marginal value of irrigation water will determine how much should be invested in providing additional water to the farmer. However, just because channel improvements can provide water to the farmer at less cost than the marginal value does not necessarily mean that they should be carried out. Many alternatives for providing supplemental water, each with its own unit costs, may exist.

Alternative supplemental irrigation water supplies include:

1. developing new surface supplies, such as building dams and enlarging canals,

2. utilizing groundwater storage through pumping programs,

3. conserving and more effectively utilizing rainwater, and

4. decreasing the losses from portions of the irrigation system, such as:

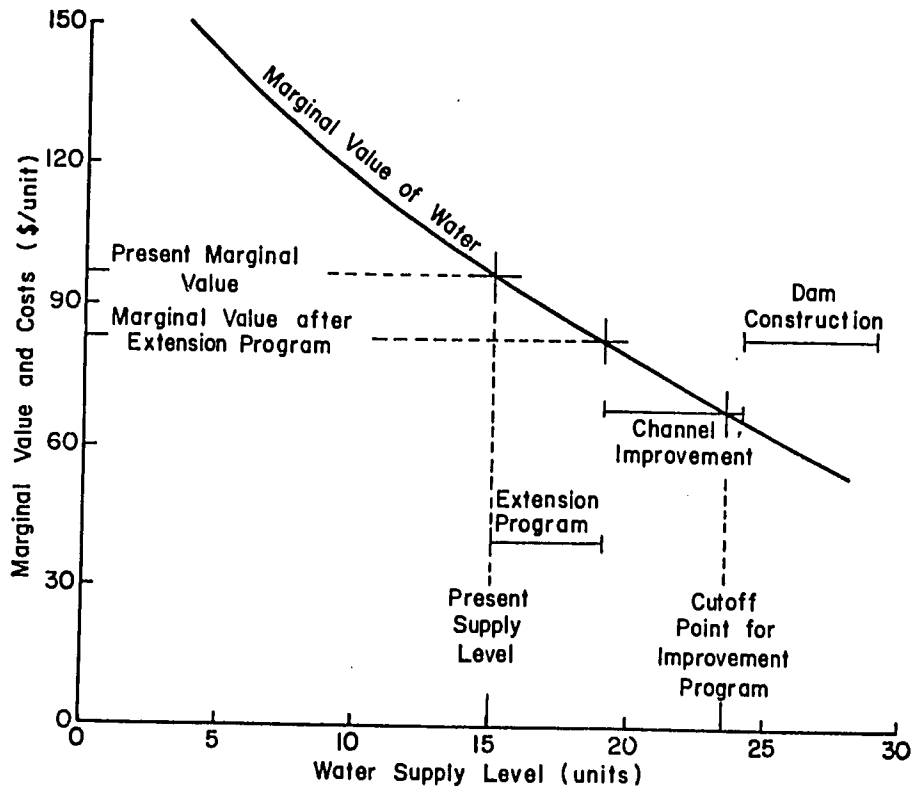
a. lining canals to reduce canal losses,

b. land leveling or extension programs to increase irrigation application efficiencies, or

c. channel improvement programs to decrease channel losses.

As long as additional water can be supplied by any of these alternatives at less than the marginal value, they are economically desirable. However, the least costly of these alternatives should be applied first to derive the highest net benefits. This implies, as demonstrated in Figure 3.14, that if an extension program designed to improve application efficiencies can provide additional water at $50/unit compared to channel improvements which cost $70/unit, the extension program should be applied first and then the benefits of additional water should be reevaluated. The marginal value of water will generally decrease as the supply increases. Channel improvements should be applied only after all less expensive sources of additional water have been exploited and only if the costs are lower than the marginal value at that supply level.

A factor that must be considered in evaluating the costs of additional supply is that the unit of water refers to water at the field, and adjustments for inefficiencies between the supply and field must be made. For example, if a new dam can supply water at the dam for $50/unit, and the canal conveyance efficiency is 70 percent, the small channel conveyance efficiency is 60 percent, and the field application efficiency is 60 percent, the true cost of that water at the field is $50/(0.70 x 0.60 x 0.60) = $198/unit. Channel improvements must also be evaluated considering the point in the system where the marginal value was evaluated. If the water is evaluated at the root zone, then application efficiencies must be taken into account.



Alternative Supplies:

1) Extension Program: Unit Cost: $50
Total Supply Supplement: 4 Units

2) Channel Improvement: Unit Cost: $70
Total Supply Supplement: 5 Units

3) Dam Construction: Unit Cost: $80
Total Supply Supplement: 5 Units

Figure 3.14. A hypothetical example showing the marginal value of water at increasing supply levels, and the cost and potential supplimental supply of three alternative programs arranged by increasing costs.

SECTION 4

DETERMINING THE CAUSES OF CHANNEL LOSSES

If irrigation channel loss measurements indicate that potential exists to increase water supplies through a reduction in conveyance losses, and the economics indicate that the benefits of saving lost water exceed the costs, channel improvement techniques to reduce the losses should be developed. But before improvement techniques can confidently be proposed, the causes of the losses in the present channels must be determined. Only if the present system's problems are understood can improvements be developed which will most effectively solve the problems.

Since different phenomena cause different types of channel losses, the first step in determining the causes of the channel losses is to quantitatively divide the losses into the various measurable categories. For example, as was explained in the previous section, while most losses will occur while the channels are flowing steadily, a portion are attributable to transient phenomena. Likewise, transient losses can be sub-divided into dead storage, short term bank breaches and outlet breaks, and initial high infiltration rates into dry banks. The causes of dead storage losses are obviously different from those which affect steady state seepage. Table 4.1 lists some of the categories into which channel losses can be divided.

Table 4.1. Categories of Channel Conveyance Losses

I. Steady State Losses

- A. Seepage into the bed and banks
  - 1. Normal infiltration into bed and bank soils
  - 2. Excess seepage into bank holes and cracks
- B. Visible leakage through and over the banks
  - 1. Overtopping
  - 2. Leakage through the banks
  - 3. Leakage through closed outlets
- C. Evaporation from the water surface

II. Transient Losses

- A. Initial seepage into dry banks in excess of normal long-term seepage rates

B. Dead storage

C. Short term leakage

1. Bank washouts and breaches

2. Outlet breaks

III. Wastage

A. Purposeful wastage due to lack of need

B. Malicious wastage

4.1 Methods to Quantitatively Divide Channel Losses into Categories

Many of the types of losses listed in Table 4.1 such as leakage through closed outlets can be measured directly. The relative importance of the more difficult to measure types can be deduced by constructing a water budget. For example, if both total operational losses (Section 3.1.1) and the amount of steady state losses are known, then the amount of transient losses would simply be the difference between the two. In some cases, due either to a lack of information or a difficulty of making measurements, an educated guess may have to be made in dividing losses into their categories.

4.1.1 Steady state losses

Section 3.1 describes both the ponding and the inflow-outflow methods of measuring total steady state losses. Either should give an adequate estimate of these losses within the limits mentioned. While these measurements are being made, visible leakage can be physically measured by collecting the leakage water in a container (Figure 4.1) or in a small pond for a measured time. The volume collected (liters) divided by the collection time (sec) will give a leakage rate (lps). If leakage is directed into a small ponded area, the depth increase in the pond over a given time interval times the surface area, divided by the time interval, gives the leakage rate. The pond should be small enough that the seepage from the pond is much smaller than the leakage collected. Measured visible leakage should be catagorized as to type; i.e.: whether it is leakage through poorly closed outlets (Figure 4.2), leakage through cracks, insect holes, or weak, thin sections of the banks, or overtopping (Figure 4.3). Visible leakage is usually easy to locate because of the wet area or standing water in adjoining fields and branch channels.

Directly measuring water surface evaporation is impractically difficult. However, fairly accurate estimates can be made if evaporation rates for the region are known. Evaporation from the surface of a small body of water such as a channel should be roughly equivalent to the pan evaporation, or 10 to 30% larger

Figure 4.1. Collecting visible leakage in a container for volumetric measurement.

Figure 4.2. Visible leakage through a poorly closed outlet.

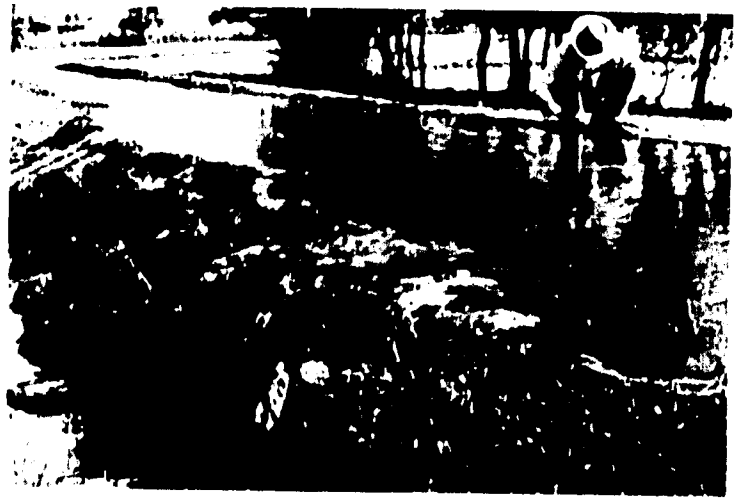


Figure 4.3. Water overtopping a channel bank.

than the potential evapotranspiration (PET). For example, a channel which averages 1.2 m wide at the water surface would have a surface area of 1.2 m^2 per meter length. If the average pan evaporation rate in the area is 1.0 cm/day, the loss rate attributable to surface evaporation would be:

$$1.2\ m^2 \times 1.0\ cm/day \times \frac{1}{100}\ m/cm \times \frac{1}{86400}\ day/sec \times 1000\ \ell/m^3 = 0.00014\ \ell ps/m.$$

This would only be 1.4% of a 1.0 ℓps/100 m loss rate, or amount to about 0.8% of 50 ℓps inflow to a 3000 m channel.

The remaining steady state loss is seepage into the channel wetted perimeter. In many channels, this category will include the majority of the total losses. This seepage is the difference between total steady state losses and all other types of measured or estimated steady state losses. Steady state seepage can be further subdivided into that portion which is the result of normal infiltration into bed and bank soils, and excess seepage caused by cracks, holes, and burrows in the banks. Separating out these two types of seepage is difficult and must be done indirectly.

The best way to estimate the normal steady state infiltration loss would be to destroy and reconstruct short channel sections insuring that there are no holes or cracks in the banks, allow the section to "age" (accumulate silt, further compact, and "grow" a surface seal) with normal use for a short time, (perhaps a month) and then measure losses by the ponding method. The amount of compaction applied to the bank soils during reconstruction will have a strong affect on infiltration rates and should be comparable to what could be expected in an improvement program. An additional indication of normal infiltration losses would be to measure ponding losses in several existing channel sections in a similar soil and assume that the lowest of the steady state seepage rates measured approach normal infiltration losses.

The difference between steady state seepage losses and the estimate of normal infiltration would be the excess seepage. Other indirect indications of the extent of excess seepage is the variability of ponding loss measurements in adjoining or similar channel sections, the variability of loss rates with flow depth fluctuations and visible evidence of highly porous banks. The standard deviation of losses measured in adjoining channel sections in Pakistan was often half as large as the mean of the measurements. Since soil types were reasonably uniform the variability was assumed attributable to excess seepage.

If bed and bank soils are fairly homogenous, the loss rate should increase as flow depths increase only because of wetted perimeter length and pressure head (water depth) increases. Generally these factors will cause losses to increase less than

proportionately with flow depth. If losses increase more rapid than depth, this indicates the permeability of the banks are higher than of the bed soils. Such a relationship would be caused by more silt deposition or algae growth on the beds, but if the loss rate varies greatly, it also would indicate excessive permeability of the bank and especially the upper bank soi Ponding loss measurements of this relationship (Section 3.1.2) will indicate if this is the case in the analyzed systems. Bot Colorado laterals and Pakistan watercourses displayed a strong exponential relationship (Eq. 6 and Figure 3.11) between loss rates and flow depths.

$$Q_L = Q_{LO}e^{b\Delta D} \tag{6}$$

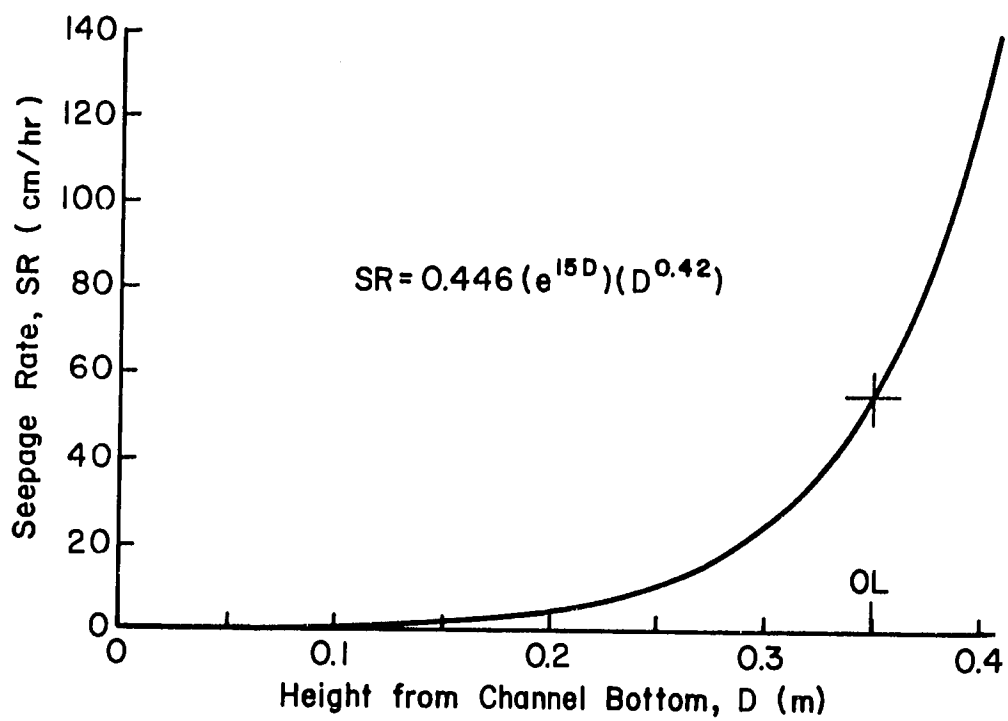


Figure 4.4. Seepage rate into the bank wetted perimeter as a function of height from the channel bottom, derived from the loss rate curve given in Figure 3.11 and assuming a parabolic shaped channel cross section.

The average value of the exponential coefficient, b, for over 100 measured sections was 15 m^{-1} resulting in a doubling of the loss rate with each 5 cm increase in flow depth. This strong relationship indicates that the water intake rates into the upper banks are 10 to 100 times greater than the intake rates of the channel beds, as shown in Figure 4.4. These extremely high intake rates of the upper banks could not be the result of normal infiltration, but must be caused by seepage into and through larger holes, cracks, and burrows in the banks.

4.1.2 Transient losses

Transient losses are those which occur while the system is not flowing under steady state conditions. Total transient losses cannot be measured directly, but can be calculated as the difference between operational (volumetric) inflow-outflow loss measurements and steady state measurements. A graphical example of this calculation is shown in Figure 4.5.

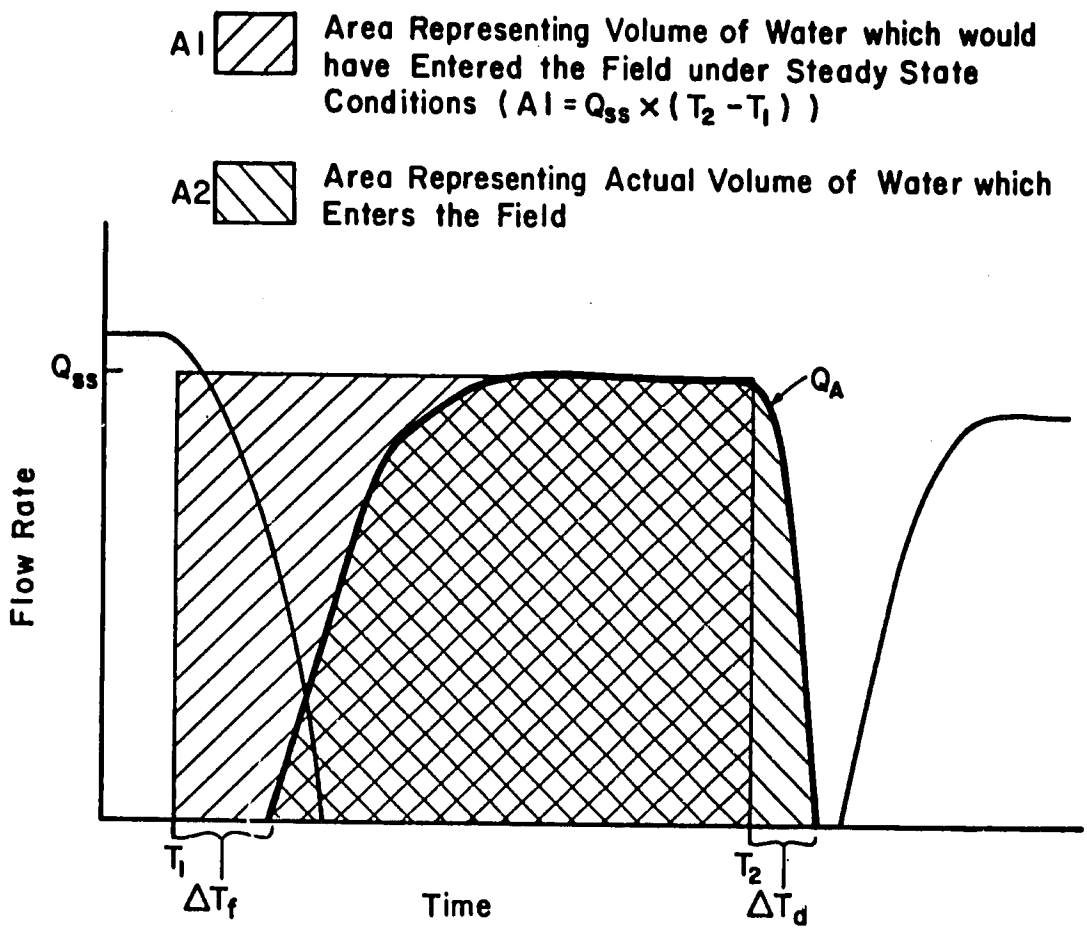
Dead storage transient loss is the water which does not drain into fields after a channel is used (Figure 4.6). This loss can be physically measured by estimating the cross-sectional area of the dead storage water soon after drainage is complete at regular intervals, along the channel, and multiplying the area by the interval length to get a volume of water loss.

Transient losses resulting from short term bank washouts and breaches, outlet breaks, and short term wastage are difficult to predict and thus measure directly. However, this can be estimated from flow rate changes at flumes installed below the breaks. During operational measurements, a short term drop in the flow hydrograph (Figure 3.2) indicates such a short term loss. As shown in Figure 4.7, graphically integrating the decrease in the flow rate from the steady state rate will indicate the volume of water lost in the break or breach. Short term losses should be categorized according to type.

The remaining transient loss — increased seepage into dry channels during filling — can be estimated as the difference between total transient losses and the other measurable transient losses. An estimate of the volume of water lost to increased initial infiltration can also be made by conducting infiltration tests in short dry channel sections. The measurement procedure is described in section 3.1.2. This initial infiltration value, when multiplied by the length of dry channels filled, will indicate the volume of initial increased seepage.

4.1.3 Estimating wastage losses

In many irrigation systems, part of the water conveyed through the system is not applied to crops either because of a lack of need for water (real or perceived), or because of malicious misuse. Although this wastage is not really a loss



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 4.5. Graphical depiction of the transient loss calculation.

Figure 4.6. Dead storage water lying in the bottom of a recently used channel.

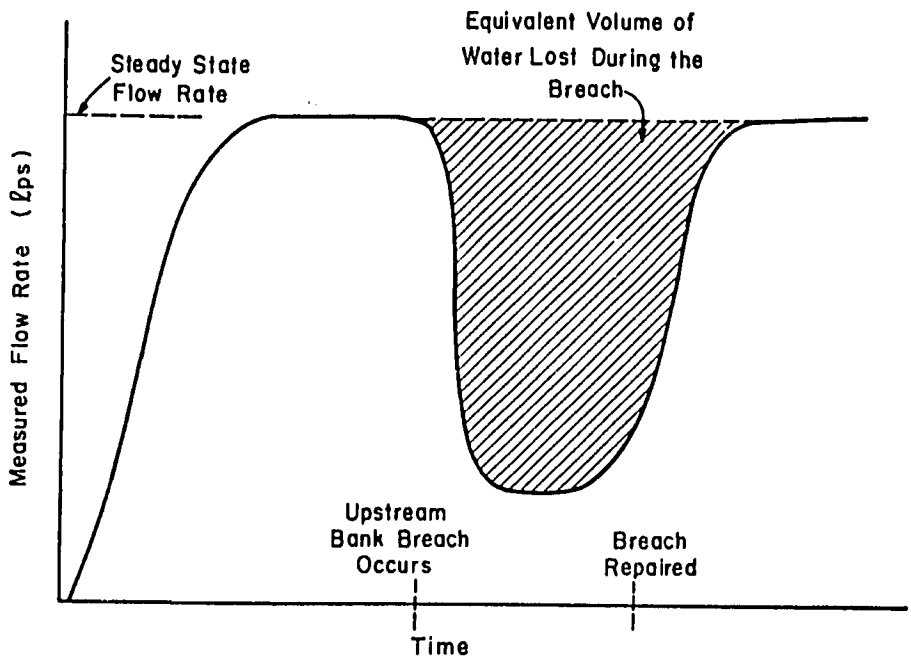


Figure 4.7. Graphical depiction of the short term bank breach loss calculation from the flow hydrograph.

from the tertiary conveyance system, and cannot be blamed on the system, it is a loss which can sometimes be partially remedied by improvements in the system, and thus should be considered.

When a water supply system is rigid, or the rotation is fixed, farmers will occasionally receive water when they have no beneficial use for it. Periods of low need may occur after rains or during harvest or planting periods between cropping seasons. Farmers may apply water to fields which have no water deficit or divert it to a drain. The amount purposefully wasted could be measured with flow measuring devices at the outlet to the drain or the outlet to an already wet field, or estimated by asking farmers how often they receive water they do not need.

Malicious wastage occurs when a person purposely causes a short term loss, either by opening an upstream outlet or cutting an upstream channel bank. The motivation for the action might be revenge or competition. Such losses are measured in the same way accidental short term losses are measured.

4.1.4 Classifying loss rates with location on the system

Determining relative differences in loss rates at different locations in the system can also help indicate the causes of water losses. Sub-systems which might be studied and compared include:

1) Constantly used main channels versus branches,

2) Primary channels versus field branches,

3) Junction areas versus normal sections, and

4) Elevated channels versus low channels.

For example, in measured watercourses, field ditches which were filled only occasionally had loss rates which were nearly double those of the primary channels. In some systems, as much as half of the total losses occurred in the vicinity of major junctions. Elevated channels tended to exhibit higher loss rates than sections built lower in the ground. Each of these findings, when the underlying cause is determined, can help determine the reasons for conveyance channel losses.

Figure 4.8 shows an example of qualitative accounting of the various types of channel losses constructed from indepth operational loss measurements on five Pakistan watercourse systems. Constructing such a flow chart is suggested in any diagnostic effort.

4.2 Determining the Causes of Channel Losses

With a flow chart of the losses such as is shown in Figure 4.8, the task of diagnosing the causes of channel losses and

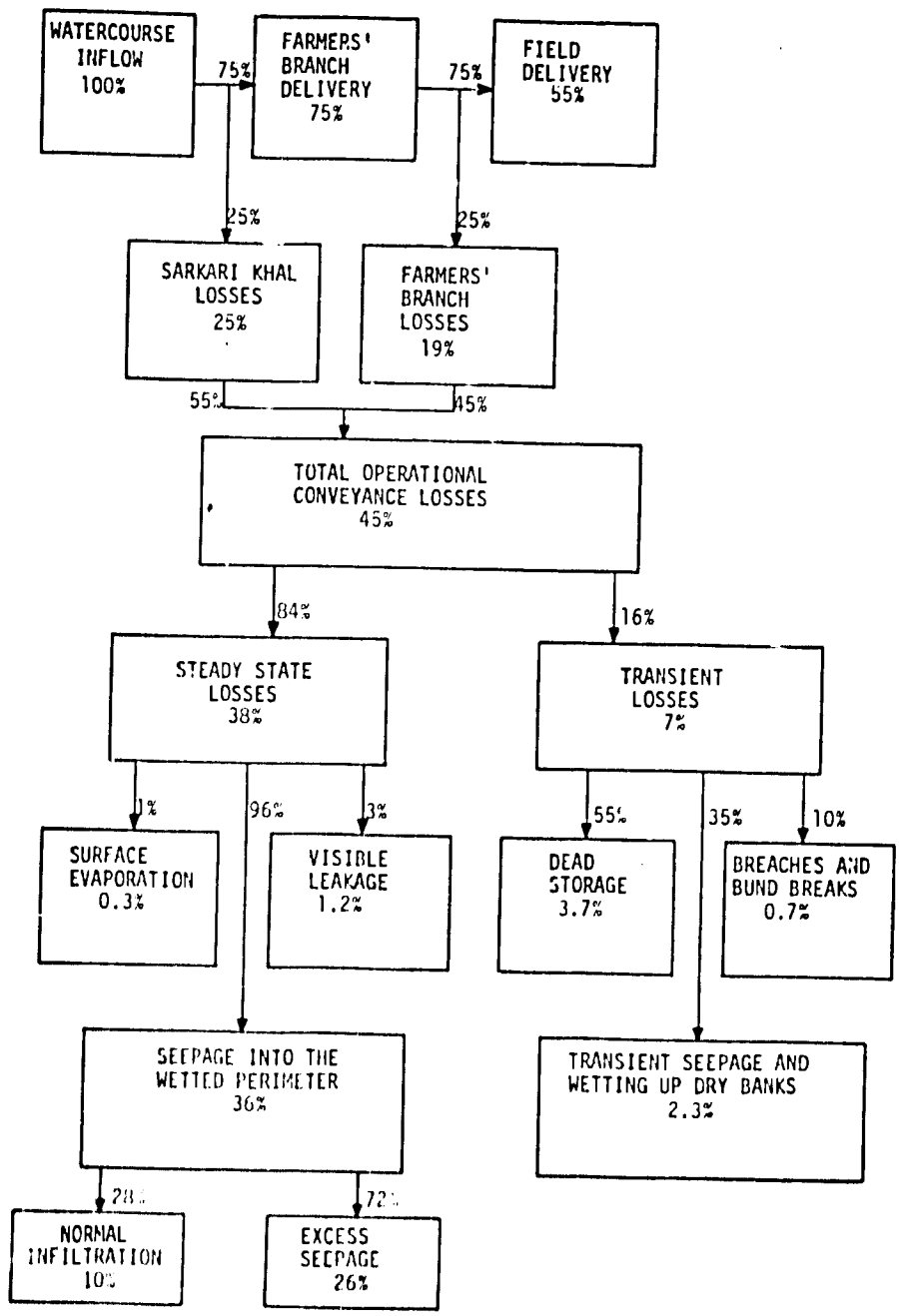


Figure 4.8. Flow chart showing a quantitative accounting of various types of losses measured in Pakistan watercourses.

evaluating the relative importance of the problems becomes much easier.

4.2.1 Causes of high steady state seepage rates

If the normal infiltration rates are high, the cause is probably large primary pores resulting from coarse soil grains. Seepage rates from channels built in gravelly or sandy soils such as the one shown in Figure 4.9, often have very high loss rates. However, channels built in sandy soils can still have fairly low loss rates if the conveyed water carries a heavy sediment load which deposits and builds up a fairly impermeable layer on the wetted perimeter. Likewise, normal infiltration losses will be accentuated by long wetted perimeters such as wide shallow cross sections or ponds where animals cross or wallow or in junction areas (Figure 4.10).

Excess seepage will generally be caused by large pores or holes in the banks or under the channel. Further investigations and perhaps hydraulic studies will need to be undertaken to isolate the causes of excess seepage. The first step in this diagnostic process should be to excavate some channel sections where excessive seepage is a problem to determine the location of and reason for the high porosity. Possible causes include poor initial soil compaction; soil loosening by frost action; flow paths left by decaying plant roots; and holes and burrows dug by insects such as ants (Figure 4.11) or mole crickets, worms, or burrowing animals such as rodents (Figure 4.12), moles, ground hogs, and crayfish. Close observation should indicate which creature or creatures are the cause of holes and burrows. Further understanding of their living and burrowing habits will be useful in determining ways to reduce the water losses they cause. A strong relationship between loss rates and flow depths will indicate that the burrows, or at least the point where water leaks into them, is concentrated higher in the banks. This would indicate that the inhabitants causing the problem attempt to stay above the water. It should be pointed out that, even though these flow paths never re-emerge on the soil surface and result in visible leakage, if the burrow system is extensive and interconnected, the full caverns will increase the effective wetted area of the channel and may lead to more permeable strata. In Pakistan watercourses, channel banks were honeycombed with interconnected rat and ant holes. The rat burrows often extended down and out under the neighboring fields. Water would occasionally be observed flowing to the surface several meters from a flowing channel.

Total seepage losses also depend upon the length of channel through which the water is conveyed. The losses can consequently be interpreted as depending upon both the distance of the field from the inflow source and the tortuosity or indirectness of the flow path. Irrigating a small compact area will involve less conveyance length and loss than supplying an equally sized but more, spreadout command area.

Figure 4.9. A channel built in sandy soil which has high normal infiltration rates.

Figure 4.10. A widened channel section with a long wetted perimeter which causes increased normal infiltration.

Figure 4.11. Ant holes visible on the inside surface of a recently cleaned channel bank.

Figure 4.12. Rat burrows visible in a partially excavated channel bank.

4.2.2 Causes of visible leakage

The causes of visible leakage are generally apparent from observation. Since visible leakage is an obvious loss of water, it indicates poor maintenance and lack of care of the conveyance system by the users. Overtopping can be caused by combinations of inadequate bank freeboards, extensive vegetation in the channel, and/or an obstruction to the flow. Leaks through the banks are often the result of burrowing activities or of trimming soil from the banks to build checks or enlarge the fields until the banks are thin and weak and holes are more likely to extend clear through the banks. Outlet leaks often result from carelessness on the part of the user, although shrinking, cracking soils often make the problem more difficult.

4.2.3 Surface evaporation losses

Surface evaporation depends upon the evaporation rate and the surface area. The evaporation rate depends on several climatic factors which cannot be economically controlled, considering the small potential in water savings. The surface area of the flowing water depends upon conveyance lengths and the water surface width, which can be affected to some extent by layout changes and proper cross-sectional designs.

4.2.4 Determining the causes of transient losses

Extra water infiltrated into dry channel beds and banks will depend upon how dry the banks are before filling, and the length of channel filled during each rotation. Channel bed and bank soils dry out from evaporation, both from the soil surface which is fairly unavoidable, and from water taken up and transpired from plants living on the banks and in the channel. These plants can consequently be considered the cause of at least a portion of wetting up losses. Estimating what portion of the loss is caused by the plants is difficult, but can be roughly approximated by an estimation of their total transpiration.

The length of channels filled during each rotation or over a given period of time depends both upon the channel layout and the operating procedures. Small field and holding sizes will usually require an extensive field ditch system. Small field and holding sizes can consequently be considered a cause of high wetting up losses. Within this size constraint, however, field shapes and branch layouts can be designed to reduce the lengths filled.

Field irrigations can also be timed to increase or decrease channel fillings. If adjacent fields are irrigated consecutively and adjacent branches are filled successively, filling lengths will be minimized. If scheduling is on demand such that water is randomly moved around the system, these lengths will be much longer, greatly increasing wetting up losses. The operational

practices are consequently an important factor affecting wetting up losses.

Dead storage losses result both from channel beds lying below the field levels and from undulations in the bed. These are both physical causes which can be observed and evaluated. Dead storage losses also depend upon the number of channels which are drained during a rotation period, since each drained section may store water. Dead storage is consequently also dependent upon channel layout and operation.

Short term leakage and washouts, like steady state visible leakage, are obvious to the users and could be considered a consequence of their carelessness or a lack of system maintenance. Adequately strong banks or well closed outlets will not wash out. However, within this condition, there is considerable variable between the structural stability of different types of soils and their tendency to erode and wash away. Sands and other non-cohesive soils will wash out much easier than clays and loams. On the other hand, ditches built with cohesive soils with high aggregate stability often have large pores between the aggregates which allow leakage and can lead to bank washouts. Soil type can consequently be considered an important factor affecting short term losses.

4.2.5 Causes of water wastage

Wastage of water due to a lack of need is caused by water delivery system not matching the water requirement. Rigidity of both supply and demand can thus be blamed for the wastage and providing flexibility in either can solve the problem. More specifically, lack of storage on the conveyance system, no provision for system bypass, lack of control of flows in or diversions from the canal system, or lack of flexibility in the turn rotation system such as trading of water turns, all cause the water supply to be inflexible. On the use side, the wastage is caused by failure to plan crop timings to meet water supplies and improper management of the soil moisture storage reservoir.

Malicious wastage is due to sociological problems. This type of wastage can be saved by either improved interrelationships or adequate policing of the system. Improved structures which make malicious activity more obvious or difficult may also reduce the problem.

SECTION 5

DESIGNING AND OPERATING SMALL CHANNEL SYSTEMS FOR IMPROVED EFFICIENCIES

Section 4 described methods to quantify the types of losses from small irrigation conveyance systems and to determine some of the causes. This section will give some examples of how to use that diagnostic information to design and operate irrigation channels for reduced losses. Since channel lining is an obvious but high cost solution for most of the diagnosed problems, the following discussion will deal primarily with lower cost alternatives in earthen channels. The design factors which will be considered include:

1. bank material preparation and compaction,
2. sediment deposition,
3. cross-sectional shape and wetted perimeter length,
4. channel elevations,
5. inflow fluctuations,
6. cleaning operations,
7. layout of the channels,
8. operation of the system,
9. removal of bank vegetation,
10. outlets, and
11. channel lining.

5.1 Reducing Steady-State Seepage

The majority of the losses from most small conveyance systems will be steady-state seepage into the channel wetted perimeter. If study of existing channels shows that much of this seepage is excessive (Section 4.1.1), potential exists to save water by reducing seepage with earthen improvements. If normal infiltration rates are high, techniques such as induced sediment deposition can reduce losses from some earthen channels, but channel lining may be the only practical technique which can reduce this loss.

Seepage into the wetted perimeter is primarily dependent upon two factors: 1) the forces pushing water into and through the soil, and 2) the ease with which water will move through the soil (i.e., the permeability of the bed and bank soils).

In small channels, the second factor is much more variable and can be more easily changed, and is thus the factor which will be discussed.

A soil material's permeability depends primarily on the size and continuity of the pores (holes) for these form the potential flow paths in the material. The pore size and distribution in turn depends both upon the sizes of the soil grains and the way the grains are packed together. The voids or holes between well-packed soil grains is termed primary porosity. The spaces between aggregates or clods of soil is called secondary porosity. Coarse sandy soil has a high permeability because the grain sizes are large, and thus the voids between grains are large. Likewise, aggregated loam soils will often have a high permeability because, although the soil grains and consequently the primary pores are small, the holes between the aggregates are large. Figure 5.1 shows the primary and secondary porosity of a hypothetical soil. Generally, normal infiltration depends upon the primary porosity while excess seepage results from secondary porosity.

Secondary porosity in irrigation channel beds and banks is usually the result of either aggregates which were present in the soil during construction and were never destroyed, soil loosening resulting from frost action, or biological activity such as plant root growth and subsequent decay or the burrowing of insects, worms, and rodents or other small animals in and under the channel bed and banks.

The seepage into a channel wetted perimeter will depend primarily on the permeability of the least permeable soil layer the seepage water must pass through. This implies that, even if the channel bed and bank materials are fairly porous, if a layer of low permeability silt is deposited on the bed or a core of low permeability clay is compacted in the bank, low seepage levels can still be achieved.

These two factors suggest two basic strategies to reduce steady-state seepage:

1. eliminate pores in the channel bed or bank soils which are larger than the pores between well-packed soil grains (primarily porosity), and

2. induce the formation of layers with low permeabilities in or on the surface of channel beds and banks.

5.1.1 Bank compaction

Soil aggregate size and thus secondary porosity can be substantially reduced during channel construction by proper soil preparation and compaction. The most important factors in soil compaction are the compaction pressure, the soil moisture at the time of compaction, and the compaction layer thickness. Akram and Kemper (1979) found that infiltration rates

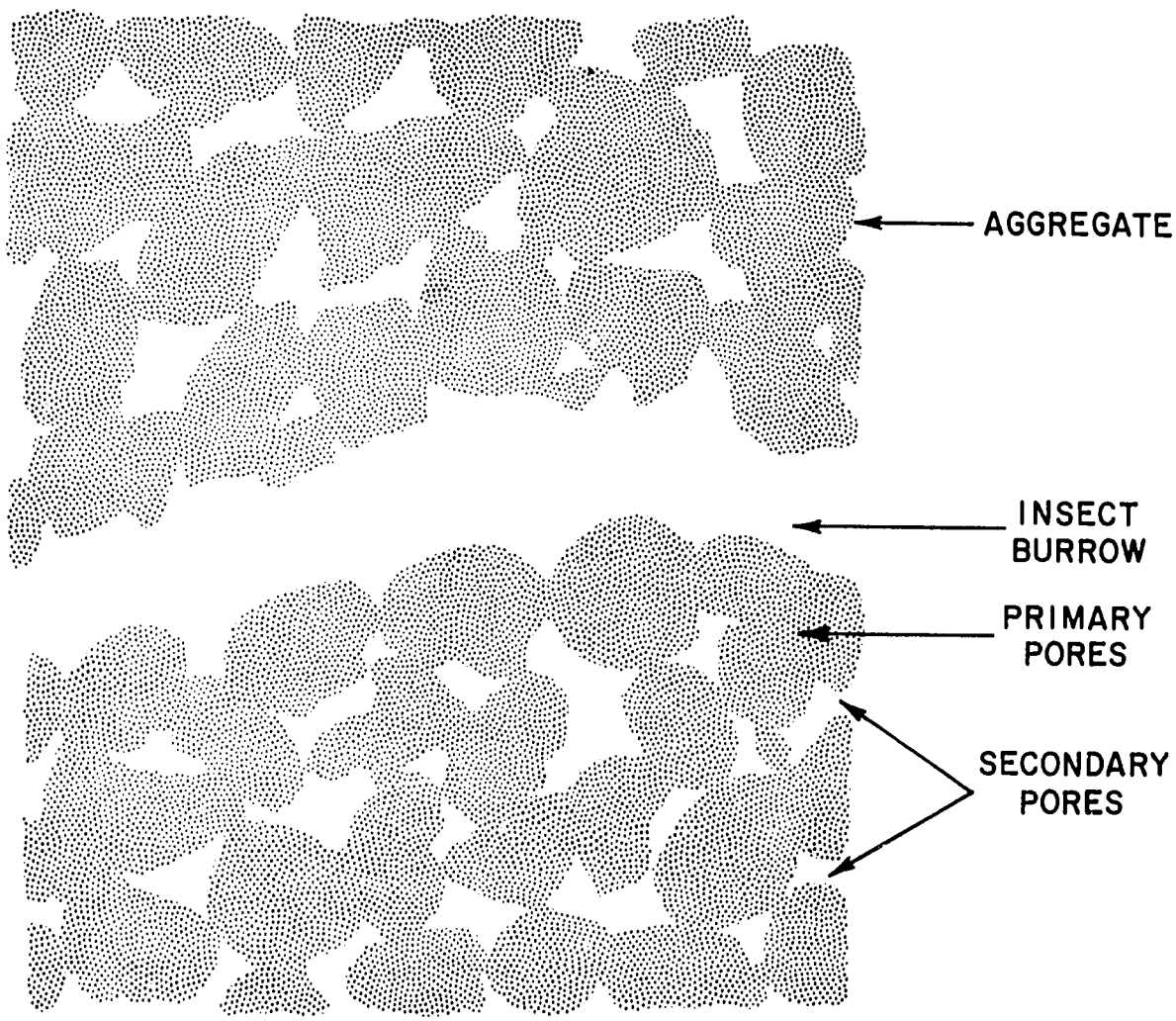


Figure 5.1. Hypothetical soil cross section showing primary and secondary porosity.

of agricultural soils (sandy loam, clay loam, and silty clay soils were studied) could be reduced to less than 1 percent of their uncompacted values with moderate compacting pressures (1.73 kg/cm^2) and to less than 0.1 percent of their uncompacted values with a compacting pressure of 3.46 kg/cm^2 if the soil moisture is near field capacity during compaction. Figure 5.2 shows their results for a clay loam soil. They concluded that the optimum moisture content for compacting with moderate pressure for most agricultural soils is near field capacity. The importance of moisture content is demonstrated in Figure 5.2 in which the relatively dry soil showed no reduction in infiltration rate with compaction. These moderate compacting pressures can be achieved with the rear wheels of a tractor, a small tractor or animal-drawn sheepsfoot roller (Figure 5.3), or a manual tamper (Figure 5.4). Even compaction of moist soils with men's (or women's) feet will achieve much of the compaction required to significantly reduce the permeability. The compaction should be done on successive layers of moist soil which are no thicker than about 10 cm. Although wetting and drying or freezing and thawing cycles tend to increase the infiltration rates of the compacted soils, Akram and Kemper (1979) found that in no case did the rate rebound to the uncompacted level.

Freezing and thawing cycles in temperate climates tend to loosen soils and increase their permeabilities. There is nothing practical that can be done to avoid this process. Periodic recompaction can lessen the effects. Frost-free tropical climates will not have this problem and a compacted soil will remain compacted indefinitely unless it is disturbed by the biological factors discussed below or wetting and drying cycles cause cracking.

5.1.2 Discouraging biological pore formation

Both plant and insect or animal activity in channel banks can cause the formation of large pores which can in turn cause high seepage rates. Even though these water passages often do not result in leakage through the banks, they are often extensive, extending out and under adjoining fields, and when filled with water, greatly increase the effective wetted area of the channel and often lead to more porous substrata.

Plant roots which grow and decay open up passages in the soil. The result is long continuous pores varying in diameter from a fraction of a millimeter to over a centimeter through which water can pass freely. The easiest solution to this problem is to discourage the initial growth of plants with large roots by maintaining clean channel beds and banks. This could be accomplished manually or with herbicides.

Discouraging the insect or animal activity can be much more difficult. These occupants are usually attracted to the channel bank environment for good reasons. For example,

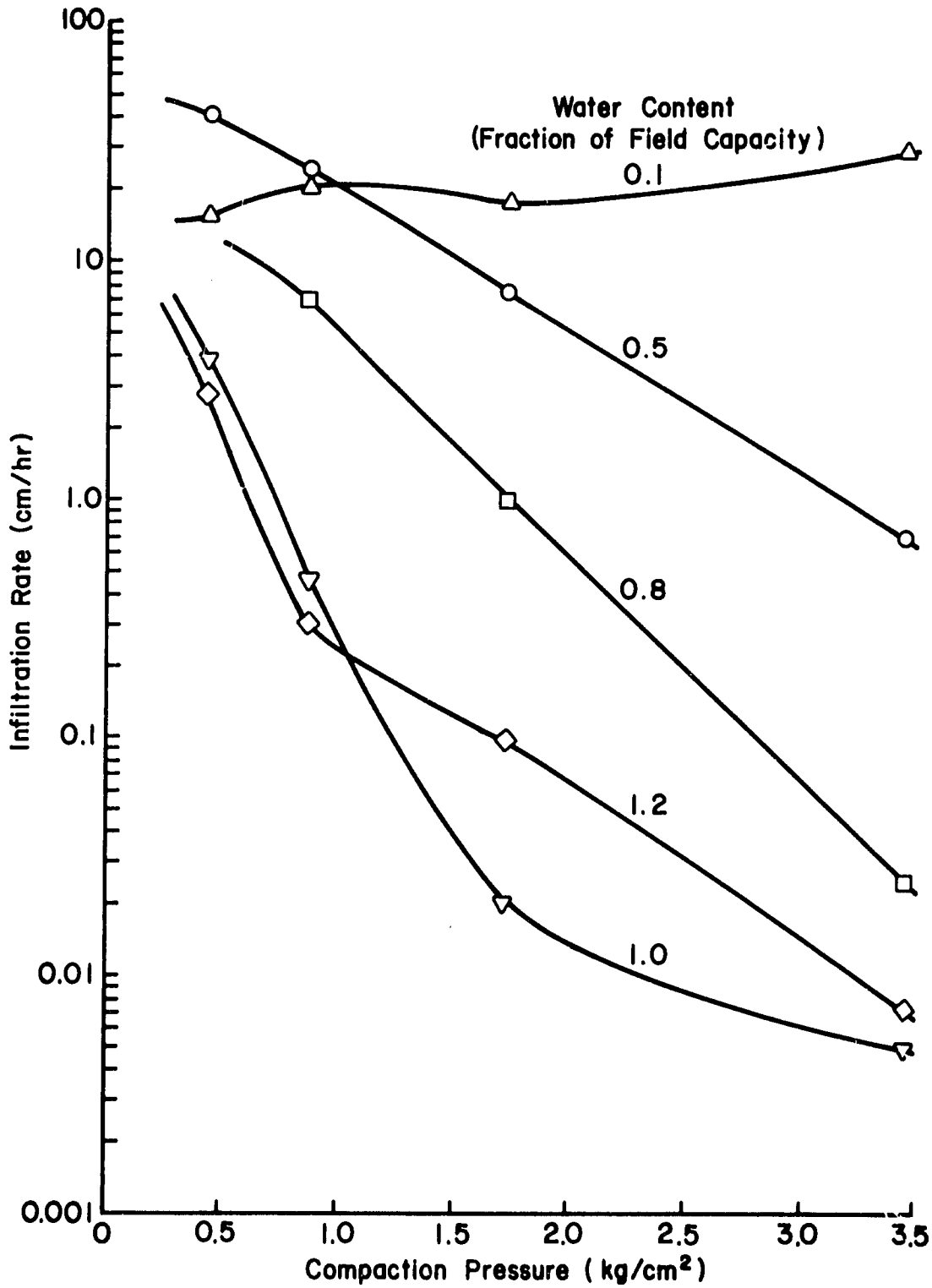


Figure 5.2. Infiltration rate as a function of compaction pressure for a clay loam soil at different moisture contents.

Figure 5.3. Small animal-drawn sheepsfoot roller.

Figure 5.4. Manual soil tamper.

channel bank soils are higher than field levels and are seldom submerged. Especially in basin irrigated areas, the channel banks may be the only large soil mass permanently above water. Channel bank soils are usually undisturbed for long periods of time, unlike the surrounding fields which are regularly cultivated. Consequently, burrows in banks are more permanent. Vegetation which often covers channel banks provides both cover and a food source for inhabitants. The vegetation is usually not harvested and removed as it is from surrounding fields. The flowing water in the channel also helps maintain a cool, moist bank environment which is desirable for living and burrowing for some inhabitants.

Chasing those burrowing inhabitants from channel banks would require changing the environment to make it less desirable, which is difficult or impossible. Channels can be built lower to the ground, but the banks still must extend above the land surface levels if water is to be delivered to adjacent fields. Keeping the banks clean of vegetation eliminates the cover and reduces the food supply but is time-consuming and costly. Destroying and rebuilding the banks regularly discourages permanent habitation, and is perhaps the best means of controlling the digging of holes in the banks, but is also time-consuming if manual labor is used. Seasonal redigging of field channels with V-ditchers and tractors is a common practice of some farmers in the western U.S. This practice discourages permanent inhabitants, destroys old burrows, and removes vegetation. If this practice is to be adopted, the use of permanent structures must be minimized.

Another control option is to kill the digging insects and burrowing animals with the use of poisons. Since some of these occupants are also pests which damage field crops, such a control program may have benefits beyond reduced leakage. However, since the conveyed water is often used for many purposes, this can be a dangerous operation, and utmost care must be used. Also, many of the inhabitants regenerate quickly and migrate freely, so such a control program must be both extensive and continual to be effective in the long term.

An alternative means of decreasing the high seepage or leakage rates attributable to the bank holes and burrows of insects and animals is to design or operate the channels in such a way that isolates these large flow paths from the flowing water. One method may be to lay out and operate the conveyance system such that the primary channels are full most of the time. Since most insects and animals will not burrow in submerged soils or live in flooded holes, they will be driven out of these sections or at least be forced to concentrate their activity above the water level. Similarly, maintaining a fairly stable water surface level in the channels would tend to force most digging activity in the bank to take place above the water surface where the holes would not cause leakage.

Section 5.2 discusses ways to maintain constant water surface levels. Another possible option would be to compact a dense soil core near the inner channel bank surface in the hopes that burrowing activity would be concentrated in the outer, softer soils.

A final obvious solution to the problem of seepage and leakage into insect- and animal-dug pores would be to construct an impermeable barrier between the flowing water and the porous soils, or line the channel.

5.1.3 Silt and microbiological seals

The second strategy to reduce seepage into the channel wetted perimeter is to create or cause the formation of a relatively impermeable layer on the wetted perimeter surface or in the bank. Two natural processes in earthen channels, sedimentation and anaerobic microorganism growth and secretions will tend to seal the wetted perimeter surface. Facilitating both of these processes can help reduce seepage losses.

Sediment deposition will depend on the size and amount of sediment in the water and the velocity of the flowing water. Although sediment will not deposit in some channel sections during steady flow conditions, velocities change during transient conditions (filling or draining of channels) allowing sediment to deposit. Installing partial temporary checks across channels will increase flow depths, decrease flow velocities, and induce sediment deposition in channel sections where flow velocities are normally too high.

If little sediment is present in the flowing water, fine- to medium-textured soils can be mixed in the water and allowed to deposit. A second method of artificially inducing sediment deposition would be to pond water in sections of the channel by constructing check dams across the channel, mixing fine soil in the ponded water, and allowing it to deposit on the wetted perimeter. Although the sediment seal in a channel section in which sediment normally deposits will be fairly permanent, an artificially induced seal will tend to erode away in time and must be renewed periodically. Working the finer soil into the top few centimeters of the wetted perimeter by puddling can extend the effective life of the seal.

The effectiveness of sediment seals in Pakistan watercourses is graphically demonstrated by two situations. First, in most channels, the channel sides are regularly cleaned of vegetation during which the surface sediment layer is also scraped away, while sediment is allowed to accumulate on the beds. Ponding measurements show that the seepage rate into channel banks is as much as 100 times higher than into the bed (Figure 4.4). Second, in watercourses serving very sandy soils, the channel sections near the inlet where sediment deposition

is greatest have loss rates comparable to those normally measured in fine soils, while branches near the tail where little sediment deposits exhibit loss rates three times higher.

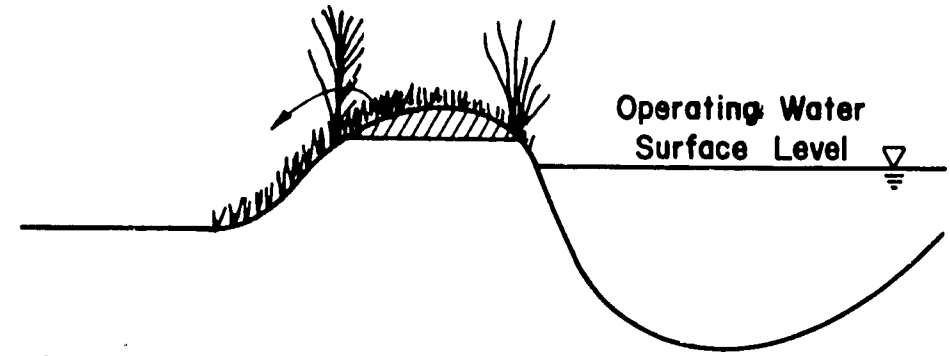
More sediment tends to deposit on channel beds than on the banks. This is partially because of the steep angle of the banks and can be partially remedied by constructing and maintaining flatter side slopes. Manual cleaning operations which remove the sediment seal from the banks or which tend to make them more vertical should be avoided if possible, although this often isn't practical.

Soils which are under water for periods of time tend to seal up. Several researchers (i.e. Allison, 1947) attribute this impermeable seal to the growth and secretions of anaerobic microbiological organisms. Irrigation channel wetted perimeters which are full for several consecutive days or are regularly filled a high percentage of the time tend to have lower seepage rates. Part of the decrease is attributed to this microbiological activity. The layout and operation of channel systems should consequently be designed to extend the filled time of main channels in order to promote this microbiological sealing and reduce seepage rates. Channel layout design which will increase filled time of the primary channels is discussed in Section 5.3.

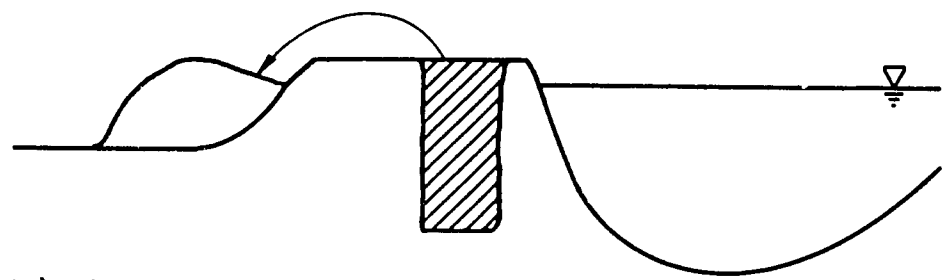
Another means of creating a relatively impermeable layer through which seepage water must pass is to compact a dense core of soil in the banks. This alternative is especially attractive if the beds tend to seal with sediment but the seepage rates into the banks are high. Such compaction can eliminate all secondary porosity and interrupt any insect or animal burrow flow paths in an existing bank and thus greatly reduce seepage through the bank.

Core compaction can be accomplished by digging a narrow trough in a moist bank, compacting the bottom of the trough with a tamper or heavy narrow wheel, and replacing the moist excavated soil in layers while compacting each layer. This core compaction process is depicted in Figure 5.5. The depth of the core will depend upon the area through which seepage should be reduced. The compacted cores should extend above the water level.

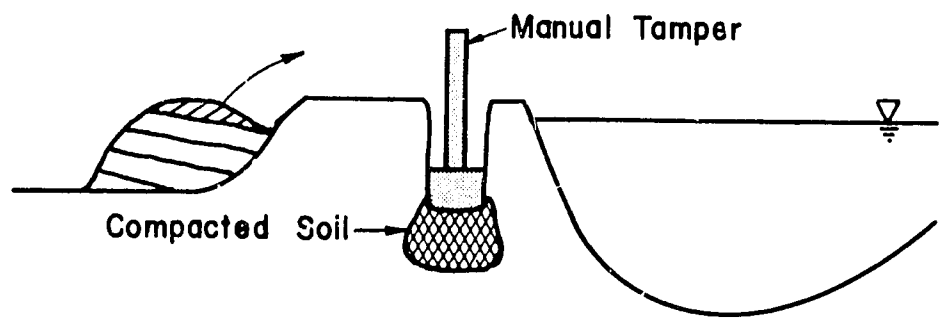
Compacted cores form a viable and relatively inexpensive means to reduce leakage and seepage through the banks of existing channels. In Pakistan, using a narrow spade and tamper, one man could compact a core in 15 to 20 m of bank per day. Compacted cores should be effective until freezing and thawing action loosens them or burrowing animals and insects penetrate them. If the bank coring procedure proves successful in reducing losses and a longer life improvement is desired, more durable materials such as concrete or brick masonry can be placed in the core trough instead of compacted soil.



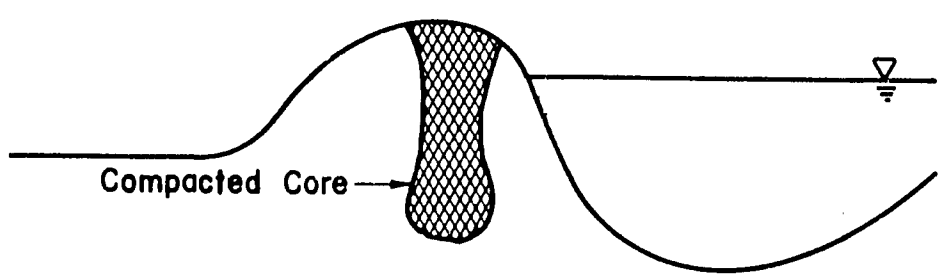
a) Remove the Bank Top and Vegetation.



b) Dig a Narrow Trough in the Bank Near the inside Edge.



c) Compact the Soil at the Bottom of the Trough with a Tamper and Replace the Soil in Layers.



d) Compact Each Layer of Replaced Soil with the Tamper Until the Trough is Filled and the Bank Top is Replaced.

Figure 5.5. Procedure for compacting soil cores in channel banks.

5.2 Maintaining Constant Inflow Rates and Vegetative Roughness to Reduce Seepage

Ponding loss measurements in Pakistan (Trout, 1979a), Colorado (Akram, et al., 1980), and New Mexico (Hanson, 1966), show that the losses in earthen channels generally increase rapidly as the water depth increases. This increase, shown in Figure 3.11, is much quicker than can be attributed to increased wetted perimeter length or pressure head, and must be the result of bank soils being much more permeable than the channel bed. Figure 4.4 shows the variation in seepage rate with height from the channel bottom for the loss rate data in Figure 3.11. This graph indicates that, although the seepage rate of the bed soils is very low, water seeps into the upper banks very rapidly--much more rapidly than normal seepage into medium textured agricultural soils. Some causes of highly variable seepage rates into channels were given in Section 4.2.1.

Figures 5.6 and 5.7 show how fluctuations in inflow rates or roughness coefficients will cause the depth of flow to change in a channel with a cross-sectional shape described by Eq. 5. In channels with a loss rate which changes with flow depth, these fluctuations will also affect water loss rates from the channels. Figure 5.8 depicts the relative change in loss rate with a change in either flow rate or roughness coefficient for the exponential loss rate: flow depth relationship described by Eq. 6 and a channel cross-sectional shape described by Eq. 5. An increase of either the flow rate or roughness coefficient by 30 percent will cause a depth increase that will about double the loss rate. Although the degree of this relationship will vary from channel to channel and from region to region, the basic relationship will usually be about the same.

This analysis shows the importance of maintaining fairly constant inflow rates and roughness coefficients for reducing losses in small channels. If extra water is put into a channel, the loss rate will increase and a surprisingly small portion of the extra water will reach the field. Failure to redesign and enlarge channels when inflows are increased, and the consequent high rates of water loss is a primary reason why supplemental water provided to an earthen channel system often does not produce proportionate increases in crop production. Consequently, inflows to a channel system should not be increased without redesigning and enlarging the channels for the increased flow. Also, if the primary channels in a conveyance system are renovated to reduce the loss rate, then more water will reach the downstream branches and much of the saved water will again be lost before reaching the fields. If an improvement program is undertaken, it is important that the improvement be continued in some form even if only in enlarging the channels, compacting bank cores, or cleaning the vegetation from the branches, to the field outlet.

