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SUSTAINABLE USE OF WATER RESORUCES AND IRRIGATION SYSTEMS DESIGN

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AND
DEPARTMENT OF WATER MANAGEMENT, NWFP
AGRICULTURAL UNIVERSITY PESHAWAR - PAKISTAN

MARCH, 2004

FOREWARD

This book is second in series of Training Manuals produced by the International Arid Lands Consortium (IALC) and University of Illinois at Urbana Champaign (UIUC) under the Human Resource Development Project of Afghanistan. The first Training Manual entitled "Irrigation Infrastructure Development and Water Management Practices" was published in Pushtu language, which, was based on the training course, conducted by the IALC/UIUC at the NWFP Agricultural University (NWFP AU) Peshawar in August 2003. This Manual entitled "SUSTAINABLE USE OF WATER RESOURCES AND IRRIGATION SYSTEM DESIGN" is the output of training course conducted by the IALC/UIUC at NWFPAU during February and March 2004, which was sponsored by the United States Agency for International Development (USAID). Twenty-one participants from Afghanistan attended this course representing Ministries of Irrigation, Water Resources and Environment and Agriculture and Animal Husbandry and Non-Governmental Organizations engaged in agricultural activities in Afghanistan.

This Manual discusses the irrigation and water management problems and solution thereof and is organized in nineteen sections. The first section highlights the importance of irrigation in Afghanistan, the second describes climate and water resources of Afghanistan and the third deals with water quality for irrigation, the fourth section presents the guidelines for irrigation schemes development, fifth and sixth sections deal with crop water needs and soil and water. The seventh section deals with flow measurements and eighth with irrigation canal design. The methods of peak run off assessment have been described in section nine. Hydraulic structures and water distribution have been described in sections ten and eleven respectively. Section twelve describes the surface irrigation methods. Thirteen and fourteen sections deal with sprinkle and trickle irrigation systems respectively. Ground water for irrigation has been described in section fifteen. Section sixteen deals with design of dams and rivers flow; guide banks and spurs are discussed in section seventeen. Salinity and water logging have been discussed in section eighteen. The last section deals with canal maintenance.

We hope that this book will be useful for the field engineers and agriculturalists as Master Trainers to enhance their practical knowledge in the field of irrigation and sustainable use of water. The competition for water is increasing day by day in arid and semi-arid regions and it is imperative that we all manage to make best use of available water.

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ACKNOWLEDGEMENTS

The publication of this training course material would not have been possible without a great deal of assistance from the teaching and support staff of the Department of Water Management, NWFP Agricultural University, Peshawar. Significant contribution in the production of this training material was made by Prof. Dr. M. Jamal Khan, besides that Mr. Muhammad Zubair, Dr. Tahir Sarwar, Engr. Javaid A. Tariq, and Engr. Nisar Ahmad also contributed in its preparation; their valuable contributions and assistance in this regard are highly appreciated. Mr. Tahir Shah and Mr. Tauheed Ali helped in the editing and binding of this manuscript; their efforts and keen interest are also highly appreciated and acknowledged.

Financial support for the production of the training material was provided by the University of Illinois at Urbana Champaign, United States Agency for International Development (USAID), USA and International Arid Lands Consortium (IALC), which is highly appreciated. The authors are very thankful to Prof Dr. Syed Iqbal Shah, Vice Chancellor, NWFP Agricultural University, Peshawar for providing all the facilities on the campus for the successful conduction of this course, who also participated in the closing ceremony of the course and exchanged his ideas with participants. Dr. Abdul Qayyum Khan, Director, University of Illinois Field office has maintained a desire for its publication and provided assistance to the Department of Water Management, his help and cooperation in this regard is also highly appreciated.

Prof. Dr. Muhammad Jamal Khan

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Medium-scale traditional (informal) surface water systems being supplied by river flow diverted with the help of temporary brush weirs. Defined as a separate category, these systems are characterized by size (100 ha to 2000 ha) and by the fact that only a single village community is using irrigation water through a single intake. The schemes are operated and maintained in a similar way as other small-scale and large-scale irrigation systems.

Large-scale traditional (informal) surface water systems being supplied by river flow diverted with the help of temporary brush weir. Extending over an area of up to 2,00,000 ha (Hish-Dah-e Nahr scheme at Balkh), they are mainly located in flat plains and along the main valleys. Although they are called informal, their operation and maintenance was highly structured, involving several communities of sometimes different ethnic origins. Each village has at least one water master (mirab), who delegates his authority to sub water masters responsible for the allocation of water to different plots of the schemes. Lawyers (vakil) support the mirabs in disputes over water rights and provide the linkages to governments authorities for the registration of land and water rights. Repair and maintenance works are executed by mobilizing large gangs of labour for a long period, and farmers in the command area have to contribute in labour, cash or kind. Historically, large parts of these schemes have been abandoned because of the impacts of wars, water logging and salinization, particularly in the Harrud, Farah Rud, Balkhab, Murghab and Helmand valleys.

Modern irrigation Systems

Modern (formal) surface water systems without storage have a permanent intake structure which is operated and maintained by the Irrigation Department of the Ministry of Water and Power. The management of the irrigation scheme itself follows the rules of the large-scale traditional surface water schemes described above. However, the significant difference is that the regulation of water flow to the system depends on a functioning interaction between government authorities and the village communities.

Modern (formal) surface water system with storage. Organized large-scale irrigation system development is a relatively recent innovation, the most important systems being the Helmand (Kajakai dam) and Arghandab (Arghandab dam) irrigation schemes. Land tenure was different from traditional systems in that some parts of the schemes were operated under private land ownership agreements, while others were operated as State farms owned by the government. The management of these schemes was under complete government control and heavily subsidized leaving the farmers little choice of crop selection or farming practice.

Modern (formal) ground water systems. Very little is known about the few irrigation schemes supplied by ground water from deep and shallow wells. In Khost/Paktia provinces. In the 1970s about 100,000 ha were said to have been under sprinkler

irrigation (private and government owned) and plans existed to introduce drip irrigation.

The overall irrigation efficiency is between 25 to 30 percent for both modern and traditional irrigation schemes due to the following reasons.

- High conveyance losses in traditional schemes with earth canals.
- High operation losses in modern schemes with lined conveyance canals;
- High on-farm distribution losses (over-irrigation, poorly leveled land) in both traditional and modern schemes.

Additionally, there is usually a wastage of irrigation water in the traditional schemes during the first half of the growing season due to unregulated flood water entering the conveyance canal and a shortage of water during the second half when river flow decreases to its annual minimum.

1.1 Problems of Irrigated Agriculture

Absence of watershed management

Irrigated agriculture is usually found in river valleys, on alluvial terraces situated on either or both sides of a river. Irrigation water is usually river water, diverted into canals with the help of temporary intake structures (brush weirs) in informal systems or permanent intake structures in modern systems. Irrigation water may also be groundwater extracted with the help of underground tunnels (Karez systems) or lifted from shallow wells with the help of Persian wheel (arhad). The above-mentioned (primary) irrigation canals generally follow a pathway situated at the upper limit of an area to be irrigated and usually have to cross a number of (torrential) tributaries descending from the surrounding hills or mountains. The irrigation systems are affected by a number of problems:

Sediment accumulation in irrigation canal and Karez systems.

Shifting of main river channel and subsequent uselessness of intake structures

Stream bank erosion and damage to or destruction of intake structures, either due to annual floods or to exceptional floods occurring e.g. once every 5 or 10 years.

Damage to irrigation canals and field due to (torrential) flashfloods, especially due to spring rainstorms, in the numerous tributaries (washes) originating in the hills and mountains on both sides of the valleys.

Low cropping intensities

The northern provinces from Faryab to Takhar, clearly reflect the differences in overall water constraints. In the northwestern provinces (Faryab, Jawzjan, Balkh and Samangan), where rivers are not or hardly fed by the perennial snow in summer, the cropping intensities are lowest (3-9% double cropping, 15-40% single cropping and 48-80% intermittent cropping), whereas in the northeastern province (Kunduz, Baghlan and Takhar) the cropping intensities are highest (36-73% double cropping, 24-34% single cropping and 3-39% intermittent cropping). Kunduz province is by far Afghanistan's best performing province with 73% double cropping, 24% single cropping and only 3% intermittent cropping. Worst off appears to be Nimroz province where water shortages reduce the cropping intensities to almost entirely intermittent cropping.

Low Yield

In the table below yields recorded in 1966 (eastern provinces) and in 1991 are compared.

Table 1.1 Yield of selected crops.

Crop	1966 (kg/ha)	1991 (Kg/ha)
Irrigated wheat	700 – 1400	1400 – 3150
Rainfed wheat	350 – 1400	0 – 1400
Rice	1400 – 2800	1400 – 3500
Maize	1050 – 2100	1050 – 2450
Barley	350 – 1400	0 – 2450

The average (irrigated wheat yield appears to have increased between 1966 and 1991 but hardly any increase is being recorded for the other crops listed.

Salinity

Because of the arid to semi-arid climate prevailing in most of Afghanistan, saline soils may occur n almost any irrigation area. Salinity of the soil horizons in Afghanistan is generally the result of :

- Upward and lateral movement of solutes from groundwater
- Deposition by evaporation of saline runoff waters in fields or in natural catchments basins
- Weather and re-deposition from geologic formations
- Results of irrigation or flooding with saline waters of those soils which have inadequate drainage; and
- Continued irrigation with limited amounts of saline waters of soils with adequate drainage so that no percolation occurs and salts accumulate at or near the surface.

For areas surveyed in southern Afghanistan the following estimates concerning salinity conditions were recorded. Severe to very severe salinity (147, 800 ha of which 65,600 are considered not reclaimed) and moderate to slight salinity was observed in 228,900 ha. Salinity affects an approximately 50% of irrigated areas and former abandoned irrigation areas in the southern half of Herat-Farah lowlands, Southwestern stony deserts and Helmand valley's Sistan basin zones: approximately 20% of the soils are severely to very severely saline, half of which un-reclaimable, whereas about 30% of the soils are slightly to moderately saline. FAO identified the following approximate percentages of soils affected by salinity limitations:

Table 1.2 Salinity affected areas in Afghanistan.

S.No	Area	Approx %	Est Area (ha)
1	Kabul Valley	10	10,000
2	Ghazni/Jilga	15	58,000
3	Nahar	20	25,000
4	Ab-i-Istada/Lora	30	90,000
5	Waza Khawa	15	8,000

6	Farah	50	25,000
7	Hari Rud	2 to 6	8,000 to 25,000

Wind erosion

Wind erosion is a serious hazard in parts of the Marja, Shamalan, Darweshan, Garmser and Chakansur areas (Helmand valley irrigation schemes): drifting sand produces small hummocks around desert shrubs and bunch grasses and small dunes; large active dunes and sparsely covered sand hills occur to the windward side of structures in the Central and Lower Darweshan, south Marja and south Shamalan. The menace of drifting sand during periods of strong winds which are common to the lower areas in the Helmand valley is threefold Filling of ditches and drains creating an annual high cleanout cost.

1.2 Problems of Rainfed Agriculture

According to FAO (1972) rainfed agriculture occupied approximately 4,835,700 ha or 8% of the country's total area. Important rainfed agriculture is found in northern provinces (Faryab, Badghis, Balkh, Samangan, Baghlan and Takhar).

Rainfed agriculture in Afghanistan is an extensive and of opportunistic nature and appears to be a very neglected sector. Rainfed wheat is generally an economically critical crop to the poorer farming communities but is highly vulnerable to plague insect attack and rainfall variations. Yields are generally low and soil erosion is common. Yield for rainfed wheat are generally low and vary widely from year to year, probably ranging from 300 to 1400 kg/ha according to rainfall, the region and farmers' practices. The reason for low yields are mainly erratic rainfall, the almost total absence of fertilizer application, the lack of improved seeds, inadequate planting and weeding practices and the incidence of crop pests and diseases. The prominent lack of soil and water conservation measures certainly contributes to the problem of low yields and is probably the major reason for recurrent flashfloods and subsequent damage to irrigation systems. The risks associated with rainfed farming are probably too high for farmers to invest in fertilizers. Farmers seem to rely rather on a rotational system to regenerate soil fertility. Runoff, erosion, flashflood and sedimentation are the common problems in the rainfed areas

1.3 Water Resources Management

In the absence of any national or provincial institution capable or authorized to monitor ground water extraction or surface water use, there is virtually no water resources management taking place at present. Water allocation for irrigation schemes is handled in a mix of traditional water rights and arbitrary orders by local commanders and or authorities. In traditional as well as in modern irrigation schemes the dominant irrigation method is basin/border irrigation for cereals and furrow irrigation for vegetables and grapes. Main constraints are (i) shortage of water after the peak (June/July) of snow melt, and (ii) shortage of labor for the cleaning of canals.

In spite of considerable efforts in the rehabilitation of irrigation infrastructure undertaken during the last years, there remains the enormous task of rehabilitating about half of

Afghanistan's irrigation schemes. This includes about 1.4 million ha of traditional irrigation schemes supplied by perennial rivers and streams as well as about 50,000 ha supplied by springs and karez, all operated and maintained by the farmers themselves, and about 300,000 ha of modern irrigation schemes formally operated and maintained, and subsidized, by the Government. "Rehabilitation" comprises all activities, which have to be undertaken to safeguard the irrigation scheme against floods and to enable the farmers to manage their schemes, at best efficiently and economically, or at least in the way it was operated before 1978. Irrigation rehabilitation activities can be classified into the following types of rehabilitation works:

Verification:	Problem identification, survey, (re)design, project formulation
Maintenance:	Cleaning of canals and karez
Construction:	Permanent intake structures: spillway, distribution and drainage system structures canal lining and canal bank stabilization, river training (flood protection) and rectification of flow gauge stations
Management:	(Re)establishment and improvement of Mirab/Vakil system reservoir operation and intake regulation, watershed conservation and ground water recharge
Training:	Engineers, technicians, craftsman, mirabs, extensionists, water user groups, (acc. To their tasks)

Irrigation, being one of the most important input of agriculture becomes an equally important component of the rural infrastructure development. The basic requirements and facilities which are essential for development of the regions using local resources can be defined as the infra-structural for the rural development.

1.4 Irrigation

The scientific application of water artificially to the soil to make up the deficiency of moisture under natural conditions for the profitable growth of crops, which could not be assured otherwise through precipitation etc. Irrigation can also be defined as the controlled application of water to arable land to supply crop requirements of water not satisfied by rainfall.

Factors necessitating Irrigation.

1) Insufficient rainfall
   a) Total rainfall < 100 cm.
   b) Not at the proper time
2) Non uniform distribution of rainfall.

Need for Irrigation

- Increase production
- Increase income
- Increase labor employment
- Increase standard of living.
- Leaching

- reclaim sodic soil
- Frost protection.
- To lessen risk of catastrophes
- Population redistribution
- National security.

Intimate plant Environment

1) Climate
2) Soil moisture
3) Soil
4) Fertility
5) Pest.
6) Natural site condition
7) Cultivation

Good Irrigation System

- To deliver water in efficient quantity and quality
- Timely
- Low cost (Economical)
- Will meet crop need
- Result in uniform distribution.

Surface Irrigation System

It consists of four sub system (Figure 1.1)

I) Water Supply subsystem (storage for intake or diversion woks)
II) Water Delivery subsystem (conveyance, control and other hydraulic Structures)
III) Water use subsystem
IV) Water Removal subsystem

I) Water Supply Subsystem

- Diversion from river or streams
- Pumping from ground water basins or rivers
- Diversion from surface impoundment

II) Water Delivery Subsystem

It consists of an irrigation network of canals, distributaries, minors, water courses and structures destined to convey, regulate, control and distribute water obtained from a natural stream, reservoirs or well to fields where crops are grown.

III) Water use Subsystem

It includes the methods of water application to the fields and field ditches.

Irrigation application methods:

A. Surface		B. Pressurized	
1)	Flood	1)	Sprinkler
2)	Furrows	2)	Side roll
3)	Basin	3)	Handling
4)	Border	4)	Solid set
		5)	Linear move big gun
		6)	Traveler

IV) The water removal subsystem

It consists of drainage net work for disposal of extra irrigation and rainfall water.

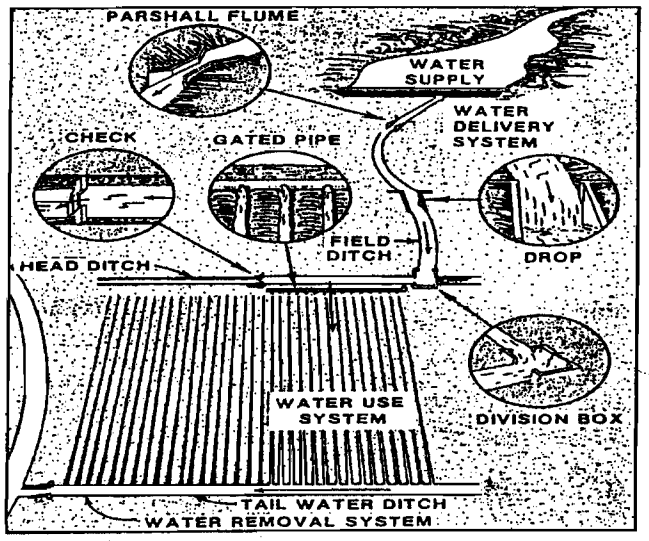

Figure 1.1 View of the typical Irrigation sub-systems

Sources of irrigation water

- Water for irrigation is obtained from surface and ground water sources which includes:
- Streams
- Reservoirs
- Ground water i) Wells and springs
- Precipitation (rainfall and snow) storage

2 CLIMATE, LAND AND WATER RESOURCES OF AFGHANISTAN

2.1 Climate of Afghanistan

Due to Afghanistan's rugged mountainous relief characterized by snow covered peaks (Kuh) of high altitude (up to 7500 masl), deeply incised fertile valleys and large desert plains (dasht), the climate is very heterogeneous, and hence not easily described at a country-wide scale. The typical continental climate varies form arid in the South and Southwest to semi –arid in most other parts of the country. The high mountain ranges of Hindu Kush and Pamir are moderate humid and covered by permanent snow and glaciers at altitudes above 5000 m. With a few exceptions of some locations receiving sufficient rainfall in spring (Northern slopes of Hindu Kush above 1000m altitude), the climate is not favorable for rain fed agriculture: during winter, temperatures are low and precipitation occurs in form of snow whereas during summer, temperatures are high and rainfall is virtually nil. This unfavorable climate for rain fed agriculture is, however, rather favorable for irrigated agriculture. Snow stored in the Hindu Kush during winter is melting in late spring providing irrigation water at a time when crop water requirements are at their peak in early summer.

Climatic Zones and Their Characteristics:

Climatic zoning is based on the modified geographic zones system (Dupree, 1973) proposed in the Land Management Consultancy Report of this Mission. As water availability for irrigation purposes is mainly a function of effective rainfall and surface as well as ground water resources which depend in turn on the amount and distribution (time and space) of precipitation 6 climatic zones (Table 2.1) using precipitation as the most decisive parameters are as follows:

Table 2.1 Climatic Zones of Afghanistan

Zone	Name	Precipitation (mm)	Dry (months)	Frost (months)
1	Badakhshan (without Wakhan)	300 - 800	2-6	1-9
2	Central and Northern Mountains	200 - 600	2-9	0-8
3	Eastern & Southern Mountains	100 - 700	2-9	0-1
4	Wakhan corridor and Pamir	<100 - 500	2-5	5-12
5	Turkestan Plains	<100 - 400	5-8	0-2
6	Western + S. western Lowlands	<100 - 300	6-12	0-3

In general, there is a strong correlation between precipitation, length of growing season and altitude: the greater the altitude, the greater the amount of precipitation and the shorter the growing season due to frost hazard. Zones 1-4 are mountainous, and where rainfall is sufficient, the limiting factor is frost. Zones 5 and 6 are flat, the growing season is sufficiently long even for double cropping under irrigated conditions, but the limiting factors are effective rainfall (rain fed agriculture) and irrigation water availability (irrigated agriculture).

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Precipitation, Temperature, and Evapotranspiration

On average 12 year records of precipitation, temperatures and evapotranspiration are available from 19 meteorological stations located in 18 provinces. In Table 2.2, the altitude of each station is given, the mean annual precipitation, the range between the mean monthly minimum (December/January) and maximum (July/August) temperature, the mean annual reference evapotranspiration (ETP = potential evapotranspiration of grass), and the mean daily evapotranspiration during the moths of July and August when evapotranspiration reaches a maximum.

Precipitation:

Generally, precipitation is directly correlated with altitude varying from less than 10 mm per year at altitudes below 1000 m and 800 mm per year above 5000 m (Table 2.2). However, there are several distinctive regions which do not comply with this rule due to their surrounding mountains: the Wakhan corridor situated in Badakhshan province, several valleys in the provinces of Ghor and Bamyan and the Kabul plains of Nangarhar province receive disproportionately little rainfall whereas Badakhshan (without Wakhan corridor), Takhar, Kunduz and Faryab province benefit form a disproportionately high amount of rainfall. Almost half of the country is covered by the Hindu Kush Mountains at altitudes higher than 2000 m asl. With the exception of parts of Ghor, Bamyan, Nangarhar and Badakhshan provinces, there high altitude areas receive a mean annual precipitation of 500mm varying from 300 mm to 800 mm depending on altitude, exposure and monsoon influence. Generally, over 90 per cent of all precipitation occurs in the Hindu Kush between November and May and in the W3estern and Southwestern plains between December and April. Only the provinces of Paktia and Paktikca situated in the southeast of the country are under monsoon influence and hence benefit from rainfall during summer, too.

Table 2.2: Precipitation, Temperatures and Evapotranspiration

Location	Altitude (m asl)	Precipitation (mm/a)	Temperature degree C	Annual ETP (mm/a)	Daily ETP (mm/d)
Shiberghan	360	214	-2 - + 38	1420	8
Mazar-I-Sharif	378	190	-2 - + 39	1530	9
Kunduz	433	349	-2 - + 39	1390	8
Baghlan	510	271	-2 - + 37	1100	6
Jalalabad	580	171	+3 - + 41	1350	7
Farah	660	77	~0 - + 42	1610	8
Lashkargah	780	89	~0 - + 42	1720	8
Maimana	815	372	-2 - + 35	1310	7
Hirat	964	241	-3 - + 36	1720	10
Qandahar	1010	158	~0 - + 40	1790	8
Khost	1146	448	-1 - +35	1390	6
Faizabad	1200	521	-5 - +35	1020	6
Qadis	1280	323	-3 - + 30	1240	6
Jabul Saraj	1630	499	~0 - + 31	1610	9
Kabul	1791	303	-7 - + 32	1280	7
Karizimir	1905	433	-7 - + 31	1100	6
Ghalmin	2070	222	-8 - + 29	1100	6
Ghazni	2183	292	-11 - + 31	1420	7
Lal Sarjangal	2800	282	-21 - + 25	950	5

Temperature

The daily (20-30 °C) as well as the seasonal (35 – 40 ^{0}C) variations of temperatures prevailing all over the country lead to distinctive different lengths of growing season, and require a careful selection of the most suitable crop for an area. This is reflected in many regions very well known for their particular agricultural product (et grapes, melons, rain fed wheat) or their natural forest cover (pistachio trees, pines). In the South western desert plains, frost can occur in any month of the year even which temperatures reach a daily maximum of up to 40 °C. Daily minimum temperatures in the Northern (Turkestan) plains can be as low as –20 °C, in winter and as high as +50 °C in summer at the same location.

Evapotranspiration

Annual evapotranspiration rates are relatively low in the Hindu Kush (1000-13000 mm) because of severe and long winters. They vary between 1300mm and 1500mm in the Northern plains and reach values of up to 1800 mm in the southern and southwestern plains. However, seasonal (summer) evapotranspiration rates are high everywhere showing a daily peak of 6-8 mm in July/August. Due to strong winds occurring particularly in Mazari Sharif and Hirat (Bad-e sad-o bist rus), maximum daily evapotranspiration rates are 9mm and 10mm, respectively.

About 80% of the water resources of Afghanistan has their origin in the Hindu Kush mountain range. The amount of water received through precipitation is estimated to be 180,000 million m3 for the whole country. Afghanistan is part of three large river basins. The Amu Darya basin in the North separated by the Hindu Kush mountain range from the desert basin in the south, and Indus basin in the east.

Table 2.3: Monthly, potential Evapotranspiration and Precipitation at different locations.

