IRRIGATION WATER DISTRIBUTION SYSTEM FOR TUBEWELLS AND LOW-LIFT PUMPS IN BANGLADESH

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CARE-BANGLADESH

The CARE Deep Tubewell Irrigation and Credit Programme (DTICP) assist Bangladeshi farmers to expand the capacity and coverage of their deep tubewells and thereby increase their incomes. CARE's project has doubled the command area and the average yield per acre of the target tubewells. However, a major constraint of this and many similar irrigation projects operated by the Bangladesh Government and Private Voluntary Organizations is the general lack of technical information regarding irrigation techniques, practices and facilities. CARE felt that an irrigation manual would be of great use to the CARE project as well as to other small irrigation projects operated by the Bangladesh Government and Private Voluntary Organizations.

This manual will provide the needed reference material as well as guidance on how to plan and design efficient small irrigation systems.

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TABLE OF CONTENTS

LIST OF ILLUSTRATIONS

LIST OF TABLES

FOREWORD

A primary development goal in Bangladesh is self-sufficiency in food production. A major factor in the realization of this goal is increased production through irrigation. The large cost of irrigation facilities such as pumps and wells, as well as the rising cost of energy, make it necessary to utilize these facilities with more efficiency and increased capacity. This can be accomplished to a large extent by improving farm water management.

This book attempts to explain and illustrate some of the important concepts of proper practice in irrigation planning, design and operation of small systems. Reference material is provided which is needed in system design and operation. The text attempts to show the reader how to synthesize this reference material into efficient irrigation systems. Several options are presented in design of the systems as possibly being appropriate in Bangladesh. These options are by no means exclusive or the best possible but are presented to give the user a beginning or foundation in planning and design.

The publication is written for engineers and technicians working in planning, design and operation of small systems. It is assumed the user has prior training in engineering hydraulics. While recognizing that organization and management of water user groups is the primary key to success this component is beyond the scope of this book. Hopefully, this manual will be used by technicians who are part of a larger inter-disciplinary team working on all components of irrigation water management.

USEFUL EQUIVALENTS

1 Acre-inch/hour = 1 Cusec
1 Cusec = 2 Acre-feet/day
1 Cubic foot = 7.48 U.S. Gallons
1 Cubic foot of water = 62.4 pounds
1 Imperial Gallon of water = 10.0 pounds
1 Acre = 43,560 square feet.
1 Imperial Gallon = 1.198 U.S. Gallons

INTRODUCTION

There are about 10,000 deep tubewells (DTW's), 15,000 shallow tubewells (STW's) and 40,000 low lift pumps (LLP's) operating in Bangladesh, each with a capacity between 0.5 and 3.0 cubic feet per second (cusec). Taken in total, this is a huge irrigation potential in such a small area by any standard. Numerous studies are available relating to the actual average command area of these units. Based on crop water requirements, acceptable distribution efficiency and pump capacities, the actual command areas are about 1/3 to 1/4 the potential. A huge increase in food production could be realized with better utilization of existing facilities. New facilities should be provided in such a way that better utilization is assured.

Typically, pumps and wells are supplied at a subsidized rate by the Bangladesh Agricultural Development Corporation (BADC), or through a loan agreement between Bangladesh Krishi Bank (BKB), a franchised dealer, and the recipients. Except for shallow tubewells, pumps and wells are virtually always received by a group of farmers, since landholdings are so small and fragmented as to preclude personal ownership. When farmers apply for irrigation facilities such as pumps and wells, little or no technical assistance is available for planning and design of distribution systems. Each thana supposedly has an irrigation overseer under the Ministry of Local Government, Rural Development & Cooperatives (MLGRD&C) Thana Irrigation Program (TIP) who is charged with this responsibility. In fact, this post is often vacant, or the level of technical training and experience is inadequate and the necessary materials and equipment such as survey and mapping capability, reference materials, calculators and transport are lacking. Just as important, or more so, is the absence of administrative direction and on-the-job training.

The lack of technical assistance is one of the main reasons why projects fail to reach their potential. Usually facilities are located in less-than-ideal places from the standpoint of topography, soils and land use. The potential command area is seldom defined in quantity of land or location. All the problems associated with distribution, water scheduling, measurement and control are left to the farmers. A network of crude ditches evolve and the command area boundary is defined by whether adjacent landowners outside can negotiate a small trickle of water from neighbors inside. Farmers in the potential command area, but not adjacent to the actual command area boundary, have little hope of receiving water because of all the dry plots between them and the source, which would receive water first. Most farmers within the potential command area never realize that they could receive water or feel any identification with the water user group. Without an official potential command area boundary and network of channels, water control and outlet structures, farmers in the far reaches of the boundary have no incentive or means to try to improve and expand the system.

Existing systems can be roughly classified into three categories. The worst systems are operated by individual landowners, with no group cooperation. If a member wants water, he buys and brings a small quantity of diesel fuel, cranks the pump and runs some water to his plot. Unless his plot is adjacent to the pump, a significant portion of the water is lost in wetting and filling the channel. It is not hard to imagine the problems in equipment and distribution system maintenance and repair when dealing with such an individualistic group.

The next rough classification of system has some cooperative arrangement in fuel purchase and transport, operation and maintenance, and distribution system excavation and maintenance. In these systems the pump will discharge into a network of channels that are divided and subdiv ded until the stream sizes are a fraction of the pump discharge ; each farmer simultaneous trying to receive water. Some conveyance will be overland sheet flow from flooded plot to plc This is an extremely inefficient method of distribution due to the small stream sizes used, as we as the fact that the total channel network is used simultaneously, greatly increasing channel losse Again, the command area boundary is defined or evolved by whether those just outside ca negotiate a trickle from those just inside. Those inside are just getting enough from the subdivide flow and are reluctant to provide the necessary channel, increased risk and increased wear c the pump.

The best systems incorporate some kind of irrigation blocks and rotation system. These ar rarely formed with outside assistance or motivation, but by a member or group manager who ha some entrepreneurial tendencies and sees that individuals as well as the group can benefit fror cooperation. Typically, he will organize the group and provide the leadership to set up and operat scheduling in irrigation blocks, group excavation and maintenance of channels, etc. Many time: this man will collect a fee from all the members before the season in cash or kind, and the guarantee water. He shoulders the responsibilities of fuel procurement and transportation, main tenance and operation, group organization and dispute settling in matters such as providing channe access and water sharing. He may pocket a substantial amount of profit at the end of the season therefore he has the incentive to expand the systems as far as possible, bringing in as man paying members as he can. He knows that he must sell water at a price that leaves a profi margin for the farmers and he must deliver water in a timely fashion, or he will be unable to collect the next year and lose face with the group. Most of these systems still only irrigate abou 1/2 of their potential.

Another problem related to command area expansion is a pervasive belief among farmers and technicians alike, that, like draft animals, pumps and motors must be shut down periodically to take rest and cool off. The better systems operate only 12-14 hours per day in peak demand, bu the average is more like six hours.

In view of these problems, it is obvious that a systems approach must be taken in command area development. Instead of supplying wells or pumps and then considering improvement of a section of channel, as is the present practice, the total system should be planned, organized, designed, constructed and operated as a package unit, from water source to the last plot in the potential command area.

BASIC PRINCIPLES

It is beyond the scope of this manual to go into the complexities of soil-plant-water relations and irrigation hydraulics. It is assumed that the reader has a background in these subjects or can review partinent works listed in the bibliography. Only a brief and simplified description of the basic principles will be given, along with some thoughts on how these apply to the unique situations in Bangladesh. Flooded rice irrigation is basically different from crops such as wheat and vegetables in unsaturated soil and therefore will be treated separately.

FLOODED RICE

Most high yielding varieties of rice produce best when growing in two to five inches of water during most of the growing season [5]. The main advantage of continuous flooding is apparently weed control. Rice grown at or near field capacity moisture (see section on upland crops) with good weeding will produce equivalent yields with a large saving in water. In fact, most of the irrigated rice crops in Bangladesh are not continuously flooded, but are kept wet by frequent applications. The following applies to continuously flooded systems only.

Pump and tubewell systems used primarily for irrigation of flooded rice during the winter and spring, when little or no rainfall can be depended on, should be designed for this extreme condition. Before land preparation for transplanting, the paddies must be saturated and ponded for tillage. The main purpose of saturated soil tillage is to break up soil structure and fill soil pores and passages with fine particles to prohibit downward percolation of the ponded irrigation water. This takes about five inches of water, depending on the soil and antecedent moisture conditions, and may well be the period of maximum water demand.

After planting, there are a number of ways pumped irrigation water can be lost :

1. It can be lost from the channel system by evaporation, seepage or leakage.
2. It can be lost to the atmosphere from the irrigated paddies by evapotranspiration.
3. It can infiltrate into the soil and groundwater reservoir below the paddies.
4. It can drain out of the irrigated area by gravity surface flow and be lost as tailwater outflow.

Evaporation from the water surface in field channels is comparable to evapotranspiration from flooded paddy and can be treated as additional area. Since the paddies adjacent to field channels are continuously flooded and the soil is saturated, downward percolation from the channels is also minimal and only serves to keep the soil saturated. Again, this can be treated as additional paddy area.

Evapotranspiration is the water lost to the atmosphere from direct evaporation plus transpiration from plant leaves. Evapotranspiration from paddies is a function of many elements, including temperature, wind, solar radiation, humidity, crop type, stage of growth and cover. Many methods for estimating evapotranspiration have been used with varying success. Values of evapotranspiration for design purposes may be estimated from Tables 1 and 2. Table 1 gives estimated monthly value of **potential evapotranspiration** (ETP) and **dependable rainfall** (DR) for five regions in Bangladesh. Potential evapotranspiration is the amount of water evaporated and transpired by an actively growing thick cover of grass which is not short of water. Dependable rainfall is defined

Table—1

POTENTIAL EVAPOTRANSPIRATION AND DEPENDABLE RAINFALL IN BANGLADESH

	REGION*	1	2	3	4	5
JAN.	ETP**	3.5 in	3.4 in	3.5 in	2.1 in	2.3 in
	DR***	0.0 in	0.0 in	0.0 in	0.0 in	0.0 in
FEB.	ETP	5.0	4.5	4.9	3.4	4.0
	DR	0.1	0.1	0.1	0.1	0.1
MAR.	ETP	7.2	6.6	6.8	5.2	5.1
	DR	0.1	0.2	0.5	1.0	0.5
APR.	ETP	8.0	7.2	5.9	5.8	5.4
	DR	0.4	1.2	2.8	5.7	1.8
MAY	ETP	6.8	6.3	6.2	5.9	5.6
	DR	5.0	3.9	6.4	13.5	6.0
JUNE	ETP	5.7	5.4	5.7	5.2	5.1
	DR	7.1	7.3	8.9	15.1	15.0
JULY	ETP	6.2	5.2	5.7	5.2	5.1
	DR	9.1	9.7	10.4	9.8	16.0
AUG.	ETP	5.4	4.9	5.6	5.0	4.8
	DR	8.7	7.7	10.4	9.9	15.2
SEP.	ETP	4.8	4.3	5.0	4.3	4.4
	DR	7.2	5.7	6.9	8.0	8.8
OCT.	ETP	4.4	4.1	4.6	3.7	3.9
	DR	3.2	2.6	2.8	3.7	5.1
NOV.	ETP	3.7	3.5	3.7	2.6	3.3
	DR	0.0	0.0	0.0	0.0	0.0
DEC.	ETP	3.4	3.1	3.4	2.3	3.1
	DR	0.0	0.0	0.0	0.0	0.0

* Region 1 : Rangpur, Dinajpur, Bogra, Rajshahi, Pabna.
Region 2 : Kushtia, Jessore, Faridpur, Khulna.
Region 3 : Tangail, Dacca, Comilla, Mymensingh, Jamalpur.
Region 4 : Sylhet
Region 5 : Chittagong, Noakhali, Patuakhali, Barisal.

** Potential evapotranspiration (ETP) from green grass.

*** Dependable Rainfall. (DR). 75% probability of being equalled or exceeded.

here as the amount of rain for the given month which has a 75% probability of being equalled or exceeded. Table 2 gives coefficients (KC) for multiplying ETP to find the evapotranspiration of specific crops. The given KC values are for the maximum monthly water use during the peak stage of plant growth.

The values given for ETP were calculated by the radiation method as described in reference (6), listed in the Bibliography. Crop coefficients were also taken from this publication. Climatological data for use in the radiation method calculations were taken from reference (9). The data was compiled from weather stations at Bogra, Jessore, Narayanganj, Srimangal and Chittagong. If more reliable local data is available it snould be used. Even though the given values will vary with locality as well as with time, they are of sufficient accuracy for design and management of small systems.

For design of most flooded rice systems, deep percolation will be the most difficult to estimate but the most important parameter of water loss to be determined. Since it is a function of local soil strata and groundwater levels, it can vary greatly within a small area as well as with time. Therefore, approximations and assumptions will be made here.

Probably the best way to estimate percolation is to measure the rate of fall of water in a ponded paddy. This should be done between dusk and dawn, when evapotranspiration is minimum. Be sure to check the bunds around the paddy to assure that there is no inflow or outflow from adjacent areas and that the total surface is ponded. Use a relatively large area to minimize the effect of lateral seepage through bunds. Also, the paddy area should have been puddled, wet tilled and ponded for at least several days before measurements are made. To determine a composite value of a larger area, measurements should be made on several paddies with representative soil types and relief.

Table—2

CROP EVAPOTRANSPIRATION COEFFICIENTS
AND EFFECTIVE ROOT ZONE DEPTH

Crop	**Effective Root Depth**	**Coefficient (KC)•**
Flooded rice	—	1.25
Wheat & grains	3.0 ft.	1.1
Sugarcane	3.0	1.1
Bananas	4.0	1.0
Pulses	2.0	1.1
Vegetables	2.0	1.1
Vine crops. melons	2.5	1.0

• Crop evapotranspiration is calculated by multiplying KC by ETP given in TABLE 1. The KC values given are for the crop at maximum vegetative growth stage. Evapotranspiration before and after will be less.

If the estimated percolation is much higher than evapotranspiration, the area is not considered suitable for flooded rice irrigation using expensive pumped water. The alternatives would be to grow rice under unsaturated conditions, to grow another crop or to abandon the site for a better location which would warrant an investment in expensive equipment. Calculation of a maximum permissible value of percolation is a subject worthy of study but beyond the scope of this paper. The author feels that percolation rates above 0.3 inches per day are probably too high for economical use of water pumped from tubewells.

During percolation of ponded water, one of two situations can exist. Either the groundwater level is below the field surface and ponded water is held by low permeability soil layers, or the subsoil is completely saturated and the water table is at the ground surface. If ponded water is perched above unsaturated subsoil, then there must be a layer of very low permeability soil near the surface to eliminate large water loss from deep percolation. If the complete soil profile is saturated, then percolation will be limited by lateral movement and displacement of groundwater with a small hydraulic gradient. To determine which of these conditions exists, select a site in the middle of a paddy that has been ponded for at least several days and bore a six-foot deep hole with a soil auger through the saturated surface layer and any under-lying hardpan. Isolate the hole from the ponded paddy water with a small bund or wall around its periphery and fill the hole with water to the level of water in the paddy. If the water in the hole remains close to the same level as in the paddy, then a completely saturated soil profile is indicated. If the paddy water is perched, then the water in the auger hole will drop well below the surface and may even dry up.

Why is it important to know which condition exists? If the complete profile is saturated and the ponded surface water is supported by a high water table, then irrigation from a well could drop the water table and greatly increase the percolation rate. Conversely, irrigation from a surface source could raise the water table, thereby causing a decrease in percolation rate. These factors need to be considered in the planning and design of a system.

Unless there are salinity problems, such as in the coastal areas of Bangladesh, loss of water by tailwater drainage should be avoided Systems should be designed to avoid this loss. For our design purposes, it is assumed that it is non-existent, at least during the peak demand period.

The system can be looked on as a storage reservoir. The summation of inflow and outflow for a given period is represented by a change of storage or water level in the paddies. This rather large quantity of stored water serves to cushion the effect of days with extremely high evapotranspiration and is replenished during cool or cloudy days.

Crops on Unsaturated Soil

Irrigation of crops grown under unsaturated conditions is somewhat different from flooded rice. Storage of moisture for plant use in the soil root zone instead of surface flooding requires more sophistication in design and operation of distribution system.

If one submerges a dry towel in a bucket of water, all the pore spaces in the cloth will fill with water. This condition is called **saturation** If the towel is taken out of the water and hung on a line, water will run or drip from the towel by gravity for a period of time. Eventually, the water will cease dripping, even though the towel is very wet. In irrigation terminology, the remaining moisture content is called **field capacity**. The towel continues to dry at a much slower rate by evaporation of moisture from its surface. This continues until it is in equilibrium with the surrounding air and feels dry. However, there is still some water held by the towel called **hygroscopic moisture,** which can only be removed by heating it to a high temperature.

Table—3

AVAILABLE MOISTURE CAPACITY OF BANGLADESH SOILS*

Siol Parent Material Unit	Textural Class 1/	Drainage Class 2/	Available Moisture (inches water per foot soil) Range	Average for SiCL with 34% clay
Tista, old Himalayan Piedmont Jamuna flood plain	FSL	I-MW	1.4-2.0	
	L-SiL	I-MW	2.4-3.2	
	SiL	P	3.6-5.2	3.2
	SiCL - CL	P	2.4-3.7	
	SiC - C	P	0.7-2.8	
Ganges river flood plain	SiL	I-MW	2.0	
	SiL	P	3.5-3.6	2.6
	SiCL	P	3.0	
	SiC-C	P	1.3-2.6	
Old Brahmaputra, old Meghna estuarine flood plain	SiL	I-P	1.9-2.6	
	SiCL-CL	P	2.0-3.2	2.2
	SiC-C	P	0.6-3.2	
Madhupur Clay, north Mymensingh piedmont plain	SiCL-CL	I-W	1.3-1.8	
	SiC-C	I-W	1.1-1.4	
	L	P	1.8-2.8	1.6
	SiCL-CL	P	0.8-2.4	
	SiC-C	P	0.7-1.4	

1 FSL = fine sandy loam
L = loam
SiL = silt loam
CL = clay loam
SiCL = silty clay loam
SiC = silty clay
C = clay

2 P = poorly drained
I = imperfectly drained
MW = moderately well-drained
W = well-drained

* Data from H. Brammer, FAO

Soils hold moisture in a similar way to the towel discussed above. Most crops are retarded or killed when the root zone becomes saturated for more than a few hours. Therefore, the ideal soil is one which is deep, well-drained and fairly permeable. These are opposite characteristics of a good flooded rice soil. The **gravity moisture**, or difference between saturation and field capacity, is not available to plants, since it drains below the root zone rather quickly and is lost to deep percolation. This raises the water table, which may be detrimental to crop growth. Plants remove moisture from the soil root zone, which is transpired into the air through leaves. Some soil moisture is evaporated directly from the soil surface. Water is also lost to the atmosphere through evapotranspiration. Plants continue to extract water and the soil moisture content continues to decrease in the root zone until the plant begins to wilt or to be significantly stressed from lack of soil moisture. The remaining moisture content is called the **wilting point**. The quantity of water held by a particular soil between the field capacity content and wilting point content is called **available moisture**. Available moisture is a function of soil type. It is lowest for sand and heavy clay and highest for loam. Table 3 gives approximate values of available moisture for design purposes. Ideally, available moisture should be tested using soil samples, as described in reference (7)•. It is general practice to supply water before 50% of the available moisture has been depleted, otherwise the crop yield may be reduced.

Table—4

AVERAGE PERMEABILITY OF BANGLADESH SOILS•

Soils	**Topsoil (inches/day) Av.**	**Range**	**Subsoil (inches/day) Av.**	**Range**
A. Grey terrace soils used for transplanted rice (Barind, Madhupur tracts)	1	0-4	40	4-236
B. Red and brown terrace soils (mainly Madhupur tract)	20	4-30	47	24-60
C. Silt loam/silty clay loam floodplain soils used for transplanted rice	0.2	0-1	3	0.32
D. Silty clay/clay floodplain soils used for transplanted rice	1	0-2.5	12	0.72
E. Silty clay/clay floodplain soils used for broadcast rice	12	0-40	20	0-120
F Imperfectly drained flood plain ridge soils (silt loam/silty clay loam)	6	4-8	16	4-32
G. Moderately well drained flood plain ridge soils (silt loam/silty clay loam)	24	4-40	28	8-48
H. Old Himalayan piedmont soils ("sandy" Thakurgaon soils, actually mainly loam/ sandy clay loam)	80	40-120	100	12-160

• Data from Hugh Brammer, FAO

• Because it is frequently desirable to estimate the soil texture in the field, the feel of the soil can be tested between the thumb and the finger or in the palm of the hand. If the soil is wet, sand particles feel gritty, silt has a rather smooth and floury feeling and clay is plastic or sticky.

If moist soil is left fallow into the dry season, water will evaporate and the surface will become dry. However, after the top few inches are dry, movement of moisture becomes very slow and lower depths will stay moist. Therefore, if land is kept fallow after the monsoon, water will be stored in the soil well into the dry season and the first irrigation need only provide enough water to wet the dry layer so that seeds will germinate.

The root depth of most annual crops extends 1 to 1.5 feet each month of active growth, unless penetration is restricted by compacted soil layers or a high water table. Areas that have been recently cultivated for rice may have a compacted layer below the soil surface that will restrict the root penetration of winter crops, thereby reducing the effective soil depth for storing irrigation water. These compacted layers can easily be identified and located by soil borings with a hand auger.