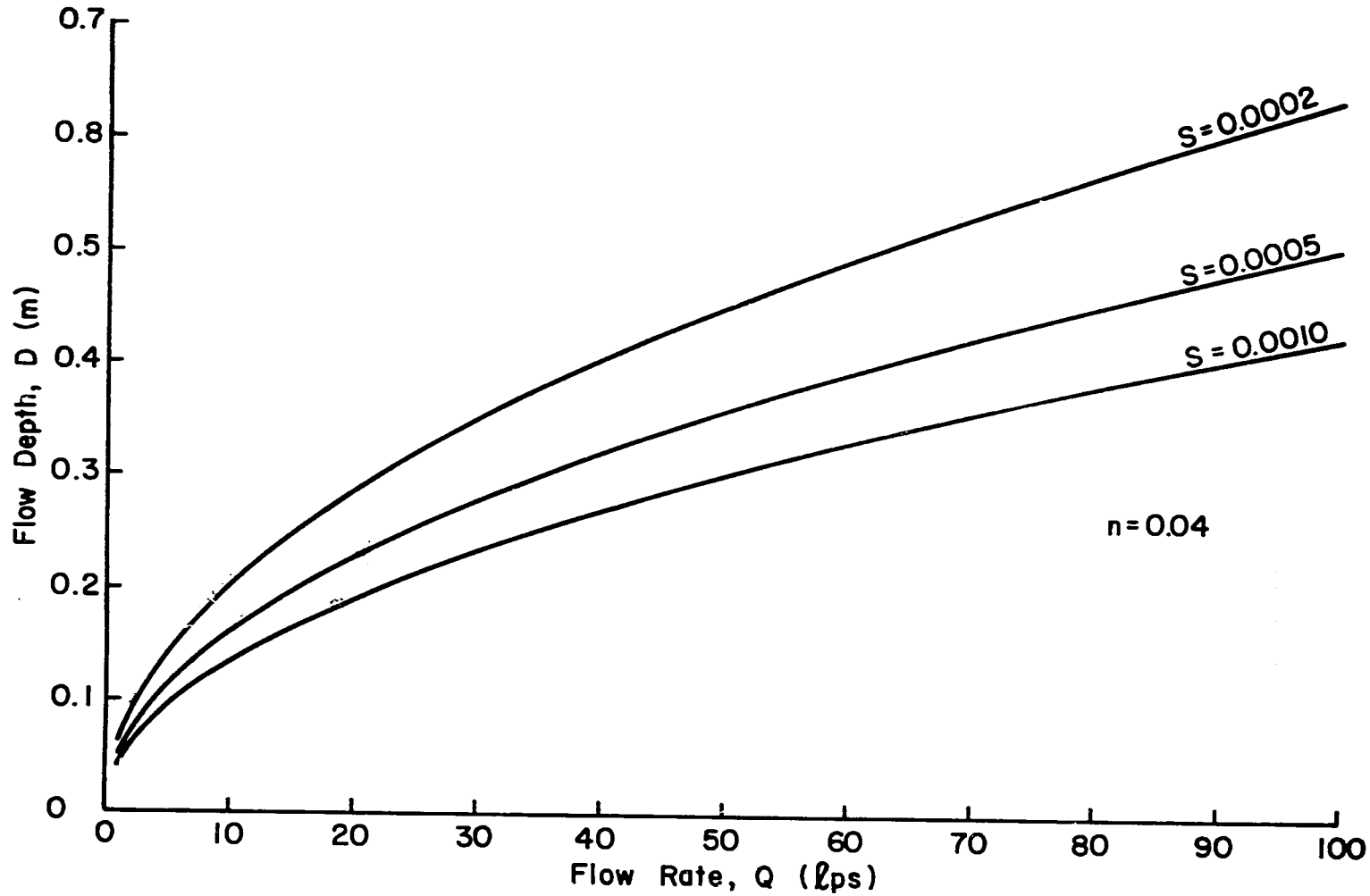


Figure 5.6. The effect of flow rate changes on flow depths (calculated for a channel with a cross-sectional shape described by Eq. 5 with $a_1 = 2$ and $a_2 = 3$).

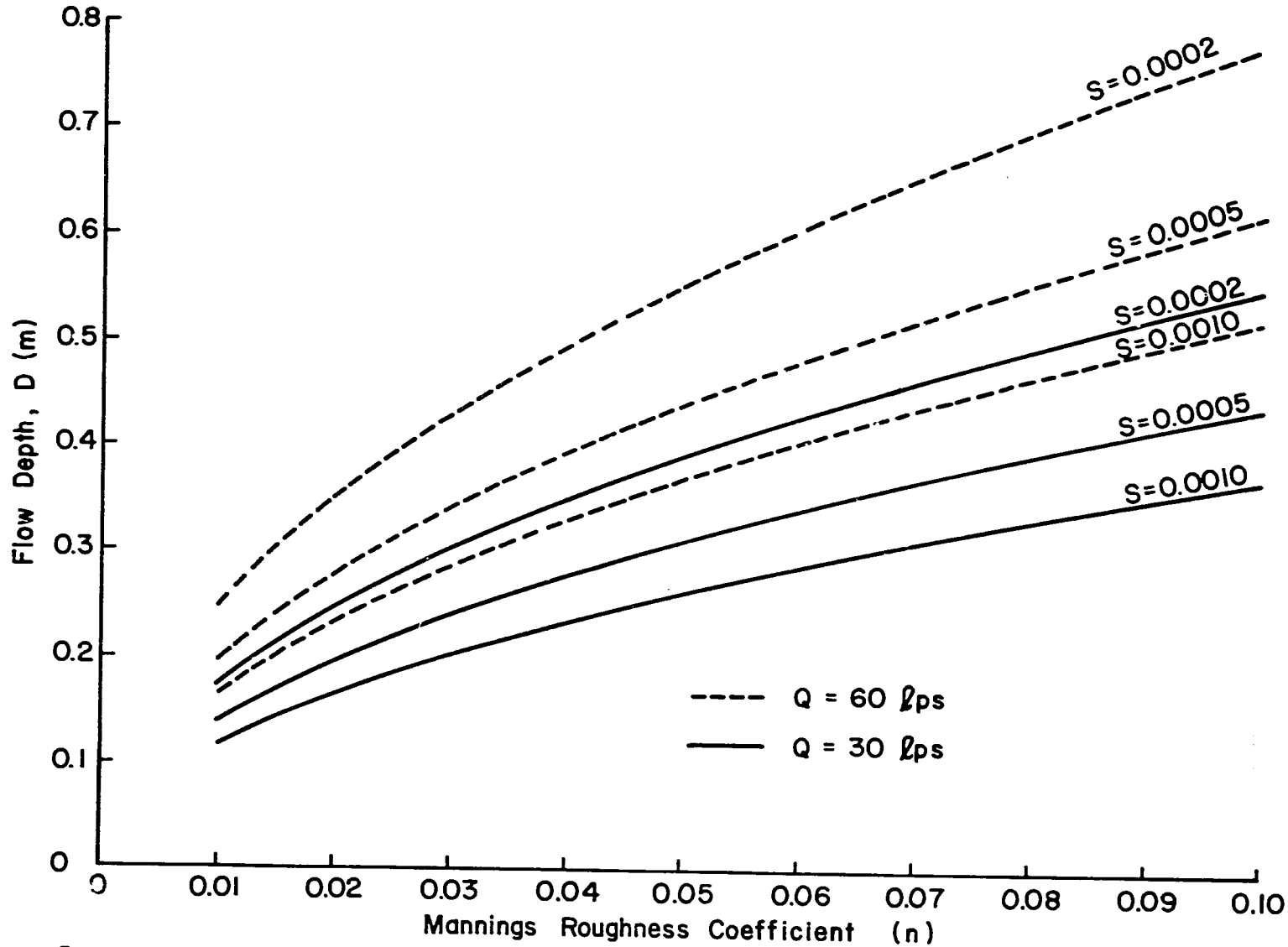


Figure 5. The effect of roughness coefficient changes on flow depths (calculated for a channel with a cross-sectional shape described by Eq. 5 with $a_1 = 2$ and $a_2 = 3$).

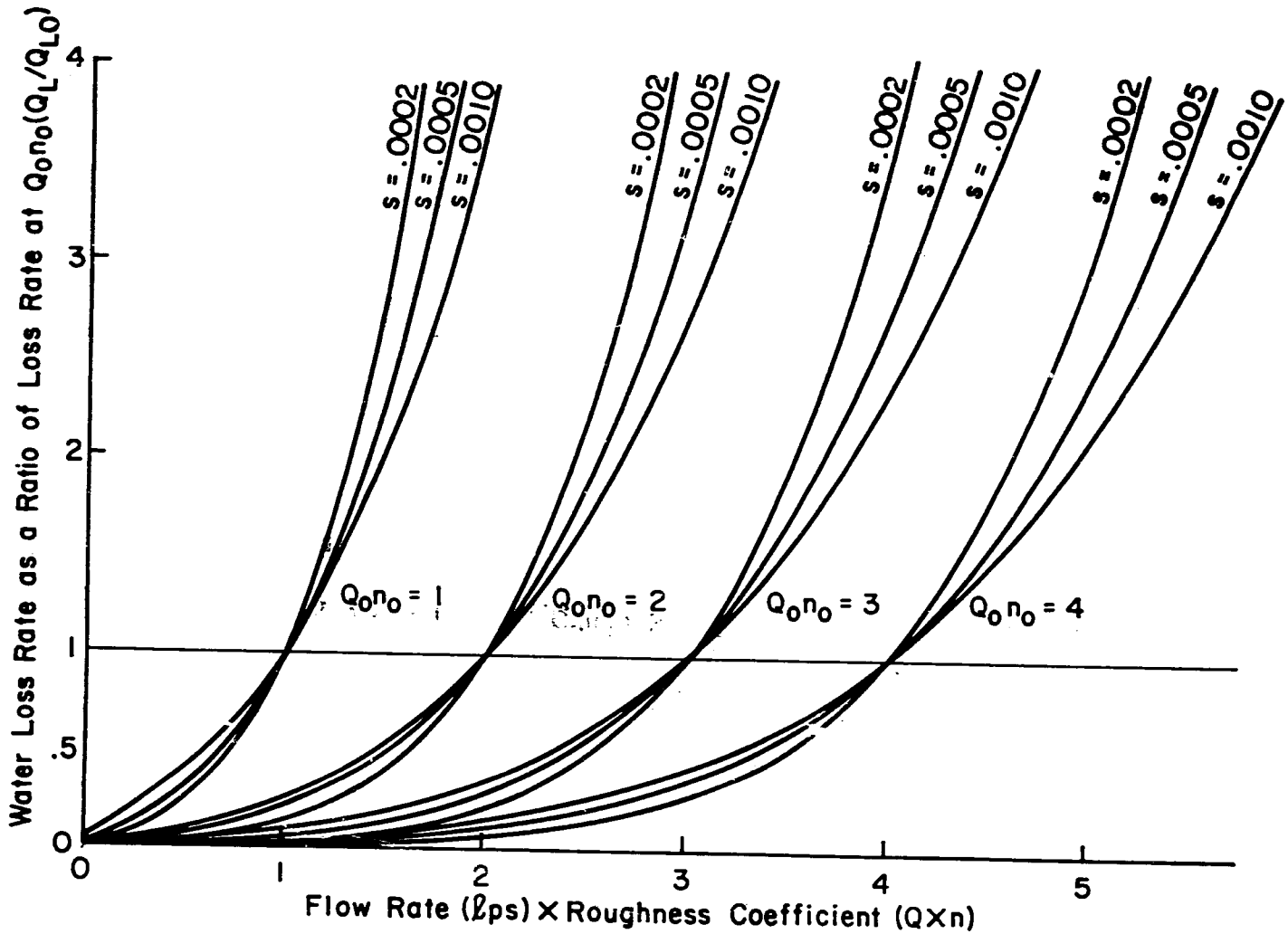


Figure 5.8. Relative change in loss rate with a short term relative change in the product of flow rate and roughness coefficient (based on Eqs. 5 and 6 with $a_1 = 2$, $a_2 = 3$ and $b = 15m^{-1}$).

Maintaining clean channels will also result in a stable water surface level and reduced channel conveyance losses. Figure 5.9 depicts the percent conveyance losses over time on a hypothetical channel system like that described in Figure 5.8 for three different cleaning schedules in which a clean channel is assumed to have a roughness coefficient of 0.03 and a weedy one, after six months without cleaning, an n value of 0.06. Figure 5.10 shows the resulting annual excess conveyance losses in a year due to increased roughness and flow depths with different cleaning intervals. These figures emphasize the potential water saving by simply cleaning existing channels and the importance of a regular cleaning program in any channel renovation project.

Another implication of bank seepage rates being much higher than those through the bed is that channel cross sections with wider beds and shallower depths may have reduced losses. Instead of designing cross sections strictly for minimum wetted perimeter length, cross sections should tend toward smaller depth-to-bottom width ratios.

5.3 System Layout and Operation to Reduce Losses

Both the layout of fields and channels and the operation of the channels will affect total channel lengths and, in a rotational turn system, total channel usage and the number of times each section is filled in a year. When total length of channels in a system is less, each section will be full for a larger portion of the time since more land will be commanded from each branch. As stated, keeping channel sections full more is desirable since the resulting increased microbiological sealing and decreased insect and animal burrowing would decrease steady-state leakage and seepage. With more land commanded from each branch, fewer channels will need to be filled which will also reduce dead storage and initial filling transient losses. Reducing the number and/or length of branch channels will consequently reduce conveyance losses.

One means of reducing the total length and number of channels would be to convert rectangular basins into long narrow borders or increase the run lengths of sloping borders or furrows. Figure 5.11 shows how the total field branch length in one area could be cut in half by rearranging small rectangular level basins to long narrow borders while leaving average field sizes unchanged. In the reorganized layout, transient losses would be cut nearly in half (a savings of 3 percent of the inflow in Pakistan) and total time of branch usage would be doubled which would result in further savings. This, of course, assumes that water turns rotate from field to field. In a constant flow system, where water flows into each branch continually little water would be saved by such layout changes. Although the long narrow fields would require more total length of dikes, the land area gained from the unneeded channels will more than compensate for the extra land needed for the dikes.

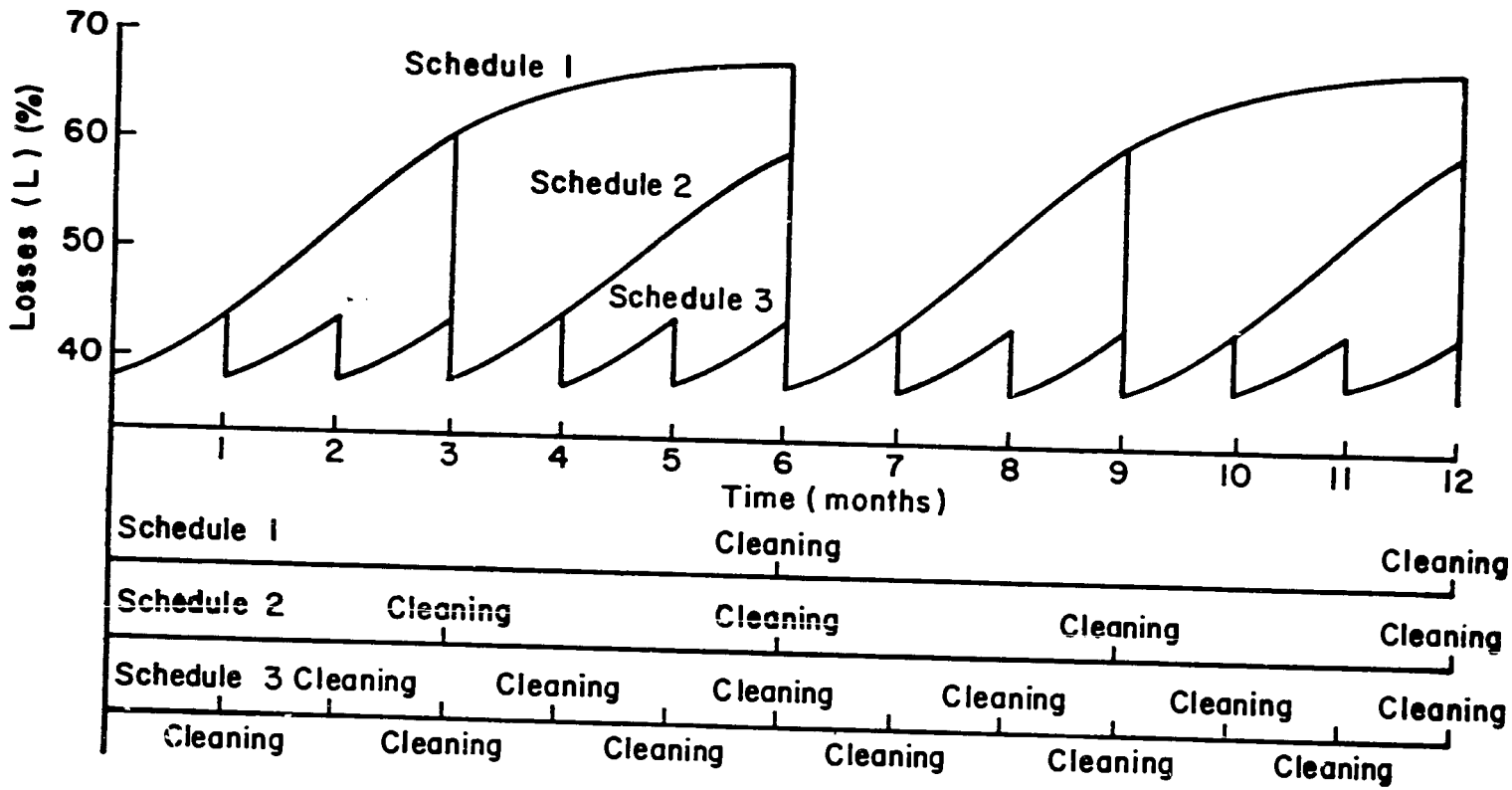


Figure 5.9 Percent conveyance losses over time on a hypothetical conveyance channel system under a semi-annual, quarterly and monthly cleaning schedule, based on a watercourse model with the roughness coefficient (n) fluctuating between 0.03 and 0.06 (from Trout, 1979a).

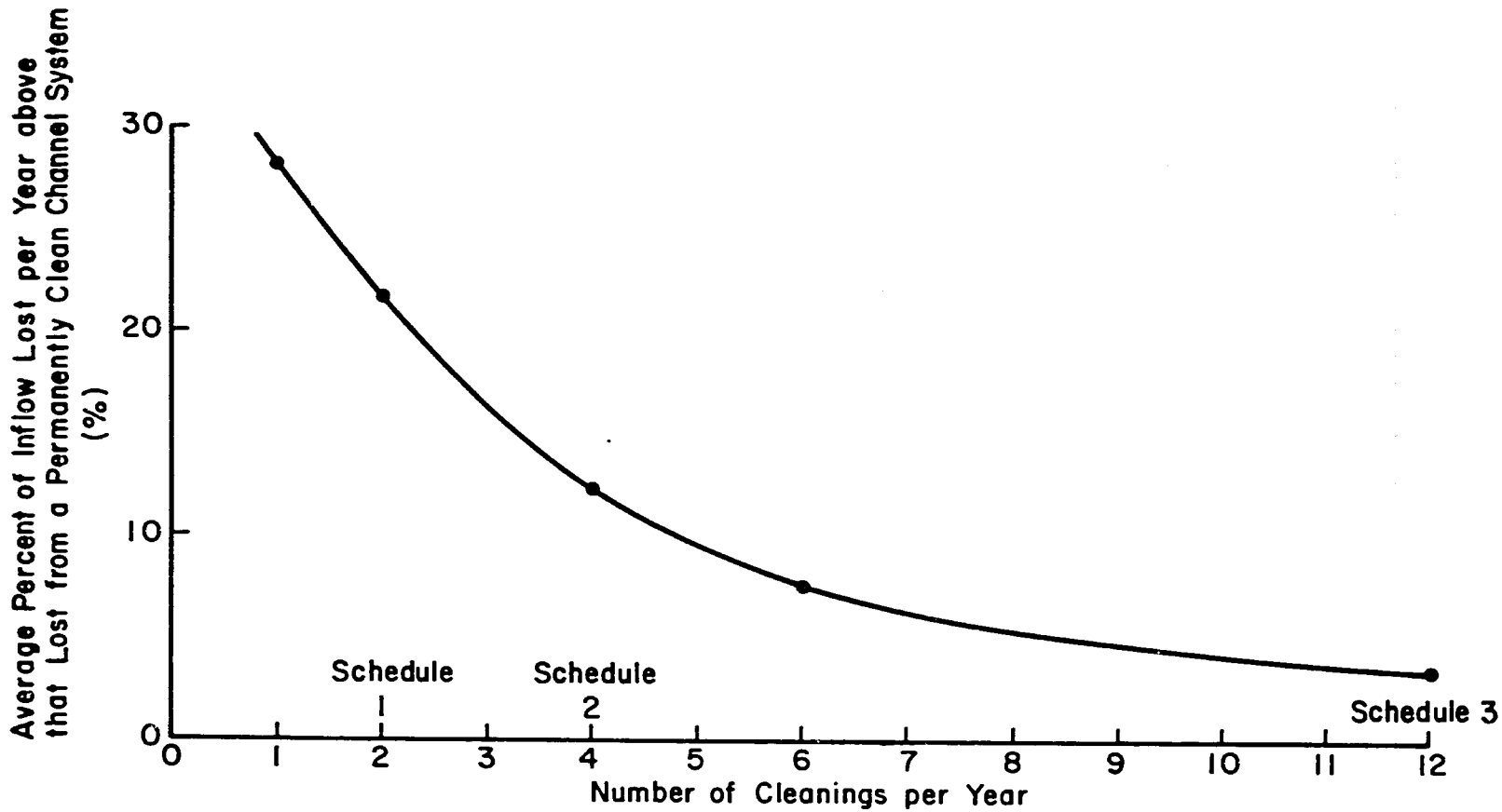
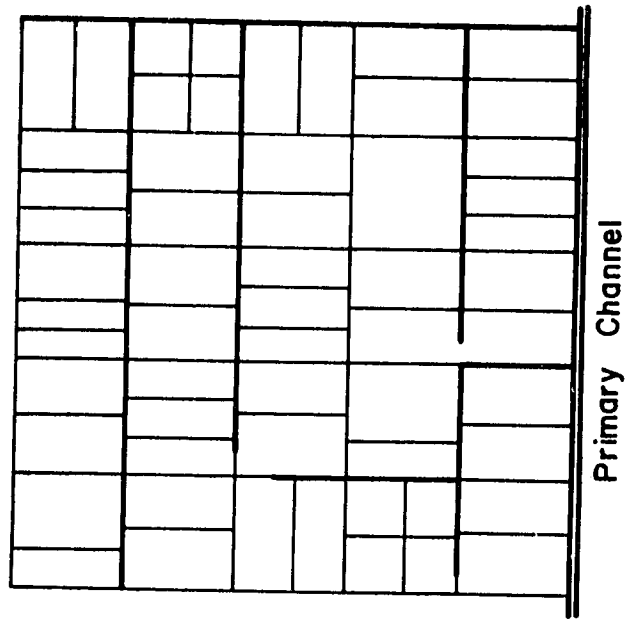
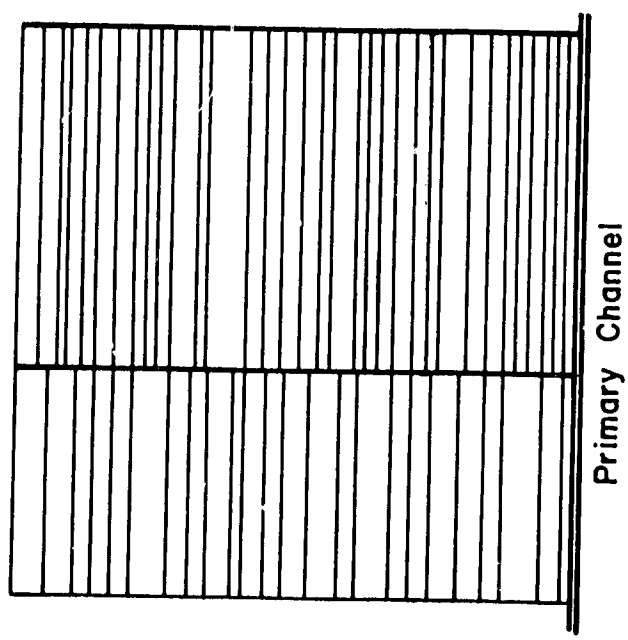


Figure 5.10. Excess conveyance water lost per year from a hypothetical channel system (the same as was used in Figure 5.9) versus the number of cleanings.



a) Present Layout on a 10 ha "Square" of Land Showing 1300m or 130m/ha of Field Channels.



b) Proposed Layout Showing 640m or 64m/ha of Field Channels.

Figure 5.11. An example of field and channel layout changes which result in a 50 percent reduction in field branch length.

Organizing the channel layout such that there are a minimum number of main channels will also maximize the percent of time that the average section of the main channel is used and lead to reduced seepage losses in the main channels, although the potential for such changes is often small. Figure 5.12 shows a hypothetical command area in which the layout changes decreased primary channel length and thus increased average main channel usage by about 50 percent while not increasing total channel length. Care must be taken in such reorganization that the average length of the field branches, which often exibit higher loss rates due to less usage, is not increased to the extent that savings in the primary channels are more than offset with increased losses in the branches.

The way water is rotated through a channel system can greatly affect the transient losses. If the rotation follows an orderly progression in which neighboring farmers follow each other and they in turn irrigate adjoining or nearby fields, the total length of channels filled and drained in one rotation interval, and thus, the transient losses, can be minimized. At the other extreme would be an allocation based purely on farmer demand in which the water might be randomly moved around the system from one distant branch to another. Although a demand system can, if properly managed, lead to increased field application efficiencies, these gains will often be more than offset by increased transient losses if farm and field sizes are small. The weekly "warabundi" turn rotation system in Pakistan which proportionately allocates water to successive farmers down each primary channel minimizes channel filling and transient losses.

5.4 Removing Bank Vegetation to Reduce Losses

The major type of transient loss is usually the water absorbed into dry channel wetted perimeters during initial filling of the channels. The intake rates of dry soils are generally higher than of wet soils. Wetted perimeter soils dry out both from surface evaporation and from the water taken up by the roots of plants growing on the banks of the channels. Consequently, channel bank vegetation can directly affect seepage rates and increase losses. The amount of losses attributable to these plants is difficult to estimate (Section 4.2.4). However, when this direct induced transient loss is added to the indirect effects of the vegetation of providing food and cover for bank inhabitants, decaying roots creating flow channels in the banks, and general interference with channel monitoring and maintenance operations to the farmers, the evidence is clear that trees and other vegetation should not be allowed to flourish on primary channel banks but should be controlled as much as is practical. If it is felt necessary to grow beneficial trees, bushes, or grasses on channel banks, they should be concentrated on less often used field branches and kept off primary channels.

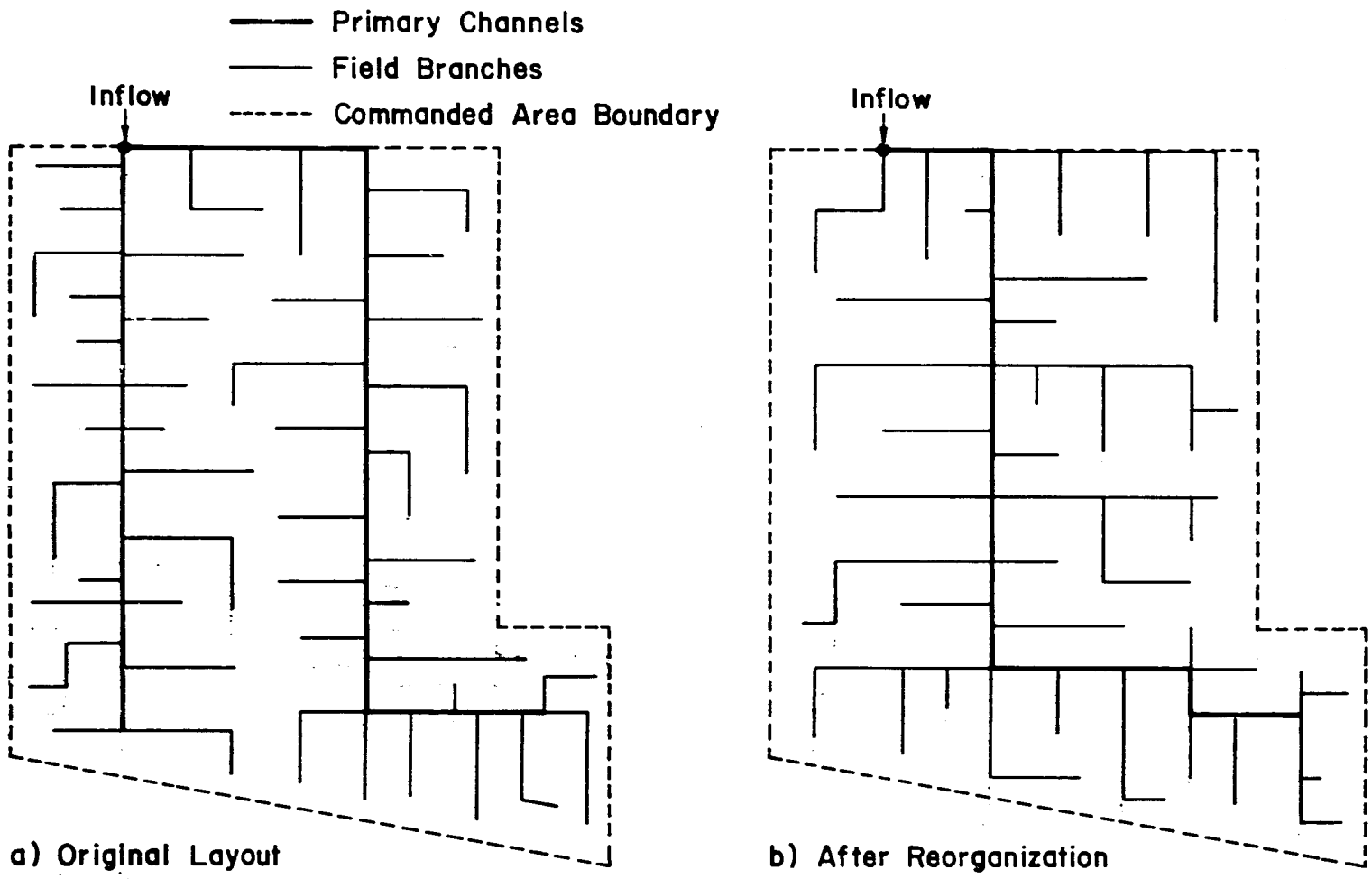


Figure 5.12. An example of reorganization of the channel network on a hypothetical command area such that primary channel length is reduced by 50 percent, and thus each section is filled an average of 50 percent more of the time.

5.5 Raising Channels to Reduce Dead Storage

A second type of transient loss, dead storage water which will not drain into fields but lies in channel bottoms until it evaporates or seeps away, can be eliminated by raising the channel bottom to the field level. However, this requires a higher channel with higher, thicker banks. Larger banks require more soil and are more costly to construct. The thicker banks also require more land area which usually must be taken from productive fields. Preliminary studies indicate that higher channels have significantly higher seepage rates. Leaks through or washouts of banks of higher channels lose more water. Erosion can also be a problem from higher outlets requiring more costly structures. Because of these detrimental effects of raising channels, they generally should not be constructed higher than is required to serve the fields with the desired working head (Section 6.2.3). In fact, in some cases losses may be reduced by constructing the channels lower in the ground and checking the water up to the required level to irrigate the fields.

Within this constraint, dead storage losses can be minimized by constructing channels with evenly sloped beds so all water drains to the tail, reorganizing channel layout to reduce the number of tails, rotating the water through the system to minimize the number of branches used and consequently drained, and allowing farmers with low fields at the tails to drain any remaining dead storage into their fields.

5.6 Permanent Structures to Reduce Junction and Outlet Losses

In some channel systems, water lost from and in the vicinity of junctions and outlets is significant. This loss can take the form of leakage through structures, washouts of outlet dikes or gates, or leakage and seepage in the vicinity of junctions where banks are thin and freeboards small due to soil borrowing or traffic. Leakage through earthen dikes can be significant in heavy clay soils which shrink and crack upon drying. Washouts of earthen dikes will especially be a problem in lighter sandy soils which have little cohesion. Junction deterioration will be a problem whenever earthen junctions are not maintained well.

If outlet and junction losses are a problem, good quality permanent structures can eliminate most of these losses and prevent channel deterioration. Good quality implies both durability and low leakage. Poorly made structures will often leak more than temporary earthen dikes, and their leakage is more difficult to stop. Section 7 describes several types of outlet structures for small channels.

Even if permanent structures cannot be justified, based on water losses, they will improve water control, reduce labor

required for irrigation, and can form the basis for motivating farmers to support and participate in a channel renovation program.

5.7 Summary

Several design techniques and operating procedures have been described which can affect specific problems which cause losses in small earthen conveyance systems. If diagnosis of a channel system shows that these problems exist, the suggested techniques should be tested, evaluated and adapted before an extensive channel improvement program is undertaken. Listed in Table 5.1 are a summary of potential loss-reducing techniques and the types of problems or losses they affect. Specific local conditions and problems will lead to additional loss-reducing techniques beyond those listed here.

Table 5.1. Summary of design techniques and operating procedures which can reduce water loss and the problems or losses they affect.

Loss Reducing Technique	Affected Problem
1. bank compaction	a) secondary porosity-induced excess seepage b) insect, animal, and plant pore formation
2. core compaction	a) secondary porosity-induced excess seepage b) insect-, animal- and plant-caused pore-induced excess seepage c) insect, animal, and plant pore formation
3. managed sediment deposition	a) seepage
4. channel operation changes to maximize usage time of main channels	a) seepage (via microbiological seals) b) insect, animal, and plant pore formation
5. channel operation changes to minimize the number of channels filled per rotation	a) transient losses
6. field branch and field layout changes to reduce the number of channels	a) transient losses
7. maintaining banks clean of vegetation and trees	a) initially high seepage transient losses b) insect, animal, and plant pore formation
8. cleaning vegetation from the wetted perimeter to maintain low roughness	a) excess seepage into upper banks b) insect, animal, and plant pore formation below the operating level
9. maintaining constant inflows or redesigning for increased inflows	a) excess seepage into upper banks b) insect, animal, and plant pore formation below the operating level
10. continuing renovation at some degree to the field	a) excess seepage into upper banks in field branches
11. permanent structure installation	a) leakage through outlets b) channel deterioration in junction areas

SECTION 6

IRRIGATION CHANNEL DESIGN

6.1 Open Channel Design: Manning's Equation

The flow of water in channels is controlled primarily by two opposing forces: gravity, which pulls the water downhill and friction, which resists the flow. Since the pull of gravity is essentially constant, the force exerted on the water depends upon the angle of the flow with respect to the downward gravity force or the slope of the channel. The friction which resists the flow is primarily a result of the "rubbing" of the flowing water against the bed and banks of the channel. The amount of resistance depends upon the degree of roughness (i.e., the length of vegetation, the size of clods or rocks in the bed, and the nonuniformity of the cross section), the area of contact (wetted perimeter length), and the velocity of the flowing water.

When the flow is steady and uniform, these two opposing forces--the driving and resisting forces--are equal and can be balanced, which leads to the Chezy equation:

$$V = C\sqrt{RS} \tag{9}$$

where: V = the flow velocity (m/sec),

C = a coefficient of roughness,

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

S = the slope of the water surface (m/m),

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

WP = the wetted perimeter length (m).

Manning derived an empirical equation adapting the Chezy equation for open channel flow in which the roughness coefficient, C, is equal to $R^{1/6}/n$:

$$V = (1/n)R^{2/3}\sqrt{S}\ , \tag{10}$$

where n is the Manning's roughness coefficient which has the dimensions of sec-$m^{-1/3}$. Manning's equation is the most commonly used design equation for steady, uniform open channel flow, and is the equation which will be used in this manual.

Since the flow rate, Q (m^3/sec), is equal to the average velocity times the cross-sectional area, Manning's equation can be written in terms of the flow rate:

$$Q = (1/n)AR^{2/3}\sqrt{S}. \tag{11}$$

While designing channels, the problem usually is to design the shape and size of the channel for a desired flow rate, an available slope, and the design roughness coefficient. Manning's equation is most conveniently used if these two sets of factors are separated out:

$$AR^{2/3} = Qn/\sqrt{S} \tag{12}$$

The problem will be to determine the cross section which will give an $AR^{2/3}$, termed the hydraulic section, equal to the desired factors on the right-hand side of Equation 12.

The hydraulic section is a complex expression for most cross-sectional shapes. For example, for the trapezoidal shape

$$A = BD + ZD^2, \text{ and} \tag{13}$$

$$WP = B + 2D\sqrt{Z^2 + 1}, \text{ so} \tag{14}$$

$$AR^{2/3} = A^{5/3}/WP^{2/3} = \frac{\{BD + ZD^2\}^{5/3}}{\{B+2D\sqrt{Z^2+1}\}^{2/3}}, \tag{15}$$

where: B = bottom width (m),

D = flow depth (m), and

Z = the side slope (horizontal/vertical).

These geometric parameters are shown in Figure 6.1. Note that the side slope value is the inverse of a slope in the usual sense.

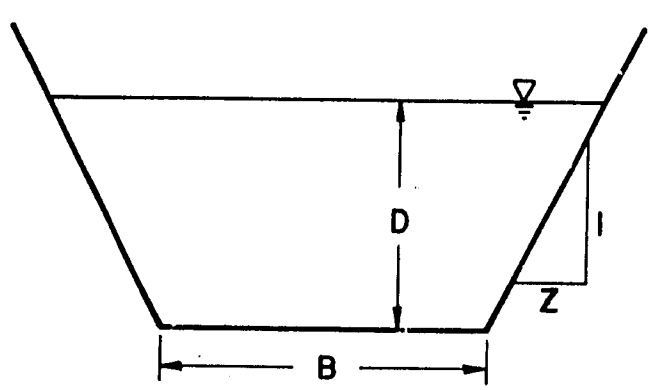


Figure 6.1. The trapezoidal channel cross section showing the geometric parameters used in Eqs. 13 to 15.

A solution for this equation for the three variables B, D, and Z, is possible only by trial and error, and is quite time-consuming. Consequently graphs or nomographs are used to assist with the design process.

The trapezoidal shape shown in Figure 6.1 is the most common shape for both lined and unlined small channels, and

so is the shape which will be used. It is fairly efficient (has a large capacity for a given wetted perimeter length), is easy to design and build since a flat bottom and two straight sides are involved, and is adaptable to materials with various stabilities. The three design parameters required are defined in the figure. The rectangular shape is simply a trapezoidal channel with vertical sides (Z = 0).

The design procedure will generally involve calculating the required hydraulic section, $AR^{2/3}$, from the flow rate, Q, slope, S, and roughness coefficient, n, according to the right side of Eq. 12. Then values for two of the three design parameters are chosen and the third value is determined from a solution graph. The side slope, Z, is usually constant for a given soil or lining material and so is usually one of the predetermined values. Commonly, the bottom width, B, is held fairly constant for a given channel or is varied in increments, and the depth, D, is allowed to fluctuate to adapt to changing slope, flow rate, or roughness values. Consequently, most of the solution graphs are designed to determine D for chosen values of B and Z, and given values of S, A and n.

Four sets of solution graphs are given. The first three assume that Z and B are given and the depth is to be determined. The fourth requires that only Z is chosen and both B and D can vary. This set will be most useful for lined channels where material costs are high but can be minimized with the proper selection of B and D. The selection of which of the first three sets should be used will depend upon the computational ability of the designer. As the computational ability decreases, increasingly more graphs are required to cover the desired solutions.

6.1.1 Graphical solution number 1: dimensionless method

The first solution graph, Figure 6.2, defines the relationship between two dimensionless quantities: D/B and $(Qn/\sqrt{S})/B^{8/3}$. One graph (printed on semilog paper to expand the abscissa scale) is sufficient for all small channel designs. Side slopes other than those given in Figure 6.2 must be interpolated between the curves. Use of the graph requires that the hydraulic section, $AR^{2/3}=Qn/\sqrt{S}$, divided by a chosen bottom width to the eight-thirds power, be calculated. The required depth-to-bottom width ratio, D/B, is then found using the curve for the chosen side slope value, Z. The design depth is then simply this ratio times the initially chosen bottom width (i.e. (D/B)xB).

An alternative use of the graph would be to choose a D/B ratio and determine the required $(Qn/\sqrt{S})/B^{8/3}$ value for the proper side slope. The bottom width which is required would then be:

$$B^{8/3} = (Qn/\sqrt{S})/[(Qn/\sqrt{S})/B^{8/3}], \text{ or} \qquad (16)$$

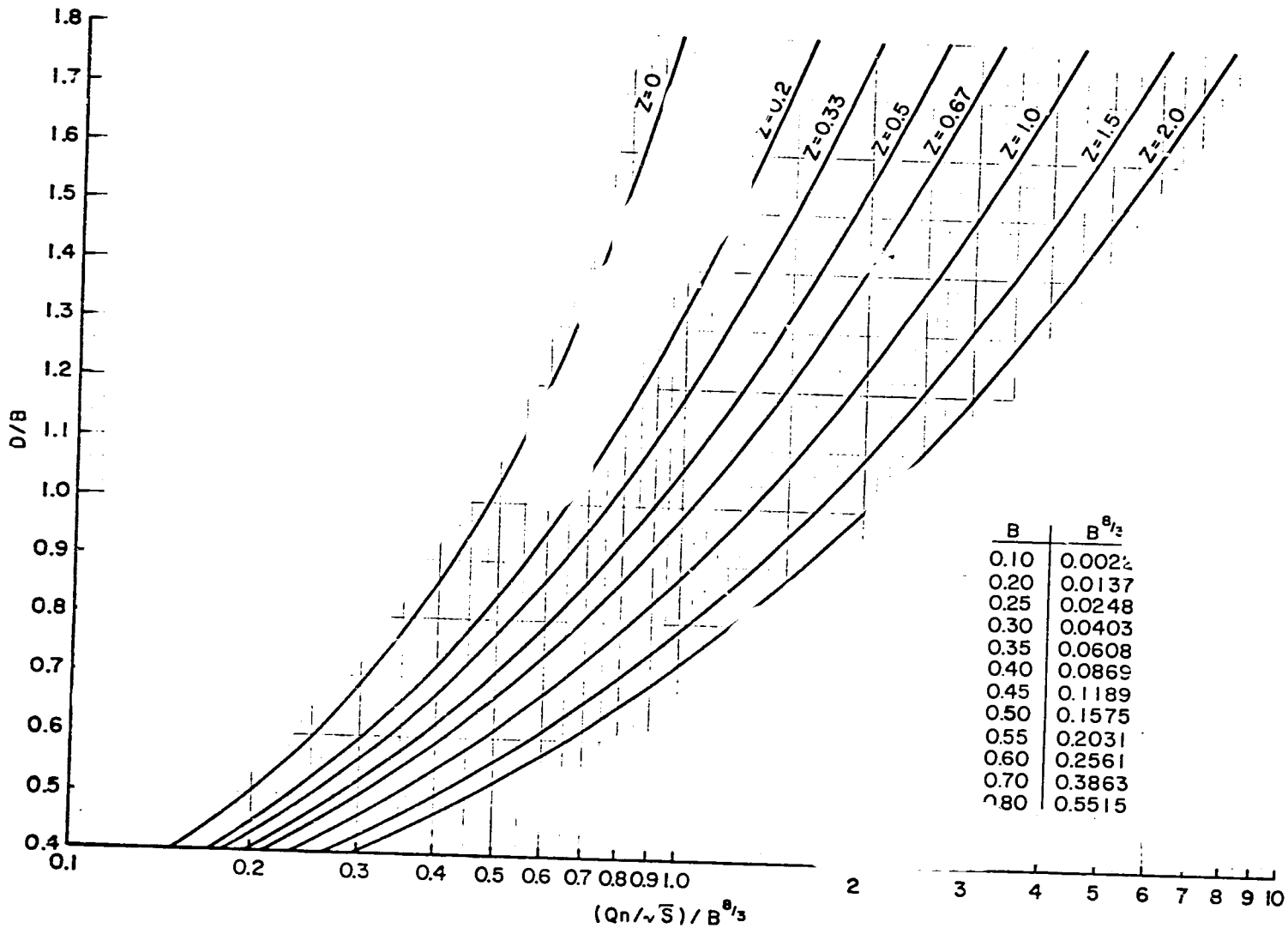


Figure 6.2. Manning's equation solution graph #1: Dimensionless Method

$$B = \{(Qn/\sqrt{S})/[(Qn/S)/B^{8/3}]\}^{3/8} \tag{17}$$

where the value in the square brackets, [], is taken from the graph.

The method can be easily carried out if a calculator with a power function is available; or, if only the square root function is available, the $B^{8/3}$ value can be picked off of the table provided in Figure 6.2.

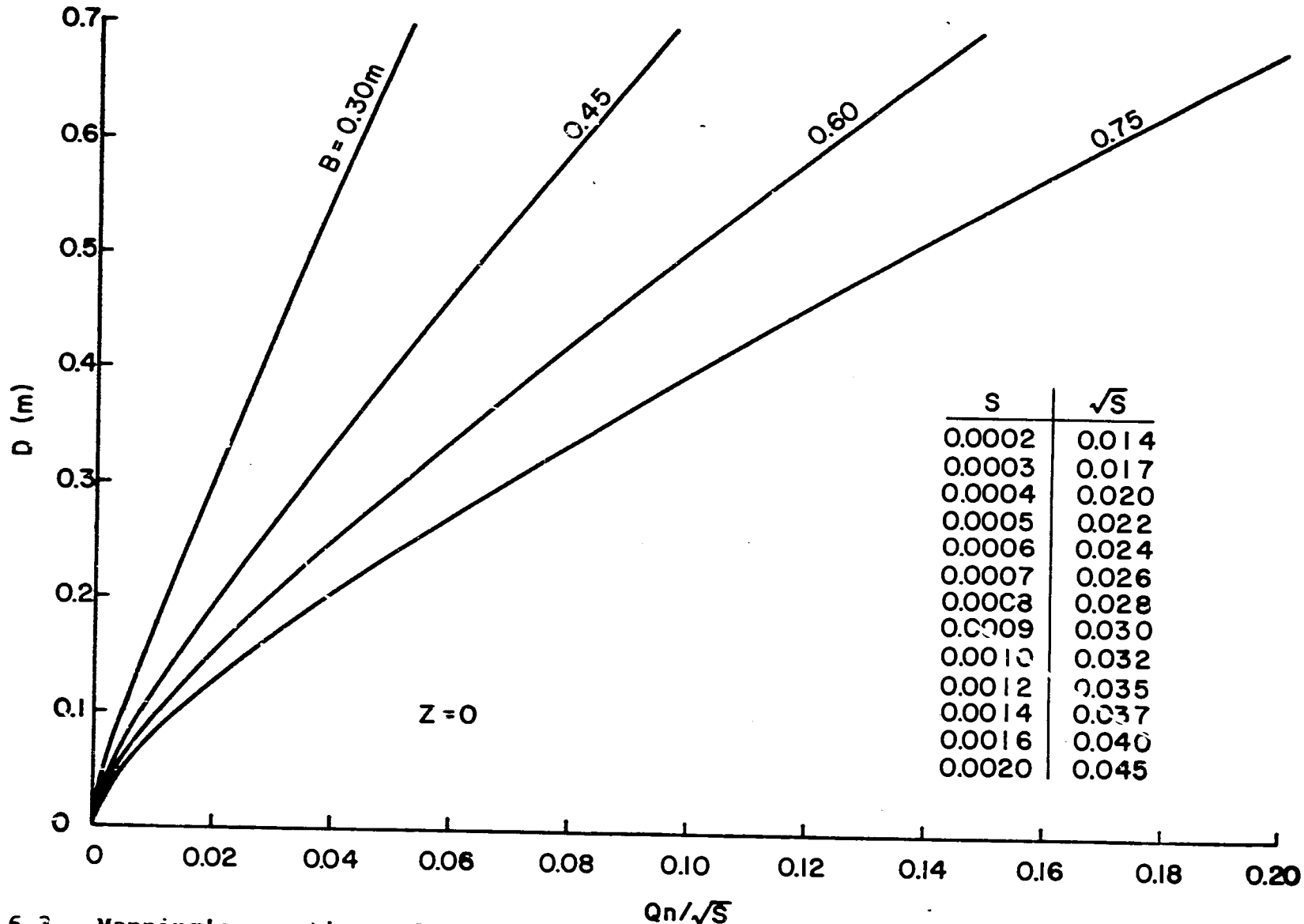
6.1.2 Graphical solution number 2: depth versus hydraulic section

The second graphical solution method shown in Figures 6.3 to 6.5 involves a separate graph for each side slope value and a separate curve for each bottom width value. In this way, only the hydraulic section must be calculated, and the design depth is determined from the proper graph and curve. Since only one or two side slope values will probably be used on a given project, this technique can be very practical if a hand calculator with a square root function is available. Slope square root values can also be taken from the table on Figure 6.3. Solution graphs for side slope values of Z = 0, 1.0, and 1.5 are given in Figures 6.3, 6.4, and 6.5 respectively.

6.1.3 Graphical solution number 3: no calculation method

The third solution technique given in Figures 6.6 to 6.11 requires no calculations on the part of the designer, but requires a new graph for each combination of bottom width, side slope, and roughness coefficient. This simple method will be practical if the requirements are fairly uniform. For example, if all channels are designed with 1:1 (Z = 1) side slopes, and roughness coefficients of 0.04, and only three bottom width values are used, only three graphs and no calculations are required. Solution graphs for n = 0.04, Z = 1.0 and 1.5, and B = 0.30, 0.45, and 0.60 m are provided.

The solution technique involves selecting the graph with the proper B, Z, and n value, and determining the design depth from the curve at the proper flow rate, Q, and the required slope, S. The proper depth curve is the one nearest where a vertical line from the proper slope value and a horizontal line from the design flow rate value cross. Curves are given for each 2 cm depth increment. Intermediate depths can be interpolated between existing curves, although using the nearest curve will result in no more than a 1 cm error in depth which is insignificant. An advantage of the method is that the average flow velocity values, V, for the designed cross section can also be determined directly from the graph.



S	√S
0.0002	0.014
0.0003	0.017
0.0004	0.020
0.0005	0.022
0.0006	0.024
0.0007	0.026
0.0008	0.028
0.0009	0.030
0.0010	0.032
0.0012	0.035
0.0014	0.037
0.0016	0.040
0.0020	0.045

Figure 6.3. Manning's equation solution graph #2-a: depth as a function of hydraulic section for Z = 0.

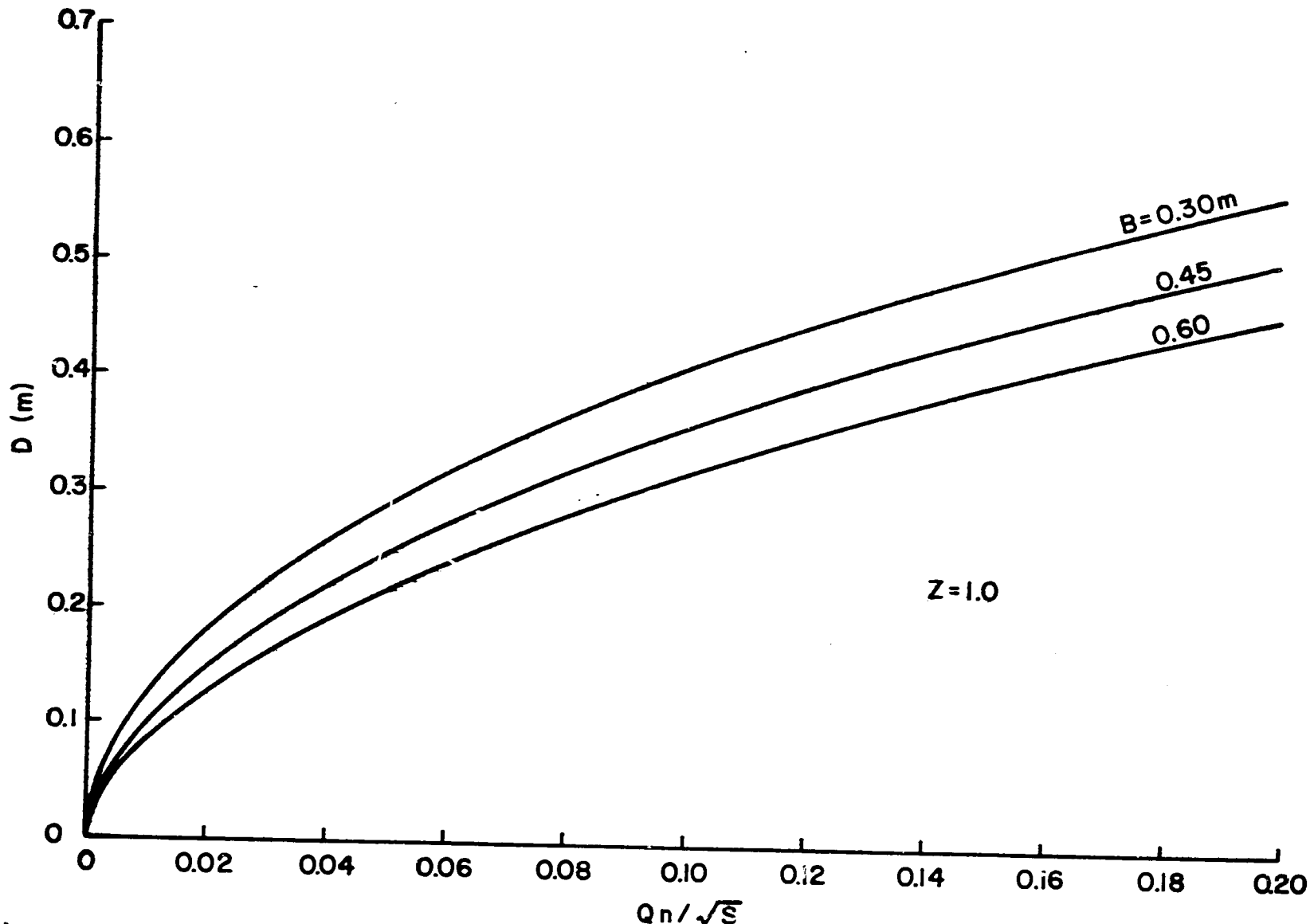


Figure 6.4. Manning's equation solution graph #2-b: depth as a function of hydraulic section for Z = 1.0.

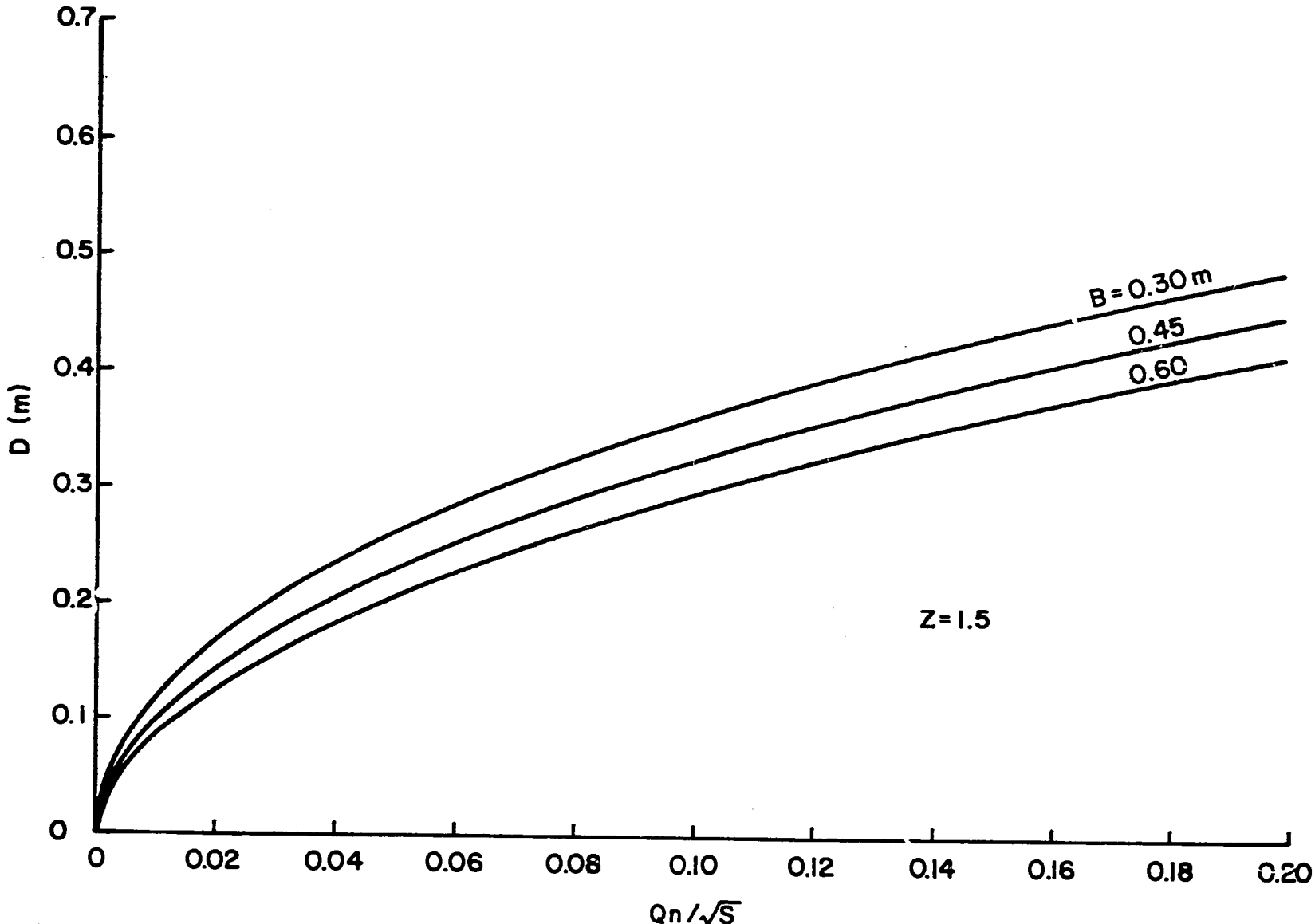


Figure 6.5. Manning's equation solution graph #2-c: depth as a function of hydraulic section for Z = 1.5.