Station		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
Baghlan	ETo (mm/day)	0.8	1.1	1.9	2.8	4.3	5.6	5.6	4.9	4.1	2.6	1.4	0.8
	ETo (mm/mo)	25	31	5	84	133	168	174	152	123	81	42	25
	PPt (mm/mo)	30	40	62	62	30	0	0	0	0	0	24	23
	Pe (mm/mo)	29	37	56	56	29	0	0	0	0	0	23	22
Bust	ETo (mm/day)	2.0	2.4	3.8	4.8	6.4	7.2	7.5	7.2	6.4	4.1	2.6	2.1
	ETo (mm/mo)	62	67	118	144	198	216	233	223	192	127	78	65
	PPt (mm/mo)	21	16	21	13	0	0	0	0	0	0	3	12
	Pe (mm/mo)	20	16	20	13	3	0	0	0	0	0	3	12
Faizabad	ETo (mm/day)	0.7	1.0	1.7	2.7	3.8	5.3	5.7	4.9	3.8	2.3	1.2	0.8
	ETo (mm/mo)	217	28	53	81	118	159	177	152	114	71	36	25
	PPt (mm/mo)	43	80	101	99	88	8	11	1	2	28	29	31
	Pe (mm/mo)	40	70	85	83	73	8	11	1	2	27	28	29
Farah	ETo (mm/day)	1.7	2.3	3.5	4.3	5.9	6.9	7.8	6.6	5.9	4.0	2.4	1.6
	ETo (mm/mo)	53	64	109	129	183	207	242	205	177	124	72	50
	PPt (mm/mo)	18	24	13	8	2	0	0	0	0	0	3	9
	Pe (mm/mo)	17	23	13	8	2	0	0	0	0	0	3	9
Ghazni	ETo (mm/day)	1.1	1.4	2.4	3.6	5.2	6.6	6.7	6.7	5.7	3.8	2.2	1.4
	ETo (mm/mo)	34	39	74	108	161	198	208	208	171	118	66	43
	PPt (mm/mo)	41	45	67	60	24	2	14	1	0	1	11	26
	Pe (mm/mo)	38	42	60	54	23	2	14	1	0	1	11	25
Ghelmin	ETo (mm/day)	0.9	1.0	1.7	2.8	4.2	5.4	5.8	5.2	3.8	2.5	1.5	1.0
	ETo (mm/mo)	28	28	53	84	130	162	180	161	114	78	45	31
	PPt (mm/mo)	33	30	44	50	20	0	2	0	0	8	7	28
	Pe (mm/mo)	31	29	41	46	19	0	2	0	0	8	2	27
Herat	ETo (mm/day)	1.4	1.9	3.1	3.8	5.9	8.0	9.7	8.9	6.3	4.0	2.2	1.5
	ETo (mm/mo)	43	53	96	114	183	240	301	276	189	124	66	47
	PPt (mm/mo)	45	35	48	32	8	0	0	0	0	1	9	33
	Pe (mm/mo)	42	33	44	30	8	0	0	0	0	1	9	31
Jabul Siraj	ETo (mm/day)	1.1	1.5	2.8	3.7	5.8	8.0	8.9	7.8	6.1	3.7	2.3	1.2
	ETo (mm/mo)	34	42	87	111	180	240	276	242	183	115	69	37
	PPt (mm/mo)	52	86	109	142	35	1	3	1	3	9	22	36
	Pe (mm/mo)	48	74	90	110	33	1	3	1	3	9	21	34
Jalalabad	ETo (mm/day)	1.3	1.9	2.7	3.7	5.3	6.6	6.6	5.7	4.5	3.2	2.1	1.3
	ETo (mm/mo)	40	53	84	11	164	198	205	177	135	99	63	40
	PPt (mm/mo)	19	22	36	33	18	1	6	2	3	7	8	16
	Pe (mm/mo)	18	21	34	31	17	1	6	2	3	7	8	16
Kabul	ETo (mm/day)	1.0	1.3	2.2	3.2	4.8	6.3	6.7	5.8	4.5	3.0	1.8	1.1
	ETo (mm/mo)	31	36	68	96	149	189	208	180	135	93	54	34
	PPt (mm/mo)	37	59	67	78	21	0	6	0	2	3	11	19
	Pe (mm/mo)	35	53	60	68	20	0	6	0	2	3	11	18
Qandahar	ETo (mm/day)	2.4	2.6	3.9	5.5	7.1	7.7	7.9	7.0	5.9	4.4	3.0	2.4
	ETo (mm/mo)	74	73	121	165	220	231	245	217	177	136	90	74
	PPt (mm/mo)	51	35	31	12	1	0	2	0	0	2	6	18
	Pe (mm/mo)	47	33	29	12	1	0	2	0	0	2	6	17
Karizimir	ETo (mm/day)	1.0	1.3	2.0	2.9	4.2	5.2	5.6	4.9	4.0	2.7	1.6	1.1
	ETo (mm/mo)	31	36	62	87	130	156	174	152	120	84	48	34
	PPt (mm/mo)	44	60	99	121	31	3	7	1	3	7	27	30
	Pe (mm/mo)	41	54	83	98	29	3	7	1	3	7	26	29

Station		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
Khost	ETo (mm/day)	1.6	2.0	2.9	3.8	5.2	6.1	5.7	5.1	4.7	3.6	2.5	1.8
	ETo (mm/mo)	50	56	90	114	161	183	177	158	141	112	75	56
	PPt (mm/mo)	15	39	55	76	40	17	82	52	33	7	10	22
	Pe (mm/mo)	15	37	50	67	37	17	71	48	31	7	10	21
Kunduz	ETo (mm/day)	0.8	1.2	1.9	3.0	4.9	7.1	7.7	7.1	5.6	3.4	1.7	0.9
	ETo (mm/mo)	25	34	59	90	152	213	239	220	168	105	51	28
	PPt (mm/mo)	39	55	75	67	41	0	2	0	0	8	26	36
	Pe (mm/mo)	37	50	66	60	38	0	2	0	0	8	25	34
Lal	ETo (mm/day)	0.6	0.8	1.4	2.2	3.6	4.6	5.0	4.6	3.6	2.3	1.2	0.8
	ETo (mm/mo)	19	22	43	66	112	138	155	143	108	71	36	25
	PPt (mm/mo)	24	44	43	68	37	4	0	0	0	16	25	21
	Pe (mm/mo)	23	41	40	61	35	4	0	0	0	16	24	20
Maimana	ETo (mm/day)	1.0	1.4	2.1	2.9	4.8	6.5	6.9	6.4	5.0	3.3	1.9	1.2
	ETo (mm/mo)	31	39	65	87	149	195	214	198	150	102	57	37
	PPt (mm/mo)	48	56	90	68	34	2	1	0	0	10	28	35
	Pe (mm/mo)	44	51	77	61	32	2	1	0	0	10	27	33
Mazar-i-Sharif	ETo (mm/day)	1.0	1.3	2.2	3.4	5.7	7.5	9.0	7.9	5.5	3.6	1.9	1.1
	ETo (mm/mo)	31	36	68	102	177	225	279	245	165	112	57	34
	PPt (mm/mo)	26	33	40	33	12	2	0	0	0	5	16	23
	Pe (mm/mo)	25	31	37	31	12	2	0	0	0	5	16	22
Qadis	ETo (mm/day)	1.7	1.4	2.2	3.0	4.1	5.6	6.0	5.8	4.6	2.8	1.8	1.9
	ETo (mm/mo)	53	39	68	90	127	168	186	180	138	87	54	59
	PPt (mm/mo)	46	39	66	48	28	0	1	0	2	23	22	48
	Pe (mm/mo)	43	37	59	44	27	0	1	0	2	22	21	44
Sherberghan	ETo (mm/day)	1.1	1.4	2.2	3.3	5.8	7.2	7.5	6.9	5.3	3.5	2.0	1.2
	ETo (mm/mo)	34	39	68	99	180	216	233	214	159	109	60	37
	PPt (mm/mo)	38	39	41	33	7	0	0	0	0	12	15	29
	Pe (mm/mo)	36	37	38	31	7	0	0	0	0	12	15	28

2.2 Hydrologic Cycle

Hydrology is the science of water, which deals with occurrence, circulation and distribution of water on earth surface and beneath it. The hydrologic cycle represents the interdependence and continuous movement of all forms of water provide the basis for the concept of hydrologic cycle (Figures 2.1 and 2.2). For agricultural purposes only a part of the general hydrologic cycle is relevant to agriculture is considered. This part is called the *agro hydrologic cycle*. The central unit in the agro hydrologic cycle is the part of the soil which contains the crop roots. This unit is usually referred to as the *rootzone*. The depth of the rootzone differs per crop. Alfalfa for example has a rootzone of approximately 2.4 metre while wheat can have a rootzone of 1.8 metre and onions one of 0.9 metre. In the agro hydrologic cycle, agriculturists are mainly interested how much water enters this rootzone, how much is stored, and how much goes out again. The main sources of water in the agro hydrologic cycle are precipitation and in irrigated areas also irrigation. Part of this water is discharged to small streams or rivers by surface runoff, another part directly evaporates from the surface, and the rest of the water infiltrates into the soil. A part of the infiltrated water leaves the rootzone either by evaporation or by absorption by the roots of the crops. The absorbed water is brought back into the atmosphere by transpiration.

Another part of the infiltrated water can be stored in the soil of either the rootzone, or the zone below the rootzone and above the water table. The rest of the infiltrated water leaves the rootzone by percolation to the water table. Besides precipitation and irrigation there is another possible source of water in the rootzone: capillary rise. If the water table depth is not too deep, water can enter the rootzone by capillary rise. This water can either be

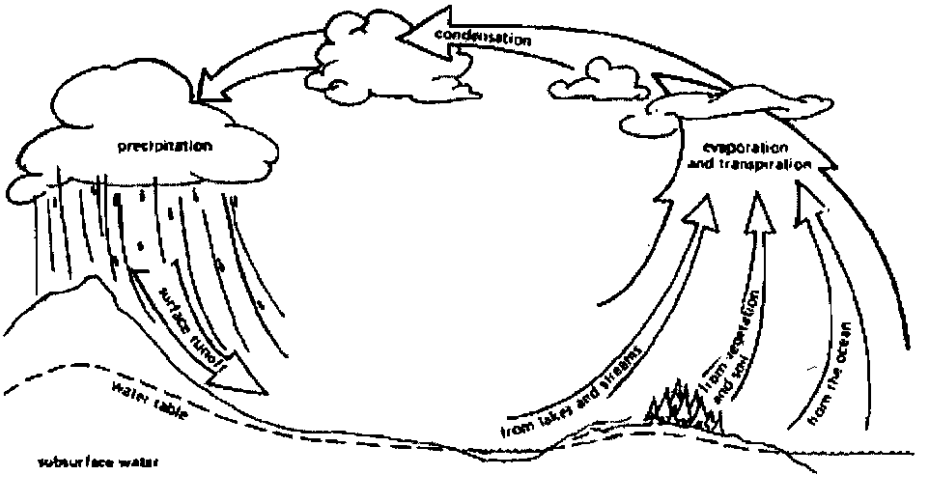


Figure 2.1. Schematic diagram of the hydrologic cycle.

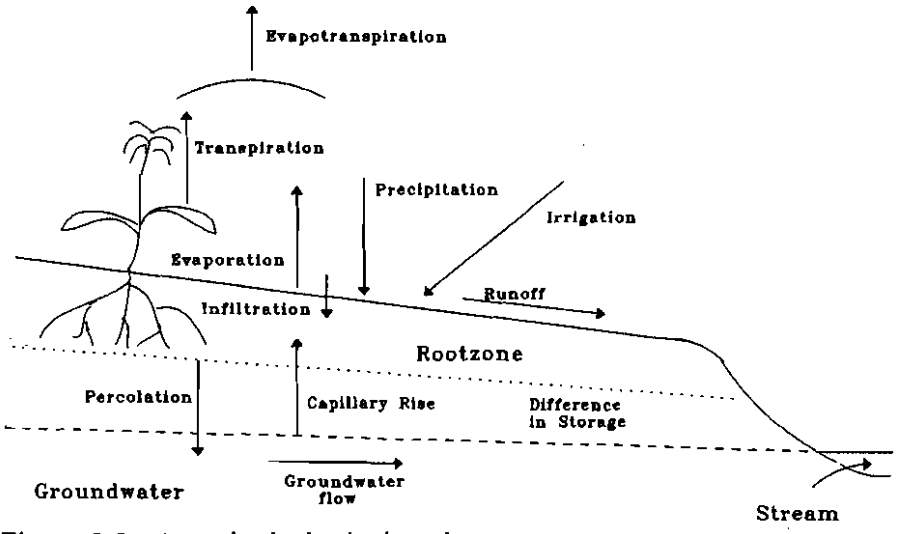


Figure 2.2 . Agro-hydrological cycle.

transpired by the plant or evaporated at the ground surface. If more water percolates to the groundwater than leaves the groundwater by capillary rise, water has to be discharged by groundwater flow to streams and rivers. Figure 2.2 above gives a schematic overview of the whole agrohydrologic cycle. It should be noted that in reality rain or irrigation water can directly evaporate from the ground surface or the soil moisture can rise to the ground surface and evaporate. For the sake or simplicity, we will assume that only soil moisture evaporates from the ground surface. Since it is difficult to measure the evaporation from the ground surface independently from the transpiration of a plant, they are often joined together and named *evapotranspiration.*

2.3 Water Resources of Afghanistan

Excluding fossil ground water, over 80 % of the country's water resources have their origin in the Hindu Kush mountain ranges at altitudes above 2000 m which function as a natural storage of water in form of snow during winter and thus support perennial flow in all major rivers by snow melt during summer. The amount of water received in these are^{3}as through perception is estimated to be in the order of 150,000 million m^3. The other half of the country receives only about 30,000 million m^3 annually through rainfall resulting in a total amount of 180,000 million m^3 for the whole country. There were over 140 stations located all over the country where rainfall and or river flow was measured regularly. At present, most of these stations are destroyed and their measurement equipment looted. This, none of them is operational.

Hydrological Classification

Afghanistan is part of 3 large river basins: The Amu Darya basin in the North separated by the Hindu Kush mountain range form the Desert basin in the South, and the Indus basin in the East (Table 2.4). Because of practical reasons related to the quantification of available surface water volumes, the hydrological classification chosen is based on principal watershed units. These are sub divided into main catchments areas whose river either joins a river of the principal watershed within the country or outside of the country, or disappears in a plain within the country because of topographical reasons. Eight basins can be distinguished.

Amu Darya Basin

North eastern river basins: Pamir/Panj river having common boundary with Tajikistan;
Northern river basins: Murghab, Kashan, Jshk and Gulran flow out to Turkmensitan, Samangan, Balkhab, Saripul and Shirin Tagab do not join Amu Darya; and
Hari river (Harirud) basin: partial common boundary with Iran.

Desert Basin

South western river basins:	all rivers drains into depressions situated along Iranian border
Helmand river basins:	entering Helmand lake (Hamun-e Helmand) partially situated in Iran; 26m^3/s to be allocated to the Iranians throughout the year according to the international agreement between Afghanistan and Iran signed in 1972; and
Southern river basins:	only ephemeral rivers, outflow to Baluchistan/Pakistan.
Indus Basin	outflow to NWFP/Pakistan
South Eastern river basins:	watershed of the tributary Kunar.
Kabul river basins:	outflow of Kabul River to NWFP-Pakistan

Although the country features eight river systems only one river system (4) can be exclusively used by Afghanistan itself, another one (2) to a great extent. Two river systems (1 and 3) have a common border with other countries (Tajikistan and Iran respectively), and four rivers flow out of Afghanistan to Iran (5) and Pakistan (6,7 and 8). It should be well understood that any reduction of the discharge of Amu River by potential water users must be seen in the context of ongoing rehabilitation measures for saving Lake Aral.

There are plenty of individual discharge data of many of Afghanistan's rivers, particularly from the Kabul and the Helmand River as well as from their tributaries. However, there is no reliable documentation available about the systematic quantification of surface water resources at watershed level. Additionally, no specific rainfall/run off relationship for any of the country's rivers is known to the Author.

Table 2.4. Surface water resources in Afghanistan

No.	Name of River Basin	Drainage Area (km^2)	Mean Annual Vol. (million m^3)
	Amu Darya Basin		
1	Northeastern river basin		
	Panj	27800 (+29000 in Tajikistan)	36420
	Kokcha	21100	5700
	Kunduz	37100	6000
	Total	86000 (+29000 in Tajikistan)	48120
2	Norhtern river basin		
	Murghab	26200	1350
	Kashan, Kushk, Gulran	12200	110
	Samangan	8300	60
	Balkhab	19300	1650
	Saripul	10800	40
	Shirin Tagab	12100	100
	Amu Darya deserts	27100	30
	Total	116000	3340
3	Hari river basin	39000	1600
	Desert Basin		
4	South western river basin		
	Farah	27800	1250
	Harut	23800	210
	Gulistan	9100	40
	Khash	10500	170
	Kajrud	20800	60
	Total	92000	1730
5	Helmand river basin		
	Ghazni	19200	350
	Helmand	42200	6000
	Musa qala	3700	220
	Arghandab	53000	820
	Lower Helmand	47900	110
	Total	166000	7500
6	Southern river basin	70000	70
	Indus Basin		
7	South eastern river basins		
	Gomal	10700	350
	Margo	8300	400
	Total	19000	750
8	Kabul river basin		
	Panjir	11000	3130
	Kunar	13000 (+14000 in Pakistan)	15250
	Kabul	30000	2540
	Total	54000 (+14000 in Pakistan)	20920
	Grant Total	642000	84000

2.4 Ground Water Resources

Ground water is usually abundant in aquifers along all major river valleys where infiltration of surface water is high (Table 2.5). If an amount of only 10% of the total annual precipitation volume is adopted – which is rather conservative estimate - the annual groundwater recharge volume would be in the order of 18,000 million m^3. Ground water quality is generally good but varies from place to place. In the lower reaches of river valleys intensively irrigated by the diversion of surface water, ground water is frequently found to be saline or brackish and not usable for neither drinking nor irrigation purposes.

Table 2.5. Estimated surface and ground water balance (million m^3 per year)

Water Resource	Potential	Present use	Balance	Future use	Balance
Surface water	57000	17000	40000	30000	27000
Ground water	18000	3000	15000	5000	13000
Total	75000	20000	55000	35000	40000

3 WATER QUALITY FOR IRRIGATION

The quality of irrigation water is determined by the amount of suspended and dissolved material it contains. Some water may not be fit for crops. It can contain salts, which may be injurious to crop growth. Therefore it is necessary to determine the suitability of water before application for agricultural purposes. The quality of suitable irrigation is very much influenced by the constituents of the soil which is to be irrigated (Sharma,1993). Particular water may be harmful for irrigation on a particular soil, but the same water may be tolerable or even useful for irrigation on some other soil. The various types of impurities, which makes the water unfit for irrigation are classified as:

(i) Sediment concentration in water.
(ii) Total concentration of soluble salts in water.
(iii) Proportion of sodium ions to other cat ions.
(iv) Concentration of potentially toxic elements present in water.
(v) Bicarbonate concentration as related to the concentration of calcium plus magnesium.
(vi) Bacterial contamination.

The effects of these impurities are discussed below:

3.1 Sediment

The effect of sediment, present in the irrigation water, depends upon the type of irrigated land. When fine sediment from water is deposited on sandy soils, the fertility is improved. On the other hand, if the sediment has been derived from the eroded areas, it may reduce fertility or decrease soil permeability. Sediment water creates troubles in irrigation canals, as they get silted up and their maintenance cost is increased. In general, ground water or surface water from reservoirs, etc. does not have sufficient sediment to cause any serious problems in irrigation.

3.2 Total concentration of Soluble Salts

Salts of calcium, magnesium, sodium and potassium, present in the irrigation water may prove injurious to plants. When present in excessive quantities, they reduce the osmotic activities of the plants, and may prevent adequate aeration, causing injuries to plant growth. The injurious effects of salts on the plant growth depend upon the concentration of salt left in the soil. The concentration of salts in water may not appear to be harmful to plants, but the concentration of salts which remain in the soil after the saline water is used up by the plants is much more than the first, and may prove to be harmful. In other words, at the beginning of irrigation with undesirable water, no harm may be evident, but with passage of time, the salt concentration in the soil may increase to a harmful level, as the soil solution gets concentrated by evaporation.

The salt concentration is generally expressed by ppm (parts per million) or by mg/l (milligram per liter), both units being equal. The critical salt concentration in the

irrigation water, depends upon many factors. However, amounts in excess of 700 ppm are harmful to some plants, and more than 2000 ppm are injurious to all corps.
The salt concentration is generally measured by determining the electrical conductivity of water. They are directly proportional to each other. Electrical conductivity is expressed in micro mhos per centimeter.
When value is upto 250 micro mhos/cm at 25 °C, is called Low Conductivity Water (C-1).
between 250 to 750, it is called Medium Conductivity Water (C-2)
between 750 to 2250, it is called High Conductivity Water (C-3); and the values above 2250 are classified as very High Conductivity Water (C-4).

Table 3.1.Suitability's of waters based on total concentration of soluble salts (Sharma,1993)

S.No.	Type of water	Use in irrigation
1.	Low salinity water (C1), Conductivity between 100 to 250 micro mhos/cm at 25°C	Can be used for irrigation for almost all crops and for almost all kinds of soils. Very little salinity may develop which may require slight leaching; but it is permissible under normal irrigation practices except in soils of extremely low permeability.
2.	Medium Salinity water (C2). Conductivity between 250 to 750 micro mhos/cm at 25°C.	Can be used, if a moderate amount of leaching occurs. Normal salt-tale-rant plants can be grown without much salinity control.
3.	High salinity water (C3). Conductivity between 750 to 2250 micro mhos/cm at 25°C.	Cannot be used on soils with poor drainage. Special precautions and measures are undertaken for salinity control, and only high salt tolerant plants can be grown.
4.	Very high salinity water (C4). Conductivity more than 2250 micro mhos/cm at 25°C.	Generally not suitable for irrigation.

3.3 Proportion of Sodium Ions to other Cations

Most of the soils contain calcium and magnesium ions and small quantities of sodium ions. The percentage of the sodium ions is generally less than 5% of the total exchangeable cations. If this percentage increases to about 10% or more, the aggregation of soil grains breaks down. The soil becomes less permeable and of poorer tilth. It starts crusting when dry and its pH increases towards that of an alkaline soil. High sodium soils are, therefore, plastic, sticky when wet, and are prone to form clods, and they crust on drying.

The proportion of sodium ions present in the soils, is generally measured by a factor called Sodium-Absorption ratio (SAR) and represents the sodium hazards of water. SAR is defined as:

$$SAR = \frac{Na}{\sqrt{(Ca + Mg)/2}}$$

where, the concentration of the ions is expressed in equivalent per million (epm); emp is obtained by dividing the concentration of salt in mg/1 or ppm by its combining weight (i.e. Atomic weight / Valence).
When the value of SAR lies between 0 to 10, it is called Low Sodium Water (S1);
When between 10 to 18, it is called Medium Sodium Water (S2);
between 18 to 26 it is called High Sodium Water (S3);
When values of SAR are more than 26, called Very High Sodium Water (S4).

The suitability of these four kinds of water for irrigation is given in Table 3.3. The SAR value can be reduced by adding gypsum ($CaSO_4$) to the water or to the soil.

Table 3.2 Suitability of four type of water based on SAR (Sharma,1993)

S.No	Type of water	Use in irrigation
1.	Low sodium water (S1). SAR value lying between 0 to 10.	Can be used for irrigation on almost all soils and for almost all crops except those which are highly sensitive to sodium such as stone-fruit trees.
2.	Medium sodium water (S2).SAR value lying between 10 to 18.	Appreciably hazardous in fine textured soils, which may require gypsum, etc. but may be used on course-textured or organic soils with good permeability.
3.	High sodium water (S3). SAR value lying between 18 to 26	May prove harmful on almost all the soils, and do require good drainage, high leaching , gypsum addition, etc. for proper irrigation.
4.	Very high sodium water (S4). SAR value above 26.	Generally not suitable for irrigation.

The quality of water falling in various group as developed by Sharma,S.K., 1997 is given below:

Table 3.3 Various groups of water Quality

Salinity Group	Quality of irrigation water
C1-S1 C2-S1	Good
C1-S2 C2-S2 C3-S2 C3-S1	Moderate
All other groups	Bad

3.4 Concentration of Potentially Toxic Elements

A large number of elements such as boron, selenium, etc. may be toxic to plants. Traces of Boron are essential to plant growth, but its concentrations above 0.3. ppm may prove toxic to certain plants. The concentrations above 0.5 ppm is dangerous to nuts, citrus fruits and deciduous fruits. Cotton, Cereals and certain truck crops are moderately tolerant to boron, while Dates, Beets, Asparagus etc. are quite tolerant even for the most tolerant crops, the boron concentration should not exceed 4 ppm. Boron is generally present in various soaps. The waste water containing soap.etc. should, therefore, be used with great care in irrigation. Selenium, even in low concentrations, is toxic, and must be avoided.

3.5 Bicarbonate Concentration and Bacterial Contamination

High concentration of bicarbonate ions may result in precipitation of calcium and magnesium bicarbonates from the soil-solution, increasing the relative proportion of sodium ions and causing sodium hazards.

Bacterial contamination of irrigation water is not a serious problem, unless the crops irrigated with highly contaminated water are directly eaten, without being cooked. Cash crops like cotton, nursery stock. etc. which are processed after harvesting, can therefore, use waist waters, without any trouble.

4 GUIDELINE FOR IRRIGATION SCHEME DEVELOPMENT

4.I Minor Irrigation Works

Minor irrigation includes a large variety of schemes which, differ widely in their technical features and scope. Some require specialized knowledge while others may just depend on local skills and talent for satisfactory completion. The schemes also differ vastly in terms of ownership some of them are private-owned, other owned by the community and state. These schemes include the following:

- Diversion from river or stream
- Lift irrigation schemes
- Repair and improvement of existing irrigation channel
- Desilting of irrigation channels
- Construction of tank /pond for irrigation in case of small spring
- Desilting and improvement of Karez system
- Construction of wells or tube well for irrigation

Over Riding Considerations for Minor Irrigation Works

There are definite considerations which make a strong case for implementing Minor irrigation over Major and Medium irrigation. Major and Medium irrigation involve extremely heavy capital and operational expenditure. Minor irrigation on the other hand cost less. In many cases part of the expenditure is borne by the farmers themselves. Minor irrigation can serve the small farmers directly. Minor irrigation projects take less times in construction and therefore, bring in immediate results. On the other hand the time required by the heavy projects before they become productive is extremely long.

4.2 Guidelines for Minor Irrigation Scheme Development

Before undertaking the preparation of a detailed project for a new work, a preliminary report, and rough estimate and plan, giving an idea of the approximate cost of the project based on actual data should be prepared in consultation with the Department concerned wherever necessary. This preliminary rough project should be got countersigned by the officer of the Department for whom the work is required and then submitted to the competent authority for administrative approval. After such estimate is approved, a separate estimate should be got prepared by the Executive Engineer or the special duty officer placed in charge of the project for conducting accurate and detailed survey.

After collecting the data and statistics required during the survey season, the plotting of the sections and the preparation of plans should be done in the office. Experienced upper subordinates or other departmental men should be set to the work of preparing the plans and detailed estimates for the different works composing the project.

In preparing estimates, only the current rates for items of work as per the schedule of rates of the Division in which the work falls, should be taken. If any substantial variations are

necessary, the circumstances demanding these may be explained in the body of the report or the general description of the project.

This freshly prepared project should again be got countersigned by the officer of the Department concerned before submitting it for sanction. The specific source from which the expenditure could be met should be indicated. On receipt of sanction, the funds required for the work should be applied for and on actual allotment of funds work should be taken up.When submitting the project or work for sanction, the Executive Engineer or the officer in charge should note concisely and clearly in report all points deserving consideration in connection with the proposals embodied in the project.

The following details should be furnished in the reports accompanying the estimates.
History:A brief review of the circumstances leading to the necessity for undertaking the work.
Scope: The extent of the work involved.
Specifications and Drawings: Whether these are attached.
Cost of work, allocation of charges under budget or other head of account. Whether there is budget provision for the expenditure.
Data Sheet: In respect of rates provided for items of work not contemplated in the sanctioned schedule of rates, data sheets as to know each rate is arrived at, should be furnished.
Revenue-Expected: revenue in respect of revenue yielding works.
Counter-signature: Whether the estimates have been got countersigned by competent authorities as required by rules.
Execution: Proposed method of execution whether by Department or other agencies or with out tenders.
Establishment: Whether the regular staff will do for the work or any extra establishment is required.
Work-Establishment: If any temporary work establishment is required, details should be furnished in the Estimate.
Time: When the work is expected to be commenced and completed.
Practices: Demand for Minor Irrigation Projects from different sources , i.e individual, community, tribe and government.

4.3 Five important phases of Minor Irrigation Projects

- Feasibility Report for Project Identification and Formulation
- Detailed plan and Estimate
- Construction Phase
- Monitoring and Evaluation
- Operation and Maintenance Phase

4.4 Key Issues in Water Management

Community Management of Small Scale Irrigation

i) Mobilization of local resources (which include not only labor but also financial and material resources

ii) Maximizing production and employment

iii) Ensuring equitable water distribution

Improvement and Rehabilitation of Small Irrigation Schemes

Both improvement and rehabilitation can have a major impact on the operation and management of small system, in particular on the functioning of existing community organizations.

Self-reliant Management of Small Irrigation Schemes: Pre-conditions and incentives required for the effective organization and functioning of self reliant organizations some of which could be as follows:

- A democratically elected and accountable leadership,
- Adequate remuneration system and benefits for the leaders,
- A high degree of financial self-reliance achieved through the mobilization of labor, financial resources etc (through membership fees, fines and so on)
- A set of rules and regulations evolved preferable by the water users themselves
- based on local traditions, customs etc and their strict enforcement,
- Equitable distribution of obligation and benefit (especially labor contribution and other payment assessed according to share of water received.

4.5 Irrigation criteria for minor project development

Three types of criteria were used by the PATA project Pakistan for development of minor irrigation project.

1. Irrigation Criteria 2. Hydro-Geological Criteria 3. Social Criteria

Irrigation Criteria

- The maximum elevation between the lower and the upper part of the command area may not exceed 12.5 m for a Dug Well and 20 m for a Tube Well Scheme.

- The maximum drop of a single terrace may not exceed 5.00 m.