The design irrigation frequency and depth of water to be applied are functions of the crop evapotranspiration rate and the available moisture-holding capacity in the root zone. For example, assume the effective root zone depth is 3.0 feet, available moisture is 2.4 inches per foot of root zone depth, evapotranspiration is 0.15 inches per day and irrigation is supplied when 50% of available moisture is depleted. The design irrigation depth is $0.5 \times 3 \times 2.4 = 3.6$ inches, and the frequency is $3.6/0.15 = 24$ days. This is the maximum depth and minimum frequency that should be used under ideal conditions. It may be advantageous to apply less water at more frequent intervals because of organizational or management reasons. This is acceptable, but it should be kept in mind that distribution efficiency and uniformity generally decrease with depth of application. Also, it is generally better to under-irrigate than over-irrigate. Most field crops will produce about 90% of potential yield when supplied about 75% of optimum water requirements. The increased profit from expanded command area and/or savings in fuel from under-irrigation is usually greater than the small per-acre decrease in yield.

After an irrigation system is operational, irrigation should be scheduled from soil moisture measurements or from soil moisture budgets. Design values of evapotranspiration and maximum root zone depth for various crops in Bangladesh are given in Tables 1 and 2.

For the small systems under consideration **irrigation efficiency** can be defined as the actual crop evapotranspiration divided by water delivered by the pump. It is impossible to have 100% efficiency because of conveyance losses, uneven distribution over fields, percolation below root zones and tailwater runoff. However, efficiencies ranging from 50% to 70% are possible. When planning a system these water losses must be accounted for when determining the quantity of water needed, or area of land that can be irrigated with a given flow rate.

IRRIGATION SITE SELECTION

There are many factors that should be taken into account when evaluating or selecting a site for an irrigation project. These can be broadly categorized under water availability, land suitability, availability of agricultural inputs and extension services, and the needs and potential of involved farmers.

The water supply must be suitable in quantity and quality. Surface sources are usually cheaper to exploit than groundwater if the land to be irrigated is near the supply. Care must be taken to assure that surface sources will be sufficient during the driest months of the year and that over planning for water use will not take place. Irrigation can raise the water table, restricting root growth and soil aeration. Furthermore, significant withdrawals from a surface source must be weighed against adverse effects to navigation and fisheries. Groundwater sources must be evaluated from the standpoint of specific yield and depth of the aquifer as well as recharge potential. Extensive groundwater development may lower water table levels, causing nearby ponds or shallow wells to dry up. In coastal areas groundwater development may cause salt water intrusion. Water high in salt or mineral content can be detrimental to crops and irrigation equipment.

The land to be irrigated should have high potential for production. For flooded rice, it should be level and impermeable. For crops grown under unsaturated conditions it should be well-drained and have high moisture holding characteristics. Land should have enough slope so that water can be conveyed by gravity, but not so steep that many drop structures are needed in the conveyance system. Slopes of 0.001 to 0.01 are ideal. The slope should be gentle and uniform in one direction, or in two directions from a ridge. Ideally there should be no high spots or depressions and the area should be roughly square. Irregular boundaries greatly increase problems of distribution, system layout and irrigation scheduling.

The water user group needs access to agricultural inputs such as fertilizer, seed, fuel, maintenance of equipment, credit and extension advice in order to reap maximum benefits from the irrigation system. The system performance is totally dependent on the organization of water users. The crucial factor is their cooperation in matters of water management, scheduling, labor, right-of-way, system maintenance and costs. A team spirit in optimization of the total system is imperative for successful operation, During all phases of planning, design and construction, the group of farmers served by the system should be involved to the maximum extent possible.

SYSTEM PLANNING AND DESIGN

In order to realize the full irrigation potential of a tubewell or low lift pump, the irrigation system should be planned and designed as a complete package unit, from the source to the potential command area boundaries. If this is not done, the pump will likely be placed in a less-than-ideal location and be of the wrong size. Landholders adjacent to the pump will gain control and be reluctant to provide right-of-way, investments, and the extra operating hours necessary to expand the command area. Proper planning and design also minimize the equity problems associated with tubewells and pump irrigation systems.

Land consolidation and leveling would be advantageous to command area development from a technical standpoint if it could be implemented. However, there has been little indication that this is feasible in the present social structure and therefore is not considered here.

It is important that irrigation system planning, design, organization and operation be carried out by an interdisciplinary team, working together within the same agency. The team should include necessary expertise in engineering, agronomy, organization, management and sociology. Instead of offering pumps or wells, the agency should offer complete irrigation systems and use the wells and pumps as incentive for the farmers to organize, give right-of-way, labor and funds for system implementation. The major steps in system planning, design, organization, construction and operation are listed below.

1. Initial discussions and planning between water use group and irrigation team.
2. Mapping of the area
3. Assessment of the water source
4. Organization of water user group
5. Definition of potential command area
6. Definition of irrigation blocks
7. Location and design of outlet structures for each block
8. Design of scheduling and rotation system
9. Location and design of conveyance system
10. Location and design of water control and measurement structures
11. Design of pump or tubewell
12. Construction of the system
13. Operation and management

These items will be discussed separately below:

Initial Discussion and Planning: When a farmer group applies for irrigation facilities, an inter-disciplinary team from the local donor agency should meet with the group, review the group's proposal and needs and explain the group inputs such as organization, finance, etc., that are necessary to receive the requested support. The group proposal can be critiqued, with explanations of the services and options offered by the donor and those which might best suit the needs of the group.

Mapping: Assuming a positive outcome of initial discussions and planning, the proposed area should be suitably mapped, with the aid of **mouza maps**, including topography, plot boundaries, land use and suitability, roads, drainage, etc. The scale should be 1"=100 to 200 feet. For level basin farmland, spot elevations on each plot (to the nearest 0.1 ft. or 5.0 cm) are more convenient than contours, since contours will coincide with plot boundaries. Horizontal precision should be 1% or better. The most expedient way to produce this type of mapping is by use of a plane table and alidade. A more tedious and time-consuming method is using a surveyor's level, with stadia and horizontal circle.

The mapped area should extend somewhat beyond the potential command area, since it will be used to help locate the command area boundary, using the topography, land suitability, etc. Accurate maps will be an invaluable tool in all phases of system planning, design and operation.

Assessment of Water Source : Whether developing the command area of an existing tube-well or pump, or planning a new system, an assessment of the water supply should be done. For an existing pump or well, pump testing should be done. The test should include discharge and total lift, with the pump running at the design speed. Discharge measurement is discussed in a following section.

For new systems, the water source should be assessed for minimum flow and water level (surface sources) or aquifer depth, specific yield and static water level(groundwater). Data for surface and groundwater for many areas in Bangladesh are available from the Water Supply Papers and other studies published by the Bangladesh Water Development Board and BADC. Reliable data can also be obtained by questioning local residents and observing wells and pumps operating in the area.

The water source data are used to match pumps and engines to the required lift and discharge. Also, the pump or well discharge should be matched to the potential command area, whichever is limiting.

Organization of Water User Group : No irrigation system will perform better than the organization using it. The group members should be organized and trained in early stages of project planning and included to the maximum extent practical in all phases of planning, design, construction and, of course, operation and maintenance. Details of group organization and management are beyond the scope of this manual, but it should be stressed as the most essential element to successful system operation. Some possible options in group organization will be presented in the Appendix.

Definition of Potential Command Area : Knowing the discharge of the existing or planned pump or well, the potential command area can be estimated, using crop water requirements and estimated efficiency. The amount of acreage served per unit of pumping capacity can be thought of as the tension designed into the system. There is considerable latitude and there are different opinions about the tension or safety factor that should be designed into the command area of a system. The advantage of a high tension system is that maximum production and group profit can be realized. The advantage of a system with less tension is that risks are reduced and success in early years is critical in order to instill and maintain confidence among the water users. A medium tension system might be based on peak monthly crop water requirement, 70% efficiency and 20-hour per day pump operation. This would represent a great improvement in most existing systems in Bangladesh.

After the potential command area is estimated, the boundaries can be located on the map, using topographic data, land use and suitability, physical boundaries such as roads, drainage and villages, and the needs and interests of the water user group. The area should be roughly square or round, because long or irregular shapes greatly increase required water course length, organization and scheduling problems and reduce efficiency.

If the group wants to include or exclude areas other than the area recommended by the technical team, their proposals should be considered and accommodated if technically sound. Otherwise, the technical problems should be explained to the group and if full agreement cannot be reached the project should be terminated. If the well or pump is not already installed, the map should be used to locate the most suitable place for this facility and possibly to adjust the command area to fit this location.

Location of Irrigation Block Boundaries : In order to incorporate efficient irrigation water distribution, the command area must be divided into blocks of a manageable size and irrigated according to a prescribed schedule, otherwise flow will be divided and subdivided into multiple channels simultaneously, greatly increasing channel losses as well as reducing stream size to a discharge too small for uniform field distribution. To illustrate the importance of stream size, imagine a 1/3 acre plot of wheat and a small stream size of 20 gallons per minute. The present method of application is flooding small basins or plots surrounded by a low bund. For a stream size of 20 gal/min on typical upland soils it may take several hours just for the water to wet and move across the plot from the intake side to the extremities. By this time an excess has infiltrated on the intake side but more inflow is required to meet the needs of the extremities, resulting in uneven distribution and wasted water. In order to achieve uniform distribution the stream size should be large enough so that the application time into a given field should be about one-third the time it takes for the water to infiltrate. For example, if water is to be applied to a 0.5 acre basin, and the infiltration rate is 1.0 inch per hour, then the stream size should be a minimum of $1 \times 3 \times 0.5 = 1.5$ cusecs (1 0 cusec equals one acre-inch per hour).

It is equally important to be sure basins are smooth and level to assure uniform distribution. If three inches of water are applied to a basin which has two inches of slope from the high to low side, or high and low spots two inches different in elevation, the low spots will receive about twice the water as the high spots, resulting in wasted water and energy, as well as reduction in yield.

There is no set method to determine optimum block size. The author's opinion is that about five acres should be the maximum block size for the average conditions in Bangladesh, given the small size of landholdings. Blocks should be roughly square, and equal in size to the extent practical. The main key to block location is experience and judgment.

The whole purpose of block rotation and scheduling is to retain full stream size and minimum wetted channel length. Since there are many plots within a block, it is equally important to irrigate only one or two plots at a time for the same reason. The best practice is to begin irrigation of blocks and plots at the extreme end of the channels and proceed back toward the source. The command area boundary, block boundaries and irrigation schedule should be made official and posted at a conspicuous place for the information of all members. The block members must organize and coordinate for land preparation and planting as well as irrigation. There should also be some agreement and cooperation for growing crops which are mutually compatible in planting and irrigation.

Block Outlet Structures : These structures serve the dual purpose of controlling the inflow to respective blocks and serving as monuments to define the system, the only designated places where water is taken from the channels and marking the location and grade of the channels. Without well-constructed official outlet structures there is little chance of preventing farmers from taking water from the channels at undesignated times and places. Where possible, outlet structures and connecting channels should be located to serve blocks on each side in order to minimize the number of structures and channel length.

For the case of open channel distribution systems the outlet structures consists of a weir crest or outlet to divert water from the channel to the blocks, gates on the outlets and channel to control flow direction, and an apron and stilling basin at the outlet. The structure should be simple to build and operate, reasonably priced and durable. A masonry rectangular box type outlet should be suitable for the systems in question. Short rectangular sections with smooth plaster inside are provided on the outlets to accommodate a simple hand-placed stop check for controlling water. The stop checks fit inside the rectangular section diagonally and are held by gravity and hydrostatic force, eliminating the need for slide grooves and seals. Details of a typical structure are shown in the Appendix.

Hydraulic design of the outlet structures includes required water surface elevation, outlet width and crest elevation, and apron elevation and length. Water surface elevation is determined by elevation of the highest plot in the respective block and downstream channel head loss. Design water surface elevation should be about 0.5 ft. above the elevation of the highest plot to be irrigated from the respective outlet, plus the channel head loss.

The following weir formula can be used to design the crest elevation and width of the outlets.

$$Q = 3.0\, LH^{3/2} \qquad (1)^{\bullet}$$

where Q = discharge in outlet (cusecs)

L = length of weir or opening (ft)

H = height of weir crest above upstream design water surface (ft)

If Equation 1 is used, the maximum design tailwater elevation above the outlet crest should not exceed 2/3 of the upsteam head on the crest. Otherwise, the weir will submerge and increase the upstream water surface elevation. In this case the outlet can be designed with the full upstream channel cross sectional area and equal bottom elevation.

It is necessary to have an apron and stilling basin below the outlet to prevent scour under the structure and subsequent failure. The apron elevation and length are designed on the basis of assuring a hydraulic jump and subcritical flow on the apron for minimum design tailwater elevation. Related theory can be found in reference (8). A design chart for apron design is given in Fig. 1.

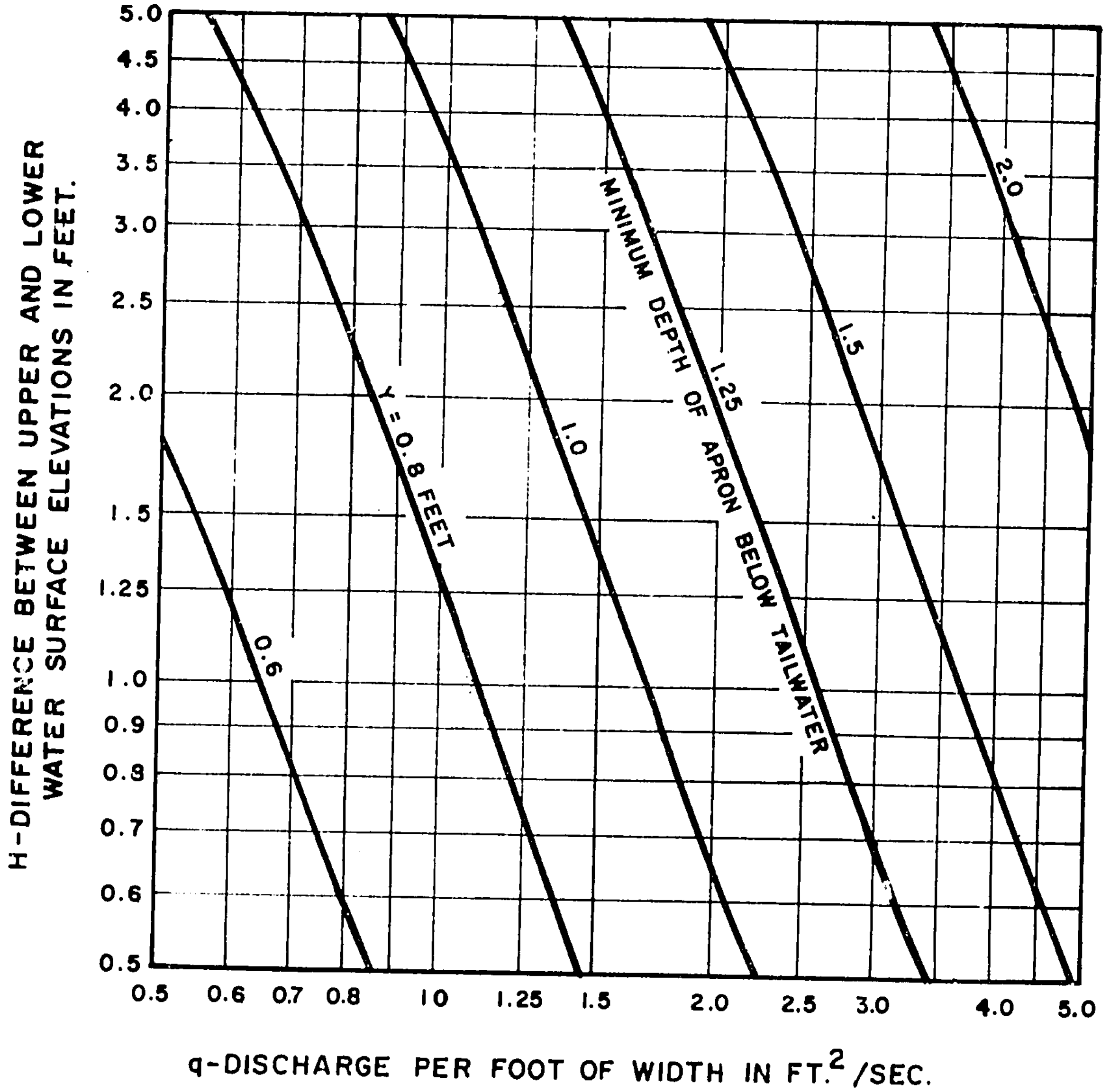
Conveyance Systems : Conveyance systems can consist of earth or lined open channels or pressure pipe or any combination. The purpose is to move water from the source to all the fields or basins in the command area. Three types will be considered : earth channels, lined channels and low pressure buried pipe. For the systems considered in this manual, channels from the pump to the block outlets will be designated primary, and from the outlets to the individual fields, secondary.

Most of the systems in Bangladesh consists of either kutcha earth channels, planned and constructed by farmers without technical assistance, or a short section of lined channel, constructed without considering the command area and system as a whole or the hydraulics (slope and cross section) unique to the site. Little work has been done on well-designed, compacted and graded earth channel distribution systems in Bangladesh.

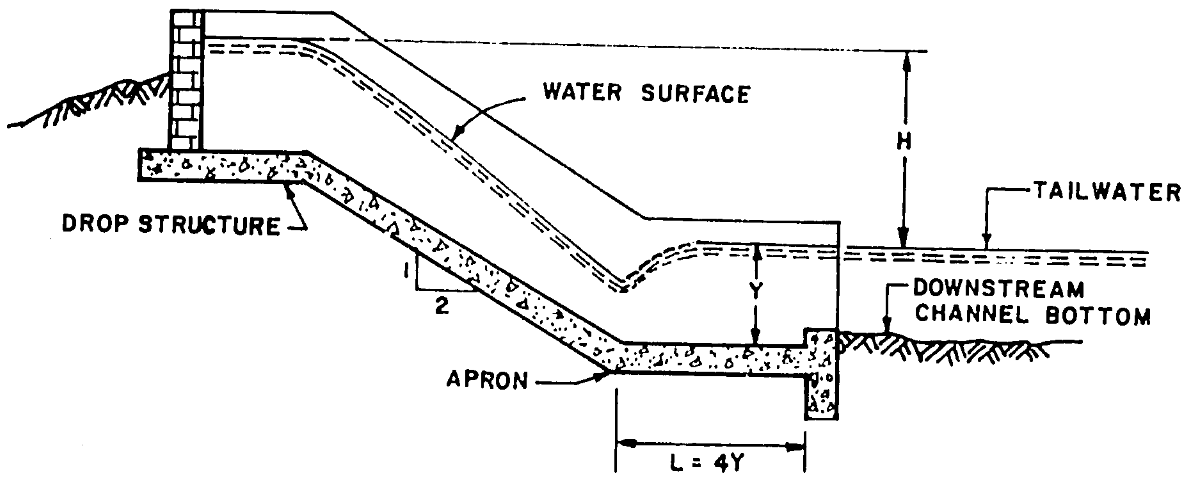
Water loss from channels reduces the potential command area, can damage crops adjacent to the channel due to waterlogging and increases pumping cost. Local research reports as much as 50% loss in 1,000 feet of 2-cusec earth channels. Since command area increases with the square of the radius (minimum channel length to boundary), it can be seen that most of the command area is at the extremities.

Most of the water loss in small earth channels in Bangladesh is not vertical percolation as in large canals, but lateral leaking through rat and crab holes and cracks. In areas with swelling clays this can be particularly bad, due to cracks that form when the banks are dry. However, most of this lateral leakage loss can be prevented with proper design, compaction and maintenance of earth channels. These are labor intensive, cheap and require no foreign exchange. Much more attention and emphasis needs to be given to good earth channels.

• All equations are represented serially in this fashion.



$$H = 2q^2/(gY^2((8q^2/(gY^3)) + 1)^{0.5} - 1)^2) + Y/2((8g^2/(gY^3)) + 1)^{0.5} - 1) - Y$$



MINIMUM DEPTH OF DROP STRUCTURE APRON BELOW TAILWATER

FIGURE - 1

Lined channels can greatly reduce water loss. However, before a channel system is lined, it should be economically justified by measuring actual losses in the field and weighing cost against savings in pumping, expanded production and reduced maintenance. The lined channel economics should also be compared with well-constructed earth channels, if they are not already in place.

Channels loss in a given reach can be estimated by taking simultaneous measurements of inflow and outflow during steady flow conditions, or by measuring rate of fall of the water surface in a channel reach that has been filled and blocked off at both ends. In both cases it is important that measurements be taken with the channel flowing or filled to its working capacity, since leaking increases greatly with water level. If flow measurements are made at the head and tail of the reach in question, accurate devices with small head losses, such as a Parshall or trapezoidal flume should be used. Since the loss is the difference between inflow and outflow, it is important that accurate measurements are made, and that both flumes are calibrated in a laboratory.

It is usually a waste of money to line a short section of channel without considering the total potential command area as a system, including blocks and scheduling. The typical short sections of lined channel seen in Bangladesh serve little purpose except transporting the distribution problems from the channel inlet to its outlet.

Most lining in Bangladesh is brick and masonry, rectangular trapezoidal. There is also some promise for trapezoidal cement-asbestos liners and handmade, fired clay, semi-circular liners.

Channel elevation should be determined by the maximum required downstream water surface elevation, keeping the channel as low as possible to minimize right-of-way and leaking. It can be seen that channel grades must be designed starting at the most remote outlets and working back toward the pump. This is one reason why it is necessary to design with the complete system in mind, rather than looking only at short reaches of channel.

Channel alignment requires experience and judgment. Generally, channels should be aligned to serve blocks on each side in order to minimize required length. To the maximum degree practical, sections should be straight. Bends should be curved to avoid excessive head loss and scouring.