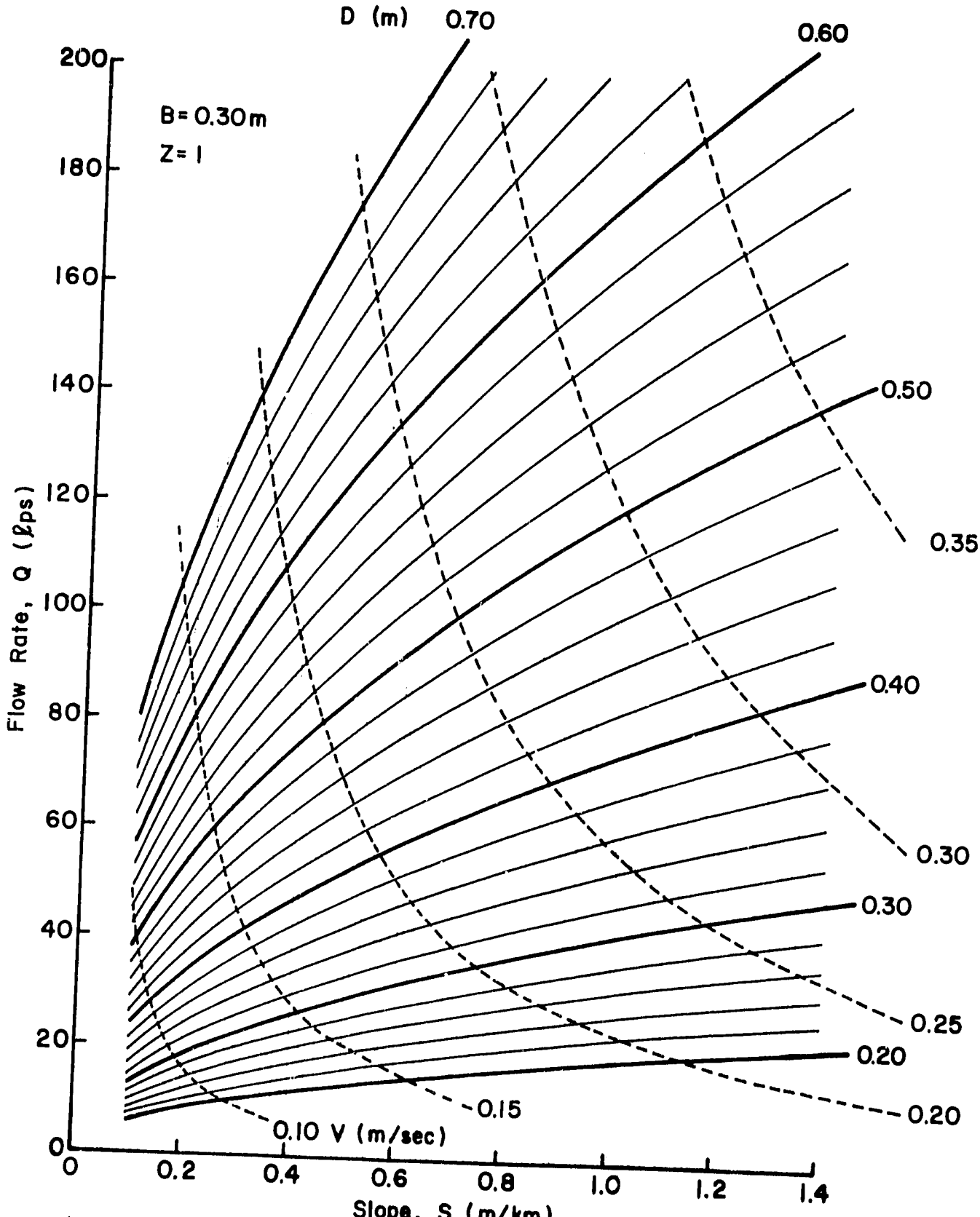


Figure 6.6. Manning's equation solution graph #3-a: no calculation method for n = 0.04, B = 0.30 m, and Z = 1.0.

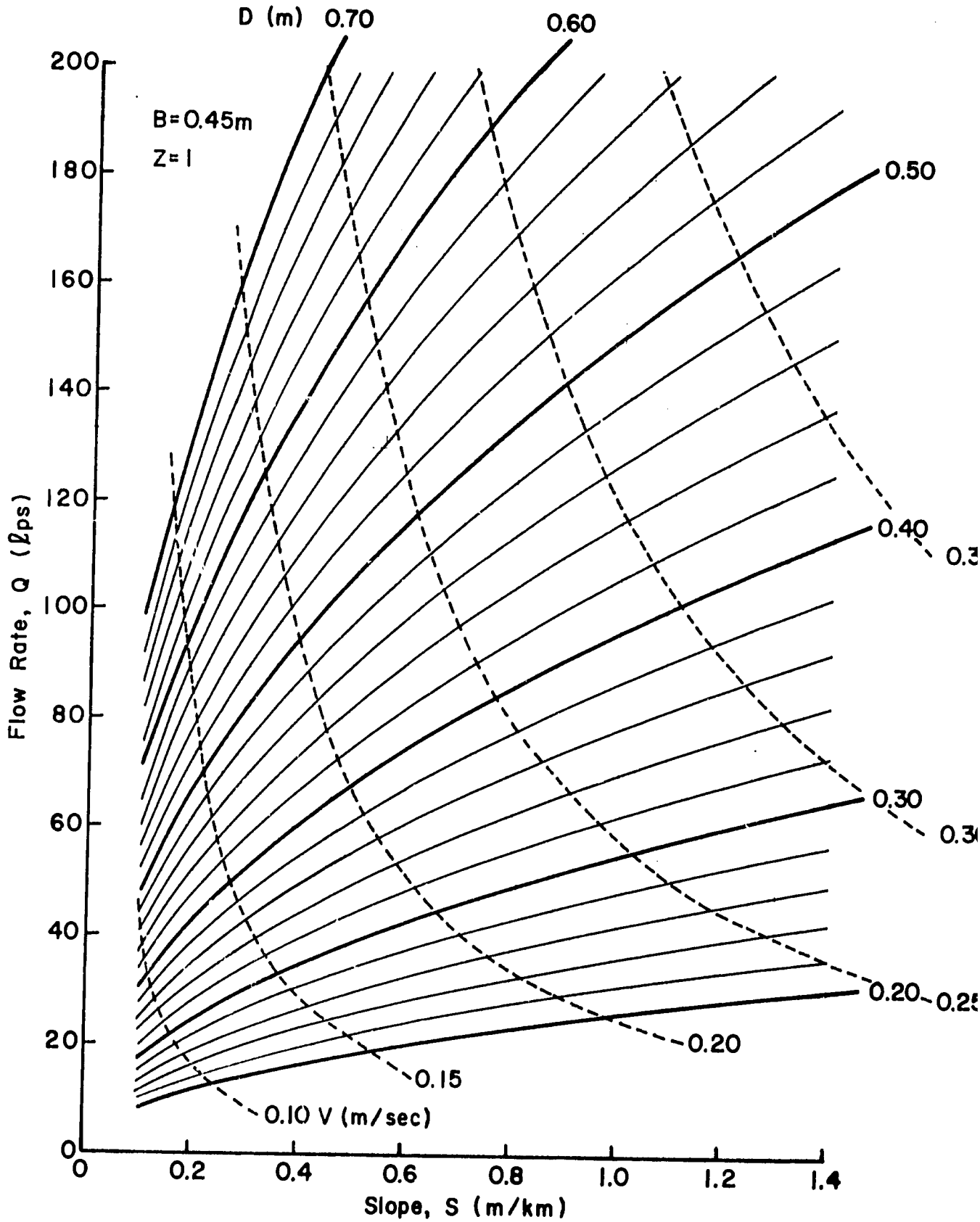


Figure 6.7. Manning's equation solution graph #3-b: no calculation method for n = 0.04, B = 0.45 m, and Z = 1.0.

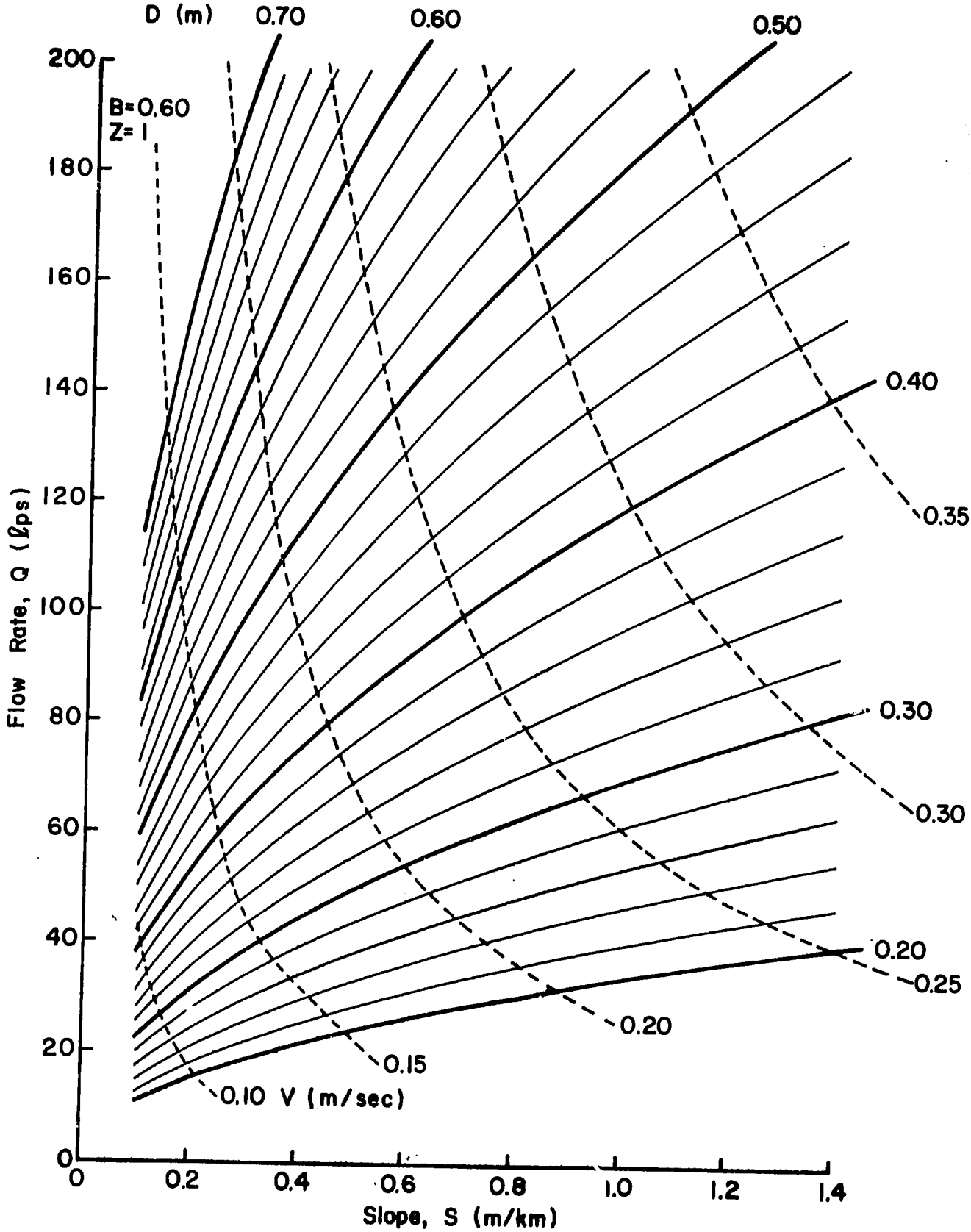


Figure 6.8. Manning's equation solution graph #3-c: no calculation method for n = 0.04, B = 0.60 m, and Z = 1.0.

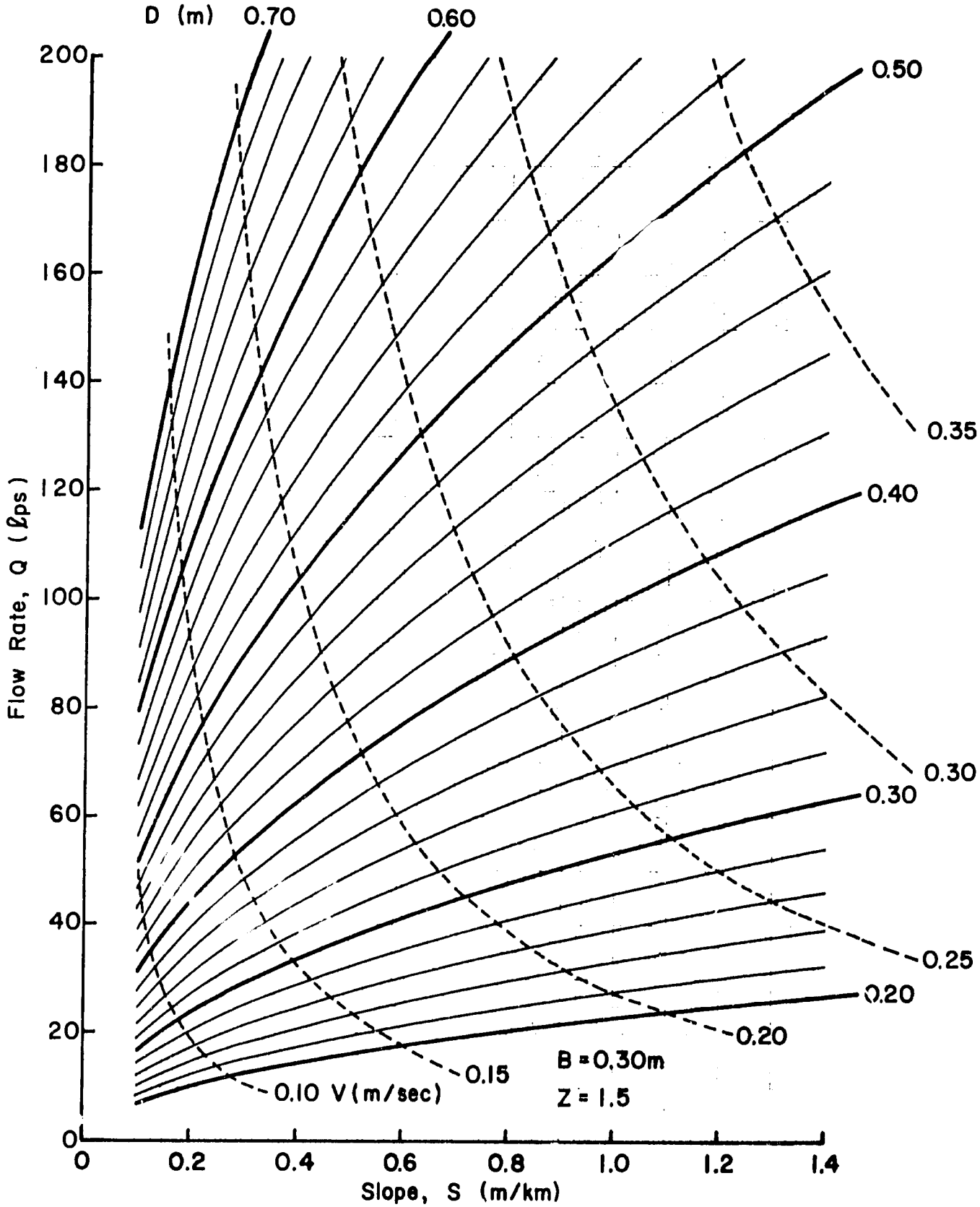


Figure 6.9. Manning's equation solution graph #3-d: no calculation method for n = 0.04, B = 0.30 m, and Z = 1.5.

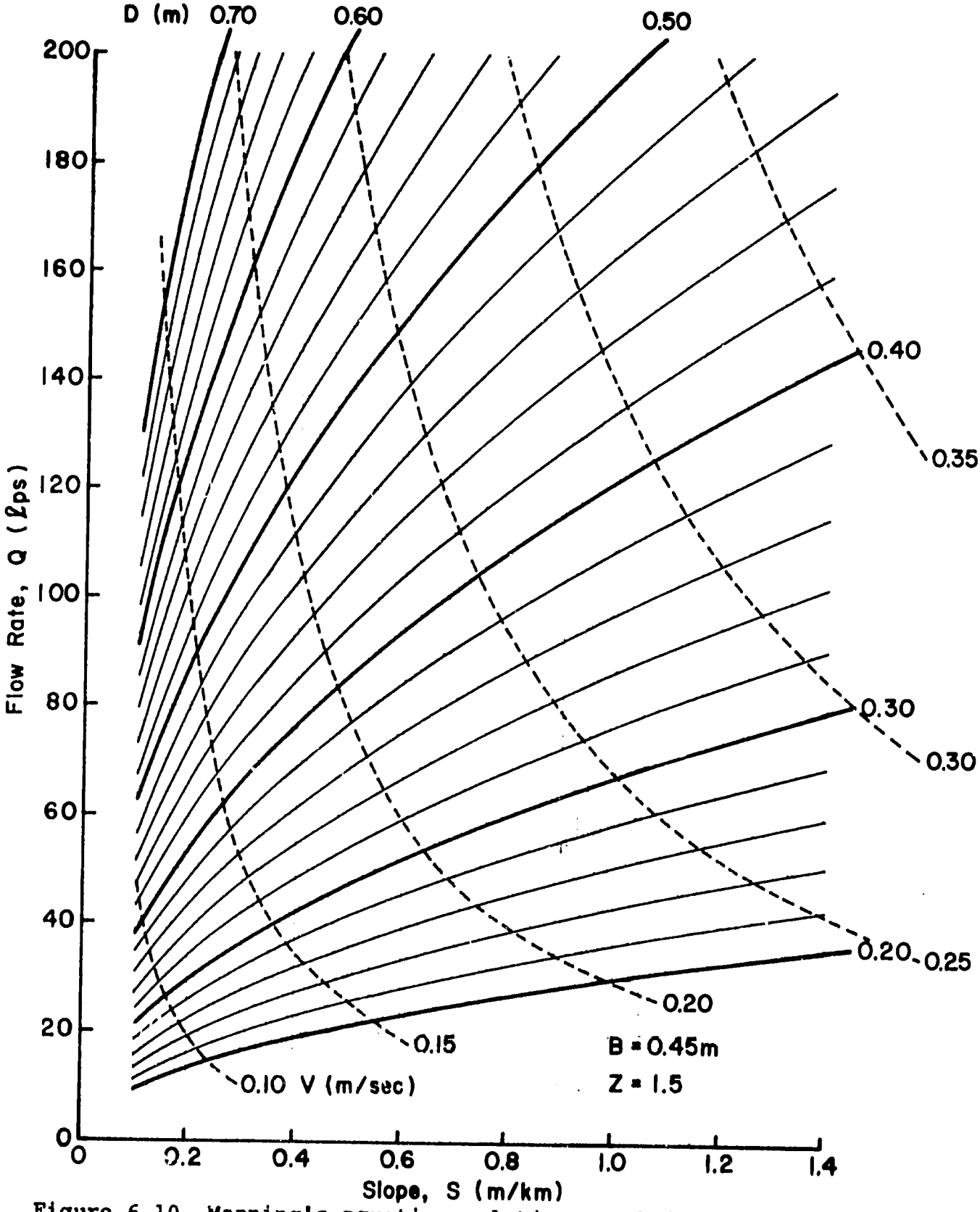


Figure 6.10. Manning's equation solution graph #3-e: no calculation method for n = 0.04, B = 0.45 m, and Z = 1.5.

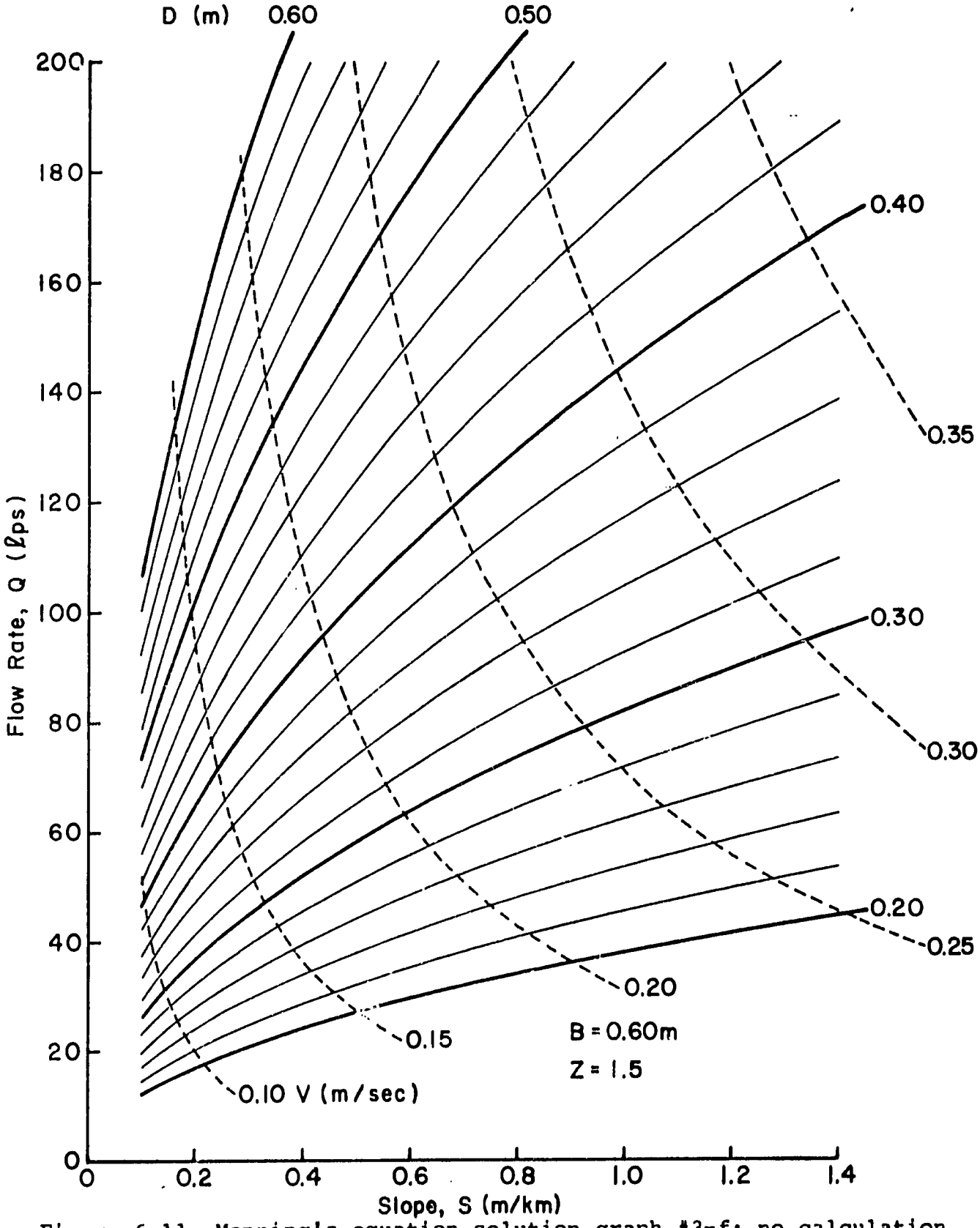


Figure 6.11. Manning's equation solution graph #3-f: no calculation method for n = 0.04, B = 0.60 m, and Z = 1.5.

6.1.4 Graphical solution number 4: minimum cost method

The fourth solution technique given in Figures 6.12 to 6.16 is similar to the second set, but allows any combination of depth and bottom width to be selected. First, the $Qn/\sqrt{S}$ value and associated curve (nearest or interpolated) for the calculated hydraulic section ($AR^{2/3} = Qn/\sqrt{S}$) is located on the graph for the chosen Z value. Any combination of B and D values which fall on this curve is a solution to Manning's equation. Although these graphs are a little more difficult to read than those of the second solution method, the lines radiating from the origin also allow a minimization of costs for lined channels. Cost minimization will be explained more fully in Section 8.3 which discusses channel lining, and in Appendix 8. It should be noted that the b/d cost ratio line radiating from the origin with a value of 1.0 will give the D/B ratio for a channel with the minimum wetted perimeter length and consequently the least area for seepage. Solution graphs for Z = 0.0, 0.33, 0.50, 0.67, and 1.0 are given.

6.1.5 Usage of the solution graphs

An example will be used to clarify the use of each of the graphical solution techniques. Assume a channel with a flow rate of 50 lps (0.050 m^3/sec) is to be rebuilt. The average available slope in the section is 0.0006, and the design roughness coefficient is 0.04. The soil of the area has been found from practical experience to be stable up to a side slope of 1:1 (Z = 1). Forms are being used to aid in construction which have bottom widths of 0.30, 0.45, and 0.60 m.

Method #1. The D/B values are determined by locating the proper $(Qn/\sqrt{S})/B^{8/3}$ value on the abscissa of Figure 6.2, following this value vertically to the curve for Z = 1.0, and moving from the intersection with the curve left to read D/B from the ordinate. This procedure is shown in Figure 6.17.

$$\text{Hydraulic Section, } AR^{2/3} = Qn/\sqrt{S} = \frac{(0.05\ m^3/sec)(0.04)}{\sqrt{0.0006}} = 0.082.$$

Then for Z = 1.0 and B = 0.30: $(Qn/\sqrt{S})/B^{8/3} = 2.02$;

D/B = 1.24, and D=0.37 m;

for Z = 1.0 and B = 0.45: $(Qn/\sqrt{S})/B^{8/3} = 0.69$;

D/B = 0.72, and D=0.33 m; and

for Z = 1.0 and B = 0.60: $(Qn/\sqrt{S})/B^{8/3} = 0.32$;

D/B = 0.48, and D=0.29 m.

The first two alternatives (B = 0.30 m and D = 0.37 m, or B = 0.45 m and D = 0.32 m) will generally be the better solutions

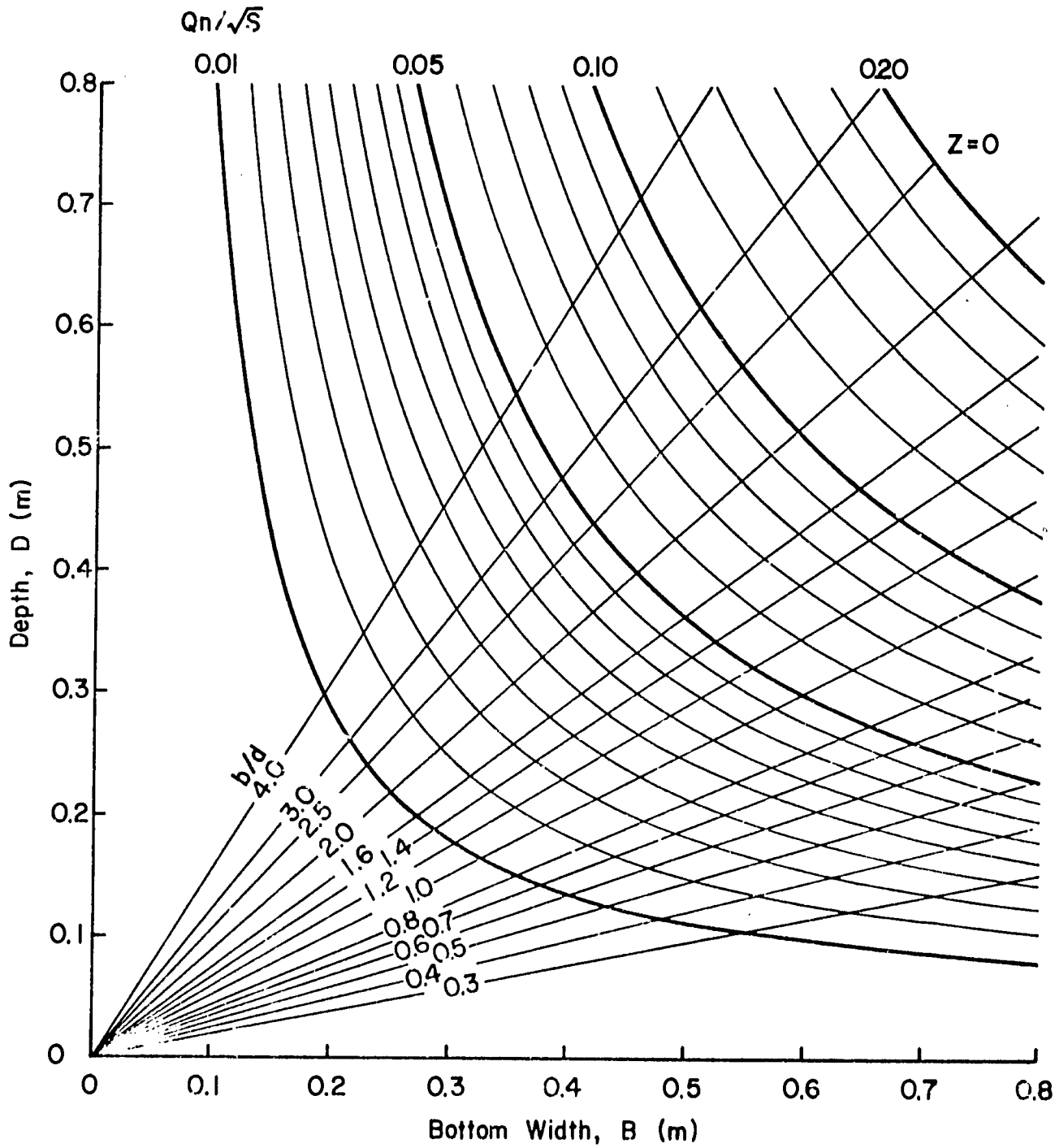


Figure 6.12. Manning's equation solution graph #4-a: cost minimization method for Z = 0.

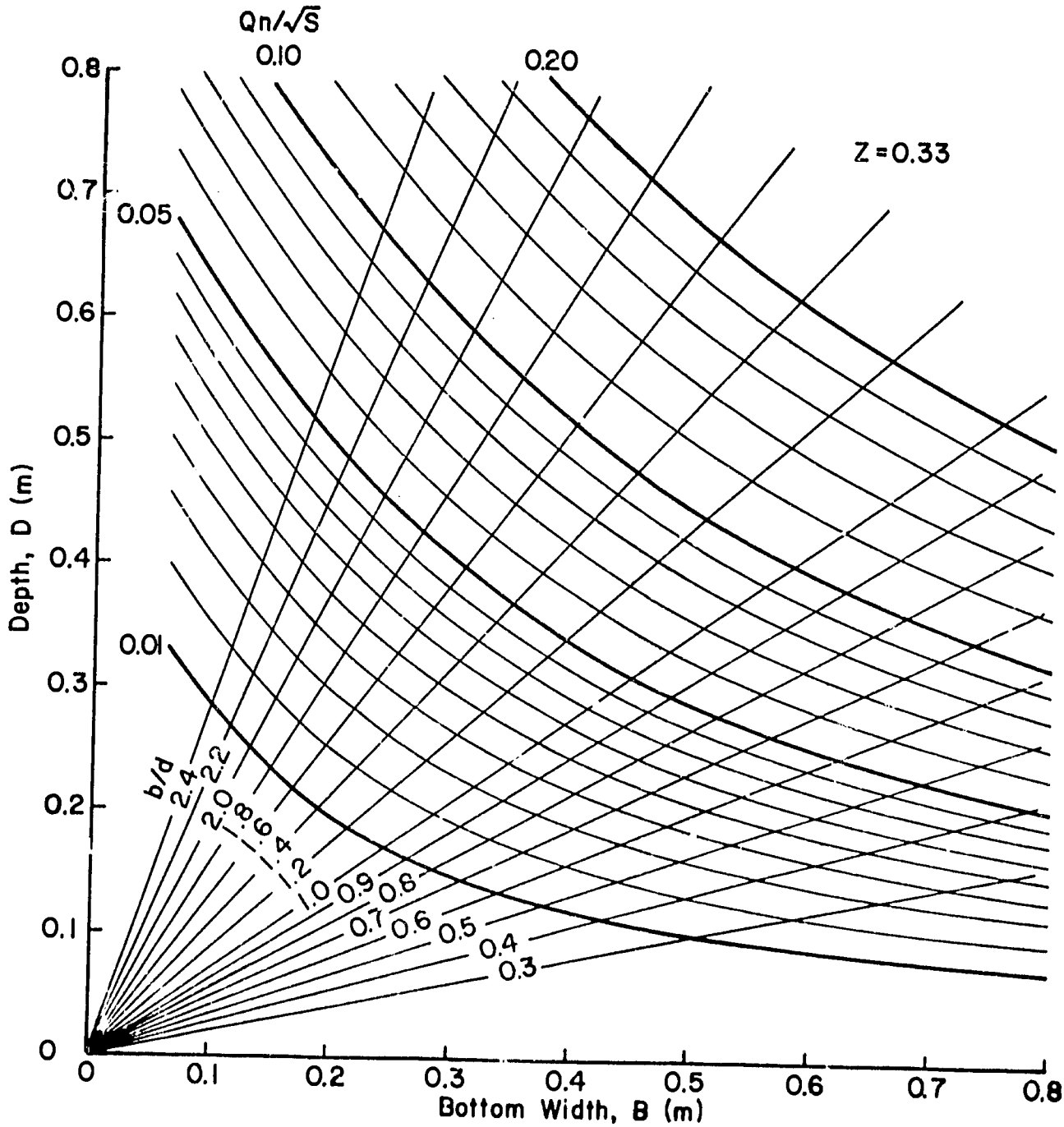


Figure 6.13. Manning's equation solution graph #4-b: cost minimization method for Z = 0.33.

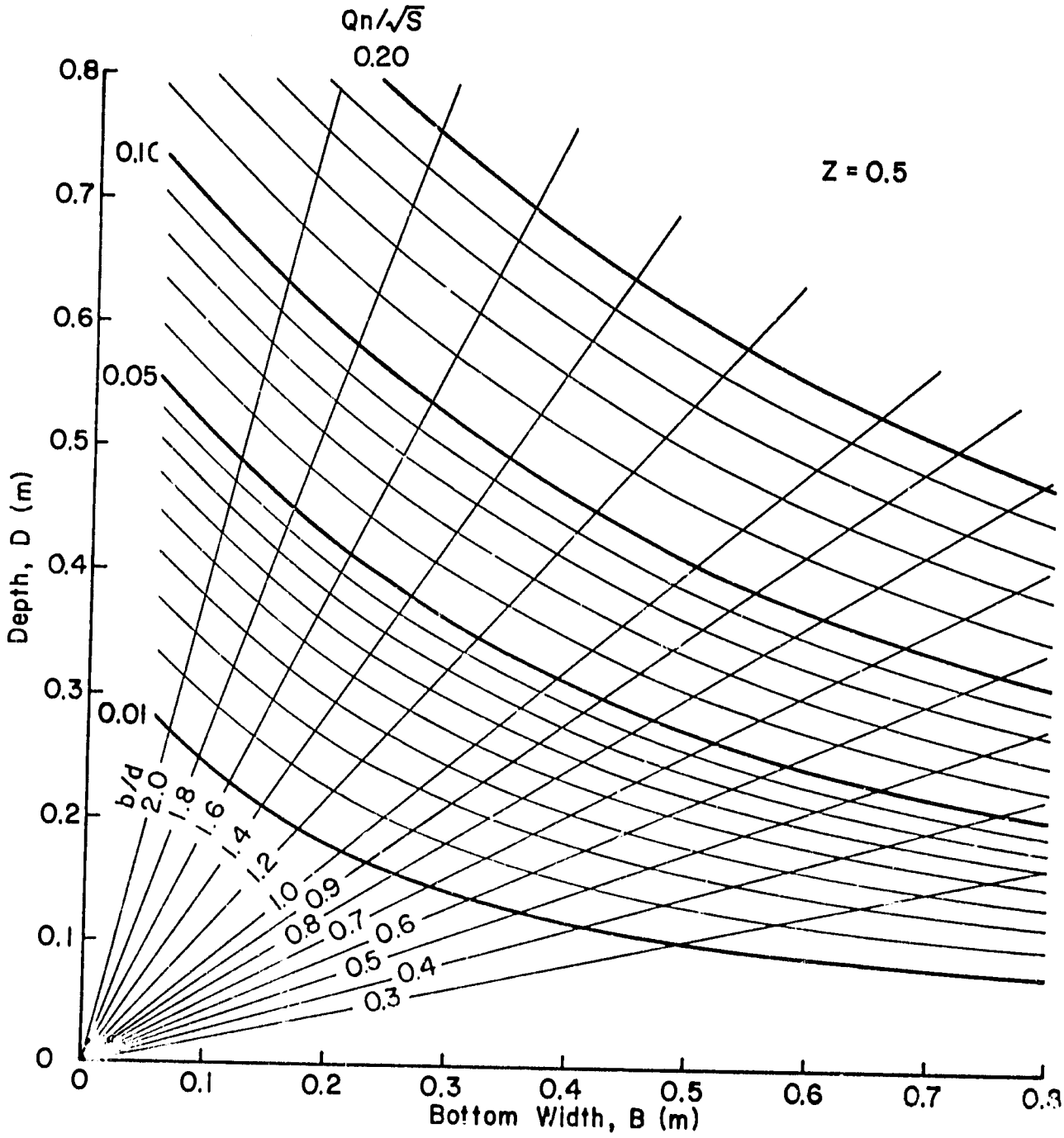


Figure 6.14. Manning's equation solution graph #4-c: cost minimization method for Z = 0.50.

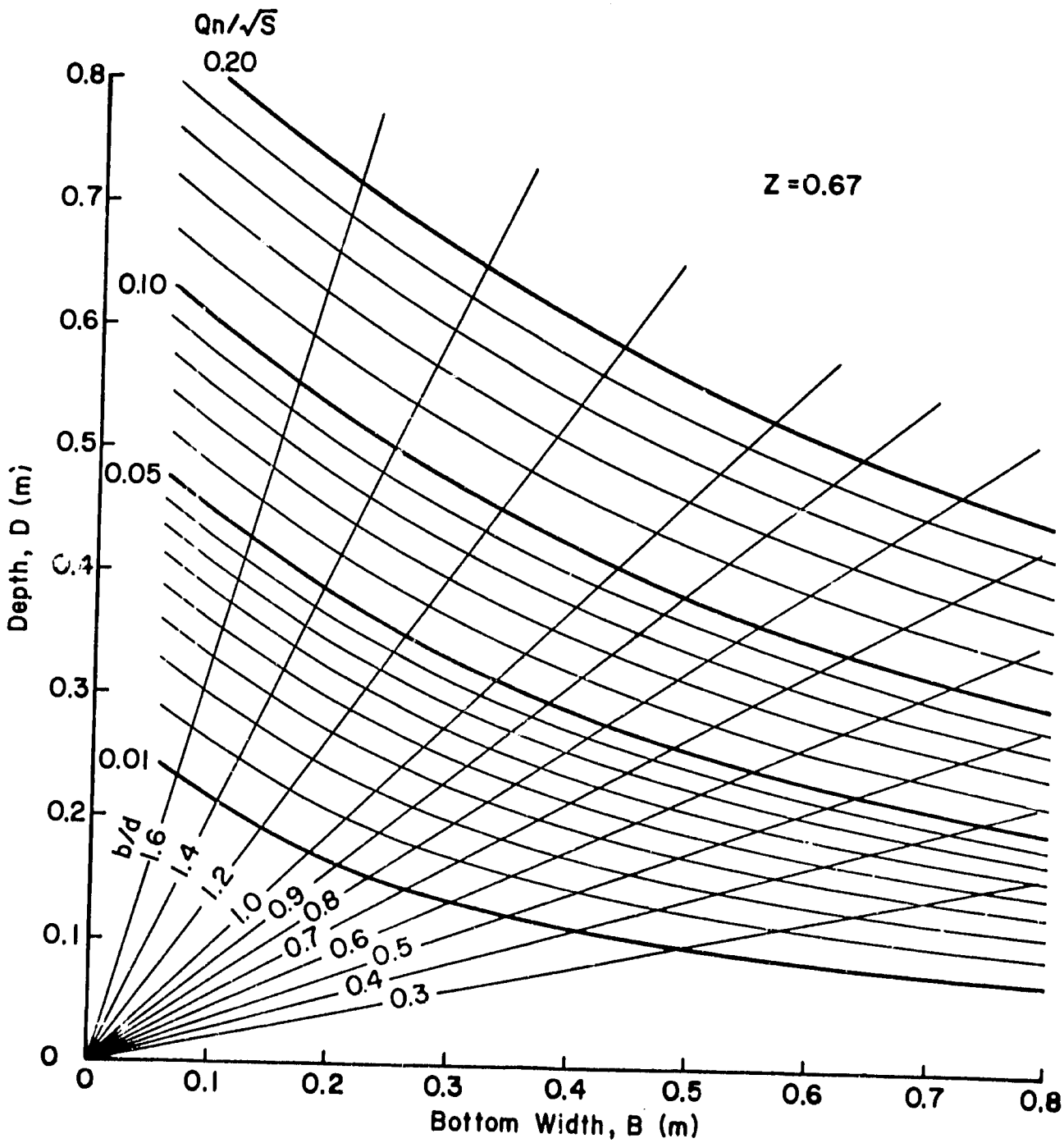


Figure 6.15. Manning's equation solution graph #4-d: cost minimization method for Z = 0.67.

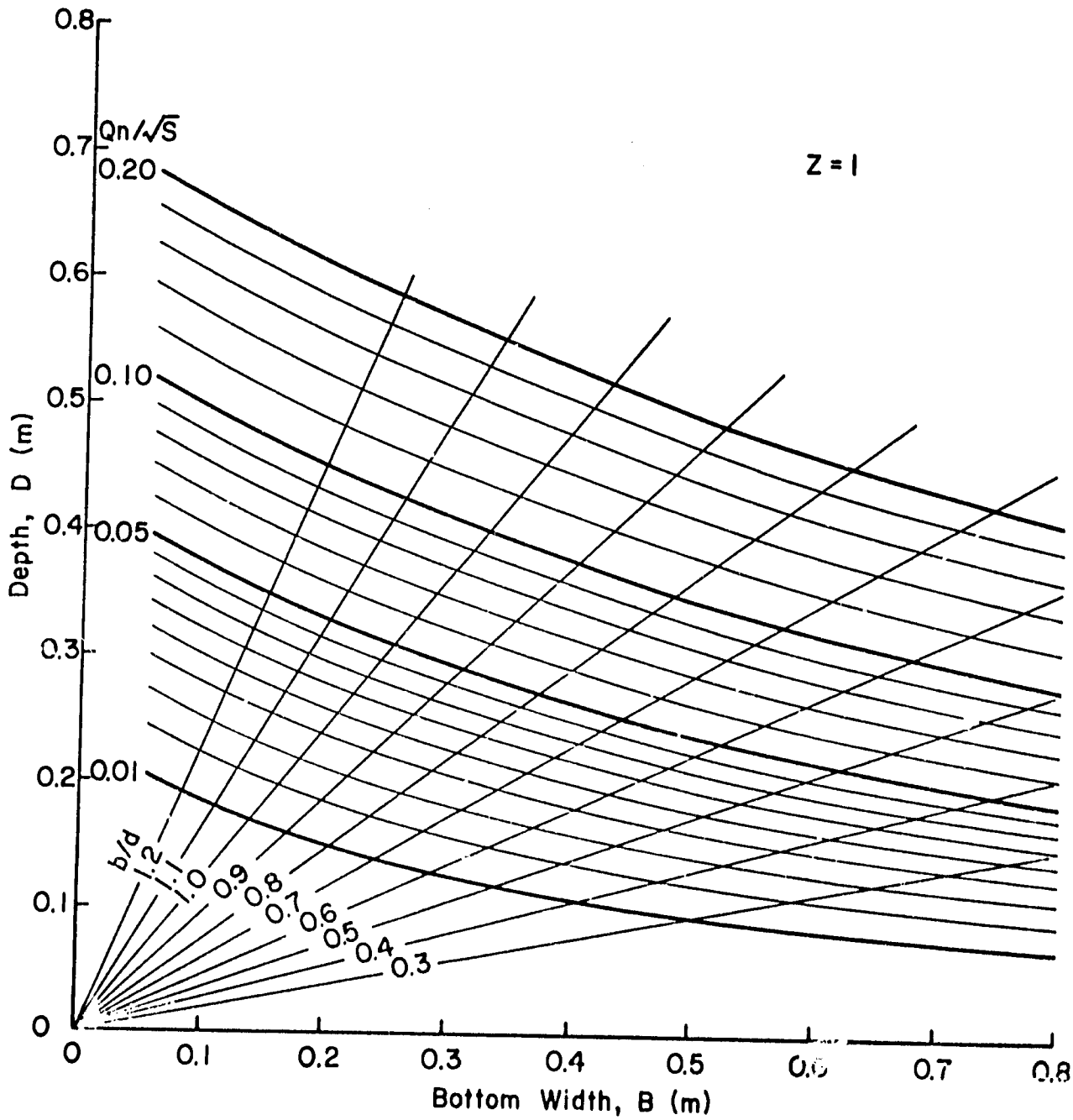


Figure 6.16. Manning's equation solution graph #4-c: cost minimization method for Z = 1.0.

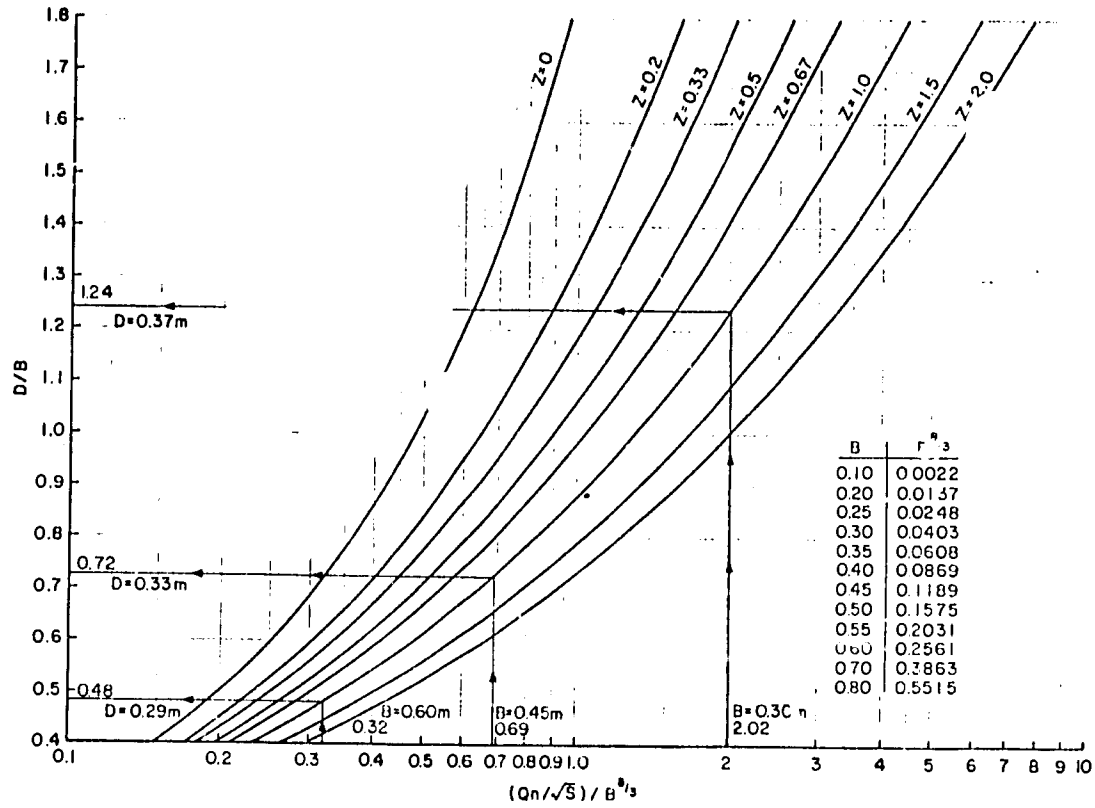


Figure 6.17. Solution of example by method #1 using Figure 6.2.

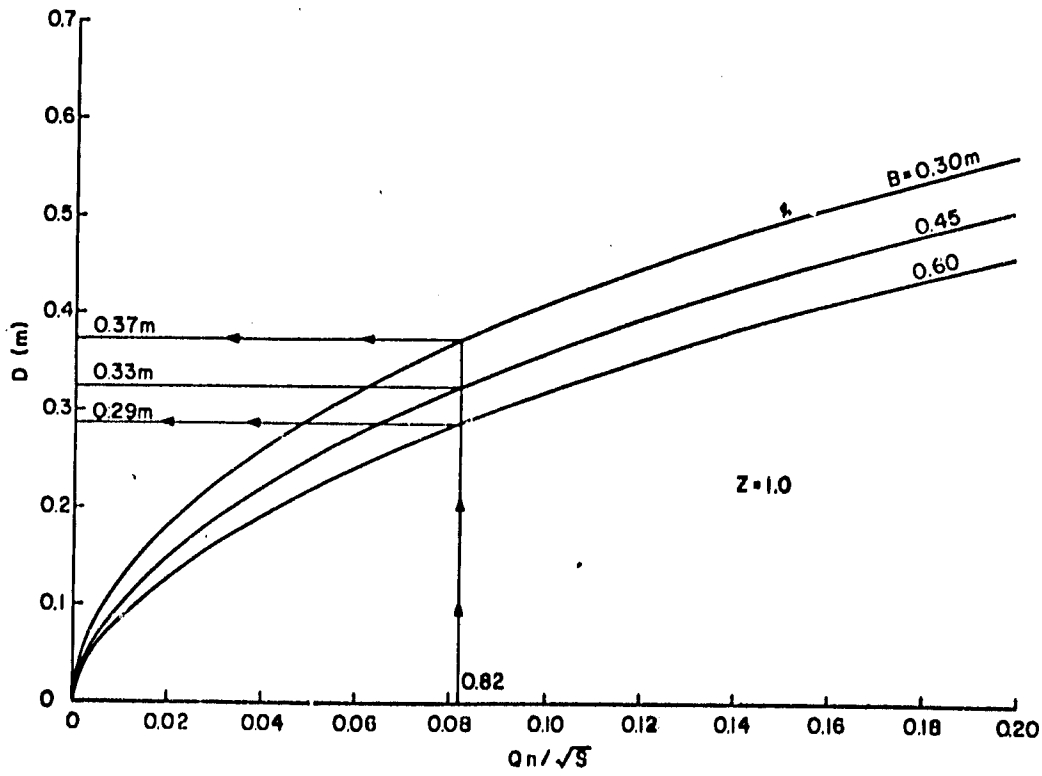


Figure 6.18. Solution of example by method #2 using Figure 6.4.

because the top width and wetted perimeter length are shorter than in the third alternative.

Method #2. On the second set of graphs, (Figures 6.3 to 6.5) the hydraulic section value (0.082) is located on the graph for Z = 1.0 (Figure 6.4). Where this value crosses the curves for B = 0.30, 0.45, and 0.60 m, the D value is read. The procedure is shown in Figure 6.18.

From the graph, for Z = 1.0 and B = 0.30 m; D = 0.37 m;

for Z = 1.0 and B = 0.45 m; D = 0.33 m; and

for Z = 1.0 and B = 0.60 m; D = 0.29 m.

Method #3. By the third technique, (Figures 6.6 to 6.11) first the graph for the proper n, Z, and B value must be found. On the proper graph, a line is followed horizontally from the flow rate value of 50 lps on the ordinate until it crosses a line drawn vertically from a slope value of 0.0006 on the abscissa. These lines cross on the graph for B = 0.30 m (Figure 6.6) between the 0.36 m and 0.38 m depth curves. The design depth is interpolated as 0.37 m. This procedure is shown in Figure 6.19. The flow velocity can also be interpolated from the velocity lines on the graph at the point where the slope, flow rate, and design depth lines cross. From Figures 6.6, 6.7, and 6.8,

for n=0.04, Z=1.0, and B=0.30 m; D=0.37 m and V=0.20 m/sec;

for n=0.04, Z=1.0, and B=0.45 m; D=0.33 m and V=0.20 m/sec; and

for n=0.04, Z=1.0, and B=0.60 m, D=0.29 m and V=0.19 m/sec.

Method #4. On the final set of graphs (Figures 6.12 to 6.16), first the curves for hydraulic section values of 0.08 and 0.09 are located on the Z = 1 graph (Figure 6.16). Since the hydraulic section is 0.082, an interpolated line can be lightly penciled in 20 percent of the way from the 0.08 line to the 0.09 hydraulic section line. At any point along this line, values of bottom width and depth can be selected from the two axes. If the curve is followed until the 0.30 m bottom width line from the abscissa is crossed, a depth value of 0.37 m can be read at the left axis, as shown in Figure 6.20. The minimum wetted perimeter length design, where b/d = 1, will be with B = 0.305 m, and D = 0.37 m.

6.1.6 Calibrated construction guide form solution method

The cross-sectional design process can be bypassed with the use of construction guide forms if channel shapes and sizes are standardized. Instead of utilizing the design graphs, solutions can be printed directly on the guide forms. These forms can then be used directly by technicians in the field.

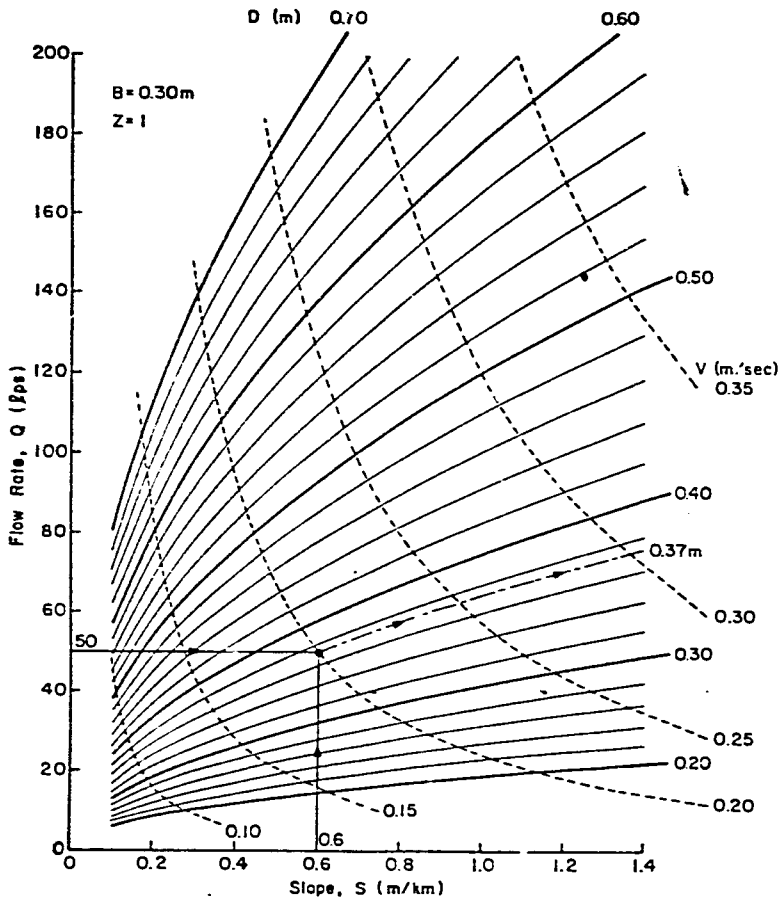


Figure 6.19. Solution of example by method #3 using Figure 6.6.

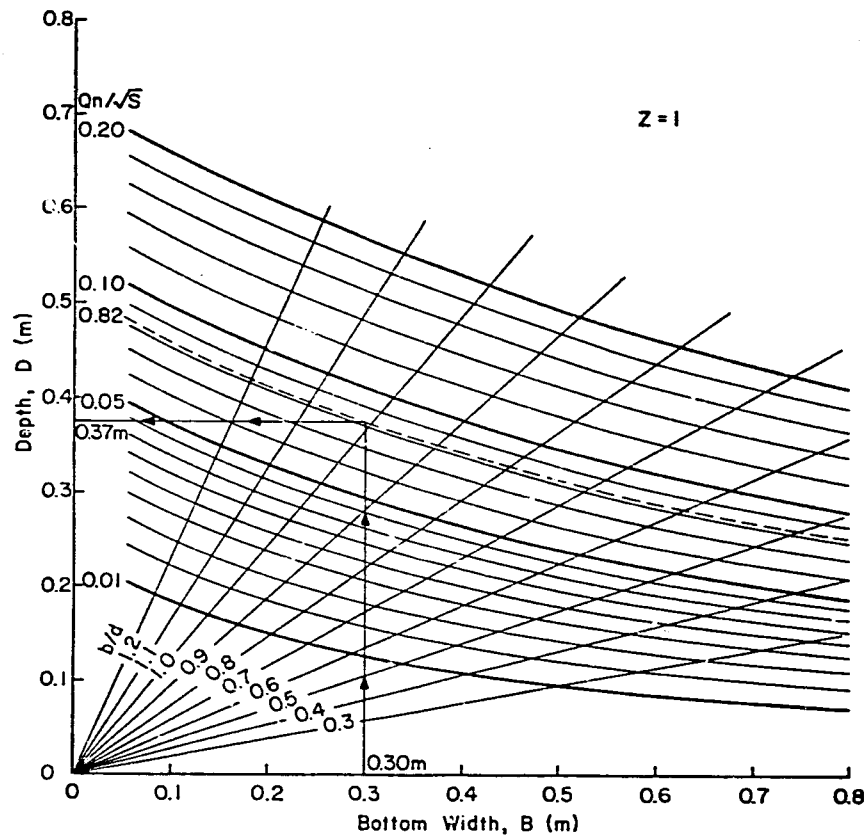


Figure 6.20. Solution of example by method #4 using Figure 6.16.

Figure 6.21 shows an example of a trapezoidal form. Hydraulic section values, $Qn/\sqrt{S}$, are marked on the sides. The forms could be used either to establish the correct channel top width and depths for construction or to check excavation work during or after construction. Once bank top or water surface elevations are established (Section 6.2.3), the form will indicate the correct bottom elevation and bottom and top width for a given required hydraulic section. Since bottom widths can easily be made adjustable, one form can be used for several bottom widths as long as scales for each width are available. A separate form should generally be used for each side slope value.

The scales on the side indicate the proper side length for a required flow rate, roughness coefficient, and slope, plus a freeboard allowance. If one standardized value of roughness coefficient is used, the scale could read in terms of only flow and slope ($Q/\sqrt{S}$). The scale can be developed analytically or from the design graphs.

Table 6.1 can be used to develop the scale for the two side slope and three bottom width values given. The table gives the side slope lengths (L_S in Figure 6.21 (b)) for hydraulic section values from 0.01 to 0.25. The table was developed by determining the design depth, D, from Figure 6.4 for proper Z and B values and for each hydraulic section value. A 0.15 m freeboard (FB) allowance was added to the design depth, and the sum was multiplied by $\sqrt{Z^2 + 1}$ to convert the total depth to a side length. Such a scale can then be attached directly to one or both sides of the form.

Another adaptation of the guide form concept is the use of straight measurement sticks calibrated in hydraulic section values to determine design top widths or total depths.

6.2 Choosing Irrigation Channel Design Parameters

Three parameters or values are required to design irrigation channels by the procedures given earlier in this section: the roughness coefficient, n; the flow rate, Q; and, the slope, S.

6.2.1 Determining the proper roughness coefficient

Manning's roughness coefficient for open channels varies from 0.01 in extremely smooth and uniform channels to over 0.10 in weedy irregular channels. Several hydraulics books list values for Manning's n for various types and conditions of channels. Table 6.2 is excerpted from Chow (1959).