- The total span of aqueducts to cross drains or gulley may not exceed 6.5 m. In addition, the irrigated land on the lower side of the aqueducts must be at least 25 % of the total command area.

- The fields in the command area must be adjacent to each other.

- The command area on the lower side of a road crossing must be at least 10 % of

the total command area.

- If erosion is strongly damaging the command area now or it is clear that it will occur in the near future, the site should not be selected.

- If a graveyard is incorporated in the command area, the consequences for the design should be evaluated.

- Soil texture is not a criteria.

- The required channel length in the command area is not a criteria.

- The existence of an approach road is not a criteria. Farmers have to provide access to the site but this point will be discussed during the Agreement process between OFWM and the Farmers Group

Hydro-Geological Criteria

- Availability of water yielding strata, verified by discharge data about drinking water wells from the Public Health department; other well test data; Irrigation Department data; WAPDA data.

- Dug Well or a Surface Lift Irrigation scheme: the estimated availability of water of at least 10.5 l/s (0.375 cusec) for 8 hectares (= 40 jerib).

- Dug Well: The maximum Head H_{tot} (=difference in level from the lowest water table level to the highest stilling basin including friction losses in the pipes) may not exceed 30 m (. 100 ft). This H_{tot} is related to a 7.5 H.P. motor and a 14 kW generator.

- Dug Well: Seasonal fluctuation in the water table might lead to a maximum number of 2 platforms to avoid too many shiftings of the pump.

- Tube Well: the estimated availability of water of at least 21 l/s (0.73 cusec) for 16 hectares (= 80 jerib).

- Tube Well: the maximum Head H_{tot}, including friction losses in the pipes, may not exceed 90 m (. 300 ft). This maximum H_{tot} is related to a 50 H.P. motor and a turbine driven pump.

- Preference will be given to sites where the Dug or Tube Well can be constructed at the highest point.
- In case of spring captation the total command area will be limited by the spring discharge.

- In case of rehabilitation of traditional Diversion Scheme the discharge of the source is not a criteria, because it is impossible to include or exclude land.

Social Criteria.

- All the landowners of the command area must be among the applicants.
- The minimum number of landowners is 5 for a Dug Well or Surface Lift Scheme and 8 for a Tube Well Scheme. There is no upper limit.
- The landownership of one member of the Farmers Group may not exceed 30 % of the total command area.
- The names of all the landowners must be provided on the applications forms by the Farmers Group together with their signature/thumb print or that of their legal representatives.
- The landowners must declare their willingness to cooperate with one another.
- The landowners must declare their willingness to give all the sociological data which the PATA Project finds useful for their selection activities and for the further implementation of the Scheme Development Progress.
- The landowners must declare their willingness to declare all the land on which the physical infrastructure will be built to be communal land.
- The landowners must declare their willingness to contribute 10 % of the investment costs.
- Preference will be given to those sites of which the landowners are present, which means: not abroad or living in other parts of the country.
- Preference will be given to sites which belong to one village or to two neighbouring villages.
- Command areas with disputed land will not be selected.
- Political affiliation is not a criteria; applications provided by local politicians shall not be accepted when they have no land in the applied site.

Other criteria for Selection of Minor Irrigation Scheme for Development

- Total Cost of the Project should not be more than certain fixed limit
- Cost benefit ratio
- Number of beneficiaries

5 IRRIGATION REQUIREMENTS

5.1 Reference Crop Evapotranspiration (Eto)

The evapotranspiration from extensive surface of 3-6 inches (9.1-15.2 cm) tall grass or 8 inches (20.32 cm) tall alfalfa of uniform height actively growing and is well watered. ET_0 can be determined by different methods. Only four methods will be discussed in following section.

5.2 Effect of Climate on ETo

Major factors affecting ET_0 are as follows:

- Air Temperature
- Sunshine
- Wind speed
- Humidity
- Altitude
- Latitude

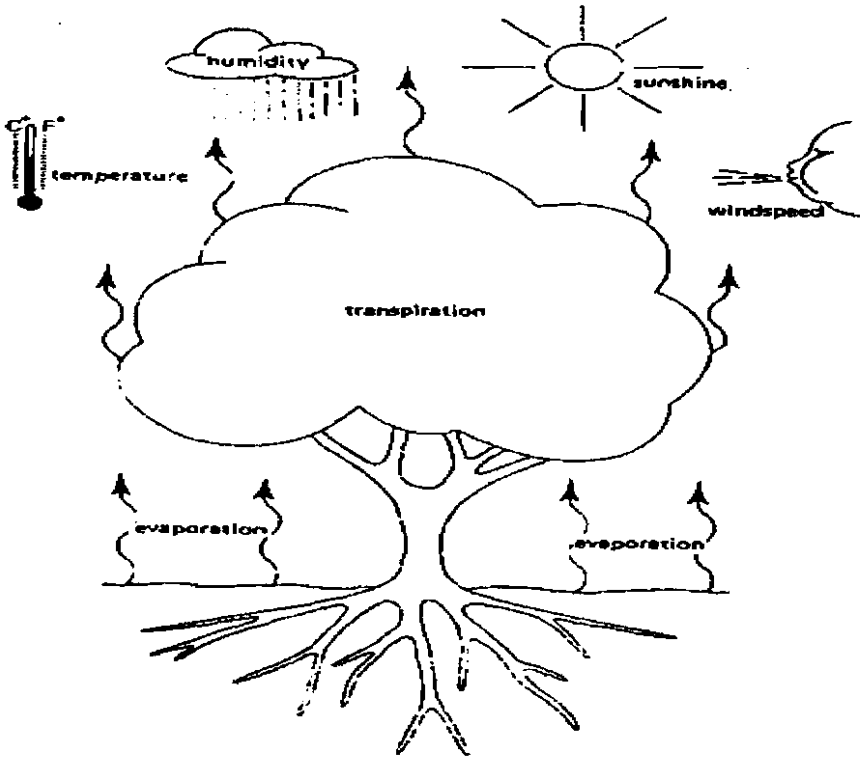
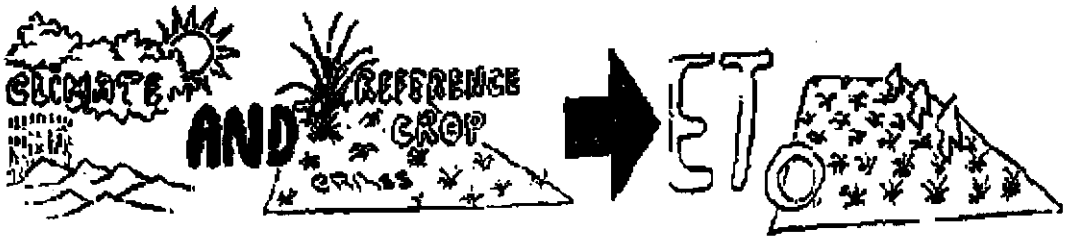


Figure 5.1 Effect of climatic elements on ET0.



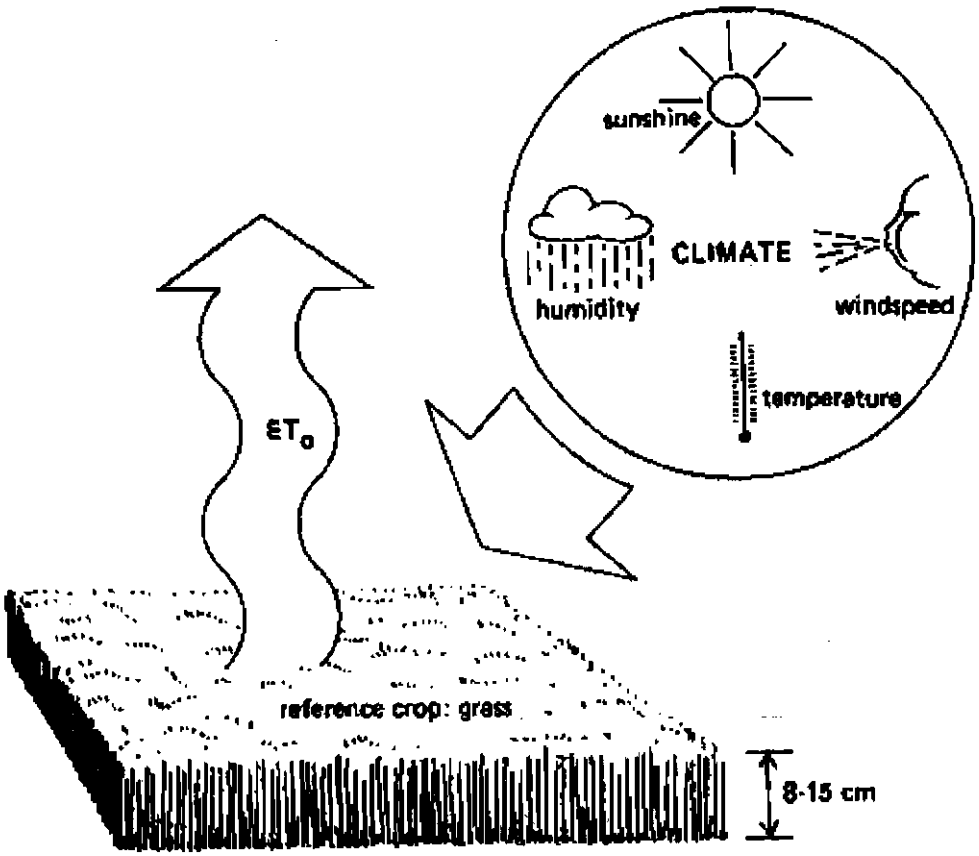


Figure 5.2 Assessment of ET0 from climatic data.

5.3 Pan evaporation

$$ET_0 = Kp \times Ep$$

Where ET_0 is reference crop evapotranspiration, Kp is pan coefficient and Ep is pan evaporation. Kp varies with relative humidity (R.H), pitch and wind speed (Table 5.1).

Table.5.1. Pan coefficient (Kp) for class A pan for different ground cover and levels of mean Relative humidity and 24 hour wind

Class A pan	Case A: pan placed in short green cropped area				Case B: pan placed in dry fallow area			
R Hmean %		Low <40	Medium 40-70	High >70		Low <40	Medium 40-70	High >70
Wind km/day	Wind ward side distance of green crop (m)				Wind ward side distance of dry fallow (m)			
Light <175	1	0.55	0.65	0.75	1	0.70	0.80	0.85
	10	0.65	0.75	0.85	10	0.60	0.70	0.80
	100	0.70	0.80	0.85	100	0.55	0.65	0.75
	1000	0.75	0.85	0.85	1000	0.50	0.60	0.70
Moderate 175-425	1	0.50	0.60	0.65	1	0.65	0.75	0.80
	10	0.60	0.70	0.75	10	0.55	0.65	0.70
	100	0.65	0.75	0.80	100	0.50	0.60	0.65
	1000	0.70	0.80	0.80	1000	0.45	0.55	0.60
Strong 425-700	1	0.45	0.50	0.60	1	0.60	0.65	0.70
	10	0.55	0.60	0.65	10	0.50	0.55	0.65
	100	0.60	0.65	0.70	100	0.45	0.50	0.60
	1000	0.65	0.70	0.75	1000	0.40	0.45	0.55
Very strong >700	1	0.40	0.45	0.50	1	0.50	0.60	0.65
	10	0.45	0.55	0.60	10	0.45	0.50	0.55
	100	0.50	0.60	0.65	100	0.40	0.45	0.50
	1000	0.55	0.60	0.65	1000	0.35	0.40	0.45

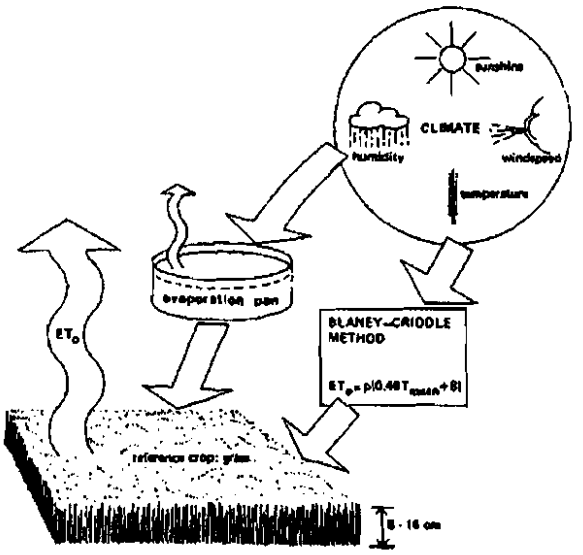


Figure 5.3 Assessment of ET0 from pan evaporation.

Many different types of evaporation pans are being used. The best known pan is the Class A evaporation pan (circular pan) (Figure 5.4).

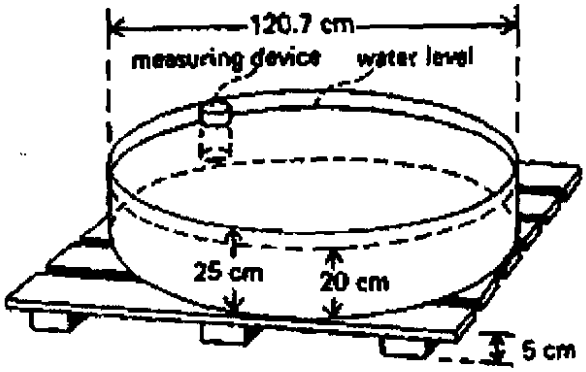


Figure 5.4 . Class A evaporation pan

The principle of the evaporation pan is the following:

- the pan is installed in the field
- the pan is filled with a known quantity of water (the surface area of the pan is known and the water depth is measured)
- water is allowed to evaporate during a certain period of time (usually 24 hours). For example, each morning at 7 o'clock a measurement is taken. The rainfall, if any, is measured simultaneously
- the amount of evaporation per unit time (the difference between the two measured water depths) is calculated; this is the pan evaporation: E pan (in mm/24 hours)
- the E pan is multiplied by a pan coefficient, K pan, to obtain the ETo.

Formula: ETo = K pan × E pan

with:
ETo: reference crop evapotranspiration
K pan: pan coefficient
E pan: pan evaporation

If the water depth in the pan drops too much (due to lack of rain), water is added (see Figure 5.5) and the water depth is measured before and after the water is added. If the water level rises too much (due to rain) water is taken out of the pan (see Figure 5.6) and the water depths before and after is measured.

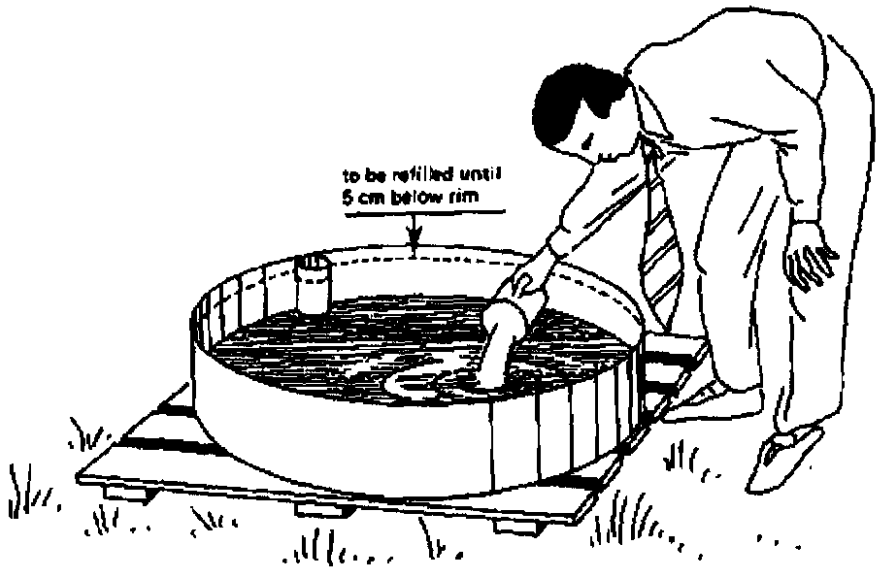


Fig. 5.5 Add water when the water depth in the pan drops too much

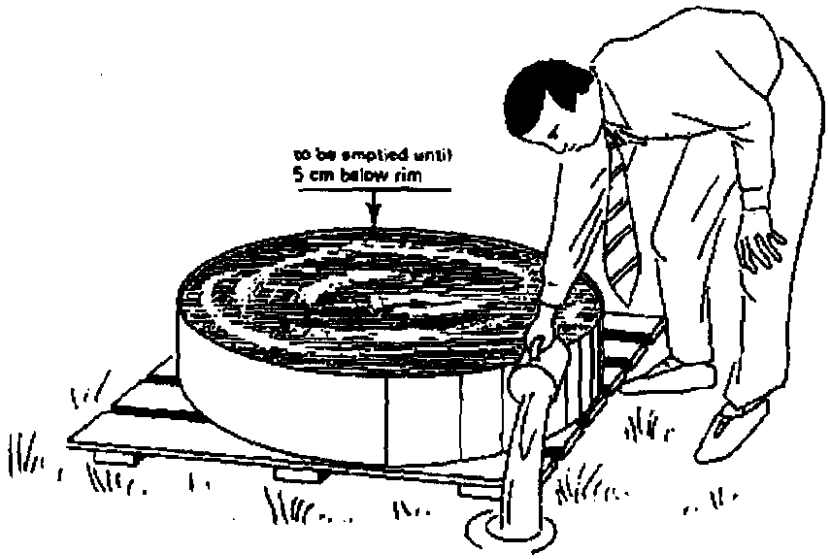


Figure 5.6 Take water out of the pan when the water depth rises too much

Determination of K pan

When using the evaporation pan to estimate the ETo, in fact, a comparison is made between the evaporation from the water surface in the pan and the evapotranspiration of the standard grass. Of course the water in the pan and the grass do not react in exactly the same way to the climate. Therefore a special coefficient is used (K pan) to relate one to the other.

The pan coefficient, K pan, depends on:

- the type of pan used
- the pan environment: if the pan is placed in a fallow or cropped area
- the climate: the humidity and windspeed

For the Class A evaporation pan, the K pan varies between 0.35 and 0.85. Average K pan = 0.70.

Table: 5.2

The K pan is high if:	The K pan is low if:
the pan is placed in a fallow area	The pan is placed in a cropped area
the humidity is high (i.e. humid)	The humidity is low (i.e. dry)
the windspeed is low	The windspeed is high

Details of the pan coefficient are usually provided by the supplier of the pan.
If the pan factor is not known the average value could be used (see box). If more accuracy is required, the pan factors given in Annex 1 should be applied. These values, however, only refer to the Class A evaporation pan.
Some examples

1) Type of pan: Class A evaporation pan
   Water depth in pan on day 1 = 150 mm
   Water depth in pan on day 2 = 144 mm (after 24 hours)
   Rainfall (during 24 hours) = 0 mm
   K pan = 0.75

Formula: ETo = K pan × E pan
Calculation: E pan = 150 - 144 = 6 mm/day
ETo = 0.75 × 6 =4.5 mm/day

Example. Class A Pan evaporation is 10 mm/day, R.H is 20% (Low), green crop up to 100 m and wind speed 7 m/s (strong), find ET0
From Table . we get pan coefficient is 0.6.

$$ET_0 = 0.6 \times 10 \text{ mm/day} = 6.0 \text{ mm/day}$$

Blaney Criddle-Method

$$ET_0 = p\,[\,0.46\,T + 8\,]$$

Where
p = mean daily percentage of total daytime hour obtained from Table 5.3
T= mean daily temperature in degree centigrade (C)

Example. The mean daily temperature of a place in May is 36 C with latitude 35 North, find ET_0.

The p from Table 5.3 is 0.31
$ET_0 = 0.31\ (0.46 \times 36 + 8) = 7.61$ mm/day

Table 5.3 Mean Daily Percentage (P) of Annual Daytime Hours For Different Latitudes in the Northern hemisphere

Latitude	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
60°	0.15	0.20	0.26	0.32	0.38	0.41	0.40	0.34	0.28	0.22	0.17	0.13
55	0.17	0.21	0.26	0.32	0.36	0.39	0.38	0.33	0.28	0.23	0.18	0.16
50	0.19	0.23	0.27	0.31	0.34	0.36	0.35	0.32	0.28	0.24	0.20	0.18
45	0.20	0.23	0.27	0.30	0.34	0.35	0.34	0.32	0.28	0.24	0.21	0.20
40	0.22	0.24	0.27	0.30	0.32	0.34	0.33	0.31	0.28	0.25	0.22	0.21
35	0.23	0.25	0.27	0.29	0.31	0.32	0.32	0.30	0.28	0.25	0.23	0.22
30	0.24	0.25	0.27	0.29	0.31	0.32	0.31	0.30	0.28	0.26	0.24	0.23
25	0.24	0.26	0.27	0.29	0.30	0.31	0.31	0.29	0.28	0.26	0.25	0.24
20	0.25	0.26	0.27	0.28	0.29	0.30	0.30	0.29	0.28	0.26	0.25	0.25
15	0.26	0.26	0.27	0.28	0.29	0.29	0.29	0.28	0.28	0.27	0.26	0.25
10	0.26	0.27	0.27	0.28	0.28	0.29	0.29	0.28	0.28	0.27	0.26	0.26
5	0.27	0.27	0.27	0.28	0.28	0.28	0.28	0.28	0.28	0.27	0.27	0.27
0	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27

Step 3: **Calculate ETo,** using the formula: ETo = p (0.46 T mean + 8)
For example, when p = 0.29 and T mean = 21.5°C the ETo is calculated as follows:
ETo = 0.29 (0.46 × 21.5 + 8) = 0.29 (9.89 + 8) = 0.29 × 17.89 = 5.2 mm/day
1.2.1 Calculation Example Blaney-Criddle
Given
Latitude - 35° North
Mean T max in April = 29.5°C
Mean T min in April = 19.4°C
Question
Determine for the month April the mean ETo in mm/day using the Blaney-Criddle method
Answer: Formula: ETo = p (0.46 T mean + 8)

Step 1: determine T mean:
Step 2: determine p: Latitude: 35° North
Month: April
From Table 5.3 p = 0.29

Step 3: calculate ETo: ETo = 0.29 (0.46 × 24.5 + 8) = 5.6 mm/day
Thus the mean reference crop evapotranspiration ETo =5.6 mm/day during the whole month of April.

Table 5.4. Indicative Values of ETo (mm/day)

	Mean daily temperature		
Climatic zone	low (less than 15°C)	medium (15-25°C)	high (more than 25°C)
Desert/arid	4-6	7-8	9-10
Semi arid	4-5	6-7	8-9
(Moist) Sub-humid	3-4	5-6	7-8
Humid	1-2	3-4	5-6

5.4 Hargreave's method

ET0 = 0.0135 (T + 17.78) Rs

T = mean daily temperature
Rs = incident solar radiation in Langleys/day
Rs = (0.25 + 0.5 n/N) Ra
n = actual sunshine in hours
N = maximum possible sunshine hours (Table 5.6)
Ra = Extra Terrestrial Radiation in mm/day (Table 5.5)

Example. The mean daily temperature of a place in May is 36 C located at altitude of 35 north, compute ET_0 by Hargreave method.
Assume no cloudiness (n/N = 1)
From Table 5.5 for latitude of 35 North and for the month of May Ra is 16.45 mm/day.

Table 5.5: Extra Terrestrial Radiation (Ra) expressed in equivalent evaporation in mm/day

Northern Hemisphere												Latitude
Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	
3.8	6.1	9.4	12.7	15.8	17.1	16.4	14.1	10.9	7.4	4.5	3.2	50°
4.3	6.6	9.8	13.0	15.9	17.2	16.5	14.3	11.2	7.8	5.0	3.7	48
4.9	7.1	10.2	13.3	16	17.2	16.6	14.5	11.5	8.3	5.5	4.3	46
5.3	7.6	10.6	13.7	16.1	17.2	16.6	14.7	11.9	8.7	6	4.7	44
5.9	8.1	11.0	14	16.2	17.3	16.7	15.0	12.2	9.1	6.5	5.2	42
6.4	8.6	11.4	14.3	16.4	17.3	16.7	15.2	12.5	9.6	7	5.7	40
6.9	9.0	11.8	14.5	16.4	17.2	16.7	15.3	12.8	10.0	7.5	6.1	38
7.4	9.4	12.1	14.7	16.4	17.2	16.7	15.4	13.1	10.6	8	6.6	36
7.9	9.8	12.4	14.8	16.5	17.1	16.8	15.5	13.4	10.8	8.5	7.2	34
8.3	10.2	12.8	15.0	16.5	17.0	16.8	15.6	13.6	11.2	9	7.8	32
8.8	10.7	13.1	15.2	16.5	17.0	16.8	15.7	13.9	11.6	9.5	8.3	30
9.3	11.1	13.4	15.3	16.5	16.8	16.7	15.7	14.1	12.0	9.9	8.8	28
9.8	11.5	13.7	15.3	16.4	16.7	16.6	15.7	14.3	12.3	10.3	9.3	26
10.2	11.9	13.9	15.4	16.4	16.6	16.5	15.8	14.5	12.6	10.7	9.7	24
10.7	12.3	14.2	15.5	16.3	16.4	16.4	15.8	14.6	13.0	11.1	10.2	22
11.2	12.7	14.4	15.6	16.3	16.4	16.3	15.9	14.8	13.3	11.6	10.7	20
11.6	13.0	14.6	15.6	16.1	16.1	16.1	15.8	14.9	13.6	12.0	11.1	18
12.0	13.3	14.7	15.6	16	15.9	15.9	15.7	15.0	13.9	12.4	11.6	16
12.4	13.6	14.9	15.7	15.8	15.7	15.7	15.7	15.1	14.1	12.8	12.0	14
12.8	13.9	15.1	15.7	15.7	15.5	15.5	15.6	15.2	14.4	13.3	12.5	12
13.2	14.2	15.3	15.7	15.5	15.3	15.3	15.5	15.3	14.7	13.6	12.9	10
13.6	14.5	15.3	15.6	15.3	15.0	15.1	15.4	15.3	14.8	13.9	13.3	8
13.9	14.8	15.4	15.4	15.1	14.7	14.9	15.2	15.3	15.0	14.2	13.7	6
14.3	15.0	15.5	15.5	14.9	14.4	14.6	15.1	15.3	15.1	14.5	14.1	4
14.7	15.3	15.6	15.3	14.6	14.2	14.3	14.9	15.3	15.3	14.8	14.4	2
15.0	15.5	15.7	15.3	14.4	13.9	14.1	14.8	15.3	15.4	15.1	14.8	0

Table 5.6. Mean daily duration of maximum possible sunshine hours (N) for different months and Latitudes

Northern	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
50	8.5	10.1	11.8	13.8	15.4	16.3	15.9	14.5	12.7	10.8	9.1	8.1
48	8.8	10.2	11.8	13.6	15.2	16.0	15.6	14.3	12.6	10.9	9.3	8.3
46	9.1	10.4	11.9	13.5	14.9	15.7	15.4	14.2	12.6	10.9	9.5	8.7
44	9.3	10.5	11.9	13.4	14.7	15.4	15.2	14.0	12.6	11.0	9.7	8.9
42	9.4	10.6	11.9	13.4	14.6	15.2	14.9	13.9	12.6	11.1	9.8	9.1
40	9.6	10.7	11.9	13.3	14.4	15.0	14.7	13.7	12.5	11.2	10.0	9.3
35	10.1	11.0	11.9	13.1	14.0	14.5	14.3	13.5	12.4	11.3	10.3	9.8
30	10.4	11.1	12.0	12.9	13.6	14.0	13.9	13.2	12.4	11.5	10.6	10.2
25	10.7	11.3	12.0	12.7	13.3	13.7	13.5	13.0	12.3	11.6	10.9	10.6
20	11.0	11.5	12.0	12.6	13.1	13.3	13.2	12.8	12.3	11.7	11.2	10.9
15	11.3	11.6	12.0	12.5	12.8	13.0	12.9	12.6	12.2	11.8	11.4	11.2
10	11.6	11.8	12.0	12.3	12.6	12.7	12.6	12.4	12.1	11.8	11.6	11.5
5	11.8	11.9	12.0	12.2	12.3	12.4	12.3	12.3	12.1	12.0	11.9	11.8
0	12.1	12.1	12.1	12.1	12.1	12.1	12.1	12.1	12.1	12.1	12.1	12.1

$Rs = (0.25 + 0.5 \times 1) \times 16.45 = 12.34$ mm/day

$ET_0 = 0.0135 (36 + 17.78) \times 12.34 = 8.96$ mm/day

5.5 Jensen-Haise's Method

$ET_0 = Rs (0.025 T + 0.08)$
Rs = incident solar radiation in Langleys/day
T = mean daily temperature in degree C

Example . The mean daily temperature of a place in May is 36 C located at altitude of 35 north, compute ET_0 by Jensen-Haise method.
Assume no cloudiness (n/N = 1)
From Table 5.5 for latitude of 35 North and for the month of May Ra is 16.45 mm/day.
Assume n/N = 1
$Rs = 16.45 (0.25 + 0.5 \times 1) = 12.34$ mm/day
$ET_0 = 12.34 (0.025 \times 36 + 0.08) = 12.09$ mm/day

5.6 Crop Evapotranspiration (Etc)

$ETc = Kc \times ET_0$
Where
Kc is crop coefficient (Table 5.9) for different growth stages.