Buried pipe has not been used extensively in Bangladesh but shows promise and has the following advantages :

1. No right-of-way is necessary. This is not only an economic benefit but a practical one, when perhaps 100 plots must be crossed and negotiated for a 150-acre command area.
2. It is not necessary to follow plot boundaries, reducing conveyance length, cost and head loss.
3. The pump does not need to be located in the highest part of the command area.
4. Water can be conveyed to high blocks that cannot be reached by open channels.
5. Water cannot be taken out at undesignated places.
6. With proper construction conveyance, efficiency is nearly 100% between pump and riser.
7. Conveyance system maintenance is minimal and life is maximal.
8. Water control and measurement are simple.
9. Drainage and transportation are not restricted (this is a continual problem with other systems, and is linked to increased maintenance).
10. Weed seeds are not spread throughout the command area.

Appropiate materials for buried pipe in Bangladesh are concrete, cement-asbestos and possibly vitrified clay. Concrete is cheapest and can be cast on site, but much skill and supervision are required to assure quality control of pipe and joints. Cement-asbestos is locally made together with fittings and rubber seals. It is more expensive than concrete, but can be easily placed with unskilled labor under technical supervision with minimum work and risk of leaks. Cement-asbestos also has less friction loss than concrete pipe with morter joints. Plastic pipe is currently too expensive to be used in Bangladesh.

EARTH CHANNEL DESIGN

The relation between discharge, slope, shape, size and surface roughness of a channel is given by Manning's equation.

$$Q = 1.49/N(A) \times (A/P)^{2/3} S^{1/2} \qquad (2)$$

where Q = flowrate (cusecs)
A = cross sectional area (sq.ft)
P = wetted perimeter of channel (ft)
S = slope of the water surface (dimensionless)
N = channel roughness coefficient (see Table 5)

Velocity of water in a channel is related to flowrate and cross sectional area by:

$$Q = AV \qquad (3)$$

where V = average velocity in feet per second

The design of earth or unlined channel is governed by the maximum non-erosive velocities and side slopes given in Table 6. If the land is steep, channel velocities will exceed the maximum permissible and cause erosion. In such cases drop structures must be placed at appropriate intervals, so that the channel will have a sufficiently mild slope and low velocity. Equations 2 and 3 are used to determine permissible slopes.

Earth channel cross sections are usually trapezoidal. With the high value of agricultural land in Bangladesh it is safe to say that the bottom width and side slopes should be as small as is practical to construct and maintain. Most of the small earth channels in Bangladesh are too wide and shallow.

Table-5

MANNING'S ROUGHNESS COEFFICIENTS (N) FOR IRRIGATION CHANNELS

Type of channel	Roughness Coefficient (n)
Cement-asbestos	0.013
Smooth concrete	0.013
Rough concrete	0.015
Bricks with mortar plaster	0.013
Smooth bricks without plaster	0.015
Rough bricks	0.018
Smooth clean earth	0,025
Normal earth	0.035
Rough earth with weeds	0.050
Cement-asbestos pipe	0.011
Mortar joint smooth concrete pipe	0.013
Rough concrete pipe	0.015
Vitrified clay pipe	0.014

Table-6

MAXIMUM PERMISSIBLE VELOCITIES AND
SIDE SLOPES FOR NON-ERODING EARTH CHANNELS

Material	Maximum Permissible Velocity (ft/sec)	Side Slope (horizontal : vertical)
Fine sand	1.5	3:1
Silt loam	2.0	2:1
Firm loam	2.5	1.5:1
Stiff clay	3.5	1:1

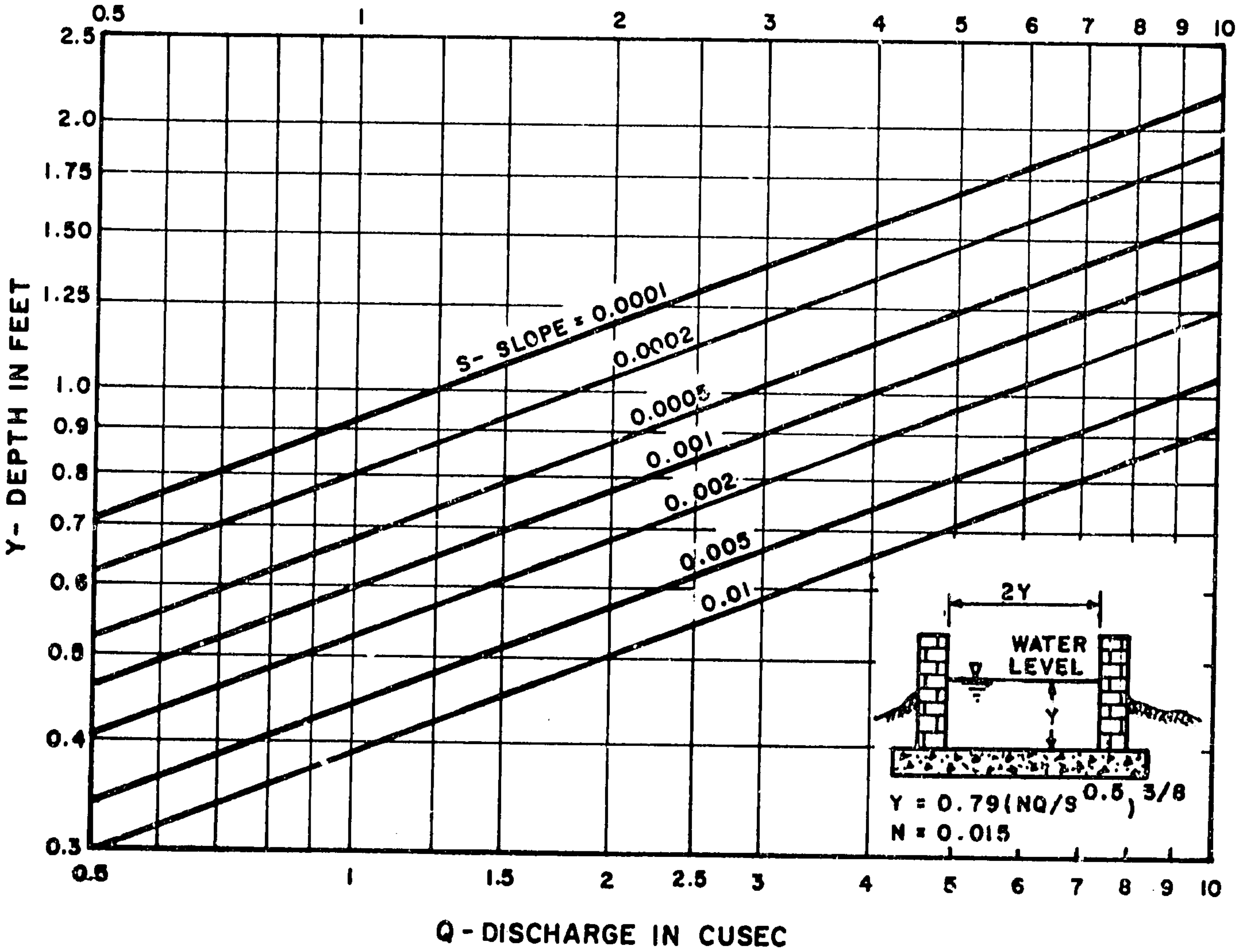
Some freeboard (additional height of channel sides) must be added above the design water surface to provide a safety factor to prevent spilling over the top. Field channels of less than 10-cusec capacity should have minimum freeboard of about 6 inches. This should be larger for rough channels or systems with irregular flow. Earth channel embankments should have a berm or top width of at least one foot on one side to provide stability, and at least two feet on the other to permit passage of pedestrians and livestock.

LINED CHANNEL DESIGN

Lined channels are simple to design because velocities are seldom limiting. The suitable cross section is calculated from the discharge and slope between the source and delivery point. **The best hydraulic section is defined as the cross section which will convey maximum water with minimum wetted perimeter, or area, for a given shape.** The best of all hydraulic sections is the half circle, but it is difficult to construct. The best trapezoidal section is half a hexagon. The best rectangular section is half a square. Because of the small width and right-of-way requirements and ease of construction with bricks, a rectangular channel is frequently used in Bangladesh.

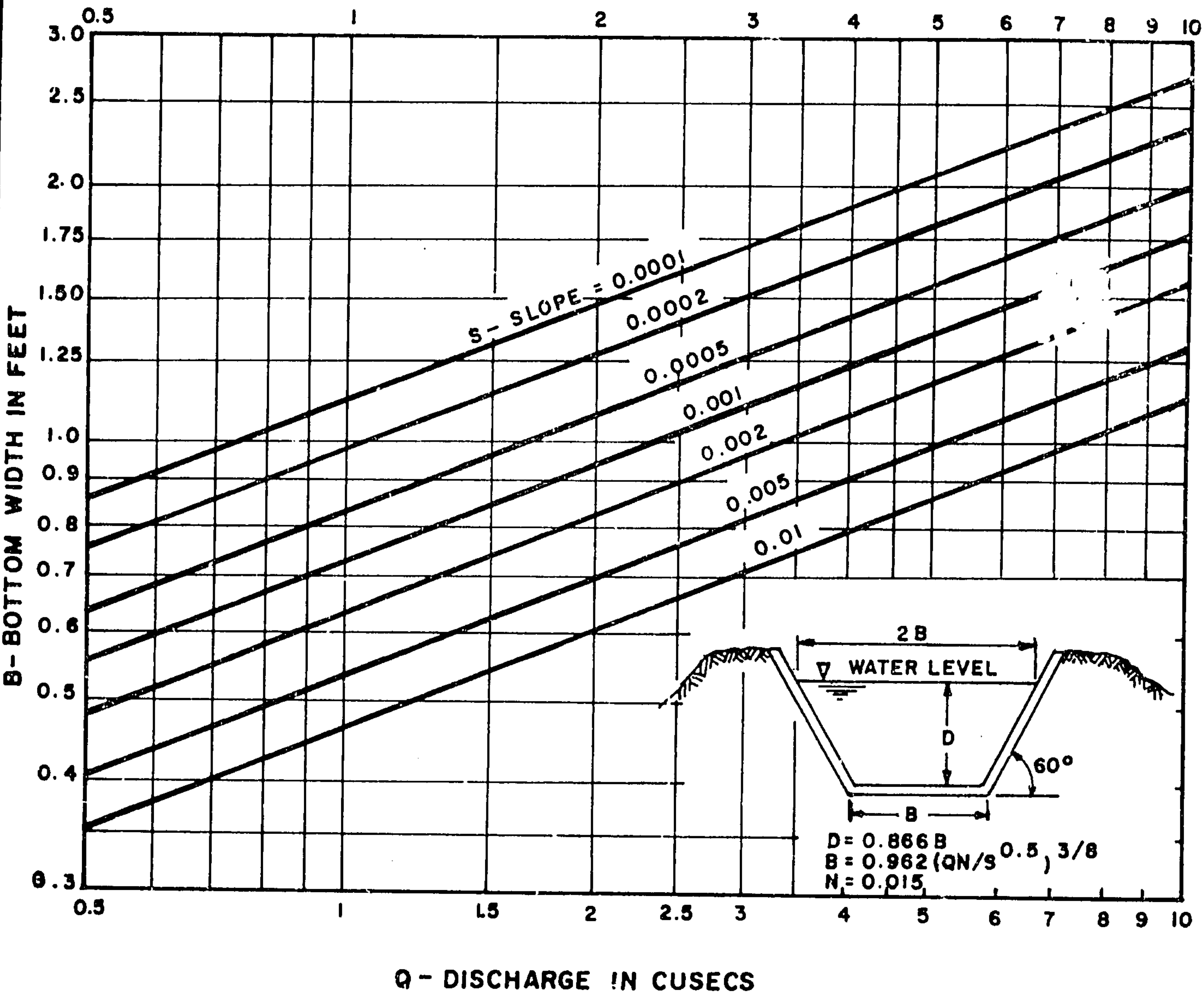
Once a geometric channel section has been selected, design can be greately facilitated by making a logarithmic graph of channel depth as a function of discharge for various slopes. Design charts for best masonry lined rectangular, trapezoidal and semi-circular hydraulic sections are given in Figs. 2,3 and 4.

If lined channels are placed on freshly-cut earth, the cut should be properly graded and smoothed, and any layer of plant or organic material should be removed. If the channel is placed on earth fill, it must be properly compacted and graded to assure structural stability.



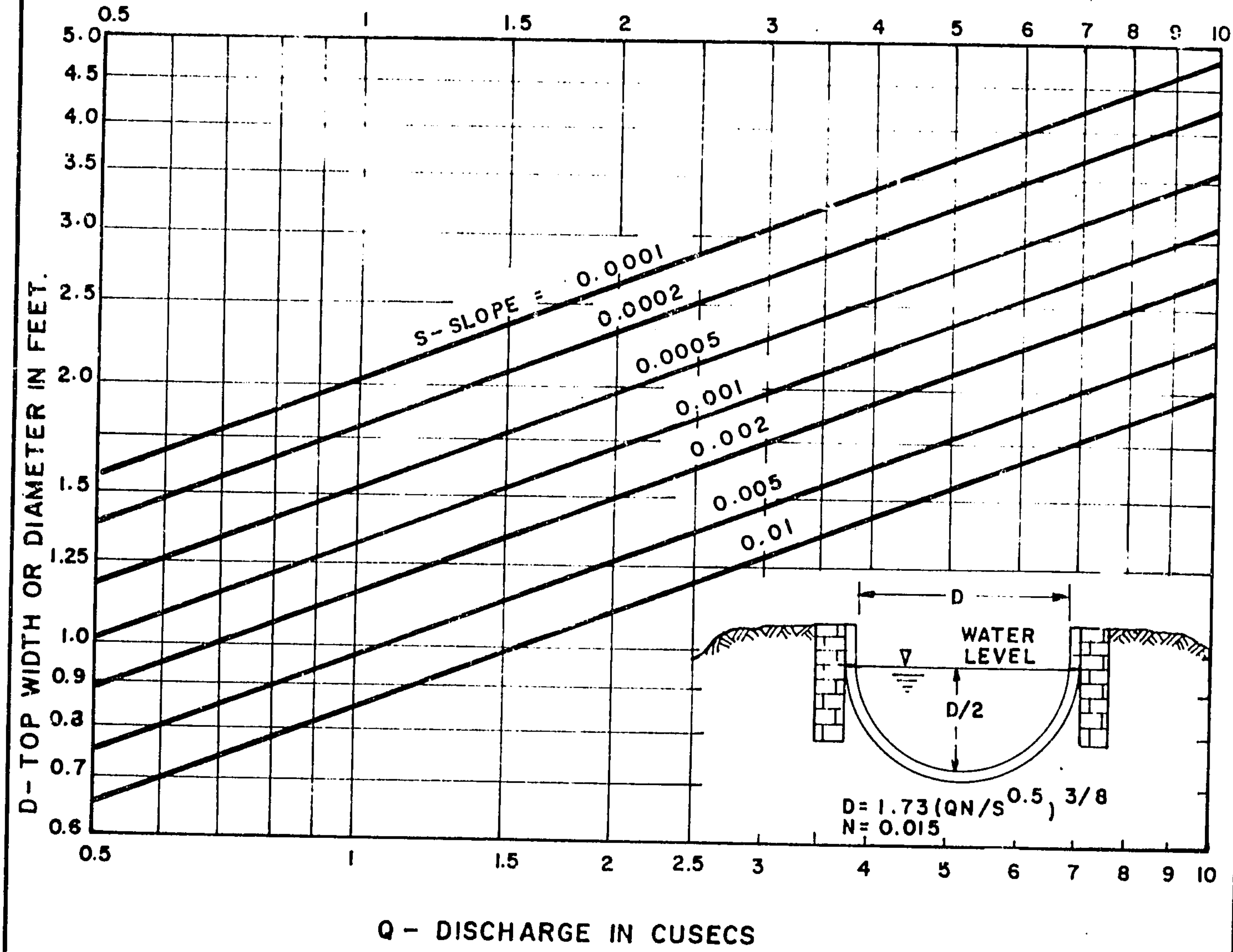
DEPTH VS. DISCHARGE
BEST RECTANGULAR HYDRAULIC SECTION

FIGURE - 2



BOTTOM WIDTH vs. DISCHARGE
BEST TRAPEZOIDAL SECTION

FIGURE - 3



DIAMETER vs. DISCHARGE
SEMICIRCULAR HYDRAULIC SECTION

FIGURE - 4

BURIED PIPE DESIGN

Pipe design and construction is a specialized field which must be carried out by experienced, qualified people to avoid errors which would greatly reduce effectiveness of the system. An excellent reference, from which much of the following material is taken, is listed in the bibliography (12).

This manual will consider lower pressure pipe (less than 15 ft of pressure head). Pipe lines will have risers with suitable valves to serve each block in the command area. The system will be designed so that only one block is served (one valve open) from a given pipe line or loop at a time. This is important in order to retain full stream size, as well as prevent differences in flow between risers due to differences in outlet elevation and pipe friction loss.

Water is pumped directly into an appropriate standpipe for delivery into the pipeline. The standpipe provides the necessary head in the pipeline, and dampens surges or water hammer during unsteady flow. The standpipe can also serve as a sand trap if the pump discharges sand or silt.

The additional head at the pump necessary for a pipe system must be accounted for in the design. If a pipe system is to be added to an existing pump, the efficiency characteristics should be known at the higher head to assure proper performance. The characteristics can be determined by looking at curves for the pump and engine if the total dynamic head is known, or by field measurements of discharge, head, pump speed and fuel consumption. The latter tests should be made to find the optimum speed of operation of any pump and engine used for irrigation and will be discussed in a following section.

Pipe size Selection: Size selection should be based on the criteria of maximum permissible velocity, head loss and economics of pipe and energy cost. Under no circumstances should velocities exceed five feet per second. The height of the pump standpipe is determined by friction loss in the pipeline, plus or minus the elevation difference between the standpipe and respective risers, and, of course, must be designed for the most extreme condition. To avoid problems in design, operation and maintenance the author recommends selecting pipe on the conservative (large) size when designing a low pressure system. It is important to remember that the cost saved using smaller pipe is reduced by higher pumping cost, higher standpipe and possibly increased maintenance due to higher pressure. Also the absolute safety factor in design of standpipe freeboard increases with use of smaller pipe.

In many cases design pipe size and cost can be significantly reduced by using pipe loops or double loops instead of lines. The principle will be explained in the Appendix in example designs.

Pipe friction can be calculated by several methods, including Manning's equation (2), or Hazen and Williams' formula. Values for Manning's roughness coefficient are given in table 5. Hazen and Williams' formula for pipe friction is given by:

$$H = 841 \times 10^{6}(Q/C)^{1.85}D^{-4.87} \qquad (5)$$

where H = friction loss in feet per 1000 feet of pipe

Q = Discharge (cusecs)

D = pipe diameter (inches)

C = roughness coefficient

Type of Pipe	C (Hazen & Williams)
Extremely straight and smooth asbestos cement	140
Smooth wood and masonry	120
Vitrified clay, concrete mortar joint	100
Rough and pitted concrete	60 - 80

Table 7

FRICTION LOSS IN CEMENT-ASBESTOS PIPELINES (Feet Per 1,000 ft. of Pipe)

Discharge	Pipe Size (Inside Diameter) 4-Inch	6-Inch	8-Inch	10-Inch
0.1 cusec	2.0			
0.2	7.2	1.0		
0.3	15.2	2.1		
0.4	**25.9**	3.6		
0.5	39.1	5.4	1.3	
0.6	54.8	7.6	1.9	
0.8	93.2	**13.0**	3.2	1.1
1.0	141.0	19.6	4.8	1.6
1.2		27.5	6.8	2.3
1.4		36.5	9.0	3.0
1.6		46.7	**11.5**	3.9
1.8		58.1	14.3	4.8
2.0		70.6	17.4	5.9
2.2			20.8	7.0
2.4			24.4	8.2
2.6			28.3	**9.6**
2.8			32.5	11.0
3.0			36.9	12.5

Based on Hazen & Williams, with C = 120.
Numbers below dotted lines have velocities above 5 feet per second and should be used with special caution.

Table 8

FRICTION LOSS IN MORTAR JOINT CONCRETE PIPE (Feet Per 1000 ft. of Pipe)

Discharge	**Pipe Size (Inside diameter)** **6-Inch**	**8-Inch**	**10-Inch**	**12-Inch**	**15-Inch**
0.3 cusec	3.0				
0.4	5.0	1.2			
0.5	7.6	1.9			
0.6	10.7	2.6	0.9		
0.8	**18.2**	4.5	1.5		
1.0	27.5	6.8	2.3	0.9	
1.2	38.5	9.5	3.2	1.1	
1.4	51.2	12.6	4.3	1.8	
1.6	65.5	**16.2**	5.5	2.2	
1.8		20.1	6.8	2.8	
2.0		24.4	8.2	3.4	1.1
2.2		29.1	9.8	4.0	1.4
2.4		34.2	11.5	4.8	1.6
2.6		39.7	**13.4**	5.5	1.9
2.8		45.5	15.4	6.3	2.1
3.0		51.7	17.5	7.2	2.4
3.5			23.2	**9.6**	3.2
4.0			29.7	12.2	4.1
4.5			37.0	15.2	5.1
5.0			44.9	18.5	6.2
6.0				25.9	**8.8**
7.0				34.5	11.6
8.0				44.1	14.9
10.0					22.5

Based on Hazen & Williams formula, with C = 100.
Numbers below dotted lines have velocities above 5 feet per second and should be used with caution.

Table 7 and 8 give average friction losses for cement-asbestos and concerete mortar joint piper.