Measurements of Manning's n were made in Pakistan earthen watercourses (Trout, et al., 1979). It was concluded from the study that the roughness coefficients of the small watercourse channels tended to be slightly higher than the values listed

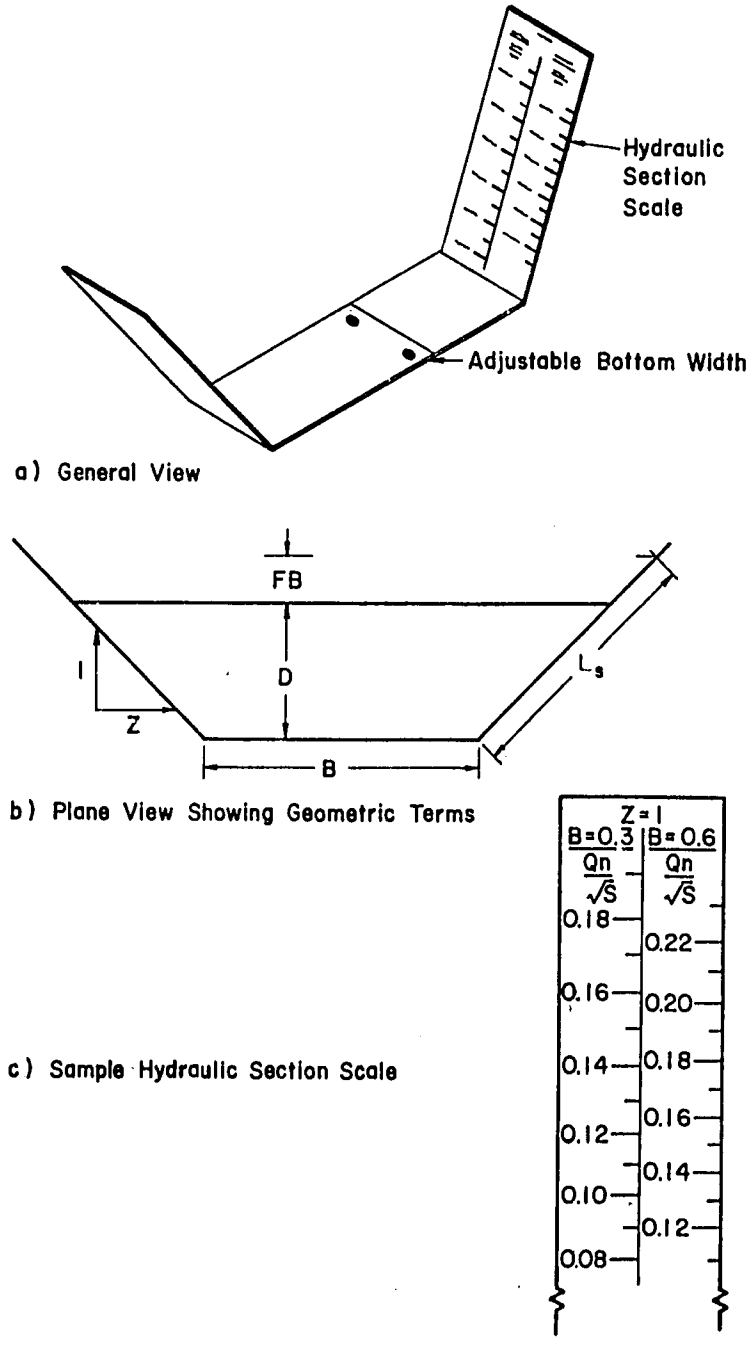


Figure 6.21. Trapezoidal construction guide form.

Table 6.1. Side length values, L_s(m), for side slope values, Z, of 1.0 and 1.5 and bottom width values, B(m), of 0.30, 0.45, and 0.60 for various hydraulic section values, $Qn/\sqrt{S}$.

$Qn/\sqrt{S}$	Z = 1			Z = 1.5		
	B=0.300 m	0.450	0.600	B=0.300 m	0.450	0.600
0.010	0.389 m	0.355 m	0.334 m	0.481 m	0.445 m	0.421 m
0.020	0.471	0.425	0.395	0.571	0.527	0.494
0.030	0.532	0.480	0.443	0.638	0.588	0.550
0.040	0.583	0.527	0.485	0.694	0.640	0.598
0.050	0.627	0.567	0.522	0.742	0.685	0.640
0.060	0.666	0.603	0.555	0.784	0.725	0.678
0.070	0.702	0.637	0.586	0.822	0.762	0.713
0.080	0.735	0.667	0.614	0.857	0.796	0.745
0.090	0.765	0.696	0.641	0.890	0.827	0.775
0.100	0.794	0.723	0.667	0.920	0.856	0.803
0.110	0.821	0.749	0.691	0.949	0.884	0.829
0.120	0.846	0.773	0.713	0.976	0.911	0.855
0.130	0.870	0.796	0.735	1.002	0.936	0.879
0.140	0.894	0.818	0.756	1.027	0.960	0.902
0.150	0.916	0.840	0.777	1.050	0.983	0.924
0.160	0.937	0.860	0.796	1.073	1.005	0.945
0.170	0.958	0.880	0.815	1.095	1.026	0.966
0.180	0.977	0.899	0.833	1.116	1.047	0.986
0.190	0.997	0.918	0.851	1.136	1.067	1.005
0.200	1.015	0.936	0.868	1.156	1.086	1.024
0.210	1.033	0.953	0.885	1.175	1.105	1.042
0.220	1.051	0.970	0.901	1.194	1.123	1.060
0.230	1.068	0.986	0.917	1.212	1.140	1.077
0.240	1.084	1.003	0.932	1.229	1.158	1.094
0.250	1.101	1.018	0.948	1.247	1.175	1.110

Table 6.2. Values of Manning's roughness coefficient, n, for earthen and lined channels (from Chow, 1959, p. 110).

Type of channel and description	Roughness Coefficient Minimum	Normal	Maximum
A. Excavated Earthen Channels			
a. Straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
b. Winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.028	0.030	0.035
5. Stony bottom and weedy banks	0.025	0.035	0.040
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest state of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140
B. Lined or Built-Up Channels			
a. Cement			
1. Neat, smooth surface	0.010	0.011	0.013
2. Mortar	0.011	0.013	0.015
b. Concrete			
1. Trowel finish	0.011	0.013	0.015
2. Float finish	0.013	0.015	0.016
3. Finished, with gravel on bottom	0.015	0.017	0.020
4. Unfinished	0.014	0.017	0.020
c. Brick			
1. Glazed	0.011	0.013	0.015
2. In cement mortar	0.012	0.015	0.018
d. Masonry			
1. Cemented rubble	0.017	0.025	0.030
2. Dry rubble	0.023	0.032	0.035

in the literature. Table 6.3 lists these measured watercourse roughness values. Figure 6.22 shows examples of watercourse channels with different n values.

The roughness coefficient can be measured in existing channels with fairly uniform slopes during uniform, steady state flow by measuring the water surface slope and cross-sectional shape (area and wetted perimeter length). Figure 6.22(a) shows these measurements being made. These values are inserted into Manning's equation (Eq. 11) which can be solved for roughness coefficient. Appendix 3 shows a data collection sheet which can be used to collect information needed to calculate Manning's n for small channels.

The coefficient chosen for design should be based not upon the desirable, or even intended condition of the channels, but upon the realistically expected condition. Although clean uniform channels can have n values as low as 0.02, to design for such a condition would be unrealistic. If it is truly expected that the maintenance will be good and vegetation will be regularly cleaned from the channels, a roughness value of 0.03 could be used. A value of 0.04 is probably more realistic for conditions in earthen channels. Lined channels kept clean from vegetation will have a more constant n value and can be designed closer to the value in a new channel of 0.015 to 0.020.

There are both advantages and disadvantages of using a conservative roughness estimate. A conservative estimate leads to larger and more costly channels, and also gives the cultivators more leeway to allow their channels to get in bad condition. A low estimate can, if the channels are properly designed, force the users to maintain their conveyance system in good condition, but will also allow less safety factor if unusual circumstances should cause a high roughness. The freeboard allowance should, however, be sufficient to allow for most such circumstances.

Figure 5.5 illustrates the relationship between flow depth and roughness coefficient for a typical channel. The depth is more sensitive to the channel roughness in larger (higher Q) and in flatter (lower S) channels. For example, if a channel designed for an n value of 0.03 develops a real roughness of 0.06, the depth would be increased by between 0.10 and 0.15 m. Likewise, a very clean earthen channel (n = 0.02) which had been designed for n = 0.04, would flow about 0.07 to 0.14 m below its designed depth. Flow depths are sensitive to roughness coefficient values so both conservative values and freeboard allowances should be used to insure against overtopping channel banks. A roughness coefficient of 0.04 was used in designing Pakistan earthen watercourses. To overtop a channel with 0.15 m of freeboard, a roughness value of 0.10 would have to be reached. In concrete and masonry lined channels, a less conservative estimate of 0.017 was used due to the high costs of larger cross sections.

Table 6.3. Manning's roughness coefficient values measured in Pakistan watercourses for various channel conditions.

Channel Condition	Measured Roughness Coefficients
lined with brick masonry	0.018
earthen, newly built, uniform, clean	0.17 - 0.032
earthen, winding, with no vegetation	0.030 - 0.035
earthen, uniform, with short grasses	0.026
earthen, winding, with grass and some weeds	0.035 - 0.055
earthen with dense weeds	0.05 - 0.20

(a) n = 0.025

(b) n = 0.035

(c) n = 0.045

Figure 6.22. Pakistan watercourse channels with various roughness coefficients (n).

(d) n = 0.08

(e) n>0.10

Figure 6.22. Pakistan watercourse channels with various roughness coefficients (n). (Cont'd.)

6.2.2 Determining the design flow rate

The design flow rate should be the maximum value which is expected in the channel. A first estimate of the value in an existing system would be the measured flow rate, but several adjustments to measured values might be required.

A quantitative estimate of whether the flow rate at the time of measurement is less than the maximum rate can often be obtained from the users. An estimate of the maximum flow must be made from their estimates in conjunction with the measurements. It must be determined whether water from other sources, such as wells, is also conveyed in the channel, and if the other water is mixed with or conveyed separately from the primary source.

Irrigation officials should also be able to indicate how much the flow rate fluctuates and its relative value during the measurement. They might also know the design flow rate for the turnout, as well as whether any changes in the design rate are anticipated. Turnout design flow rates should always be checked with measurements in the field.

Flow measurements in different sections of the existing conveyance system will indicate how much flow is reaching each location at the present loss rates. These loss rates will decrease after improvement with the amount of decrease depending on the type of improvement. This value can be determined by experience. Once an after improvement loss rate is assumed, the design flow rate at the head can be adjusted downward for sections at some distance from the head by subtracting this loss rate times the conveyance distance from the head design rate.

$$Q_D = Q_{DH} - Q_L \times L \qquad (18)$$

where:

Q_D = the design flow rate in any channel section (lps),

Q_{DH} = the design flow rate at the system inflow (lps),

Q_L = the assumed loss rate after improvement (lps/100 m), and

L = the distance from the head to the section (100 m).

For example, if measurement plus Irrigation Department records indicate that the maximum inflow rate to a channel system is 55 lps, and experience has shown that an earthen renovation program reduces losses to about 0.5 lps/100 m, the design flow rate into a branch located 1600 m from the inflow point would be 47 lps.

Figure 5.6 demonstrates the sensitivity of flow depths to fluctuations in the flow rate. A 50 percent increase in flow rate from 40 to 60 lps will cause a 5 to 9 cm increase in the flow depth in a trapezoidal channel with B = 0.40 m, n = 0.04, and varying slopes.

6.2.3 Determining the design slope

The most difficult of the parameters required for channel design to determine is the slope, S. Slope is more complicated and time-consuming to measure in the field. It is also the design parameter which must be adjusted if the channel design is not satisfactory (i.e., flow velocities are too fast or slow).

The slope of an open channel really refers to the change in energy of the water in the channel over some distance. In a steady and uniformly flowing channel, this is equivalent to the change in elevation of the water surface over a given length of channel. Consequently, the slope of a channel refers to the slope of the water surface and not necessarily of the channel bottom. Designing for the water surface is also convenient since the water surface elevation determines whether fields can be adequately served or whether banks will be overtopped. Channel bottom elevations are calculated later by subtracting the design depth, D, from the water surface elevation; and bank top elevations will be the water surface elevation plus a freeboard allowance.

The most thorough and accurate procedure to determine channel design slopes will involve determining the water surface elevation required at each point in the system to adequately irrigate the land. Channel elevation and slope calculations will consequently begin at the fields at the tail of the system and move up channel including progressively more of the commanded fields in the design. The channels will follow the natural land surface as close as possible so that construction costs are minimized. This design process will continue until the head of the channel system is reached, at which point a check must be made that the system can be served by the water supply source.

Design slopes for channels are dependent upon six factors:

1. the channel layout,

2. the water surface elevation required to adequately irrigate each field,

3. the elevation of the water supply source; i.e., canal turnout or well outflow,

4. the topography of the land around the channels,

5. any head losses in the system, and

6. limiting velocities of the water in the channels.

The first information needed to design conveyance channel slopes is a base map of the commanded area, from which the locations and lengths of all the channels can be determined. The method of obtaining a base map will vary. If the area has been previously surveyed, perhaps for the original channel construction or for the purpose of delineating land ownership, a prepared base map may be available from the proper government offices. The scale and accuracy of the maps should be checked in the field. Aerial photographs, if available in a large enough scale, can also be used to construct or verify base maps.

If no previous maps or photos are available, a base map must be made from field measurements. If the command area and fields are relatively rectangular in shape, and the channels are relatively straight, such a map can be made by drawing a sketch and taping or pacing the distances. If the field shapes are irregular and channels meander, a plane table and peep-sight alidade can be used to construct a base map. The equipment required for plane tabling is simple and inexpensive, but the procedure requires a large amount of time. Appendix 4 describes the use of this equipment for irrigation system mapping. The alternative to the plane table method is the use of a transit and regular surveying techniques, which requires expensive equipment in addition to a large amount of time. The accuracy obtainable with a transit is not required.

Once a base map is available indicating channel locations and lengths and field locations, a topographic survey must be conducted to determine the elevations of the commanded fields. The objective of the measurement is to determine the required elevation of the water in the channel to adequately irrigate the fields. If the fields are level basins, the desired elevation will be of the average land surface in each field. If the basins are relatively flat and small, one elevation determination per field should be sufficient. If they are larger and uneven, several measurements might be necessary to determine an average value. If the fields are irrigated down slopes using either borders or furrows, the elevation at the top of the field near the outlet is required since that establishes the required water supply elevation. The precision required of the topographic survey will depend upon the general slope of the area. Small topographic fluctuations will be more important in a relatively flat area, while only a few shots are required to determine the critical elevations of an area where the channels have more slope. The first step in a topo[illegible] survey is to survey bench marks scattered around the [illegible] area from which field surveys can be run. Appendix 5 bri[illegible] describes bench-mark survey techniques. The bench marks [illegible]ed to conduct the topographic survey should be sufficiently permanent that they will still be available during the c[illegible]ruction phase.

During the topographic surveying, all permanent structures (those which will not be removed or rebuilt during channel improvement) should also be noted, and their elevations measured. For example, the top and bottom elevations and size of road culverts, inverted siphon entrances and exits, and junction box elevations should be measured. Also, the elevation of the water supply source should be determined. If the source is a canal turnout, the water surface elevation below the turnout should be measured as well as the elevation of the turnout structure and of the water surface in the canal. If the source is a well, the outlet elevation of the well should be measured. Field and structure elevations should be added to the base map.

Topographic lines can be sketched on the base map to indicate general land patterns and slopes and to act as a check for erroneous elevation measurements. Figure 6.23 shows a base map with field elevations and topographic lines of the small level basins commanded by a primary branch channel. The topographic map shows all existing field channels and outlets, proposed check locations and the location, elevation, and cross-sectional area of an existing culvert. Appendix 6 gives an example of data collection techniques for and construction of a topographic map of a command area.

The information from the topographic map should next be transferred to a channel profile sheet to determine the design slope. Figure 6.24 shows a channel profile sheet for the primary channel shown in the topographic map. The horizontal axis follows the channel along its length. Location identifiers such as outlets, bends, and structures should be marked on the horizontal axis. The vertical axis of the profile sheet is elevation relative to the bench mark used in the topographic map.

The next information needed on the profile sheet is the elevation required at each outlet to serve the fields. The outlet elevations can be determined from the high fields on each field branch, the fields' distances from the primary channel outlet, and an assumed slope of the branch. If the field branches are also to be rebuilt, a designed slope would be used. Since field ditches are usually no more than a few hundred meters in length, proper slopes aren't so critical if sufficient freeboard is built into the system. In the example since the area is relatively flat a field channel slope of 0.0002 was assumed. For every 100 m distance from the primary channel outlet to the field, 0.02 m was added to the field elevation to derive a required outlet elevation. Because of this slope requirement, the highest field will not always require the highest outlet if fields farther from the outlet are only slightly lower. If the outlet elevation for one field is significantly higher than for the remaining fields, the elevation for the second highest field should also be marked on the profile sheet. Top and bottom elevations of permanent structures should also be noted on the profile.

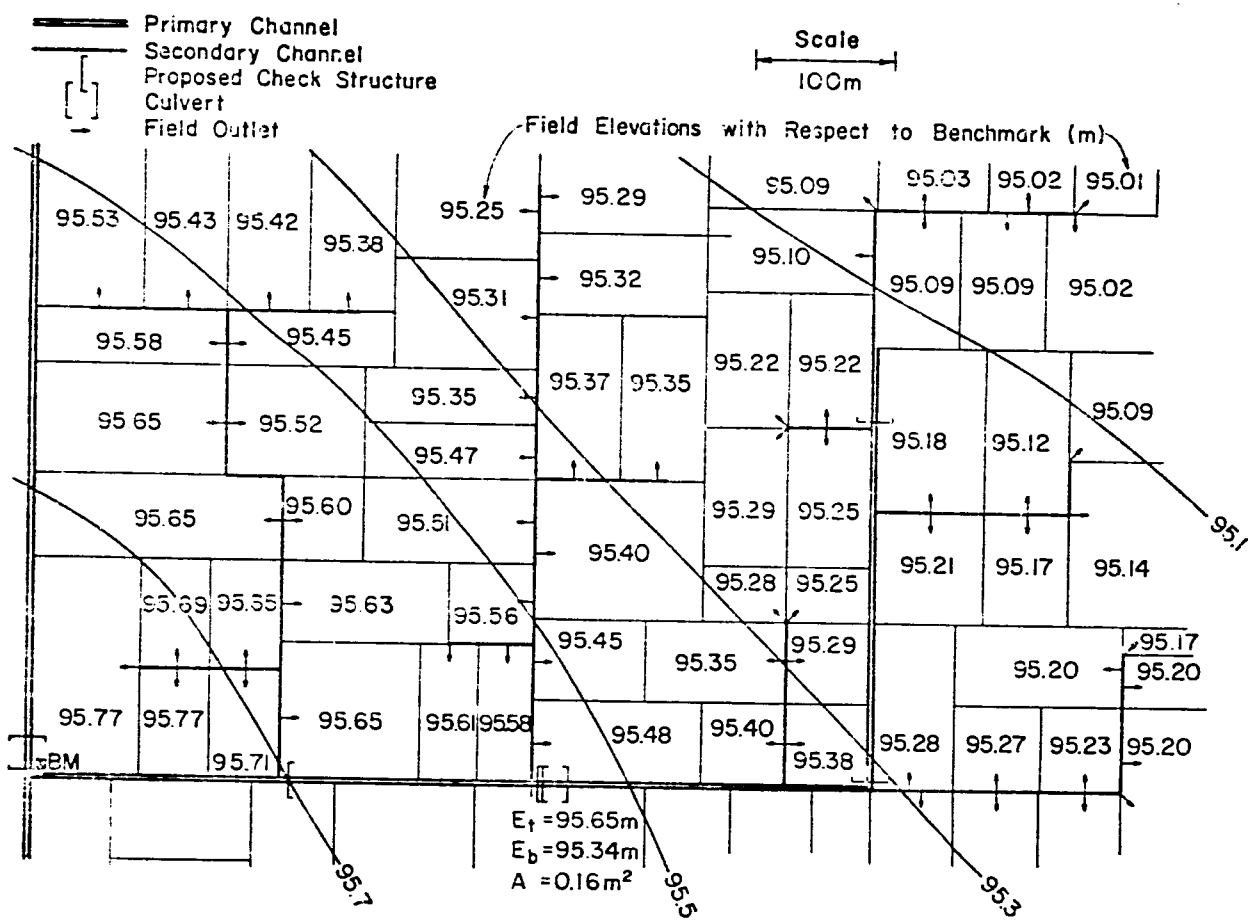
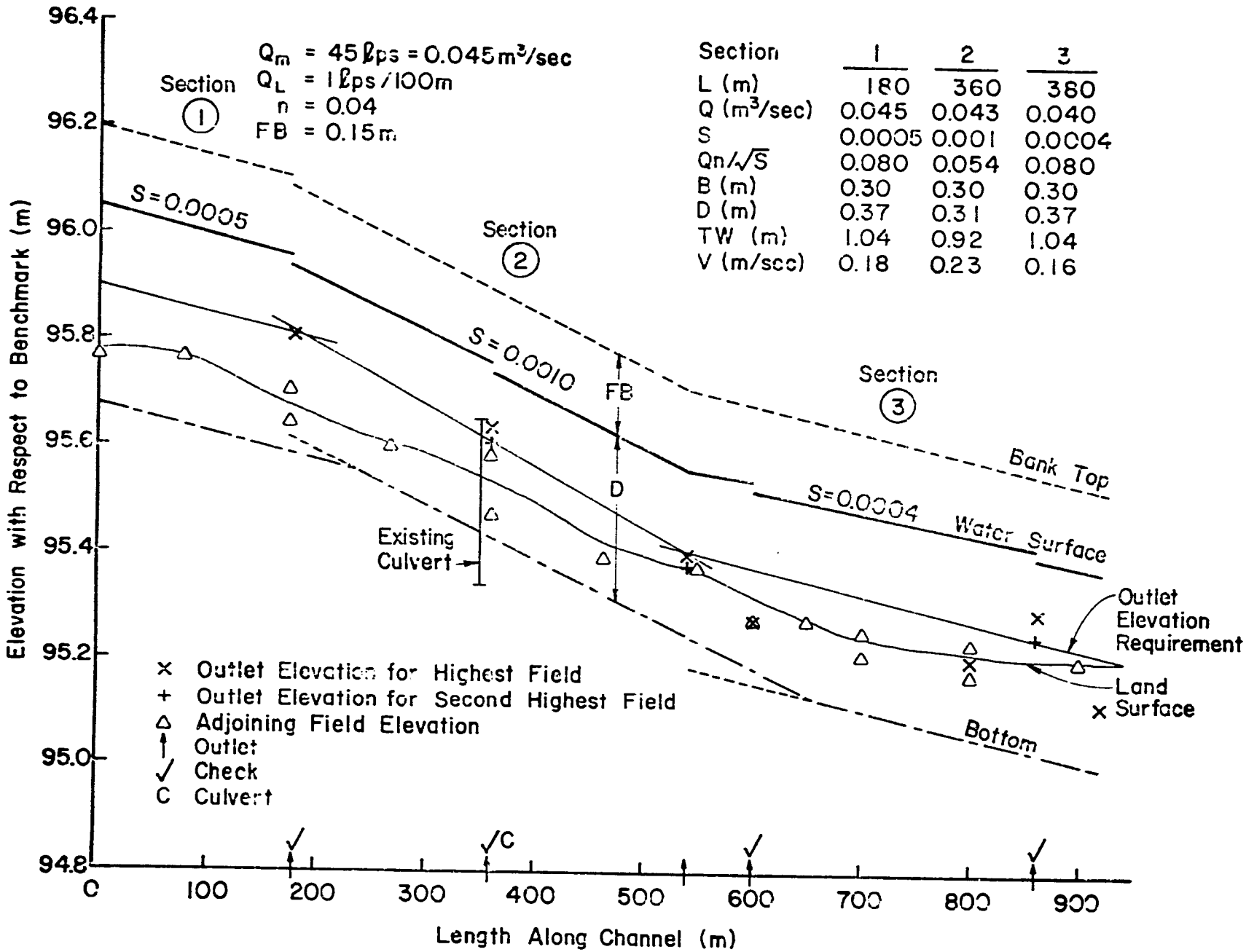


Figure 6.33. Topographic map of the rectangular basins served by a primary branch showing channel layout and topographic lines.



Section	1	2	3
L (m)	180	360	380
Q (m³/sec)	0.045	0.043	0.040
S	0.0005	0.001	0.0004
$Qn/\sqrt{S}$	0.080	0.054	0.080
B (m)	0.30	0.30	0.30
D (m)	0.37	0.31	0.37
TW (m)	1.04	0.92	1.04
V (m/sec)	0.18	0.23	0.16

Fig. 6.24 Profile sheet for branch channel shown in Figure 6.23.

Once outlet elevations are marked on the profile, the channel should be divided into sections where several outlets can be roughly connected with a straight line. These lines should pass through or above the outlet elevation marks. The number of sections should be kept as small as possible to simplify the design, yet large enough that the line doesn't pass too far above several consecutive outlet elevations.

It is desirable to build the channel as low as possible both to reduce construction costs and to reduce losses. Consequently, if one or two fields representing a small proportion of the total commanded area are higher than the others, the slope can be designed for the second or even third highest fields, as was done at the second and fifth outlets in the sample profile. Since a working head and freeboard allowance will be added to these water surface elevations, the high fields can still be served although not as easily. Plotting the elevations of fields adjacent to the channel on the profile will indicate whether the channel is high or low relative to the surrounding land.

Very flat slopes should be avoided as much as possible. Flat slopes require large channel cross sections, increasing construction costs and losses, and result in low velocities which can lead to sedimentation if the water is carrying sediment. Figure 6.25 shows how design flow depths (and thus cross-sectional areas) increase as slopes decrease. Slopes less than 0.0002 are discouraged and greater than 0.0004 are preferred.

Steep slopes cause high flow velocities which can be erosive in earthen channels and can be difficult to control at bends and outlets. Maximum non-erosive velocities in earthen channels will depend upon the soil type, construction techniques, and vegetation cover. Table 6.4 from Booher (1974) lists maximum recommended flow velocities for earthen channels in various soil types and for lined channels. If erosion is a problem in newly built channels, bank compaction, gradual initial wetting up of the channels, and establishment of vegetation can reduce potential erosion problems. Average flow velocity, V (m/sec), can be calculated from the channel cross-sectional area by the formula:

$$V = Q/A, \tag{19}$$

where the area, A, is as defined for a trapezoidal channel in Eq. 13, or it can be taken directly from the third set of solution graphs (Figures 6.6 to 6.11) for the design flow and desired slope. If steep slopes are unavoidable, the excess energy can be dissipated and thus velocities controlled through the use of drop structures which will be discussed in Section 7. Drop structures effectively stair-step water down a steep slope. The channel profile design will indicate where drops are required.

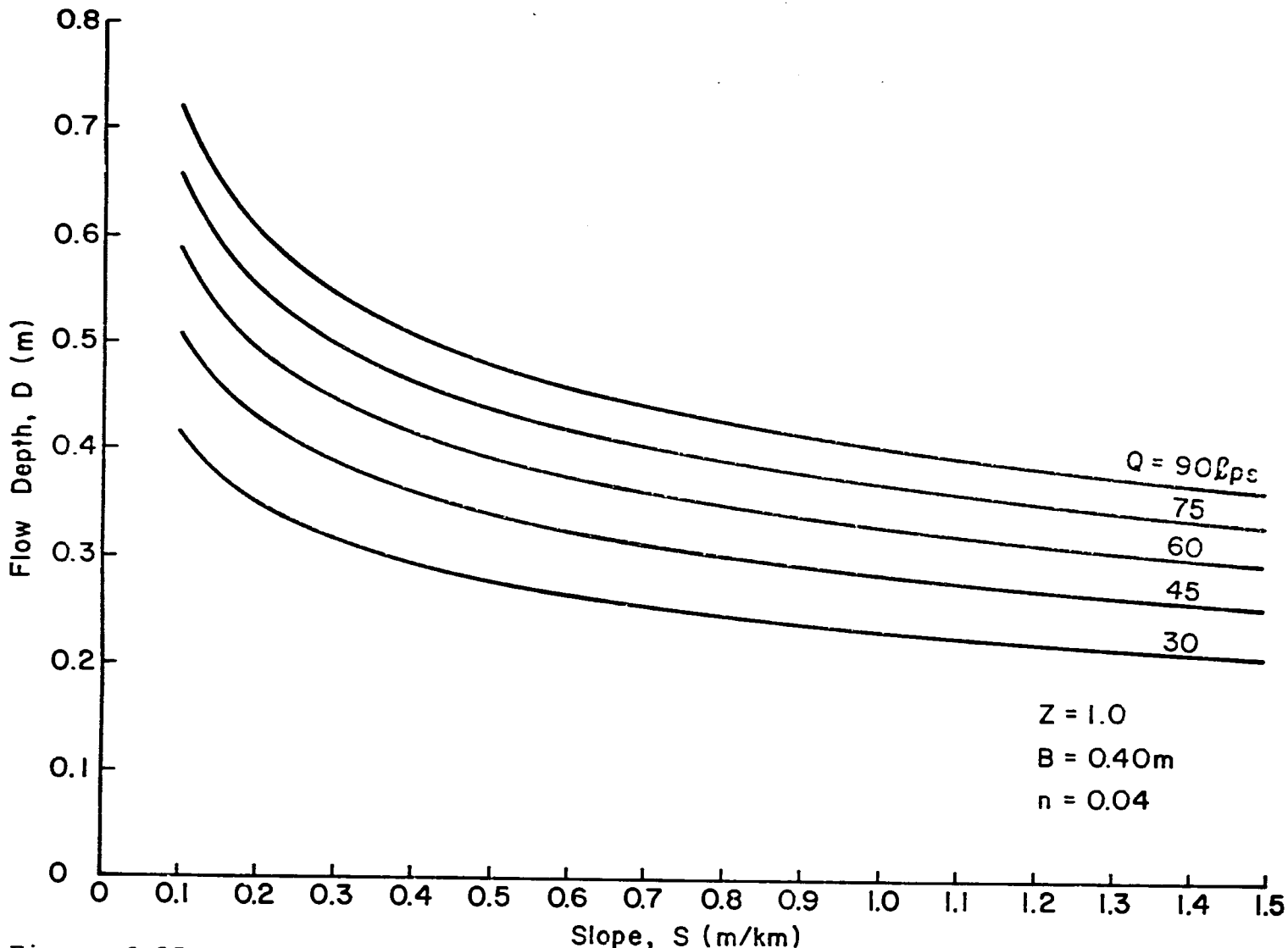


Figure 6.25. The effect of slope on flow depths in trapezoidal channels.

Table 6.4. Suggested maximum flow velocities and side slopes for lined and unlined channels (Booher, 1974).

Type of Surface	Maximum Flow Velocities m/sec	Maximum Side Slopes (Z)
Unlined Ditches		
Sand	0.3 - 0.7	3
Sandy loam	0.5 - 0.7	2 - 2½
Clay loam	0.6 - 0.9	1½ - 2
Clays	0.9 - 1.5	1 - 2
Gravel	0.9 - 1.5	1 - 1½
Rock	1.2 - 1.8	¼ - 1
Lined Ditches		
Concrete		
Cast-in-place	1.5 - 2.5	1 - 1½
Precast	1.5 - 2.0	1½
Bricks	1.2 - 1.8	1½
Asphalt		
Concrete	1.2 - 1.8	1 - 1½
Exposed membrane	0.9 - 1.5	1½ - 2
Buried membrane	0.7 - 1.0	2
Plastic		
Buried membrane	0.6 - 0.9	2½

Once tentative design sections and slopes are chosen, the water surface profile can be plotted taking into consideration the outlet elevation requirement, permanent structure elevations, the working head allowance, and head losses in structures. The working head is the elevation required to push the water through the outlets and into the fields. If the working head is small, water will flow slowly from the channel into the field until the water level builds up in the channel. If channel freeboard isn't sufficient, the banks may eventually overtop unless additional field outlets are cut. Higher fields will also tend to receive less water than low fields due to this in-channel storage change. If the working head allowance is large, the channel must be built higher which increases both construction costs and losses. The recommended working head for small irrigation channels is 0.15 m. This is generally sufficient to push the water from the channels onto the fields, provide equitable distribution to most fields, and provide a safety factor for surveying or calculation errors or unexpected head losses in the system without causing the channels to be too high and increasing construction costs and water losses. The tentative water surface profile, previously drawn at the required outlet elevations, should be redrawn to reflect the working head requirement. In the example (Figure 6.24) the profile lines were raised 0.15 m.

A check must be made to insure that the proposed channel elevations are compatible with the structures which will be retained. If the water level is too low, it must be raised or the structures must be lowered. If the water level is higher than the structures, the structures can be raised or in the case of culverts, entrance and exist structures can be built to allow the culvert to function as an inverted siphon.

Once the proposed water surface elevation at the head of the channel is determined, whether this level is attainable from the water supply source must be checked. If the source is flexible, such as a well outflow, the water source outflow elevation can be adapted to the designed elevation of the channel. If a preexisting or otherwise fixed source is higher than the design surface elevation, the chosen level is OK. However, if the source outflow is below the proposed surface level, slopes must be reduced, working head reduced and/or some higher fields excluded from the commanded area. In the example given, the channel is one branch of a system and the main channel will be designed to the required branch inlet elevation.

If permanent check structures, culverts, drop structures, or other structures which will impede the water flow are to be built in the channel, the head loss (water surface elevation drop) they will create should be taken into account. The head loss through a structure will depend primarily on two factors:

1. The amount of flow constriction, or the difference between the structure flow area and the channel

cross-sectional flow area; and

2. The abruptness of the constriction--gradual converging and diverging sections will reduce head loss.

In practice, the first of these two factors is the most important. Head loss is generally proportional to the change in the velocity squared of the water as it passes through the constriction; and since velocity is inversely related to flow area (Eq. 19), head loss is also proportional to the difference between the square of the flow area of the channel and of the structure. Figure 6.26 shows the head loss through submerged orifice check structures based on the equation:

$$\Delta h = 1/C^2 \; V_s^2/2g, \tag{20}$$

where:

Δh = head loss (m),

C = an empirical coefficient ($C = 0.8$ in Figure 6.26),

V_s = velocity through the structure (m/sec) = Q/A_s,

A_s = the structure flow area, and

g = acceleration of gravity (9.81 m/sec^2).

Figure 6.26 and Eq. 20 assume the area of the orifice is significantly less than the channel cross-sectional flow area, and that the flow is constricted on all sides. The head loss predicted by Eq. 20 and in the figure can be corrected for the "approach" velocity by subtracting out the velocity head in the channel, h_v(m):

$$h_v = V^2/2g, \tag{21}$$

where:

V = the flow velocity in the channel = Q/A. This velocity head correction will generally be less than 10 percent if the orifice area is less than 1/3 of the channel flow area. If a portion of the orifice is suppressed or does not constrict the flow due to the bottom or sides being even with the bottom or sides of the channel, the head loss will also be decreased somewhat making the value calculated in Eq. 20 a conservative estimate. If the orifice is not flowing full or the structure is open at the top, again the head loss will be less than that predicted by Eq. 20. It should be noted that the area, A_s, used to calculate V_s in Eq. 20 is the flow area and not the orifice area if the orifice is not full. For the purpose of design, it can be assumed that culverts have the

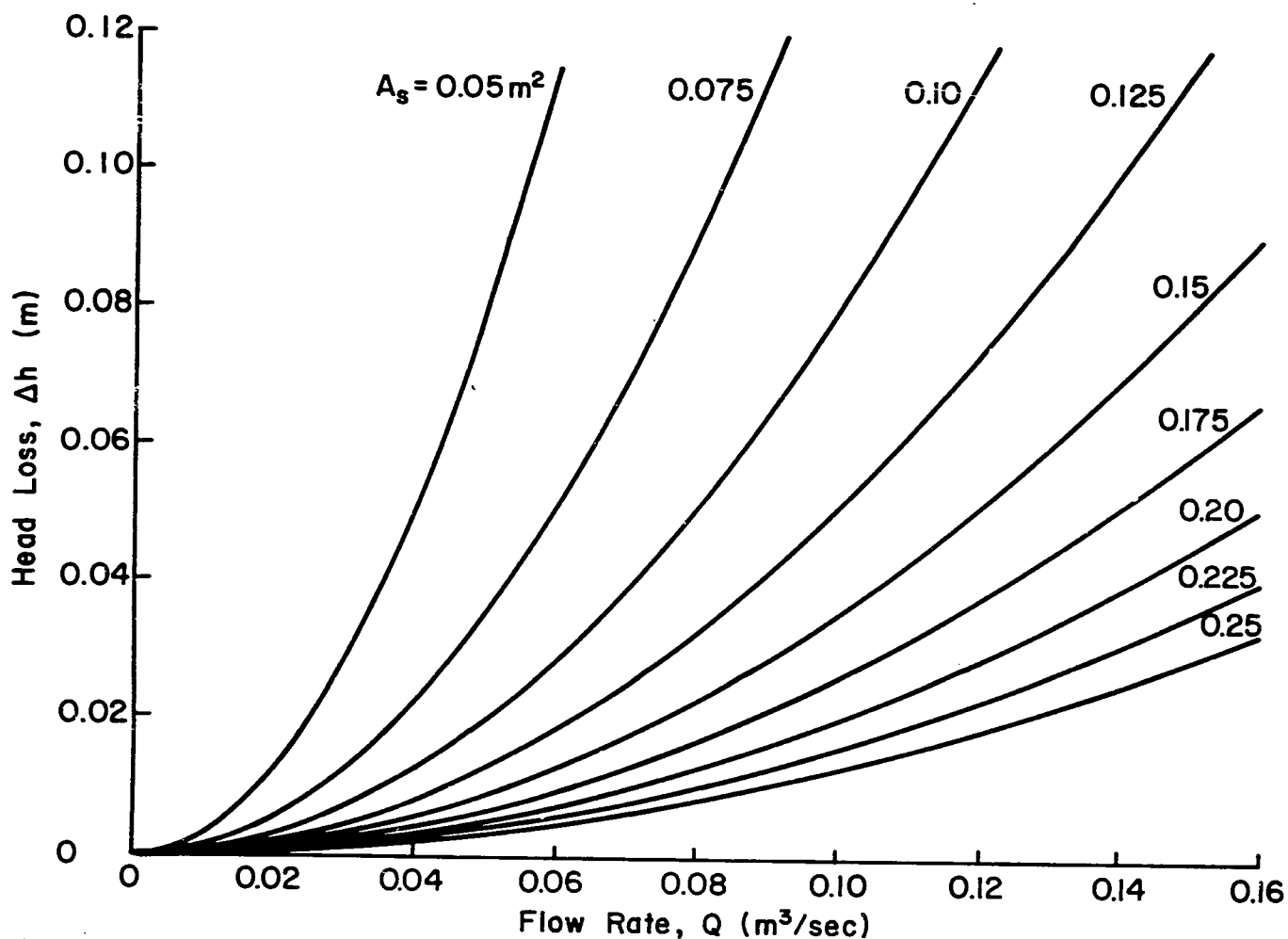


Figure 6.26. Head loss through submerged orifices of various flow areas, A_s, as a function of the flow rate according to Eq. 20 with C = 0.80 and assuming low approach velocities.

same head loss as orifices of the same flow area. In the sample profile, in Figure 6.24, it was assumed that 0.35 m diameter circular orifice checks were being used, which have a flow area of 0.09 m^2, and at a flow rate of 0.045 m^3/sec, a head loss of 0.02 m.

Structures with smaller cross sections and abrupt constrictions are generally less expensive and easier to build, and are often easier to use. Consequently, there is usually a trade-off between reducing head loss and reducing costs, which must be considered in the design process. The head loss through existing structures such as culverts can be measured directly with a surveyor's levels or plastic tubing as will be explained later. It must be remembered that flow rates will probably increase after renovation and the head loss estimates should likewise be increased.

The location of each proposed structure should be noted on the profile. The slope of each section can now be calculated by determining the elevation drop for each section, subtracting the structure head losses, and dividing by the section length.

$$S = \frac{EL_1 - EL_2 - \Sigma\Delta h}{L} \tag{22}$$

Where:

EL_1 = the water surface elevation at the beginning of each channel section (m),

EL_2 = the water surface elevation at the end of each channel section (m),

$\Sigma\Delta h$ = the sum of all the structure head losses in the section (m), and

L = the section length (m).

If the slopes are fairly steep (>0.0008) and the structure losses are not large (< 0.04 m), structure losses do not need to be considered because the final channel design is not sensitive to the relatively small slope decrease.

The method of determining the required outlet elevations by completing a topographic survey of the fields is, as stated, the most accurate. However, the method is very time-consuming and requires fairly expensive equipment and trained surveyors. If area slopes are steep enough that accurate slopes are not critical to the final design, fields are more easily served, and field surface elevation fluctuations are not as important, other less accurate methods can be used.

An alternative method is to determine the slope of the present channel and construct the new channel on the same slope and at the same elevation. Several techniques are available to measure the profiles of existing channels. One is to run a channel profile survey along the channel with a surveyor's level. The survey can be of the channel bottom but preferably should be of the water surface when the water is flowing to the tail. A plot of the present channel profile can be used on the profile design sheet in place of the outlet elevations.

A second method which can be used to determine the slope of an existing channel is to pond water in several adjoining channel sections and measure the elevation drop of the ponded water between sections and the length of the sections. The ponds should be as long as possible such that standing water will still be ponded at the upper end. The technique is most easily applied by building consecutive check dams across a channel beginning at the downstream end. As the ponded section nears being filled, the upstream dam can be built or installed which in turn serves as the lower end of the next section. A series of completed ponds is illustrated in Figure 6.27. After movement in the ponded sections has ceased, the elevation drop across the checks can be measured with clear plastic tubing filled with water, submerged in the higher pond, and turned up at the lower pond water surface. Such a device is shown in Figure 6.28. The elevation of the water in the tubing above the water surface in the lower pond is equal to the water elevation difference across the check.

A third measurement technique utilizes a long plastic tube manometer such as that shown in Figure 6.29. Both ends of the clear flexible tubing should be attached to a rule which can be set on or inserted into the channel bed. The tubing should be as long as is practical. Thirty to fifty meters is recommended. The tubing should be filled with water to about midscale on the rule so that it acts as a manometer. Both rules should be fixed upright in the channel with the tubing stretched out between. The difference in the readings on the two manometers is equal to the elevation differences between the bases of the two rules. The rules can be fixed relative to the channel bottom or, more preferably, relative to the flowing water surface. A variation of this method is to move the upstream end while leaving the downstream rule fixed and insuring that no water is lost from the tube. Then the elevation drop in the section is equal to the reading change in each fixed end when the other end is moved. Manometer readings should not be made until the fluctuations of the water surfaces in the tube have stopped.

Either of the second two methods can be used to plot a channel profile, such as that shown in Figure 6.30, from which slopes of present channels can be determined. If the present channels are effectively serving all commanded areas, the same slopes should be adequate for renovated channels. The

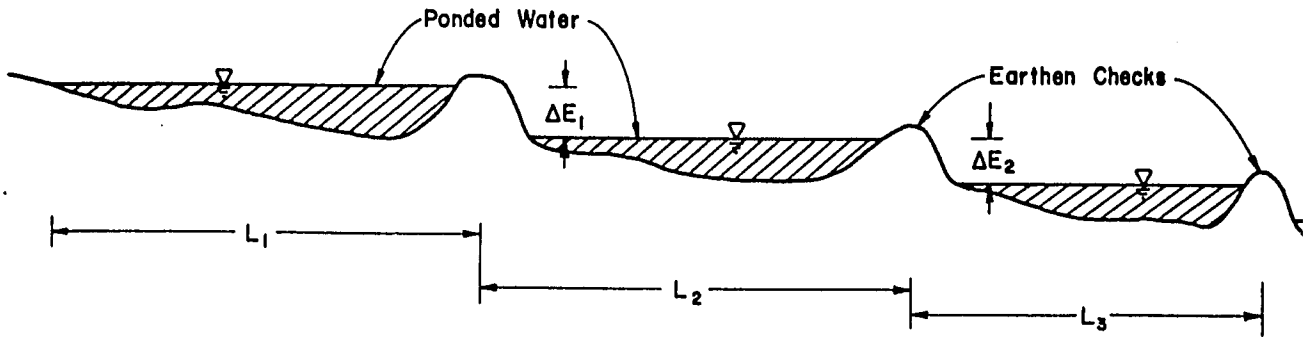


Figure 6.27. An illustration of the ponded water technique to determine the slope of a channel.

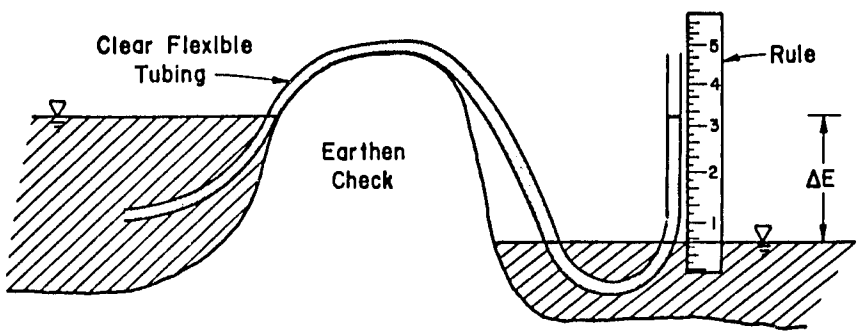


Figure 6.28. Utilizing a clear plastic tube to determine the elevation difference, ΔE, between two ponds of water.

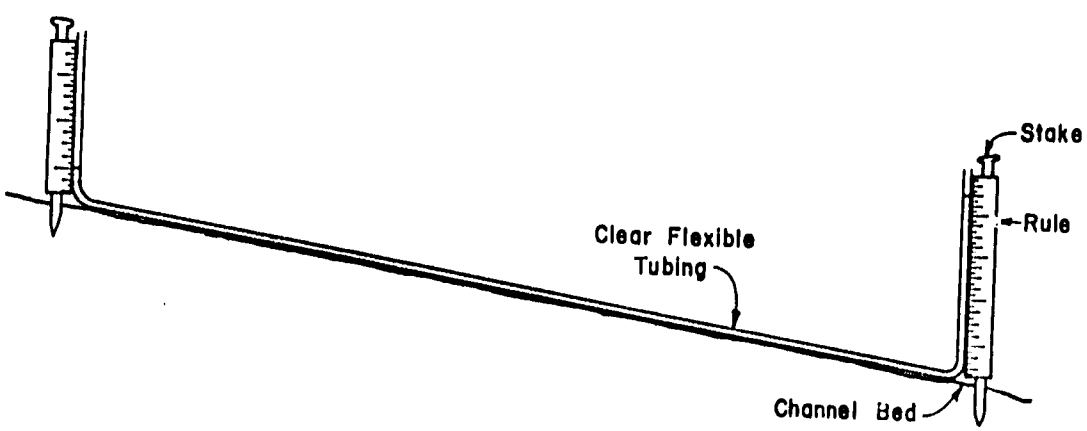


Figure 6.29. A long clear plastic tube manometer for determining elevation changes along channels.

inclusion of some additional working head in the new design will add a safety factor. The measured slopes can be adjusted for head losses in proposed structures by Eq. 22.

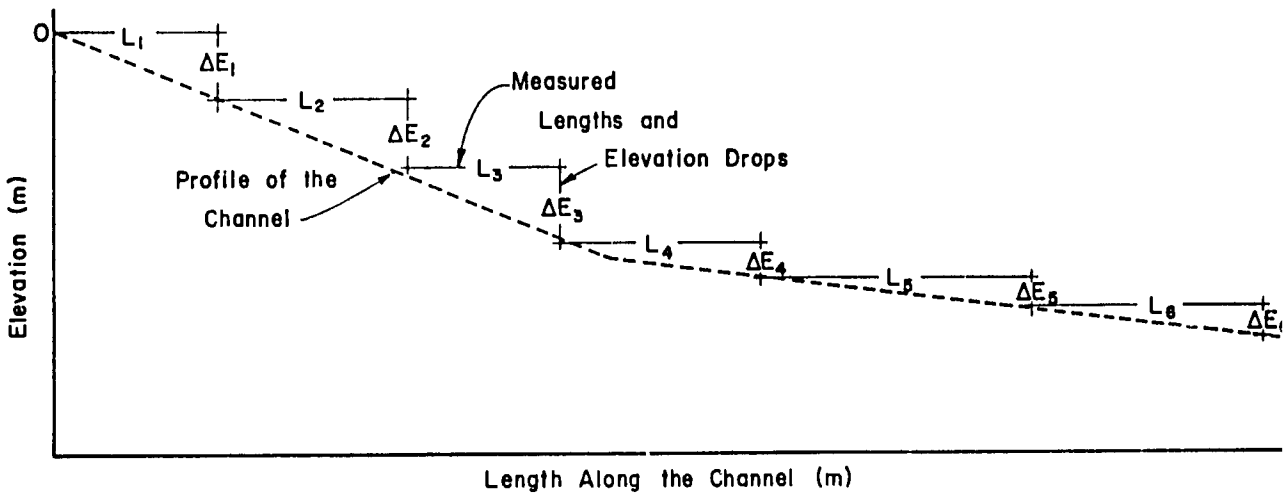


Figure 6.30. Plotting ponded water or manometer measurements to determine a channel profile or slope.

A problem with these alternative techniques is that no bench marks are established from which the rebuilt channel can be positioned after the old channels are destroyed. One alternative is to establish bench marks (wooden stakes are sufficient, permanent structures are preferred) relative to the present water surface which can remain fixed during reconstruction. The bench marks can be fixed either in the channel bed using a rule or outside the channel using the flexible tubing. The tubing manometer can then be used to establish intermediate design elevations between bench marks during construction by reversing the slope measuring process.

6.3 Completing the Design

Once the slope, flow rate, and roughness coefficient are determined, the channel cross section can be designed using Figures 6.2 through 6.16. Two geometric parameters, the depth-to-bottom width ratio, and the side slope, must still be chosen before the final cross section can be determined. The side slope will depend both upon the stability of the materials from which the channel is built, and the desired cross-sectional shape. Earthen channel banks built in unstable soils will sluff and erode if the side slopes are too steep. Table 6.4 lists recommended maximum side slope values for different soil types. These values are conservative for small irrigation ditches. Experience and observation of existing channels commonly indicates that steeper slopes are feasible. In Indus Basin loam and clay loam soils, new channels were successfully built with 1:1 (Z = 1) side slopes. Even steeper side slopes in consolidated or well packed banks were often possible.

Steeper side slopes are often desirable since the resulting channels have a narrower top width which presents less hinderance to passage, decreases the costs of structures, and require less land; and a shorter wetter perimeter which decreases the potential water seepage area. The trapezoidal channel with the minimum wetted perimeter length has a side slope value of 0.58, although this is generally too steep for earthen channels, and a channel with a Z value of 1.0 can be built with only a 5 percent longer wetted perimeter. Side sluffing which often occurs immediately after construction in earthen channels can be minimized by bank compaction and gradual initial filling and drainage of the channel.

A factor operating against this general rule that steeper side slopes are better is the tendency for shallower side slopes in earthen channels to collect more silt and thus have lower infiltration rates. If silt deposition is part of the loss-reducing design, shallower side slopes should be considered and measurements made to determine the optimum side slope.

Depth-to-bottom width ratios should normally be chosen to reduce channel wetted perimeter lengths. Top width really isn't very sensitive to the D/B value and so does not need to be considered. In trapezoidal channels, the wetted perimeter length will be the shortest for a given side slope value when the depth-to-bottom width ratio, D/B, is:

$$D/B = \frac{1}{2\sqrt{Z^2 + 1} - 2Z} \tag{23}$$

According to this equation, when:

$Z = 1.0$, $D/B = 1.2$,

$Z = 1.5$, $D/B = 1.7$, and

$Z = 2.0$, $D/B = 2.1$.

Consequently, as a general rule to minimize wetted perimeter length, the depth should be somewhat larger than the bottom width, with the D/B value increasing as the side slope value increases. If loss measurements show that most losses are from the sides rather than the beds of earthen channels, D/B values less than those which minimize wetted perimeter length should be used.

The final design information and parameters should be noted directly on the channel profile sheet such as was done in Figure 6.24. The flow velocity for each section should be calculated and design adjustments made if velocities are too high or low.

Bank top elevations will be the water surface elevations plus a freeboard allowance. The freeboard allows for increased roughness coefficients, inflow rates above the design level, temporary obstructions in the channels, and errors in the survey, design, or construction work. A freeboard of 0.15 m is recommended for small earthen channels.

Channel bottom elevation will be the water surface elevation minus the design flow depth. A jump or drop in the bottom elevation will occur between sections when slopes change. Instead of an abrupt change which will lead to dead storage losses or silt deposition, a smooth transition should be designed between successive sections such that both abrupt drops and negative slopes are avoided. The bank top and channel bottom elevation profiles should be drawn on the design sheet, as shown in Figure 6.24.

Final design dimensions and elevations can be recorded directly on the profile sheet, as was done in Figure 6.24, or on a separate design sheet, such as is illustrated in the main channel design example given in Appendix 7. By either method, all information required in the field to construct the channel should be contained on one design sheet. Channel bottom elevations, widths, and alignments, and bank top elevations and widths can be staked with a surveyor's level using the same bench marks as in the original topographic survey. If the cross section construction forms described in the previous chapter are used, only bottom elevations and alignments are required.

Experience has shown that after final construction, the earthen bank top elevations are generally below the design level. This is usually due to settling of the bank soils. Unless banks are well compacted during construction, a settling allowance should be added to the freeboard allowance to compensate for this. It was found in Pakistan that an additional 5 cm must be added to the designed bank elevations during construction to allow for settling of the uncompacted bank soils.

SECTION 7

IRRIGATION CHANNEL STRUCTURES

Most irrigation channel improvement programs will include the installation of some types of permanent structures. The purposes of irrigation channel structures are to improve water control, prevent channel degradation, reduce losses, measure water flows, and aid the community in working around their water conveyance system. Irrigation channel structures include:

1. checks,
2. outlets,
3. division boxes,
4. energy dissipators (drops),
5. sediment control devices (silt traps),
6. flow measurement devices,
7. culverts, and
8. community structures such as clothes washing stations and animal watering and bathing stations.

7.1 Water Movement Control Structures: Checks, Outlets, and Division Boxes

Farmers need to be able to direct the water through the conveyance system to their fields. Checks, outlets, and division boxes are the water gates which accomplish this purpose. Checks are placed across a channel to stop and redirect the water flow or to control the flow depth. Outlets are the gates which allow water to flow from one channel into a branch, or from a channel into the field being irrigated. Division boxes are designed to divide water into two or more channels. At junctions in a conveyance system, these three purposes sometimes merge. As water gates, these structures should:

1. have minimal leakage when closed,
2. not obstruct the flow when open,
3. be easy for the farmer to use, and
4. be inexpensive to construct and install.

Check and outlet structures can be either permanent or temporary, and the decision of which type to use will depend upon the usage. Generally, regularly used checks and outlets

at major junctions and outlets to branches, especially if cooperatively used by several farmers, should be permanent installations; while structures required to divert water into an individual field can be temporary. Temporary water control structures include earthen dikes or canvas or rigid dams for check structures, and cuts through earth banks or siphon tubes for outlets. Temporary structures will not be discussed in this report, although the use of temporary dams and siphon tubes should be considered as part of a conveyance system improvement program.

Several factors must be considered in choosing materials to be used for constructing permanent water control structures. The first, of course, is that it can be fabricated into a product that can fulfill the design requirements. It must produce an effective water gate which is durable, has a long life and is simple and easy to use. It is desirable that the material be locally available and can be utilized by local or at least regional fabricators. In this way, the growth of small local industries would be encouraged, and farmers would have ready access to replacements. The product should also be as low cost as possible. This would reduce the cost of watercourse improvement programs, or alternately increase the number of structures which could be installed on a given budget. Also, a structure that is sufficiently inexpensive that a small farmer would purchase it for his own use, or could at least purchase replacements, would be desirable.

Materials which are used for the construction and installation of check and outlet structures include steel, cast iron, concrete, brick, stone, or concrete block masonry, soil cement, wood, and fiberglass. Each of these materials have drawbacks under some conditions. Steel is expensive and has a scrap value which could lead to theft losses. Wood has a fairly short life and a reuse value as fuel. Concrete is heavy and, unless of good quality, subject to chipping and breakage. Fiberglass is relatively expensive and usually not locally available. Brick or stone masonry and soil cement require high labor inputs in the field. The choice of materials will depend primarily upon the local conditions and costs.

Check and outlet structures will usually be composed of a lid, a seat or slide for the lid which is built into a frame or panel, and, if placed into an earthen channel, an installation structure to support it, prevent leakage around the structure, create an opening for water passage, and prevent erosion. Division boxes will consist of two or more rigid or adjustable openings and an installation structure. Checks and outlets can act as division boxes if their gates are adjustable.

Many types and designs of water control structures are in use around the world, although most of the structures described in the literature are designed for larger canals and are generally

over-designed and quite expensive for small channel use. Kraatz and Mahajan (1975) describe several types of control structures for small channels. Four common types of control structure gates, shown in Figure 7.1 are:

1. wood flash boards or a metal sheet which slides in a concrete or masonry groove;

2. rectangular sheet metal gate which slides in a metal groove, sometimes fitted with a rubber strip in the groove;

3. a cast iron flap or slide which fits over a round outlet hole often used with pipe outlets and usually with a machined sealing surface; and

4. precast concrete lids and panels of various shapes.

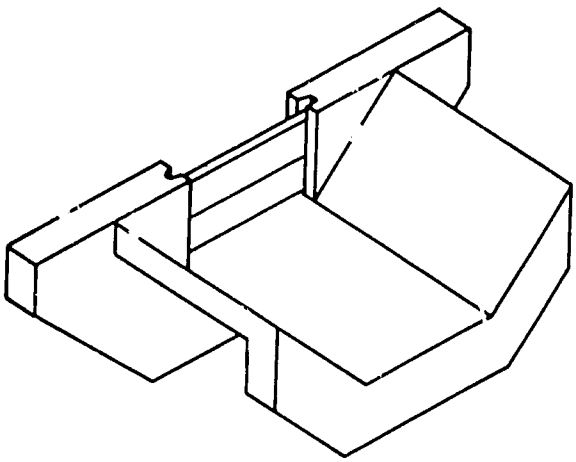
The first type is easy to construct and inexpensive but usually allows significant leakage. The second and especially the third type can be quite leak-proof if properly constructed but are more expensive and difficult to fabricate. The concrete gates can have low leakage if carefully and properly constructed but can be heavy and difficult to use in large channels.

Standard designs, such as the one for a poured concrete trapezoidal division box shown in Figure 7.2, are available from organizations such as the U.S. Soil Conservation Service, but such designs must always be adapted to local materials, conditions, and requirements. In this report the structures developed for Pakistan watercourse systems will be described as an example of the control structure development process.

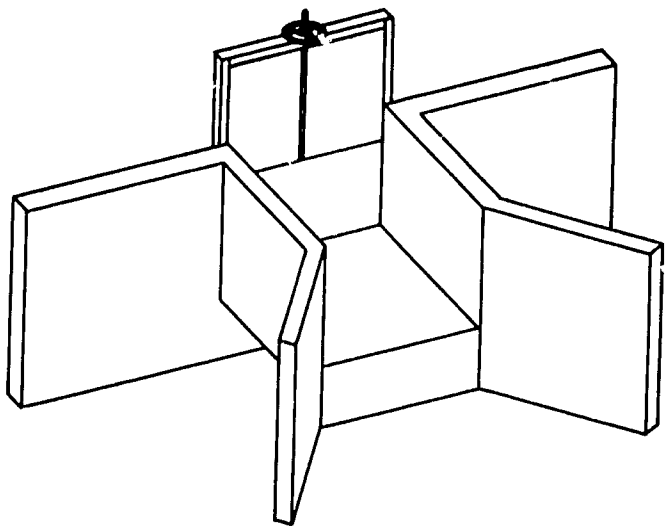
7.1.1 Control structure development in Pakistan

Irrigation water is rotated to each farmer on Pakistan watercourses on a weekly basis. Consequently, outlets from main to branch channels are used regularly. The water is seldom divided, so the gates do not need to regulate flows--only direct them. Field sizes are small, and thus the conveyance system is extensive requiring numerous branch outlets along the main channels so outlet leakage per structure must be small. Most of the water delivery channels have minimal slope, so structure head losses in the system should be as small as possible.