5.7 Net Irrigation Requirements

Net irrigaion = ETa - ER
Where
ETa = actual evapotranpiration and
ER = effective rainfall
Effective rainfall is taken some fixed percentage of rainall.
Both ET crop and ETo are expressed in the same unit: usually in mm/day (as an average for a period of one month) or in mm/month.

The crop factor, Kc, mainly depends on:
- the type of crop
- the growth stage of the crop
- te climate

Kc and the type of crop
Fully developed maize, with its large leaf area will be able to transpire, and thus use more water than the reference grass crop: Kc, maize is higher than 1. Cucumber, also fully developed, will use less water than the reference grass crop: Kc, cucumber is less than 1.

Kc and the growth stage of the crop
A certain crop will use more water once it is fully developed, compared to a crop which has just recently been planted.

Kc and the climate
The climate influences the duration of the total growing period and the various growth stages. In a cool climate a certain crop will grow slower than in a warm climate.
Thus, to determine the crop factor Kc, it is necessary, for each crop, to know the total length of the growing season and the lengths of the various growth stages.
The determination of the Kc values for the various growth stages of the crops involves several steps:
Step 1 - Determination of the total growing period of each crop
Step 2 - Determination of the various growth stages of each crop
Step 3 - Determination of the Kc values for each crop for each of the growth stages

Determination of the Total Growing Period
The total growing period (in days) is the period from sowing or transplanting to the last day of the harvest. It is mainly dependent on:

- the type of crop and the variety
- the climate and
- the planting date

As the growing period heavily depends on local circumstances (e.g. local crop varieties) it is always best to obtain these data locally. Only if no data are available locally should Table 5.7 be used.
As can be seen from Table 5.7 there is a large variation of values not only between crops but also within one crop type. In general it can be assumed that the growing period for a certain crop is longer when the climate is cool and shorter when the climate is warm.

Table 5.7 Indicative values of the total growing period

Crop	Total growing period (days)	Crop	Total growing period (days)
Alfalfa	100-365	Millet	105-140
Banana	300-365	Onion green	70-95
Barley/Oats/Wheat	120-150	Onion dry	150-210
Bean green	75-90	Peanut/Groundnut	130-140
Bean dry	95-110	Pea	90-100
Cabbage	120-140	Pepper	120-210
Carrot	100-150	Potato	105-145
Citrus	240-365	Radish	35-45
Cotton	180-195	Rice	90-150
Cucumber	105-130	Sorghum	120-130
Eggplant	130-140	Soybean	135-150
Flax	150-220	Spinach	60-100
Grain/small	150-165	Squash	95-120
Lentil	150-170	Sugarbeet	160-230
Lettuce	75-140	Sugarcane	270-365
Maize sweet	80-110	Sunflower	125-130
Maize grain	125-180	Tobacco	130-160
Melon	120-160	Tomato	135-180

Determination of the Growth Stages

Once the total growing period is known, the duration (in days) of the various growth stages has to be determined.

The total growing period is divided into 4 growth stages.

1. **The initial stage:** this is the period from sowing or transplanting until the crop covers about 10% of the ground.
2. **The crop development stage:** this period starts at the end of the initial stage and lasts until the full ground cover has been reached (ground cover 70-80%); it does not necessarily mean that the crop is at its maximum height.
3. The **mid - season stage:** this period starts at the end of the crop development stage and lasts until maturity; it includes flowering and grain-setting.
4. The **late season stage:** this period starts at the end of the mid season stage and lasts until the last day of the harvest; it includes ripening.

Table 5.8 shows the duration of the various growth stages for some of the major field

crops. For each crop the "minimum" and "maximum" duration of total growing period have been taken and sub-divided in the various growth stages.

Table 5.8 Approximate duration of growth stages for various field crops

	Total	Initial stage	Crop Development stage	Mid season stage	Late season stage
Barley/Oats/Wheat	120	15	25	50	30
	150	15	30	65	40
Bean/green	75	15	25	25	10
	90	20	30	30	10
Bean/dry	95	15	25	35	20
	110	20	30	40	20
Cabbage	120	20	25	60	15
	140	25	30	65	20
Carrot	100	20	30	30	20
	150	25	35	70	20
Cotton/Flax	180	30	50	55	45
	195	30	50	65	50
Cucumber	105	20	30	40	15
	130	25	35	50	20
Eggplant	130	30	40	40	20
	140	30	40	45	25
Grain/small	150	20	30	60	40
	165	25	35	65	40
Lentil	150	20	30	60	40
	170	25	35	70	40
Lettuce	75	20	30	15	10
	140	35	50	45	10
Maize, sweet	80	20	25	25	10
	110	20	30	50	10
Maize, grain	125	20	35	40	30
	180	30	50	60	40
Melon	120	25	35	40	20

	Total	Initial stage	Crop Development stage	Mid season stage	Late season stage
Millet	105	15	25	40	25
	140	20	30	55	35
Onion/green	70	25	30	10	5
	95	25	40	20	10
Onion/dry	150	15	25	70	40
	210	20	35	110	45
Peanut/Groundnut	130	25	35	45	25
	140	30	40	45	25
Pea	90	15	25	35	15
	100	20	30	35	15
Pepper	120	25	35	40	20
	210	30	40	110	30
Potato	105	25	30	30	20
	145	30	35	50	30
Radish	35	5	10	15	5
	40	10	10	15	5
Sorghum	120	20	30	40	30
	130	20	35	45	30
Soybean	135	20	30	60	25
	150	20	30	70	30
Spinach	60	20	20	15	5
	100	20	30	40	10
Squash	95	20	30	30	15
	120	25	35	35	25
Sugarbeet	160	25	35	60	40
	230	45	65	80	40
Sunflower	125	20	35	45	25
	130	25	35	45	25
Tomato	135	30	40	40	25
	180	35	45	70	30

Example. Carrots: the "minimum" growing period is 100 days. This growing period corresponds with the following duration of growth stages:

Initial stage	: 20 days
Crop development stage	: 30 days
Mid-season stage	: 30 days
Late season stage	: 20 days
Total	100 days

For the "maximum" growing period of 150 days the following values apply: respectively 25, 35, 70 and 20 days.
Should, under certain local circumstances, the duration of the growing period be 120 days, the duration of the growth stages could be estimated as follows:

Initial stage	: 25 days
Crop development stage	: 35 days
Mid-season stage	: 40 days
Late season stage	: 20 days
	Total 120 days

With respect to Table 5.8 the following should be noted:
The table always refers to "sown" crops. When the crop is transplanted, the length of the initial stage should be reduced. For example:

Tomatoes:	growing period 180 days from sowing
Direct sowing:	initial stage 35 days
Transplanted:	(estimated) initial stage 15 days

The growing period from transplant is thus (180 - 20) = 160 days

Direct sowing			
Initial stage	crop development stage	mid-season stage	late season stage
35 days	45 days	70 days	30 days
Transplanting			
Initial. Stage	crop development stage	mid-season stage	late season stage
15 days	45 days	70 days	30 days

When a crop is harvested "green" or "fresh" the late season stage is short.

Compare, for example, green beans with dry beans (Table 5.8). The duration of the late season stage is 10 and 20 days respectively.
If a crop is planted in the winter or is growing in the cool season the total growing period is long. The same is the case with the individual lengths of growing stages. The difference will be most pronounced for the stage during which the temperature is the lowest.
It should be kept in mind that the influence of variations in the total growing period on the crop water need is very important. Less important is the choice of the various lengths of growth stages.
In other words: it is important to obtain (preferably locally) an accurate estimate of the total growing period. The duration of the four growth stages can be estimated with the help of Table 5.8. Note: The sum of the four growth stages should always equal the total growing period.

Example. Estimate the duration of the four growth stages for the following crops: cotton (190 days), lentils (160 days), sweet maize (100 days), potatoes (130 days), tomatoes/transplanted (140 days from transplant), etc. (the figures in brackets refer to the total growing period).

Crop	Initial stage (day)	Crop dev. stage (days)	Mid-season stage (days)	Late season stage (days)
Cotton (190 days)	30	50	60	50
Lentils (160 days)	25	30	65	40
Sweet Maize (100 days)	20	30	40	10
Potatoes (130 days)	30	35	40	25
Tomatoes/Transplanted (140 days from transplant)	15	40	60	25

5.8 Determination of Crop Factors

Per crop, **four** crop factors have to be determined: one crop factor for each of the **four** growth stages. Table 5.9 indicates per crop the Kc values for each of the four growth stages.

Table 5.9 shows average Kc values for the various crops and growth stages. In fact, the Kc is also dependent on the climate and, in particular, on the relative humidity and the windspeed. The values indicated above should be reduced by 0.05 if the relative humidity is high (RH > 80%) and the windspeed is low (u < 2 m/sec), e.g. Kc = 1.15 becomes Kc = 1.10. The values should be increased by 0.05 if the relative humidity is low (RH < 50%) and the windspeed is high (u > 5 m/sec), e.g. Kc = 1.05 becomes Kc = 1.10.

Table 5.9 Values of the crop factor (kc) for various crops and growth stages

Crop	Initial stage	Crop dev. Stage	Mid-season stage	Late season stage
Barley/Oats/Wheat	0.35	0.75	1.15	0.45
Bean, green	0.35	0.70	1.10	0.90
Bean, dry	0.35	0.70	1.10	0.30
Cabbage/Carrot	0.45	0.75	1.05	0.90
Cotton/Flax	0.45	0.75	1.15	0.75
Cucumber/Squash	0.45	0.70	0.90	0.75
Eggplant/Tomato	0.45	0.75	1.15	0.80
Grain/small	0.35	0.75	1.10	0.65
Lentil/Pulses	0.45	0.75	1.10	0.50
Lettuce/Spinach	0.45	0.60	1.00	0.90
Maize, sweet	0.40	0.80	1.15	1.00
Maize, grain	0.40	0.80	1.15	0.70
Melon	0.45	0.75	1.00	0.75
Millet	0.35	0.70	1.10	0.65
Onion, green	0.50	0.70	1.00	1.00
Onion, dry	0.50	0.75	1.05	0.85
Peanut/Groundnut	0.45	0.75	1.05	0.70
Pea, fresh	0.45	0.80	1.15	1.05
Pepper, fresh	0.35	0.70	1.05	0.90
Potato	0.45	0.75	1.15	0.85
Radish	0.45	0.60	0.90	0.90
Sorghum	0.35	0.75	1.10	0.65
Soybean	0.35	0.75	1.10	0.60
Sugarbeet	0.45	0.80	1.15	0.80
Sunflower	0.35	0.75	1.15	0.55
Tobacco	0.35	0.75	1.10	0.90

1.1 Calculation of the Crop Water Needs

This section explains how the crop water need ET crop is calculated on a monthly basis, using the formula: ET crop = ETo × Kc.

Although the formula to calculate ET crop is easy to apply, there are still some practical problems to be overcome, which can best be explained using an example.

Crop Water Need Calculation Example

Determine the crop water needs of tomatoes in Kabul

GIVEN

Month	Mar	Apr May	Jun	Jul	Aug	Sep
Eto (mm/day)	2.2	3.2 4.8	6.3	6.7	5.8	4.5
Humidity	medium	(60%)				
Windspeed	medium	(3 m/sec)				
Duration of growing period (from sowing): 150 days						
Planting date: 1 March (direct sowing)						

CALCULATION

Step 1: Estimate the duration of the various growth stages, using Table 5.8.

Crop	Total growing period (days)	Initial stage	Crop dev. Stage	Mid-season stage	Late season stage
Tomatoes	150	35	40	50	25

Step 2: Indicate on table, as per example below, the ETo values and the duration of the growth stages.

Note: When calculating the crop water needs, all months are assumed to have 30 days. For the calculation of the reference crop evapotranspiration (ETo), the actual number of days of each month is used e.g., January 31 days, February 28 or 29 days, etc.

Planting date	1 March
Initial stage, 35 days	1 Mar-5 Apr
Crop development stage, 40 days	6 Apr-15 May
Mid season stage, 50 days	16 May-5 Jul
Late season stage, 25 days	6 Jul-30 Aug
Last day of the harvest	30 Aug

Step 3: Estimate the Kc factor for each of the 4 growth stages, using Table 5.9 and bearing in mind that the humidity and windspeed are medium

Kc, initial stage = 0.45

Kc, crop development stage = 0.75

Kc, mid season stage = 1.15

Kc, late season stage = 0.8

The Kc values are inserted in the table

It can be seen from the table above that the months and growth stages do not correspond. As a consequence the ETo and the Kc values do not correspond.
Yet the ET crop (= ETo × Kc) has to be determined on a monthly basis. It is thus necessary to determine the Kc on a monthly basis, which is done as follows:
(see also the table above)

March: Kc Mar = 0.45

April: 5 days: Kc = 0.45
25 days: Kc = 0.75

NOTE: The Kc values are rounded to the nearest 0.05 or 0.00.
Thus Kc, March = 0.70

May: 15 days: Kc = 0.75
15 days: Kc = 1.15

Thus Kc, April =0.95

June: Kc, May = 1.15

July: 5 days: Kc = 1.15
25 days: Kc = 0.80

Thus Kc, July =0.85

In summary:

Step 4: Calculate, on a monthly basis, the crop water need, using the formula:
ET crop = ETo × Kc (mm/day)

March: ET crop = 2.2 × 0.45 = 1.0 mm/day

April: ET crop = 3.2 × 0.70 = 2.2 mm/day

May: ET crop = 4.8 × 0.95 = 4.6 mm/day

June: ET crop = 6.7 × 1.15 = 6.0 mm/day

July ET crop = 6.7 × 0.85 = 5.7 mm/day

In summary:

Step 5: Calculate the monthly and seasonal crop water needs.
Note: all months are assumed to have 30 days.

March: ET crop = 30 × 1.0 = 30 mm/month

April: ET crop = 30 × 2.2 = 66 mm/month

May: ET crop = 30 × 4.6. = 138 mm/month

June: ET crop = 30 × 6.0 = 180 mm/month

July ET crop = 30 × 5.7 = 171 mm/month

The crop water need for the whole growing season of tomatoes is 585 mm.

Special Cases

In the previous sections it has been indicated how the crop water need is calculated for a variety of field crops: ET crop = Kc × ETo. However there are some crops that do not directly fit this model: their crop factor Kc is determined in a different way. Once the crop factor Kc has been determined, the same procedure is used to calculate the crop water needs.

The special cases include:

alfalfa, pasture and clover

rice

sugarcane

olives

grapes

Alfalfa, Pasture and Clover

Alfalfa, pasture and clover are regularly cut during the year. Just after cutting they are in the "initial stage", while just before the next cutting, they are in the "late season stage". To determine the crop water need it is best to use an average value of the crop factor Kc. The average Kc values are given in Table 5.9

Table 5.10 Kc values for alfalfa. Pasture and clover

Climate:	Alfalfa	Pasture	Clover
Humid Light/medium wind	0.85	0.95	1.0
Dry Light/medium wind	0.95	1.0	1.05
Humid or Dry Strong wind	1.05	1.05	1.05

Citrus

The crop factor Kc for clean cultivated citrus is 0.70 year-round. This value is applicable for large mature trees, covering some 70% of the ground surface. If there is no weed control, a Kc value of 0.90 year-round should be used.

Rice

For paddy rice the values indicated in Table 5.11 should be used.

Table 5.11 Kc values for paddy rice

Climate	Little wind		Strong wind	
Growth stage (days)	dry	humid	Dry	Humid
0-60 days after transplant or direct sowing	1.1	1.1	1.1	1.1
Mid-season	1.2	1.05	1.35	1.3
last 30 days before harvest	1.0	1.0	1.0	1.0

Example: Suppose the total growing season of rice from sowing to harvest =150 days in a humid climate with strong wind

Kc day 0 to 60: 1.1

Kc day 60 to 120: 1.3

Kc day 120 to 150: 1.0

Sugarcane

Crop coefficients for sugarcane vary widely depending on climate and sugarcane variety. It is best to use locally available data. If such data are not available. Tables 5.9 and 5.8 can be used.

Table 5.12a refers to a virgin sugarcane crop from establishment to first harvest, which is assumed to take 18 months. Table 5.12b refers to a ratoon crop, which is the regrowth after the harvest of the virgin crop. The regrowth is assumed to take 12 months. Sometimes, depending on local practices, a virgin crop is followed by 2 or 3 ratoon crops.

Table 5.12a Kc values for virgin sugarcane

Climate	Little wind		Strong wind	
Growth stage (days)	dry	humid	dry	humid
0-2	0.4	0.5	0.5	0.6
2-4	0.8	0.8	0.8	0.85
4-6	1.1	1.0	1.2	1.1
6-12	1.25	1.05	1.3	1.15
12-17	0.95	0.8	1.05	0.85
17-18	0.7	0.6	0.75	0.65

Growth stage (days)	dry	humid	dry	humid
0-1	0.4	0.5	0.5	0.6
1-2	0.8	0.8	0.8	0.85
2-4	1.1	1.0	1.2	1.1
4-10	1.25	1.05	1.3	1.15
10-11	0.95	0.8	1.05	0.85
11-12	0.7	0.6	0.75	0.65

Olives

For mature olive trees the following year-round Kc values are recommended:

- wide spacing of trees: Kc = 0.4
- close spacing of trees: Kc = 0.7

Grapes

With the assumption that the grape harvest starts some 5 months after the first leaves appear, the Kc values indicated in Table 5.12c can be used for the months starting with the first leaf appearance.

Table 5.12c Kc values for grapes (35% ground cover)

Months after first leaves appear	1	2	3	4	5	6	7	8	9
Kc	0.25	0.45	0.65	0.75	0.75	0.7	0.55	0.45	0.35

Indicative Values of Crop Water Needs

Table 5.13 gives indicative values of the crop water needs for the total growing period of various important field crops. The values indicated in the table provide a rough estimate and should only be used if the crop water needs cannot be calculated more accurately due to lack of data.

Table 5.13 Indicative values of crop water needs and sensitivity to drought

Crop	Crop water need (mm/total growing period)	Sensitivity to drought
Alfalfa	800-1600	low-medium
Banana	1200-2200	High
Barley/Oats/Wheat	450-650	low-medium
Bean	300-500	medium-high
Cabbage	350-500	medium-high
Citrus	900-1200	low-medium
Cotton	700-1300	Low
Maize	500-800	medium-high
Melon	400-600	medium-high
Onion	350-550	medium-high
Peanut	500-700	low-medium
Pea	350-500	medium-high
Pepper	600-900	medium-high
Potato	500-700	High
Rice (paddy)	450-700	High
Sorghum/Millet	450-650	Low
Soybean	450-700	low-medium
Sugarbeet	550-750	low-medium
Sugarcane	1500-2500	High
Sunflower	600-1000	low-medium
Tomato	400-800	medium-high

6 SOIL AND WATER

6.1 Soil Composition

When dry soil is crushed in the hand, it can be seen that it is composed of all kinds of particles of different sizes. Most of these particles originate from the degradation of rocks; they are called mineral particles. Some originate from residues of plants or animals (rotting leaves, pieces of bone, etc.), these are called organic particles (or organic matter). The soil particles seem to touch each other, but in reality have spaces in between. These spaces are called pores. When the soil is "dry", the pores are mainly filled with air. After irrigation or rainfall, the pores are mainly filled with water. Living material is found in the soil. It can be live roots as well as beetles, worms, larvae etc. They help to aerate the soil and thus create favorable growing conditions for the plant roots (Fig. 6.1).

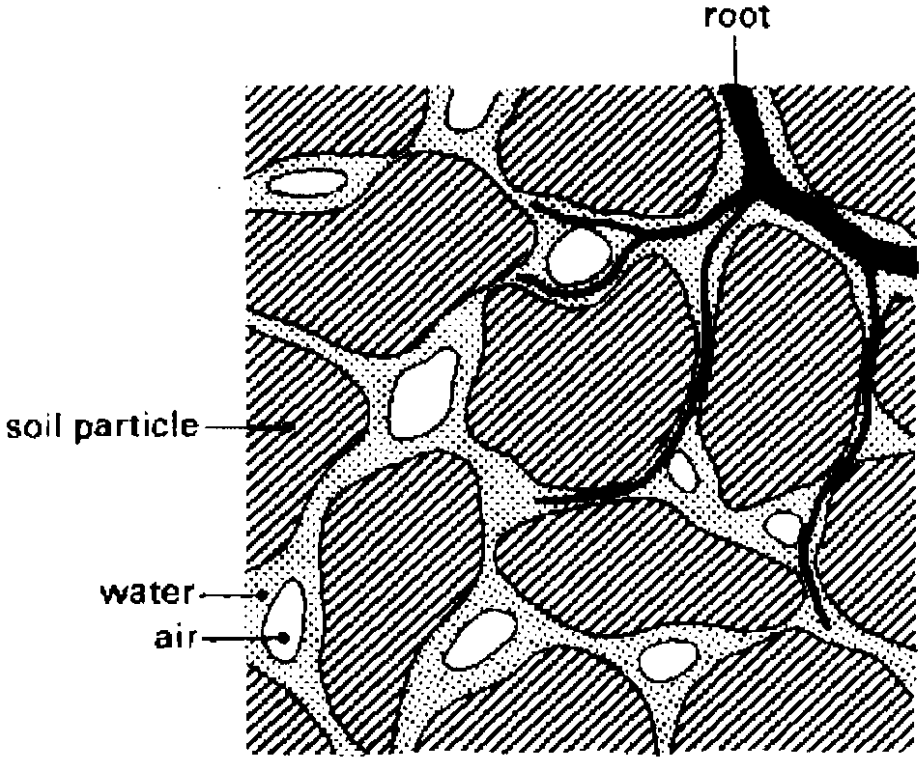


Figure 6.1. The composition of the soil

Soil profile

If a pit is dug in the soil, at least 1 m deep, various layers, different in color and composition can be seen. These layers are called horizons. This succession of horizons is called the profile of the soil (Fig. 6.2).

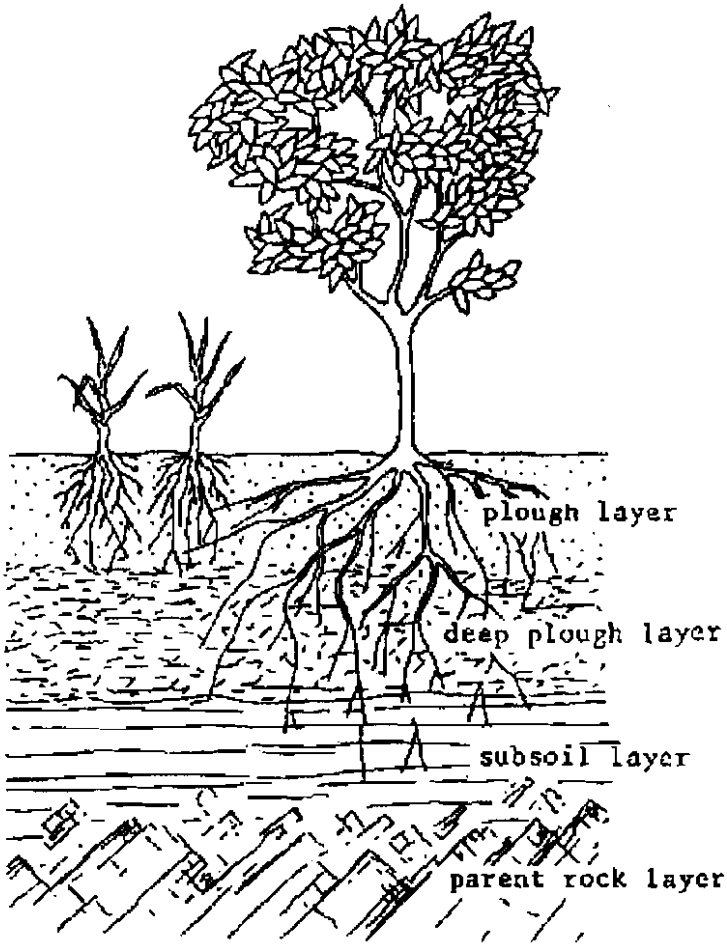


Figure 6.2. The soil profile

A very general and simplified soil profile can be described as follows:
The plough layer (20 to 30 cm thick): is rich in organic matter and contains many live roots. This layer is subject to land preparation (e.g. ploughing, harrowing etc.) and often has a dark color (brown to black).
The deep plough layer: contains much less organic matter and live roots. This layer is hardly affected by normal land preparation activities. The colour is lighter, often grey. and sometimes mottled with yellowish or reddish spots.
The subsoil layer: hardly any organic matter or live roots are to be found. This layer is not very important for plant growth as only a few roots will reach it.
The parent rock layer: consists of rock, from the degradation of which the soil was formed. This rock is sometimes called parent material. The depth of the different layers varies widely: some layers may be missing altogether.

6.2 Soil texture

The mineral particles of the soil differ widely in size and can be classified as follows:

Table 6.1 Sizes of different material particles

Name of the particles	Size limits in mm	Distinguisable with naked eye
Gravel	larger than 1	Obviously
Sand	1 to 0.5	Easily
Silt	0.5 to 0.002	Barely
Clay	less than 0.002	Impossible

The amount of sand, silt and clay present in the soil determines the soil texture.
In coarse textured soils: sand is predominant (sandy soils).
In medium textured soils: silt is predominant (loamy soils).
In fine textured soils: clay is predominant (clayey soils).
Farmers often talk of light soil and heavy soil. A coarse-textured soil is light because it is easy to work, while a fine-textured soil is heavy because it is hard to work.

Expression used by the farmer	Expression used in literature	
Light	Sandy	Coarse
Medium	loamy	Medium
Heavy	clayey	Fine

The texture of a soil is permanent, the farmer is unable to modify or change it.