Friction in pipe loops or parallel pipes can be analyzed by substituting an **equivalent pipe**. An equivalent pipe is any pipe which will carry a given flow with the same head loss as the system which it replaces. To find the equivalent pipe for a pair of parallel pipes, a head loss is assumed, and the flow in each pipe is computed. An equivalent pipe is computed using the combined flow at the assumed head loss.

An equivalent pipe for a number of pipes in series is computed by assuming a flow rate in the system and calculating a total head loss. Again, any single pipe which will carry the assumed flow with the computed head loss is an equivalent pipe.

Friction loss in pipe transitions and fittings can be estimated using the equation :

$$H = KV^2/2g \qquad (6)$$

where H = head loss (ft)

K = fitting coefficient

V = average velocity (ft/sec)

g = acceleration of gravity (32.2 ft/sec²)

The following "K" values can be used for design purposes :

Table 9

DISCHARGE COEFFICIENTS FOR PIPE FITTINGS

Fitting	**K**
Square edge inlet or contraction	0.5
Square edge enlargement	1.0
90° sharp elbow	1.0
45° sharp elbow	0.5
Curved 90° elbow (R = 2D)	0.2
TEE (line flow)	0.1
TEE (branch flow)	1.0

Riser Design: Vertical risers with an appropriate valve are used to discharge from the buried pipe to the ground surface or open channels. For the low pressure systems considered here, each riser will be designed to convey the full flow of the pipeline. A sketch of a typical riser and valve are shown in Fig. 5.

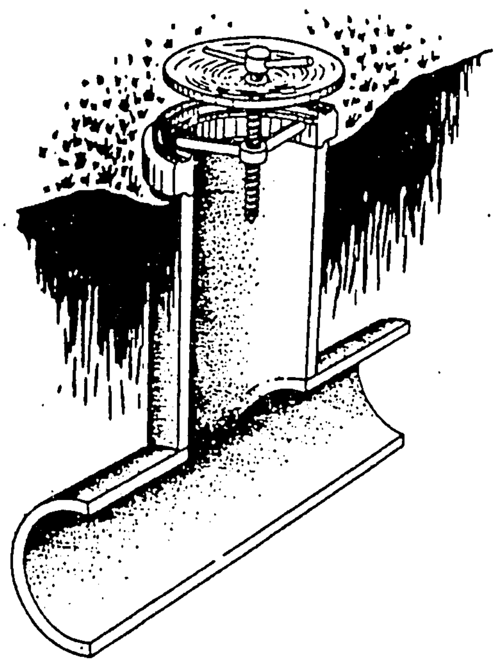


Fig. 5. Section of an Alfalfa valve mounted on concrete pipe

An **Alfalfa valve** is a screw valve grouted to the top of a riser pipe. A handle and cap plate is attached to a threaded rod that moves up and down as the handle is turned. When the valve is closed, the cap plate fits the circular edge of the valve case to make it watertight. When the plate is lifted by turning the handle, water is released from all sides of the valve. A complete line of riser valves can be procured from Mohinder & Co., Allied Industries, Kruali, Punjab, India, or Waterman Industries, Inc., P.O. Box 458, Exeter, Ca. 93221, U.S.A. There are numerous local foundries which could produce riser valves, given proper designs.

For low head systems the riser and valve should be the same size as the pipeline. Maximum design capacities for alfalfa valves are given in Table 10. Head loss through alfalfa valves can be computed by:

$$Q = 0.7A\,(2gH)^{0.5} \qquad (7)$$

where Q = flow rate (cusecs)

A = normal port area (square feet)

H = head loss through valve (feet)

g = acceleration of gravity (32.2 ft/sec^2)

The design water surface elevation at the riser is determined by the elevation of the highest respective plot and head loss in the channel between the plot and riser. The riser outlet elevation should be designed at a level low enough to prevent fall into the channel, with resulting head loss and scouring. The outlet crest elevation should be at or below the channel bottom grade. A concrete slab should be cast at the top of the riser pipe with some reinforcing wire for tying into and grouting the alfalfa valve. A masonry structure should be built above the slab to protect the riser and valve and serve as an outlet into the open channels. This structure can be designed according to the same criteria as open channel field outlets if flow is to be diverted into more than one channel. Figure 5(a) is an example of a riser discharging into an earth channel.

Table 10

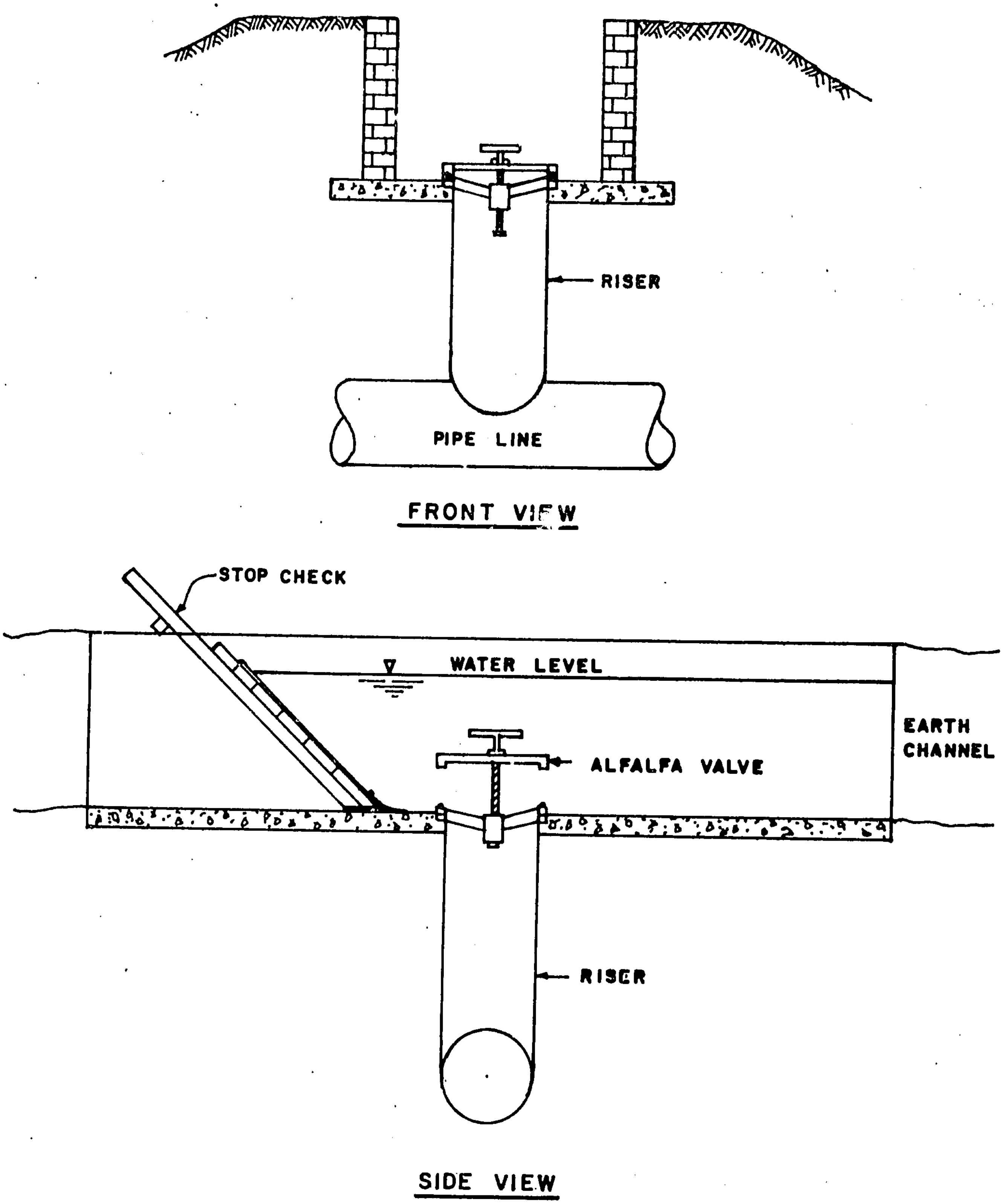
ALFALFA VALVE—MAXIMUM DESIGN CAPACITY

Inside Diameter of Riser and Valve	**Maximum Design Discharge**
6 inches	0.8 cusec
8	1.4
10	2.2
12	3.1
14	4.3

Pump Standpipe Design: The pump should discharge directly into a vertical standpipe or reservoir, which in turn connects to the pipeline. A typical design is shown in Fig. 6. The standpipe can be made of concrete pipe, cast concrete or brick, but care should be taken in design to assure structural stability, including hydrostatic forces.

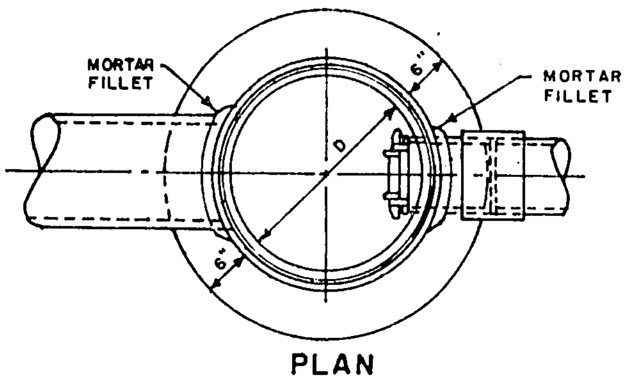
The inlet pipe (pump discharge) should be fitted with a flap or check valve to prevent water from flowing from the standpipe and pipeline back into the well when the pump is switched off, possibly causing damage to the equipment. The cross sectional area of the standpipe should be large enough to assure a maximum downward velocity of 2.0 ft/sec. The pump inlet pipe and pipeline outlet should be vertically offset by a minimum of the sum of their diameters. If the standpipe is to serve as a sediment trap, the downward velocity should not exceed 0.25 ft/sec, the inlet and outlet should be offset a minimum of twice the sum of their diameters, and the outlet invert should be at least two feet above the standpipe bottom. Access must be provided for cleaning. The height of the standpipe should be designed with a minimum of two feet freeboard.

Flow Division in Pipelines: In many cases it may be advantageous to divide flow from the pump outlet equally or proportionally into two or more pipe lines or loops. This can be done using a vertical standpipe fitted at the top with overflow weirs to discharge into adjacent standpipes



RISER, VALVE, AND WATER CONTROL STRUCTURE

FIGURE - 5a



NOTES :

1. WHEN D ≧ 27" OR WHEN D_2 IS GREATER THAN 1/2 D ELIMINATE FLAP GATE AND USE A CHECK VALVE IN PUMP DISCHARGE PIPE.

NOMENCLATURE

D- DIAMETER OF VERTICAL PIPE

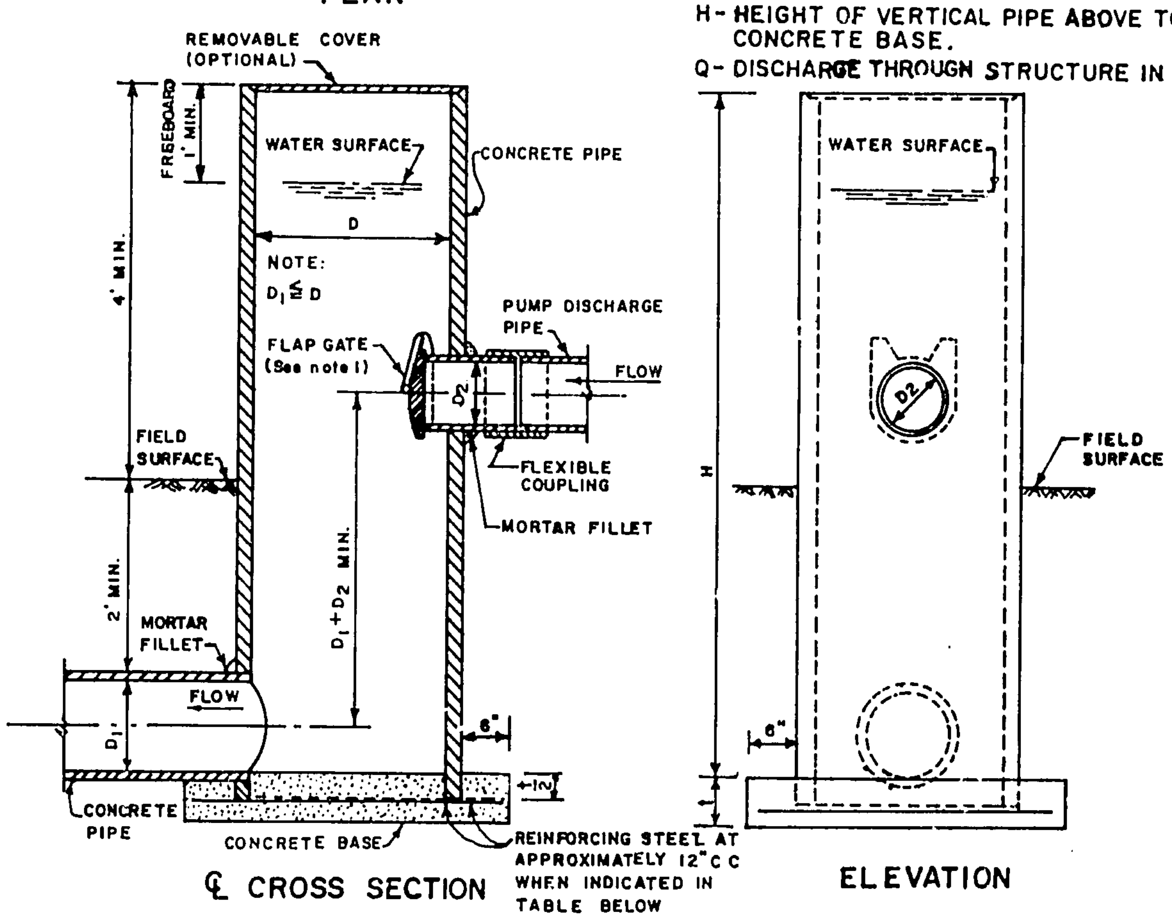
D_1- DIAMETER OF UNDERGROUND PIPE

D_2-DIAMETER OF PUMP DISCHARGE PIPE

t - THICKNESS OF CONCRETE BASE

H- HEIGHT OF VERTICAL PIPE ABOVE TOP OF CONCRETE BASE.

Q- DISCHARGE THROUGH STRUCTURE IN CFS.



Max. Q	D	A.S.T.M. Spec.		Concrete Base					
				H=10' or less		H=more than 10'		Reinforcing steel	
cfs	Inches	No.	Type	t	Cu.yd.	t	Cu.yd.	Size	Length
0.79	12	C-118	Concrete Irrigation Pipe	4"	0.05	6"	0.07	-	-
1.07	14			4"	0.05	6"	0.08	-	-
1.23	15			4"	0.06	6"	0.09	-	-
1.40	16			4"	0.06	6"	0.10	-	-
1.77	18			4"	0.07	6"	0.11	-	-
2.18	20			6"	0.13	8"	0.17	-	-
2.41	21			6"	0.14	8"	0.18	-	-
3.14	24			6"	0.16	8"	0.22	-	-
3.98	27	C-76	Class II Reinforced Concrete Pipe	6"	0.20	8"	0.26	3/8"	19'
4.91	30			6"	0.23	8"	0.30	3/8"	21'
5.94	33			8"	0.35	8"	0.35	3/8"	22'
7.07	36			8"	0.39	8"	0.39	3/8"	23'
9.62	42			8"	0.50	8"	0.50	3/8"	38'
12.57	48			8"	0.62	8"	0.62	1/2"	46'

PUMP STAND FOR CONCRETE PIPE

FIGURE- 6

and respective pipelines. The weirs can be used as measuring devices as well as proportional dividers. Weir crest elevations must be equal and above the lower design water surface under the most extreme conditions to prevent submergence.

Pipeline Anchors: Any abrupt change in pipeline grade or alignment must be secured with an appropriate anchor. This is to balance the internal forces caused by hydrostatic pressure, surges and change in momentum. As long as the pipe is in line, these internal forces oppose each other and cancel, but using the example of a 10 inch, 90° elbow or riser with a pressure of 10 lbs/sq. in., the thrust from the static pressure alone is 785 lbs (force equals pressure times area).

Anchors for bends can be constructed of concrete poured to fill the space between the pipe and the undisturbed earth at the side of the trench on the outside of bends. The anchors should be to the full height of the outside diameter of the pipe and should have a minimum thickness of six inches and a length in feet normal to the direction of thrust equal to;

$$L = 98\,HD\,\text{Sin}\,(a/2)/B \qquad (8)$$

where L=length of anchor (feet)

H=maximum working height in feet

D=inside diameter of the pipe in feet

B=the allowable passive pressure of the soil (lbs per sq. ft)

a=the deflection angle of the pipe bend

The pipe should be clean and wet when placing the anchor in order to provide a good bond between anchor and pipe. Where adequate soil tests are not available, the allowable passive soil pressure should be considered to be 500 lbs per sq. ft. All ends of rubber gasket jointed pipelines should be anchored.

Vent Stands: If air becomes entrained in a pipe line at the pump standpipe or any other location it can cause discontinuous flow, with resulting reduction of conveyance capacity and high pressure induced by surges or water hammer. If the pipe system has significant changes in relief, vent stands should be placed at summits in the pipeline or where there are changes in grade in a downward direction of more than ten degrees. The height of a vent standpipe should be about two feet above the maximum-design hydraulic grade line. The diameter of a vent stand should be at least half the diameter of the pipeline. Another option is to cap the vent pipe about one foot above its junction with the pipeline and reduce the vent pipe riser to a smaller size (not less than two inches).

It is good practice to put a vent standpipe just downstream from a pump standpipe if there is any possibility of air entering the pipeline. The vent standpipe distance from the pump standpipe can be determined by the equation L=1.8 VD, where L is the downstream distance in feet, V is the maximum design velocity in the pipeline in feet per second and D is the inside pipeline diameter in feet. A typical vent stand is shown in Fig. 7

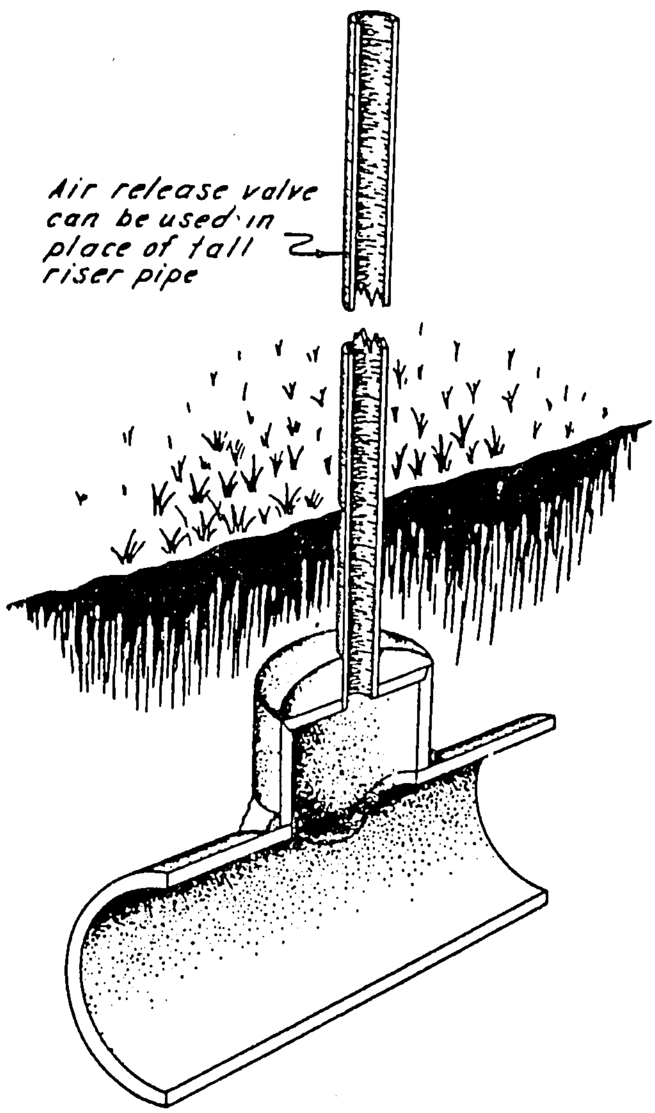


Fig. 7. Section of a Capped Vent

CONSTRUCTION OF CONCRETE PIPELINES

Grade and Alignment : Pipelines should be straight and of uniform gradient from reach to reach or between stands or anchors. This can be controlled by proper excavation of the trench. The alignment and grade for the bottom of the trench should be properly established prior to the excavation of the trench.

Excavation : Trenches should be reasonably straight, with the bottom reasonably free of undulations and humps. To facilitate economical pipe laying, all trenches should be excavated with vertical sides and a well-graded bottom. All excavated material should be deposited on one side of the trench and the pipe placed on the other side.

Dimensions : All pipe should be placed deep enough below the land surface to protect it from hazards imposed by traffic crossings, farm operations or soil cracking. The minimum depth of cover should be 18 inches for pipe sizes up to and including 12 inches in diameter, and for pipelines greater than 12 inches the minimum depth of cover should be 24 inches in cultived fields, in loams and coarser textured soils. In heavy soils such as clay, the depth of cover should be increased an additional six inches. This requirement should be governed by the usual texture of the soil horizon at the pipe depth, rather than by the texture of the surface soil.

The width of the trench should be sufficient to permit laying of pipe correctly and for proper bonding or finishing of the joints. There should be a minimum clearance of six inches from the outside of the pipe to the sides of the trench.