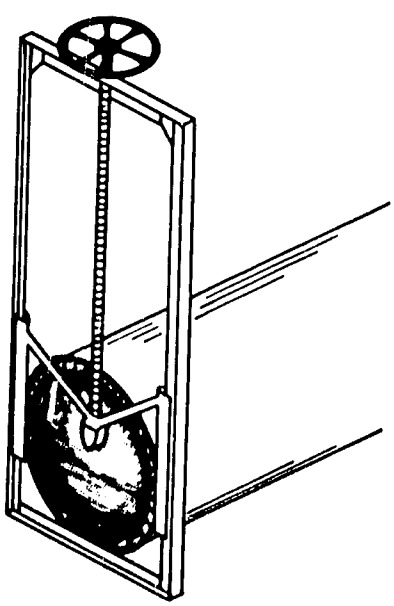
Cement, sand, and bricks are readily available and relatively cheap. Concrete fabricators are located in every town and brick masons live in every village. Also, unlike steel and wood, concrete and brick masonry has practically no reuse value and thus stealing is not a problem. Consequently, check and outlet designs were adapted to these materials.



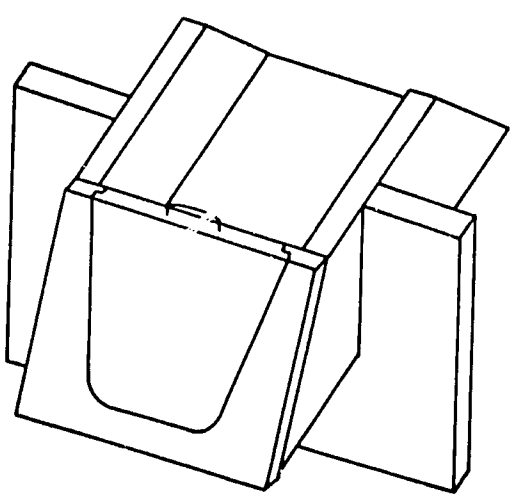
a. wood flash boards in concrete grooves.



b. sliding sheet metal gate.

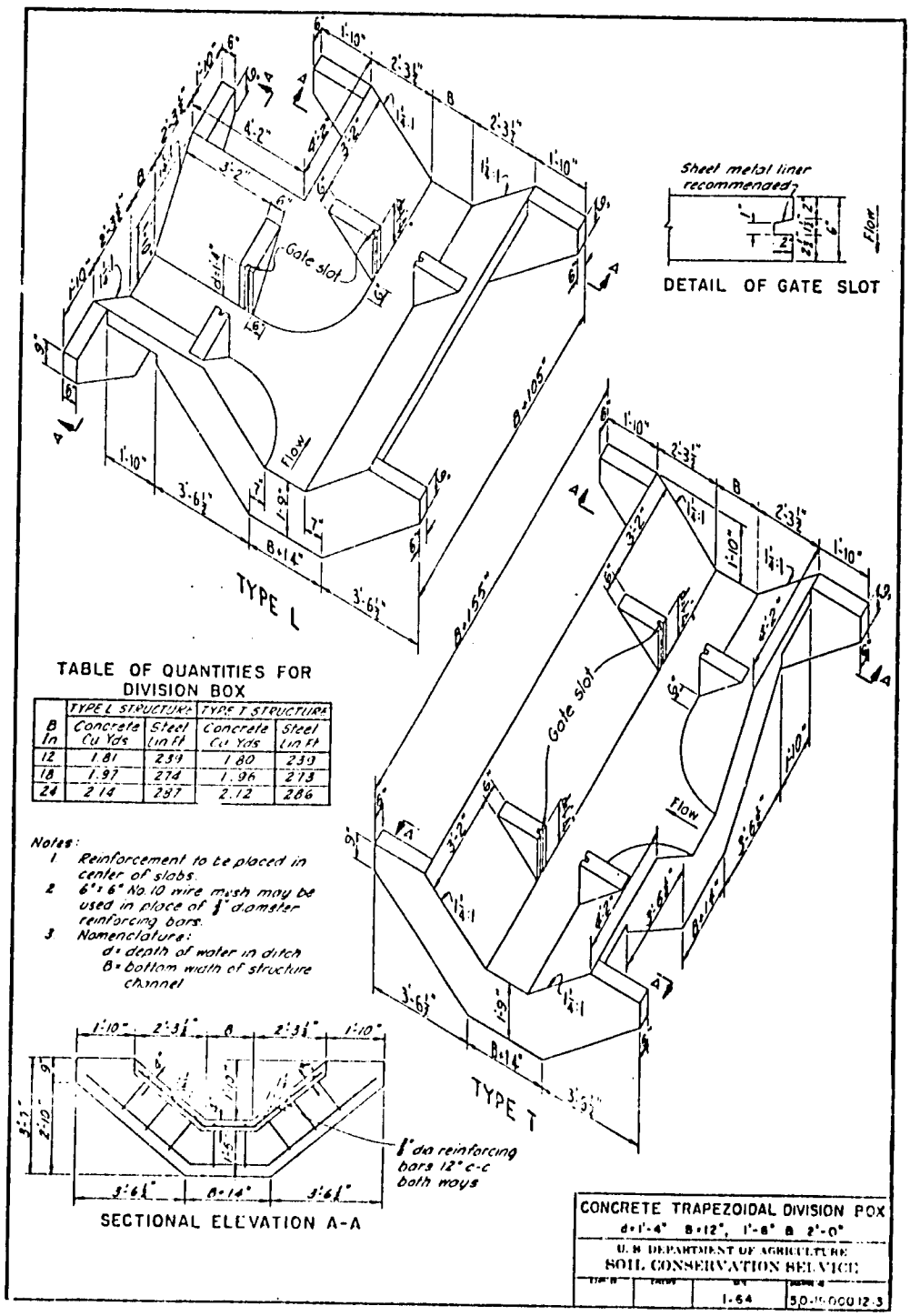


c. sliding cast iron gate.



d. concrete panel and lid.

Figure 7.1. Examples of small channel water control structures.



B In	TYPE L STRUCTURE Concrete Cu Yds	TYPE L STRUCTURE Steel Lin Ft	TYPE T STRUCTURE Concrete Cu Yds	TYPE T STRUCTURE Steel Lin Ft
12	1.81	239	1.80	239
18	1.97	274	1.96	273
24	2.14	287	2.12	286

Figure 7.2. U.S. Soil Conservation Service Concrete Trapezoidal Division Box Standard Design.

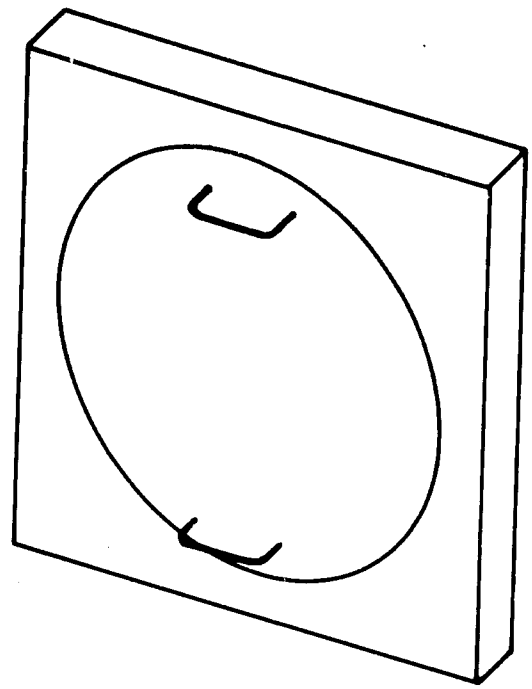
Based on these requirements and material preferences, concrete panel orifice gates (Figure 7.3) were developed to fulfill the control structure needs. A durable structure with low leakage and which was not too heavy was required. The leakage problem was solved by the use of machined steel molds to form the concrete components. The round shape was chosen because local machinists could make precise molds on their lathes. The round shape also provides both a structurally strong lid and a submerged opening which gives a maximum flow area:lid weight ratio.

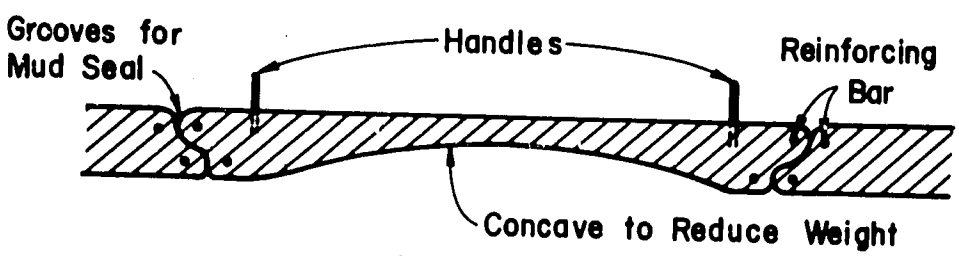
The rounded shape of the sealing surface shown in cross section in Figure 7.3 evolved in response to chipping and subsequent leakage encountered with square edges. The rounded shape distributes the impact of the lid on the panel over a wider area and makes closing easier since the rounded surfaces tend to slide together, as compared to the initially developed lids with square edges. The rounded seal shape allowed lids to be cast directly into previously cast, grease coated panels insuring a perfect fit. Casting the lid over a convex plate reduced the center thickness and weight and made the gate easier to use. Turning the lid in its panel for a few minutes after curing ground the sealing surface smooth and helped create a leak-free fit. A groove left at the front could be packed by farmers with mud if leakage should still pose a problem. The use of a high quality concrete mixture, vibration, and curing in a pond for at least a week improved the strength and durability of the product. Figure 7.4 illustrates the fabrication process.

Because the orifice outlets often required that farmers enter the channel to open the lid from in front, a trapezoidal panel outlet shown in Figures 7.5 and 7.6 utilizing similar sealing surface shape and casting techniques was also developed. Although the structure is easy to use in smaller sizes, the weight of the lid becomes excessive in larger sizes since, unlike the orifices, a freeboard allowance must be included in the lid surface area. It was thus adaptable to small channels or where head loss was no problem. Such structures have proven quite successful in Spain (Figure 7.6(b)).

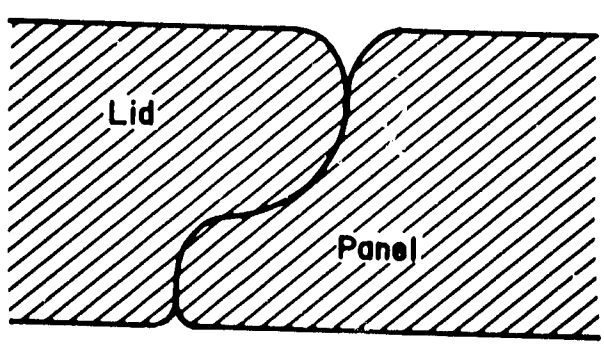
Thousands of the orifice outlets in several sizes are presently in use in Pakistan. Most are installed in a brick masonry structure constructed in place such as is shown in Figures 7.7 and 7.8. In response to the large amount of time required to install these structures, a precast installation shown in Figures 7.9 and 7.10 was developed. Such structures not only speed up construction time and free technicians for other jobs, but they insure uniform quality and can be dismantled and moved in the future if required.

The soil replaced around structure installations must be firmly compacted or seepage along the walls will quickly begin





Cross Section

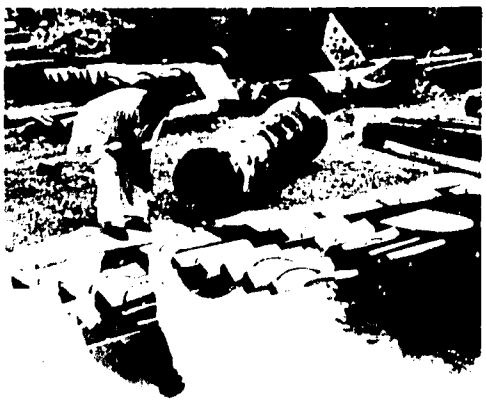


Enlarged Cross Section of the
Sealing Surface

Figure 7.3. Circular concrete orifice panel outlet.

(a) Casting the panel with a machined steel mold.

(b) Casting the lid into the panel and vibrating to remove bubbles.



(c) Curing the panels and lids in a pond for at least one week.

(d) Turning the lid in the panel to grind the sealing surfaces to provide a tighter fit.

Figure 7.4. The process of fabricating concrete orifice outlet gates in Pakistan.

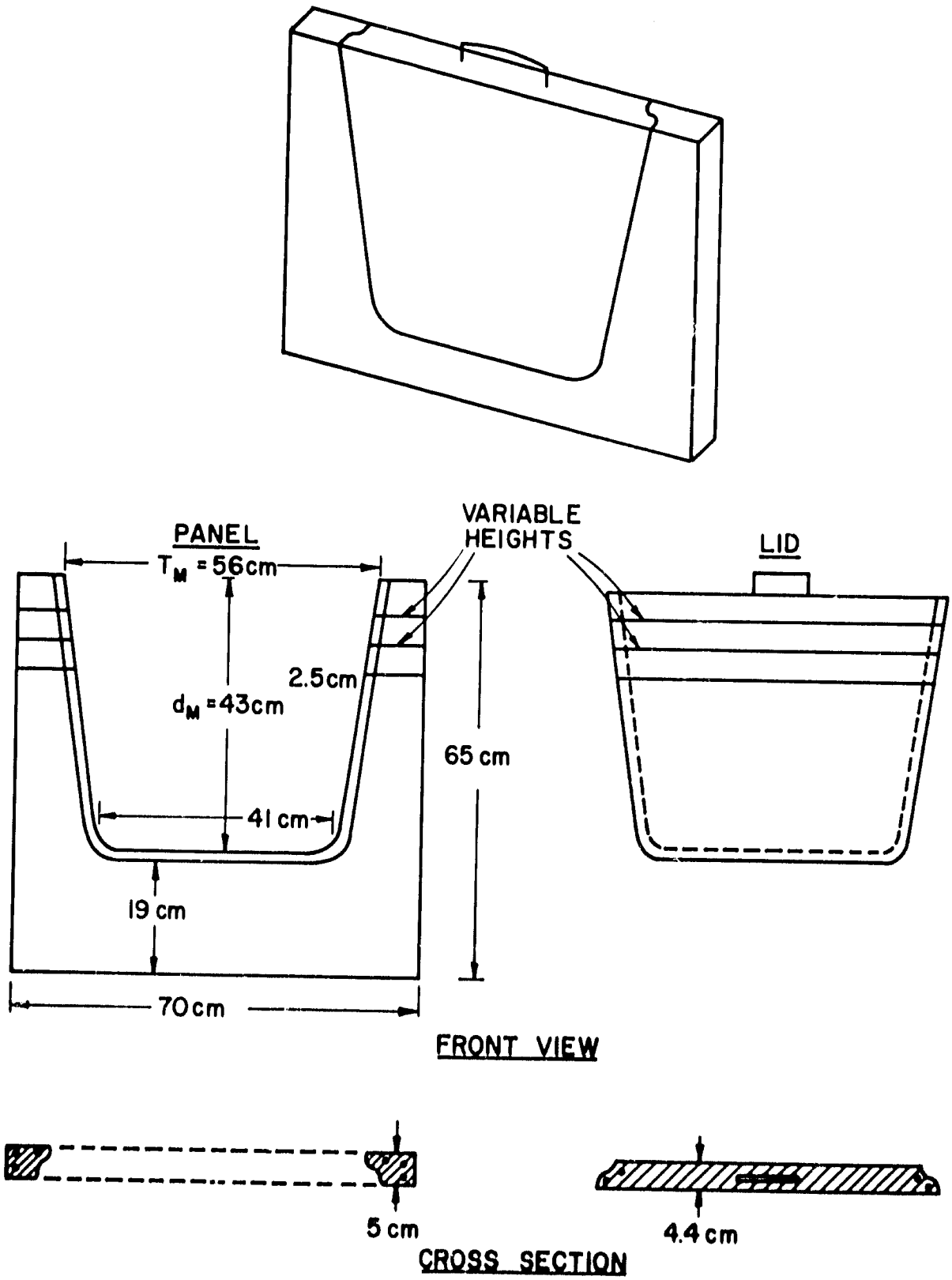
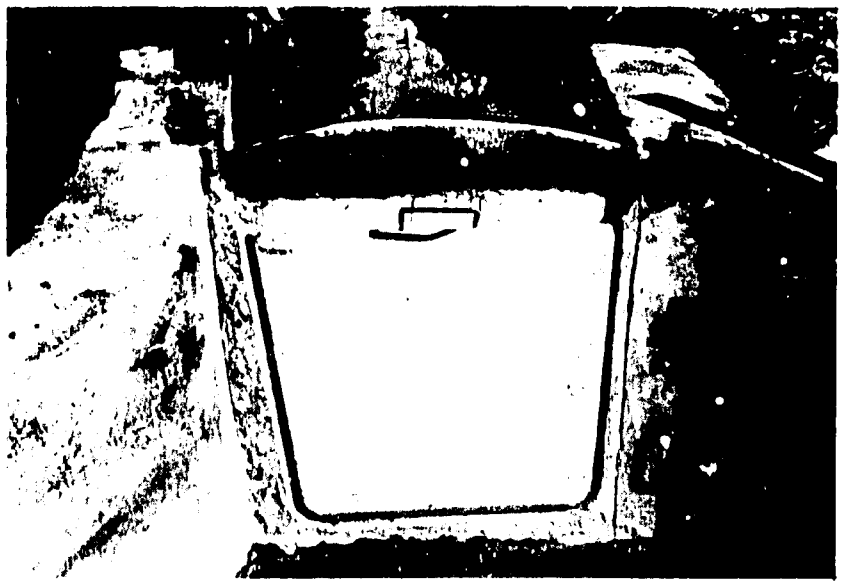


Figure 7.5. Trapezoidal panel outlet.



(a) Pakistan

(b) Spain

Figure 7.6. Trapezoidal concrete outlets from Pakistan and Spain.

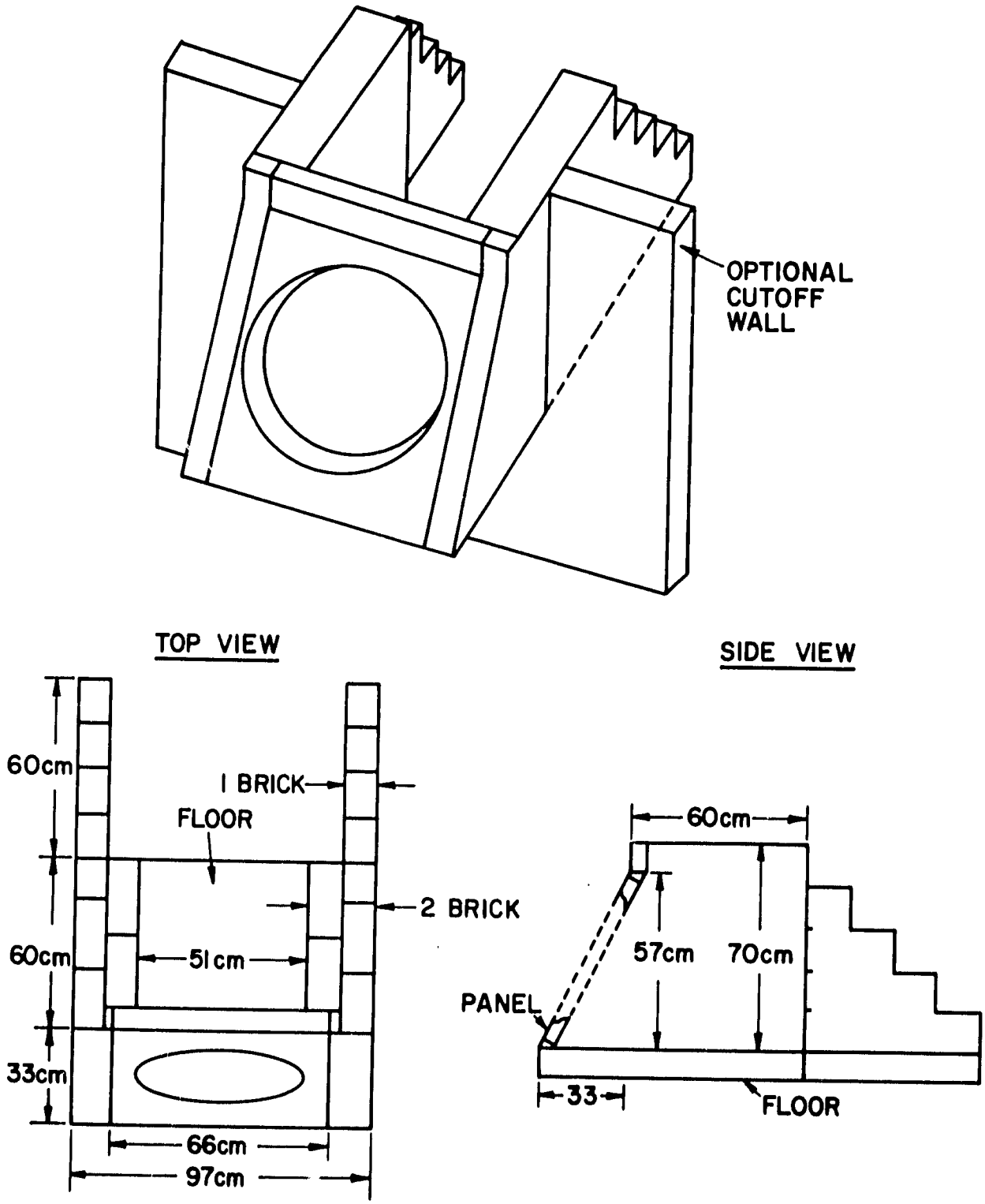


Figure 7.7. Brick masonry installation for panel outlets.

Figure 7.8. Brick masonry installation for concrete panel outlets.

Figure 7.9. Precast concrete slab installation for panel outlets.

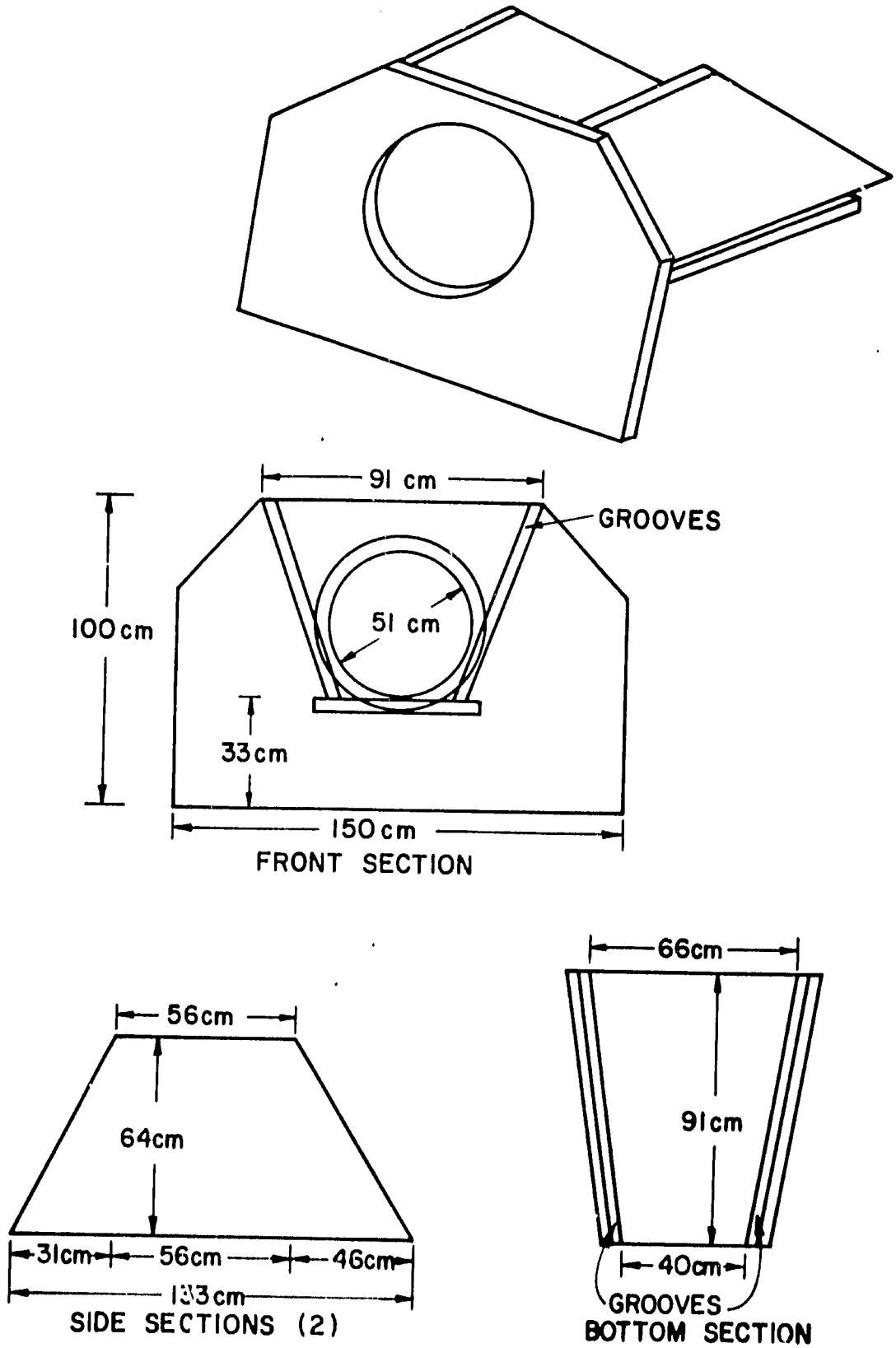


Figure 7.10. Precast concrete slab installation for panel outlets.

piping leaks and eventually wash out the bank. Cutoff walls as shown in Figure 7.7 will reduce the possibility of piping and subsequent water losses and washouts but do not eliminate the need for compaction. In cohesive soils cutoff walls should not be required if the compaction is sufficient. On some improved watercourses in Pakistan, more water is lost leaking around structures than through them due to poor compaction. Soils replaced around structures should be moist (near field capacity) and placed and compacted in layers. A hand tamper is the best tool to apply a compacting force around structures.

To simplify construction, the same gates are generally used as outlets and checks. Occasionally, however, larger or deeper check structures are required to minimize head loss or raise the water level, and the lids become too heavy to handle. For these specialized purposes, gates were designed in which the concrete lid does not need to be removed from the panel but swivel on a pivot point or slide up and down on a slanted track. These rectangular gates had higher leakage rates than the previously described outlets and were thus only used as checks when the needs demanded large openings. Although several designs were tested, no concrete gates were developed which could accurately regulate or divide flows which did not allow significant leakage when closed.

7.2 Control Structures to Prevent Channel Degradation

Two types of irrigation channel structures are sometimes required to reduce deterioration of the channel cross section from sedimentation or erosion. Silt traps collect sediment in one place which would otherwise deposit along the channels. Drop structures are used to reduce channel slopes and thus flow velocities in order to prevent erosion.

7.2.1 Sediment traps

If sediment which enters a conveyance system in the irrigation water cannot all be conveyed to the fields, usually because of flat slopes and low velocities, it will deposit in the channels and eventually fill up the cross section. The only solution is then to manually or mechanically clean out the channels and either deposit the sediment on the banks or haul it away. Continual throwing of silt on the banks eventually leads to high banks which often become covered with vegetation. These silt piles make cleaning and maintenance operations difficult and spill over into adjoining fields taking land out of production.

Sediment deposition problems are difficult to solve, but in many cases the deposition can be concentrated in one area with the use of a sediment trap and thus be made easier to monitor and control. A sediment trap is essentially a channel section with an enlarged cross section such that the flow

velocity is low enough that the desired amount of sediment drops out of the water and accumulates on the bed. Unless sediment deposition is not desired on the fields, all of the sediment should not be retained in the trap, but only the larger particles which would otherwise accumulate in the channels.

Two parameters are required for sediment trap design: the cross-sectional area and the length. The cross-sectional area should be large enough to slow the water to a velocity at which the desired sized particles will fall, and the length should be such that the particles have sufficient time to drop to the bed. Of course these two factors are inversely related, and a shorter trap of large cross section can accumulate the same amount of sediment as a longer trap with a small cross section, so the final design will depend upon the amount and shape of the space available. The values of these parameters will depend upon the local sediment load and channel velocities and can be established by field trials. In Pakistan, sediment traps were normally designed to have about six times the downstream channel cross-sectional area and to be a least 30 meters long.

Sediment traps do not need to be lined although lining can make cleaning easier. The traps must be cleaned regularly to be effective, and they must be located such that the sediment can be stored or disposed of. Sediment piled near roads, villages, or brick kilns can be used in making adobe or fired bricks, or used to enrich the soils in garden plots.

7.2.2 Drop structures

As explained in the previous section, channels with steep slopes will erode due to the high velocity of the flowing water. The slopes and thus water velocity can be reduced by stair-stepping the water through drop structures. Drop structures are designed to lower the elevation of flowing water while dissipating its energy and thus not allowing the velocity to increase.

In large canals, the design of drop structures becomes very complicated, but in small channels, a simple overflow check with a vertical drop into an erosion protected basin is sufficient. Such structures can be used to drop water as much as 0.60 m. A series of such small structures is usually less expensive than constructing one structure such as a chute for a large drop. Figure 7.11 shows examples of many types of both vertical overfall and chute drop structures for small channels. Kraatz and Mahajan (1975 Volume II) and USBR (1974) describe the design of several types of drop structures.

The purpose of the overfall check of a drop structure is to prevent the flowing water from drawing down and thus accelerating when approaching the drop. The high velocity may cause

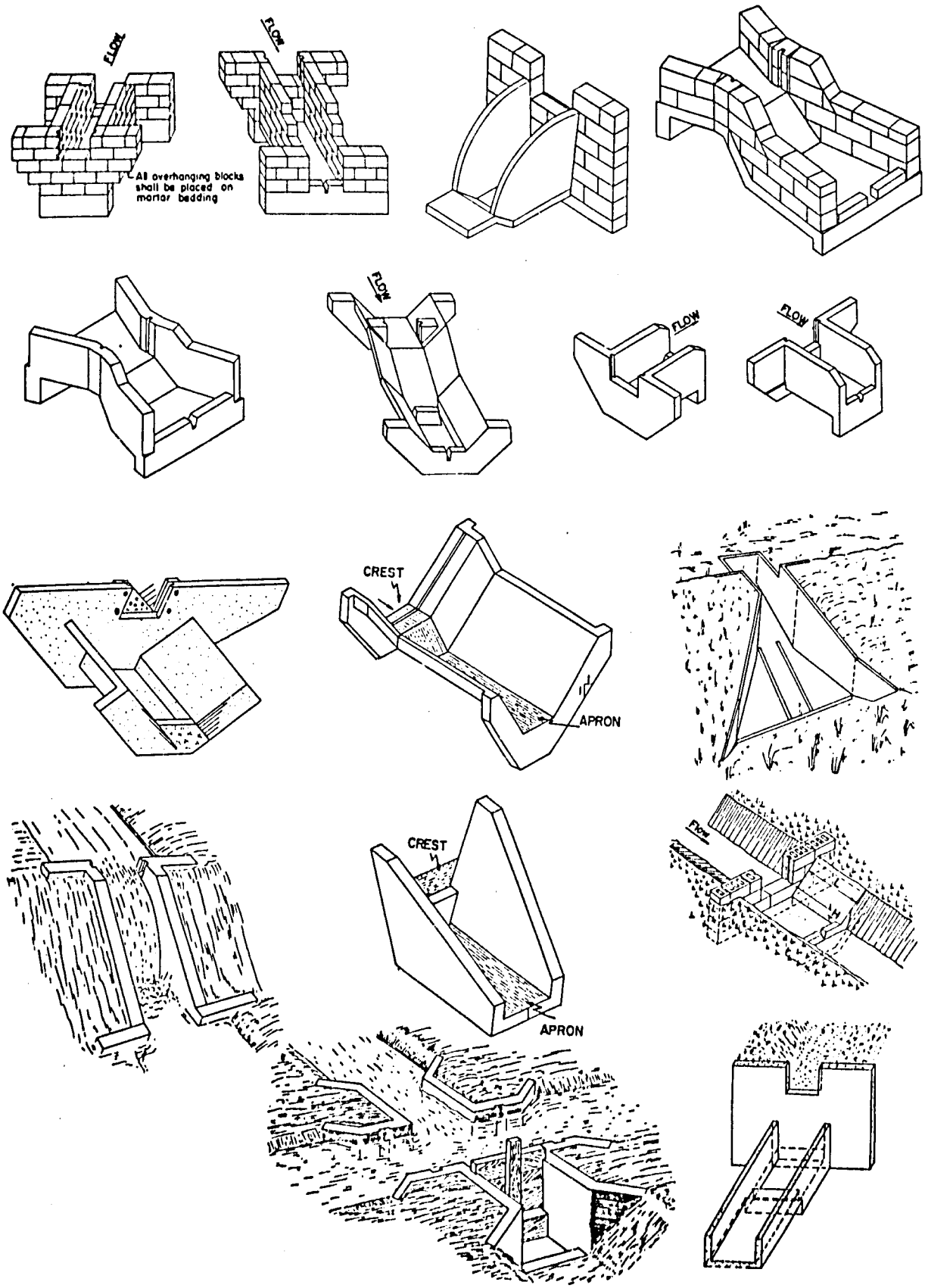


Figure 7.11. Drop structures for small channels.

erosion upstream from the structure. The overflow width and bottom elevation can be designed from weir formulas (USBR, 1967). The crest height should be below the upstream designed water surface elevation by the predicted head on the overflow weir. The head on the weir can be estimated by:

$$H = C_W \ (Q/L_W)^{2/3} \tag{24}$$

Where:

H = the head on the weir (m),

C_W = a weir coefficient depending on geometry,

Q = the flow over the weir (m^3/sec), and

L_W = the width of the weir opening (m).

The coefficient, C_W, will vary from 0.67 for a fully contracted sharp-crested weir to 0.70 for a suppressed broad-crested overfall. The bulkhead into which the overfall weir is built should be large enough to prevent seepage around the structure which could quickly wash it out.

The length of the protective apron downstream will depend on the length of the fall and should generally be at least twice as long as the fall height but not less than 0.8 m. Rip rap (large gravel) can be used in place of an impermeable lined apron.

It is often convenient to combine check and drop structures using the open check as the overfall weir. The check opening again must be set high enough that the water level upstream does not draw down. Small drops (less than 10 cm) can often be accomplished by only using small check structures through which the head loss at the design flow rate is equal to the desired drop (see Figure 6.23).

7.3 Water measurement structures

Efficient irrigation requires knowledge of the amount of water being applied which in turn requires the use of flow measurement structures. If water measurement is to be part of a comprehensive water management program, it is usually preferable to install permanent devices in the conveyance system which can be used to measure flows. The numbers and locations of these installations will depend upon the program needs as well as the available slopes in the system since simple and accurate flow measurement devices require significant head loss.

The various flow measurement devices available for small open channels, as well as the advantages and disadvantages of each, is described in Section 3.1.1. As permanent structures,

flow measurement devices can often be combined with other control structures. For example, a drop structure can easily be designed as a standard weir, or regular drop structures can often be calibrated as flow measurement devices. Likewise, orifice or pipe outlets or checks, if they are submerged and create sufficient head loss, can be calibrated to measure flow. Often the additional cost of redesigning control structures so they can be used to measure flow is small if sufficient slope is available in the system.

When channel gradients are small and other structures cannot be used to measure flows, flumes are probably the best choice as permanent flow measurement devices since they can operate with minimal head loss, are accurate over a fairly large flow range, and seldom have problems with sediment deposits or clogging with trash. Cutthroat flumes (Skogerboe et al., 1973) are easy to construct and install as permanent installations. They have a flat bottom which can be poured-in-place concrete or a flat precast concrete plate. The sides which have only one bend can be precast and set on the bottom or poured in place from simple forms. Figure 7.12 shows a permanent concrete Cutthroat flume installation.

The elevation at which flumes or any other measurement device are set is often a critical and difficult to determine factor, especially in low gradient systems. As discussed in the last chapter, variations in channel vegetation (roughness), inflow rates, or field elevations can cause a significant fluctuation in channel flow depths which can submerge an otherwise properly operating device. Likewise, setting a device too high in a low gradient system will require elevated channels and often create channel storage which farmers dislike. In Pakistan, farmers often removed permanent flumes because they felt they were "holding back" their water, even though engineers tried to convince them that, once the channel was flowing steadily, the devices did not really decrease their water supplies.

In order for measurement devices to be useful to irrigators, they must be easy to use. This can be accomplished in unsubmerged devices by calibrating the upstream staff gage directly in units which are understandable to the irrigators, for example, in ha-cm/hr (ac-in/hr). In submerged devices such as orifices, this simplification is not possible since a head loss reading is necessary requiring two gage readings plus a chart to convert to flow rate.

7.4 Community Structures

7.4.1 Culverts

Irrigation channels often intersect established paths or travel routes and thus hinder the movement of people, animals, and vehicles. Culverts are required to minimize this hindrance especially if the channel is wide. By concentrating the passage

Figure 7.12. Permanent Cutthroat flume installation with poured concrete floor and precast concrete sides.

to certain areas, the deterioration of the channel banks can also be reduced. In very heavily trafficked areas such as through villages, it may be desirable to line the channel utilizing a cross section with a narrow top width or even burying it in a pipeline so that passage is not hindered.

Reinforced concrete pipe, if available, will often be the cheapest form of culvert. The diameter of the pipe chosen will depend upon the allowable head loss. Head loss through short pipes such as are used in culverts can be estimated by Figure 6.23. Head loss is least in culverts when they are flowing about 80 percent full. Culverts should not be set to flow less than 80 percent full so they can be installed as low as possible thus maximizing the depth of soil covering and minimizing the chance of breakage. Setting culvert pipe low will also reduce the size of the hump which often must be crossed at the culvert.

Pipe wall thicknesses should be designed to provide sufficient strength to insure against breakage. The thicker the soil covering, the more evenly weight is distributed to the pipe, and the less pipe strength is required. At least 15 cm, and preferably 30 cm of soil should cover any culvert used for vehicular traffic. Preparing an even, rock-free bed for the pipe and compacting the replaced soil at the sides will reduce the chance of breakage as well as the possibility of water washing out soil along the pipe.

Culvert lengths will depend upon the usage. Masonry or concrete walls at the ends of the pipe will hold the soil covering in place and prevent driving over and chipping or breaking the ends. Figure 7.13 shows a pipe culvert with wing walls. Another alternative which may be lower cost and easier to install would be to increase the pipe length by about two diameters and allow the soil to just slope down at the ends. An alternative to pipe culverts for major crossings are masonry culverts such as shown in Figure 7.14. The cap of these culverts could be a brick arch, concrete or rock slabs, or a half concrete pipe.

For smaller culverts where only people or animals will cross, wood boards or concrete slabs laid across the bank will often be a lower cost alternative to pipe or masonry culverts.

7.4.2 Washing stations

In some areas, the water conveyed in irrigation channels is used by the community for purposes other than irrigating crops. For example, village women may wash clothes in the flowing water or may utilize it as a household water supply. The water may serve as a drinking supply for animals, or the animals may be bathed in the cooling flow. An irrigation

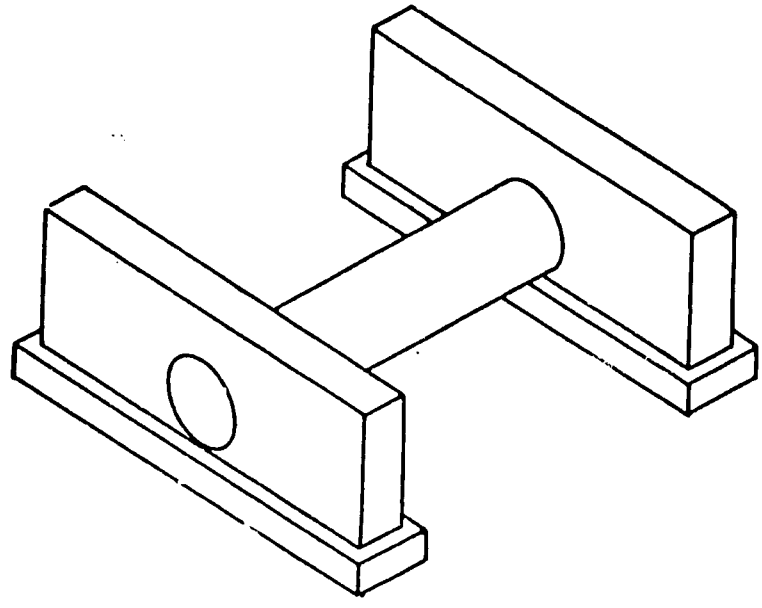


Figure 7.13. Concrete pipe culvert with wing walls.

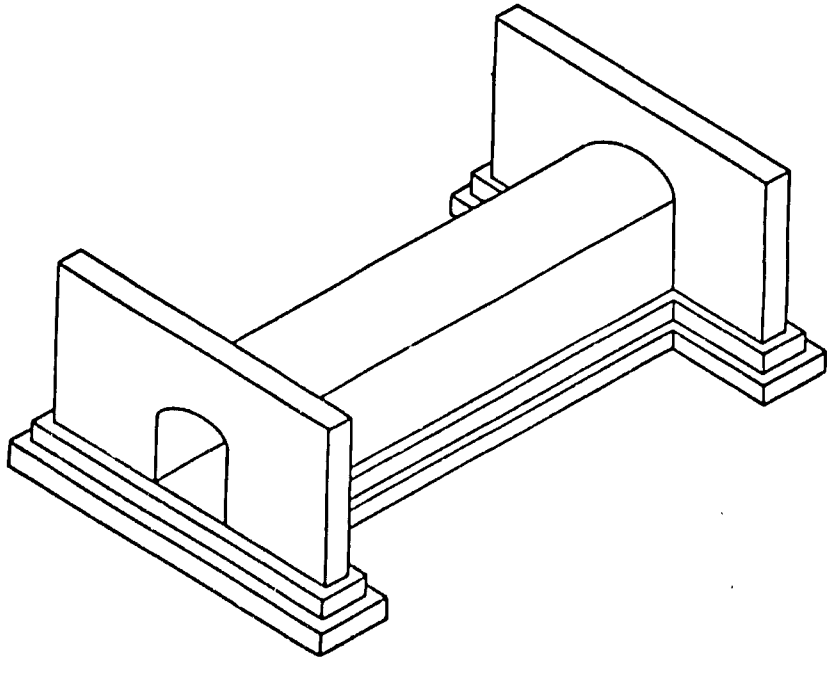


Figure 7.14. Masonry culvert with wing walls.

channel improvement program which provides for these conjunctive community uses will be more widely and enthusiastically accepted especially by those villagers who own no land and thus derive no other benefits from the improvements. Providing permanent locations and structures for these needs can also concentrate these activities at designated locations and reduce deterioration of unprotected channel banks.

Clothes washing stations composed of a short section of lined or partially lined channel with a durable concrete or masonry pad on top of one bank will provide both easier access to the water and a working surface. Reducing the freeboard at the pad to about 10 cm will make access to the water easier. Washing pads would be inexpensive to provide at otherwise lined sections. The potential users should be consulted concerning their specific needs before washing station locations are chosen.

7.4.3 Animal baths

Animal drinking and bathing stations can be anything from an enlarged channel section where the activity is concentrated to a concrete or masonry basin in or at the side of a channel with entrance ramps. A practical medium cost design is to line the two sides of a widened channel section which are perpendicular to the flow direction to prevent the section from lengthening and reduce the wetted perimeter, and add coarse sand to the bed to prevent erosion due to the movement of the animals. If it is desired to not place the tank directly in the flowing water, water can be circulated through an adjoining tank by connecting the tank with the channel with two submerged pipes and creating a small head loss in the channel between the two pipe inlets. Figure 7.15 shows a buffalo bath with three brick masonry walls and a sand ramp.

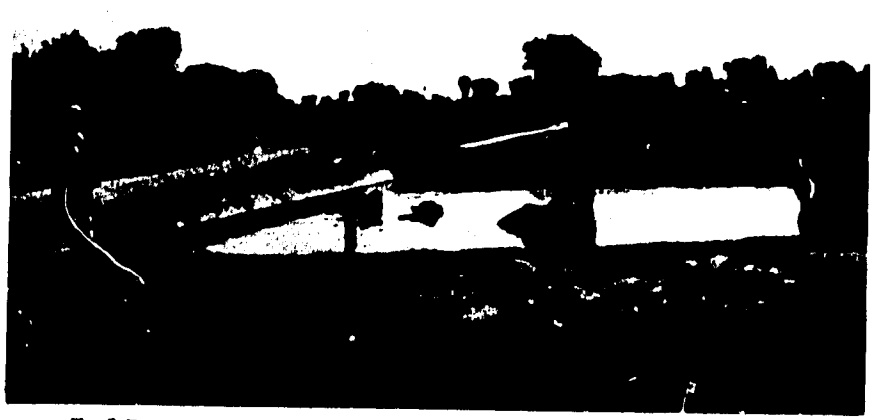


Figure 7.15. Buffalo bathing pond at the side of a watercourse.

SECTION 8

IRRIGATION CHANNEL RENOVATION STRATEGIES

Possible techniques to reduce water losses from small conveyance systems range from the cleaning of vegetation from the banks to complete channel lining. Which technique is chosen for a given section will depend upon the amount and types of losses diagnosed, the cost of the program and the benefits derivable from the saved water, the resources (personnel, material, and financial) available, the priorities set, and the time available. Of the continuum of possible channel improvement techniques, three general types will be discussed. They are:

1. cleaning and repair,
2. earthen renovation, and
3. channel lining.

These examples represent progressively increasing cost and potential water-saving alternatives.

The channel renovation strategy chosen will often involve mixtures of techniques applied at different times and to different systems or portions of one system. Methods will be described to optimally apply different techniques to a system and to evaluate the improvement strategies.

8.1 Irrigation Channel Improvement Techniques

8.1.1 Channel cleaning and repair

Channel cleaning and repair involves stopping the easiest-to-control forms of channel losses with a minimum amount of initial capital inputs. The potential water savings will depend upon the types of losses diagnosed (Section 4). Channel repair includes plugging holes to stop visible leaks, raising banks to stop overtopping, strengthening thin banks to prevent washouts, improving deteriorated junction areas to strengthen banks and prevent leaks and washouts, smoothing out channel bottom fluctuations to reduce dead storage, and removing excessive vegetation from the banks which hinder cleaning, maintenance, and monitoring operations. Repair could also include compacting earthen cores in the banks (Section 5.1) to reduce excessive seepage through the upper banks. Cleaning conveyance channels includes both the cleaning of vegetation from the wetted perimeter and removing excess accumulated silt from the bed, both of which hinder water flow, increase flow depths, and increase seepage losses into the upper porous banks, as was explained in Section 5.2.

Cleaning and repair involves no redesign of channel elevations and cross sections and thus requires no engineering input. The program could be administered through an existing or bolstered extension program and requires minimal retraining of technicians or other personnel. Extension agents who have seen or participated in cleaning and repair programs on other systems and who have a basic idea of the causes of water losses and the effects of channel cleaning and repair on losses, could introduce the program to farmers.

The program does not include the installation of permanent structures or lining and thus the only construction costs are for labor. Although labor could conceivably be hired directly by the government or through contractors, because the labor requirement is ongoing and must be available to meet channel maintenance needs, it is strongly recommended that the farmers supply the labor. By supplying the labor and noting the changes in water delivery as a result of the effort, farmers become more aware of the importance of maintenance and of their ability to improve their own water supply. With farmer-supplied labor, low training needs, and a program that is sufficiently basic and simple that it can be administered and supervised by existing agencies, a cleaning and maintenance program can be low cost and quickly applied to a wide area.

The specific procedure to follow in undertaking a cleaning and repair program will depend upon the problems diagnosed. Each channel should be observed while operating to note visible leaks, locations where the freeboard is inadequate or banks are thin, junctions where leakage is a problem and banks are deteriorated, obstacles to flow, and excess vegetation on the banks which hinders monitoring activities. While the channels are draining after use, depressions in the bottom or accumulated silt piles which cause dead storage should be noted. Also, the length of vegetation in the channel should be noted to determine whether cleaning is required. Losses should be measured in the channels. This diagnosis process should be carried out cooperatively with the farmers.

Once the losses are measured and some of the problems are diagnosed, this information should be shared with the farmers. If the farmers are convinced that they are losing a significant portion of a limited water supply and that they can, through cleaning and repair, reduce these losses, they will be more likely to support and provide labor for an improvement effort. The mechanics of carrying out the work will depend upon the local social structures and traditions. All work could be carried out cooperatively or the system could be divided into units for which individuals are given responsibility. Some factors to consider in organizing farmers for a cooperative effort are discussed in Section 9.1.

How much water can be saved through channel cleaning and repair will depend upon the amount and nature of the losses. In Pakistan, conveyance losses were reduced from 3.9 percent per 100 m before cleaning and repair to 1.4 percent/100 m immediately following the work on one set of watercourses which had high loss rates. Another study showed that just cleaning the vegetation from channels could reduce loss rates to 25 percent of their pre-cleaning value. A conservative estimate of how much water can be saved by a good and regular cleaning and repair program would be 30 percent of the losses. The actual savings should be determined by actual measurements of loss before and after improvement.

Although the potential savings of this simple program may be large, but the benefits are of limited duration. If vegetation growth is rapid, the insects and rodents are active, and earthen junctions again deteriorate rapidly, half of the benefits of the improvement can be lost within a few weeks. Consequently continued benefits require regular recleaning of the channels and periodic repair. Figures 5.9 and 5.10 show an example of the variation in water losses under various cleaning schedules. A cleaning and repair program consequently must be an ongoing effort if it is to achieve continued benefits. The main responsibility of the government agent will be to instill in the farmers the need for ongoing maintenance.

A primary requirement for ongoing maintenance is a signal that tells the cooperators when it is time to do the job again. Seasonal changes in sediment load and vegetative growth and other factors change the rate of deterioration so that cleaning at regular time intervals is generally not the best solution. The loss rate can provide the signal if structures are available to measure flow at the head and tail and farmers or government technicians are trained and available to take regular measurements and announce when cleaning and repair are needed. The strong correlation between loss rate and elevation of the water surface in the watercourse discussed in Section 5.2 allows the level of water in the watercourse to provide a good signal of when the watercourse needs cleaning. Markers on control structures or culverts can indicate the designed water levels at which losses are normal and higher levels at which losses are unacceptably high. Then when the level of the water reaches the higher levels the farmers know that cleaning and maintenance should take place.

Economic values of water and labor are also a primary factor determining when cleaning should take place. During certain seasons (e.g. harvest of the primary crop) the value of labor will be extremely high and will often exceed the value of lost water. So the best economic decision will be to lose the extra water until the harvest is completed and the value of labor falls below the value of water being lost. The "best" cleaning and repair schedule will take into consideration both the

availability of labor, the water savings that can be achieved and the seasonal value of that water. In most cases the farmers themselves will be best qualified to set the cleaning schedule. However, in many cases they need the flow measurements or elevation markers to indicate loss rates so that they have a clear signal that the irrigation channels need attention. Since some of the farmers tend to resist the call to work for the common good, leaders responsible for organizing the cleaning and repair generally appreciate clear signals as to when it is needed so they have objective backing for their call to work. If he has gained credibility with the farmers the government agent can assist the leaders by helping them make and interpret the measurements and backing their call to work. The farmers' leaders can then set the date for cleaning and repair depending on when labor will be available. Local water users associations of some type (Section 9.1) which give local leaders authority to issue the call to work will be needed to carry on the program.

In spite of the regular required reapplication, a labor-intensive cleaning and maintenance program, when organized and sustained, showed the highest benefit:cost ratio (although not the highest net returns) of the three presented alternatives in Pakistan.

8.1.2 Earthen renovation

Earthen renovation of small channels involves the complete destruction of old channel banks and reconstruction to specifications based upon hydraulic design, and the installation of permanent structures at junctions and major outlets. This technique falls between the cleaning and repair program just described and channel lining in both costs and water-saving potential. Even if farmers supply the manual labor, the program still requires significantly more time to complete than the cleaning and repair program because of more extensive earth work, the installation of structures, and the need for trained personnel to carry out the design work and oversee the construction.

Diagnostic procedures such as those described in Section 4 will indicate whether earthen renovation has a potential to save a significant portion of the losses. If much of the loss is avoidable (i.e., leakage and excess seepage), earthen renovation will be a good investment in terms of program costs and water savings.

As with the cleaning and repair program, the mechanics of carrying out channel renovation will depend upon the local conditions, although some of the processes can be generalized. The first step will probably involve the removal of trees and large vegetation from the old banks. The banks can then be destroyed either manually or with the help of bulldozers, tractor blades, or animal-drawn scrapers. The organic and vegetation covered bank soils should not be reused in the new

banks because they will not compact well, the decaying vegetation will leave pore spaces, the organic matter will attract insects and animals, and the live roots may begin to grow again. The old bank soils should be used to fill the pits from which soil is borrowed to make the new banks or spread on adjoining fields where the organic matter will benefit crop production. Likewise, the soil brought to construct the banks should be free of vegetation and trash. This may require scraping surface vegetation from an area before excavating the soil to be used for bank construction. Figure 8.1 shows a suggested procedure for manual reconstruction of earthen banks, assuming only shovels and hand tampers are used. Figure 8.2 shows a similar procedure for achieving the same ends when tractor-powered equipment is available such as blades, scrapers, V-ditchers, and compactors.

A difficult but most important part of the earthen renovation process is the proper compaction of the banks. The benefits of compaction in water-loss reduction is discussed in Section 5.1.1. The benefits of extended life of the channel is more difficult to quantify, but significant. Compaction can be carried out by many different methods including the feet of men or animals, manual or power tampers, tractor tires, or tractor- or animal-drawn small sheepsfoot rollers. As explained in Section 5, the soil should be near field capacity for proper compaction. Field capacity is the moisture content a soil will reach after it has been irrigated and allowed to drain for a couple of days. If the soil to be used is too dry, it should be wetted. This can be accomplished by irrigating the borrow area, or the pad, a couple of days before excavating the soil. The time between wetting and compacting will depend upon the soil texture (sandy soils will drain in one day) and evaporation rates. If the soil is too wet for easy working, it is also too wet for proper compaction and should be allowed to drain longer. Properly moist soil will be easier to work than dry or wet soil. Once the excavation and compaction process is begun, it should be completed before the soil dries out. Preparing and working with soils at the proper moisture content will require proper planning and an organized effort, but can increase the rate at which construction proceeds.

The compaction should always take place before layers of loose soil exceed 10 to 15 cm thick depending upon the soil and compaction force, since the full effect of a compaction force affects only the surface soil layer and decreases rapidly with depth. Figures 8.1 and 8.2 demonstrate bank construction in layers. Because compaction decreases the volume of a given amount of soil, more soil must be brought to the channel site than would be calculated from the bank cross-sectional area, and the banks must be initially built higher than is recommended in the design to allow for compaction shrinkage.

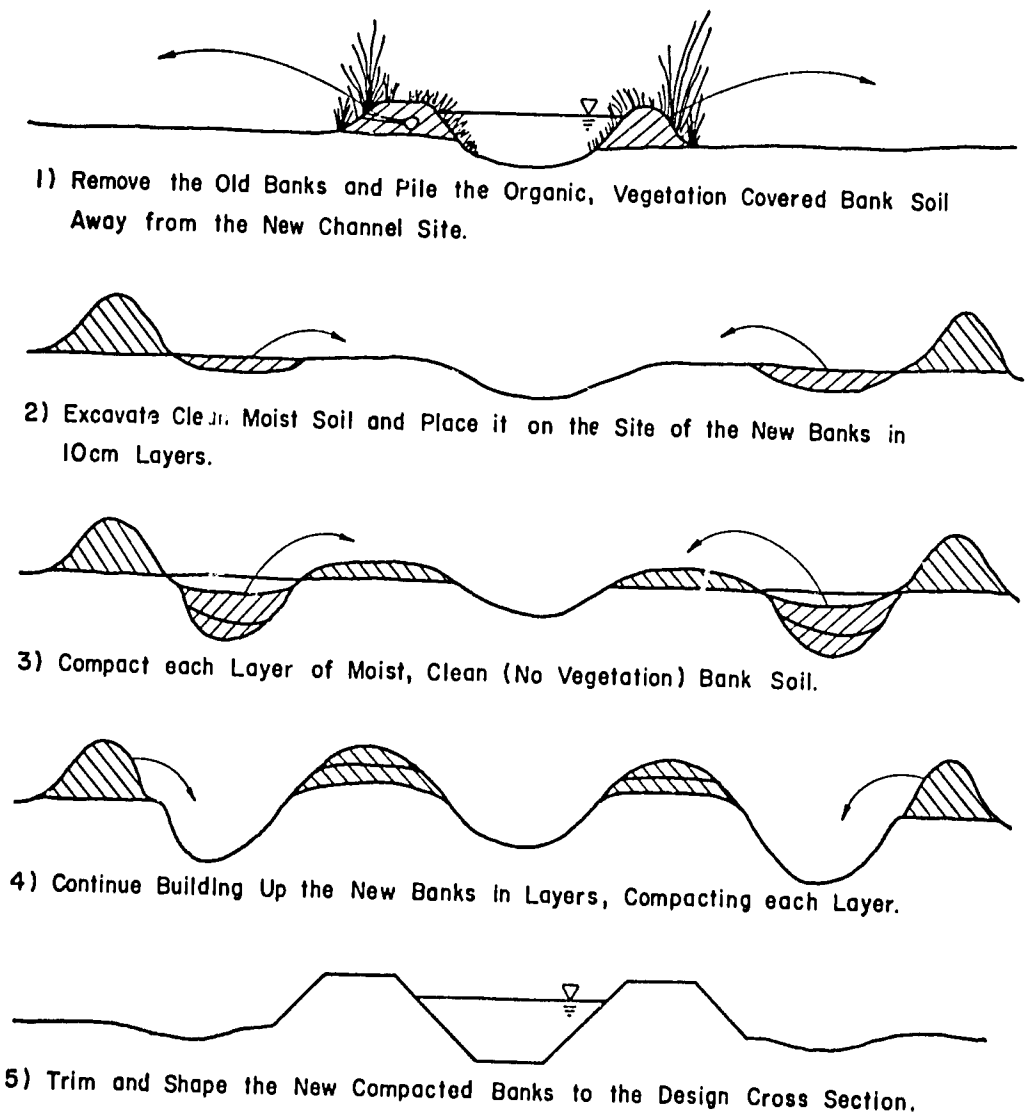


Figure 8.1. Suggested procedure for manual reconstruction of earthen channels.