6.3 Soil Structure

Soil structure refers to the grouping of soil particles (sand, silt, clay, organic matter and fertilizers) into porous compounds. These are called aggregates. Soil structure also refers to the arrangement of these aggregates separated by pores and cracks (Fig. 6.3).
The basic types of aggregate arrangements are shown in Fig. 6.4 granular, blocky, prismatic, and massive structure.

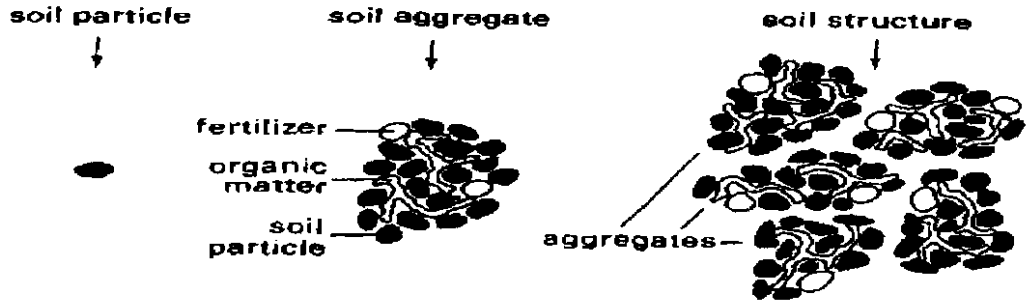
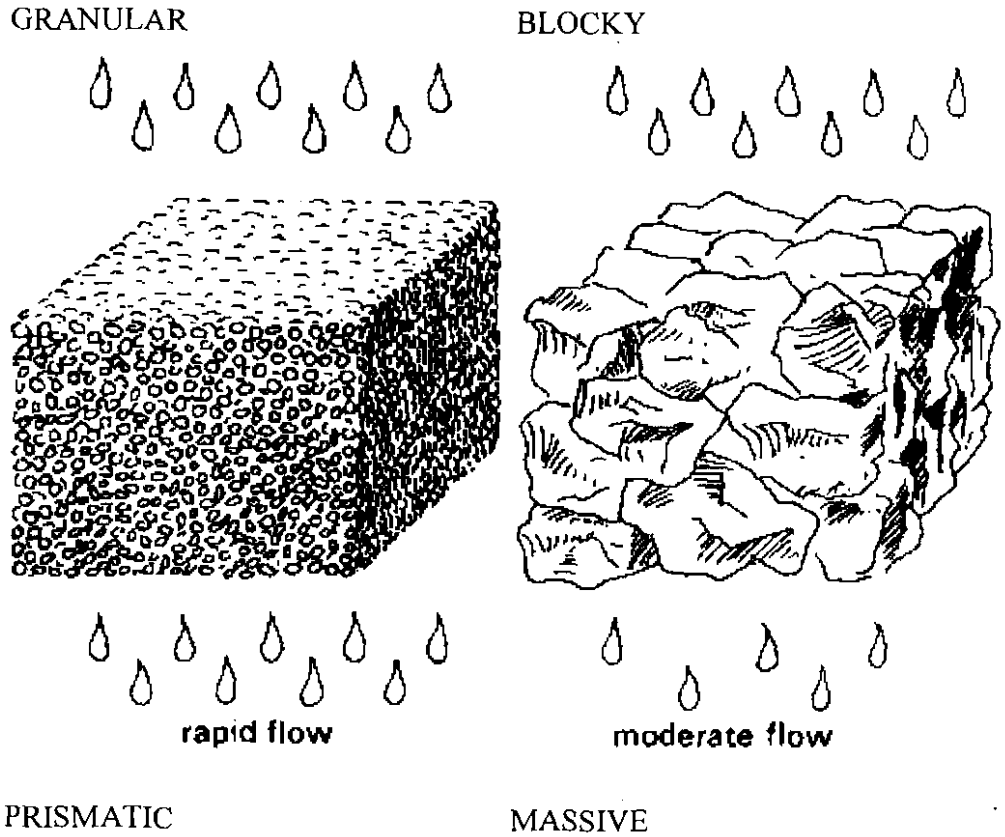


Figure . 6.3. The soil structure

When present in the topsoil, a massive structure blocks the entrance of water; seed germination is difficult due to poor aeration. On the other hand, if the topsoil is granular, the water enters easily and the seed germination is better.
In a prismatic structure, movement of the water in the soil is predominantly vertical and therefore the supply of water to the plant roots is usually poor.
Unlike texture, soil structure is not permanent. By means of cultivation practices (ploughing, ridging, etc.), the farmer tries to obtain a granular topsoil structure for his fields.



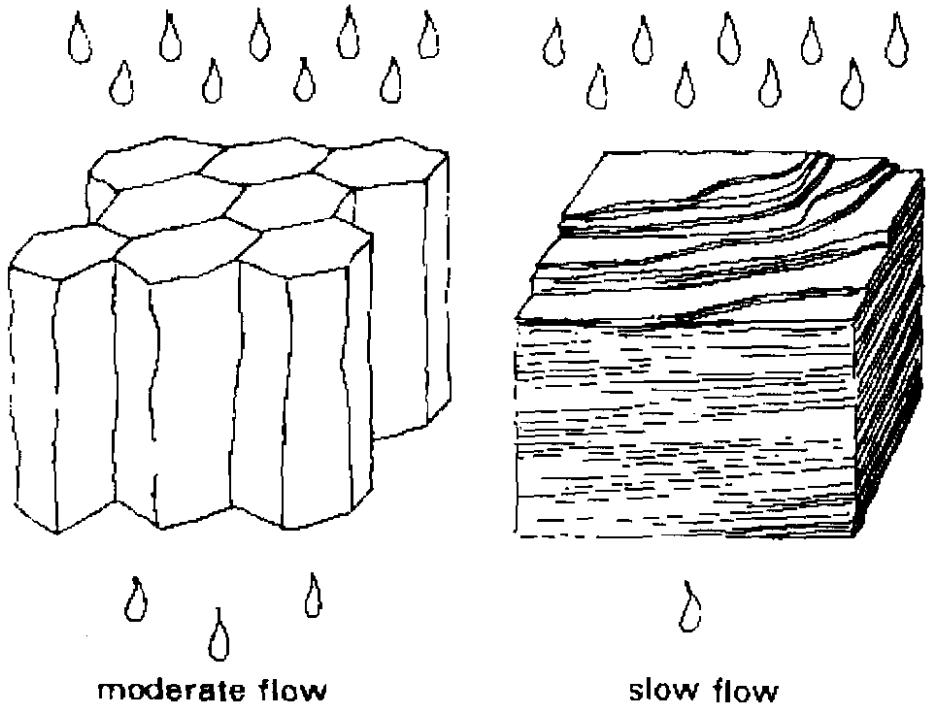


Figure 6.4. Some examples of soil structures

6.4 Entry Of Water Into The Soil

The infiltration process

When rain or irrigation water is supplied to a field, it seeps into the soil. This process is called infiltration.

Infiltration can be visualized by pouring water into a glass filled with dry powdered soil, slightly tamped. The water seeps into the soil; the color of the soil becomes darker as it is wetted (see Fig. 6.5).

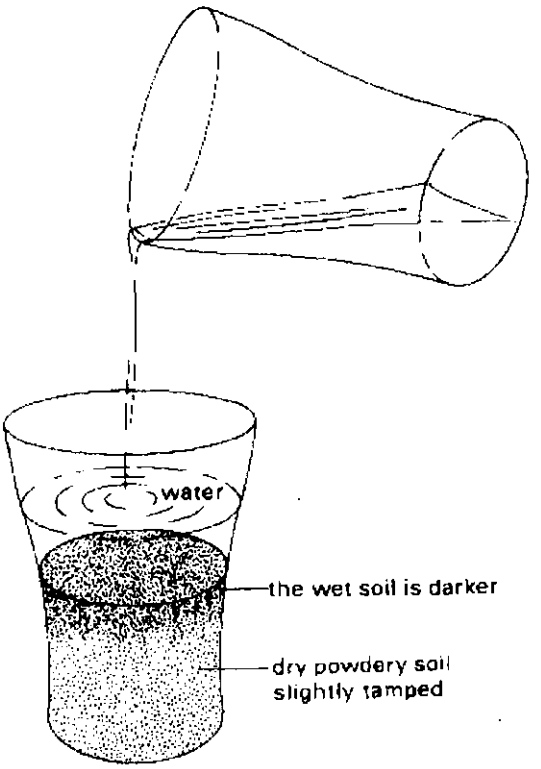
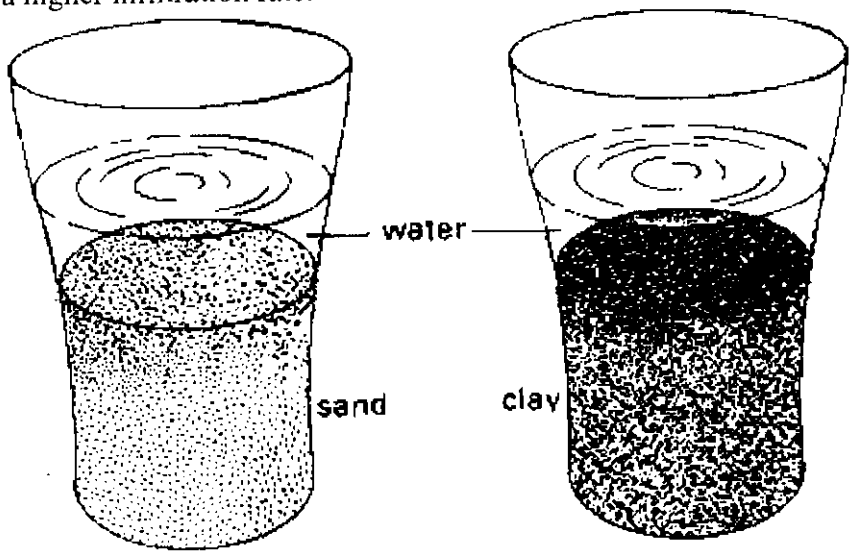


Figure 6.5. Infiltration of water into the soil

Infiltration Rate

Repeat the previous test, this time with two glasses. One is filled with dry sand and the other is filled with dry clay (see Fig. 6.6).

The infiltration of water into the sand is faster than into the clay. The sand is said to have a higher infiltration rate.



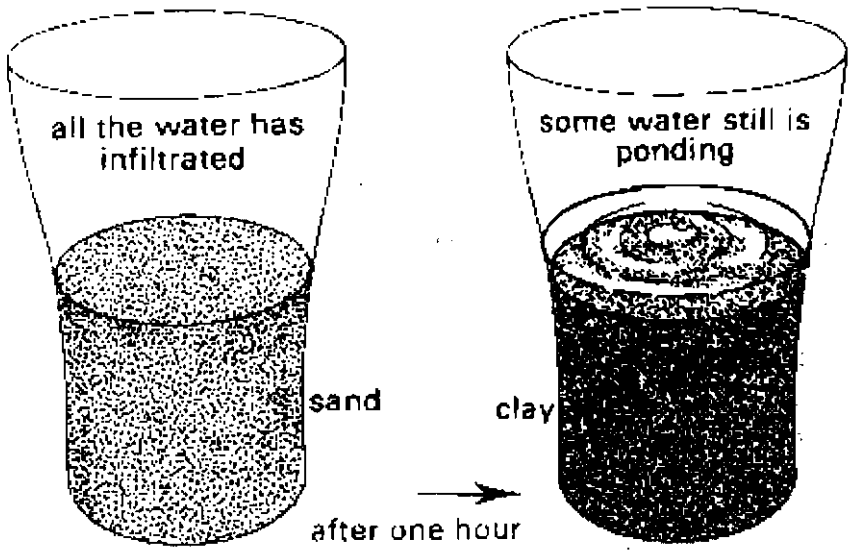


Fig. 6.6. After one hour the water has infiltrated in the sand, while some water is still ponding on the clay

The infiltration rate of a soil is the velocity at which water can seep into it. It is commonly measured by the depth (in mm) of the water layer that the soil can absorb in an hour.
An infiltration rate of 15 mm/hour means that a water layer of 15 mm on the surface of the soil, will take one hour to infiltrate (see Figure 6.7).

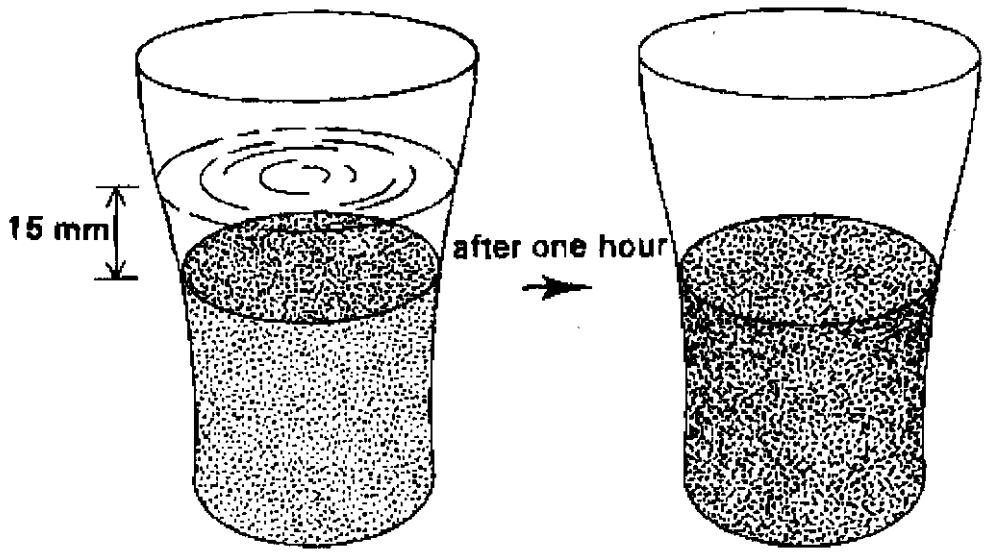


Figure 6.7. Soil with an infiltration rate of 15 mm/hour

A range of values for infiltration rates is given below:

Low infiltration rate	less than 15 mm/hour
medium infiltration rate	15 to 50 mm/hour
high infiltration rate	more than 50 mm/hour

Factors influencing the infiltration rate

The infiltration rate of a soil depends on factors that are constant, such as the soil texture. It also depends on factors that vary, such as the soil moisture content.

i. Soil Texture

Coarse textured soils have mainly large particles in between which there are large pores. On the other hand, fine textured soils have mainly small particles in between which there are small pores (see Fig. 6.8).

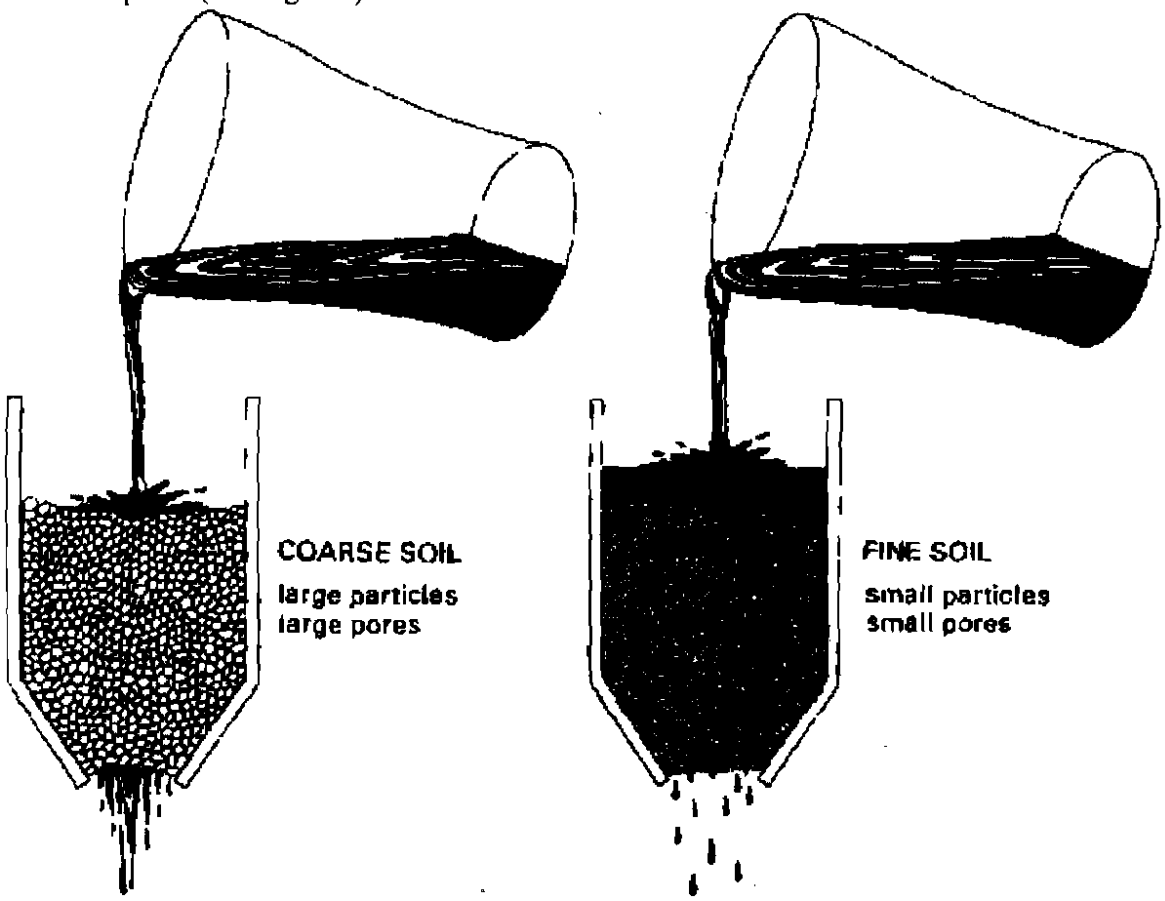


Fig. 6.8. Infiltration rate and soil texture

In coarse soils, the rain or irrigation water enters and moves more easily into larger pores; it takes less time for the water to infiltrate into the soil. In other words, infiltration rate is higher for coarse textured soils than for fine textured soils.

ii. The soil moisture content

The water infiltrates faster (higher infiltration rate) when the soil is dry, than when it is wet (see Fig. 6.9). As a consequence, when irrigation water is applied to a field, the water at first infiltrates easily, but as the soil becomes wet, the infiltration rate decreases.

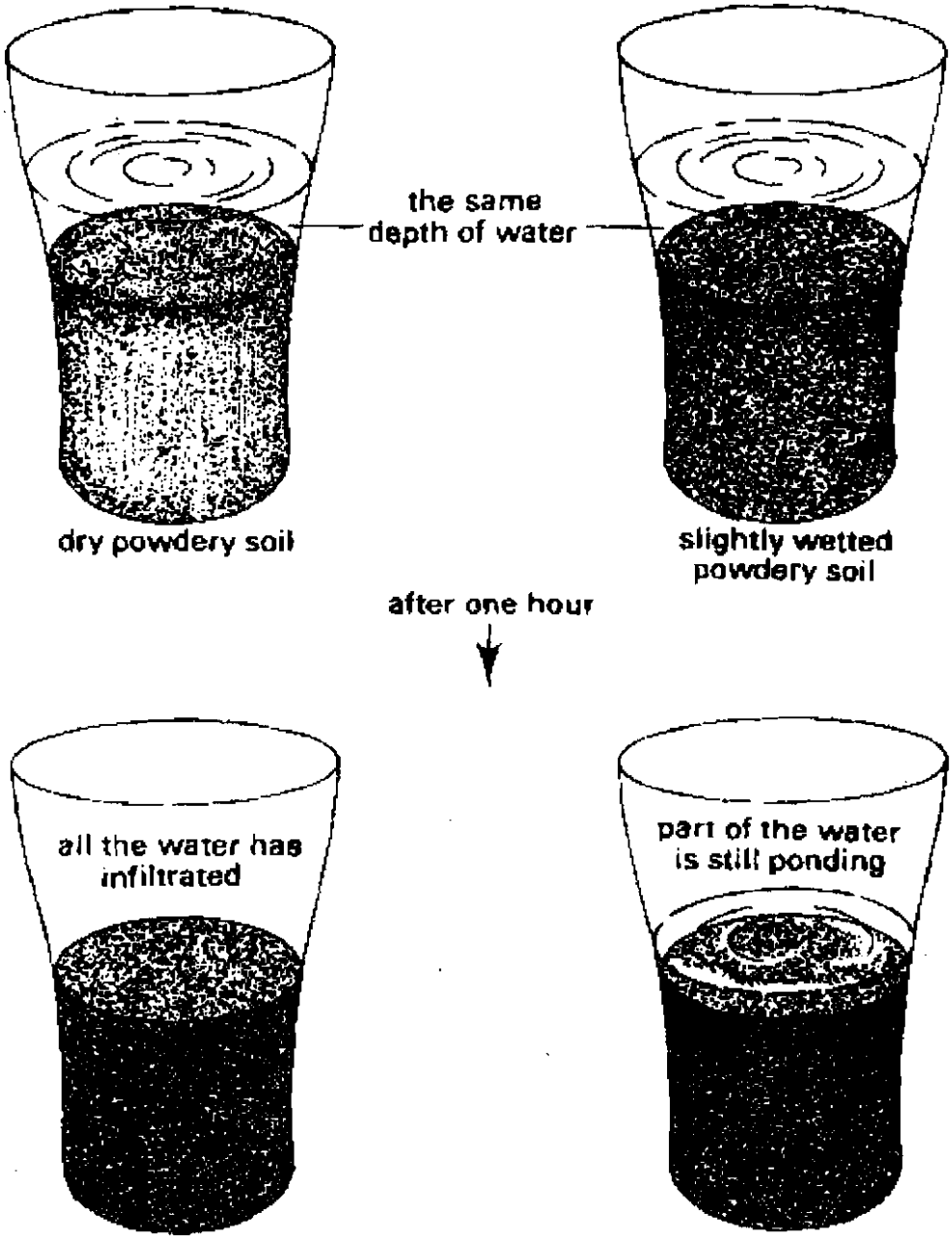


Figure 6.9. Infiltration rate and soil moisture content

iii. The soil structure

Generally speaking, water infiltrates quickly (high infiltration rate) into granular soils but

very slowly (low infiltration rate) into massive and compact soils.
Because the farmer can influence the soil structure (by means of cultural practices), he can also change the infiltration rate of his soil.

6.5 Soil Moisture Conditions

Soil Moisture Content
The soil moisture content indicates the amount of water present in the soil.
It is commonly expressed as the amount of water (in mm of water depth) present in a depth of one metre of soil. For example: when an amount of water (in mm of water depth) of 150 mm is present in a depth of one metre of soil, the soil moisture content is 150 mm/m (see Fig.6. 10).

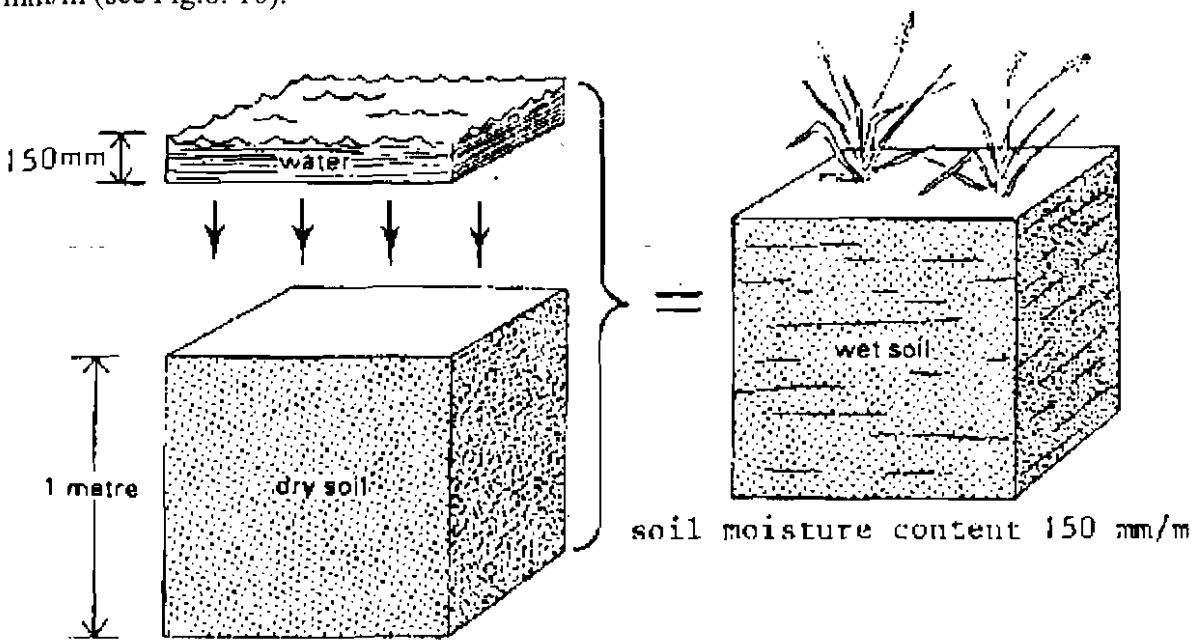


Fig. 6.10. A soil moisture content of 150 mm/m

The soil moisture content can also be expressed in percent of volume. In the example above, 1 m^3 of soil (e.g. with a depth of 1 m, and a surface area of 1 m^2) contains 0.150 m^3 of water (e.g. with a depth of 150 mm = 0.150 m and a surface area of 1 m^2). This results in a soil moisture content in volume percent of:

$$\frac{0.150\,m^3}{1\,m^3} \times 100\% = 15\%$$

Thus, a moisture content of 100 mm/m corresponds to a moisture content of 10 volume percent.
Note: The amount of water stored in the soil is not constant with time, but may vary.

Saturation

During a rain shower or irrigation application, the soil pores will fill with water. If all soil pores are filled with water the soil is said to be saturated. (see Fig. 10a). It is easy to determine in the field if a soil is saturated. If a handful of saturated soil is squeezed, some (muddy) water will run between the fingers.

Plants need air and water in the soil. At saturation, no air is present and the plant will suffer. Many crops cannot withstand saturated soil conditions for a period of more than 2-5 days. Rice is one of the exceptions to this rule. The period of saturation of the topsoil usually does not last long. After the rain or the irrigation has stopped, part of the water present in the larger pores will move downward. This process is called drainage or percolation.

The water drained from the pores is replaced by air. In coarse textured (sandy) soils, drainage is completed within a period of a few hours. In fine textured (clayey) soils, drainage may take some (2-3) days.

Field Capacity

After the drainage has stopped, the large soil pores are filled with both air and water while the smaller pores are still full of water. At this stage, the soil is said to be at field capacity. At field capacity, the water and air contents of the soil are considered to be ideal for crop growth (see Figure 6.11.

Permanent Wilting Point

Little by little, the water stored in the soil is taken up by the plant roots or evaporated from the topsoil into the atmosphere. If no additional water is supplied to the soil, it gradually dries out.

The dryer the soil becomes, the more tightly the remaining water is retained and the more difficult it is for the plant roots to extract it. At a certain stage, the uptake of water is not sufficient to meet the plant's needs. The plant looses freshness and wilts; the leaves change colour from green to yellow. Finally the plant dies.

The soil water content at the stage where the plant dies, is called permanent wilting point. The soil still contains some water, but it is too difficult for the roots to suck it from the soil (see Fig. 6.11).