Foundations for pipelines should be firm but should yield slightly and uniformly when pipe is bedded to establish line and grade. Where a firm foundation is not encountered, due to soft, spongy or other similarly unsuitable material, all such unstable material under the pipe and for a width of not less than one diameter on each side of the pipe should be removed and the space backfilled with suitable material, such as coarse sand, and properly compacted to provide adequate support for the pipe.

Trench in Clay or Expansive Soils : Where trenches are excavated in clay soils or those that show evidence of shrinking or swelling, the trench should be excavated four to six inches below grade and brought up to grade with a compacted layer of loam or coarser textured soil. In all clay or expansive soils the pipe should be deep enough so that the soil mass surrounding it will not be subject to changes of moisture and temperature.

Water in Trench : When water is encountered in the trench, it should be removed by draining or pumping. Should water get into the trench before the pipe is laid, the laying of pipe should be postponed until the trench has dried sufficiently to provide a firm foundation for the pipe, or else the mud and softer material should be removed and grade reestablished by backfilling with suitable compacted material.

Placement : At the time of laying, the prepared trench should be in a reasonably dry condition. Necessary facilities should be provided for lowering and properly placing the sections of pipe in the trench without damage. Immediately before placing each section of pipe in final position for jointing, the bedding for the pipe should be made by scraping away or tamping under the body of the pipe and not by wedging or blocking.

The interior of the pipe should be kept free from dirt and other foreign material as the pipe laying progresses, and left clean at the completion of the work. Any pipe which is not in true alignment, or which shows any undue settlement after laying or is damaged, should be taken up and relaid. No pipe should be laid which is cracked, checked, spoiled or damaged beyond specification tolerances, and all such sections of pipe should be permanently removed from the work site. Curves are allowed with a maximum deflection of five degrees in mortar joints and a maximum of three degrees in rubber gasket pipe, with all joints receiving an equal amount of deflection.

In advance of jointing sections of pipe, the ends of each section should be washed clean with a wet brush, and immediately prior to placing mortar and jointing the sections, the ends should be thoroughly wetted. The groove or socket end of the next section of pipe is wetted and filled with mortar. This section is then tipped over carefully so as not to dislodge the mortar and the two sections of pipe firmly pressed together in such a manner that the groove or socket end of the pipe fits truly and snugly over the tongue or spigot end to which it is to be fitted, so that mortar is squeezed out from the inner and outer surfaces. Care should be taken that no mortar falls from the groove or socket end during the abutting operation. Because extruded mortar would reduce flow and increase friction, the inside of the pipe should be wiped smooth of any surplus mortar. In pipes too small for a man to work inside, wiping may be done by dragging a swab or long handled brush on the inside. This is done after the pipe is placed true to line and grade, because any movement thereafter may cause the joint to leak.

Rubber Gasket Joints : The first section of pipe laid should be firmly bedded in the center of the trench to establish line and grade, with the socket end pointing in the direction to be followed by the pipe laying. Rubber "O" rings are available to seal cement asbestos pipe manufactured in Bangladesh. The "O" ring is placed just on the end of the pipe to be fitted into the socket, and it will roll into place between the pipe and socket as they are pushed together. In order that rubber rolls instead of slipping, it is important that the pipe be dry. If it is wet, it can be dried by dusting both surfaces with dry portland cement. If, while making the joint, the gasket becomes loose or crooked and can be seen through the exterior joint when the joint is pulled up to within one inch or closer, the pipe should be removed and the joint remade to the satisfaction of the engineer.

Oakum (Hemp or Jute Fiber) and Mortar Joints A closely-twisted gasket of oakum, of the diameter required to support the spigot of the pipe at the proper grade and to make the joint concentric, should be used. The joint packing should be in one place, of sufficient length to pass around the pipe and should lap at the top. This gasket should be thoroughly saturated with neat cement grout. The socket of the pipe should be thoroughly cleaned with a wet brush and the gasket laid in the socket for the lower third of the circumference and covered with mortar. The spigot of the pipe should be thoroughly cleaned with a wet brush, inserted in the socket and carefully shoved in. A small amount of mortar should be inserted in the annular space for the upper two-thirds of the circumference. The gasket should be lapped at the top of the pipe and driven into the annular space with a caulking tool.

The remainder of the annular space should then be filled completely with mortar and beveled off at an angle of approximately 45° with the outside of the socket. The finishing of this joint should be at least five joints behind the laying operation.

Collar Joints : Most concrete pipe available in Bangladesh has plain ends, and is butted together and joined with a concentric concrete collar. Before a pipe is laid, a collar is slipped on its ends. The butted pipe ends are caulked with mortar or jute rope dipped in hot bitumen. If rope and bitumen are used, the pipes must be pressed or squeezed together before placing the collars. The butted joints are squeezed by placing a jack in the trench horizontally at the last pipe. Four or five pipes can be squeezed at one .time.

When the butted joints are prepared and stabilized, the collars are slipped over so that half the collar width covers each side of the joint. In order to maintain uniform clearance between the pipe and collar, wooden battens or wedges are plugged in both sides of the collar between the collar and pipe. The gap between the pipe and collar is then filled with mortar and tampec in with an appropriate tool. Once the gap is sealed satisfactorily the collar is lined with morta and beveled at an angle of 45° to the pipe.

The mortar for the joints should consist of not less than one part portland cement to twc parts of clean, well-graded sand that will pass a 2.36 mm (U.S. No. 8) sieve. The quantity ot water in the mixture should be sufficient to produce a soft workable mortar, but should in nc case exceed six U.S. gallons per cubic foot of cement. The consistency of mortar should be such as to adhere to the ends of the pipe while being laid and easily squeezed out of the joints when the pipe sections are pressed together. Banding mortar should be plastic and of such consistency that it will readily adhere to the pipe.

Another acceptable method of sealing bell and spigot and collar joints is use of a dry mix of cement and sand in a 1:1 ratio. The dry mixture is compacted into the joints with a flattened steel chisel and hammer. When the joint is filled, it is finished with a band of wet cement mortar beveled to 45°. All mortar should be used within thirty minutes after mixing with water.

Joint Curing and Backfilling : Openings in all concrete pipelines should be covered to prevent air circulation, except when work is actually in progress. Such openings should be kept closed until the pipeline is to be filled with water. There should be an initial backfill of soil around the pipe and covering the pipe to a depth of at least six inches for the full width of the trench and not more than seven sections behind the laying. Care should be taken in placing such earth around the pipe to avoid injury to the joints. Mortar joints should be protected from drying out. If the soil used in the initial backfill is not thoroughly moist, the joints should be covered with a layer of jute bags which are kept moist. If laying ceases for two hours or more, the initial backfill should be brought up to and over the last completed joint. Nothing prohibits the complete backfilling while mortar bands are still plastic as long as care is taken. In case complete backfilling is not done at this time, the completion should be delayed at least 20 hours. In placing the backfill the earth should be placod on each side of the line at the same time to avoid displacement or injury to the green mortar joints and bands. It is desirable to backfill the lines as soon as practical to avoid shrinkage of the joints and bands. Where the backfill is to be flooded to expedite its consolidation, it is absolutely necessary to fill the pipe with water; otherwise it may float. Backfill should be placed in uniform layers. Each layer should be carefully and uniformly compacted or consolidated in such a manner so as to completely fill the voids under the pipe haunches and around the pipe. Uncompacted fill should be mounded over the top of the trench to allow for settlement.

In expansive or clay soils, a loam or coarser textured soil should be used for backfill. This type of backfill should be placed up to at least half the diameter of the pipe. Water should not be turned into the pipeline until all backfilling is completed and in no case within 24 hours of finishing the pipe joints. Maximum hydrostatic pressure should not be applied to the pipe within three days of finishing the pipe joints.

Openings in Pipelines : All openings cut into the concrete pipe for outlets and connections should be full size, within one inch of the inside diameter of the connecting pipe or fittings. However, if at all possible such openings should be avoided. All connections should be cut to fit closely and should be strongly cemented to the pipe with banding mortar, and where possible both the inside and outside of the joint should be brushed smooth. In all cases pipes should be clean and wet before mortar is applied. No trash or other obstructions should be left in the pipeline.

Bases for Stands : The first section of pipe for stands should be placed on the base of concrete before initial set of the concrete, or the pipe can be placed first and the concrete poured and tamped in around the pipe. Concrete bases of stands should have a diameter at least ten inches greater than the outside diameter of the stand and should be at least four inches thick for stands not over 18 inches inside diameter or over ten feet high above the base. After initial set and before final set of the concrete base, water to a depth of four to six inches should be carefully poured into the stand, or it should be loosely covered with about six inches of most soil.

Inspection of Finished Joints: Tests should be made to assure that all pipelines function properly at design capacity. At or below design capacity there should be no objectionable surge or water hammer. It is objectionable if there is either a) continuing unsteady delivery of water, b) damage to the system or c) overflow from vents or stands. Pipelines should be tested for leaks by observing their normal operation any time after a period of two weeks of continuous wetting. All detectable leaks should be repaired. Losses for mortar-jointed pipelines should not exceed 0.05 cubic feet per square foot of inside surface in 24 hours. Losses from rubber gasket lines should not exceed 0.02 cubic feet per square foot of inside surface in 24 hours. Leak rate tests can be made by closing all the riser valves in the pipe system and filling the pump standpipe to the design head and measuring the rate of fall in the standpipe. It is important to assure there is no air in the pipe system and no leakback through the pump and well during this test.

Sand, dirt and other suspended matter should not be permitted to settle and remain in a pipeline. This especially applies to the low points where such material naturally accumulates. Besides reducing the capacity of the pipeline, constrictions thus caused may develop an abnormal stress on the line. A sediment trap at the pumphead will obviate this difficulty.

To avoid sudden application of excessive pressures, valves in the pipeline must be opened and closed gradually. In filling a pipeline where water is standing in the low points, extreme care must be exercised to avoid excessive pressure due to trapped air. By keeping gates and valves closed, excessive rusting and pitting of seals is avoided.

WATER MEASUREMENT

Water must be measured to determine application amounts during irrigation scheduling. If a reservoir is used for storage, a measurement device must be built into the outlet structure to regulate outflow. Studies to determine channel losses and feasibility of lining or upgrading involve flow measurement into and out of a channel reach. Water must be measured to determine and optimize pump-engine efficiency as a function of pump speed. Methods considered to be the most appropriate for typical systems in Bangladesh will be presented.

The Porshall Flume: This is a standard device for accurate measurement of discharge in an open channel. Its advantages are that it requires a minimum of head loss compared to other structures; is self-cleaning and has a high degree of accuracy and standardization. Due to the characteristic of minimum head loss it is considered to be ideal for discharge measurement and regulation of flow from storage reservoirs or channel loss studies. The disadvantage of the Parshall flume is that it is somewhat difficult to construct. However, there is no reason why suitable flumes cannot be made in Bangladesh by skilled craftsmen with bricks and concrete, or sheet metal.

The flume is made to contract flow in an open channel and cause critical depth in the throat. When critical depth occurs in the throat, discharge is a discrete function of the upstream depth and is unaffected by downstream water surface fluctuations.

Fig. 8 shows a standard Parshall flume, with the corresponding dimensions given in Table 11. Rating curves for nine inch and one foot flumes are shown on Fig.9. One of these flumes should cover the range of discharge encountered in the systems described.

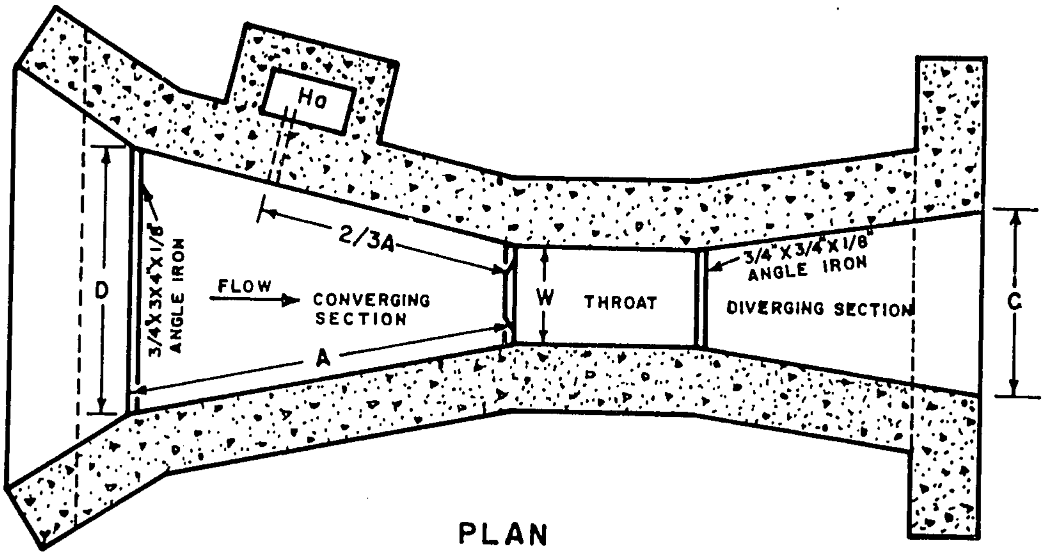
For channel loss studies, two portable flumes are used at the head and tail of a channel reach, measuring difference in flow. Since the difference may be a small percentage of the total discharge, the flumes must be accurate and should be calibrated in a laboratory.

When placing a flume in a channel system, it is critical that the flume crest be high enough in relation to the downstream channel water surface. If H is the water depth at the upstream gauge point in the flume, and the water surface in the downstream channel is at a vertical distance Y below the gauge point, then the submergency of the flume is defined as:

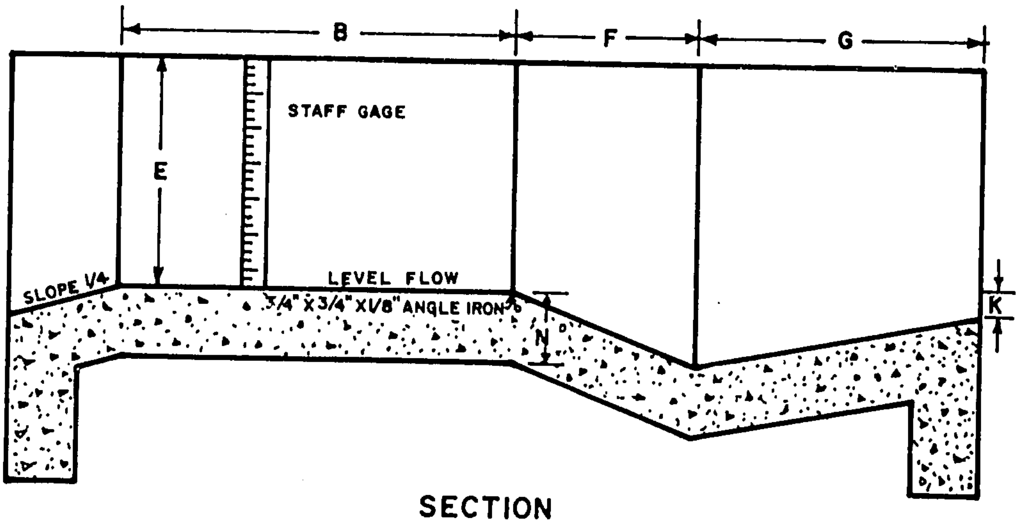
$$S = (H - Y)/H \tag{9}$$

If submergence is increased above a certain value, critical depth will no longer be established in the flume contraction, and the free discharge flow relations no longer hold true. The maximum free-flow submergence for various size flumes is included in Table-11. Equations relating discharge to head under free-flow conditions are given below:

STANDARD PARSHALL FLUME



PLAN



SECTION

Plan and elevation of a concrete Parshall measuring flume. Lettered dimensions are shown as follows:

W - size of flume, in inches or feet
A - length of side wall of converging section
2/3A - distance back from end of crest to gage point
B - axial length of converging section
C - width of downstream end of flume
D - width of upstream end of flume
E - depth of flume
F - length of throat
G - length of diverging section
K - difference in elevation between lower end of flume and crest
N - depth of depression in throat below crest
(See Table 11 for actual dimensions for various sizes of flumes)

FIGURE-8

Table 11

DIMENSIONS AND CAPACITIES OF THE PARSHALL FLUME FOR VARIOUS THROAT WIDTHS(w). LETTERS REFER TO DIMENSIONS ON FIGURE 8

Throat Width (w)	**Feet 0**	**Inches 6**	**Feet 0**	**Inches 9**	**Feet 1**	**Inches 0**	**Feet 1**	**Inches 6**
A	2	7/16	2	10-5/8	4	6	4	9
B	2	0	2	10	4	4-7/8	4	7-7/8
C	1	2-1/2	1	3	2	0	2	6
D	1	3-5/8	1	10-5/8	2	9-1/4	3	4-3/8
E	2	0	2	6	3	0	3	0
F	1	0	1	0	2	0	2	0
G	2	0	1	6	3	0	3	0
K	0	3	0	3	0	3	0	3
N	0	4-1/2	0	4-1/2	0	9	0	9
Maximum Capacity	3.0 cfs		8.8 cfs		16.1 cfs		24.6 cfs	
Maximum Sub-mergence	0.56		0.60		0.62		0.64	

Throat Width (w)	Equation
3 inch	$Q=0.992H^{1\cdot547}$
6 inch	$Q=2.06H^{1\cdot58}$
9 inch	$Q=3.07H^{1\cdot53}$
12 inch to 8 feet	$Q=4WH^{1\cdot522w^{0\cdot026}}$

where Q=discharge (cusec)
H=depth of water at gauge point in flume (feet)
W=size or throat width (feet)

Weirs: When conditions are favorable, the weir is one of the simplest, cheapest and most reliable devices for measuring the flow of water.

The following terms are used in connection with weirs:

Weir : A bulkhead placed across a ditch or stream with an opening cut in the top through which the water is allowed to pass. The opening is called the weir notch.

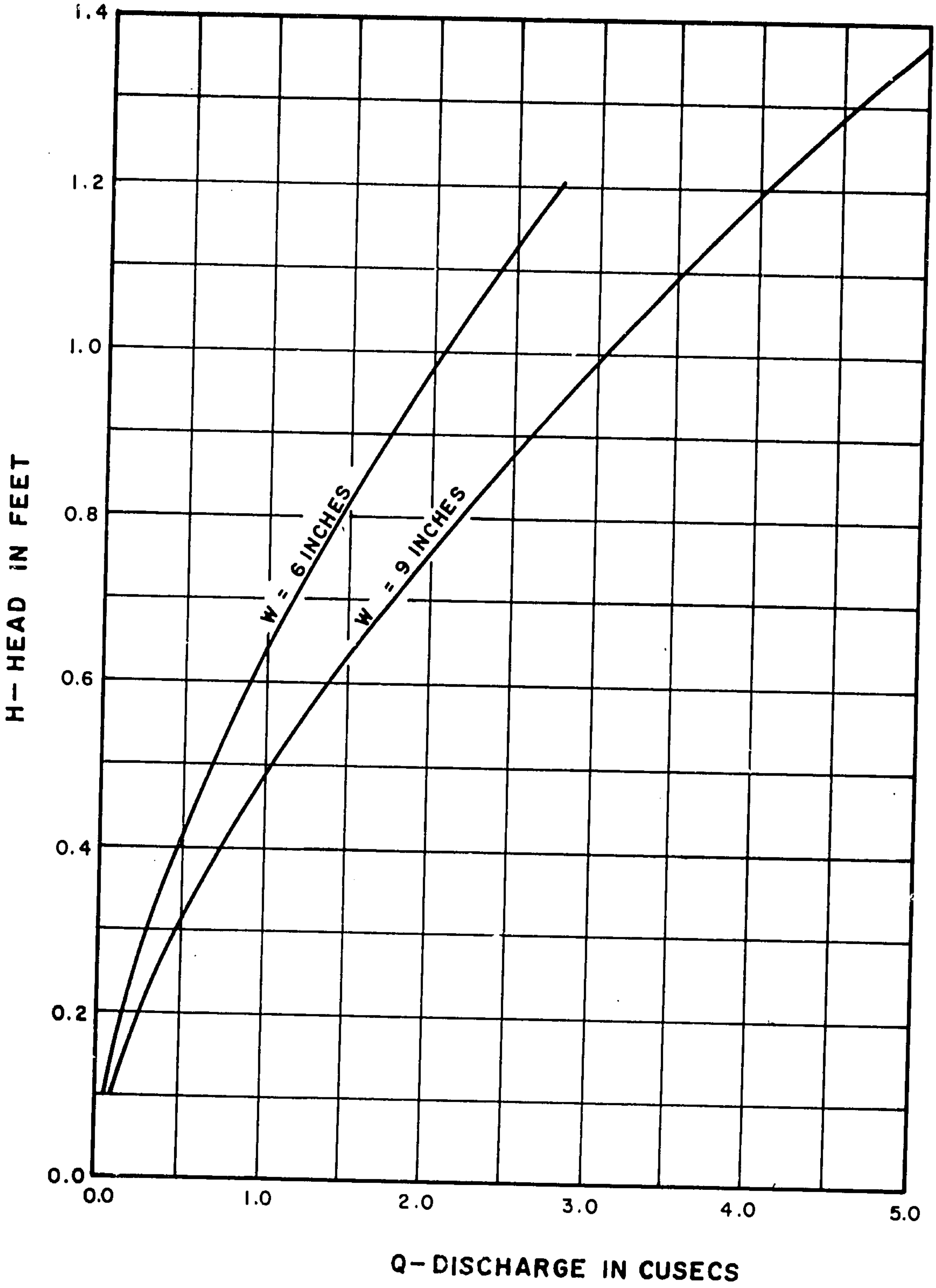
Weir Pond : The portiom of the ditch immediately upstream from the weir.

Weir Crest : The bottom of the weir notch.