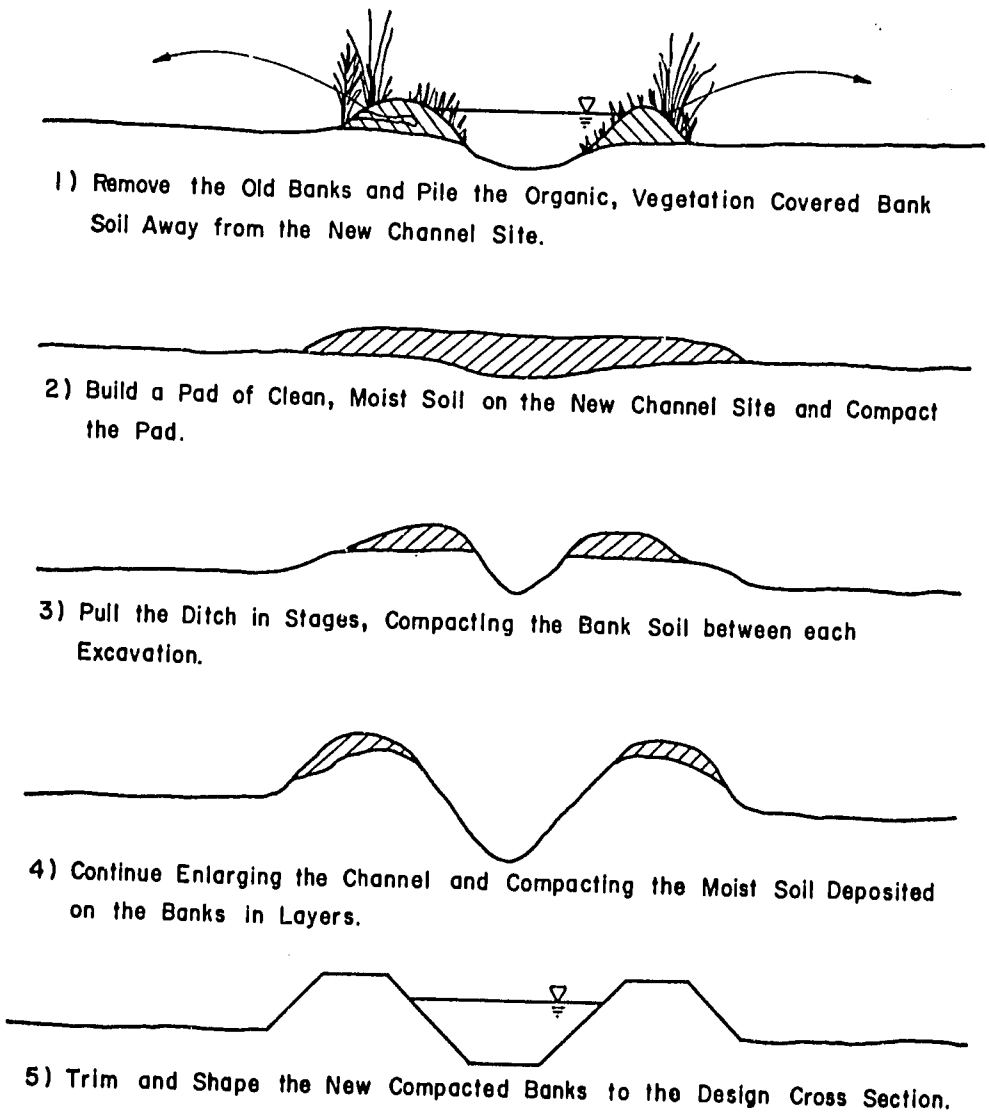


Figure 8.2. Suggested procedure for mechanical reconstruction of earthen channels with a tractor drawn scraper, ditcher, and compactor.

Completing the earth work before the structures are built will generally be easier, especially if mechanization is used. It is not difficult to excavate a short section of new bank to install a structure. As stated in Section 7.1 recompacting the soil around a newly installed structure to prevent piping leaks from forming is very important. A hand tamper (Figure 5.4) is the best tool for this process.

Proper elevations and cross sections for the rebuilt channels will be listed on design sheets described in Section 6.3. Using this information, a surveyor's level, and the original topographic survey bench marks, stakes can be placed to indicate the required bed and bank levels and widths. This staking process is best done roughly at first so that pad elevations can be established and a sufficient amount of soil is brought to the site. During final bank trimming and shaping, more elaborate and precise staking will be required. Adjustable cross-sectional guide forms (Section 6.1) can simplify this final shaping process.

The process of organizing and carrying out an earthen channel renovation will depend on local traditions and preferences. Labor can be provided by the government, hired through contractors, supplied by farmers, or can be the farmers themselves. Work can be done manually or with machinery, or any combination of the two. Supervision can be totally provided by government agents, farmers can cooperate in the decision making and dispute-settling process or the farmers can take primary responsibility with the government engineers acting only as technical advisors. The next section (Section 9) discusses several of these organizational aspects. A brief description of the watercourse renovation process developed in the Pakistan On-Farm Water Management Pilot Project will give an example of one possible procedure. Figure 8.3 illustrates the renovation process.

After a preliminary survey of the watercourse system, project agents approach the farmers who use the conveyance system and explain their program. They describe some of the problems with the channel system and how much water is being lost, and tell how earthen renovation can solve the problems and save water. They often show the farmers other watercourses which have been previously improved and let them discuss the program with those farmers.

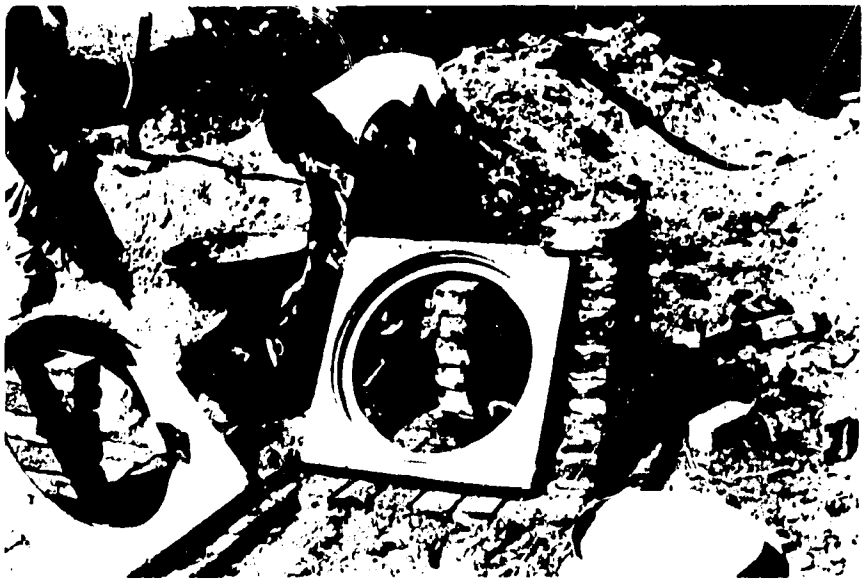
If the farmers are sufficiently convinced of the benefits, they sign a petition (at least 75 percent must sign) requesting the Project to help them renovate their channels and agreeing to provide the required labor and masons. The farmers then elect an executive committee to act as the intermediary between themselves and the project engineers. The executive committee is also responsible for arranging the required labor, settling disputes between farmers, and making some decisions on design options and working procedures.

(a) Discussing the watercourse losses and the renovation process with farmers.

(b) Farmers destroying the old channel banks.

(c) Constructing the new banks under the engineer's supervision.

Figure 8.3. The watercourse earthen renovation process.



(d) The village mason installing the outlet structures.

(e) The engineer checking the finished channel.

(f) A finished renovated watercourse.

Figure 8.3. The watercourse earthen renovation process. (Cont'd.)

The first responsibility of the committee is to see that all trees and large vegetation are removed from the right-of-way of the channel sections chosen for renovation. While the trees are being removed, the government technicians survey the area, make topographic maps, and design the new channels. Once tree removal and channel design work is completed, the earth work begins. The work is concentrated in specific sections to minimize engineer supervision time. Farmers choose the work schedule. Minimizing the disruption to their irrigation water supply is generally the primary concern. The executive committee distributes the work to the landowners, usually based on the percent of the total commanded land owned by each.

The farmers are given the option of choosing the method of completing the earth work. If they have access to a tractor and all are willing to contribute towards providing fuel for it, they will often choose to use a scraper and ditcher loaned by the Project to assist and speed up the earth-moving process. Final shaping is always completed manually. A project engineer provides all channel elevations and alignments and supervises the construction.

The executive committee along with the project engineer decides where structures are required. The Project provides the structures and installation materials, the farmers transport the materials to the chosen locations, and a mason provided by the farmers and trained by the engineer installs them.

When the farmers sign the channel improvement agreement, they agree to maintain their improved channels. An extension agent is assigned to the watercourse to train the farmers in proper maintenance techniques as well as to educate them to use their improved water supply more efficiently. The intent is that the executive committee will remain active after construction is complete and will organize and supervise cleaning and maintenance work and solve future disputes between the water users.

Earthen renovation should save all of the losses which cleaning and repair can save, plus additional losses associated with porous vegetation covered banks and uneven, irregular channels. Rebuilding banks will eliminate excess seepage resulting from insect and animal burrows. Good compaction of rebuilt banks can significantly reduce bank porosity and permeability and discourage the renewed activity of these pests. Complete removal of bank vegetation will eliminate flow paths resulting from decaying roots, reduce bank drying and subsequent wetting during channel filling, and eliminate the cover and food supply for much of the animal life which lives and burrows in the banks. The thicker, higher banks reduce possibilities of bank washouts and overtopping, and make monitoring operations easier. Uniform cross sections can reduce roughness coefficients and wetted perimeter lengths and thus seepage area.

The uniform bank side slopes collect more sediment on the upper portions than is collected on commonly found parabolic sections which can also lead to lower seepage rates. Permanent structures can reduce outlet leakage and washouts. Measurement of conveyance losses in renovated channels will determine the actual benefits. In Pakistan, about 50 percent of the channel losses were saved with earthen renovation.

In addition to water savings, the earthen renovation process results in channels which are easier to monitor, easier to clean and maintain, and which can act as paths on which farmers can easily move among their fields. The structures make the control of water easier and quicker, allowing more timely water control with less labor. Designed working heads make it easier for the farmers to irrigate their fields and helps distribute the water more evenly among fields. The economic value of these additional benefits of channel renovation are often difficult to measure, but are important to farmers.

Although earthen renovated channels will retain their improvement longer than those which are only cleaned and repaired, many of the benefits are still temporary, and continual maintenance is critical to maintaining the increased water supply. Figure 8.4 shows a hypothetical example of how the benefits of earthen channel renovation might decrease with time under a good and poor maintenance program. Under poor maintenance, much of the improvement benefits may be gone in as little as three years, although some increase in delivery efficiencies will continue indefinitely. A good maintenance program implies regular cleaning (perhaps bimonthly) of wetted perimeter vegetation and keeping the banks clean of trees, bushes, and weeds; controlling any erosion and cleaning out excess sediment deposits; reparing any degraded bank sections resulting from sluffing, human or animal traffic, or soil borrowing; maintaining designed channel cross sections; firmly plugging visible cracks or holes in the banks; and repairing or replacing cracked or chipped outlet structures. Maintenance could also include a program to control insects and animals which inhabit the banks. Even with good maintenance, the losses would be expected to increase as the bank inhabitants return and continue their burrowing activities, and it is expected that an earthen renovation program will need to be repeated perhaps each 10 years. Good bank compaction will be, other than maintenance, the most effective method of extending the improvement life. Because of the decrease of delivery efficiencies over time, a sample of the improved channel systems should be regularly monitored for conveyance losses to better determine the effectiveness of various maintenance activities, to establish the effective life of earthen renovated channels, and to better define the benefits of the improvement process. Permanently installed flow measurement devices will facilitate such monitoring.

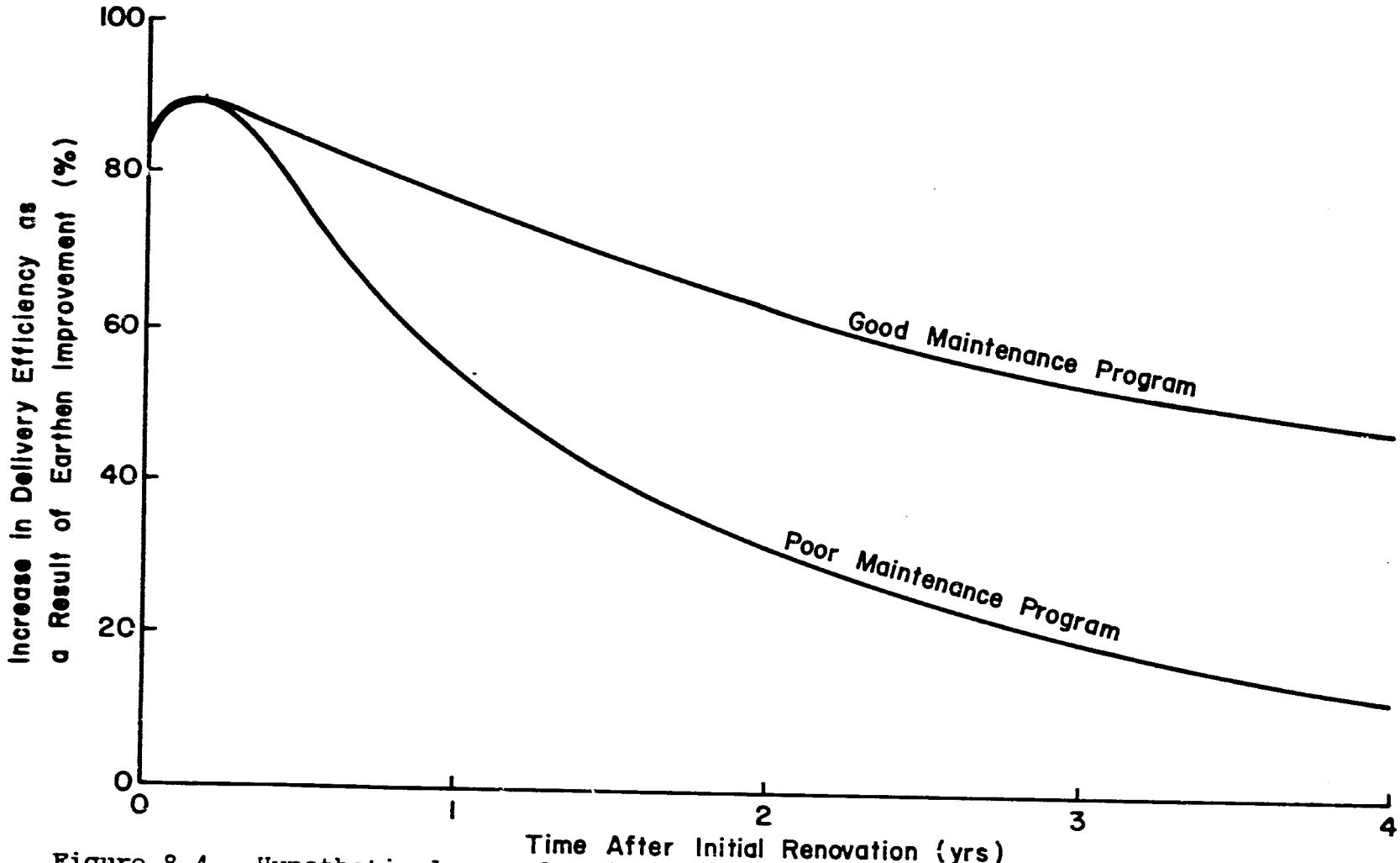


Figure 8.4. Hypothetical example of the decrease of water savings from earthen renovation with time with a good and poor maintenance program.

8.1.3 Channel lining

Channel lining is a high cost improvement technique which can achieve high delivery efficiency. Lining can often be justified when:

1. high cash value crops are grown and water is a limiting input,
2. water seepage rates are high due to coarse textured soils,
3. loss rates are high due to uncontrollable insect or animal burrowing,
4. usage time of main channel sections is high,
5. the channel passes through a village or other high traffic area and would otherwise deteriorate rapidly,
6. maintenance costs of earthen channels are high, or
7. low cost lining materials are available.

Extensive lining programs will usually require:

1. high capital input,
2. large amounts of materials,
3. a large supply of trained designers and construction supervisors, and
4. a significant amount of time to complete the project.

Channel lining can potentially eliminate all conveyance losses except evaporation, outlet, and wastage losses.

Lining techniques which have been used in different parts of the world, for small channels include: buried concrete pipelines, trapezoidal channels of poured concrete, rectangular and trapezoidal brick or stone masonry, precast concrete or soil cement block channels, semicircular channels of precast concrete, parabolic sprayed concrete or asphalt channels, and concrete, brick masonry, or soil cement block partial (side) linings. Figure 8.5 depicts cross sections of some of these lining techniques. Kraatz (1977) discusses the design and construction of many types of linings for small channels.

The choice of lining material will depend primarily on the local cost, availability of materials and the familiarity of local craftsmen with the materials. If cement, aggregate and sand are relatively cheap, concrete is generally a good choice

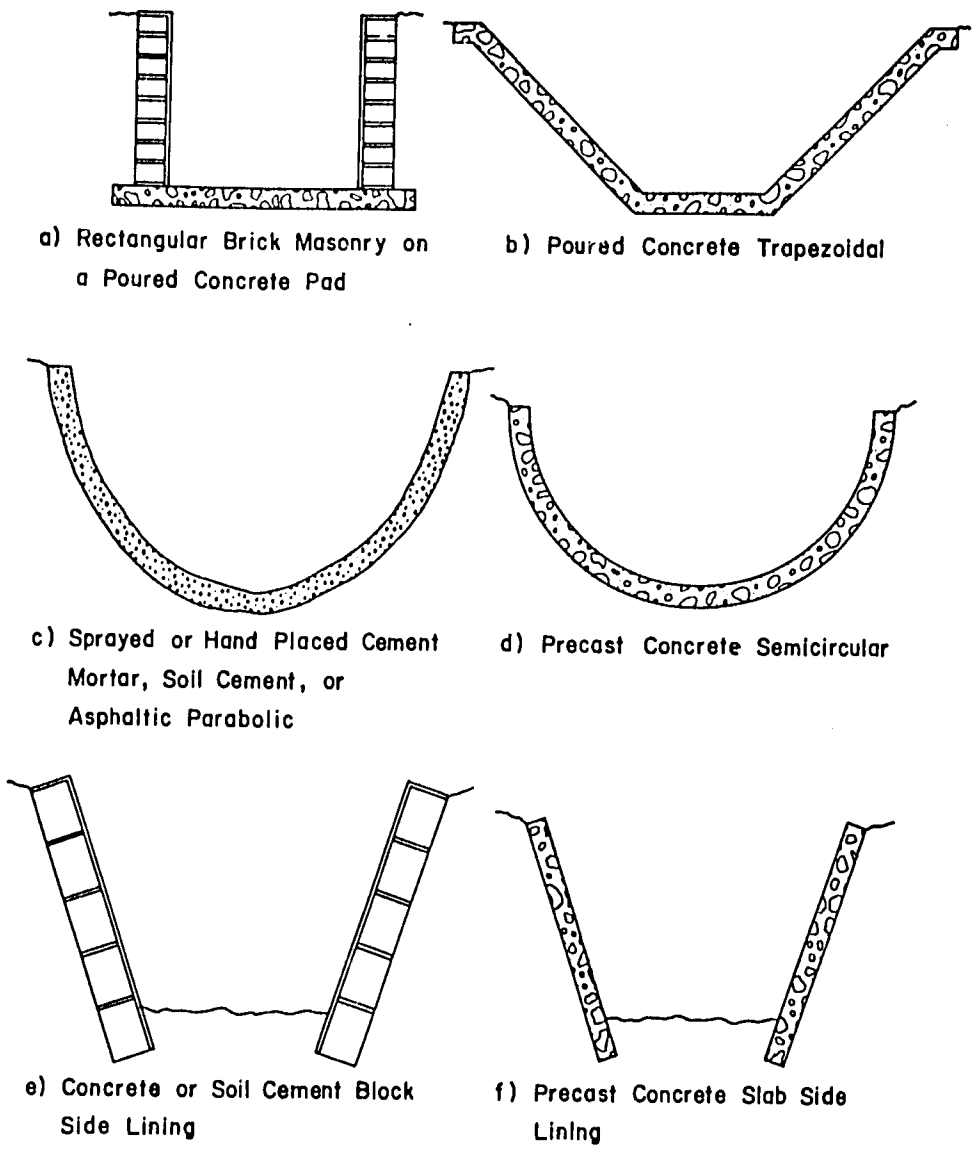


Figure 8.5. Types of small channel linings.

as a lining material. If materials are expensive but labor is cheap, precasting concrete shapes, slabs, or blocks which uses less material but requires more labor might be a less expensive alternative. If aggregate is not available but cement is cheap, soil cement may be the best choice. If a local fired brick industry produces cheap bricks or if construction stone is locally available, and on-site masons are inexpensive, brick or stone masonry should be considered. If cement is expensive and petroleum products cheap, asphalt may be the cheapest alternative material.

Once the material is chosen, the cross-sectional shape should be chosen which will fulfill the purpose and minimize costs. The lowest cost shape, material wise (that having the shortest wetted perimeter length), is the semicircle. However, a semicircle is generally difficult to construct unless precast as half concrete pipe. Rectangular shapes require a relatively longer lining cross section but are the easiest shapes to construct if bricks or blocks are used. Trapezoidal shapes require less material than rectangular shapes both because wetted perimeter lengths are shorter and because less wall thickness may be required. Less wall thickness is often possible because the weight of the conveyed water will be distributed to the underlying soils if they are well compacted, and the back pressure on the walls of an empty trapezoidal channel are countered by the weight of the sloping lining. Rectangular channels must be structurally strong enough to resist back pressures. Appendix 8 describes techniques for minimizing the costs of channel lining by choosing efficient cross sections.

As discussed in the previous section, many small channels have fairly impervious beds but highly permeable banks. If ponding loss measurements show this to be the case (indicated by very low loss rates when the water level is low) a low-cost lining alternative would be to line only the channel sides and leave the bed as compacted earth. Examples of side linings are shown in Figure 8.5. Figure 8.6 shows brick masonry side linings being constructed.

Since side linings, being unconnected, have no inherent structural strength, their stability will depend upon compacted bank soils to support the weight of the walls and water when the channel is full, and the weight of the sloping walls to resist the back pressure of saturated bank soils or possible traffic on the banks when the channel is empty. Adequate soil compaction is thus more critical in the construction of partial linings.

Because there is no lining on the bottom, minimizing costs for side lined channels would result in an infinitely wide and shallow channel. In practical terms, assuming the compacted and silt-covered beds exhibit very low seepage rates, such linings should tend to be wide and shallow to reduce lining

Figure 8.6. Brick masonry side linings being constructed.

costs. Shallower depths also result in less stress on the side linings. Steeper side slopes (lower Z values) will also reduce side lining costs, so the lowest Z value for which the walls exert sufficient force to resist soil back pressures should be used. Side slope values of 0.3 to 0.5 are recommended. The sides should extend below the bottom of the channel by about 20 percent of their height to add stability and to reduce the possibility of bed erosion undermining the lining. If vegetation does not grow in the bed, roughness coefficients for the lining material can be used for the channel. If vegetation grows on the bed, more conservative (higher) n values must be used.

Construction of channel linings will begin with earth work. Except when rectangular cross sections are to be built, a compacted earthen channel must first be constructed upon which the lining is applied. Linings for rectangular channels are generally constructed on flat pads and the earthen support banks are added later.

Because costs of lined channels are high, they should be carefully engineered and supervised. If supervision can be shared between farmer representatives and the government agent, both will tend to keep the other more honest since the farmer's primary concern should be to get a good product, while the agent's might be to minimize costs, either to save or to make money.

Because large numbers of skilled masons and large amounts of materials are required, private contractors who are experienced in recruiting and supervising masons and purchasing and handling materials may be a desirable means to carry out the work. This is the method used in a large-scale lining program in northern India. An alternative would be for the farmers to supply all of the labor or at least to supply the unskilled labor. Regardless of who does the lining, the farmers could carry out the required earth work. When farmers are involved in the construction, supervision, or decision-making process, they are more likely to feel responsibility for and pride in the finished product and try harder to maintain it.

Channel lining should eliminate nearly all conveyance losses in the lined portions. However, improper material apportioning or mixing, inadequate soil foundation preparation and compaction, and poor construction techniques can result in significant seepage and leakage through cracks in linings. For example, in one experimental brick masonry lined system in Pakistan, improper mixing of mortar, insufficient cement usage, the tendency of masons to leave holes between bricks, and poor compaction of the underlying soil allowed leaks to form between bricks, cracks to form between joints, and in one case, a complete lined section to settle and collapse (Figure 8.7). Water loss rates nearly as high as those in unlined channels were measured, although later plastering of the masonry sections reduced losses to negligible levels. Consequently, it cannot be assumed that losses will be very low in lined sections. Loss rates should be measured, preferably by the ponding method (since loss rates would be expected to be low) both to determine whether the lining techniques are adequate and to evaluate the water savings.

As with any construction work, the lined channels must be maintained to extend their life and extract the maximum benefits. Lining maintenance includes patching holes, maintaining earth in support banks, raising the bank height in settled sections, replacing damaged structures, cleaning silt from inside the channel, and preventing vegetation growth in the channel or through the lining.

Figure 8.7. A collapsed brick masonry rectangular lined section.

8.2 Irrigation Channel Improvement Strategy

After potential channel renovation techniques are delineated, tested, and evaluated, they must be combined into an improvement strategy. The strategy will involve when and where each technique should be applied. Under varying conditions, one technique alone will not give the best benefit/cost ratio or utilize the available resources most efficiently, and thus combinations of methods will be preferable.

8.2.1 Time strategy

A renovation technique such as cleaning and repair can be applied quickly but has a short effective life. A channel lining program may provide the highest net returns but in most situations will require several years to complete. Because of these variations, the choice of strategy will often depend on time constraints.

For example, if the water supply to an irrigation system is predicted to be critically low during a cropping season (i.e. because of a low snowpack in the watershed) and a decision is made to increase water use efficiencies to save the planted crops, a technique such as cleaning and repair which can be applied quickly would be preferable. If, instead, the government decides to make a long-term investment in improving irrigation water supplies to farmers, a longer-term method such as earthen renovation or channel lining would be preferable.

If funds are available for long-term capital intensive improvements, but there are also needs for immediate increases in food production, the techniques can be applied consecutively. An immediate benefit can be gained by implementing a regular cleaning and repair program. Simultaneously, an earthen renovation or channel lining program can be begun with field tests and evaluations, technician training, and the start of a pilot project. As the long-term program spreads, it will supercede the short-term program. Since the life of the short-term technique is short, little will be foregone in its being followed by a more permanent improvement. This progression could pass through more than two stages, especially if there are capital constraints or the value of water is projected to increase. A cleaning and repair program could be superceded by an earthen renovation program which could, in turn, as the earthen improvements begin to lose their effectiveness, be gradually replaced by a channel lining program.

8.2.2 Regional strategy

Saved irrigation water will have different values in different regions or areas within one region. The value will vary with climate and crop values, and other factors such as groundwater pumping potential or whether salination or waterlogging is resulting from the water loss. The channel water losses and potential savings will also vary with regional variations in soils and topography. Consequently, the improvement technique which will achieve the greatest benefit will also vary.

For example, if an irrigated basin is underlain by some saline and some pumpable freshwater aquifers, the value of water saved will probably be higher above the saline aquifers, and higher cost and water-saving techniques will show higher benefits there making the higher cost techniques economically feasible. Likewise, if seepage from higher areas in a basin is salinizing lower lands, higher cost water saving techniques can be justified to avoid this extra cost of the water loss. In areas with sandy soils, channel lining may be necessary to reduce high seepage rates, while in other soils, earthen renovation might provide the desired savings.

Consequently, one improvement technique should not necessarily be applied uniformly to a country or even region

or basin, but the areas must be divided into zones with similar conditions and water values, and an improvement strategy must be decided for each zone. Even if, for the purpose of program simplification, a uniform strategy is chosen, regional evaluation will allow the strategy to be applied first to the areas where the needs and potential benefits are greatest. This ordering of application is especially important for programs which require large capital investments and long times for completion.

8.2.3 System strategy

The costs of a channel improvement program are generally proportional to the length improved, while the water savings and consequent benefits are proportional to both the length and the amount of time a channel is used. Likewise, the costs of a lining program generally decrease as the channel capacities (size) decrease, but not as quickly as the loss rates decrease with capacity. Consequently, the benefit/cost ratio of an improvement technique will vary within conveyance systems depending on the total channel usage time in a rotational system, and the design flow rates in a constant flow system where the supply is progressively subdivided.

A hypothetical example shown in Figure 8.8 demonstrates how the benefit/cost ratio can vary in a system where flows are subdivided. In the example, 120 lps flows into a system but is constantly subdivided until the field branches only receive about 10 lps. Loss rates are assumed to decrease 80 percent as fast as the flow rates, which is the relationship measured in Pakistan watercourses. Since the benefits are generally proportional to the reduction in loss rates, the potential benefits of saving a certain portion of the losses similarly decreases in the smaller channels.

Likewise, the improvement costs will be less in smaller channels but will generally not decrease as quickly as the potential water savings. Lining costs are generally proprotional to the wetted perimeter length which, in efficiently designed channels, decreases about 3/8 as fast as channel capacity (as indicated by the exponents on the equations given in Table A.8.1). Thus, if lining is economical (benefits exceed costs) in the upper channel sections, a flow subdivision point will usually be reached where lining is no longer economical. This cutoff point in the example (Figure 8.8) is at a flow rate of 72 lps. Earthen renovation costs will also be somewhat less in smaller channels although the relationship will not be as strong as with lining. But again the net benefits (benefits minus costs) will decrease in smaller channels and a cutoff point may be reached after which earthen renovation is no longer economical.

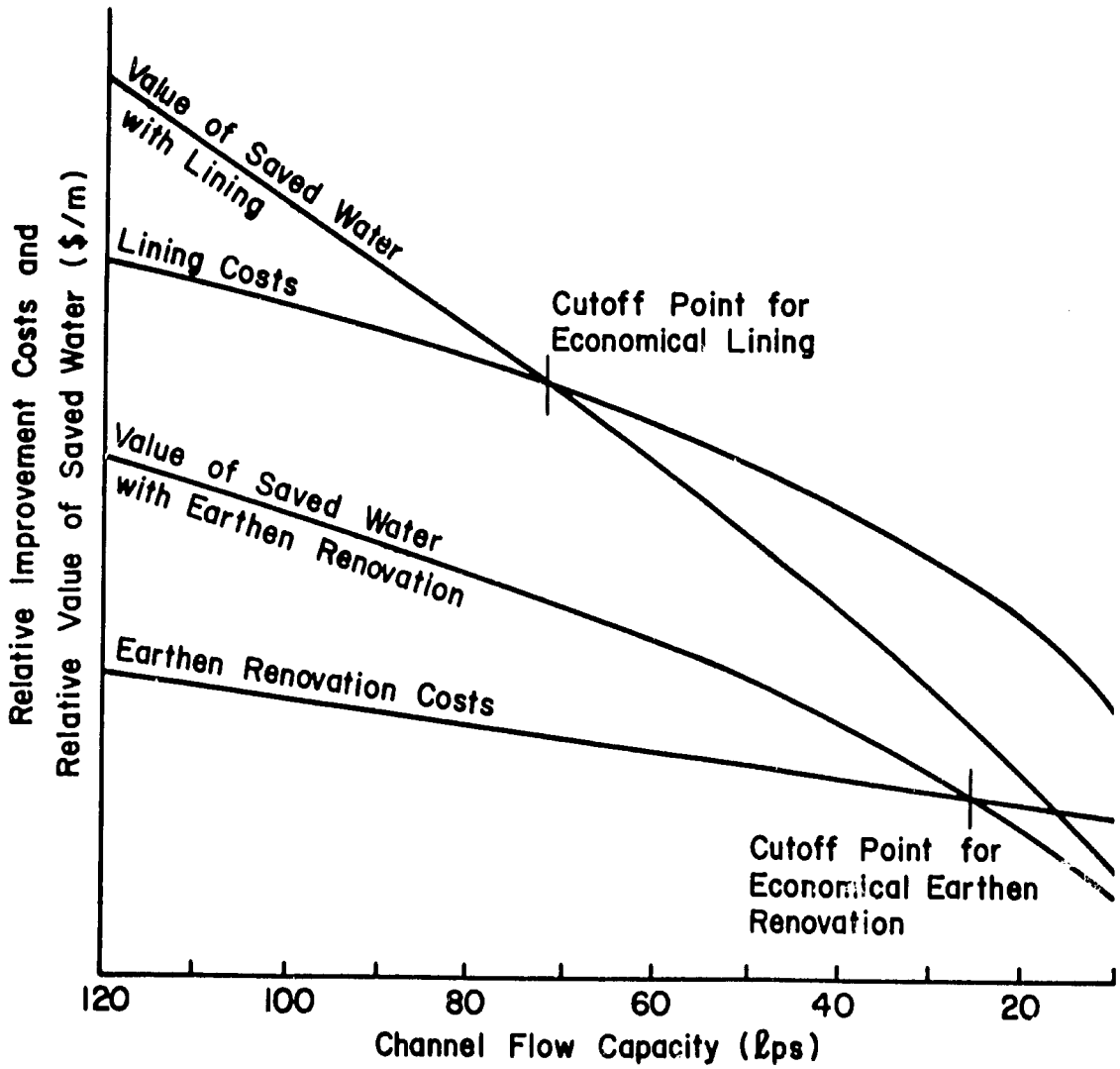


Figure 8.8. Example of the relative variation in benefits and costs with channel flow capacity for channel improvement programs.

The relationship between relative losses and channel capacity is difficult to determine, making this analysis rather impractical unless numerous similar systems are being improved. But an estimation of the realtionship and an understanding of the principles involved should lead to a better allocation of resources.

Most small irrigation conveyance systems are not constant flow, but the water is rotated through the channels to different fields at different times. Thus, different portions of the system are being used varying amounts of time. In a conveyance system branching from a single inflow source, the initial section will be full all the time while the tails of field ditches may be used only once or twice per month.

Figure 8.9 shows the percent usage time of the various sections of the primary branch channel shown in Figure 6.20 assuming water flows into the branch 20 percent of the time and the water is evenly allocated to the irrigated area. The usage of the main channel might be similarly shaped but would begin at 100 percent. The field branches shown in Figure 6.20 would also have a similar use distribution, but water flows into each branch an average of only 3 percent of the time. In Pakistan where a rotation system is followed, the average channel reach in portions of the conveyance system designated as primary or "official" channels, which make up about 15 percent of the total channel length, are carrying water an average of 1/3 of the time, while the average field branch section (85 percent of the total system) is full only 2 percent of the time. The variations of channel usage within each system will vary from 100 percent to less than 1 percent.

Because channel layout affects usage times, different layouts will have different usage time distributions and thus will require different improvement strategies. Figure 5.10 shows two alternative primary channel layouts for a commanded area. In the layout with two main channels, only about 3 percent of the main is used more than 50 percent of the time and only 65 percent of the mains are full more than 20 percent of the rotation period. In the layout with one main, nearly 50 percent of the primary channel is full over half the time and about 80 percent is full more than 20 percent of the rotation period. As the layout becomes more branched, an improvement program targeted at channels utilized more than 20 percent of the time would include less and less of the total system. Thus, layout and usage variations between systems will also affect the improvement strategy.

Water losses are generally proportional to the usage time (assuming uniform loss rates), and thus benefits of loss-reducing techniques will likewise decrease with usage time. However, since improvement costs are usually not directly related to usage but only to length of improvement, the costs will not

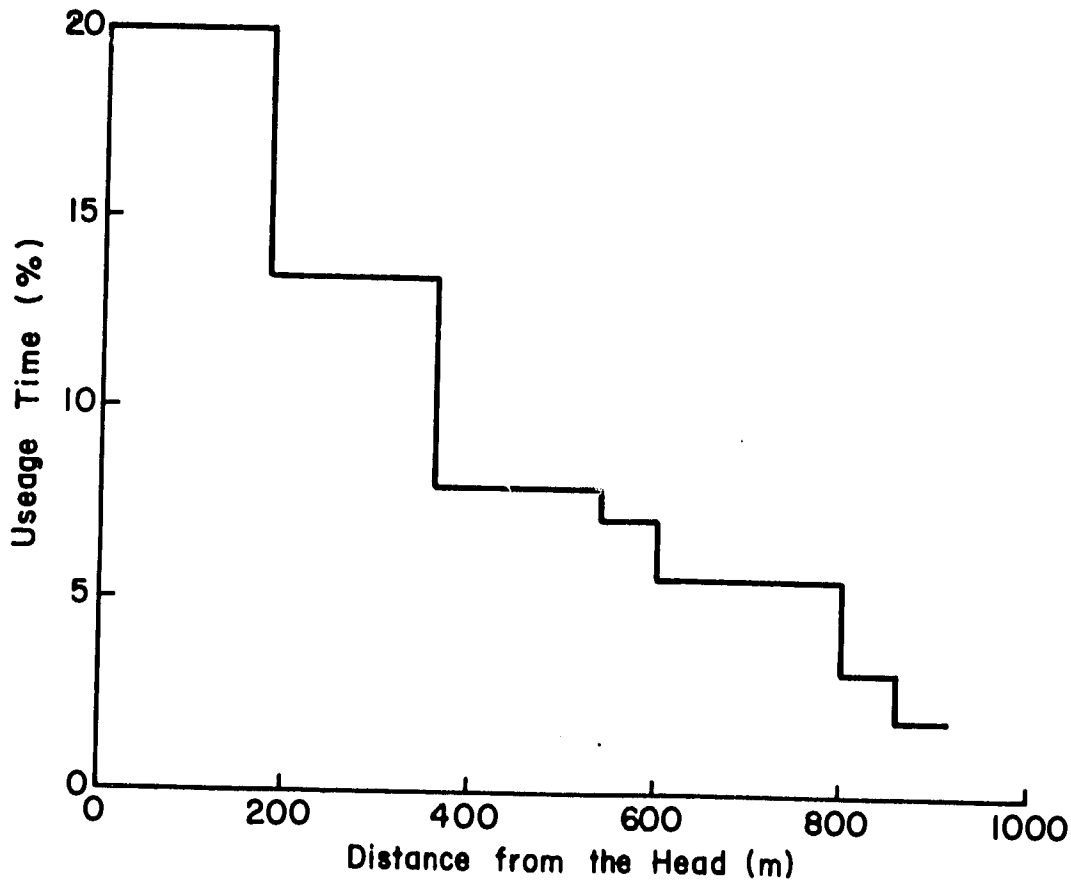


Figure 8.9. Percent usage time for the primary branch shown in Figure 6.20 (assuming water is allocated based on commanded area and 20 percent of the total system is commanded from the branch).

vary with usage. Consequently, in a turn rotational system, the economics of channel improvement will vary greatly within one system and the optimum improvement strategy will normally involve the application of successively lower cost techniques along a system starting from the head.

Figure 8.10 depicts how the benefits and thus economics of channel improvement vary with usage time of the channels. The figure depicts the total value ($/unit length) of saving 90%, 50%, and 25% of an initial water loss (assuming a uniform loss rate, Q_L), and the hypothetical costs of the improvement which results in those savings ($/unit length). As stated, while savings are proportional to usage time, costs are constant. Wherever the benefits are greater than the costs, the improvement technique would be economical. In the figure, any channels which are utilized more than 80 percent of the time could be economically improved by the high cost technique, any used more than 45 percent of the time by the intermediate technique, and any channels full more than 18 percent of the time could be economically improved by the low cost technique.

Reuss (1980) describes the theoretical principles and presents computational techniques for optimally applying improvement techniques to rotational systems based on the annual cost of the improvement per unit length, the expected water savings associated with each improvement alternative, and the value of water. These computational procedures which allow the determination of how much time a section must be utilized for a given technique to be economical, and at what point a change from one technique to another is desirable, are presented in Appendix 9.

One factor which must be remembered in designing channel improvement strategies is that, by improving some sections of a conveyance system, the losses will usually increase in the downstream portions. This is caused by the unimproved channels being incapable of efficiently conveying the increased flows as explained in Section 5.2. Consequently, the economics of applying improvement techniques will change as the system is improved. This change will require that, in order to receive the expected water savings, either some type of improvement must be continued through the branches and field ditches, or that the larger deliveries must be split into more than one branch at the end of the improved sections. These downstream improvements could involve simply enlarging the cross sections or increasing freeboards to convey the increased flow at the same efficiency as the smaller flows were being conveyed, or a regular cleaning and repair program to maintain low channel roughness and reduce excess losses.

8.3 Evaluating Irrigation Channel Improvements

In order to know whether an irrigation channel improvement program is beneficial and should be continued and expanded

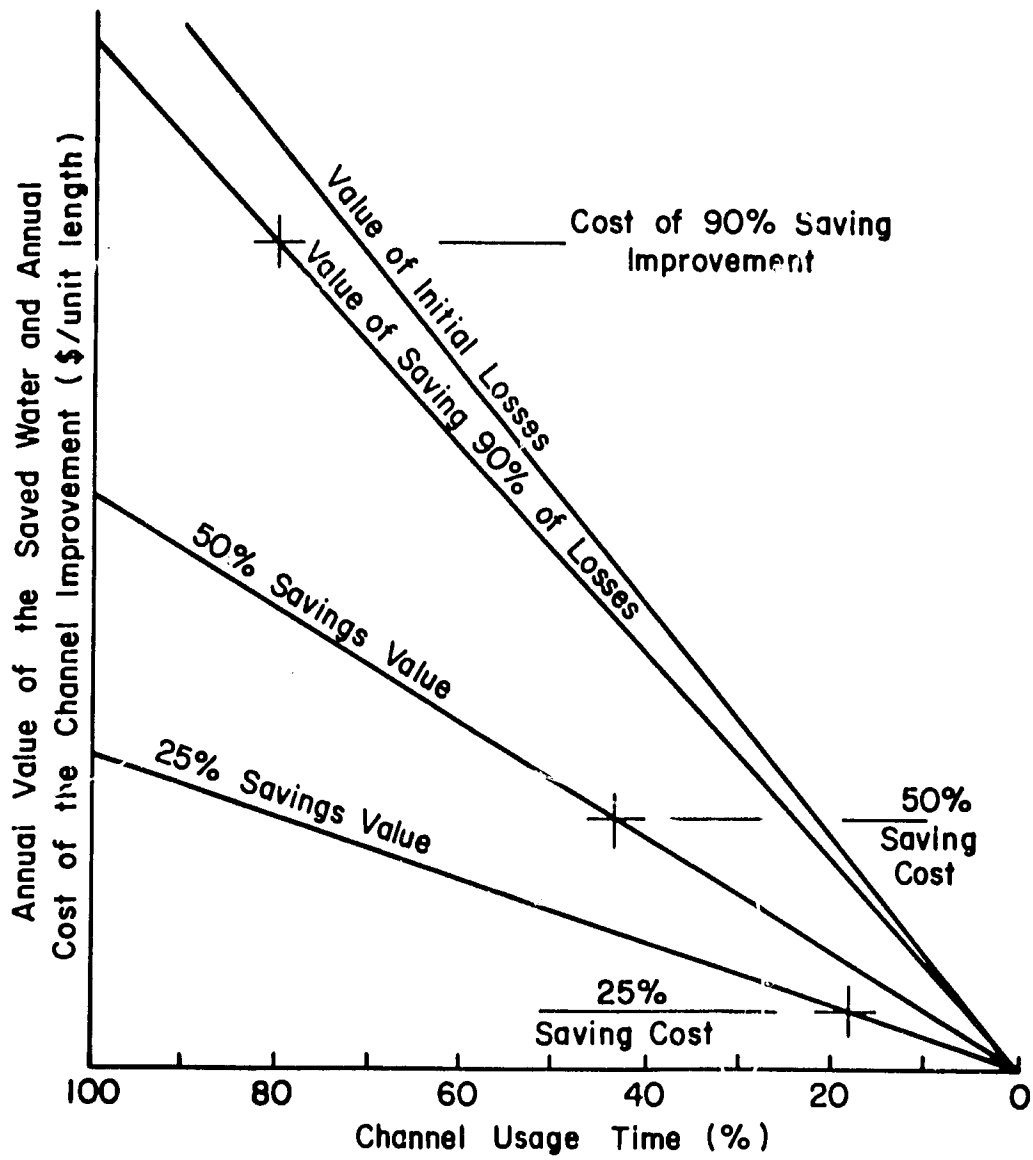


Figure 8.10. Graphical depiction of how costs and benefits of channel improvements vary with usage time (assuming uniform loss rates).

or changed or even discontinued, the program must be evaluated. The evaluation will be primarily an economic one although several factors will be difficult to put in economic terms.

8.3.1 Project costs

The costs of the program will depend upon the component costs, and may include:

1. the costs of consumed materials,

2. the costs of manual labor, skilled and unskilled,

3. the costs of machinery and tools,

4. the costs of design and supervisory technicians including training, support staff, and support facilities, and

5. the administrative costs.

Costs of materials, labor, and tools are the product of a quantity or hours utilized and a unit cost or price. Determining quantities is a bookkeeping problem. Setting prices can vary from simple (market prices) to difficult. An example of the possible difficulties is the process of pricing labor which is over and under utilized during different periods. This process is dependent upon local conditions and will be left to the local economists and not discussed here. The costs associated with technicians and administration will depend heavily upon the project scale and utilization efficiency. Real costs of personnel and facilities should be allocated to the improvement program.

Besides this initial cost there will be recurring costs associated with:

1. the increased annual cleaning and maintenance prescribed by the program, and

2. the annual costs of periodic repair and replacement of damaged structures and facilities.

The first of these can be predicted reasonably well while the second can initially only be estimated with the estimates later improved based on experience.

These two types of costs can be combined either into a total improvement cost over the life of the project, or an annual cost using a reasonable discount rate. Both require an estimate of the life of the improvement, which, like repair requirements, can only be established by experience. Regular measurement over time of project benefits, as explained later, is the best way to establish an effective project life.

8.3.2 Project benefits

The benefits of channel improvements can be calculated by two different methods. The first is a direct valuation of water savings, and the second, an evaluation of the added production attributable to the increased water supply. The total volume of water saved as a result of the improvement times the value of water, as established by the procedures outlined in Section 3.2, yields the benefits. If the established value of water is accurate for the relevant conditions, this can be a good, direct evaluation of the program benefits. One complication with establishing a price for the saved or additional water is that the per-unit value of water will be a diminishing function of the amount saved, and thus will vary from technique to technique. The farm budget and linear programming technique discussed in Section 3.2 can be used to establish this water value relationship.

Determining the value of the increased water supply requires knowledge of the volume of water saved by the improvement. This can be determined by measuring water losses by the techniques explained in Section 3.1. Two factors must be remembered when measuring losses. The first is that the value of water is realized at the field and thus the flow rates or water quantities must be measured at the field. Even if only a portion of the system is improved, the measurement of water savings should not be made only in that section, because it cannot be assumed that all the water saved in one portion of the system will be conveyed by the unimproved portions to the field. In Pakistan, a large part of the water saved by lining main channels was lost in the field branches where the increased flows could not be efficiently conveyed.

The second important factor affecting loss measurements is time. As depicted in Figures 8.4 and 5.7, water savings will not be static but will change with aging and fluctuate cyclically with cleaning and repair. Several loss measurements must consequently be made over time to determine the "life" of the savings. Total savings could be calculated as the area under a curve such as that shown in Figure 8.4, or the average of several measurements over a given period of time. The effective "life" of the improvement or the time when the renovation should be reapplied, could be set at a certain cutoff point when the savings have decreased below a certain level. Because of the cyclical fluctuations, the measurements should be made when the channels are in "average" condition as regards to vegetation length and repair.

A second method of determining the benefits of a channel improvement program is to measure the value of additional production which can be attributed to the resulting increased water supply. The simple concept involves comparing the agricultural production and net value of that production of a command area after improvement with that before improvement, with

concurrent measurements in control (unimproved) areas to allow for climatic and other variations. The relative differences between the improved and control measurements should reflect the result of the channel improvement program.

Achieving accurate measures of additional crop production is often not easy. Accurate determination of crop yields is labor intensive involving sample crop cuttings and measurement of crop areas or measurement of total harvested crops. Selection of sample farmers or plots can reduce the labor and time requirements with acceptable losses in accuracy if done carefully. Estimating yields with farmer interviews is a much less costly alternative, but involves the danger of random errors because of inaccurate estimates and biased values from farmers who have a preconceived opinion of the value of their improved system.

A second problem with measuring production benefits resulting from channel improvement is that a portion of the production increase could be the result of other factors. A farmer who recognizes the potential of a better water supply may begin using additional fertilizer, improved varieties, or more involved cutural practices, all of which involve additional production costs. Not only must the additional costs of these other factors be measured and subtracted from the value of the production, but a decision must be made as to how much of the production increase should be attributed to the increased water supply, and how much is attributable to other cultural practice changes.

Because of these complications, evaluating the benefits of water channel improvements with production increases should involve both physical measurements of yields and cropped areas of at least a sample group and interviews to determine changes in cultural practices, on both command areas where improvements have taken place and control areas, both before and for several seasons after the improvement. Then a decision must be made as to how much of the change is a result of the increased water supply.

Each of these techniques has its advantages and disadvantages. The water measurement method evaluates the factor directly affected by the improvement program--water supply. However, increased water supply, unless put to beneficial use, has no real value. The production method, on the other hand, more directly measures the true economic benefits of the program--increased yields or incomes--but it is often difficult to separate out the affects of the channel improvement from other cultural changes. The water measurement method is quick and doesn't require too much labor, but it is very difficult to establish the true value of water for the relevant conditions. Yield levels and values can be accurately determined, but the measurement requires a large labor and time investment, and

often involves a significant lag time varying from one to several seasons, before the evaluation is complete.

Although the primary benefit will always be in terms of additional water supply and the resulting yield increases, several other benefits are also possible. Although measurement of several of these benefits can be difficult, an attempt should at least be made toward their evaluation. Improved channels reduce the labor required to irrigate both because of improved water control structures and reduced need to monitor leaks and bank washouts. Increased water levels of the channels can lead to more equal distribution of water among high and low fields. Larger flows can also lead to improved distribution of water across fields and thus higher application efficiencies. The reduced conveyance losses will nearly always lead to more equitable water distribution, and potentially income distribution, among farmers because irrigators at the tail of the channels who always receive less water than those located at the head, will receive a larger increase in their water supply as a result of the improvement. The clean, wide banks can also facilitate movement of farmers among their fields.

Perhaps the largest potential secondary benefit is the possibility of the improvement program acting as a catalyst for improved cooperation among users; adoption of improved cultural practices; and, increased care for, maintenance of, and interest in their water supply system. The water supply system is one part of a water management system which is one aspect of an agronomic system, which is one facet of a cultural system. But the water supply system is often considered a vital and central component to the whole and thus successful changes to it can serve as an example from which to expand the improvement process. Successful watercourse improvement programs in Pakistan demonstrated to farmers that:

1. they could, through their own efforts, improve their water supply,

2. they could act cooperatively with fellow farmers for their individual benefit, and

3. the government agents could serve as valuable sources of new methods and information.

With this successful example and confidence, it will be much easier to follow up with other properly planned programs to improve agriculture and village life. This "successful example" may, in fact, be the largest potential benefit of irrigation channel improvement programs.

SECTION 9

PROGRAM ORGANIZATION, ADMINISTRATION, AND TRAINING

Techniques to solve problems diagnosed in tertiary irrigation conveyance systems, and channel improvement strategies for applying these techniques to increase conveyance efficiencies have been discussed in the previous sections. The physical system has been dealt with. But improving the efficiency of the physical system requires people who are trained and organized to carry out the work. The objective of this section is to present some alternatives and make some recommendations regarding means to organize, administer, and choose and train personnel for carrying out an irrigation channel improvement program.

The fact that this topic follows those on engineering techniques and economically viable strategies is not meant to imply that, once the strategies are chosen, a socio-organizational system should be developed to meet the needs of the strategy. This socio-organizational aspect will in fact probably be the most difficult phase to design and will be just as critical, if not more so, to the ultimate success or failure of the program. Although in some cases recognition of the potential benefits of physical improvements may motivate farmers and government agencies to create the required organizations and structures to attain the improvements; in many cases socio-organizational factors are so rigid and unchangeable that the physical program must be designed to be compatable with them. Consequently, to develop techniques or design strategies without full recognition of socio-organizational and personnel constraints would be pure folly. The two phases must develop simultaneously. Without improvement strategies that are practicable, economical, and show potential for significant benefits, the program will fail. Without the social and organizational systems and the trained personnel to carry out and maintain the improvements and realize benefits from the additional water supply, the program will also fail. The success of both phases is essential.

The complexity and wide variability of the organizational, administrative, and personnel components make it difficult to forecast all the potential problems and generalize potential solutions. However, some guidelines can be presented. Three aspects will be discussed: organizational, administrative, and training. By organizational is meant the allocation of responsibility and authority and the interrelationships among the water users and between the water users and the government agency which is in charge of carrying out the program. Administrative aspects refer to the bureaucratic organization and personnel policies of the government agency. The training component will include choosing and training the personnel to design and supervise the renovation work and in turn to train the farmers to carry out their portion of the improvement process.

9.1 Allocation of Responsibility and Authority: Organization

The alternative ways of organizing the program, that is, allocating responsibility and authority between the government and the farmers, may be conceived of as a continuum between a relatively centralized, authoritarian model, with minimum farmer participation, and a decentralized model with a large degree of farmer participation in the decision making process.

9.1.1 Two organizational models: centralized and decentralized

In the centralized approach, primary responsibility and authority is in the hands of the government; the role of the farmers, especially in the decision-making process, is minimal. The government decides which irrigation channels will be improved, how they will be reconstructed, and may carry out the reconstruction itself or through contractors. An example of this approach to reconstruction of irrigation channels is the Command Area Development Program currently underway in some states of India.

A decentralized organization is one of which farmers take the primary responsibility, with the government perhaps providing technical advice and/or financial assistance. In this approach farmers have a relatively large amount of authority and responsibility in the improvement and management of their portion of the irrigation system. They are involved in decisions concerning the reconstruction of their irrigation channels, and are responsible for their subsequent maintenance, management, and further improvements. Examples of decentralized local-level water management may be found in Taiwan, the Philippines, Indonesia, Spain and the U.S.A.

9.1.2 Comparison of the two models

Which type of organization is chosen for carrying out an irrigation channel reconstruction program will depend on many factors specific to the country, system, and proposed improvement program, including previous history of organizing farmers, governmental and legal traditions, and the attitudes of the planners and political leaders themselves. The approach chosen may contain elements of both models. Ultimately, the decision is as much political as it is technical; nevertheless the approach chosen will be a very significant factor in the ultimate success or failure of the program, especially in the long run. The two models will be compared and contrasted in terms of relative speed of implementation, relative costs, likelihood of maintenance following reconstruction, relative flexibility, and the implications for long-term "development."

One advantage of the centralized model might be that a reconstruction program could be initiated quickly and could

proceed rapidly, at least in the early stages. This is because there would be little necessity of educating, motivating, negotiating with, and organizing farmers. Manpower and financial resources could probably be mobilized relatively quickly.

In a decentralized approach with farmers taking the primary responsibility, initial progress in the physical reconstruction of channels may be slow. It will be necessary to educate the farmers as to the benefits of the project, convince them it is worth the cost, and help them to organize so that they can effectively achieve their goals.

Centralized, bureaucratic programs typically emphasize showing rapid progress, as the program tends to be measured in terms of physical progress. Therefore, such programs often depend on hired labor or contractors to carry out the reconstruction project because negotiating with farmers to do the work is time-consuming. If hired labor or contractors are used, the cost to the government of reconstruction will be high; this is true even if farmers are asked to repay the cost over the long run, as is true in the present program in India. If the government uses its powers to force the farmers to do the work themselves on their irrigation channel, the quality of the work may suffer, and the farmers may take a negative attitude toward their channel and be little inclined to maintain it or use it more efficiently.

On the other hand, when the farmers are given responsibility for and are involved in the project, the cost to the government can be reduced considerably. Farmers can be made responsible for the labor (as they are in the Pakistan On Farm Water Management Pilot Project) and can also share the burden of administration, such as settlement of disputes and procurement of materials, thus freeing the government technical personnel to provide technical services to more areas.

As was explained in Section 8.1, maintenance of the renovated channel is crucial to reap the benefits of any channel improvements. In a centralized system, the government would have primary responsibility for insuring that the channel is maintained. If the government does the maintenance itself, the cost will be very high, even if the farmers are asked to pay the direct costs. The government could order the farmers to do the maintenance themselves and take punitive action if they do not. The success of this approach will depend on the ability of the government to inspect channels and enforce sanctions. It is unlikely, for reasons given below, that the farmers will voluntarily maintain their channel if they have not participated actively in its reconstruction.

In a centralized approach, especially if physical conditions are reasonably uniform, the project can be simplified

technically: engineers could be given specific design procedures to follow, and variations among systems as well as the wishes of the farmers can be ignored. For example, outlets can be installed according to specific criteria such as one outlet per so many hectares; there would be no necessity to negotiate with farmers desiring extra outlets. This too would contribute to making rapid physical progress and may have the additional advantage of enabling the project to utilize more paraprofessionals for designing and constructing channels.

On the other hand, if procedures and standards are set rigorously at the top, there may not be sufficient flexibility to meet differing needs and problems on different irrigation systems. This would lead to inappropriate designs on some channels, and dissatisfied farmers.

The question of flexibility versus strict standard procedures leads to another consideration. There is now considerable evidence that a primary reason for the decline of early irrigation-based civilizations, especially Mesopotamia, was environmental degradation brought about by mismanagement of the irrigation system; and experts have suggested that this mismanagement was the result of overcentralization of the system. Briefly, the argument is that in highly centralized systems where local subsystems are managed by central authorities, those authorities make decisions in terms of perceptions of needs of the larger system and are insensitive to local perturbations and problems. They may perceive a need for increasing food production because of rising population pressures, and may respond by expanding local irrigation systems or intensifying them in such a way as to lead to waterlogging and salinity. The central authorities become aware of and respond to this environmental degradation only when it has become a large-scale problem of the system requiring massive investments to cure (if indeed it is curable). The extension of government authority and control to the lowest level of the irrigation system, implicit in a centralized approach to a channel reconstruction program, may therefore reduce the long-term viability and productivity of a modern system just as it seems to have in previous irrigation-based societies.

Related to this consideration is the question of what is meant by "development" of a nation, society, or more narrowly, an irrigated agricultural system. If the objectives of "development" include increasing peoples' capacity to improve their own lives, helping them to be active participants in improving productivity and social life and not simply building more and more physical structures, then the long-term consequences of a centralized versus a decentralized approach will be very different. In a centralized approach, since farmers do not participate actively in decisions regarding their irrigation subsystem and have no real authority and responsibility for it, there will be no development of farmers' pride or "civic sense"

toward their irrigation channel, and thus no sense of responsibility to maintain it, use it efficiently, and even to improve it further. On the other hand if farmers participate actively in the process of reconstruction and subsequent maintenance and management, they will take a greater interest in utilizing the water properly and further improving their irrigation subsystem. This will be true of course only if they are organized properly. This process of developing self confidence and local ability to organize and work for the common good may contribute more to the development of agriculture in the long term than the reconstruction project itself.

9.1.3 Legal considerations

Whether a centralized or decentralized approach is utilized, certain changes may be required in the laws relating to irrigation in order to carry out the project efficiently. In some countries water law may be sufficiently well developed that changes are not required; in others such as Pakistan and India, water laws have not been substantially changed since the colonial period and their provisions are inappropriate to modern needs.