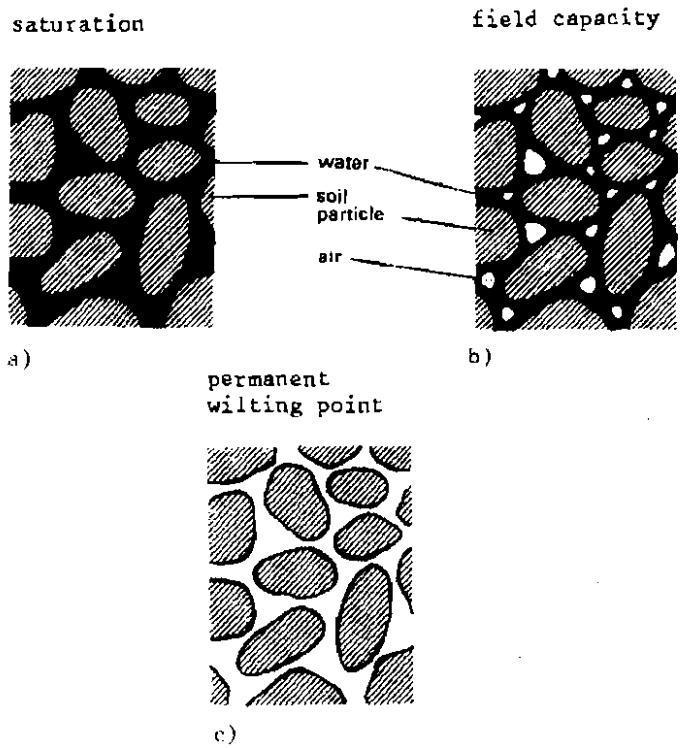


Fig. 6.11. Some soil moisture characteristics

Available Water Content

The soil can be compared to a water reservoir for the plants. When the soil is saturated, the reservoir is full. However, some water drains rapidly below the rootzone before the plant can use it (see Fig. 6.12). When this water has drained away, the soil is at field capacity. The plant roots draw water from what remains in the reservoir. When the soil reaches permanent wilting point, the remaining water is no longer available to the plant.

The amount of water actually available to the plant is the amount of water stored in the soil at field capacity minus the water that will remain in the soil at permanent wilting point. This is also illustrated in Fig. 6.12.

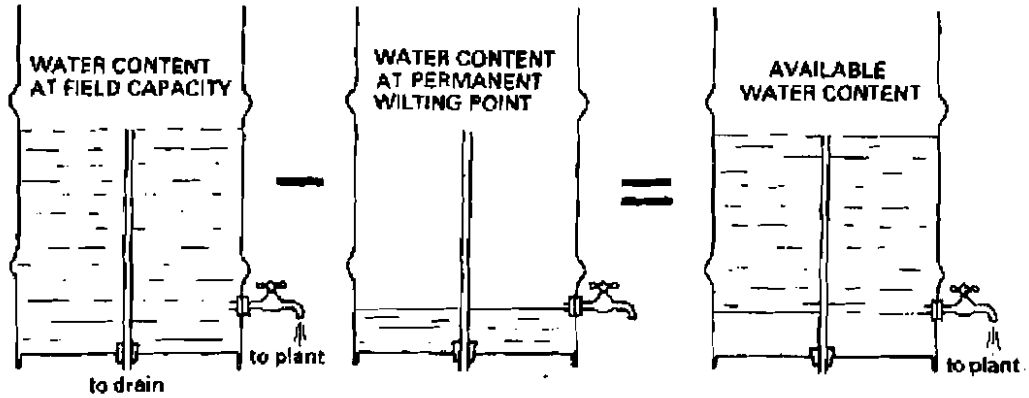


Fig. 6. 12. The available soil moisture or water content

Available water content = water content at field capacity - water content at permanent wilting point

The available water content depends greatly on the soil texture and structure. A range of values for different types of soil is given in the following table.

Soil	Available water content in mm water depth per m soil depth (mm/m)
sand	25 to 100
loam	100 to 175
clay	175 to 250

The field capacity, permanent wilting point (PWP) and available water content are called the soil moisture characteristics. They are constant for a given soil, but vary widely from one type of soil to another.

6.6 Groundwater Table

Part of the water applied to the soil surface drains below the rootzone and feeds deeper soil layers which are permanently saturated; the top of the saturated layer is called groundwater table or sometimes just water table (see Fig. 6.13).

Depth of the Groundwater Table

The depth of the groundwater table varies greatly from place to place, mainly due to changes in topography of the area (see Fig. 6.14). In one particular place or field, the depth of the groundwater table may vary in time. Following heavy rainfall or irrigation, the groundwater table rises. It may even reach and saturate the rootzone. If prolonged, this situation can be disastrous for crops which cannot resist "wet feet" for a long period. Where the groundwater table appears at the surface, it is called an open groundwater table. This is the case in swampy areas. The groundwater table can also be very deep and distant from the rootzone, for example following a prolonged dry period. To keep the rootzone moist, irrigation is then necessary.

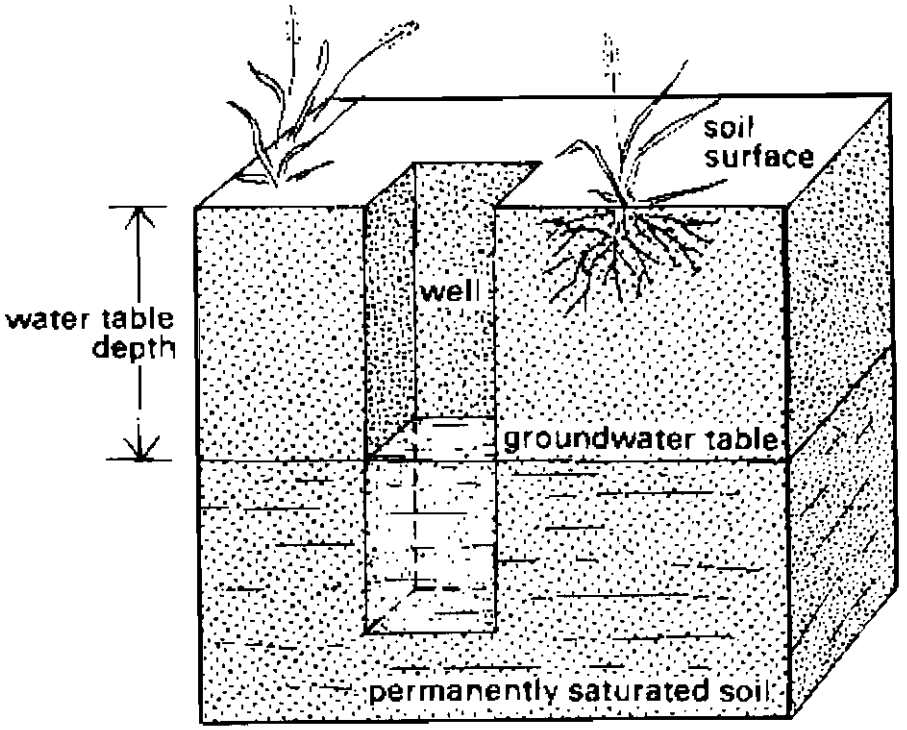


Fig. 6.13. The groundwater table

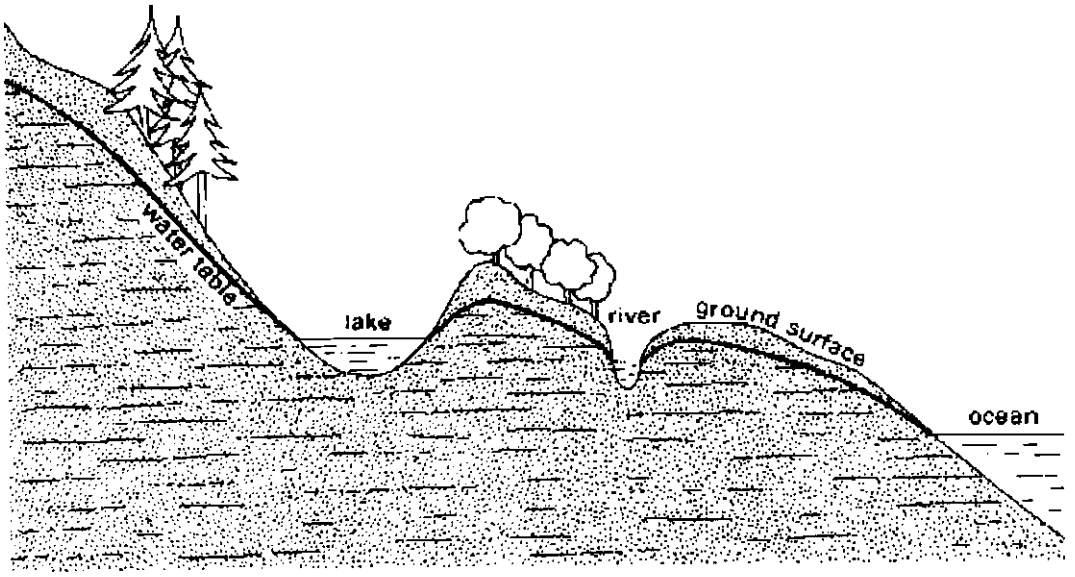


Fig. 6.14. Variations in depth of the groundwater table

7 FLOW MEASUREMENT

Irrigation management and operation requires knowledge of the amount and rate of flow. All irrigation system includes flow-measuring devices for determining flow rate. In the following section techniques and devices that are used to measure flow in open channels as well as pipe flow are described.

7.1 Common Units of Flow Measurements

Volume

1 cubic meter	=	1000 liters
1 liter	=	1000 cm^3
1 m^3	=	35.31 ft^3
1 m^3	=	264.2 gallons (U.S)
1 acre-foot	=	1233.3 m^3
1 acre-foot	=	12 acre-inch
1 acre-foot	=	325,829 gallons

Flow rate

1 l/min	=	60 l /s
1 m^3/s	=	1000 l/s
1 l/s	=	15.85 gallons/min
1 m^3/s	=	35.31 ft^3 /s
1 gallon per minute	=	3.7851 l/s
1 gallon per hr	=	3.7851 l/hr
1 ft^3/s	=	1.9835 acre-feet per day

7.2 Flow Measurement In Open Channels

Volumetric method

In this method the time required for the flow to fill a container of known volume is measured.

Q = Volume /time

Q = flow rate in m^3/s,

V =Volume in m^3 and

T= time in seconds

Velocity-Area Method

In method flow velocity is measured and is multiplied by the cross sectional area of the channel.

Q = AV

V= flow velocity in m/s and A = cross-sectional area in m^2 .

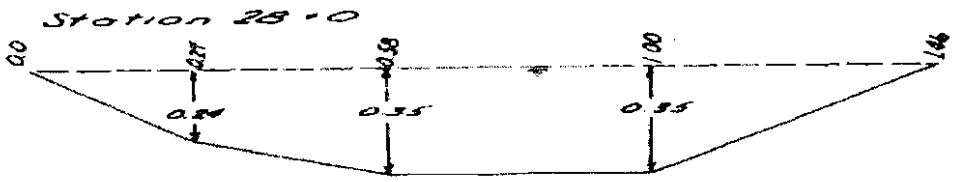


Fig 7.1. Irregular shape Cannel cross section

Area

In irregular shaped channel the cross section (normal to flow) is divided into several segments as shown in Figure 7.1. The area of each segment is computed and are summed to determine the total cross sectional area.

Velocity

The flow velocity in a channel varies significantly at different segments of the channel. A current meter is used when more accurate measurements are needed. For rough estimate float method is also used flow velocity. Besides that flow velocity can also be determined by chemical tracers.

7.3 The Float Method

The weighted float method is based on velocity - area relationships, the velocity in the open channel is measured directly and the rate of flow determined by multiplying this velocity by the cross sectional area of the channel.

The rate of flow passing a point in a ditch or other open channel can be determined by multiplying the cross sectional area of water by the average velocity of the water. Normally, the cross sectional area can be determined by direct measurement of the channel dimensions. In this method the section should be straight, uniform and free from free from surface disturbances. The average velocity of the stream is computed with the following equation

$$V = C_f V_f$$

Where V_f is the surface velocity and
C_f is Depth correction factor (Table 7.1)

Table 7.1. Correction factor (C_f).

Average flow depth(m)	C_f -	Average flow depth(m)	C_f -
0.3	0.66	1.8	0.76
0.6	0.68	2.7	0.77
0.9	0.70	3.7	0.78
1.2	0.72	4.6	0.79
1.5	0.74	>6.1	0.80

Source: U.S Bureau of Reclamation water measurement manual (1975)

Procedure for Determining Flow Velocity by Float Method

The velocity can be estimated by timing the passage of a small float through a measured length of channel. The procedure for estimating rate of flow by the float method is as follows:

Select a straight section of ditch with fairly uniform cross sections. The length of the section will depend on the current, but 30 meters usually will be adequate. A shorter length may be satisfactory for slow flowing ditches.

Make several measurements of depth and width within the trial section, to determine the average cross section area, it is expressed in square meters.

Place a small float in the ditch about a meter upstream from the upper end of the trial section. Determine the number of seconds it takes for the float to travel from the upper end of the trial section to the lower end. Make several trials to get an average time of travel. The best floats are small rounded objects which float nearly submerged. They are less apt to be affected by wind or to be slowed by striking the side of the channel. Among small objects which make good floats are a long necked bottle partly filled with water and caped, a rounded block of wood, or an orange. A wooden sphere, like a croquet ball, is excellent.

Determine the velocity of the float in meters per second by dividing the length of the section (meters) by the time (seconds) required for the float to travel that distance.

Determine the average velocity of the stream. Since the velocity of the float on the surface of the water will be greater than the average velocity of the stream, the float velocity must be multiplied by a correction coefficient to obtain a better estimate of the true average stream velocity. The correction factor varies with the type of float used and

with the shape and uniformity of the channel. With floats that sink about 2 to 5 centimeter below the water surface, a coefficient of about 0.80 should be used for most unlined farm ditches.

Compute the rate of flow. The rate of flow is obtained by multiplying the average cross sectional area by the average stream velocity. The accuracy of these estimates of flow rates is dependent upon the preciseness with which average cross sectional areas and float velocities have been determined and upon the section of the proper correction coefficient.

Velocity Data

Time for float (wooden sphere) to travel 30 meters

Trial No.	1	2	3	4	Average
	95	91	90	80	91

Float Velocity=30/91 = 0.33 meters per sec.
For the depth of 0.6 m the correction factor is 0.68 and
average velocity (V) = 0.68x0.33 = 0.224 m/s

Flow Rate

$$Q = AV = 0.4\ m^2 \times 0.224 m/s = 0.0896\ m^3/s \text{ or } 89.6\ l/s.$$

7.4 Weirs

The weir is one of the oldest and simplest methods of measuring water in open channels and can be used where there is sufficient head available (usually 15 cm or more). Depending on the shape of the opening through which the water passes they are classified as rectangular,

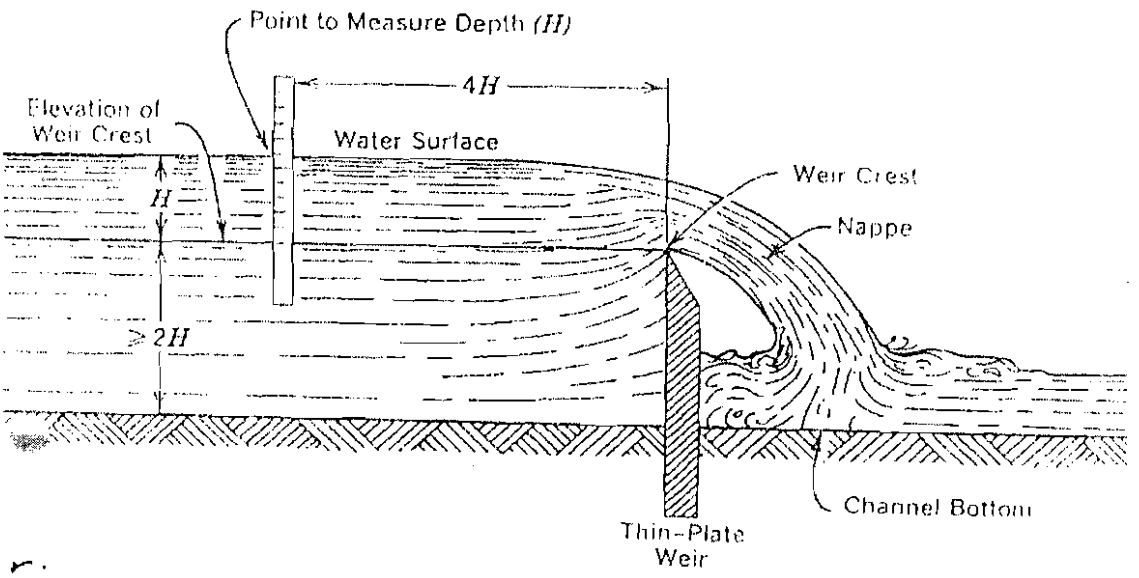


Fig 7. 2. Thin Plate Weir

Trapezoidal or Cipolletti and triangular Using an orifice flowing partially full, a general equation for weir flow may be derived which has the following form:

$Q = C\,L\,H^{3/2}$

Q is the rate of flow
C is the coefficient of discharge
L is the length of crest
H is the head on the weir

The thin plate weirs, also called sharp crested weir, consist of a smooth, vertical flat plate installed across the channel and perpendicular to flow. The thickness of the crest is less than 2 mm in the direction of flow. If the material is thicker than this it should be chamfered at a 45^0 angle on the downstream side. Weirs should always operate under free flow conditions with the maximum water level downstream at least 6 cm below the crest. Thus the total head loss across the weir will be the head on the weir plus at least 6 cm. Figure 7. 3 shows the profile of a sharp crested weir along with the head measurement location and the elevation of the weir crest with respect to the upstream channel bottom.

Rectangular Weirs

Rectangular weirs may be divided into two classes. Weirs placed in a rectangular channel where the sidewalls act as the ends of the weir have no contractions are said to be suppressed. Discharge values for suppressed rectangular weir are presented in Annexure A (Table A1). The equation for discharge for suppressed conditions is:

$$Q = 1.84\,LH^{3/2}$$

where Q = flow in m^3/sec
L = crest length in m
H = head in m

Example: L = 1.0 meters
H = 0.30 meters
$Q = 1.84*1.0*(0.3)^{1.5} = 0.302\ m^3/s$ or
Q= 302.12 l/s.

Contracted Rectangular Weirs

Theses are more commonly used in irrigation canals for flow measurement.

The more common case of a contracted weir as shown in Figure 7.3 allows unconstrained lateral approach to the weir. Discharge values for contracted rectangular weir is presented in Table 7.3 (Annexure A i.e Table A2). In this case a correction must be made in the discharge and is given by:

$Q = 1.84\ (L - 0.2H)*H^{3/2}$

Q= flow in m^3/sec
L= crest length in m
H= head in m

Example L=1.2 meters
H=0.50 meters

From the above mentioned equation

$Q = 1.84 \times (1.2 - 0.2 \times 0.5) \times 0.5^{1.5} = 0.716\ m^3/s$ or 716 L/s

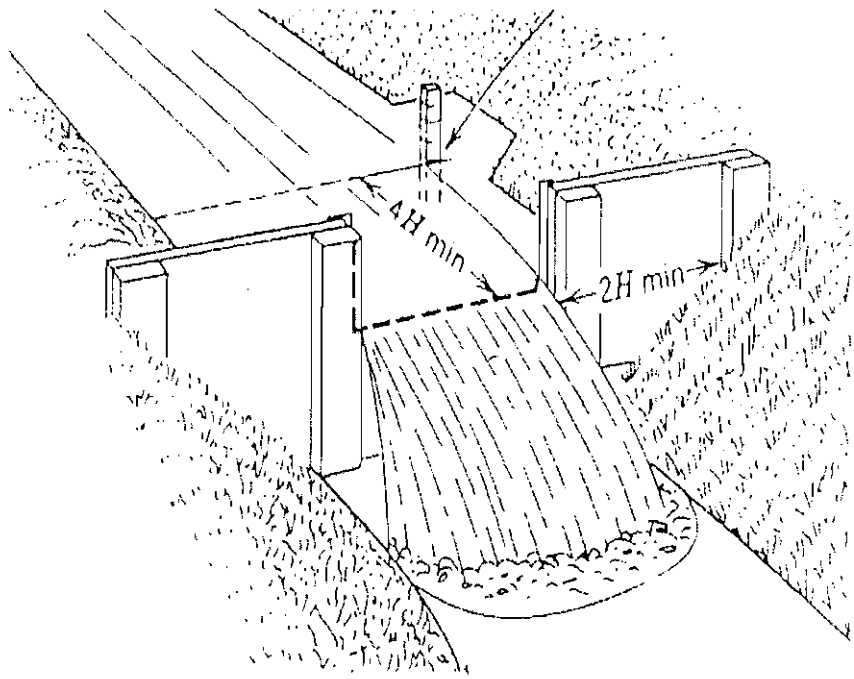


Fig7. 3. Rectangular Weir

Trapezoidal or Cipolletti Weirs

Trapezoidal weirs are sometimes referred to as Cipolletti after the engineer who developed them to eliminate the need for correction of the crest length of rectangular weirs see Annexure A (Table A3). They have sloping sides of 1 horizontal to 4 vertical and various crest lengths. The equation for discharge is:

$Q = 3.367\ L\ H^{3/2}$

Q is in m^3/sec

L is the crest length in m

H is the head in m

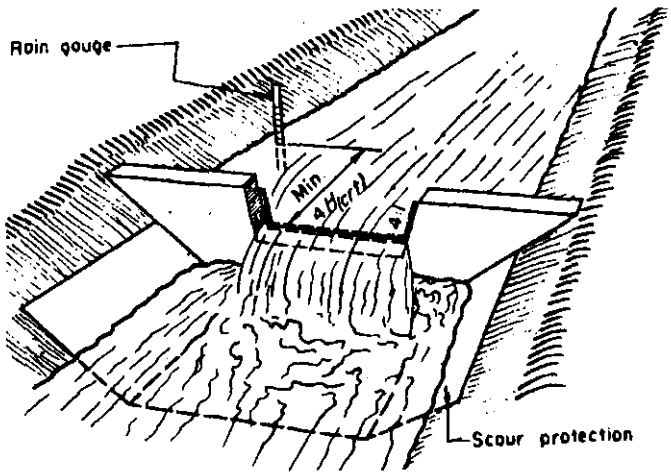


Fig 7.4. Trapezoidal or Cipolletti Weirs

Example L = 1.0 meters

H = 0.30 meters

From the above equation

$Q = 3.367 \times 1.0 \times (0.3^{1.5}) = 0.553\ m^3/s$ or 553 L/s

4. The upstream corners of the notch must be sharp. They should be machined or field perpendicular to the upstream face free of burrs or scratches and not smoothed off with abrasive cloth or paper. Knife edges should be avoided because they are difficult to maintain.

5. The downstream edges of the notch should be relieved by chamfering if the plate is thicker than the prescribed crest width. This chamfer should be at an angle of 45° or more to the surface of the crest.

6. The distance of the crest from the bottom of the approach channel (weir pool) should preferably be not less than twice the depth of water above the crest and in no case less than 30 cm.

7. The distance from the sides of the weir to the sides of the approach channel should preferably be no less than twice the depth of water above the crest and never less than 30 cm.

8. The overflow sheet (nappe) should touch only the upstream edges of the crest and sides.

9. Maximum downstream pool level should be 6 cm below the crest elevation.

10. The measurement of head on the weir should be taken as the difference in elevation between the crest and the water surface at a point upstream from the weir a distance of four times the maximum head on the crest.

11. For contracted weirs the cross-sectional area of the approach channel should be at least 8 times that of the overflow sheet at the crest for a distance upstream from 15 to 20 times the depth of the sheet.
If the weir pool is smaller than defined by the above criteria the velocity of approach may be too high and the staff gauge reading too low.

7.5 Cutthroat Flume

The cutthroat flume is used to measure the rate of flow in an open channel. It has been used successfully for a number of years in Pakistan. The cutthroat flume has two important operating characteristics; 1) operation is satisfactory under both free and submerged flow conditions, 2) head loss through the flume is low, even less than the long-used Parshall flume.
It gives satisfactory accuracy in water measurement in the flat gradient channels common to Pakistan. Furthermore, it is easy to construct because of the flat bottom. Figure 7.6 shows a dimensionless design for the cutthroat flume.
The free flow discharge rate is given by the equation:

$$Q = K \times W^{1.025} \times H_a^n$$

Triangular Weirs

Triangular weirs may be many sizes but the generally accepted standard is a 90° V-notch. The head is measured from the vertex of the inverted angle. It has the advantage of being able to accurately measure low flows and can still be used over a wide range of flows see Annexure A (Table A4). The equations for discharge is:

$Q = 1.34257\ H^{5/2}$

Q is in m^3/s and H is head in m

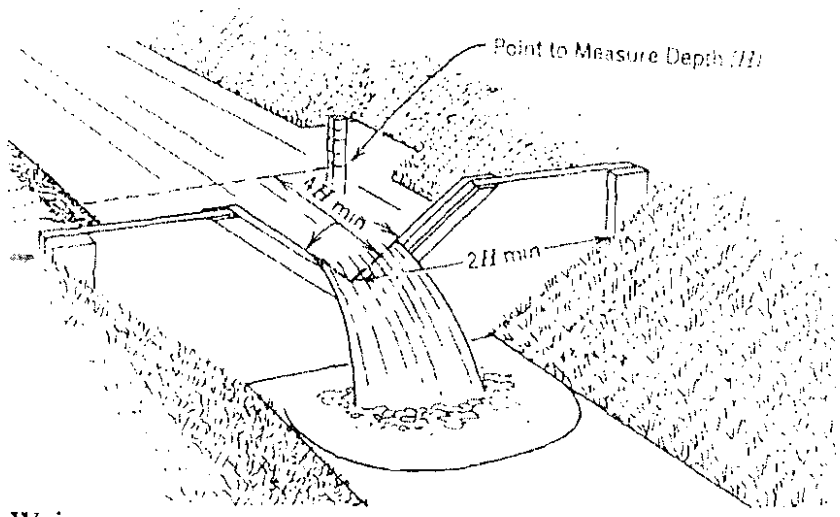


Fig 7.5. Triangular Weirs

Example H= 0.30 m

From the above equation

$Q = 1.34257 \times (0.3^{2.5}) = 0.06618$ m3/s or 66.18 L/s.

Weir Installation:

The installation of weirs should be done according to the following recommendations:

1. The upstream face of the bulkhead should be smooth and in a vertical plane perpendicular to the axis of the channel.

2. The upstream face of the weir plate should be smooth straight and flush with the upstream face of the bulkhead.

3. The entire crest of the rectangular and Cipolletti weirs should be a level plane surface which forms a sharp right-angled edge where it intersects the up-stream face. The thickness as the crest.