Head-on Crest: The depth of water flowing over the weir crest measured at some point in the weir pond.

Sharp-Crested Weir : A weir having a thin-edged crest and sides such that the overflowing water touches the crest at only one point.

End Contraction : The horizontal distance from the weir crest to the opposite side of the weir pond.



RATING CURVES FOR PARSHALL FLUMES

FIGURE-9

Bottom Contraction : The vertical distance from the weir crest to the bottom of the weir pond.

Weir Scale or Gauge : The scale fastened on the side of the weir or on a stake in the weir pond to measure head-on crest.

Weirs may be divided into two general classes : 1) sharp-crested and 2) broad-crested. The sharp-crested may again be divided into weirs with end contractions and weirs without end contractions. Only the sharp-crested weir will be discussed here.

Weirs may be built as stationary structures or they may be made portable. Portable weirs are usually made of wood or sheet steel, and are placed in the ditch where a measurement is desired. The stationary structures may be built of wood, steel or concrete. In the wood and concrete structures the notch is usually faced with a metal strip to form the sharp crest.

The discharge through a weir notch is proportional to the head of the crest and is affected by the condition of the crest, the contraction, the velocity of approach and the elevation of the water surface downstream from the weir. Each type of weir has its own discharge characteristics, which can be defined by a formula, tables or curves. Therefore, in order to properly measure water with a weir, it should be constructed and installed in a manner similar to that for which the formula, tables and curves were developed. The following are some general requirements for the proper setting and operation of weirs :

1. The weir should be set at the lower end of a pool sufficiently long, wide and deep to give an even smooth current with a velocity of approach of not over 0.5 feet per second.
2. The longitudinal axis of the weir should be perpendicular to the direction of the flow. If a weir box is used, the center line of the weir box should be parallel with the direction of flow.
3. The face of the weir should be vertical.
4. The crest of the weir should be horizontal, so that the water passing over it will be the same depth at all points along the crest.
5. The height of the crest above the bottom of the pool should be about three times the depth of water flowing over the weir crest. The sides of the pool should be at a distance of no less than twice the depth of the water passing over the crest from the sides of the crest.
6. The gauge or weir scale may be placed on a stake at any point in the weir pond or box, provided it is sufficiently upstream or to one side of the weir to be free from the downward curve of the water surface as it passes over the weir crest. The zero of the weir scale or gauge must be placed at the same elevation as the weir crest.
7. The crest should be placed high enough so that the water will fall freely below the weir, leaving an air space under the over falling sheet of water. If the water below the weir rises above the crest elevation, free fall is not possible and the weir is then said to be submerged. Unless complicated corrections are made, measurements on submerged weirs are unreliable.
8. For accurate measurements, the depth of water flowing over the crest should be no more than one-third the length of the crest.
9. The depth of water flowing over the crest should not be less than two inches. With smaller depths the sheet of water tends to cling to the downstream side of the crest and the relationship between the depth of water on the crest and the discharge no longer holds true.
10. To prevent erosion below the weir, the ditch downstream should be protected by loose rock, stilling basin or other non-erosive material.

11. The crest of a weir must be kept free of debris, and the pond or box above the weir must be kept free of sediment which would restrict end or bottom contraction.

Discharge over a rectangular sharp-crested weir with bottom and end contraction as outlined above can be calculated by :

$$Q = 3.3(L - 0.2H)H^{1.5} \qquad (10)$$

where Q = discharge (cusecs)

L = length of the weir crest (feet)

H = head on the weir (feet)

The reduction in the crest length by 0.2H in equation (9) compensates for the contracted ends of the weir. For calculation of discharge through a sharp-crested weir without end contractions (suppressed weir), the actual crest length without the 0.2H reduction is used.

The 90° vee notch weir is especially adapted to measurement of small discharges ranging for about 0.1 to 3.0 cusecs and is ideal for measuring discharge from small pumps. Discharge through a vee notch weir can be calculated by :

$$Q = 2.52H^{2.47} \qquad (11)$$

where Q = discharge (cusecs)

H = head above the lowest point in the weir notch (feet).

Discharge from Pipes : Frequently it is convenient to estimate discharge from a round pipe using coordinates of the free jet of water. Such methods have limited accuracy. If the issuing jet is so near a pump or pipe fitting, such as an elbow, which causes extreme turbulence, or if the pipe is flowing partially full, this method should not be used.

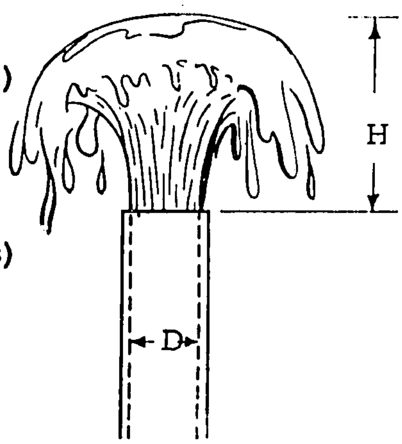
If the water is flowing from a vertical pipe, and the height of the jet reaches a distance greater than 1.4 times the pipe diameter above the end of the pipe, then the discharge can be expressed by :

$$Q = (D^2H^{0.53})/90 \qquad (12)$$

where Q = discharge (cusecs)

D = pipe diameter (inches)

H = maximum height of the jet above the end of the pipe (inches)



Discharge from a horizontal pipe can be estimated by measuring the horizontal and vertical coordinates (X and Y) of the jet at some distance from the end of the pipe. The approximate discharge is given by:

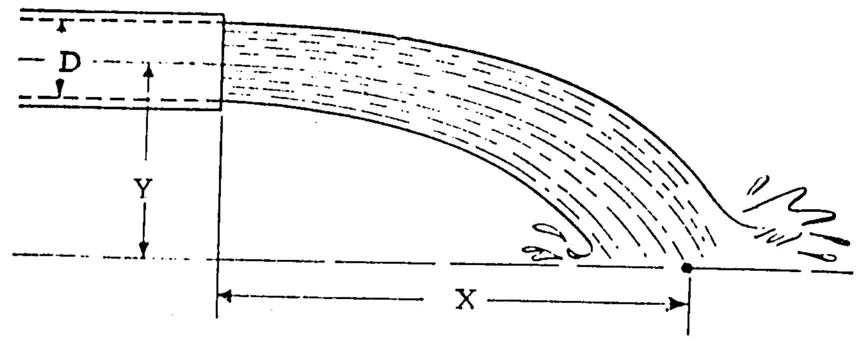
$$Q = D^2 X/(158Y^{0.5}) \tag{13}$$

where Q = discharge (cusecs)

D = pipe diameter (inches)

X = horizontal distance of jet from pipe end (inches)

Y = vertical fall of jet from pipe end (inches)



Equation 12 also applies to a pipe at an angle to the horizontal if the X ordinate is measured parallel with the pipe axis and the Y ordinate is measured vertically (1 cusec equals 448 gals per min.).

For buried pipe systems it may be necessary to measure discharge rate or volume in a closed pipe. Flow rate can best be measured with an in-line orifice meter fitted with an appropriate differential manometer. Flow volume over a period of time can be measured with an in-line propeller flow meter.

Orifices: The general equation for flow through an orifice is given by:

$$Q = KA(2gH)^{0.5} \tag{14}$$

where Q = flow rate (cusecs)

K = orifice coefficient

A = orifice area (sq. ft)

H = head differential across orifice (feet of water)

g = acceleration of gravity (32.2 ft/sec^2)

For a sharp edge orifice with pressure taps one inch upstream and downstream from the orifice place, approximate values of "K" as a function of the ratio of orifice to pipe diameter (d/D) are shown in Fig. 10. This is for design purposes only, and orifice meters should be calibrated in a laboratory in the design flow range. The differential head loss is best measured with an appropriate manometer.

Propeller Flow Meters: Irrigation meters essentially consist of a conical propeller connected to a registering head by a gear train. They are operated by the kinetic energy of the flowing water. The propeller is suspended, facing the center of flow, in the pipe, tube or conduit and is rotated by the flow of water. The speed of the propeller (rpm) is proportional to the average velocity of flow within the tube (ft/sec), and since the cross sectional area of the tube is known and remains constant, the propeller speed is proportional to the rate of flow. The rotating propeller actuates the registering head through the gear train. This head registers total flow on a counter-type clock. The total flow is recorded directly in standard volumetric units, such as gallons or cubic feet.

There are two basic requirements for accurate operation of the meter: 1) the tube must flow full at all times, and 2) the rate of flow must exceed the minimum for the rated range. Meters are given a volumetric calibration test at the factory, and adjustment or recalibration in the field is not normally required.

Irrigation meters have a number of advantages over other methods of water measurement. Registration is independent of variations in the line pressure or in the rate of flow within the rated range, thus eliminating frequent readings and checks. Since the meters total flow directly, no time-consuming computations are involved and human errors are eliminated. The principal disadvantages of these meters are their susceptibility to clogging with moss or debris. If used with a surface water pump, the intake water must be screened to eliminate debris.

Fig. 11 shows a low pressure line meter installed in a section of steel pipe. Note the straightening vanes installed ahead of the propeller to eliminate turbulence.

Fig. 10. RATIO OF ORIFICE TO PIPE DIAMETER (d/D)

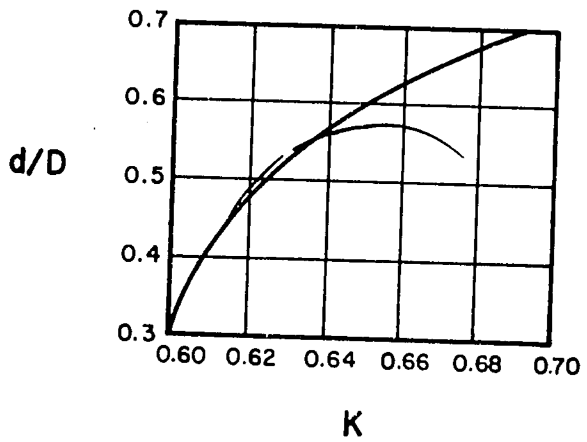


FIGURE-10

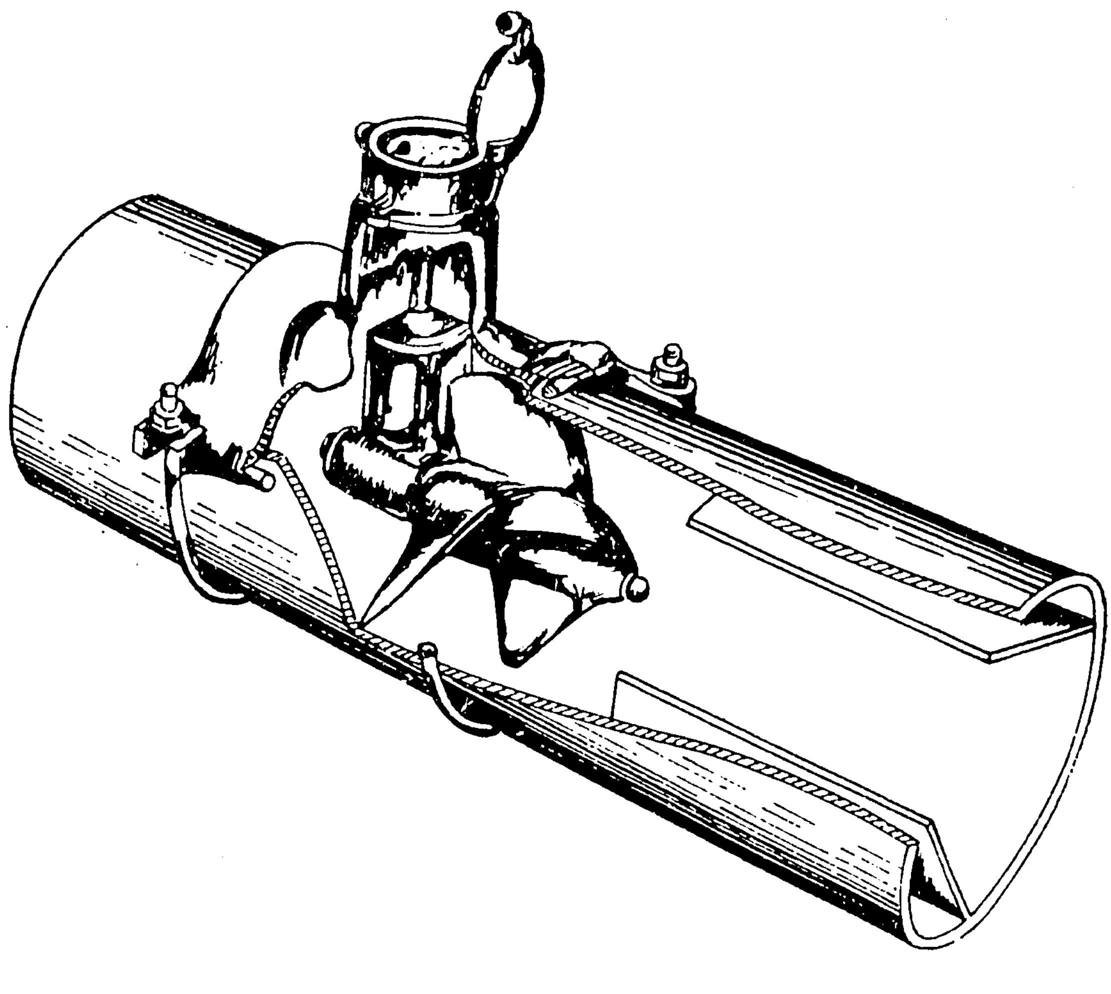


Fig. 11. Flow Meter

STORAGE RESERVOIRS

In many cases it is advantageous to include a small elevated storage reservoir in irrigation water distribution systems. The reservoir is located next to the pump and is used to store water pumped at night for use the following day. This eliminates the need of farmers staying up all night to manage their irrigation during peak use and increases the working stream size. The reservoir can also be managed for fish culture. If managed properly it can yield more profit per acre from fish than the loss from the land taken out of crops.

Design of the Reservoir : The reservoir should be approximately round or square to minimize the earthwork and land area required. The effective storage volume required can be computed by :

$$V = 3600(Q)\ (I-O) \qquad (15)$$

where V = storage volume (cu. ft)

Q = reservoir inflow or pump discharge (cusecs)

I = hours of pump operation per day

O = time of irrigation or reservoir outflow (hours per day)

The reservoir surface area is given by :

$$A = V/D \qquad (16)$$

where A = surface area (sq. ft)

D = effective depth of storage (ft)

The minimum design water surface elevation in the reservoir is determined by the design water surface in the channel at the reservoir outlet, plus the head loss in the reservoir outlet structure. For a given usable storage volume, the reservoir surface area (and land requirement) decreases and the embankment height and earthwork volume increase as the effective storage depth increases. Economic optimization and judgment should be used in selecting effective storage depth. In most cases it should be between two and four feet. The embankment should have a freeboard at least two feet above the maximum design water level, a minimum top width of six feet, and side slopes given in Table 6. The embankment fill can be taken from inside the reservoir if it is suitable from the standpoint of stability and permeability, and should be well-compacted, graded and planted with a suitable cover for stabilization.

Design of Reservoir Outlet Structure : An outlet structure must be designed to discharge the total volume of water pumped in one day during the required irrigation time period and the pressure head range of the reservoir. A simple design is an outlet controlled with an adjustable gate or valve and outflow-measuring device such as a Parshall flume. The discharge is regulated and periodically adjusted to maintain the desired outflow, as the head in the reservoir falls. For example, if the inflow is 2.0 cusecs around the clock, and the outflow is needed for eight hours daily then the outflow must be controlled to 6.0 cusecs and periodically adjusted as the head in the reservoir falls.

The outlet conduit is designed to conduct the necessary discharge at minimum head conditions, i.e., when the water level in the reservoir is lowest. Assuming a square outlet, the following equation can be derived from energy principles and Manning's equation :

$$H = Q^2/D^4\ [(1+Ke)/64.4+2.86LN^2/D^{4/3}] \qquad (17)$$

where H = head loss, or difference between minimum reservoir water elevation and channel water elevation at the outlet (feet)

Q = required outlet discharge (cusecs)
Ke = conduit entrance loss co-efficient (see Table 9)
L = length of the outlet conduit (ft)
D = inside dimension (width or height) of the conduit (ft)
N = Manning's roughness coefficient (see Table 5)

If the outlet is round, then the equation becomes

$$H = 1.62Q^2/D^4\,[(1 \quad Ke)/64.4+2.86LN^2/D^{4/3}]$$

where D = pipe diameter and other symbols are the same as in equation 17 above.

By substituting the necessary coefficients and parameters into these equations, the required pipe size can be solved by trial and error. The top of the outlet pipe must be at or below the downstream water surface elevation.

A WORD ON ENGINES, PUMPS AND EFFICIENCY

There is a general concern in Bangladesh that increasing irrigation will ultimately deplete the water resources. This is a legitimate concern but in such a water-rich country, energy for lifting water is an issue of equal or greater importance. Performance of pumps and wells is generally evaluated in terms of command area. An equally important parameter is food production per unit of energy input, which is only partially reflected in command area. We have discussed distribution efficiency, which directly relates to energy efficiency. It is equally important to design and maintain pumping units to give maximum discharge per unit of energy input. This involves proper matching of engines and pumps with discharge and head, as well as routine maintenance to assure that the unit is operating at optimum speed.

For a given lift there is one unique speed at which a pump and engine will deliver maximum water for a unit of fuel input. Tubewells and low lift pumps are set for average conditions when commissioned, but seldom optimized for specific sites or periodically adjusted. In order to optimize the efficiency of a pumping unit, simultaneous measurements of discharge, fuel consumption rate and speed can be made. Discharge of water per gallon of diesel is calculated and plotted against pump speed to find the speed which gives peak efficiency. Water level should also be recorded to assess total dynamic head. Discharge can be measured with an appropriate flume or weir. Pump speed is measured with a techometer or revolution counter and stopwatch. Fuel consumption rate can be measured using a removable graduated pipette fitted in the fuel line, with a tee between the fuel tank and fuel filter. A valve must be in line between the tank and pipette in order to isolate the tank during measurement. Data can be taken during normal irrigation without interruption. Once the site-specific optimum speed is determind, the pump should be periodically adjusted to assure peak performance. The improvement in efficiency should be noted and calculated into annual fuel savings in order to convince the users of the importance of such adjustments.

There has been a dramatic rise in the number of small shallow tubewell sets used in Bangladesh. The priming system that has evolved in these units is to weld a socket into the elbow between the tubewell and pump suction pipe and attach a standard No. 6 cast iron pump. The system is checked at the power pump discharge pipe by closing with a suitable flap (or farmers' belly), and the system is filled up to the impeller with the No. 6 priming pump. Invariably, the checks and seals in the priming pump become worn and leak some air back into the suction side and impeller of the power pump, once the system is operational. The entrained air reduces pump efficiency but is usually not recognized as a problem until flow is greatly reduced or the system loses its prime. Also, the No. 6 pump is an expensive item to be used only in occasional priming of a power pump.

Virtually all power pumps are supplied with a plug built into the discharge side for the purpose of fitting a suitable priming pump. In this way, there is no chance of air leaking to the impeller and reducing efficiency. The Joint Caritas CRS Irrigation Project in Mirpur is currently designing a low cost priming pump to be fitted to the discharge side of STW's which will eliminate the above problem, as well as be significantly cheaper than the priming method currently used.

APPENDIX

Example Designs

Problem 1.

A farmer group in Dacca District wants a distribution system for their low-lift pump project. The system will be used for Boro rice irrigation. The water source is a canal leading from a perennial river.

Planning and Design : The source is found to be sufficient, with a minimum discharge of 55 cusecs. At low flow during March, the lift from the water surface to the bank crest is about 12 feet. The pump is tested/and found in good condition, giving a discharge of 1.8 cusecs at the design speed and lift conditions at the site.

The soil in the area is a swelling silty clay. Infiltration tests made on puddled paddies indicate an average infiltration rate of about 1 inch per day, which is too high for continuously flooded rice. A system will be designed to provide frequent light irrigations to keep the soil near field capacity.

Crop Water Requirement : The peak demand for Boro rice is in March, and from Tables 1 and 2 ETP and KC are found to be **6.8** and **1.25** respectively. Dependable rainfall is ignored due to the short irrigation interval that will be used. The design daily crop water requirement is (6.8) (1.25)/(31) = **0.275 in/day.**

Potential Command Area : Assuming an efficiency of 75%, a design discharge of 1.8 cusec, and 20 hr/day operation, the design command area is: (1 Acre-inch/hr/cusec)(1.8 cusec)(20 hr/day)(1 day/0.275 in)(0.75) = **98 Acres.**

Irrigation Blocks : The command area can be divided into 20 blocks of approximately 5 acres each.

Mapping : An area of about 200 acres in mapped at the project site for helping to locate the command area and block boundaries. Part of the map is shown in Fig. 12. By using the map and working with the farmers the command area and block boundaries are defined on the map. A permanent bench mark should be placed at the site for future reference, and if practical, referenced to PWD data.

Design of Field Outlets and Channels : The system will use masonry lined rectangular hydraulic sections from the pump to field outlets (primary channels) and trapezoidal earth channels within the blocks (secondary channels). The clay soil is stable, and side slopes can be 1:1 (horizontal : vertical). For earth channels, the bottom width will be equal to the depth to keep right-of-way from being excessive.

Blocks are designated as B1, B2. ... etc. Field outlets are designated 01, 02, ... etc. Secondary channels are designated by the block number followed by the letter sequence of the channel from the outlet. For example, S1A is the secondary channel leaving the field outlet in B1. S1B is the first secondary branching from SIA. Primary channels are designated by the numbers of the field outlets they connect. For example P12 is the primary channel between 01 and 02. Division boxes are designated D1, D2 ... etc. All channels and water control structures will be designed with a freeboard of 0.5 feet.