A centralized, authoritarian approach primarily will require provision of legal authority, with adequate sanctions, to enable the government department involved to carry out its duties effectively. Legal provisions may be required, for example, to enable the government to reconstruct irrigation channels even if some farmers are opposed, to acquire right-of-way for improved channel alignments, to tax the farmers to pay their portion of the costs, to require farmers to contribute labor, and to enforce proper use and maintenance of the channel after its reconstruction. The details will vary with the nature of the program.

A decentralized approach, in order to be successful, may require even greater legal support and changes than an authoritarian approach. Experience in Pakistan suggests that a successful program cannot be based on informal non-binding understandings and organization. Existing laws are often not adequate to deal with the specific problems of improving water management. For example, existing laws usually do not contain provisions for punitive sanctions against those who do not do their share of the work or contribute their share toward the "collective good". An appropriate legal structure in a decentralized approach must define the responsibilities and authority of the various parties involved, it must provide for organizing the farmers to enable them to fulfill their responsibilities, and it must provide enforceable sanctions to insure that all parties involved do in fact fulfill their responsibilities. The specific content of such an irrigation law will depend on the characteristics of particular irrigation systems, the social structure and culture of rural areas, and the administrative and legal traditions of the country. Here, we can

only suggest some general principles which ought to form the basis of an irrigation law for a decentralized system:

1. A federal structure, with two or three tiers, will be appropriate in most cases. The lowest tier would consist of organizations of farmers on the smallest natural units of the irrigation system, that is, the irrigation channels to be renovated. At this level, membership of all farmers on the channel must be compulsory; the rights and duties of the farmers vis-a-vis each other must be clearly spelled out; and procedures of decision-making, choosing officers, and settlement of disputes must also be specified. Organizing these local water user associations into federations with substantial responsibility in management and maintenance of the irrigation supply system may be a longer-range goal worthy of consideration. The function of these federations could include providing support to constituent lower-level organizations and functioning as a court of appeal for settlement of disputes that cannot be settled locally. This would increase the viability of the local-level organizations themselves.

2. The law should provide for a maximum of participation of farmers in decision-making processes, operation, maintenance, labor, and financing.

3. Equity should be a basic principle built into the law. All farmers should be insured equitable access to water, and there should be a strong attempt to minimize the likelihood of undue dominance by, or advantages to, larger farmers at the expense of smaller farmers.

4. It is important that the law and policies of the government encourage the institutionalization of processes to settle disputes (including the use of technical "experts") and enforce sanctions locally, while including a rapid appeal process to an objective external body such as the federations discussed previously.

5. To insure that all farmers contribute their fair share (labor and financial) to the reconstruction, operation, and maintenance of the system and to prevent abuses (water theft, damage to the channel, etc.), adequate sanctions and provisions for their enforcement will be required.

6. The organizations should be designed to make them effective mechanisms for the continuing education of the farmers in effective water management.

7. The provisions of the law, as well as government policy, should encourage farmer initiative to further improve their local subsystem.

8. Finally, it is important that the local customary procedures and processes (decision-making, dispute settlement, etc.) be utilized or adapted for use as much as possible. It would usually be counter-productive to impose procedures that are contrary to local cultural values.

Radosevich, et al. (1976) gives a more indepth treatment of the legal aspects of irrigation organizations and describes the existing water laws in several countries. The suggestions given here for organizing a program for reconstruction of irrigation channels are based in part on a judgment of how some of the problems faced in Pakistan ought to be solved (using some hindsight), partly on observations in Thailand, Sri Lanka, India, and the U.S.A., and partly on published reports describing experience with local water management projects in Taiwan and the Philippines, where the governments have encouraged farmers to organize to improve channel systems, operate tubewells, and allocate water.

In Pakistan, although it was realized early in the experimental work that a decentralized form of organization would most likely facilitate the watercourse improvement program's goals, little actual research into the practical aspects of organizing farmers was carried out until much of the technology for channel reconstraction was developed. Because the organizational structures were not well developed, and because the researchers failed to adequately emphasize to the implementing agency the importance of farmer involvement at all levels, the pilot project evolved with a relatively centralized organizational structure with some anomalous decentralized elements. The tendency of the government and implementing agency officials of judging the project personnel on the number of systems improved also forced the engineers and technicians to more authoritatively dictate designs and manage construction work in order to increase the efficiency of the improvement process.

In the Pakistan On Farm Water Management Pilot Project, the government sets the design standards, and the engineer makes most of the decisions concerning the reconstruction of the channels. However, he has no legal authority to impose decisions, and one powerful obstructionist can stop the whole project. There is a farmer committee that is responsible for organizing labor and representing the farmers, but it too has no specific authority and in practice no real role in decision-making. After reconstruction the committee was expected to continue to function to manage the maintenance of the channel, but this has not happened, and maintenance has proved to be a major problem. Since the committee was not given authority

or responsibility during construction, they have generally not felt responsibility to maintain the systems. Also, they have not been given sufficient authority to carry out the more tedious and less exciting maintenance work. Many of these and other organizational problems and anomalies could have been avoided if there had been more research into existing patterns of farmer organization and testing of proposed solutions to the problem of organization beginning at the early stages.

This experience coupled with similar experiences in other countries illustrate the need for a continuous effort to involve the farmer's leaders in the decision making processes and give them the required authority so they will develop the self-confidence and initiative essential to continual improvement and maintenance of their water delivery system.

9.1.4 Development and testing of organizational forms

Modes of organization appropriate and successful in one place cannot necessarily be transplanted to another place where conditions are different. Organizations must be developed for specific traditions and needs. Therefore, an integral part of the early stages of an irrigation channel improvement project must be research into local forms of organization and experimentation with alternative organizational forms. This sociological research should be done in conjunction with the research into physical problems on the system allowing observation of the interactions between the social and physical factors. Freeman and Lowdermilk (forthcoming) discuss the conceptual aspects of diagnosing socio-organizational problems in village level irrigation systems. An example of such sociological research is given in Mirza and Merrey (1979). The stages of the research and development process are the same as those discussed in earlier sections concerned with solving problems in the physical system. The first step is to diagnose the problem: research initially will be focused on the existing forms of organization of the local system, and processes by which the system is managed. This research should have two goals; first, identifying the relationship between the organizational forms (including but not limited to the organization of the water users themselves) and the physical problems identified; and, second, to identify structures and processes that could be used or modified for improving the organization of the system. In Pakistan, for example, one of the primary causes of the inefficiency and poor maintenance of the irrigation channels is the lack of any effective social and legal mechanism for insuring regular maintenance.

After better understanding the present socio-organizational system and diagnosing its problems, the next stage is to propose some possible solutions, for example, alternative modes of organizing farmers. Radosevich and Kirkwood (1975) describe organizational systems found in many countries of the world.

These alternatives should then be tested on a small scale in conjunction with the testing of solutions to the problems identified in the physical system. During the next stage solutions that appear to work--a particular mode, or modes, or organization for example--must be evaluated on a larger scale, in conjunction with solutions to the physical problems, in a pilot project. The pilot project stage should be long enough and on a sufficient scale to fully evaluate the proposed socio-organizational solutions and their interactions with the physical solutions and modify them as necessary before initiating a large-scale project.

9.2 Program Administration

Administration refers to the bureaucratic structure and processes by which the channel improvement program is carried out. It is again not possible to make recommendations in this area that would be applicable in all situations; like the questions of solutions to the physical problems and organization of the water users, the particular form of administration adopted will depend on many specific factors including the present structure of the government, administrative traditions, and whether a centralized or decentralized approach is followed. Here we can only point out some of the administrative options and basic problems that must be solved if the project is going to succeed.

9.2.1 Bureaucratic structure

The basic bureaucratic structure of the agency can, like the program organization, tend towards either centralized or decentralized models. Concentrating authority and decision-making at the top is often regarded as more "efficient", but in practice, the bureaucratic procedures of centralized systems are frequently cumbersome and inefficient. A problem with centralized administration is that the field worker usually has less flexibility to adapt the improvement program to local conditions, needs, and preferences. Conversely, a field worker who can select from several options must be adequately trained to exercise his flexibility wisely or the final decisions may be worse than a centrally dictated one.

A second structural option will be whether to form a new administrative structure or agency to carry out the program, or to expand an existing agency to administer it. A decision must be made whether the established traditions, personnel, and hierarchy of an existing agency will make the project more efficient, quicker starting, and less expensive; or will stifle the flexibility required to develop and test solutions in a program which must in its initial stages, have some experimental components.

9.2.2 Job differentiation

Many distinct tasks will need to be carried out in the process of improving irrigation channels. These will depend upon the improvement techniques and organizational mode selected, but will often include:

1. diagnosis of problems and development of solutions,
2. selection of project areas and channel systems,
3. design of renovated channels,
4. organization of farmers,
5. supervision of construction,
6. purchasing of supplies,
7. arranging for and hiring of contractors,
8. instructing farmers in maintenance,
9. instructing farmers in efficient water use, and
10. evaluating of the program.

All except the first and last of these tasks is an integral part of the improvement activity and will probably be carried out by the same agency. The diagnostic, developmental and evaluation stages could also be carried out by that agency.

One question that must be dealt with is the degree of differentiation of job responsibility. On the one hand, one could have several "specialists" i.e.: surveyors, designers, supervisors, organizers, and purchasing agents. This may reduce the amount of training needed for individual personnel. On the other hand, a generalist who can select channels, design improvements, and supervise construction, as well as organize and train farmers will be better able to carry out improvements which meet the physical and social needs. But he will also need to be thoroughly trained in all aspects of the improvement process. The degree of administrative specialization chosen will be determined to a large part by the personnel available and the time and money available for training. However, it is felt that:

1. one government agent should be assigned as the chief contact person with the farmers on each channel system being improved from start to finish to give the process continuity,

2. the technician supervising the construction should have been involved in the design process,

3. the designer and supervisor should have sufficient contact with farmers to know their needs and have their confidence, and

4. trained engineers should be burdened as little as possible with tasks which can be carried out by farmer leaders such as organizing labor, purchasing supplies, and settling disputes among farmers.

Separating the initial improvement duties from the follow-up activities may enable more efficiency in both the training and improvement processes. However, an emphasis on the construction of physical improvements should not lead to a deemphasis of the follow-up program or cause a lower status to be given to the extension personnel involved.

The people involved in the improvement process should also evaluate the systems they have improved so that they can gain satisfaction from their successes and learn from their failures. A more comprehensive evaluation of the program by personnel from outside the action agency would be desirable to eliminate possible biases, to gain new ideas and opinions and to make objective decisions concerning continuing or expanding the program. This evaluation should include the administrative structure and personnel policies as well as the improvement process.

9.2.3 Personnel

A major constraint of an improvement program may be the availability of trained or trainable personnel. This constraint may necessitate the selection of less desirable improvement techniques and limit the extent of the program. Several options are available to help relieve this possible limiting factor. One is to make the required positions attractive both so that personnel would be more willing to transfer from other less important jobs, and so that the project's present trained and experienced personnel would be less likely to transfer to other agencies or jobs. Making a position desirable requires providing sufficient incentives and recognition. More incentives will be required to keep a qualified technician in a field job in rural areas where work is hard and facilities are usually minimal, than are needed for a man in a traditionally higher status desk job in a city. This need must be realized and provided for. Technicians should be recognized and rewarded for good work. By giving the technicians more flexibility and responsibility in their interactions with farmers, they can gain the respect of the farmers and rightfully feel pride in their completed work. Recognition of and reward for good work within the administrative structure such as the support of

superiors and promotions based on performance rather than seniority will also be an incentive to continue good work. As a last resort, if proper incentives cannot be provided, it may be necessary to limit the transfer rights of freshly trained personnel, at least until some return on the training investment has been realized.

A second long-term option to prepare more personnel for an improvement program is to offer training in channel design and water management skills as part of the curriculum of existing agricultural and engineering colleges and vocational schools, and to expand the numbers admitted to these programs.

A shorter-term option involves hiring less qualified people and training them for specific jobs. Use of such paraprofessionals will probably require very specific definiton of job responsibilities, but, when combined on a team with a more broadly trained leader, the paraprofessional could relieve the leader of some time consuming responsibilities. Also, the use of less qualified people will probably require a larger investment in training programs which is undesirable but may be necessary to relieve the manpower shortage. The use of less qualified personnel may limit the technical complexity of the improvement program. For example, channel cleaning and repair may have to be undertaken in place of more desirable earthen renovation work, at least in the initial stages of a project, because of a lack of trained design engineers.

9.3 Training

Training of engineers and other water management specialists for the improvement of irrigation channels and the instruction of farmers in maintaining the improvements and efficiently utilizing the water is another important facet of a successful project. A successfully designed project will fail without proper application at the field level. The type of training required will depend upon the type of personnel available and their previous training and experience, the type of improvements undertaken and specific job requirements, and the organization of the program. For example, preparing extension agents to instruct and motivate farmers to clean and repair their channels will involve a different training program than teaching engineers to design channels. The previous subsection discusses some of the options in personnel types and job requirements. A channel improvement program may very well involve two or more separate training programs to prepare people from different backgrounds to carry out different jobs. In the Pakistan OFWM Pilot Project, two independent training programs are carried out: one to train engineers to design and supervise the renovation of watercourses and a second to train water management specialists to in turn instruct and motivate farmers to maintain the improved channels and efficiently utilize their water.

The organizational makeup of the project will also affect the training requirements. If a centralized approach is chosen in which the responsibilities of farmers are minimal, farmers will need to be instructed only in how to use the water more efficiently; and the engineers and other specialists will require only technical training--there will be little necessity for training in "social skills".

If, on the other hand, a decentralized form of organization is adopted with a high degree of farmer participation and responsibility, then both farmers and technicians will require considerable training. Farmers, as the ultimate users of the water and growers of the crops, will have to be trained to:

1. accept the responsibilities and authority designated to them under the new set of rules and new organizational form,

2. understand the value and best methods of reconstructing and subsequently maintaining and operating their irrigation channels, and

3. use the water effectively in conjunction with other inputs for increasing crop production.

As with other aspects of a channel improvement program, the training will need to be tailored to the specific needs. Only some guidelines will be given here in the selection of candidates and training techniques. Outlines of the two training programs developed in Pakistan are listed in Appendices 10 and 11. The outlines list examples of topics which might be covered.

9.3.1 Selection of candidates

The best candidates for staffing a channel improvement program will probably be persons who themselves are from a rural, small farmer background--that is, the same background as their clients. More important than background, however, is that the candidate have a positive attitude toward working in rural areas directly with farmers, and that he have an ability to work with others, to take initiative, and to solve problems. In regard to technical background, he need only have the basic skills (mathematics, agronomy, knowledge of the language of his clients) upon which the training can build, plus a general problem-solving ability. This means that formal educational requirements should be de-emphasized as a prerequisite for admission into the program. A good approach is to develop an admissions test to measure the qualifications of the candidate in terms of the above criteria. It may also be useful either to have separate training programs for "professionals" and "paraprofessionals", or, even better, to give advanced training to those paraprofessionals who do well in the field.

9.3.2 Training techniques

The training should emphasize applied field work under the conditions the trainees are most likely to face, followed by a period of internship under experienced technicians. This emphasis on field training has been an important ingrediant in the success of training water management specialists in Pakistan. At least a portion of the training should take place at or near the location where the trainee will be assigned so that he will have sufficient experience in the particular conditions of that area.

The training program should be in the language of the trainees and clients themselves and not in a foreign language. The use of English as the medium of instruction in Pakistan, traditional in academic settings, tends to reduce the comprehension of trainees, leading to the memorization of formulas and objectives without understanding their real meaning. Training in a foreign language also reduces the trainees' ability to explain in the farmer's language the principles involved (there is a tendency to use English terminology, for example).

Maintaining a very high standard of expectations is essential to the success of the project. It must be clear to the trainees that they will be examined and must pass stiff examinations in the subject matter in order to be employed.

The training should place a heavy emphasis on social skills and developing an understanding for the farmers, especially if the program organization is decentralized. This training may be imparted at several levels. At the academic level it is important for them to understand the social structure and culture of rural areas, how these constrain the behavior of the people involved, and the processes and trends of social change. At another level, the trainees should be trained in basic social skills and strategies for working with farmers, organizing meetings, helping farmers settle disputes, and encouraging farmers to take initiative and responsibility. Part of this training should also include an attempt to develop the self-confidence and self-awareness of the trainees.

The trainers should be people who themselves have considerable field experience. One possibility is to rotate some of the best field workers from the field to the training program, and then back to the field again. The trainees should be provided with a complete manual, which they can carry with them to the field, containing all the methods and data they will need for their work, to which they can refer when necessary.

SECTION 10

AFTER THE IMPROVEMENT

After the physical improvements to an irrigation channel conveyance system are complete, the most difficult part of gaining benefits from the improvement process begin--maintaining the improvements and putting the supplemental water to good use. Both of these factors have been mentioned several times in previous sections, but their importance to program success deems that they be reemphasized. To design an improvement program without considering both factors would be like giving a tractor to a group of farmers without teaching them how to maintain it or giving them the implements to utilize it efficiently. Channel maintenance will extend the life of the improvement so that the saved water is available for a longer period of time. Teaching farmers efficient use of their increased water supply will insure that real benefits are gained from the saved water, rather than the water gained from the improvement being lost from field channels or inefficient field applications, or used inefficiently due to poor crop production.

Good maintenance techniques and schedules will depend upon the type of improvement process, and can be developed and tested as were the improvement techniques. A maintenance program should be designed which will allow an improvement to reach a desired life. Farmers can most efficiently maintain earthen renovations and possibly also lined channels because they are most familiar with the system and, if properly taught, aware of the problems. But they must be taught how, organized, and motivated to carry out the work. Maintenance techniques can be taught by the original improvement technicians, although instruction may be easier after problems begin to develop and are evident to the farmers. Consequently, it might be better to assign a separate person to this task. A continuing organizational structure must be developed among the farmers to carry out maintenance work. This can be a continuation or adaptation of the organization formed for the original improvement. The organization should decide when maintenance work should be carried out and who should do it, and should be given legal support to enforce their decisions.

Farmers who feel a pride in their conveyance system and recognize the benefits of maintaining it will be motivated to carry out an effective long-term maintenance program. The water users will have more pride in and gain a better understanding of the system if they have had a direct investment in its improvement, such as providing the labor, and if they were involved in the decision-making process during improvement. They will be more likely to maintain "their" system, than a group of channels they believe are the government's responsibility. Technicians or extension personnel must

teach and demonstrate the potential problems and resulting loss of newly gained water which stem from the lack of maintenance. If the improvement program has demonstrated to farmers their ability to increase their water supply, a technically sound maintenance program which shows sufficient benefits should sell itself if it is effectively introduced.

No benefit is derived from additional water supplies at the field from any source, including channel improvement, unless the water is usefully utilized to grow crops. Many supplementary irrigation supply projects undertaken in the past have simply assumed that, since water is a limiting factor for production, the farmers will make efficient use of the supply. The resulting production increases have seldom reached the expectations, proving the assumption to be wrong. Farmers often need instruction and assistance when changes in cropping patterns and cultural practices are required. A change in water supply can represent a very basic change in a traditional farming system.

A successful irrigation channel improvement program not only provides supplemental water, but also can act as a catalyst for further improvements to the agricultural system. Farmers can gain faith in new technology, respect for government agents, and confidence in their ability to improve their conditions. To not exploit this opportunity is to forego perhaps the largest potential benefit of an improvement program.

Consequently, after the improvement process is complete, an agent must be assigned to the area to teach farmers to utilize their water efficiently and to help the farmers improve their agricultural practices. He can be the same person who teaches maintenance techniques. His contact with the farmers must be intensive after the farmers have initially realized that they do in fact have additional water, and must continue for some time at a reduced level. He must have knowledge not only of water use, but the whole agronomic system, for he may in fact find that water is not the only limiting factor to increasing yields. Since he will have the most intimate contact with the workings of the improved channel systems, he should also be given formal opportunity to offer suggestions to the program designers as to how the program can be improved.

Since this agent must be knowledgeable in several areas, his training must be more broadly based than that of the channel improvement technicians. He must be able to diagnose problems, develop solutions, and convince farmers of the importance of the changes. A list of suggested topics for the training of such a "Water Management Specialist" is given in Appendix 11. Only if this specialist can convince farmers to maintain their improved system and teach them to utilize their water supply more efficiently, will the maximum benefits of the channel improvements be realized.

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Appendix 1

INFLOW-OUTFLOW LOSS MEASUREMENT

DATA SHEET

Date _____

Experimenter _____

Locations

Location of the conveyance system _____

Location of the test channel section _____

Location of the upstream flow measurement _____

Location of the downstream flow measurement _____

Distance between flow measurement locations _____ m

Flow measurement device used _____

Flow measurement readings

1. Upstream:

Time	Staff gage readings h_a	h_b	Flow rate (ℓps)	Comments

Ave. steady state flow rate _____ ℓps

2. Downstream:

Time	h_a	h_b	Flow rate (ℓps)	Comments

Ave. steady state flow rate _____ ℓps

Is the channel flowing normally, or are the flows greater or less than usual? _____ how much? _____

What is the condition of the channel? _____.

_____

Is there overtopping or visible leakage? _____

Comments on the flow measurement _____

_____

_____

Calculations:

Constant loss rates:

$$Q_L = \frac{\text{upstream ave. st. st. flow rate} - \text{downstream ave. st.st.flow rate}}{\text{Distance between measurement}}$$

$$Q_L = \frac{\ell ps - \ell ps}{m} \times 100 = _____ \ \ell ps/100m$$

$$Q_{LP} = \frac{Q_L}{\text{upstream ave. s.s.f.r*}} \times 100 = _____ \ \%/100m$$

Loss rate proportional to flow rate:

$$Q_{LI} = \ell n \left(\frac{\text{downstream ave.s.s.f.r.}}{\text{upstream ave. s.s.f.r.}} \right) / \text{distance between measurement}$$

$$Q_{LI} = \frac{\ell n(\ \ell ps/\ \ \ell ps)}{m} = _____ \ m^{-1}$$

*Steady state flow rate.

Appendix 2

PONDING LOSS MEASUREMENT

DATA SHEET

(Always note measurement units)

Date _____

Experimenter _____

Location of conveyance system _____

Location of test section _____

Gauge reading at operational level (OL) _____

How was operational level determined _____

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 OL	_____	____, ____, ____, ____, ____,	_____
	_____	____, ____, ____, ____, ____,	_____
Final	_____	____, ____, ____, ____, ____,	_____

Additional information to diagnose the causes of the losses:

1. Bank width at OL

   Left bank ____, ____, ____, ____, ____, Average _____

   Right bank ____, ____, ____, ____, ____, Average _____

2. Elevation difference between OL and surrounding fields:

   BM. rod reading _____ BM. description _____

   OL (rod reading) _____

   Right side field rod reading ____, ____, ____, Ave ____, ΔElev _____

   Left side field rod reading ____, ____, ____, Ave ____, ΔElev _____

   Remarks (adjoining roads, etc.) _____

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

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

   _____

5. How far is the section from the head? _____

   Is the section a primary channel or on a farmer's branch?

   _____

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

   _____

7. Bank material: Estimate the soil type: _____

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. _____

Type leak	Gauge reading	Loss rate (units)

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 holes, etc: _____

    _____

    _____

Appendix 3

DATA SHEET FOR DETERMINING MANNING'S N

Date _____

Experimenter _____

1. Choose a long (500-800m) straight section with no major obstructions such as culverts.

   Location of W/C _____

   Location of test section _____

2. Flow rate at head and tail of section. Remove downstream flume before taking other data.

Inflow				Outflow			
Time	h_a	h_b	Q	Time	h_a	h_b	Q

3. Measure wetted perimeter and cross-sectional area at about 30m intervals. WP is measured directly with tape. Area is calculated from top width of water surface and five equally spaced with depth measurements (See Fig. A.3.1.)

Distance from start	Wetted perimeter	Top width	Depth 1	2	3	4	5	Area $AR^{2/3}$

4. Determine the water surface (OL) elevation at each 30m location. Only OL rod readings and turning point readings are required.

Distance from start	B.S.	F.S.	I.S.(OL)	R.L. elevation (OL)

5. Fully describe the condition of the watercourse with respect to uniformity, location of obstructions or constrictions, type, length and density of grasses and other types of vegetation, and any recent cleaning activities.

_____

_____

_____

_____

_____

6. Figure A.3.2 graphically depicts the calculation process.

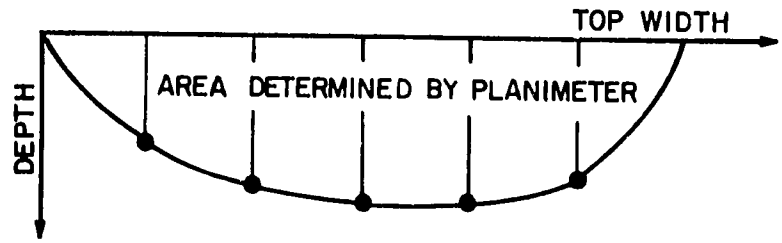


Figure A.3.1. Graphical cross-sectional flow area determination.

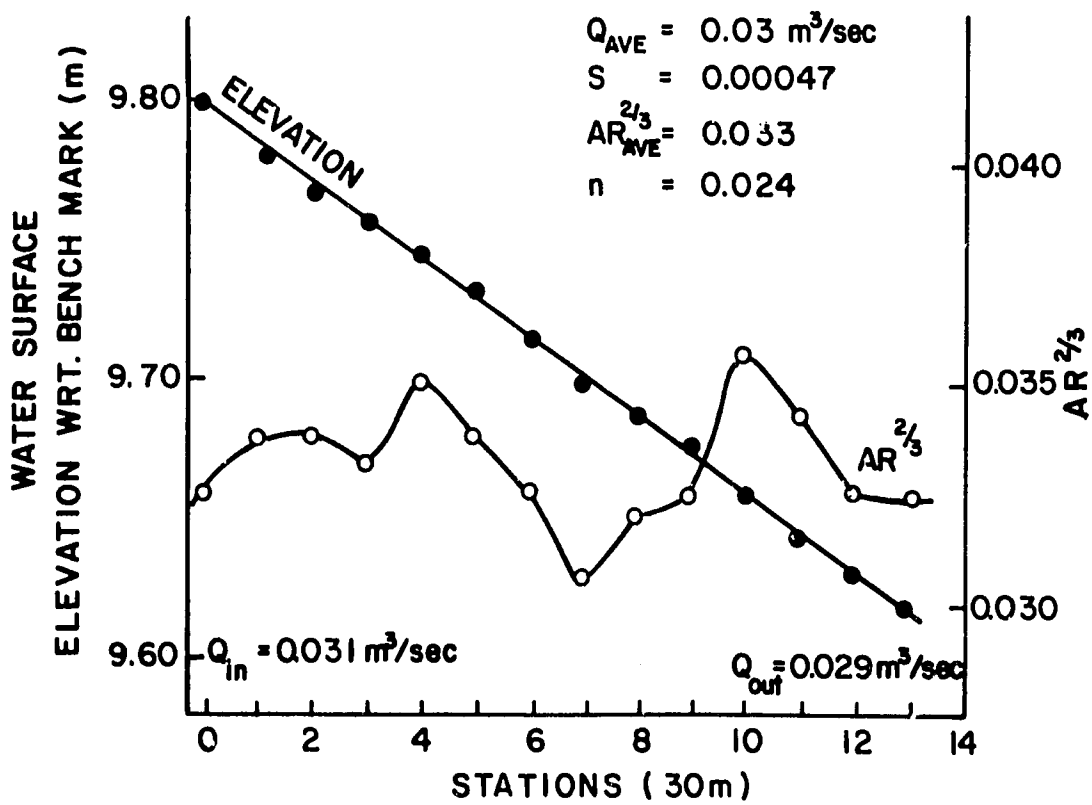


Figure A.3.2. Graphical presentation of channel hydraulic parameters.

Appendix 4

FIELD AND WATERCOURSE ORIENTATION MAPPING USING PLANE TABLE AND PEEP-SIGHT ALIDADE[1]

The plane table and alidade, Figure A.4.1, are frequently used instruments where layout maps are needed of irregular field and irrigation channel orientations. This offers the advantages of preparing the map while the surveying is done and permits checking the watercourse and field layout as shown on the map with the actual layout. The principal disadvantage of using the plane table and alidade is the extra field time necessary. When bad weather occurs, no work is possible.

The basic equipment for the plane table and alidade method of mapping consists of a tripod, drawing board, alidade equipped with peep-sight alignment hairs, tape and pencil, paper, and scale for preparing the map. Paper is fastened to the board and positions are plotted on the paper by sighting through the alidade for direction, and determining the distance by taping.

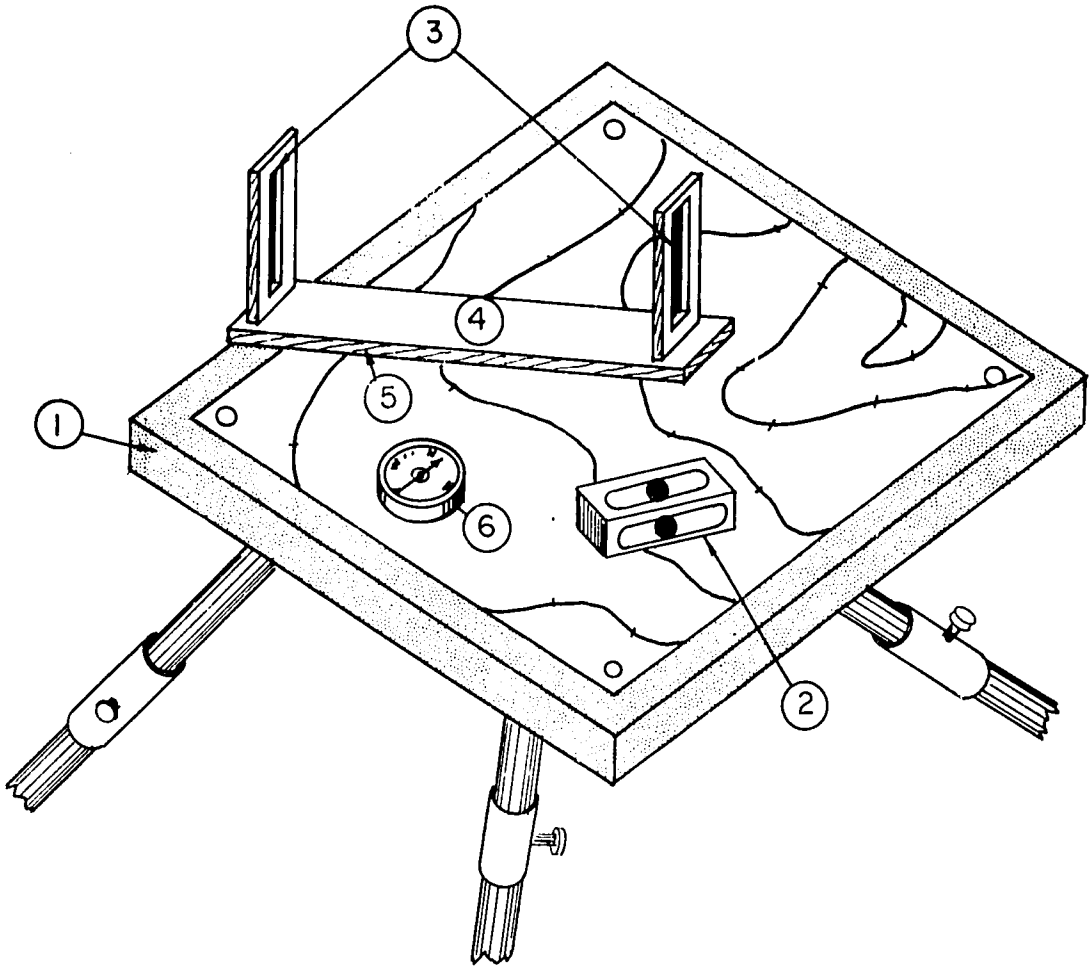
The Plane Table

The plane table (Figure A.4.1) is a rectangular board (1) usually 46 x 79 cm with a means for attaching it to a tripod. The top surface of the board is provided to attach a sheet of mapping paper.

The Alidade

The alidade in Figure A.4.1 has two peep-sight brackets (3) with the vertical hairs separated by 46 cm distance. The alidade has a linear, usually brass base (4) that supports it when resting on the table. One edge of the support (5) is usually beveled and this is the sighting edge. A separate, small magnetic compass (6) is provided for proper orientation of the plane table.

1/Taken from Water Management Methodology Series #7 by Wayne Clyma and Alan Early. Water Management Research Project, Colorado State University, Fort Collins.



1) Rectangular Board Table
2) Leveling Bubble
3) Alidade Peep Sight Brackets
4) Alidade Brass Base
5) Alidade Beveled Edge
6) Magnetic Compass

Figure A.4.1. Plane table and alidade.

Setting Up the Plane Table

The tripod and plane table are set up at an appropriate central location with respect to the line of sight for field layout of the water channel to be mapped. Using the leveling bubble (2) on the separate level provided, the table is leveled. Leveling the table is frequently difficult. The table is then rotated in a horizontal plane until magnetic north is the top of the map or until another reference direction has been selected. The location of the instrument is designated on the map either arbitrarily or by using other known points to determine instrument location. A map can then be prepared using the alidade to sight locations and the tape to determine distances.

Alidades of other types can be used to determine both elevation and stadia distances. Because of the difficulties of maintaining a precisely leveled table, the student may find it advantageous to use the plane table for locations and the engineer's level for elevations. This procedure may seem like an unnecessary duplication of instruments and equipment. However, use of the level eliminates most of the difficulties of keeping the table level for good elevation control. Where a plane table and alidade are available, a level is usually available. One experienced surveyor for both level and table can usually keep two experienced rodmen busy.

Orienting the Plane Table

Location of the instrument on the map that is to be made is usually accomplished by two methods: (1) compass and (2) backsighting.

To orient the table by compass, the alidade is placed in the direction that is desirable for a north-south line. The table is then rotated until the compass needle points to magnetic north. The board is clamped and a line is then drawn on the paper for future reference and for periodically checking the orientation of the board.

Orientation of the board by backsighting requires that two points be located in the field, They may or may not be located on the map. If not located on the map, then the orientation of the line is assumed for best use of the paper for constructing the map. Distances in all directions may be estimated and then scaled to make sure that the scale selected will permit all the area to be mapped to be included.

If two points (A and B) are already located on the map, the instrument can be located at either point. The alidade is then pointed along line A-B and the table rotated until point A (or B) is sighted and aligned with the vertical hair.

If point A is located on the map and point B is a more desirable instrument location, then the instrument is set up at point A. The table may be initially oriented by compass or by some other known line. The instrument is then sighted at point B and a line drawn. The distance to B can be measured by pacing or taping depending on the accuracy required. B is then marked. The instrument is moved to point B, set up, and the alidade pointed along line B-A on the map. The table is rotated until the vertical hair is aligned on point A.

After the table has been oriented by either of the methods, several distant objects should be located and short lines identified at the edge of the map with a description of what the object is. The orientation of the table can then be checked periodically without having the rodman return to point A. By carefully selecting well defined objects on the skyline, reorientation of the table is more accurate. The large distances require minute adjustments for proper orientation.

A straight pin is usually placed in the table at the point representing instrument location. The alidade is then kept against the pin when sighting locations. Even with the pin, care is required in keeping the line of sight of the alidade radiating from the station.

Points on the map that require precise locations may be located by intersection or triangulation. With the instrument at one location, lines are drawn that represent the line of sight to the desired points. The instrument is relocated by carefully taping the distance to the new location. Line of sight lines are drawn from the new location to the desired points. Where two lines intersect, this is the location of the point. By using three instrument locations, a triangle of error is formed around each point. These methods are indicated in Figure A.4.2.

A sample field and irrigation channel layout survey and map are illustrated in Figure A.4.3. Points A and B are points of known location. The distance between these points has been taped. The plane table is set up over point A and the mapping sheet has been set up with the points A and B separated by the scaled length. The alidade is aligned to sight from A to B and the line A-B is drawn on the map with the straight edge of the alidade. Sights and dotted line rays are then made to the corners of the field (1,2,3,4), the centerline of the channel (5,6), and the boundaries of the next field (7,8).

The plane table is moved and set up over point B and oriented to point A, as shown in Figure A.4.4. The points 1, 2, 3, 4, 5, 6, 7, and 8 are resighted and appropriate dotted line rays are drawn as illustrated. Once the intersecting rays have been obtained the field boundaries can be drawn as shown with the solid lines. In a like manner points C and D

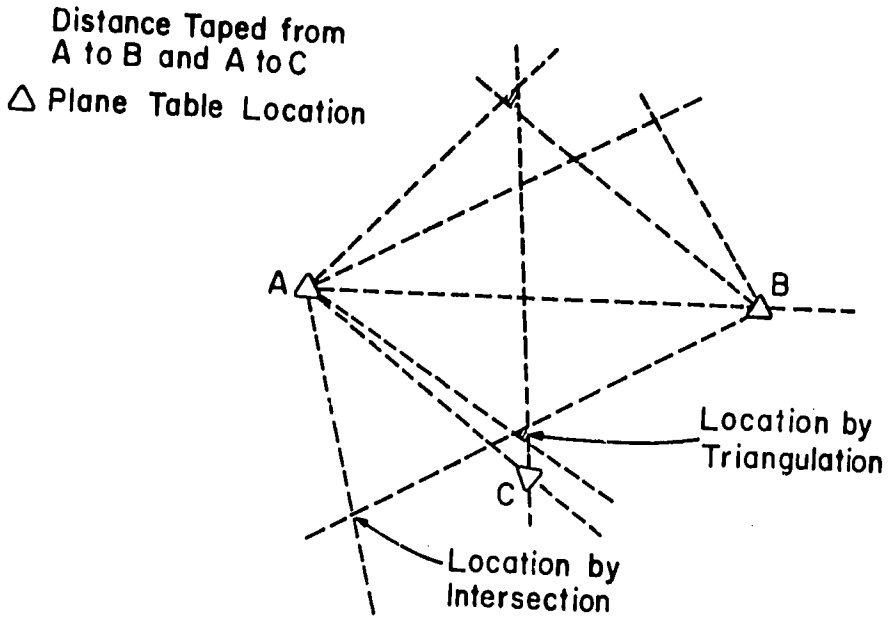


Figure A.4.2. Illustration of the intersection and triangulation methods of locating points.

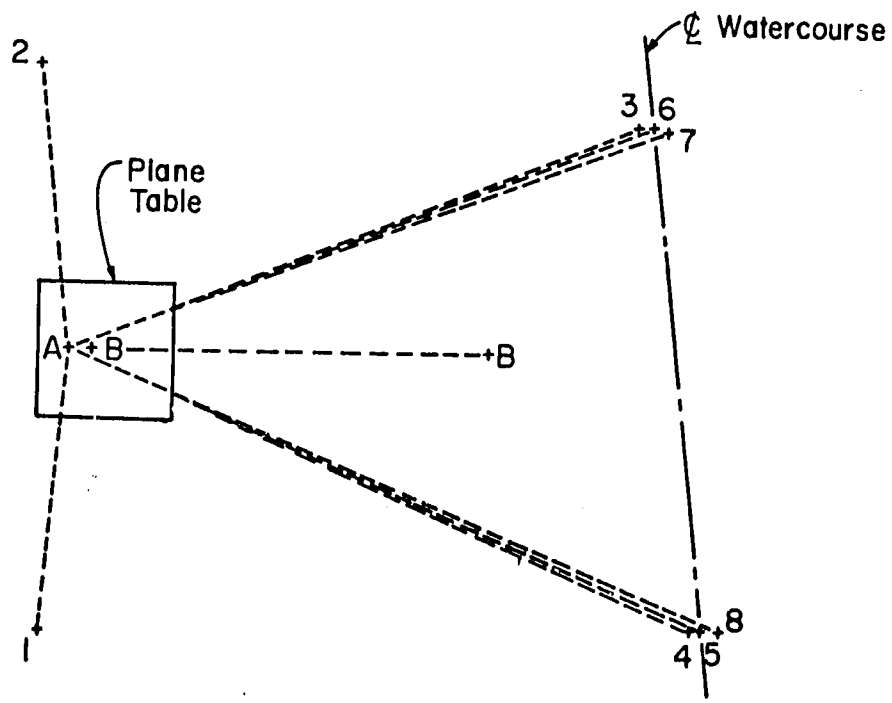


Figure A.4.3. Step 1 in plane table mapping. (Plant table over Point A.)

are chosen in the next field, as shown in Figure A.4.4. The distances from B to C and C to D are taped and the procedure continues for mapping the entire watercourse area.

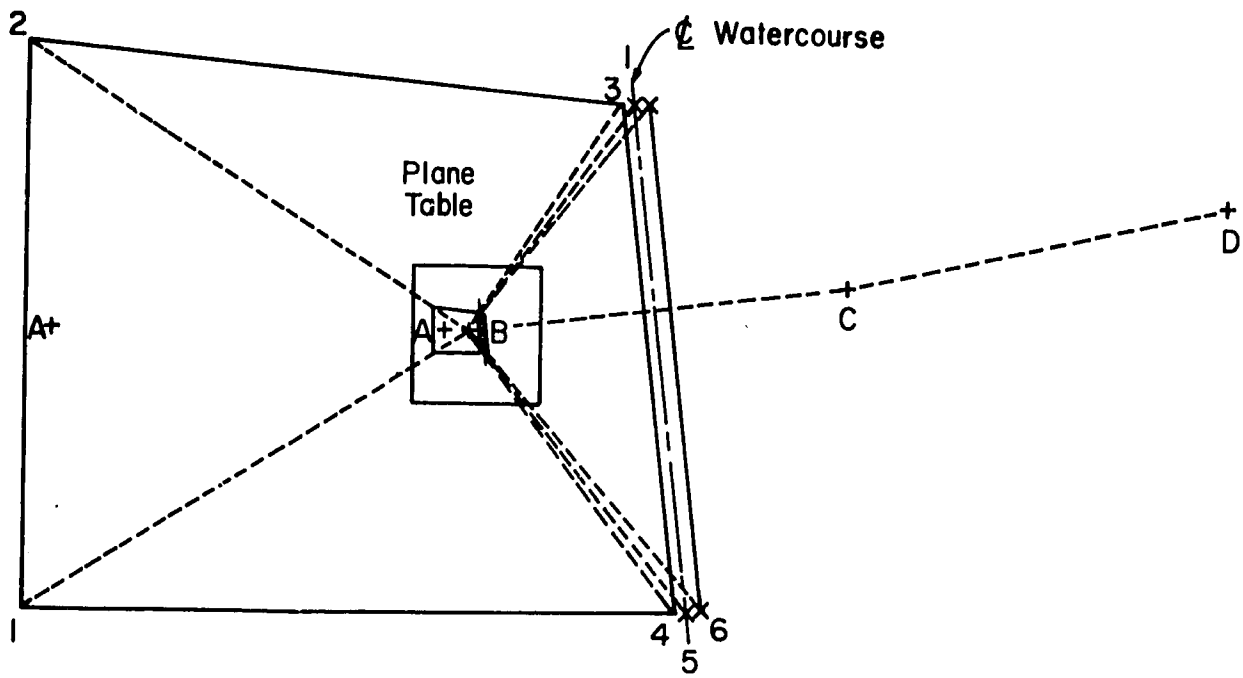


Figure A.4.4. Step 2 in plane table mapping. (Plane table over Point B.)

Appendix 5

DIFFERENTIAL LEVELING FOR BENCH MARK SURVEY OF IRRIGATION COMMAND AREAS[1]

Differential leveling is the process of finding the differences in elevation of any two points. It usually requires several setups of the instrument along a general line between the two points. Each setup requires a rod reading on a point of previously determined elevation and a rod reading on a point of unknown elevation. A bench mark survey is conducted to provide a widely spaced series of permanent or semipermanent points of known elevation from which a topographic survey is conducted at a later date.

Theory of Leveling

Leveling is the process of determining the elevations or differences in elevations of points. Figure A.5.1 illustrates the basic procedure.

A level is set up at a location approximately half-way between a bench mark (B.M.) and a turning point (T.P.). A bench mark is a relatively permanent natural or artificial object bearing a marked point whose elevation is known or assumed. A turning point is a temporary bench mark for the purpose of continuing a line of levels. Portable turning points are provided for field watercourse surveys and topographic surveys.

For example, referring to Figure A.5.1, if the elevation of the bench mark ($B.M._1$) is assumed to have an elevation of 100.000 meters, the elevation of the turning point ($T.P._1$) can be determined by leveling. First the instrument is set up approximately half-way between $B.M._1$ and $T.P._1$ and is leveled. A rod reading is taken on $B.M._1$ of 2.40. This rod reading is termed a backsight (B.S.). A backsight is a rod reading taken on a bench mark or turning point of known elevation. It is the vertical distance between the B.M. and the line of sight of the instrument. The line of sight of the instrument is almost always higher than the B.M. or T.P., therefore, the backsight is almost always positive and could be described as a plus sight.

The height of instrument (H.I.) is the elevation of the line of sight when the instrument is level. This corresponds to the line of sight of the instrument and is obtained by

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1/Taken from Water Management Methodology Series #6 of the same title by Wayne Clyma and Alan Early. Water Management Research Project, Colorado State University, Fort Collins.

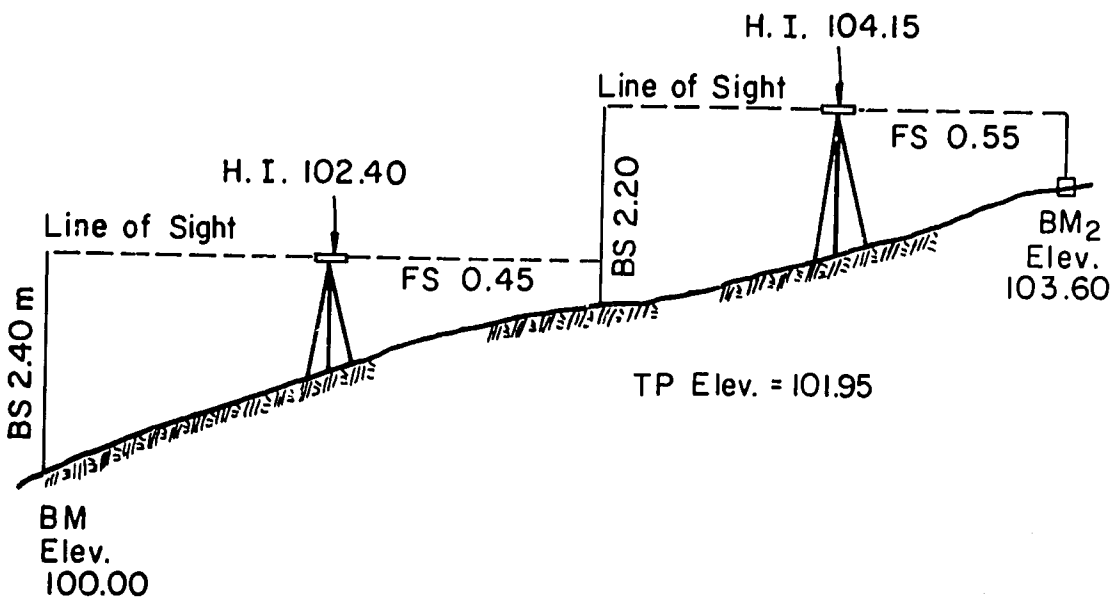


Figure A.5.1. Theory of differential leveling.

adding the backsight to the elevation of the B.M. or T.P. In Figure A.5.1, the B.S., 2.40, is added to the elevation of $B.M._1$, 100.00, to obtain the H.I., 102.40. Turning the telescope to bring into view the rod held on $T.P._1$, a rod reading called a foresight is obtained. A foresight (F.S.) is a rod reading taken on a turning point of any other point of unknown elevation for which the elevation is to be determined. The foresight is almost always subtracted from the height of the instrument and could be described as a minus sight. In the example of Figure A.5.1, the F.S., 0.45, is subtracted from the H.I., 102.40, to obtain the elevation of T.P., 101.95. Thus, by a process known as leveling, we have determined the difference in elevation of two points.

From the above definitions, we can derive two equations that are quite beneficial in leveling. These equations will be repeated numerous times during a leveling exercise and the student should become thoroughly familiar with them. They are:

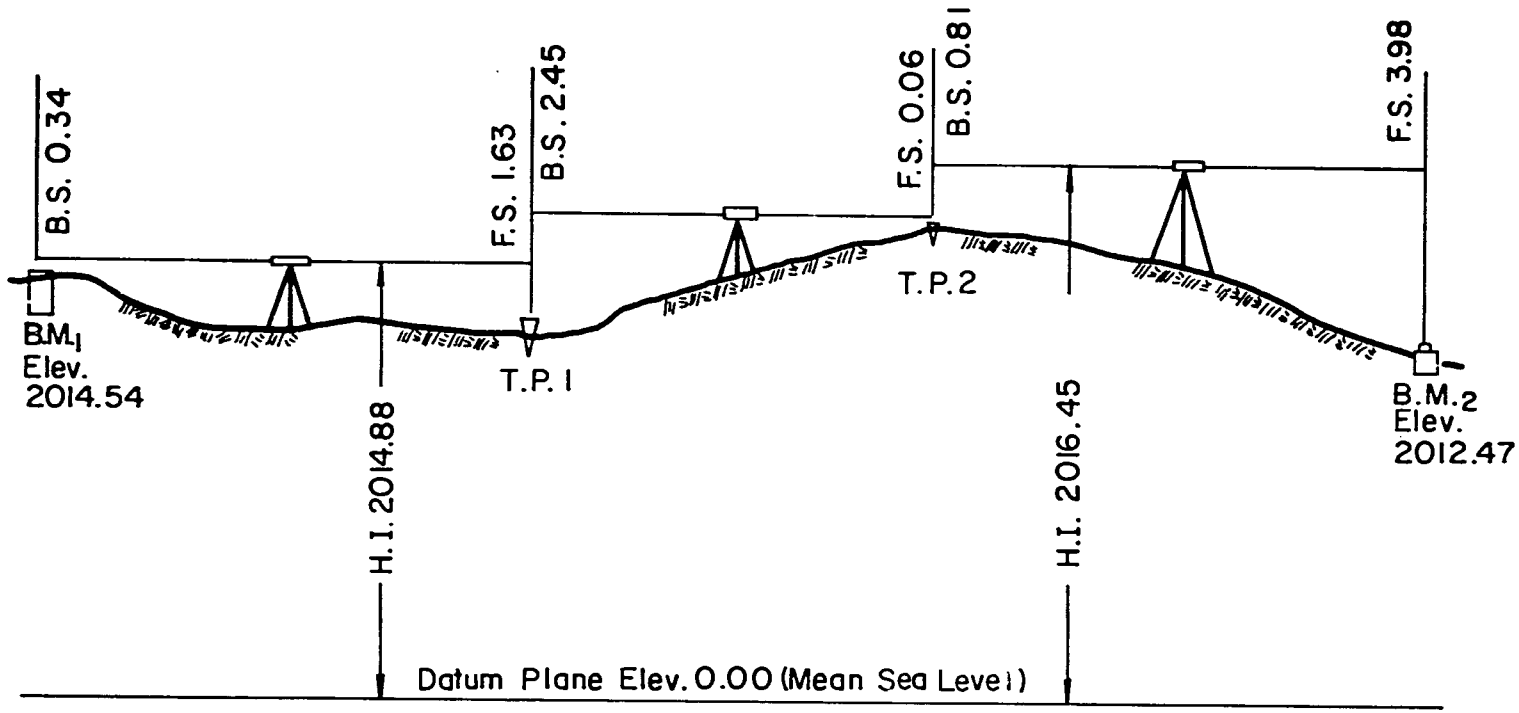
Elev. + B.S. = H.I.
H.I. - F.S. = Elev.

Procedure for Differential Leveling

Several procedures and precautions should be observed for accurate differential leveling. Refer to Figures A.5.2 and A.5.3 for sample rod readings and a sample set of notes for a differential leveling exercise.

To begin a differential survey, the rodman holds a rod on $B.M._1$ while the levelman goes forward a convenient distance (not over 100 meters) and sets up the level. The levelman takes a reading on the rod and determines where the middle crosshair strikes the rod, in Figure A.5.2 at 0.34. This is a backsight and the notekeeper records 0.34 in the B.S. column of the notes. Now the H.I. is determined by adding the B.S. to the elevation of the B.M., 100,00 + 0.34 = 100.34, and is entered in the notes in the H.I. column. For the convenience of the student, a plus sign can be placed above the B.S. column and a minus sign above the F.S. column to indicate how that column is used in note computations.

After the B.S. has been obtained, the rodman steps the distance from the B.M. to the instrument and then steps the same distance away from the instrument in the direction of $B.M._2$. This pacing is one of the acceptable methods of balancing the horizontal distances between the backsight and the foresight. This distance can also be measured by stadia, but the accuracy obtained by taping is not considered necessary. The effects of refraction, curvature of the earth, and lack of instrument adjustment are thereby eliminated. On slopes a zigzag path may be taken to utilize the longer rod length available on the downhill sights.



DIFFERENTIAL LEVELING

Figure A.5.2. Illustrating the procedure for taking backsights and foresights of differential leveling.

DIFFERENTIAL LEVELING
FOR BENCH MARKS
Alemaya College Campus

Sunny, calm

Haily S. & N
Tewolde W.
Aug. 18, 1965

Sta	BS	HI	FS	Elev.	
BM_1	0.34	100.34		100.00	$B.M._1$: the southern bolt on the pole
TP_1	2.45	101.16	1.63	98.71	nearest to the irrigation lab.
TP_2	0.81	101.91	0.06	101.10	about 5 m. from eastern side
BM_2	3.98	101.91	3.98	97.93	door
TP_3	0.09	101.19	0.81	101.10	
TP_4	1.98	100.71	2.46	98.73	$B.M._2$: Steel rod on the N.E. corner of
BM_1			0.69	100.02	the cattle guard into the live-
BS =	9.65	FS =	9.63	0.02	stock area across from poultry.

Error of closure =

9.65 - 9.63 = 0.02 check

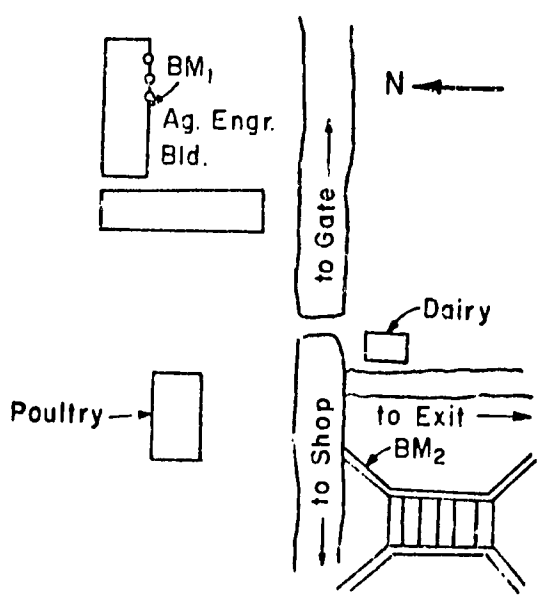


Figure A.5.3. Sample differential leveling notes.

Closed Surveys

To verify the accuracy of the leveling, a return check must always be made. That is, the line of levels must be continued from $B.M._2$ back over a slightly different route to $B.M._1$, the initial starting point. To make the return check independent of the first line of levels, after the F.S. is taken on $B.M._2$, the student should lift up the level and re-level it so that the H.I. will be at a slightly different elevation. This results in a B.S. on $B.M._2$ different from the F.S. and should result in a better check of the line of levels. When the surveyor has returned to $B.M._1$, he has completed a closed survey. All leveling exercises should be closed surveys so that a check of the accuracy of the survey can be made. Figure A.5.4 gives a sample traverse for a bench mark survey of an irrigation channel. Note that all available permanent structures are used as bench marks. These bench marks must be marked with both a waterproof marker and with a nail or screwdriver scratch for permanence. Figure A.5.5 provides the sample survey notes for the traverse of Figure A.5.4. Note the double tabling of points which are both turning points (T.P.) and bench marks (B.M.). Portable turning points are labeled PTP.

Error of Closure

If there have been no errors made in a closed survey or if the errors have compensated, then the elevation determined for $B.M._1$ by the return check will be the same as the original elevation of $B.M._1$. Generally these elevations are not the same because of errors in rod readings or instrumental errors.

The amount of which the original B.M. elevation and the B.M. elevation observed upon the return check fail to agree is called the error of closure.

Allowable errors of closure for a survey is a function of the accuracy of the instrument and the length of the survey or the number of times the instrument is set up. The allowable error of closure equals 0.01 meters per two instrument setups (0.005m/setup). For the notes shown in Figure A.5.3, there were four instrument setups so the allowable error of closure was 0.02. The actual error of closure was 0.02 so it is within the limits of error for the survey. For general leveling purposes with available equipment, the allowable error in terms of length of survey is given by:

$$\text{allowable error} = 0.0004\sqrt{\text{length of traverse in meters}}$$

If the sample traverse for Figure A.5.4 were 7000 m then the allowable error is 0.033 m or 0.03m. Bench mark surveys which do not meet this standard must be completely resurveyed until the error of closure is less than the allowable error.

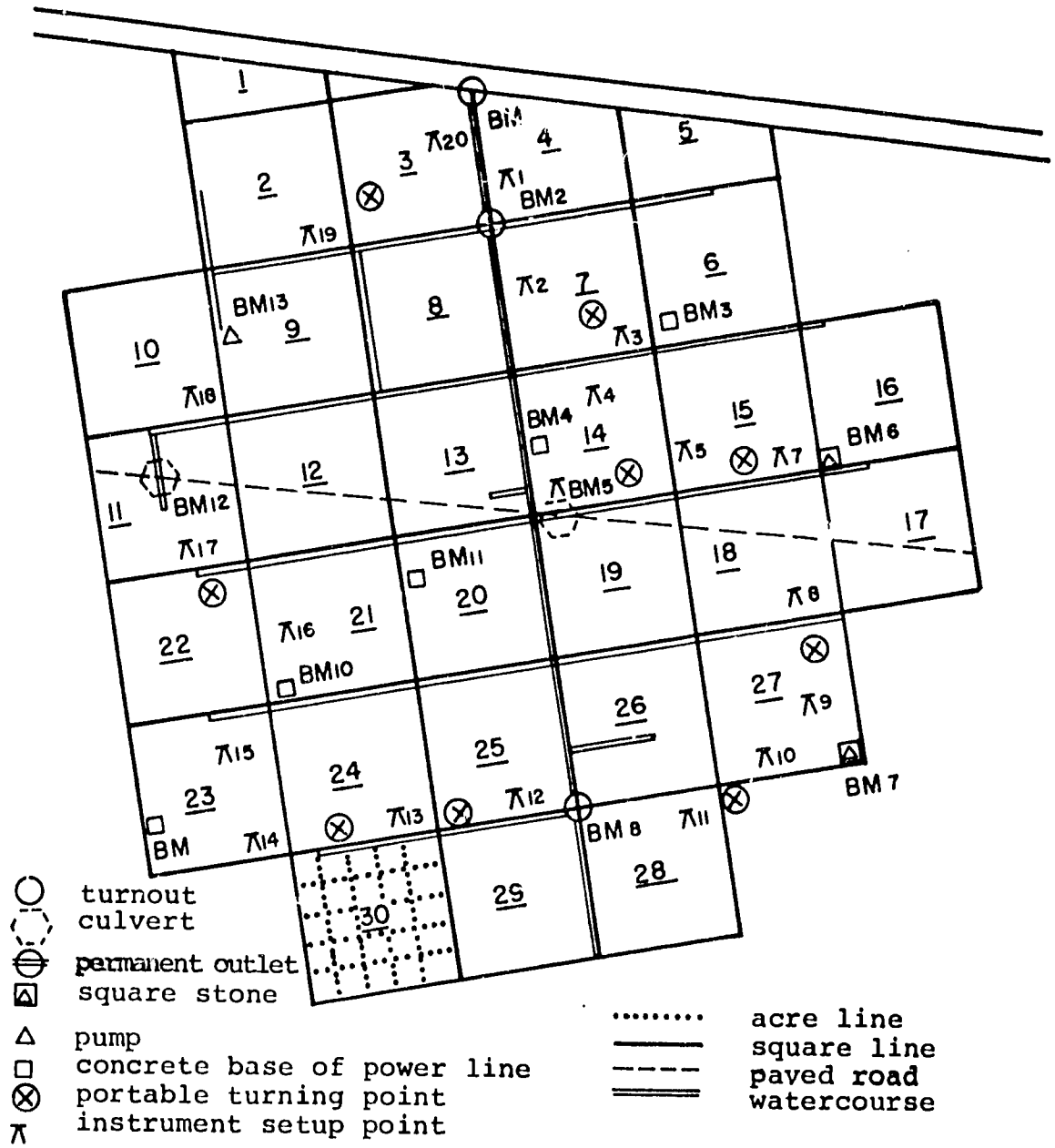


Figure A.5.4. Sample traverse for bench mark survey of an irrigation channel conveyance system.