Where:

Q = discharge in m^3/s
W = width of the throat in meters
Ha = upstream head in meters
K= coefficient dependent on the flume length (for free flow)
n= exponent for free flow

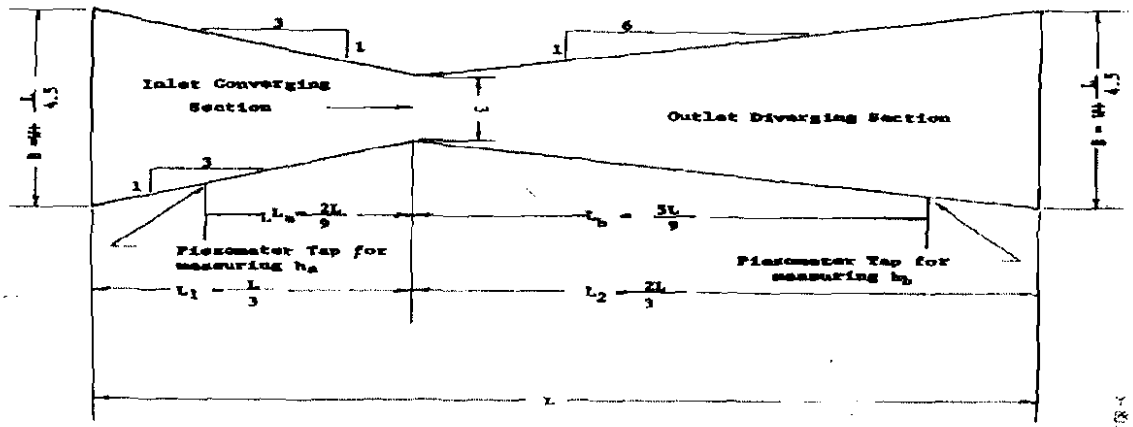


Fig 7.6. Cutthroat Flume

For submerged flow condition the Q is given by the equation:

$$Q = \frac{K_s W^{1.025} (H_a - H_b)^n}{(-\log S)^{N_s}}$$

H_b= downstream head in meters
Ks= coefficient dependent on the flume length (for submerged flow)
N_s= exponent for submerged flow
S = the submergence, H_b/H_a (in decimal form)

These values are dependent of the length and width of the flume. Values for these coefficients and exponents for commonly used Cutthroat sizes in Pakistan are as follows:

L(m)	S	n	K	N_s	K_s
0.5	0.607	2.08	6.15	1.675	3.5
0.6	0.62	1.989	5.17	1.6	2.9
0.7	0.63	1.932	4.63	1.55	2.6
0.8	0.642	1.88	4.18	1.513	2.35
0.9	0.653	1.843	3.89	1.483	2.15
1.0	0.664	1.81	3.6	1.456	2.0
1.2	0.685	1.756	3.22	1.427	1.75
1.4	0.705	1.712	2.93	1.407	1.56
1.6	0.72	1.675	2.72	1.393	1.45
1.8	0.738	1.646	2.53	1.386	1.32
2.0	0.755	1.62	2.4	1.381	1.24
2.2	0.77	1.6	2.3	1.378	1.18
2.4	0.784	1.579	2.22	1.381	1.12
2.6	0.795	1.568	2.15	1.386	1.08

Flume Selection

In order to select a flume of the optimum size, it is necessary to know the approximate flow rate, the depth of flow in the channel, and the allowable head loss through the flume. The head loss may be taken as the difference in water surface elevation between the flume entrance and exit. The downstream depth of flow will remain essentially the same after installation of the flume, but the upstream will increase by the head loss. The allowable increase in upstream or the submergence of the mogha.

Most channels in Pakistan have flat gradients with very little freeboard, thus in most cases the flumes will be installed to operate under a submerged flow condition. Good discharge measurements can be made under these conditions but the submergence should not exceed 90 percent if possible. This will reduce the energy loss. The problem building up the bund. But where conditions permit, e.g., steep-gradient channels with lots of free board, the flume should be installed for free flow conditions.

In selecting a flume, make sure the proper throat size is selected. Flow measurements are not as accurate at low heads or at high heads. Tables 6,7,8 and 9 will assist in the selection of the proper flume. Use the best judgment in selecting the proper flume to meet the specific requirements.

Installation

Although the cutthroat flume is simple in appearance, it must be installed carefully and used correctly. The first consideration prior to installing a flume, is the location or site of the structure. The flume should be placed in a straight section of channel, parallel to the direction of flow of water in the channel. Care should be taken to see that the flume is not located immediately downstream from a constriction, e.g., culvert, bridge pier, etc. If

it is to be located near a mogha, it should not be located so it will cause mogha submergence. The mogha discharge will be reduced if submerged by the flume. After the site has been selected the flume is ready for installation.

In making the flume installation it should be placed in the center of the channel. Where the channel bottom is unlevel, it may be necessary to level it out by either cutting down the high spot and/or filling the low spots. The bottom of the flume should be at same level or slightly higher than the channel bottom -- never placed lower than the channel bottom. If upstream bunds are high enough to allow an increase in water depth, it is desirable to place the flume bottom somewhat higher than the channel bottom to get a free flow condition.

After placing the flume in the channel, the floor of the flume should be properly leveled. The most desirable being the converging section of the flume, place a wooden level (about 6 inches long) in the transverse direction on floor of the flume bringing the bubble in the center. For longitudinal leveling, place the level parallel to flume length and in the throat and bring the bubble in the center. While doing this do not disturb the transverse level. This procedure may have to be done several times until the flume is level in both directions. Place soil on both side of flume and tamp, making sure that the sides and bottom are properly sealed so that no leaks occur. During the placement of the soil it will be necessary to check the level of flume to see that it has not been disturbed. In sandy soils plastic sheets or cloth may be needed to obtain an effective seal against leaks.

If the flume has to be installed in a channel with water flowing in it, make sure that two locations on the flume top are parallel with two similar locations on the flume bottom. Using the same leveling procedure as above, locate and mark these two reference points for the level. Always use these two marked positions to level the checked with reference to the flume bottom, as the flume will sometimes shift after considerable use.

Recording Flow Data

Prior to recording the gauge readings, always check underneath the flume and the sides for leaks. The inside bottom of the flume should be cleaned of any sediment or trash. Also check flume levelness, both in the longitudinally and transverse directions. If leaks are observed stop them and if level is disturbed level the flume again. Repeat this process each time that readings of the gauges are made.

Experience has shown that on flat-gradient channels the flume has to remain installed for a longer period of time before it reaches a steady-state flow condition. Short period installations can often lead to wrong conclusions. This is because the section through which the water has to flow restricts to a much smaller width with the flume installed as compared to the section width without flume. This imposes a problem of storage of water in the upstream section of the channel causing a lesser volume of flow the ditch storage to stop so that flow through the flume is the same as the normal flow through the water

channel. In other words, sufficient time should be allowed so that equilibrium is established between inflow above the flume and outflow through the flume. If the conditions on the upstream section of the channel do not change and the volume of flow has become constant, equilibrium has been reached.

Once the above conditions are reached, then gauge readings can be taken. The upstream reading is referred to as Ha and the downstream reading is referred to as Hb. These readings are then converted to flow values by using the appropriate tables (Annexure A i.e Tables A5 and A6).

7.6 Stream Gauging

Discharge, also termed as rate of flow, is defined as the volume of water flowing through a cross section in a unit time. It is expressed in terms of cubic meter per second (cuvees). The primary object of discharge observation is to establish gauge-discharge relationship for the given site so that daily measurement of discharge is not necessary and the discharge figure, corresponding to the daily gauge reading is read from gauge-discharge curve prepared on the basis of water measurement at various stages of flow. The objects of discharge measurement are as follows:

Objectives of Discharge Measurement

- Accurate assessment of the available and dependable natural water resources of a river basin.
- Comprehensive and coordinated planning for optimum utilization of water resources in the rivers, their tributaries and streams.
- Investigation and design of multipurpose river valley projects for the development of irrigation and power potentials, flood control, water supply, navigation etc.
- Issue of flood warnings on the basis of the estimated high flood of discharge corresponding to the observed high river stage.
- Regulation and equitable distribution of supplies in a canal system.
- Determination of seasonal and annual variations in run off.
- Estimation of water losses and regeneration in different reaches of the river.

Selection of Gauge Site

The reliability of the discharge depends on the consistency with which the stage discharge relationship maintained at the gauge suite, which is turn is controlled by physical features of the channel downstream from the gauge sites. The datum or zero elevation of the gauge is fixed with reference to the lowest water surface level at the gauge site so that no gauge reading indicates a negative level. The reference gauge is securely fixed to an immoveable rigid support. The datum of gauge is related by precise leveling to the national datum.

- The section should be straight and uniform for a length of about 10 to 20 times the width of the stream.

- The bed and banks of the stream should be firm and stable so as to ensure consistency of area discharge relationship, i.e. the cross section should be subject to change by silting or scouring, during different stages of flow; a smooth rock; shingle, or clay bawd is far favorable, while a fine sandy bed lies unfavorable.
- The bed and banks should be free from vegetal growth, boulders or other obstructions like bridges piers etc.
- There should be no larger overflow at flood stage. The best cross section is one with v-shape, so that there is a sufficient depth for immersing the current meter with being affected by the bed roughness of the stream.
- The part of the reach having the most regular transverse section and steady flow with the current normal to the metering section and velocities in the range of 0.3 – 1.2m/sec should be selected.

The essential requirements of a gauge site is as follows:

- Select a place, where once fixed, the gauge is permanent, i.e. immune from shifting due to silting or scouring.

- Register always the same elevation of water surface correspond to the given discharge.

- It is clearly visible and easily accessible. A gauge site meant for flood warning shall necessarily be nearest to the posts and telegraph office in the locality and sufficiently upstream of the area to which flood warning is to be issued to ensure reasonable time for carrying out safety measures in advance of the flow reaching there.

7.7 Types of Gauges

The gauge may be a direct or indirect. Direct gauges are staff gauges and indirect gauges are self recorder type and crest stage type.

Vertical staff gauges

Vertical staff gauge is preferably an enamel plate gauge graduated to a millimeter, the figures being in English. The markings are clear and sufficiently accurate with bold line indication at every 20mm interval. The graduations may be made either by etching or by painting or enameling so as to read centimeters, decimeters and meters distinctively, the smallest graduation being a centimeters.The staff gauge is generally fixed at or near the edge of water in the river so as to be easily readable and accessible and at the same time not unduly exposed to the face of the current.

Inclined gauge

Inclined gauge shall fit closely and be solidly anchored to the slope of the natural bank of the river channel. It is calibrated on the site by precise leveling. Inclined gauge may be constructed to one continuous slope or may be a compound of two or more slopes. It is

often convenient to construct a flight of steps along side the inclined gauge to facilitate taking readings.

Staff Gauge

The staff gauge is either vertical or inclined. Both the gauges are read upto hundredth of a foot. The staff gauge consists of a graduated and enameled iron or wooden section 4 inches wide and 3.33 feet or 2.5 feet long. The graduations are made at every 0.02, 0.05 or 1 ft. according to the accuracy desired. On streams where there are possibilities of silt and mud crust on the paintings, raised graduations and numerals or series of saw tooth notches on one side of the staff are used.

The inclined staff gauge usually consists of a gradated heavy timber attached to permanent foundation. Inclined gauges built flush with the stream bank are less likely to be damaged by floods, floating etc; or drift than are projecting vertical staffs,. Inclined gaugcs are used for outside, referencc gauges.

7.8 Stage Discharge Rating Curve

Because of a long time required for making each discharge measurement at a gauging station, it becomes impossible to obtain continuous record of discharge. Therefore, it becomes necessary to obtain a graphysical relation between stage and discharge by taking simultaneous measurement of stage and discharge at different water levels in the stream. This graphical relationship is known as stage discharge rating curve. Discharge on the abscissa. This graph can be used to determine continuous discharge corresponding to the measured stage on a recorder. The stage discharge rating depends on the channel control. For a permanent control this relation may be dependable for years, but for changing control, because of changes in the stream channel due to scour and fill, aquatic growth and channel bed roughness, the rating cannot be relied upon for a long time. In this frequent discharge measurements are necessary to define the stage discharge relation at any time. It may be necessary to take at least three discharge measurements a week to check the rating curve because of the random shifting of the stream channel particularly during flood season.

Shape of the Curve

Using Chezy's formula, it can be easily proved that the stage discharge relationship for uniform flow in very wide rectangular channels (width Depth) is given by:$Q= KD^{3/2}$ Where Q is the discharge, K is a constant which depends on Chezy's roughness coefficient, bed slope and width of the stream and D is the mean depth. If the datum of

the gauge is at zero flow depth D can be replaced by H, the gauge height. However, if the datum is below the zero discharge by "a" the above equation becomes:

$Q=(H-a)^{3/2}$,

Here 'a' Similarly it can be shown

that the general shape of the rating curve is:
$Q=K(H-a)^{2m+1/2}$,

Which is the equation of a parabola

Where m = 1 for a rectangular section
m = 3/2 for a parabolic shape
m = 2 for a triangular section
The general form of equation is $Q=K(H-a)^n$

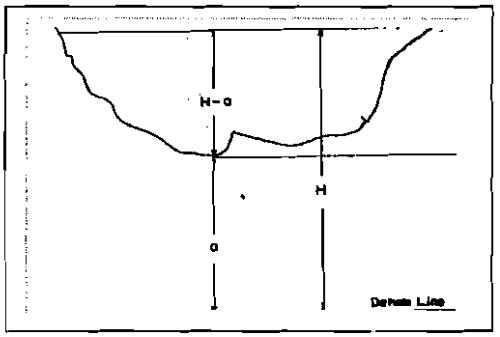


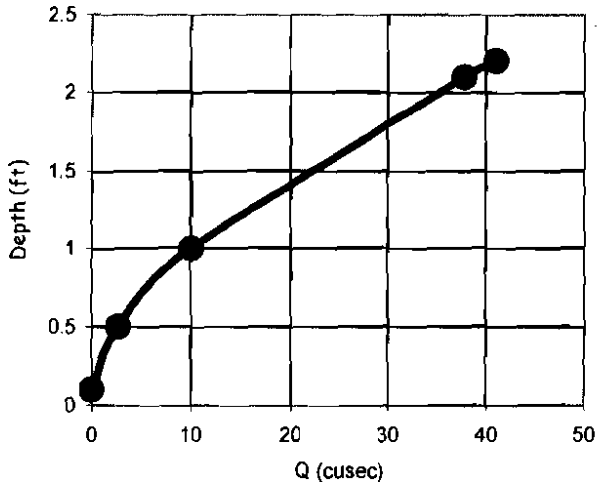
Fig 7.7. Schematic Sketch of gauge height

Among the various sections, the parabolic shape is the most common that approximate to many natural river cross sections. It may also be noted that the equation may change from one range of stage to another. Some times the curve changes from a parabolic to an odd curve or vice versa, and some time the constants and the exponents may vary. Changes in the shape of the rating curve may also occur when there is change of control. A central control may be effective for only a certain range of the stage, then the curve will take place. One has therefore, to be aware of this possible change whole constructing a stage discharge-rating curve. This change can be more easily detected if the stage discharge relationship is plotted on a logarithmic scale, where it would plot as a straight line.

relationship can change due to:

- Changing characteristics caused by weed growth ,dredging or channel encroachment,
- Aggradations or degradation phenomenon in a alluvial channel,
- Variable backwater effects affecting the gagging section and
- Unsteady flow effects of a rapidly

It may be noted that at the same stage, more discharge passes through the river during rising stages than in falling ones. Since the conditions for each flood may be different, different floods may give different loops.

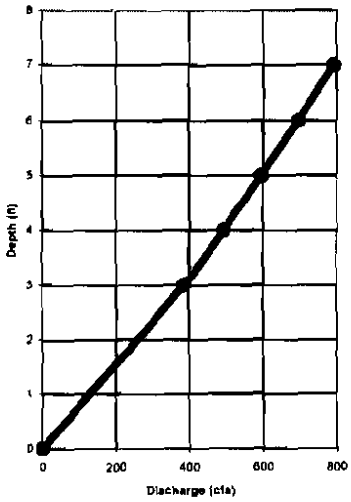


Example

Q = 38 cfs
Gauge = 2.1 ft
Depth = 2ft

Equation: $Q = KD^{5/3}$
If (Q, D) = (38, 2)⇒ K=11.97

$$Q = 11.97\ D^{5/3}$$



D(ft)	Q(cfs)
7	800
5	600
4	?

Example The data of few discharges are given in the Table. Develop depth discharge relation curve and determine the discharge when gauge depth is 4?

Using ratio proportion:
n = 0.85 and K = 151.5

$$Q = 151.5\ Q^{0.85}$$

Figure 7.8 Stage-discharge relationship.
Example :

Example :

The stream, discharge for various stages at a p[particular section were observed to be as follows. Obtain equation for the stage discharge relationship and determine the discharge at 4.9 m.

Stage (m)	1.81	2.00	2.30	2.90	3.70	4.50	5.40	6.10	7.30	7.70	8.10
Discharge (cumecs)	1.00	1.50	2.55	5.60	11.70	20.20	32.50	44.50	70.00	80.00	90.00

Equation $Q = K(h-a)^n$ where K,a and n are constant. Plot Q vs (h-a) on a log-log paper assuming a value of a constant a =0.6 m (say). The curve obtained is concave downwards. Assume a = 1.2 m (say) and the curve obtained is concave upward. Now try an intermediate value a = 0.9 m, which plots a straight line and represents the stage-discharge relationship. The slope of this straight line gives the value of the exponent n=2.2 and from the graph for *(h-a)=1, Q=1.2=K*. Now the constants are determined and the equation for the stage-discharge relationship is as follows:

$$Q = 1.2\,(h - 0.9)^{2.2}$$

It may be noted that the value of a=0.9 which gives a straight line plot is the gauge reading for zero discharge. Now the abscissa of (h-a) may be replaced by the gauge reading (stage) h, by adding the value of 'a' to (h-a) values. For example the (h-a) values of 0.1, 1, 2, 4, 6, 8 and 10 may be replaced by the h values of 1, 1.9, 2.9, 4.9, 6.9, 8.9 and 10.9 respectively. Now for any gauge reading (stage) h, the discharge Q can be directly read from the graph and the stage-discharge curve can be extended. From the graph, Fig for h=4.9 m, Q 25.3 cumecs, which can be verified from stage discharge equation.

7.9 Method of Measuring Pipe Flow

The most common need for measuring pipe flow in Pakistan irrigation is that of flow from tubewells or possibly from some natural source, such as a spring, that flows through a pipe.

Trajectory Method

This method determines the rate of flow discharging from the end of a pipe. This is particularly useful for measuring flow from tubewells to estimate the flow for sizing of Water Storage Tanks for example.

There are three essential requirements for discharge determinations (Figure 7.9):

1. The discharge pipe must have a length not less than six times the pipe diameter.
2. The discharge pipe must be nearly level. Flows can be measured on a sloping pipe but it is more difficult to make measurements.
3. The pipe must discharge into the air.

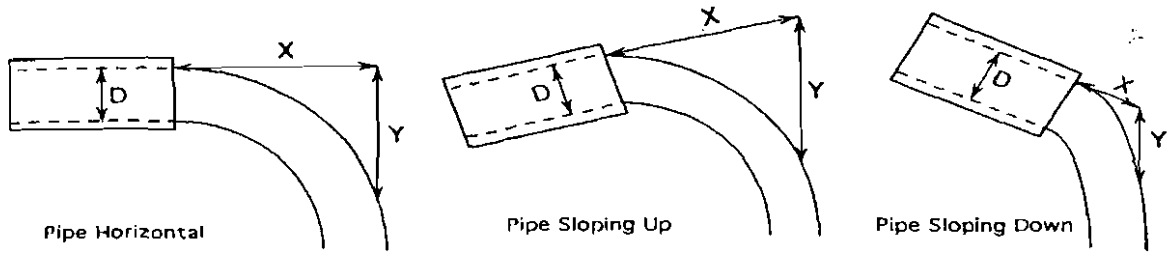


Fig 7.9. Pipe flowing full

For pipes flowing full three measurements are needed; 1) the inside diameter of the pipe in inches or centimeters; 2) the horizontal distance measured from the end of the pipe (parallel to it) for a fixed distance that allows for making a vertical measurement; 3) the vertical distance is measured down from the horizontal point to the top of the water jet. For convenience, these coordinates are measured from the top of the inside of the pipe to a point on top of the jet . See Figure. These horizontal and vertical distances are called X and Y ordinates, respectively.

For pipes flowing partly full, an additional measurement is needed. The vertical distance from the water surface at the end of the pipe to the inside top of the pipe must be determined.

After the above measurements have been made, the discharge (Q) n liters second for pipes flowing full, can be computed by the formula:

$$Q = 0.0174 \times \frac{XD^2}{\sqrt{Y}}$$

Q = flow rate in lps
D = diameter of pipe in cm
X = horizontal distance in cm from end of pipe required for the surface of the jet to drop Y centimeters
Y = vertical distance in cm measured down to surface of jet

Pipes Flowing Partly Flow

For pipes flowing partly full, the formula is:

$$Q = 0.0174 \times \frac{XD^2}{\sqrt{Y}} \times \left(\frac{a}{A}\right)$$

Q = flow rate in lps
D = diameter of pipe in cm
X = horizontal distance in cm from end of pipe required for the surface of the jet to drop

Y = vertical distance in cm measured down to surface of jet
A = the cross sectional area of the pipe
a = the area of the jet at the end of the pipe

Discharge values for both full flow and a pipe flowing partially full are given in the above figure for standard distances of; X = 15 cm, X=30 cm, and X = 45 cm respectively.
Rather than calculating "a" Figure 4, gives values of a / A in terms of measured distances for the diameter and distance down from the inside top of the pipe to top of the jet. Since discharge pipes are not always horizontal, Figure also shows the correct method for estimating flow from pipes that are sloping.

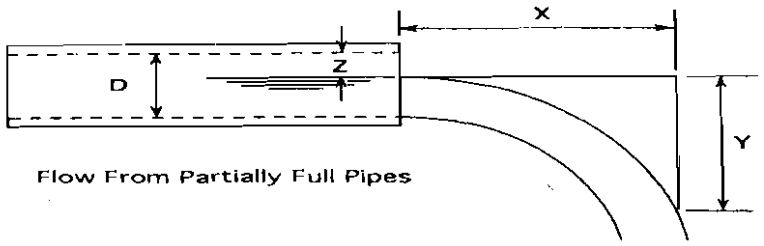


Figure 7.10 Flow from partially full pipe

7.10 Measuring Small Irrigation Streams

Siphon Tubes

Siphon tubes, used to remove water from a head ditch and distribute it over a field through furrows, corrugations, or borders, can also used to measure the rate of flow into these distribution systems (Figure 7.11). These tubes, made of aluminum, plastic, or rubber, are usually preformed to fit a half cross section of the head ditch. The normal diameter range is from 1 to 6 inches (2.54-15.24 cm), although both smaller and larger sizes are available. The smaller sizes are used with furrows and corrugations and the larger sizes with borders. Various lengths are available. Siphon tubes are portable. For this reason, the reduced number of tubes required to irrigate a given area results in low initial cost for equipment. Flow into individual furrows or borders can be controlled effectively by using the number of tubes that will divide the total head ditch flow into individual streams of the desired size.

The tubes are limited to fields with little cross slope in order to maintain a near constant operating head on each tube. A disadvantage is that they need to be primed individually. This priming is the principal labor requirement when siphon tubes are used for surface irrigation.

The discharge of a siphon tube depends on: (1) The diameter of the tube, (2) the length of the tube, (3) the roughness of the inside surface and the number and degree of bends in the tube, and (4) the head under which the tube is operating. When the outlet end of the tube is submerged, the operating head is the difference in elevation between the water

surfaces measured at the entrance and outlet ends of the tube. When the tube is flowing free, the operating head is the difference in elevation between the water surface at the entrance of the tube and the center of the outlet end.

Submerged Orifice Plates

A simple method of measuring furrow stream flow is by submerged orifice plates (Figure 7.12). The plate is placed across the furrow and the head loss through the orifice is measured under submerged flow conditions. Orifice plates consist of small sheet iron, steel, or aluminum plates that contain accurately machined circular openings or orifices usually ranging from 45-100 mm in diameter. The figure shows construction details for an orifice plate with four orifices of such diameters that any flow between about 0.14 and 6.5 1ps liters per second can be measured within a head range 1 to 100 mm. Orifice,plates have several advantages. They are simple, inexpensive, and easy to install. Furrow streams can be measured with a minimum head differential or restriction to flow, thereby minimizing the increase in the wetted perimeter of the furrow above the measuring point and the probability of overtopping. With reasonable care in setting and reading, the margin of error in the measurements will not exceed 5 percent.

In use, an orifice size is selected so as to produce a head differential within about 15-60 millimeter range, and the plate is placed in-and across the furrow with its top as nearly level as possible. Flow through the orifice must be submerged. Figure shows the installation of the orifice plate. In some cases, it may be necessary to restrict the flow downstream from the plate in order to raise the water surface on its downstream side to a level slightly above the top of the orifice. Allowing a few minutes for the head differential to become constant, this differential (the difference in the distances from the top of the plate to the water surface on the upstream and downstream, sides) is measured with an engineer's scale. Readings are taken to the nearest millimeter.

By combining the velocity equation for a free falling body $V = (2gH)^{0.5}$ and the continuity equation, $Q = VA$ the general orifice equation is obtained $Q = C\ A\ (2gH)^{0.5}$. Because of head losses and the contraction of the jet it is necessary to apply a coefficient which is generally about 0.61 for fully contracted orifices thus:

$$Q = 0.61 A\sqrt{2gH}$$

Q = flow rate in m^3/sec
A = cross sectional area in m2
H = head in m
g = gravitational constant in 9.81 m/s^2

In the case of a free discharge jet, the head is the depth of water upstream over the center of the orifice. For submerged flow the head is taken as the difference in water level upstream and downstream from the orifice. Orifices may be either circular or rectangular in shape.