In order to standardize check gates all outlet and water control structures (Fig. 13) will be built with rectangular outlets having a width of 1.5 ft. The inlets will be of the same section and elevation as the primary channel. Corners will be rounded to reduce turbulence. Outlets into primary channels with a larger section will expand at a 1:4 ratio. Details are shown on the following plans.

Outlets from secondary channels into individual plots must carry the full discharge of 1.8 cusecs. If this is done by cutting the earth channel bank, excessive erosion and scouring will damage the field and channel. An appropriate outlet can be made with several jute gunny bags filled with sand or earth (Fig. 14). The bags are stacked on top of each other to serve as a length of the channel embankment adjacent to the plot. When water is to be turned into the plot one or two bags are removed from the embankment and placed in the channel just downstream of the outlet to check and divert the water into the plot. The bottom row of bags serve as an outlet crest and apron and should be placed with their tops at the same elevation as the channel bottom and about 0.5 feet below the field.

The design starts at Block B1, because it is the highest of the blocks at the end of the channels. The controlling plot in the block is 90.0 ft. elevation and 450 feet from the field outlet. The design water surface is 0.5 feet above the plot, or **90.5 feet**.

Since there is very little slope in this block, a flat channel slope of 0.001 is chosen, to minimize the channel height and fill. From Manning's equation the channel dimensions are calculated as follows :

$$Q = (1.49/N)(A) \times (A/P)^{2/3}(S)^{1/2} \qquad (2)$$

$$A = 2Y^2$$

$$P = Y(1 + 2/\sin 45^0)$$

$$Y = 0.781\,(QN/S^{1/2})^{3/8}$$

For N=0.035 (Table 5), Q=1.8 cusec, and S=0.001, the channel depth (Y) is found to be **1.0 ft.**

Channel S1A (secondary in Block 1) will have a depth of 1.0 ft., free-board of 0.5 ft., and slope of 0.001.

The water surface elevation at the block outlet (01) is calculated by the length and slope of the secondary channel as WSEL (01) = 90.0 + 0.5 + 0.001 (450) = **91.0 ft.** Since there is no fall at the drop structure into block B1, the apron is set at the channel bottom elevation, or 91.0—1.0 = **90.0 ft.**

The water surface elevation at the inlet of secondary channel S1B is calculated using the slope and distance in S1A as 91.0—0.001 (130) = **90.9 ft.** The water surface elevation at the outlet of S1B is 90.2 + 0.5 = **90.7 ft.**

The slope of SB1 is given by the fall divided by the length, or S = (90.9—90.7)/150 = **0.001 ft/ft.** Therefore the depth (Y) is again 1.0 ft.

Block B2 has a large slope and the water surface elevations will be controlled by the highest block adjacent to the outlet (elev.87.8). The water surface at this block (and tailwater elevation at the outlet structure) is equal to the plot elevation plus 0.5 ft. (87.8 + 0.5 = **88.3 ft.).** The slope of secondary channel S2A is calculated as the fall from the high to the low end divided by the channel length, or, S = (87.8—85.5)/330 = **0.007 ft/ft.**

Using equation 2 the channel depth (Y) is calculated to be **0.7 ft.** Using Equation 3, the velocity is calculated to be **1.8 ft/sec**, which is permissable, as shown in Table 6. If the velocity was too high, the slope should be reduced accordingly, and S2A channel provided with an intermediate drop structure.

The fall (H) from the outlet structure into Block 2 is given by 91.0–88.3=**2.7 ft.** The outlet discharge per foot of width is given by 1.8/1.5=**1.2 ft^2/sec.** From Figure 1 it is seen that the apron depth (Y) is **1.1 ft**. The apron elevation is given by 88.3—1.1=**87.2 ft.**, or 0.4 ft. below the channel bottom. The apron length is given by 4 (Y), or **4.4 ft.**

The water surface elevation at 02 is determined by the required elevation in the respective blocks (B3, B4) or the downstream channel, whichever is higher. The controlling plot elevation in B3 and B4 appears to be 90.0 ft. at the end of S3B The length of secondary channel from the controlling plot to 02 is 430 ′.. Assuming a slope of 0.001 in S3B and S3A, the required water surface elevation at 02 is 90.0+0.5+430 (0.001) =90.9 ft. However, the required water surface at 01 is 91.0 ft., so the downstream elevation on the primary channel is controlling. In order to minimize fill, a relatively flat slope of 0.0005 is chosen for P12. The length of P12 is 610 ft. Therefore, the required water surface elevation at 02 is 91.0 +610 (0.0005)=**91.3 ft.** From Figure 2, the design depth (Y) of P12 is **0.9 ft.** and the width is 0 9 x 2=**1.8 ft.**

Now the secondary channels in B3 and B4 are designed (Fig. 16) using the plot elevations, slopes, and water surface at 02 in a similar fashion to B1 and B2. This process is continued up the channel to the inlet, and then repeated on the other branch.

Underdrains (Fig. 15): If distribution channels block drainage of surface runoff, drainpipes must be provided under the channel. Otherwise, crops may be damaged from flooding, or the channel may be washed out or damaged from cross flow.

It is convenient to make a list or table of the pertinent dimensions of all structures and canals. as well as drawings and profiles. A list for the example designs follows :

OUTLET STRUCTURES

Structure	Water Surface Elevation	Width of Inlet	Bottom Elevation	Left Apron Elevation	Right Apron Elevation
01	91.0	1.8	90.1	87.2	90.0
02	91.3				
03					
04					

SECONDARY CHANNELS

Channel	Depth & Bottom Width	Length	Slope	Beginning Bottom Elevation
S1A	1.0	450	0.001	90.0
S1B	1.0	150	0.001	89.9
S1C				
S2A	0.7	330	0.007	87.6
S2B				

PRIMARY CHANNELS

Channel	Depth	Bottom Width	Length	Slope	Beginning Bottom Elevation	Ending Bottom Elevation
P12	0.9	1.8	610	0.0005	90.4	90.1
P23						

Operation and scheduling: The system can be operated on a 5-day rotation period. For the twenty blocks, this allows 5 hours of water use for each block every 5 days. During cooler or rainy periods, the use can be reduced, and for periods of excessive use, such as land preparation, the blocks can use water up to 6 hours.

Within the blocks the system is designed to give the full flow of 1.8 cusecs to each plot. The time allotted to each plot is determined by the percentage of the block area in the plot. For example, if the block area is 5.0 acres, and the plot is 0.30 acres, then the irrigation time for the plot is given by (300 min) (0.3/5) = **18 minutes**. The depth of water will be about the same for each plot, and is calculated by:

(1.8 Acre — in/hr) (18.0 min) (1hr/60min) (1/0.3 Acre) = **1.8 inches**.

Irrigation scheduling begins at the most remote plots of the most remote blocks. This helps reduce the inequity of the channel losses felt by the end users.

Problem 2:

Design a storage-cum-fish culture reservoir for the same project:

The channel and operating schedule for the system are designed to cover 4 blocks each day, delivering 1.8 cusecs to each block for 5 hours. The reservoir and outlets must be designed to serve the same number of blocks, using the channels designed for 1.8 cusecs. This can be done by taking water out of the reservoir into each of the two primary branches and irrigating two blocks on each branch each day. The daily hours of irrigation can be cut from 20 to 10 by storing the additional pumped water in the reservoir. The required reservoir storage volume is calculated using Equation 15.

$$V = 3600(Q)\ (I - 10) \qquad (15)$$
$$V = 3600\ (1.8)\ (20 - 10)$$
$$V = 64{,}800 \text{ cubic feet}$$

Assuming 2 feet of usable storage, the reservoir water surface area is given by Equation 16.

$$A = V/D$$
$$A = 64{,}800/2$$
$$A = 32{,}400 \text{ square feet, or } 0.74 \text{ acres}$$

If the reservoir is square, the water surface will be 180 feet on each side.

Design of Parshall Flume: A Parshall flume will be used at the entrance of each of the two primary channel branches to measure and regulate the flow. The three primary design parameters of the flumes are: a) size, b) elevation, and c) head loss.

The flume size can be taken from Table 11. Since a 6-inch flume measures to 2.5 cusecs, it will be adequate.

To assure that the flume is free-flowing, it must be set high enough above the tailwater elevation to retain critical depth in the throat: otherwise, the tailwater will submerge the throat and the discharge-head relation will no longer be valid. The design water surface elevation on the primary channel entrance at the reservoir is found to be **92.5 feet.** From Table 11, the maximum submergence (S) is found to be **0.56.** The head on a 6-inch flume discharging 1.8 cusecs is found from Figure 9 to be **0.9 feet**. Equation 9 is used to calculate the flume elevation.

$$S = (H-Y)/H \qquad (9)$$
$$0.56 = (0.9-Y)/0.9$$
$$Y = 0.4 \text{ ft.}$$

In order to assure free discharge over the entire range of the flume, a safety factor of about 0.2 ft. is added to the flume elevation and head loss. Therefore, since the tailwater elevation is 92.5, the head (H) is 0.9 and the head loss (Y) is 0.6, the flume crest elevation is $92.5+0.6-0.9=92.2$ feet. The design headwater elevation on the flume (tailwater elevation at the reservoir outlet) is $92.2+0.9=93.1$ feet.

The flume on the other primary channel is designed in a similar fashion. The reservoir outlet must be fitted with an appropriate stilling basin and approach channel to the flume entrance to assure relatively tranquil flow. The approach channel should be the same cross section as the flume entrance, with a length about ten times the entrance width (D).

Design of Reservoir Outlet (Fig. 17) : The outlet will consist of a square box culvert fitted with two vertical lift gates at the outlet to each primary channel. The gates should be mounted in a common riser pipe for access and servicing. The total head loss in the outlet structure (culvert and gates) determines the design water surface elevation in the reservoir. The gates and culvert must be designed to give the required discharge when the pool is at the minimum storage level.

Let us assume a head loss through the gates of **0.1 feet** and through the culvert of **0.9 feet.** The minimum design water surface elevation is the tailwater elevation plus outlet head loss, or $93.1+0.1+0.9=$**94.1 feet**. The useable reservoir storage is 2 feet, making the maximum water surface elevation $94.1+2=$**96.1 feet**.

The outlet lift gate size is calculated using Equation 14, with K about 0.65.

$$Q=KA(2gH)^{0.5} \qquad (14)$$

$$1.8=0.65(A)\,[(2)(32.2)(0.1)]^{0.5}$$

$$A=1.1 \text{ square feet}$$

A 1 x 1 foot square gate will be sufficient. The gate should be set below the design tailwater, so the gate invert should be at or below $93.1-1.0=92.1$ feet. Care must be taken in design, fabrication and installation of the gates to assure a good seal during closure.

The culvert must carry the flow for both primary channels. or **3.6 cusecs.** the culvert size is designed solving Equation 17 by trial and error. Assuming a reservoir embankment with 6 foot top width, 2:1 side slopes and 3 foot freeboard, the required culvert length (L) will be about **50 feet**. The roughness coefficient (N) from Table 5 is **0.015**. The entrance coefficient (Ke) from Table 9 is **0.5.**

$$H=Q^2/D^4[(1+Ke)/64.4+2.86\ LN^2/D^{4/3})] \qquad (17)$$

$$0.9=(3.6)^2/D^4[(1+0.5)/64.4+2.86(50)(0.015)^2/D^{4/3})]$$

$$D^4-0.4633D^{-4/3}-0.3354=0$$

$$D=0.95 \text{ Foot}$$

A one-foot square culvert can be used. The culvert must be fitted with an appropriate screen on the entrance to prohibit fish or debris from entering. The screen should have enough open area to assure negligible head loss.

Problem 3

You are to design a buried pipe distribution system for a tubewell irrigating wheat in Bogra.

The soil is well-drained loam and the infiltration rate is found to be about 1 inch/hour for an application of 3 inches. The area is mapped to show plots and plot elevations.

Pump Test : The pipe system will require a head of about 10 feet, so the pump discharge is fitted with a temporary valve, manometer and flow meter for adjusting the head and measuring discharge. The diesel engine is fitted with a pipette for measuring fuel consumption rate. The pump speed is varied, holding a 10-foot head on the discharge pipe and measuring fuel consumption rate and discharge. This data is reduced to Acre-feet of water per gallon of diesel and plotted against engine speed in order to find the most efficient rpm. It is found that the optimum is **1400 rpm** at a discharge of **1.5 cusecs.**

Potential Command Area: The system will be designed to operate 20 hr/day during peak demand and is assumed to have an overall efficiency of 70%. The peak water requirement for wheat occurs in late January. ETP is estimated from Table 1 as **4.0 inches** and KC from Table 2 as **1.1**. The crop water requirement (KC)(ETP) is **4.4 inches**. The potential command area is given by:

(1 Acre-inch/hr/cusec)(1.5 cusec)(20 hr/day)(31 day/4.4 in)(0.7) = 148 acres.

We can assume a potential command area of 150 acres, or 30 blocks of about 5 acres each. The command area and blocks are drawn and adjusted on the map. The blocks are in an approximate 5×6 grid, and risers are placed between blocks so that two blocks can be served by each riser. Three pipelines serve the risers as shown on the map (Fig. 18), and the pump and well are placed at the junction of the three pipelines. Blocks are numbered B1, B2,...etc., and risers are numbered R1, R2,...etc. Distribution with the blocks is by earth channels, which are designed in the same fashion as in Problem 1.

Pipeline design: Cement-asbestos pipe will be used. Available sizes are 6', 8', and 10-inch. The branch line from the pump to R6 is the longest and serves the highest plot, and will be designed first. The plot in the southeast corner has an elevation of 93.2 feet. The field channel leading to this plot is 330 feet long. Assuming a channel slope of 0.001, and a water surface 0.5 feet above the plot level, the required water surface at R6 is given by 93.2+0.5+(330)(0.001) **=94.0 feet.**

The length of line from the pump to R6 is 2620 feet. The total friction loss in the line is the pipe friction plus head loss flowing through the risers. Head loss of water in the pipes will be calculated with Equation 5 or Table 7, and head loss of water flowing past and through the risers will be estimated with Equation 6. The roughness coefficient (C) for Equation 5 is 120. The coefficient (K) for Equation 6 will be 0.2 for water flowing past a riser, 1.0 for water flowing out of a riser, and 0.5 for the pipeline entrance loss. From Table 7 it can be seen that a 10-inch pipe is necessary to keep the head near or below 10 feet.

The pipeline loss estimated from Table 7 is 3.45(2620)/(1000) = **9.0 feet**. The pipeline velocity is given by Equation 3:

$$Q = AV \qquad (3)$$

$$1.5 = (3.14)(5/12)^2(V)$$

$$\mathbf{V = 2.75 ft/sec.}$$

$$\mathbf{V^2 = 7.56 ft^2/sec^2}$$

The total head loss through fittings and risers is given by:

$$H = KV^2/2g \qquad (5)$$

$$H = (0.5 + 4(0.2) + 1.0)(7.56)/64.4 = 0.3 \text{ feet}$$

The total loss in the line from the pump to R6 is then **9.3 feet**. The required water surface elevation at the pump standpipe is 94.0+9.3=**103.3 feet PWD**. The standpipe should have a freeboard of 2 feet, making the top crest 105.3 feet.

The riser outlet crests for each block should be set at the same elevation as the channel bottom. From Table 10 it can be seen that 8-inch valves are barely adequate to pass the design discharge of 1.5 cusecs.

Pipeline looping: If additional pipe is added to connect R6, R7 and R8, water can flow through the additional branches and possibly smaller pipe can be used. This option will be tried, using 8-inch pipe in all the branch lines and connectors. The method of equivalent pipe will be used to estimate the total head loss and required head in the pump standpipe. The first step is to determine an equivalent pipe to replace the west and central pipe branches connecting at R7. An arbitrary head loss in the two lines of 10 feet is assumed.

Using this head loss, discharge in each pipe is calculated. The sum of these two discharges is then used to calculate an equivalent pipe which would carry the same discharge at 10 feet of head. Using equation 5, the discharge in the west pipe is calculated. The length is 3,450 feet, "C" is 120, and "D" is 8.0.

$$H = 841 \times 10^6 (Q/C)^{1.85} (D)^{-4.87}$$

$$(10)/(3.45) = 841 \times 10^6 (Q/120)^{1.85} (8)^{-4.87} \qquad (5)$$

$Q = \mathbf{0.762}$ **cusec.**

Note that fitting losses are neglected, as they are small, and make calculations more cumbersome.

The center branch is 1830 feet, and using the same technique the discharge is calculated as **1.074 cusecs**. The total flow for the equivalent pipe is then 1.074+0.762=**1.836 cusecs**.

Using Equation 5, the equivalent length of an 8-inch pipe passing 1.836 cusecs is calculated as **679 feet**. In other words, if the west and center pipe branches are replaced with an 8-inch pipe 679 feet long, the hydraulics will be equivalent.

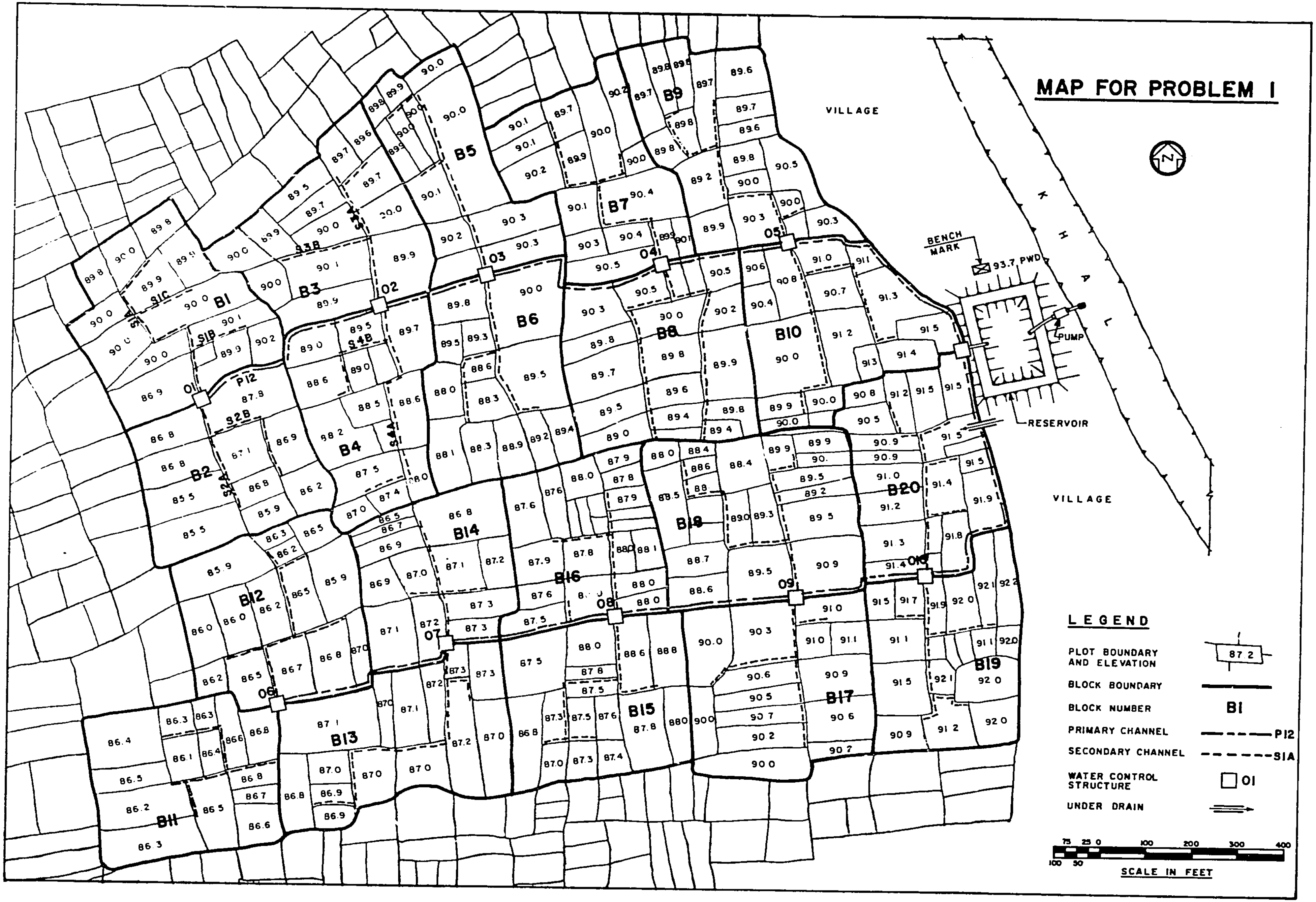
The loop connector between R7 and R8 is 1.000 feet. This, added to the 679-foot equivalent pipe gives two 8-inch branches leading from the pump to R6, with lengths of 1679 and 2620 feet. These two pipes are now replaced by a final equivalent pipe from the pump to R6, by the same technique as above. For the total network, an equivalent pipe of 8-inch diameter, and **574 feet** length is found. From Table 7 the head loss with 1.5 cusec discharge is found to be **5.9 feet**. By adding 1,875 feet of pipe into the system (the two loop connectors), the pipe size can be reduced to 8-inches and the head on the pump and standpipe is reduced by three feet. When selecting pipe size it is important to consider pumping energy cost as well as pipe cost in the economics. Smaller pipes cost less, but increase the system operating cost by increasing head and the required fuel or electricity for pumping.