IRRIGATION CHANNEL BENCH MARK
SURVEY BY DIFFERENTIAL LEVELING
Chak 110/JB, Lyallpur

Sunny — Waryam
Hot — Zahid
Calm — August 18, 1975

Sta.		BS	H.I.	FS	Elev.
	BM_1	3.02	103.02		100.00
TP_1	BM_2	4.22	103.17	4.07	98.95
TP_2	TP_2	3.89	102.05	5.01	98.16
TP_3	BM_3	6.87	104.85	4.07	97.98
TP_4	BM_4	4.48	102.44	6.89	97 96
TP_5	BM_5	4.32	101.75	5.01	97.43
TP_6	TP_6	5.10	101.60	5.25	96.50
TP_7	TP_7	4.88	101.50	4.98	96.62
TP_8	BM_6	3.37	99.61	5.26	96.24
TP_9	TP_9	3.85	99.28	4.18	95.43
TP_{10}	BM_7	4.71	99.98	4.01	95.27
TP_{11}	TP_{11}	4.85	99.56	5.27	94.71
TP_{12}	BM_8	5.17	99.44	5.29	94.27
TP_{13}	TP_{13}	4.37	98.81	5.00	94.44
TP_{14}	TP_{14}	3.99	98.28	4.52	94.29
TP_{15}	BM_9	4.00	98.37	3.91	94.37
TP_{16}	BM_{10}	6.02	100.77	3.62	94.75
	BM_{11}			/5.91/	94.86
TP_{17}	TP_{17}	6.27	101.77	5.27	95.50
TP_{18}	BM_{12}	5.33	102.82	4.28	97.49
TP_{19}	BM_{13}	4.81	103.74	3.89	98.93
TP_{20}	TP_{20}	4.68	104.72	3.80	99.94
	BM_1			4.51	100.11
BS =		98.20	FS =	98.09	0.11 = Elev. Diff.

Turnout Scratch Mark - Upstream Side

Concrete corner stone 3, 4, 7 and 8

Sq. 6: CBPL* - Scratch

Sq. 14: CBPL - Scratch

Sq. 13, 14: Concrete culvert - upstream

Sq. 15,16,17,18: Cornerstone - scratch

Sq. 25,26,28,29: Cornerstone - scratch

Concrete outlet corner of squares 25, 26, 28, 29

Sq. 23: CBPL - scratch

Sq. 21: CBPL - scratch

Sq. 20: CBPL - scratch

Sq. 11: Concrete Culvert:Scratch-Downstream

Sq. 9: Steel frame on Pump Base - scratch

Turnout

Error of closure = 98.20 - 98.09 = 0.11

*Concrete Base of Power Line - CBPL

Figure A.5.5. Irrigation channel bench mark survey notes by differential leveling.

The survey whose notes are provided in Figure A.5.5 does not meet the standard and must be repeated.

Checking the Level Notes

The computations of the level notes should always be checked by comparing the difference between the sum of the backsights (B.S.) and the sum of the foresights (F.S.) with the differences between the initial and final elevation of the B.M. used to close the survey before leaving the field. This computation checks the notes for arithmetical accuracy. The two differences must agree or an arithmetical error has been made. No set of leveling notes is complete without an error of closure computation and check of the arithmetical accuracy of the notes, before leaving the field.

Use of the Bench Mark Survey

The bench mark survey when completed to the required standard of accuracy becomes a basis for additional field surveys that are completed subsequently. The bench marks can then become starting points for any portion of the profile leveling survey of the water channels and of the topographic survey of the command area. This set of bench marks is then a set of known elevations to facilitate the completion of these other surveys.

Practical Considerations

While profile leveling, bench mark or topographic surveying, the need for intermediate turning points arises frequently. The placement of the staff rod on the ground or on the bed of a watercourse can lead to errors of major magnitude, if when the rod is rotated in contact with soil changes in rod elevation occur. To avoid this problem and source of inaccuracy, portable turning points are provided for field use. Figure A.5.6 below shows this simple surveying item. It is nothing more than a four-inch square piece of #14 or #16 sheet metal with one inch corners bent down at 90° from face to form four legs and with a rivet in center to hold the chain handle to bottom side. The rod is rotated on the rivet after portable turning point is forced into ground.

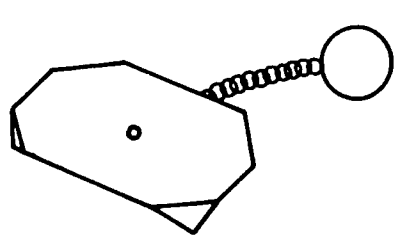


Figure A.5.6. Portable turning point.

Appendix 6

TOPOGRAPHIC MAPPING OF AN IRRIGATION COMMAND AREA

A topographic map is a map of an area which shows the elevations of the land surface. These elevations can be noted in terms of elevations at specific points or contour lines of points of equal elevations. Figure 6.23 shows a topographic map of an area commanded by one branch channel including field elevations and contour lines.

A topographic map is made by adding land surface elevations to a base map. The elevations are measured with a surveyor's level. A benchmark is used (see Appendix 5) as the standard elevation. The detail of the survey will depend upon the use to be made of the information. For the purpose of designing irrigation conveyance systems, the survey should be sufficiently detailed to insure that water is delivered to each irrigated field at the proper elevation. Thus the average elevation of each field surface must be determined for level basin irrigation fields, or the elevation of the head of the field for irrigation down slopes. The number of shots required to determine an average field elevation will depend upon field size and undulations.

Field elevation data can best be referenced during surveying by assigning a number to each field. This can be done on a copy of the base map such as is shown in Figure A.6.1, or on sketches on the field book. A survey should first be run from the nearest benchmark (BM) to an instrument setting which allows shots to be taken of several fields, using as many turning points (TP) as necessary. If the benchmarks are well placed, the benchmark shot can be made from the initial instrument setting in the area to be surveyed. Intermediate shots (IS) should then be taken of all fields within instrument range before the level is moved, via a turning point, to a new location. This process should continue until all fields in the area are surveyed, after which the survey must be closed to the original or another benchmark. Allowable closure errors are discussed in Appendix 5. During the survey, the elevations of any permanent structures which will be used in the conveyance system, such as turnouts or culverts should also be measured.

Figure A.6.2 shows the survey data taken of the fields shown in Figure A.6.1. Figure 6.23 is the final topographic map of the area. Contour lines, which divide the fields into elevation categories, were drawn every 0.2 m on the map. The lines should first be sketched in lightly along field boundaries between fields whose elevations lie above or below the chosen line elevation. They can then be

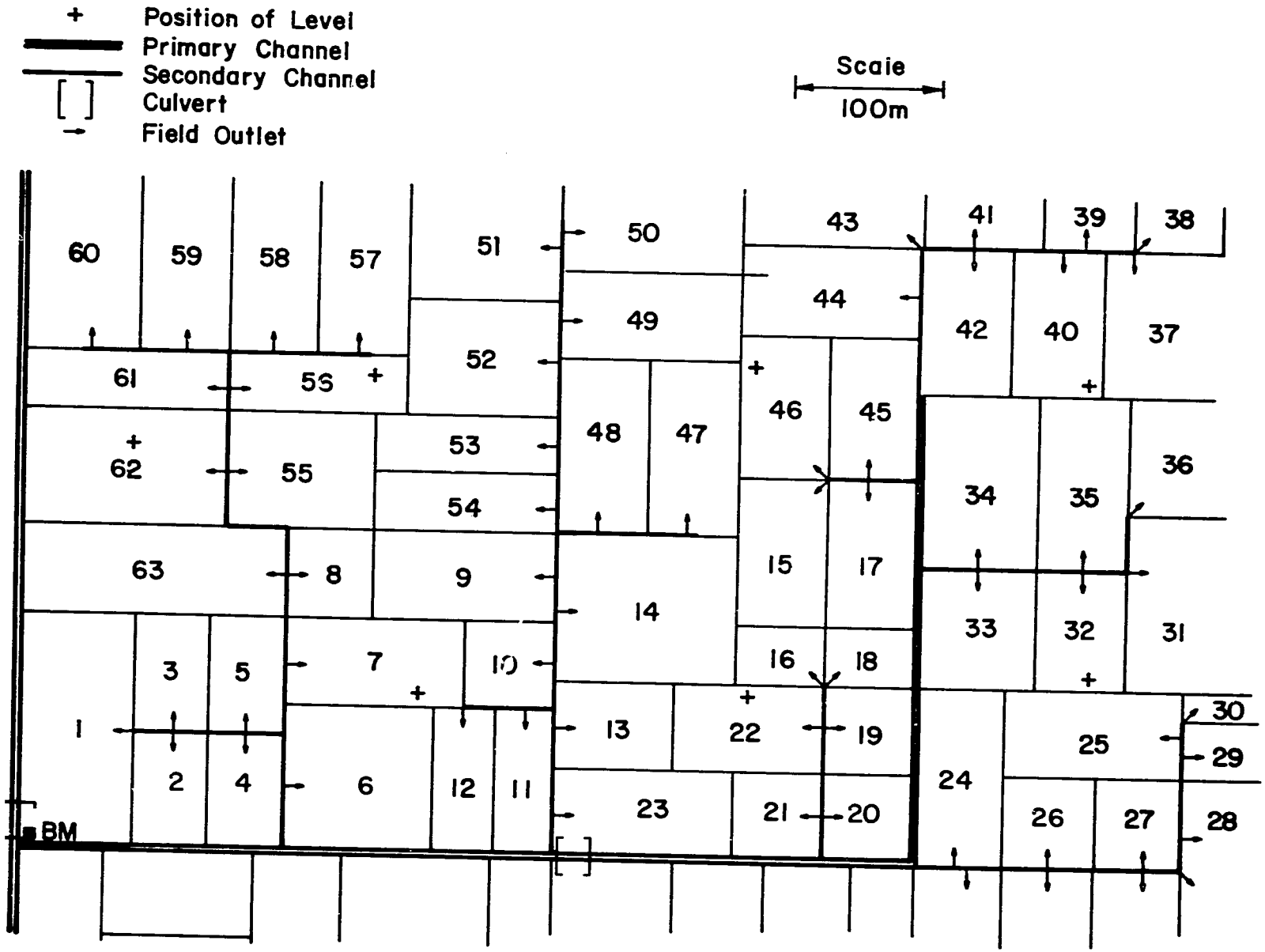


Figure A.6.1. Base map of area commanded by a primary branch showing field and channel layout and field numbering for topographic survey data collection.

Topographic Leveling of the Command
Area of Branch D of Turnout #347R
August 23, 1980; Zahid Saeed and Abdul Khaliq
Weather Cloudy and Windy

Field	BS	HI	FS	I.S.	Elev.
BM_1	1.16	97.44			96.28
1				1.67	95.77
2				1.67	95.77
3				1.75	95.69
4				1.73	95.71
5				1.79	95.65
TP_1	1.30	97.12	1.62		95.82
6				1.47	95.65
7				1.49	95.63
8				1.52	95.60
9				1.61	95.51
10				1.56	95.56
11				1.54	95.58
12				1.51	95.61
TP_2	1.23	96.88	1.47		95.65
13				1.43	95.45
14				1.48	95.40
15				1.59	95.29
16				1.60	95.28
17				1.63	95.25
18				1.63	95.25
19				1.59	95.29
20				1.50	95.38
21				1.48	95.40
22				1.53	95.35
23				1.40	95.48
TP_3	1.55	96.80	1.63		95.25
24				1.52	95.28
25				1.60	95.20
26				1.53	95.27
27				1.57	95.23
28				1.60	95.20
29				1.60	95.20
30				1.63	95.17
31				1.66	95.14
32				1.63	95.17
33				1.59	95.21
TP_4	1.52	96.77	1.55		95.25
34				1.59	95.18
35				1.65	95.12
36				1.68	95.09
37				1.75	95.02
38				1.76	95.01
39				1.75	95.02
40				1.74	95.03
41				1.68	95.09
42				1.68	95.09

Figure A.6.2. Field notes for topographic survey of the fields shown in Figure A.6.1.

Field	BS	HI	FS	I.S.	Elev.
TP_5	1.56	96.63	1.70		95.07
43				1.54	95.09
44				1.53	95.10
45				1.41	95.22
46				1.41	95.22
47				1.28	95.35
48				1.26	95.37
49				1.31	95.32
50				1.34	95.29
TP_6	1.69	97.02	1.30		95.33
51				1.77	95.25
52				1.71	95.31
53				1.67	95.35
54				1.55	95.47
55				1.50	95.52
56				1.57	95.45
57				1.64	95.38
58				1.60	95.42
TP_7	1.40	96.93	1.49		95.53
59				1.50	95.43
60				1.40	95.53
61				1.35	95.58
62				1.28	95.65
63				1.28	95.65
TP_8	1.56	97.38	1.11		95.82
BM			1.07		96.31
	12.97		12.94		

BS - FS = .03

$BM_2 - BM_1 = .03$ <u>check</u>

Figure A.6.2. Continued.

smoothed to represent natural land surface elevations and slopes. The general slope in an area in a certain direction can be found by counting the number of contour lines crossed over a given distance in that direction, multiplied by the contour interval, and divided by the distance. For example, the primary channel shown in Figure 6.23 crosses two contour lines from the first check structure to the bend, a distance of 425 m. Since the contour interval is 0.2 m, the slope of the land surface along the channel is:

$$\frac{\text{2 contours x 0.2 m per contour}}{\text{425 m distance}} = 0.0009 \text{ m/m.}$$

Narrowly-spaced contours represent steeper slopes while wider spacing indicates relatively gentle slopes. A field which does not fit into its contour interval or causes a large deviation in a contour line should be rechecked.

Appendix 7

AN EXAMPLE OF MAIN CHANNEL DESIGN

To further clarify the channel design procedure described in Section 6, the main channel shown in the base map in Figure A.7.1 will be designed. The 2000 m main channel serves the commanded area from 8 primary branches labeled on Figure A.7.1 from A to H. As stated in Section 6.2.3, the design procedure begins at the fields, progresses up each field branch and then to the primary branches before the main channel can be designed.

Branch design profiles, such as the example carried out in Section 6, will determine the required main channel outlet elevation at each branch outlet. For example, the branch designed in Section 6 is assumed to be branch D in Figure A.7.1. The branch design profile (Figure 6.24) shows a required inlet elevation of 96.05 m. The main channel outlet consequently must provide water to branch D located 1000 m from the canal turnout at this elevation or higher. This elevation along with the inlet elevation for each branch is put on a main channel profile design sheet, shown in Figure A.7.2. The horizontal axis on this profile follows the main from the canal turnout to the tail. The location of each branch is marked on the axis as well as a proposed culvert.

The vertical axis represents the elevation with respect to the bench marks. The axis should be scaled such that the highest and lowest outlets plus allowances for freeboard (+0.15 m) and bottom elevations (≈ -0.60 m) will fall within the elevation range.

The design elevation of the water surface below the canal turnout should also be designated on the vertical axis. In the example, the turnout is of the open flume, critical flow type which begins flowing freely at a water surface elevation of 96.80 m, so the elevation must be maintained below this level to insure free flow. As an added safety factor, the design elevation is decreased an additional 0.10 m to allow for flow depth increases due to sedimentation or vegetation growth without submerging the turnout. Thus, 96.70 m is chosen as the design maximum water surface elevation at the canal turnout. The elevation at the head should not be higher than this value.

As was done on the branch profile, outlet elevations should be plotted on the graph. The water must be delivered to each outlet at this elevation or higher, or else the branch channel must be redesigned to lower the elevation.

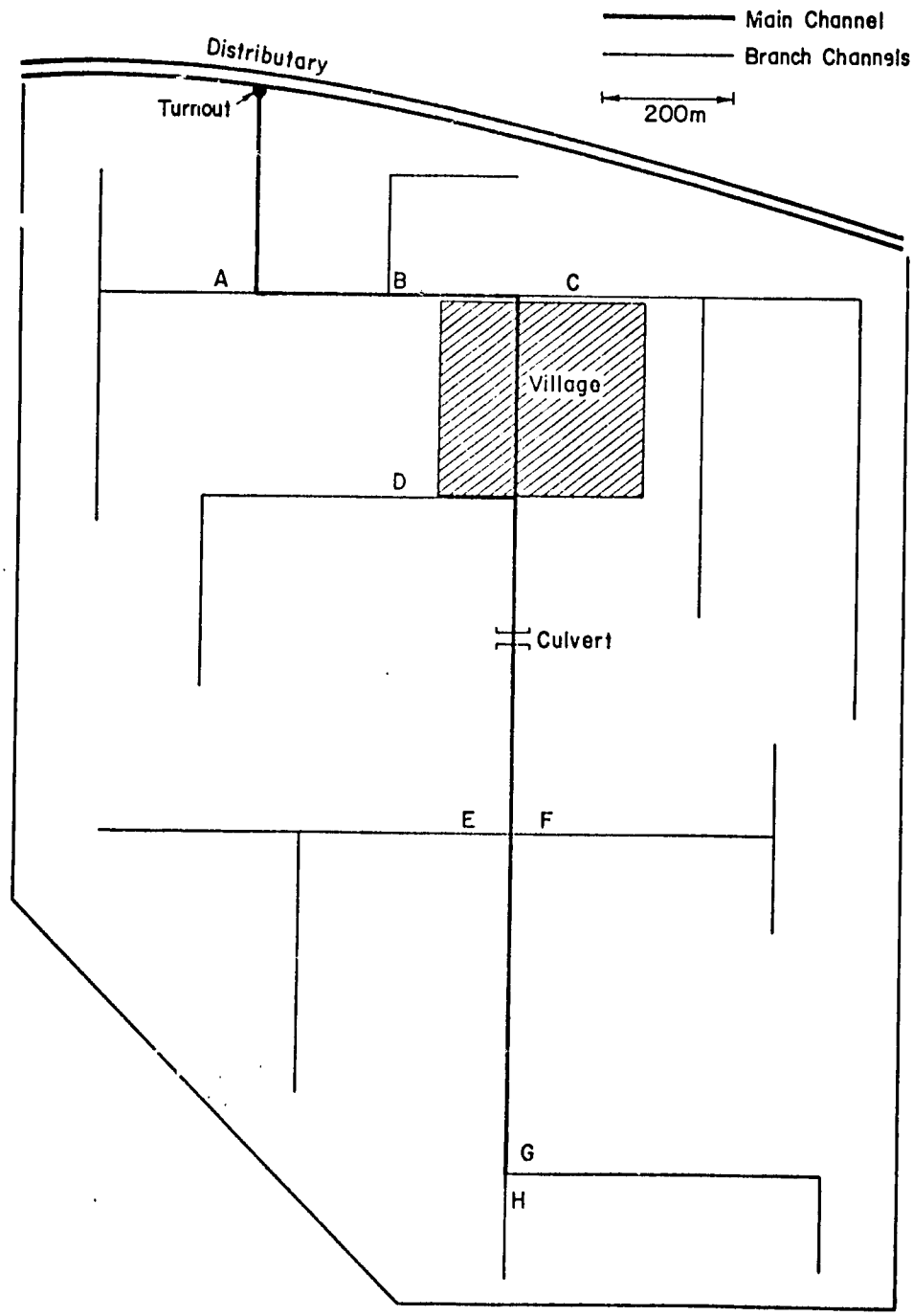


Figure A.7.1. Layout of the main and branch channels of the sample command area.

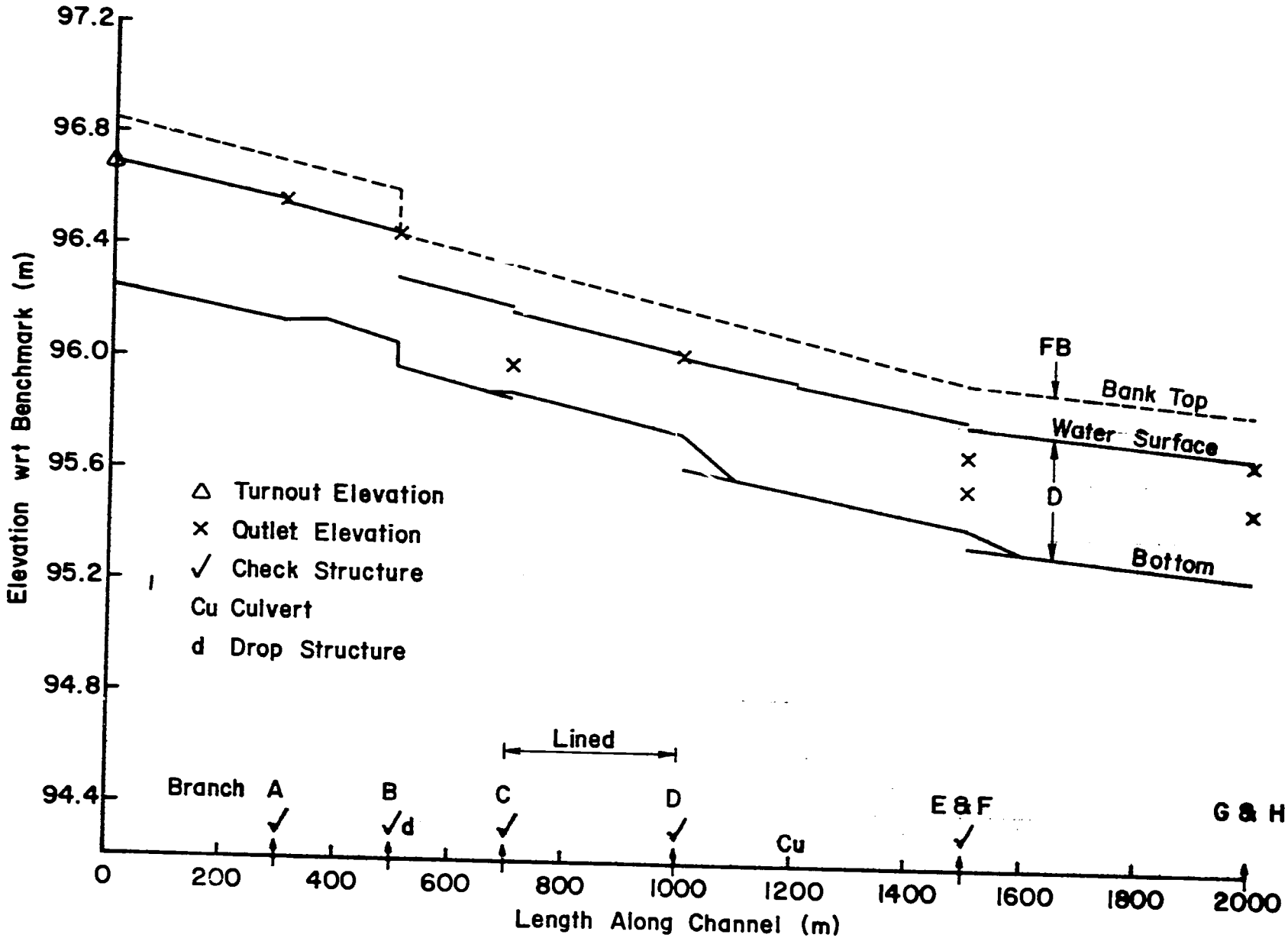


Figure A.7.2. Channel profile of the main channel shown in Figure A.7.1.

The proposed water surface profile can now be plotted connecting or exceeding each outlet elevation. For this purpose, the main is divided into 4 sections in which the outlet elevations fall roughly in a straight line. The slopes of this preliminary profile should be measured to see if they are reasonable. The initial section slope is limited by the maximum turnout elevation. The tail section slope is very flat, and is limited by the chosen minimum of 0.2 m/km. A drop is designed between the second and third section so that the channel can be built as low as possible.

Between branch C and D (700-1000 m) the main passes through the village. Because an earthen channel would deteriorate quickly in the village and the wider top width would make passage difficult, a rectangular lined section will be built. Thus, because of the lined section, the main is divided into a total of 6 subsections for design purposes.

The location of proposed checks and other structures is also added to the design profile. Each outlet is given a check in the example, and a culvert is located 1200 m from the turnout.

The information from the profile sheet should be recorded on a design sheet, shown i[illegible]gure A.7.3. Each subsection's length, preliminary sl[illegible]nd structures should be noted. Structures locatea [illegible]ween subsections are listed between subsections. The design inflow rate at the turnout and assumed loss rate should also be noted on the sheet. In the example, the maximum expected inflow rate is determined from measurements and information from the local Irrigation Department officials to be 52 lps. Previous experience shows that 1.0 lps is a reasonable loss rate in earthen improved channels. No losses are assumed in the lined sections. Design flow rates in each subsection are then calculated by Eq. 18 and listed on the sheet. Utilizing the flow rate and a graph such as Figure 6.26, the head losses through the structures are calculated and listed. In the example, 0.38 m diameter orifice checks are used as well as a 0.38 m diameter culvert. The head loss varies between 0.02 and 0.01 m for the decreasing flow rates.

Between sections 2 and 3, either a steep slope or a drop must be used. A drop of the water surface will allow the channel to be built lower, and thus is chosen. The amount of drop is determined by the elevation difference between the upstream and downstream water surfaces. In the example, this drop is 0.16 m. Such a drop can be accomplished by use of a raised, small-sized check structure with an extended downstream section to prevent erosion.

MAIN CHANNEL DESIGN SHEET

INFLOW RATE, Q (lps) 52 lps Assumed Loss Rate, Q_L (lps/100 m) = 1 lps/100 m in earthen, 0 in lined section

Maximum Inlet Elevation 96.80 m Design Inlet Elevation 96.70 m

Section	Lenth (m)	Preliminary slope (m/km)	Structure head loss (m)		OL Elevation Head (m)	OL Elevation Tail (m)	Final slope (m/km)	Design flow rate (lps)	Design cross section Z	B (m)	D (m)	TW (m)
1	300 m	0.4			96.70	96.58	0.4	51	1	.3	.42	1.12
			.40 m check:	.02								
2	200 m	0.6			96.56	96.46	0.5	49	1	.3	.39	1.04
			check/drop	.16								
3	200 m	0.5			96.30	96.20	0.5	46	1	.45	.32	1.09
			.40 m check:	.02								
4	300 m	0.5	LINED SECTION		96.18	96.04	0.47	45	0	.49*	.28	.49
			.40 m check:	.01								
5	500 m	0.5			96.03	95.9	0.42	43	1	.3	.38	1.06
			.40 m check:	.01								
6	500 m	0.2			95.80	95.7	0.2	38	1	.3	.43	1.16
			.40 m check:	.01								

Freeboard Allowance (m) 0.15m Bank Top Width (m) 0.60 M

*Assume n = .018, So $Qn/\sqrt{S}$ = .038; and b/d = 1.2

Figure A.7.3. Design sheet for main channel of sample watercourse.

Final slopes are now calculated using Eq. 22 and the elevations at each subsection head and tail taken from the profile. For example, in section 5, the elevation below the upstream check is 96.03 m, the elevation above the downstream check is 95.81 m, and a culvert in the section causes a 0.01 m head loss. The calculated slope by Eq. 22 is:

$$\frac{96.03 - 95.81 - 0.01}{500 \text{ m}} = .00042 = 0.42 \text{ m/km}.$$

With design flow rates and slopes established, the cross sections are determined from the design graphs by the method previously explained. Bottom widths of 0.30 m are chosen initially in the earthen sections because they result in depths only somewhat larger than the bottom widths. In the lined section, a wide shallow cross section is chosen to reduce costs.

In Section 3, just upstream of the lined section, the flow depth with a 0.30 m bottom width was 0.38 m. This would have caused an upward step in the bottom at the lined section of 0.08 m which would have caused some dead storage. Consequently, a wider bottom width (B = 0.45 m) and shallower depth (0.32 m) design was chosen to eliminate most of this dead storage.

The watercourse bottom and bank top elevations are now added to the design profile. The bank top elevation is drawn 0.15 m above the designed water surface profile, to reflect the freeboard allowance. The bottom elevations are equal to the water surface elevations minus the design depths. For example, at the start of section 5, the bed elevation is 96.03 m - 0.38 m = 95.65 m and the bank top elevation is 96.03 m + 0.15 m = 96.18 m. Where subsections join, there are abrupt steps in the bottom profile. These steps are smoothed out on the profile by extending straight lines from one section to the other. This smoothing out process if not considered in the design, will in fact occur naturally in earthen channels as a result of erosion and sedimentation.

A final check should be made of slope and elevation calculations by multiplying each section slope by its length, adding these calculated elevation drops to the head loss in each structure, and subtracting the total drop from the designed inlet elevation. The resulting elevation should then be compared with the main channel water surface elevation at the tail. The two values usually won't be equal because of rounding of the slope values but they should be within a few (2-4) centimeters or slope adjustments must be made. In the example, the tail elevation on the profile is 95.70 while that calculated from the design sheet is 95.69, well within acceptable limits.

Appendix 8

CHOOSING CROSS-SECTIONAL SHAPES TO MINIMIZE LINING COSTS

The efficiency of a cross-sectional shape refers to how short of wetted perimeter is required to provide a required hydraulic section or channel capacity (Eq. 12). Since lining costs are generally proportional to wetted perimeter length, a more efficient shape also requires less material. As stated in Section 8.1.3, the most efficient cross-sectional shape is a semicircle. The more sides a cross-sectional shape has, the more efficient shape is possible, and the most efficient shape for any number of sides is that shape which can be inscribed on a half circle. Figure A.8.1 depicts and Table A.8.1 describes several minimum wetted perimeter length shapes.

The most efficient trapezoidal shape has a side slope of $1/\sqrt{3}$ or about 0.58. This is often too steep for poured concrete and flatter side slopes are required. The most efficient shape for any side slope occurs when Eq. 23 is satisfied:

$$D/B = \frac{1}{2\sqrt{Z^2+1} - 2Z} \tag{23}$$

As indicated in Table A.8.1, with Z = 1, the wetted perimeter length is only 8% longer than the most efficient trapezoid (Z = 0.58) and 14% longer than the semicircle. The efficiency of trapezoidal shapes is not too sensitive to side slope values as long as proper D/B ratios for the chosen side slope are used. Even the rectangular shape (Z = 0) requires a wetted perimeter only 17% longer than the semicircle as long as a D/B ratio of 1/2 is used.

Occasionally, lining materials or thicknesses will vary between the base and sides. Trout (1979b) carried out a cost minimization procedure for this case utilizing the ratio of the unit cost of bottom and side linings, b/d, where b and d equals the cost of the base and side lining per unit area respectively for each chosen material and thickness. The graphical solution for this cost minimization procedure is given in the fourth set of solution graphs in Section 6.1.

To demonstrate the use of these graphs, assume that it is desired to construct a channel with an 8 cm thick concrete base which costs $\$3/m^2$ and 11 cm thick plastered brick masonry sides which cost $\$4.50/m^2$. The b/d ratio for this construction would be 0.67. Assume the required hydraulic section is 0.05. Since brick-masonry sides are being used, a rectangular shape is chosen. From solution graph 4-a (Figure

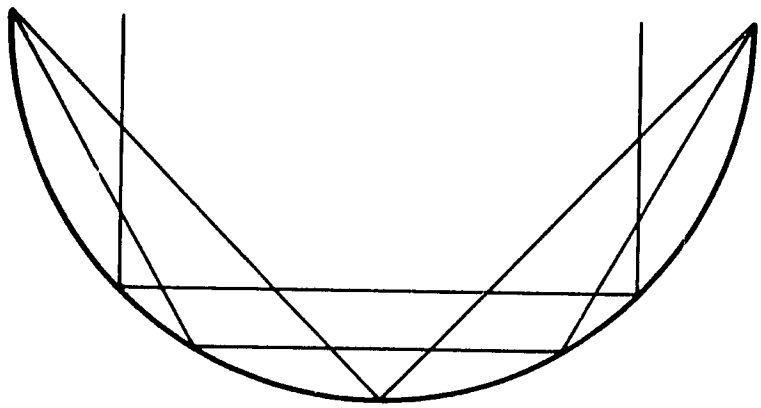


Figure A.8.1. Most efficient two and three sided cross-sectional shapes.

Table A.8.1. Design formula and relative wetted perimeter lengths of various efficient cross-sectional shapes.

Shape	Most Efficient Design Formula	Wetted Perimeter Length Relative to a Half Circ
Semicircle	$D = 1.00\ (Qn/\sqrt{S})^{3/8}$	100%
Trapezoid	$B = 1.12\ (Qn/\sqrt{S})^{3/8}$	106%
	$Z = 1/\sqrt{3} = 0.58$	
	$D/B = \sqrt{3/2} = 0.87$	
Trapezoid with Z = 1	$B = .786\ (Qn/\sqrt{S})^{3/8}$	114%
	$Z = 1.0$	
	$D/B = 1/(2\sqrt{2}-2) = 1.21$	
Rectangle (rotated triangle) (Z = 0)	$B = 1.83\ (Qn/\sqrt{S})^{3/8}$	117%
	$D/B = 1/2 = 0.50$	

6.12) (Z = 0) any combination of depth and bottom width which falls on the $Qn/\sqrt{S} = 0.05$ line will be a proper hydraulic design, but the B and D values where this 0.05 hydraulic section line crosses the b/d = 0.67 line eminating from the origin will give the lowest cost cross-sectional geometry. As shown in Figure A.8.2, these lines cross at B = 0.71 m and D = 0.26 m. A freeboard allowance must be added to this depth to get a total lining depth and to calculate the lining cost. The material costs of this cross-section would be:

$$2[(0.26\ m + 0.15\ m) \times \$4.50/m^2] + 0.70 \times \$3/m^2 =$$

$$\$5.82/m \text{ length.}$$

This is about 11% less than the costs of a rectangular cross-section in which the depth is equal to the bottom width.

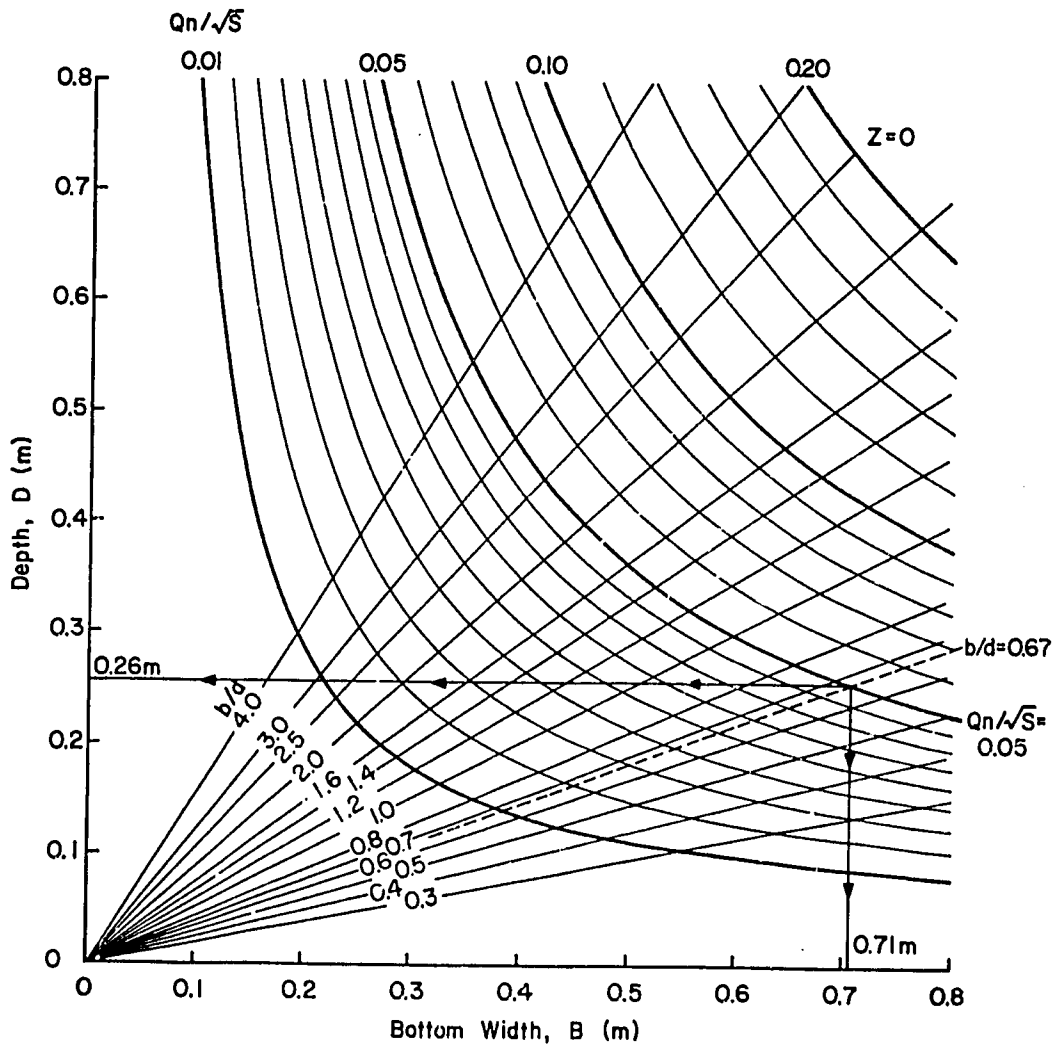


Figure A.8.2. Solution for lining problem with $Z = 0$, $b/d = 0.67$, and $Qn/\sqrt{S} = 0.05$.

Appendix 9

OPTIMALLY ALLOCATING ALTERNATIVE CHANNEL IMPROVEMENT TECHNIQUES TO A ROTATIONAL SYSTEM

(Adapted from Optimization of Lengths of Alternative Watercourse Improvement Program by John Reuss (1980).

The allocation of alternative improvement techniques to a rotational conveyance system where different portions of the system are used for different amounts of time will be done basically on economic grounds. In this analysis any technique which gives a positive net return is considered economically desirable and that which gives the highest net return is preferred. Net returns will vary with total water savings and thus with the amount of time a channel section is utilized in addition to initial loss rates and the percent of the losses which can be saved. The objective of this procedure is to determine how much time a section must be used for a given technique to give a positive net return or a higher return than any available alternatives. Once this usage time is established, the optimum improvement strategy can be found by determining how much time a channel section is used.

Net returns for a channel improvement, R ($/yr), is taken as the value of the saved water minus the costs of the improvement, both of which are calculated on an annual basis. Annual costs include the initial costs spread evenly over the project life utilizing an appropriate discount rate, plus the annual cleaning, maintenance, repair, and replacement costs. Annual costs are calculated on a per unit channel length basis, C ($/m/yr). The annual value of the saved water is the unit value of the water, V ($/m^3), times the volume saved in a year, W_s(m^3/yr). Thus:

$$R = VW_s - CL. \qquad (A.9.1)$$

Water volume saved, W_s, is equal to the flow rate increase in a channel times the amount of time the channel is used, t(fraction of a year). The flow rate increase can be represented as the initial flow loss (ΔQ)(lps) times a fractional decrease in the loss as a result of the improvement technique, f:

$$W_s = f\Delta Qt \qquad (A.9.2)$$

Both losses and costs are most easily expressed on a per unit channel length basis. When Eq. A.9.1 is converted to a per unit length basis with W_s from Eq. A.9.2 inserted, the net return per unit length becomes:

$$R_L = aVfQ_L t - C, \tag{A.9.3}$$

where:

R_L = net annual return per unit length (\$/m/yr).

Q_L = initial flow loss per unit length or the initial loss rate = $\frac{dQ}{dL}$ (lps/100m),

a = a volume and time conversion unit equal to 315 to convert from (lps/100m) to (m^3/m/yr).

As stated, an improvement is economically desirable for a channel section if R_L is positive. If two techniques are being compared, the usage time trade-off point where a switch should be made from one improvement technique to another is that point where the net returns of the two techniques are equal. If we call R_{Lh} the higher cost improvement net return and R_{Ll} the lower cost return, the trade-off point is when:

$$R_{Lh} = R_{Ll} \tag{A.9.4}$$

or when:

$$aVf_hQ_Lt - C_h = aVf_lQ_Lt - C_l \tag{A.9.5}$$

Solving this equation for usage time gives:

$$t = \frac{C_h - C_l}{315\ VQ_L(f_h - f_l)} \tag{A.9.6}$$

where:

t = the fraction of a year a given section is full,

C_h = the annualized unit costs of the higher cost alternative (\$/m/yr),

C_l = the annualized unit cost of the lower cost alternative (\$/m/yr),

f_h = the fractional water loss saved by the higher cost alternative, and

f_l = the fractional water loss saved by the lower cost alternative.

When an improvement alternative is being compared with no improvement, C_l and f_l are zero and Eq. A.9.6 reduces to:

$$t = \frac{C_h}{315\ VQ_Lf_h} \tag{A.9.7}$$

An example will clarify the use of Eq. A.9.6. Suppose three improvement techniques have shown themselves to be the best practical alternatives. The techniques, their costs, and the fraction of the losses they normally save is shown in Table A.9.1. The value of supplemental irrigation water is found to be fairly constant at $160 per hectare-meter or $0.016/m^3$. It is desired to evaluate the alternative strategies under low, medium and high initial loss rate values of 0.5 lps/100 m, 1.0 lps/100 m, and 2.0 lps/100 m.

Table A.9.1. Costs and water savings of alternative improvement techniques.

Technique	Fractional Loss Savings, f	Costs, C				
		Initial ($/m)	Projected Life (yrs)	Annualized Initial[1/] ($/m/yr)	Annual ($/m/yr)	Annualized Cost, C ($/m/yr)
Channel lining	.90	8.00	20	0.94	0.20	1.14
Earthen renovation	.50	.60	8	0.10	0.20	0.30
Cleaning and repair	.30	.10	1	0.11	0.15	0.16

[1/] At a discount rate of 10 percent.

For the medium loss rate channel (Q_L = 1.0 lps/100 m) the trade-off point between the lining and earthen renovation would be at a usage time of:

$$t = \frac{(1.14\ \$/m/yr - 0.30\ \$/m/yr)}{315\ (0.016\ \$/m^3)(1.0\ lps/100\ m)(.90 - .50)} = .417$$

or 42%. If lining were considered the only alternative, then if all channels which are used at least:

$$t = \frac{1.14\ \$/m/yr}{315\ (0.016\$/m^3)(1.0\ lps/100\ m)(.90)} = .503$$

or 50% of the time are lined, the maximum return would be gained. Table A.9.2 summarizes the optimum improvement strategies for the three alternatives at the three loss rate values. As can be seen from Table A.9.2 or Eq. A.9.6 the

Table A.9.2. Optimum allocation of improvement techniques according to channel usage time.

Improvement Technique	Combination Strategy			Single Technique Strategy		
	Loss Rate (lps/100 m)			Loss Rate (lps/100 m)		
	0.5	1.0	2.0	0.5	1.0	2.0
Channel lining	100-83	100-42	100-21	100-50	100-25	100-13
Earthen improvement	83-28	42-14	21-07	100-24	100-12	100-06
Cleaning and repair	28-21	14-11	7-05	100-21	100-11	100-05

trade-off point extends to lower usage time channel sections as the initial loss rate or the value of water increases.

Once a table such as A.9.2 is available, the only additional information required to optimally allocate the improvement alternatives is the initial loss rates and usage times of different channel sections on the concerned system. This usage information is most easily recorded on a graph such as Figure 8.9 or Figure A.9.1 which was devised for a hypothetical main channel. Note that the usage time required is the percent per year, so usage per rotation period must be multiplied by the percent of the year that the system is in use. For the channel in Figure A.9.1, if the loss rates are measured to be 1.0 lps/100 m, then, according to the values in Table A.9.2, 900 m of the channel should be lined, 700 m should be earthen improved, and 200 m cleaned and repaired to achieve the maximum net return on the improvement program. Since, due to the decreased loss rates in the improved sections, the flows will drastically increase below the improved sections, the loss rates at the tail would also be expected to increase, making it economical to extend the cleaning and repair work farther downstream. If the tail loss rates double (to 2.0 lps/100 m), 400 m should be cleaned and repaired; if they triple the improvement should be extended to 2400 m, or 800 m should be cleaned and repaired.

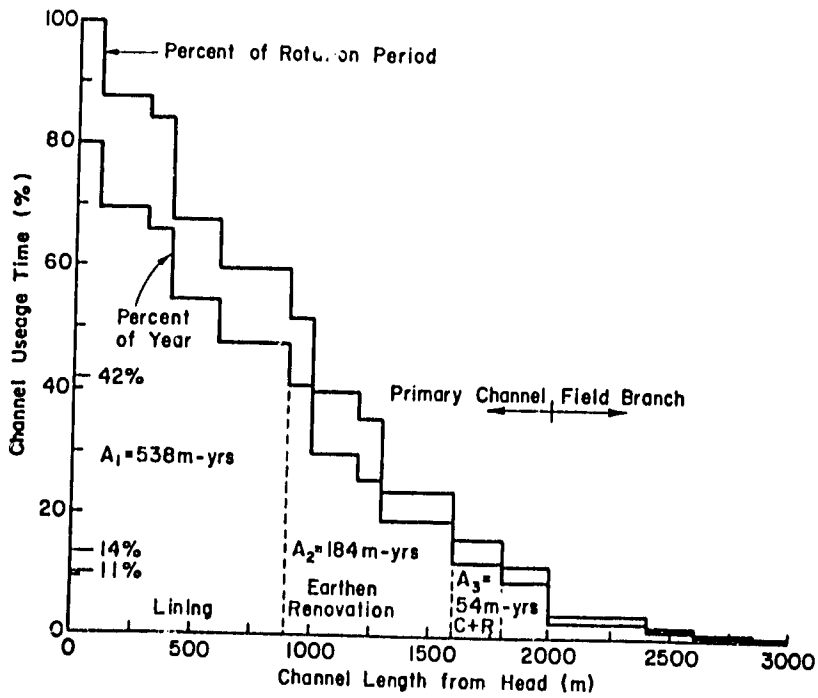


Figure A.9.1. A graph of usage time (percent per rotation period and percent per year) for the main channel on a conveyance system which is used 42 weeks per year.

Once the optimum strategy is determined, the net returns can be calculated using Eq. A.9.3 and the usage time graphs. This calculation cannot be made directly because the relationship between usage and length is not an exact relationship and thus its integral cannot be determined analytically.

$$R_n = \int 315VfQ_L\, dL - \int CL dL = 315VfQ_L \int dL - CL \qquad (A.9.8)$$

But the integral can be found by numerically integrating the usage time:length curve (i.e., Figure A.9.1). Numerical integration is equivalent to finding the area under the curve over the desired length interval, or to finding the length weighted average usage time, in units of meter-years. For example, the area under the usage curve in Figure A.9.1 in the lining section (0-900 m) is 538 m-yrs. So the net return is:

$$R_n = (315)(0.016\ \$/m^3)(.9)(1.0\ \text{lps}/100\ \text{m})(538\ \text{m-yrs}) -$$

$$(\$1.14/\text{m})(900\ \text{m}) = \$2440/\text{yr} - \$1026/\text{yr} = \$1414/\text{yr}.$$

Likewise, the net benefits for the earthen improved section (900 m - 1600 m) is $254/yr, and that for the cleaned and repaired section (1600 m - 1800 m), $50/yr, giving a total net return of $1718/yr. These calculations of course assume that all the saved water will be conveyed by the improved downstream channels to the field. The true benefits will be somewhat less because of increased downstream losses and the need to extend the improvement further downstream.

These calculations all assume a constant loss rate, Q_L, in the channels. As has been stated, the loss rates usually vary with flow rates and sometimes vary in different portions of a conveyance system. The problems with using variable loss rates is that Eq. A.9.6 cannot be solved directly unless an exact relationship between length, L, and usage time, t, is assumed. Reuss (1980) assumed a linear relationship between time and length:

$$t = 1 - \frac{L}{L_T}, \qquad (A.9.9)$$

where L_T is the total channel length, which allows the use of variable loss rates.

Another solution option is to solve Eq. A.9.6 by trial and error. Assume a length, solve the equation for time, pick the proper length for the calculated time from a usage time vs. length graph, and recalculate time. This process can continue until the proper time value is found.

Possible variable loss rate relationships include:

$$Q_L = \frac{dQ}{dL} = K_1 Q = K_1 Q_o e^{-K_1 L}, \text{ or} \qquad \text{(A.9.10)}$$

$$Q_L = \frac{dQ}{dL} = K_2 Q^P = K_2 \left[Q_o^{(1-P)} - K_2 (1-P) L \right]^{P/(1-P)} \qquad \text{(A.9.11)}$$

where:

Q_o = the inflow rate at the head of the channel (lps),

K_1 = the percent loss rate in the initial channel reach (%/100 m), and

$$K_2 = \frac{K_1}{Q_o^P}$$

Equation A.9.10 assumes that loss rates are proportional to flow rates, while Eq. A.9.11, which is more general, assumes that loss rates vary with flow rates at some relative rate, P. If loss rates change 80 percent as fast as flow rates, P = 0.8. Both of these loss rate relationships will yield lower usage time solutions and higher net returns than the constant loss rate assumption. Additional loss rate relationships are possible if the loss rates also vary with distance of the channel section from the inflow point.

Appendix 10

TOPICS TAUGHT AND ACTIVITIES OF THE IRRIGATION CHANNEL IMPROVEMENT ENGINEER TRAINING COURSE OF THE PUNJAB ON-FARM WATER MANAGEMENT PROGRAM

1. Objectives of channel improvement training and channel renovation slides.

2. Basic hydraulic units, terms, definitions and equations,

   a. water supply,

   b. water losses,

   c. types of flows,

   d. Mannings "n" factor, and

   e. ditch cross sections.

3. Tour of improved and under improvement watercourses in project areas. Discussion will be made at the sites among farmers on:

   a. conducting an effective meeting with farmers,

   b. establishing water user associations,

   c. role of farmer executive committees,

   d. social impact of water user organizations, and

   e. interviews with farmers on improved conveyance systems regarding benefits and economic aspects of improvements.

4. Basic surveying for watercourse design (i.e., note keeping, note reduction, plotting survey, data, equipment).

5. Engineering surveys for watercourse construction.

6. Watercourse design procedure:

   a. introduction to watercourse design,

   b. field information required for design

   c. preliminary design procedure,

   d. iterative design procedure, and

   e. utilization of existing culvert structures.

7. Watercourse branch design problem.

8. Engineering plan, drawing specifications and cost estimates.

9. Economics of irrigation channel renovation.

10. Essential data required for economic studies for the reconstruction of channels, its importance, and cost: benefit ratio.

11. Field problems encountered during watercourse improvement.

12. Irrigation water requirements of the channel system command area, irrigation scheduling, and planning irrigation systems.

13. Measurements of watercourse losses: importance, benefits, and use of flumes.

14. Watercourse structures:

    a. outlet installation demonstration at farm,

    b. water measurements, and

    c. use of syphon tubes.

15. Operation and maintenance procedure for the improved watercourses.

16. Watercourse record-keeping procedures.

17. Review of designing techniques and answers to questions on design problems on branches.

18. Field survey on an old channel system selected for improvement purposes:

    a. water measurements at head, middle and tail before improvement,

    b. formation of farmer committee,

    c. bench mark survey on main and branches,

    d. profile survey of main and branches of the selected channel system,

    e. topographic survey of the command area,

    f. computations of recorded data,

g. back reference to field in case of errors, mistakes or omissions and then correction of data,

h. preparation of topographic map and profile maps,

i. designing of main watercourse on the design sheet,

j. designing of branches and preparing cost estimates, and

k. completing construction of at least 330 meters of pad; alignment and staking for cross section of improved channel.

19. Final examination of channel improvement.

Appendix 11

PRINCIPLES AND SKILLS TAUGHT IN THE AGRICULTURAL OFFICER TRAINING COURSE AT UNIVERSITY OF AGRICULTURE, FAISALABAD, PAKISTAN

I. IRRIGATION AND DRAINAGE SKILLS

1. Survey techniques: taping and pacing, use of engineer's level, differential leveling, drawing profile of watercourse and surveying for watercourse design.

2. Elements of watercourse design: assumptions, field data required, general procedures, starting point, backward interaction, treatment of culverts and model problems with solutions.

3. Cleaning and maintenance of watercourse: determining needs, advantages, techniques to accomplish and advantages to farmers.

4. Water measurement with Cutthroat flumes: introduction, installation, reading, observation and maintenance.

5. Water measurement by other means: principles for head measurement, float method, weirs and flumes.

6. Soil moisture measurement: sun drying method and touch and feel method, equations, computations, examples and explanations for farmers.

7. Delivery losses and efficiencies: concepts, equations, computations, examples and explanations for farmers.

8. Application losses and efficiencies: concepts, equations, computations, examples and explanations for farmers.

9. Identification of land leveling needs and potential costs: field measurements/observations, justifications, estimating costs and predicted benefits.

10. Watercourse cleaning and maintenance: needs of cleaning and maintenance, increased water supply resulting from, benefits to farmers, and organizing and conducting a cleaning and maintenance operation.

11. Improved methods of irrigation: border irrigation, factors affecting and recommended size of borders; furrow irrigation, length, spacing and depth of furrow and construction equipment.

12. Determining irrigation requirements: elements, equations used, use of charts, calculations and amount of water to be applied.

13. Determining irrigation capability: available supplies, how much to supply, adjustments with available time and model problems.

14. Irrigation and crop scheduling on seasonal basis: basi information, computations and timetable of irrigation.

15. Determining leaching requirements: basic information, leaching requirements, expected benefits, alternate measures, removal of excess water, effects of remedial measures and predicted benefits.

16. Maintenance of level land: methods of maintaining level fields and use of proper implements to maintain level.

Practicals

Water measurement by: a. float method, b. weirs, c. Cutthroat flume

Determination of conveyance losses

Use of engineer's level

Watercourse profile survey

Topographic survey and preparation of topographic maps

Field training in layout and construction of improved watercourse

Evaluation of irrigation practices to determine land leveling needs and time farm requires to irrigate fields

II. SOCIOLOGICAL SKILLS

1. Structural attributes of village organization, social, economic and numerical significance of different caste, biradari and other solidary groups in the village.

2. The characteristics of caste endogamy and patrilineal and patrilocal family organization, extent and bases of factionalism, family dispute, social and economic dominance of one social group over the other, factional tendencies as impediments in the way of developmental activities, ways to resolve these conflicts.

3. Characteristics of village leadership, caste dominance, age, education, honesty, land ownership, etc.

4. Legal authority vs. personal influence in mobilizing collective efforts.

5. The process of collective decision making with special reference to cleaning and maintenance of common watercourses.

6. An assessment of the felt need for more water. The significance of getting more water through improving the watercourse.

7. Any alternative solutions available and their evaluation. The procedure for getting people to agree to the improvement of the watercourse. Preparation of the plan of work and its implementation.

8. Assessment of the satisfaction with the total operation.

9. Interviewing vs. participant observation as techniques of gathering sociological data.

10. Interviewing helps in reporting on values, beliefs, motivations and future plans. Participant observation highlights the problems of actual implementation of a plan.

Practicals

Preparation of an intensive interviewing schedule and involvement as participant observer for studying village attributes and the patterns of collective decision making.

Studying the relevant leadership characteristics so as to influence decision to clean watercourses.

III. AGRONOMIC SKILLS

1. Latest recommendations for land preparation, seeding rate, seeding method, fertilizer recommendations, pest control recommendations and irrigation requirements for all major crops. Principles of proper crop rotations as well as those commonly followed at present in canal and well irrigated areas.

2. Water requirements of crops, consumptive use, irrigation methods and choice of best methods.

3. Efficient use of water in relation to fertilization, influences of different levels of fertility on water requirements.

Practicals

Field trips to learn the position of the farmers in the area in regard to the agronomic recommendations, i.e., variety, cultural methods, inputs, plant protection measures, etc. What improvements can be suggested to help increase production (suggestions and improvements to the farmer). Field trips to cotton, rice and maize research centers and production areas.

IV. SOIL SCIENCE SKILLS

1. Terminology frequently applied in soil science, soil analysis and agencies analyzing soil samples, criteria for soil evaluation. Soil sampling for fertility evaluation, salinity/sodicity, and moisture determination. Management of saline/alkali soils.

2. Plant food elements, importance, source and their deficiency symptoms, recommended doses of fertilizers. Fertilizer application and efficient use of irrigation water. Determination of texture of soil in the field by feel method.

3. Quality of irrigation water, criteria for quality of irrigation water, water sampling and agencies analyzing water samples.

Practicals

Determination of soil texture by feel method, diagnosis of deficiency symptoms of plant nutrients. Field trips to observe the experiments and achievement of various agencies working on various aspects of soils.

V. EXTENSION SKILLS

Identification of actual water management problems, creation of awareness and interest in water management through mass media, mobilization of local resources for the solution of water management problems, principles of arranging demonstrations and tours, evaluation and reporting.

VI. ECONOMIC SKILLS

1. Definition, scope and goals of farm planning.

2. Farm Business Survey: Type of data required for planning, sources of data, selection of area and period, interview schedules, interviewing farmers.

3. Farm Business Analysis: Computation of farm cost, working out unit cost of production of various farm enterprises, causes of cost variation on similar farms.

4. Guiding Principles: Law of comparative advantage, enterprise and resource combination.

5. Developing Farm Plans: Analysis of existing agricultural conditions at farm level, locating merits and demerits, application of partial budgeting technique, planning land, labor and water use, developing optimum cropping patterns, economic comparison of old and new farm plans, need and procedure to revise plans over time.

Practicals

Developing a contact with the farmer. Explaining farm planning objectives and motivating farmers to participate.

Preliminary survey regarding agricultural conditions at the farm level.

Collection of basic data for application of a partial budgeting technique. Pretesting of the farm survey schedule.

VI. FARM POWER AND MACHINERY SKILLS

The use of agriculture machinery on small farms under limited capital conditions. Use of correct piece of machinery to accomplish a specific land manipulation or farming operation.

Practicals

Trainees to become skilled and confident in maintenance and operation of power-driven machinery for agriculture with particular emphasis on land leveling and planting equipment.