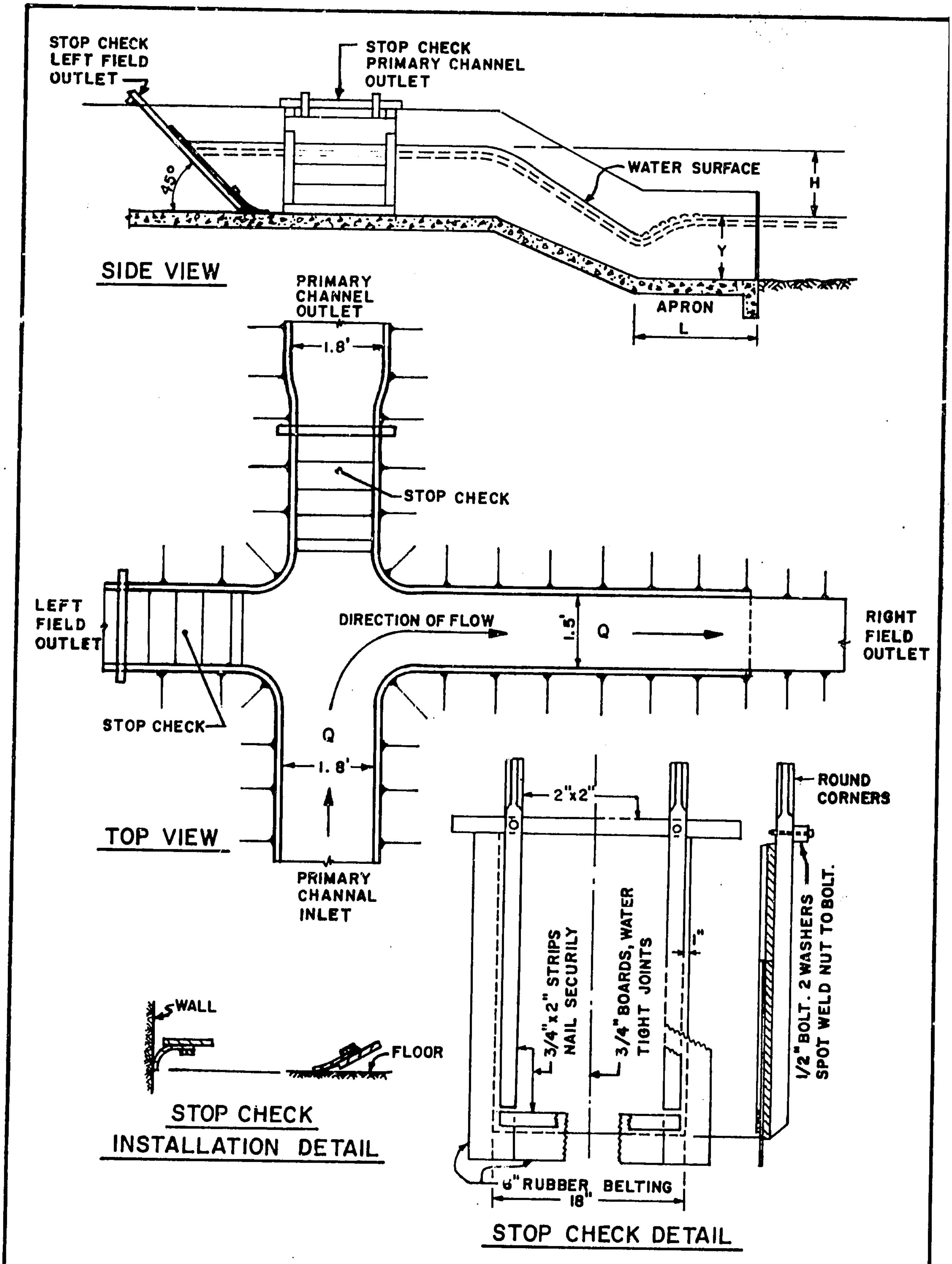
Operation and Scheduling: There are 30 blocks in the system. If 2 blocks are irrigated daily, the command area can be covered in 15 days. During peak demand, the system can be operated to supply water to each block for as much as 12 hours if necessary. The delivery to a 5-acre block in 12 hours is (1.5 acre-in/hr) (12 hr) (5.0 acre)=**3.6 inches**.

If practical, it would be advantageous to stagger the sowing of the wheat in the same manner as the 15-day irrigation rotation period. In other words, plant two blocks per day beginning in early or mid-November. The soil should have enough residual moisture left from the monsoon rains to germinate the wheat. It required, a light irrigation (about one inch or less) can be applied prior to planting. The wheat should be irrigated 15 to 20 days after planting if sufficient rain has not fallen. The required depth of irrigation at this time should be light, because, again, there should be enough residual moisture stored in the effective root zone to supply a good portion of the crop requirement as the roots penetrate.

The last irrigation should occur about 40-60 days after planting, during the milk stage of head growth. This is the period of maximum water requirement and irrigation. The effective root zone depth of the project site is 3 feet, with available moisture of 2 inches per foot. Assuming the last irrigation supplies 50% of the available moisture (3 inches), each block will receive water for the design 10 hours.

When irrigating wheat, over-watering can be more detrimental than under-watering. Plots should be level to avoid ponding of water in low spots. The application depth should never be so great that the water does not all infiltrate within two or three hours. Otherwise, the system should be operated with more frequent, lighter applications. For example, the above system could be operated to cover three or four blocks each day with smaller depths of irrigation.


MAP FOR PROBLEM I
VILLAGE
VILLAGE
KHAL
BENCH MARK
93.7 PWD
PUMP
RESERVOIR
B1
B2
B3
B4
B5
B6
B7
B8
B9
B10
B11
B12
B13
B14
B15
B16
B17
B18
B19
B20
LEGEND
PLOT BOUNDARY AND ELEVATION
87.2
BLOCK BOUNDARY
BLOCK NUMBER
B1
PRIMARY CHANNEL
P12
SECONDARY CHANNEL
S1A
WATER CONTROL STRUCTURE
01
UNDER DRAIN
SCALE IN FEET



WATER CONTROL STRUCTURE

FIGURE - 13

FIELD OUTLET OF EARTH FILLED JUTE GUNNY BAGS

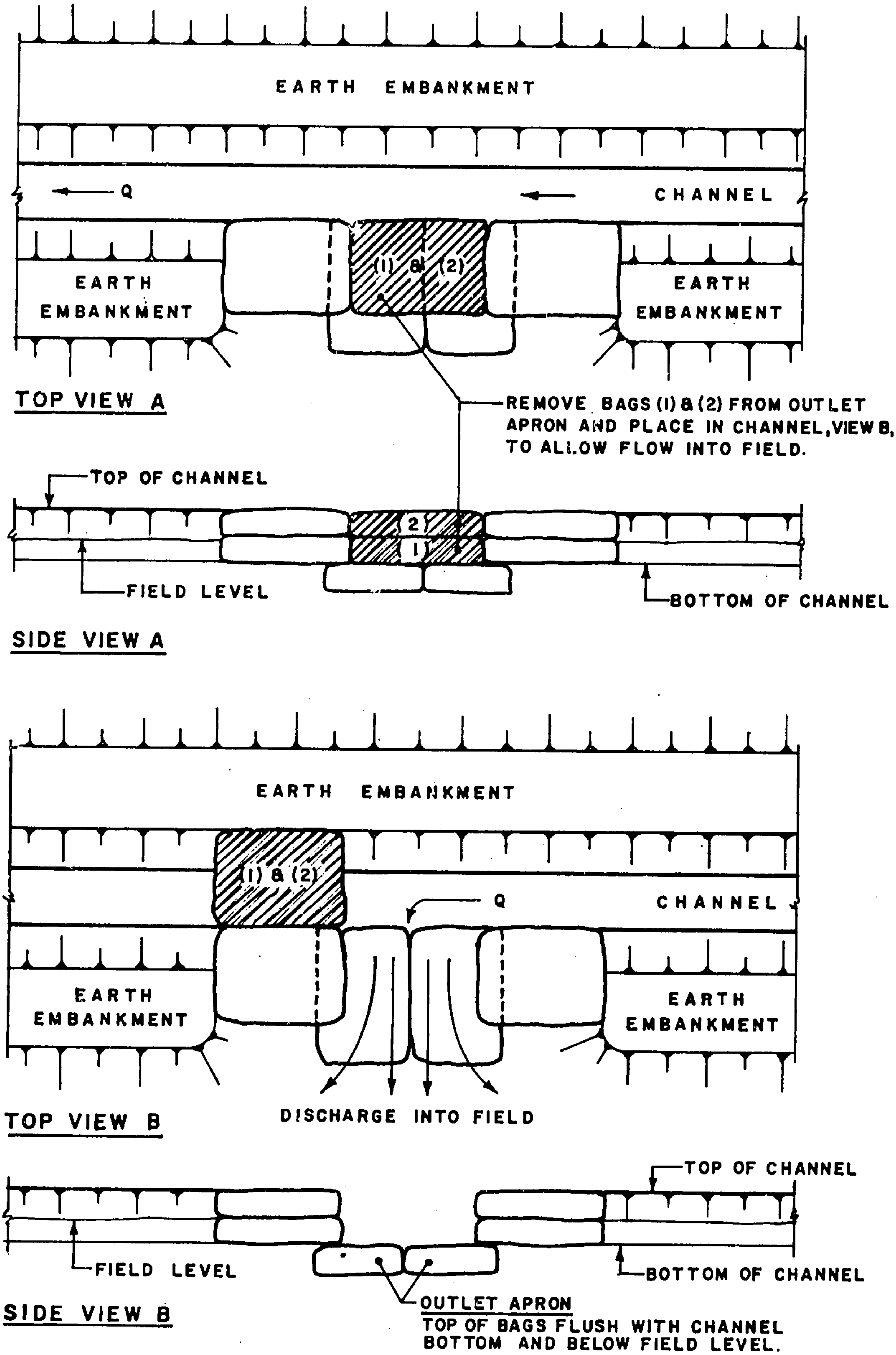
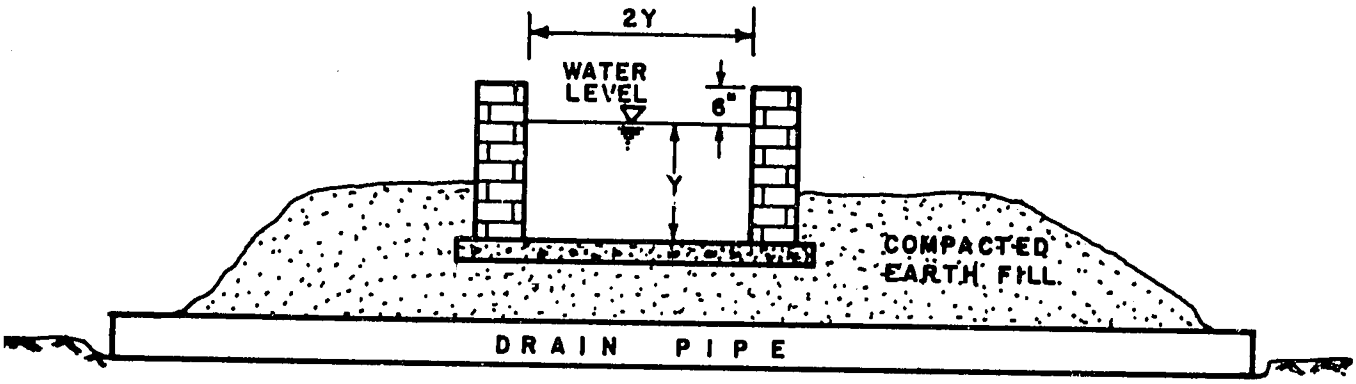
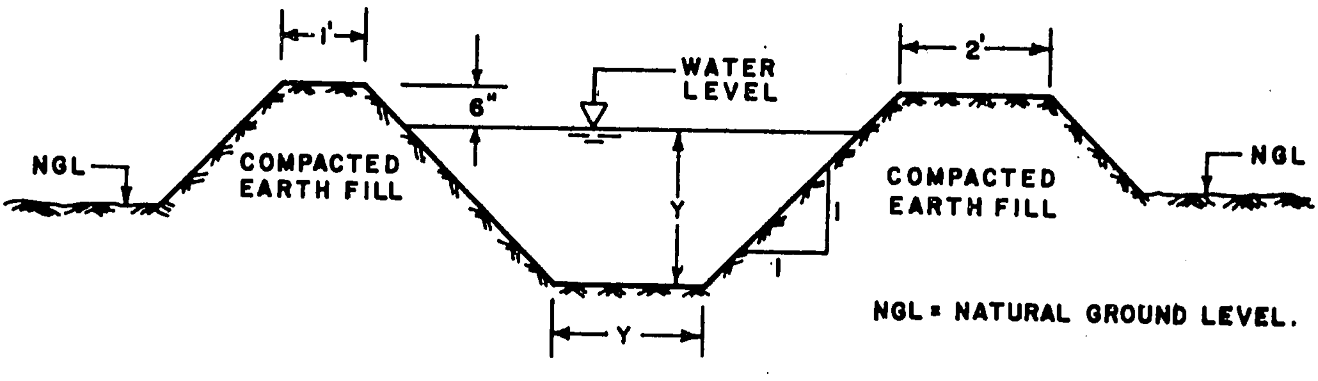


FIGURE-14



PRIMARY CHANNEL AND UNDERDRAIN

FIGURE - 15



SECONDARY CHANNEL

FIGURE - 16

STORAGE RESERVOIR OUTLET

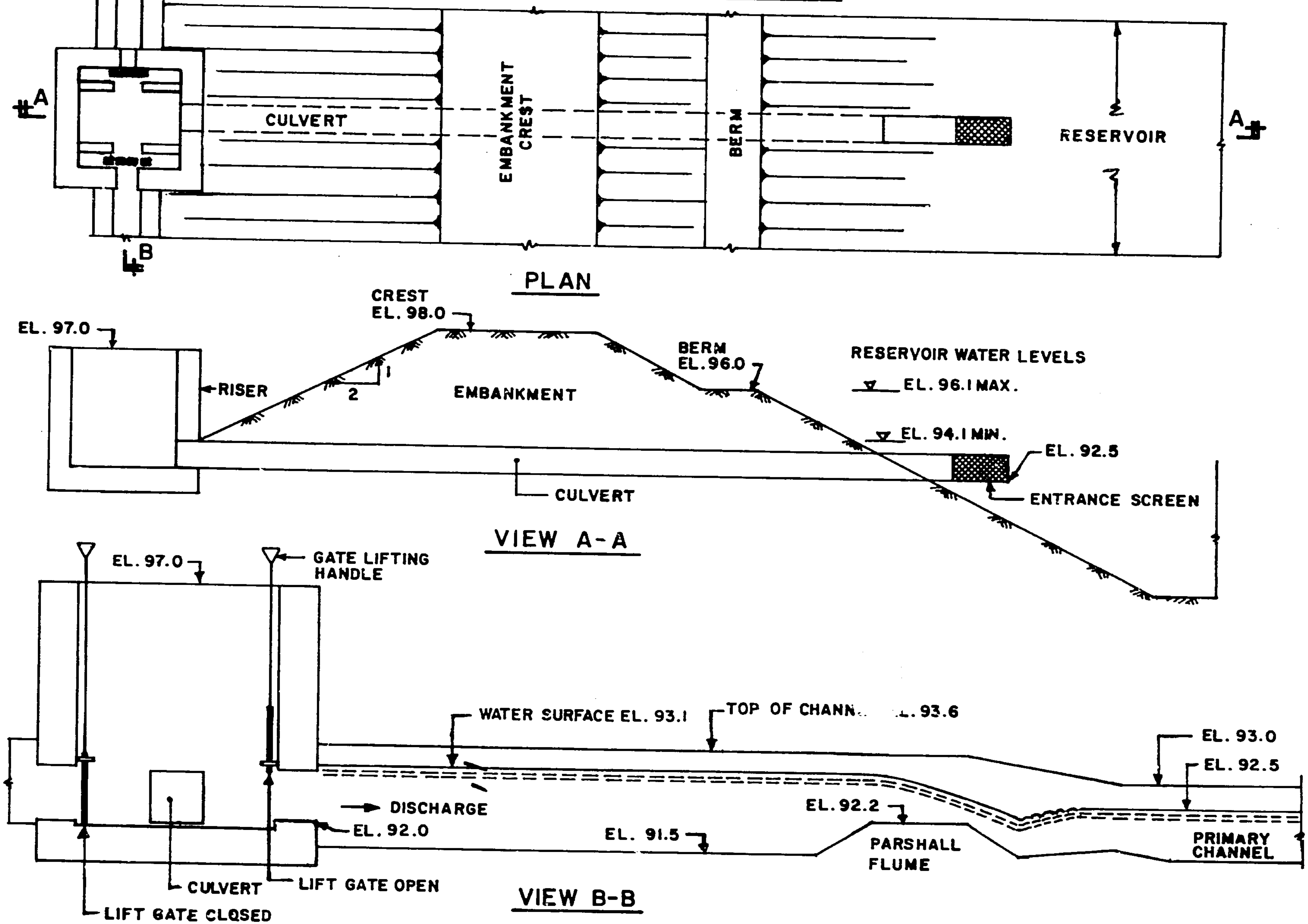
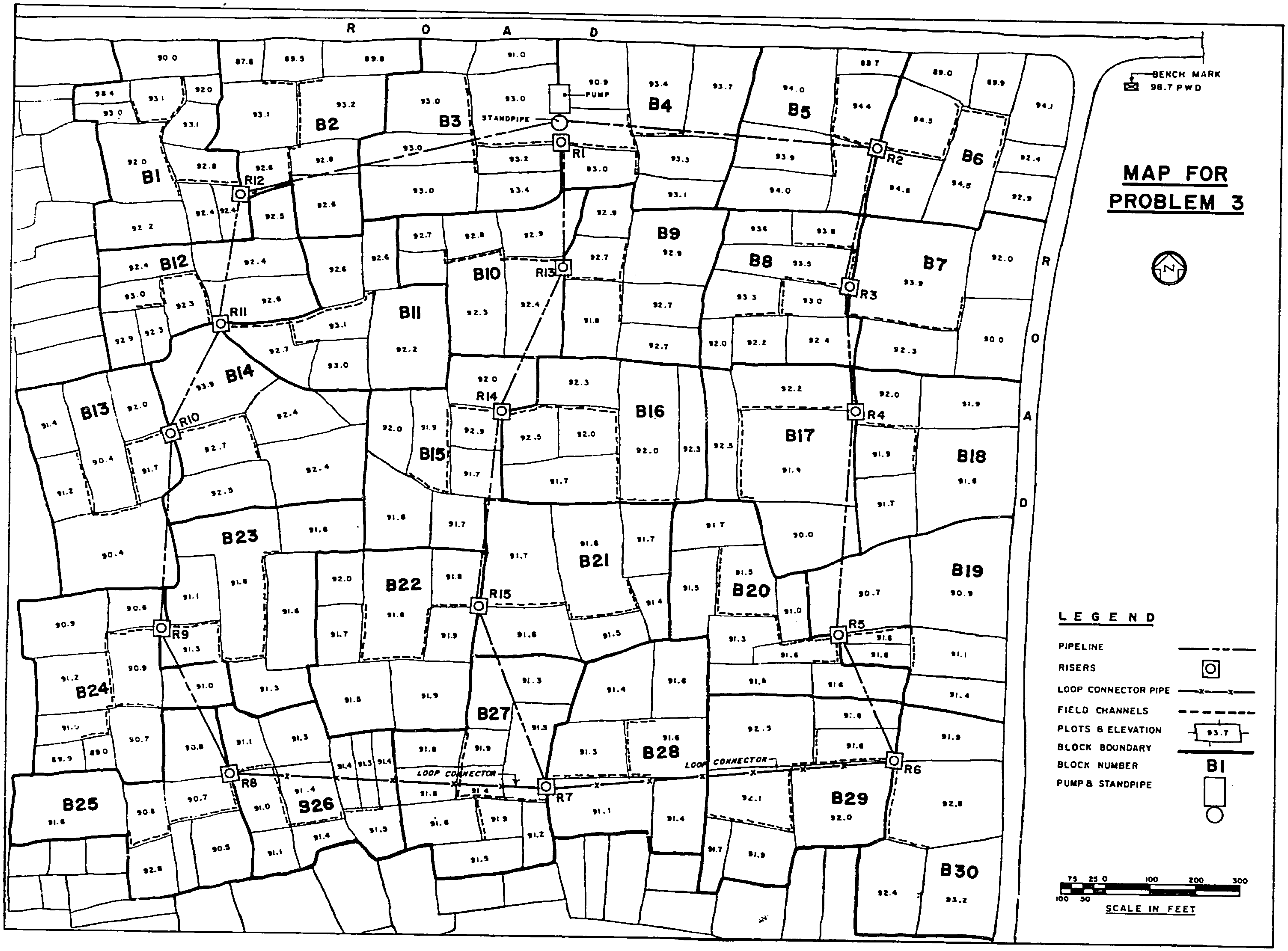


FIGURE - 17


MAP FOR PROBLEM 3
ROAD
ROAD
BENCH MARK
98.7 PWD
PUMP
STANDPIPE
N
B1
B2
B3
B4
B5
B6
B7
B8
B9
B10
B11
B12
B13
B14
B15
B16
B17
B18
B19
B20
B21
B22
B23
B24
B25
B26
B27
B28
B29
B30
R1
R2
R3
R4
R5
R6
R7
R8
R9
R10
R11
R12
R13
R14
R15
LOOP CONNECTOR
LOOP CONNECTOR
LEGEND
PIPELINE
RISERS
LOOP CONNECTOR PIPE
FIELD CHANNELS
PLOTS & ELEVATION
93.7
BLOCK BOUNDARY
BLOCK NUMBER
B1
PUMP & STANDPIPE
75 25 0 100 200 300
100 50
SCALE IN FEET

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