For example if a rectangular orifice 10 cm x 25 cm is operating under a 10 cm head the discharge is estimated by:

$$Q = 0.61 * .025 *(2* 9.81 *0.1)^{1/2} = 0.02136 \text{ m}^3/\text{sec} = 21.36 \text{ lps}$$

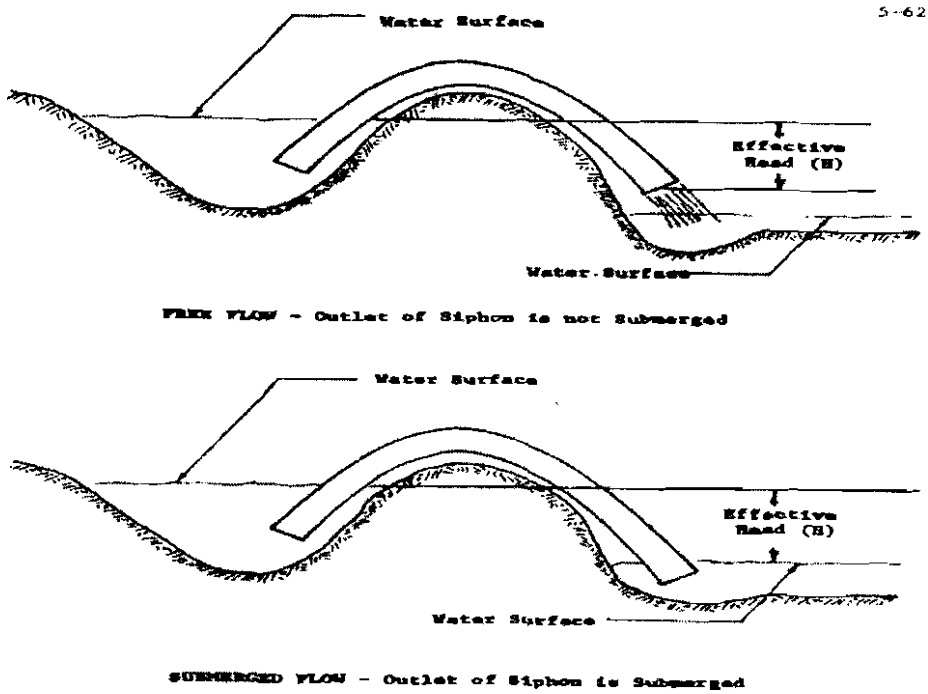


Figure 7.11. Flow through siphon tube

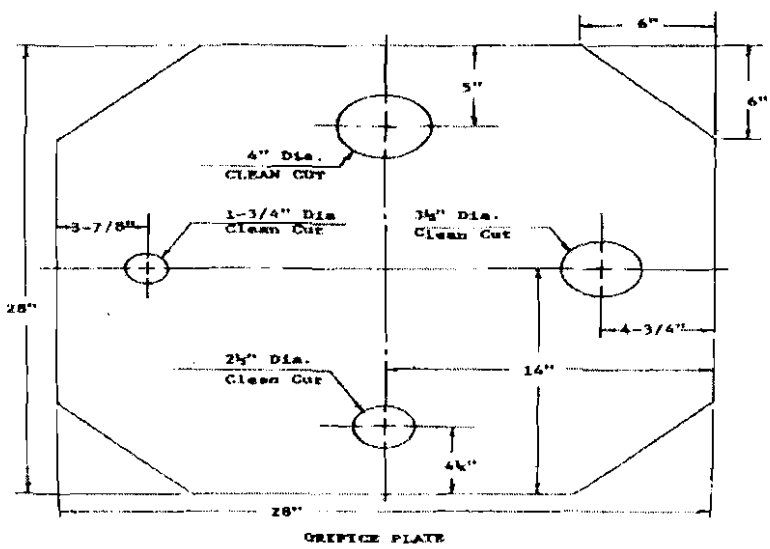


Figure 7.12. Flow through orifice plates

8 IRRIGATION CANAL DESIGN

In this section the design of canal capable of transporting water between two points in safe, cost effective manners is described. Only the hydraulics aspects of the channel design will be considered. Design of irrigation channels (I) earthen or erodible (2) lined or non-erodible will be workout. An open channel is a conduit for flow, which has free surface i.e. the boundary, exposed to the atmosphere. The free surface is the interface between two fluids of different densities. In case of flowing fluid, gravitational effects usually cause the motion and the pressure distribution is generally hydrostatics.

8.1 Types of Flow

Change of depth with respect to time and space: One of the primary criteria for of classification of flow .is the variation of depth of flow with respect to time and space

Classification Based on Time : If the depth of flow does not change with time ($dy/dt = 0$) , the flow is called steady flow. On the other hand if the depth of flow changes with time ($dy/dt \neq 0$) , the flow is called unsteady flow.

Classification Based on Space: If the depth of flow does not change along the channel ($dy/dx = 0$), the flow is called uniform flow.
On the other hand if the depth of flow changes with distance ($dy/dx \neq 0$), then the flow is called non-uniform flow.

Practically unsteady Uniform flow is not possible, but theoretically it may exist.
Non-uniform and uniform flow means steady uniform and non-uniform flow.

Further Classification of steady non-uniform flow: Rapidly varied flow: the depth of flow changes rapidly over short distance, such as hydraulic jump.
Gradually varied flow: the depth of flow changes slowly with distance such a depth of flow in a reservoir or if the flow in irrigation channel is blocked temporarily.

Viscosity, Density and Gravity: Renold Number (Re) represents the ratio of inertial to viscous forces it can be written as

$Re = UL/\nu$
Where U is the characteristic velocity of flow, often taken as the average velocity of flow
L is the characteristics length, and ν is the kinematics viscosity of the fluid

Laminar Flow: In laminar flow the viscous forces are strong relative to inertial forces.
In case of open channel for laminar flow $Re < 500$.

Transitional flow: can be classified as neither laminar nor turbulent flow
$500 < Re < 12{,}500$

Turbulent flow
In this type of flow the inertial forces are strong relatively to viscous forces.
12,500 > Re
Frounde Number (F) it show the ratio between inertial and gravitational forces, based on the F the flow is classified as follows:
Critical , $F = 1$
Subcritical , $F < 1$
Supercritical, $F > 1$

$F = U/ (g L)^{0.5}$

L is the characteristic length equal to hydraulic depth (D)
D = Area /Top width
If F = 1, the flow is in critical condition, the inertial and gravitational forces are in equilibrium condition and the flow is called *Critical flow*.

F < 1 the flow is called subcritical flow and
F > 1 the flow called supercritical flow.

8.2 Channel Types

Natural channels are developed by natural processes.

Artificial channels which are developed by human efforts
Classification of Artificial channels:
Prismatic : A prismatic channel has both a constant cross-sectional shape and bottom slope. Channels, which do not meet these criteria, are termed non-prismatic channels.

Canal: It is long channel of mild slope, may be either lined or unlined.

Flume : The term flume refers to a channel built above the ground surface to convey a flow across depression.

Chute and Drops: A chute is channel having steep slope. A drop channel has also steep slope but much shorter than chute.

Culvert: It is primarily used to convey water under road, highway, railway or pathway etc.

Section Elements of Channel
Channel Section : Cross-section of channel taken normal to the flow direction

Depth of flow Y : The vertical distance from the lowest point of a channel section to water surface; $Y = d / \cos \theta$, where θ is the slope angle of the channel bottom with a horizontal line.

Stage: The stage of a flow is the elevation of the water surface relative to a datum. If the lowest point of a channel section is taken at the datum then stage and depth of flow is equal.

Top Width: The top width of a channel is the width of a channel section at the water surface.

Flow Area: The flow area is the cross-sectional area of the flow taken normal to the direction of flow.

Wetted Perimeter: The wetted perimeter is the length of the line, which is interface between the fluid and the channel boundary.

Hydraulic Radius: The hydraulic radius is the ratio of the flow area to the wetted perimeter or
$R = A/p$

Hydraulic Depth D: The hydraulic depth is the ratio of the flow area to the top width.
$D = A/T$

8.3 Procedure for Design of Irrigation Canal

The purpose of irrigation channel is to convey irrigation water from a source to the farms to be used for agricultural production. Design objectives include; achieving maximum conveyance efficiency (low water loss) preventing silting or erosion and having the full supply level (FSL) in the watercourse high enough to provide a proper working head to all of the command area.

Small irrigation channels improvement programs are utmost necessary of today's irrigated agriculture of Afghanistan. About 30 to 50% of water is lost in irrigation channels from Karex, river diversion or other irrigation systems before it reaches to the fields. Most of the theses irrigation channels have been constructed by the water users, without proper alignment, cross-section and gradient. This resulted in varying unfavorable velocities leading to silting, scouring, and overtopping of banks, narrow banks and dead storages. Poor haphazard maintenance and inadequate cleaning has lead to the vegetative growth, breaches and overtopping. Rodents' holes, cutting of banks for turnout, and borrowing of soils from adjoining fields to build checks in the watercourse, resulting in considerable amount of water losses.

I. Selection Criteria for Improvement of Irrigation Canal

The channel selected for improvement should be those where there will be a high likelihood for success; where farmers' interests are high. In selecting irrigation channel

the following factors should be considered for improvement:
Farmers must be willing to provide Labors required to improve the irrigation channel (water course) and to organize and direct the during the planning and construction phase.
Efforts should be made to select the watercourse having high water losses.
Priority should be given to the watercourse having percentage of rural poor or small landowners.

II. Information

The objectives of the watercourse improvement should be communicated to the water users through radio, TV, newspaper, poster and personal contact in the villages.
The farmers should be informed about the benefit of the watercourse improvement in term of water saving and reduction in water logging and salinity.
The cost, in term of input required from the farmers.
Water users association should be formed for proper execution and maintenance of the watercourse improvement programme.

III. Involvement of Farmers

Involvement of farmers in phases of improvement (planning. design, construction, monitoring and evaluation) should be ensured.

IV. Planning the Watercourse Improvements project

Preparation of watercourse improvement plan is one of the essential phase of the project and it involved the following steps:

Application from the farmers for the command area for assistance.
Conduct farmers meeting to explain the project objectives.
Make a soil survey of the watercourse, to fid the high water losses portion of the watercourse.
Collect from the concerned Department map of the irrigation scheme or it should be prepared.
Find the discharge of the watercourse.
Use cutthroat flume or other devices for assessment of conveyance losses.
Determine the amount of kutcha and pikka improvement needed.

V. Final Plan and Design

The construction detailed should be discussed with the farmers and necessary changes should be made. Then the final design should be submitted for sanctioned and approval to the concerned authority.

Field Survey

First conduct a closed circuit bench mark (BM) starting at a benchmark on the outlet assumed to be 10-meter elevation. One Pucca BM should be established such as stones, pump bases, tube well structures etc at each 300m to facilitate further surveying. All

surveys should be started and closed on appoint of known elevation. If the error of closure is not within specified limits, the survey must be repeated. The permissible deviation is 0.05 cm per point. Obtain the Full Supply Level (FSL), crest elevation of the outlets.
Conduct a profile survey of bed elevation and average field elevations of the main and all branches of the main irrigation channel. The elevation of the highest fields to be served by each turnout is needed. Prepare a profile map of the main and branch watercourses by plotting the profile, the location and elevation of all the existing water control structures. Locate all the turnouts on the profile maps.

Find the maximum FSL to be able to serve the highest field. To the required FSL at turnout, add 0.15 m to the highest field elevation to provide the working head needed to move water from the watercourse to the field. If the highest field is located down stream add a minimum of 0.01 m for each 100 m of distance. Water Surface Profile Water surface profile should be plotted on the map. Select the size and shape of the watercourse and check the hydraulics. Manning's equation is generally used for flow characteristics. The design velocity should be less than erosive velocity. Sedimentation may occur if the velocity is too low. It may not be practical to change the flow depth if it varies few cm. Botto width or flow depth can be adjusted to compensate for a change in slope.

8.4 Canal Layout

In this section design procedure, which includes preparing the layout of the watercourses, preparation of the longitudinal profiles, design of watercourse section and other structures are discussed.

Canal Structures

In this section the various structures present in a tubewell irrigation system will be discussed. Besides explaining their purpose, some rules of thumb and allowable dimension will be given.

Lined watercourses: Walls of lined watercourses are constructed with bricks and the floor is constructed using concrete. All the exposed surfaces of the watercourse are plastered. Slope of the beds of watercourses within the limits of 1:250 to 1:100 are used.

Unlined watercourses: An unlined watercourse is an earthen channel having trapezoidal shape. In small schemes the bed slope of the channel should be between a minimum and maximum limit of 1:250 to 1:100 respectively.

Water Control Structures : The location of turnouts should be shown on the map. Check structures are used in the watercourse to provide working head needed at turnouts. The spacing between the check structures can be estimated by the following equation.

$L = FB/(2^* S)$

L is the distance upstream in meter a check structure can serve.

FB = Freeboard (m)

S = bed slope in m/m

Example
FB = 0.15 m
S = 0.0004 m/m
L = 0.15 m/(2* 0.0004 m/m) = 187.5 m, therefore the check structures can serve turnouts as far as 188 m upstream.

Drop structure: Two kinds of drop structure: a bedstep for a limited drop (maximum drop of 0.40 metres) and a drop structure for places where water drops within a range of 0.40 - 1.00 metres. A drop structure is a brick work construction which gives protection against the falling water and dissipates the energy. Rules of thumb for designing a drop structure are: a) for discharges up to 30 l/s the depth of the stilling basin should be 0.5 metres. The length of the stilling basin should amount 2 times the depth.

Division box: It is a structure which consists of 2 or 3 lids (nuccas). A division box regulates the direction in which water flows by closing the lids except the one in which direction the water has to flow. The diameter of a lid amounts 12" for discharges up to 30 l/s.

Culvert: A culvert is used where a road, path or track crosses the channel. They are also placed to give the possibility for every farmer to reach his field by tractor without damaging the channel. A culvert is a strong, flat object which covers the water course. The standard with of a culvert is 4 metre. The height should be as low as possible in order to simplify the construction and to make its use easier.

Nucca: A nucca is a structure to regulate and distribute water from a water course to the individual fields.

Slab: When no culvert is needed because only people or animals pass a certain place at the water course, a slab can be located. The advantage of a slab compared to a culvert is that it is much cheaper. In fact, a slab is a culvert without special reinforcements. The maximum width of a slab is 1 metre otherwise the farmers will start using it for heavy objects as well.

Designing irrigation schemes is a complex and reiterative process. While designing the designer makes many assumptions and takes many decisions. Often it appears in a later step of the design procedure that an assumption made or a decision taken in a previous step is not correct. Then the designer has to go back to the step where the wrong assumption has been made and start designing from that step onwards again. A design procedure starts with making a rough layout of the scheme, followed by drawing a longitudinal profile map and then calculating the dimensions of watercourses and other structures. The last step of design is to make a final layout of the scheme.

Canal Layout: The first step in the design the layout will be prepared. A layout map indicates all irrigation infrastructure, it includes stilling basins, watercourses, secondary channels, field channels, position of field nuccas and other structures such as culverts (tractor paths) and washing place (if required by farmers' wives). The topographic map and contour map of a scheme are used to prepare a layout map. The layout of an irrigation system can be prepared in the following steps:

a. locate the position of the stilling basin(s) which must be at the highest point(s) in the scheme;
b. starting from the stilling basin locate the watercourse while taking the existing field boundaries as indicated in the topographic map and the elevations on the contour map into consideration;
c. locate the position of field nuccas, division boxes and culverts (tractor paths).

All structures on the layout map require to be labelled so that their identification becomes easy. Prepare also a readable legend for your layout design. The abbreviations commonly used are as follows:

Structure	Abbrev.
watercourse	W.C.
secondary channel	S.C.
field channel	F.C.
drop structure	D.S.
Nucca	N
Culvert	C.T.
Slab	S
Pipe	P

An example of layout map prepared for a PATA project scheme is given in Figure 8.1. It is important to indicate the different structures (eg. division box, nucca, culvert, etc.) in a different way, so, that it becomes easy to distinguish them.

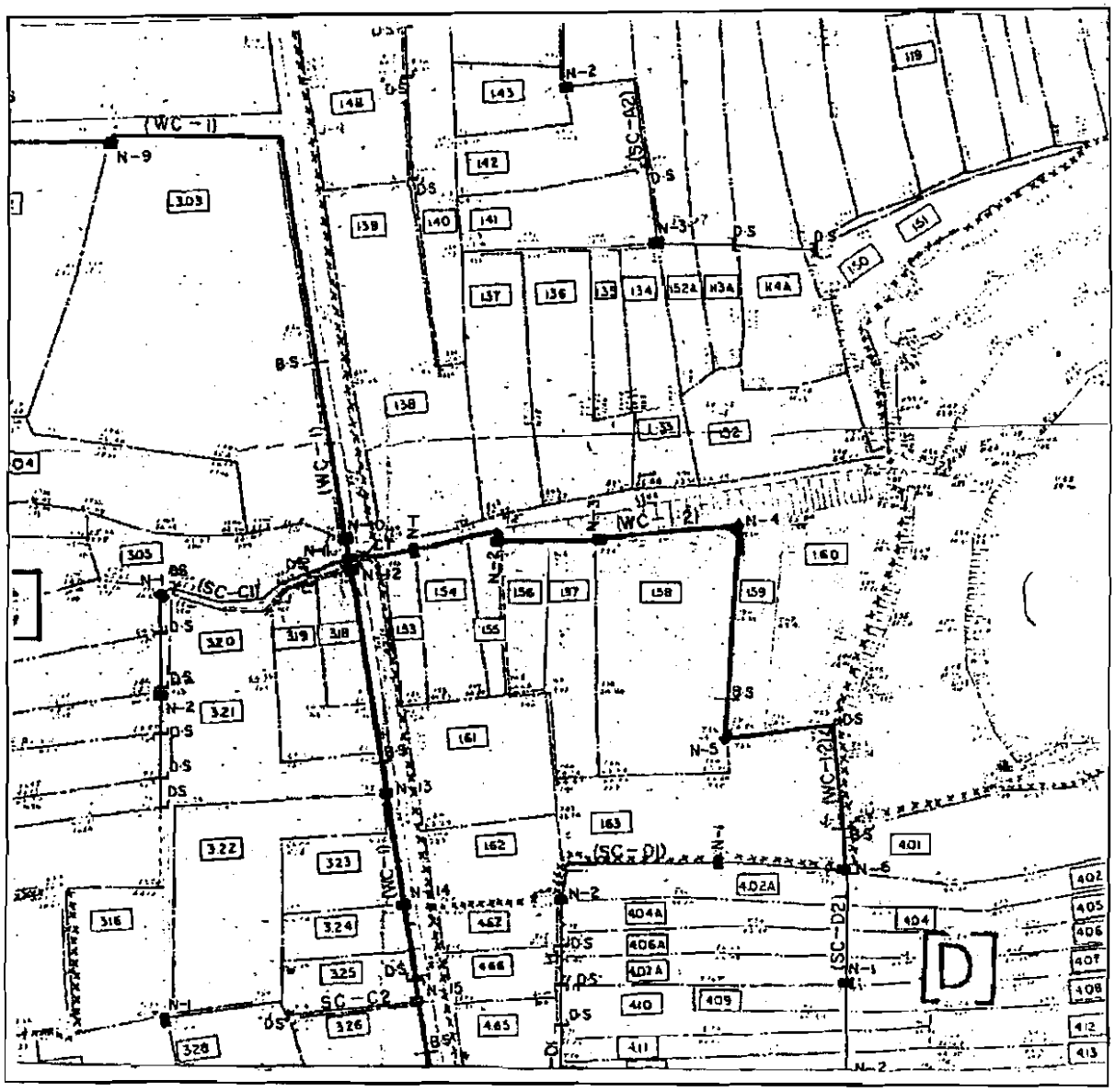


Figure 8.1. Example of layout of an irrigation scheme.

Longitudinal Profile

After preparing the layout map of the scheme, the next step is to draw a longitudinal profile of the existing ground levels of the watercourses. The longitudinal profile maps are required by the construction engineers in the field for construction of various structures. The profile maps are drawn on graph paper. Elevations of the canal bed, canal walls, and the water level are indicated on the Y-axis (vertical) while the distance is on the X-axis (horizontal). Before the drawing a longitudinal profile has started, suitable vertical and horizontal scales have to be selected.

A design of a longitudinal profile of a channel can be developed in the following steps:

a. Select suitable scales for the elevations (vertical) and distance (horizontal). For example a scale of 1:100 means that 1 mm on the map represents 100 mm or 10 cm in reality.

b. Draw the existing ground level profile on which the watercourse is to be constructed. An example of a ground level profile is line 1 in Figure 8.2.

c. Decide what will be the bed slope of the new watercourse. Note that a designer will always try to minimize cut and fill in order to reduce construction costs. However, the slope of a watercourse should always be chosen within the allowable range. Draw the bed of the watercourse according to the selected slope(s). An example of the bed slope of a watercourse is represented by line 2 in Figure 8.2.

d. Drop structures can be placed at points where the difference in height between the bed slope line and the ground level profile line cross a certain limit (the maximum fill allowed is normally 1.00 meters). Drop structures should be located in such a way that cut and fill are minimized.

e. The next step will be to indicate the height of the watercourse walls, water depth and the position of nuccas on the profile map. In order to do so, you have to compute the exact dimensions of the watercourse. These dimensions are calculated using the Manning formula.

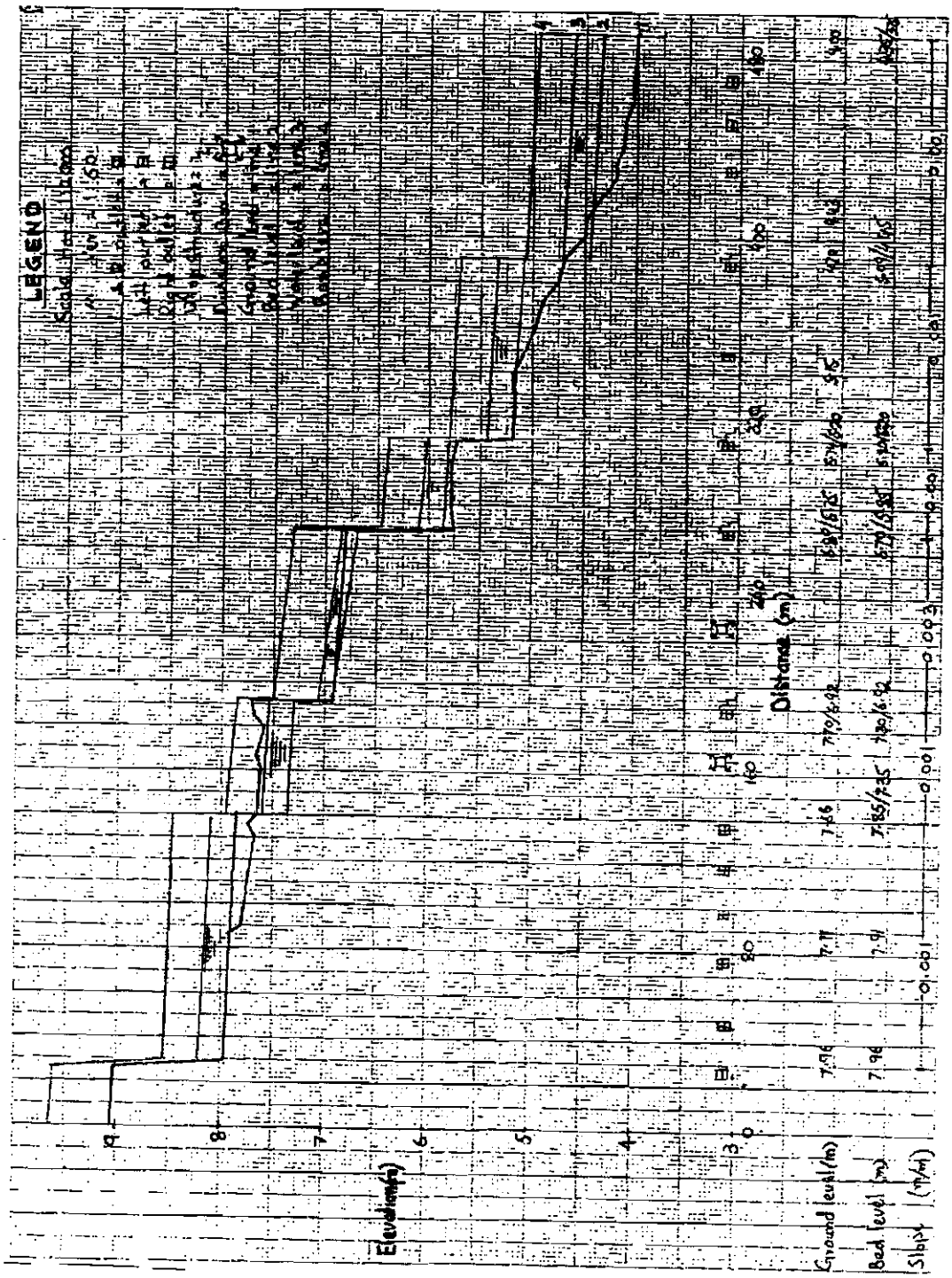


Figure 8.2 . Example of a longitudinal profile design of a watercourse.

Manning's Formula

Manning formula is the most widely used equation for the design of canals and watercourses is given below

$$Q = AV$$

$$Q = \frac{1}{n} R^{2/3} \sqrt{S}$$

Q = flow rate in m^3/s, V = flow velocity in m/s.
A = Flow area in m^2, P = wetted perimeter in m and R hydraulic radius in m.

Detailed Design of Structures

After calculating the cross section of the watercourse, it has to be drawn on graph paper. The other special structures like the drop structures, division boxes, and culverts have to be designed as well. The design of these structures consists of a top-view and a side-view (cross section). The design of complex structures often require separate designs for different cross-sections. Note that all detailed designs should be on scale.

Final Layout

On the basis of the previous steps, the exact layout indicating all special structures can be made. Before the final layout is made, PATA organizes a design meeting with the farmers in which the design is explained to the farmers with the help of the design model and a scheme walk. During these meetings, the different design options and their advantages and disadvantages can be discussed with the farmers. In this way, the suggestions of the future users can be included in the final design.

Permissible Velocity:

Flow velocities in earthen canal should be non-erosive for the soil material in which it is constructed. Erosion will occurs if the velocity is too high. For checking velocities, the lower expected "n" values of the newly constructed or cleaned should be used. Drop structures may be required to reduce the channel slope and velocity Table 8.1 and 8.2. The permissible velocity in a channel depends on its cross-section and longitudinal gradient. The longitudinal gradient adopted shall be such that as to produce a velocity as high as possible without scouring bed and sides of the canal and sacrificing command.

Table 8.1. Non-erosive velocities in m/s

Channel bed material	Water with sediment (m/s)	Clean water (m/s)
1. Very fine sand	0.75	0.45
2. Sandy loam	0.75	0.55
3. Silty loam	0.90	0.60
4. Alluvial silt	1.00	0.60
5. Dense clay	1.00	0.75
Source Manual of surface Drainage Engineering Vol 1. B.Z. Kinort, Elsevier publishing Co. N.Y., 1970. In case of mixing of clean water with muddy water, weighted average may be calculated to obtain permissible velocity		

Table 8.2. Maximum permissible velocities for different types of soils.

S. No	Description of the Soils	Maximum Velocity m/s
1	Light sand	0.50
2	Loamy sand	0.75
3	Clay soft	1.00
4	Clay hard	1.25
5	Soft rock	1.5-2.5
6	Hard rock	2.5-3.5

Minimum non-silting and non-weed velocity is taken as 0.5 m/s.

The minimum permissible velocity refers to the lowest velocity, which will prevent both sedimentation and vegetative growth. In general, an average of 0.61 to 0.91 m/s will prevent sedimentation when the sediment load is low. A velocity of 0.76 m/s is usually sufficient to prevent the growth of vegetation which could significantly affect the conveyance of the channel.

Freeboard:

Freeboard is bank height above the maximum designed water surface elevating that should be provided to allow for the most sever operating condition. Minimum free board requirements are as follows: Earthen channel 1/3 of the design depth or 15 cm whichever is greater. Rectangular lined channel 10 cm and for Trapezoidal channel 7.5 cm.

Freeboard is provided for berm settlement, siltation, or expected poor maintenance. Minimum freeboard requirements are given in Table 8.3 for small irrigation channels (discharge up to 300 L/s).

Table 8.3 Minimum Freeboard requirements for irrigation channels

1. Earthen Channels	1/3 of the design flow depth or 15 cm whichever is greater
2. Rectangular lined channels	10 cm
3. Trapezoidal lined channels	7.5 cm
Source : On-farm Water Management Field Manual , Govt. of Pakistan	

For unlined large canal the freeboard generally ranged from 0.3 m for shallow depth of flow to 1.2 m for channel carrying 85 m3/s at relatively large depth. A preliminary estimate of freeboard for an unlined channel can be obtained from

$$F = (C\,Y)^{0.5}$$

Where F = Freeboard in feet

Y = design depth of flow, feet

C = coefficient which varies from 1.5 at Q = 20 ft^3/s (0.57 m^3/s) to 2.5 for Q = 3000 ft^3/s (85 m^3/s)

Table 8.4: Free Board for large canal.

Full supply discharge, cfs	Free Boards, ft.
0 – 250	1.0
251 – 500	2.0
501 – 1000	2.5
1001 – 5000	3.0
5001 – above	4.0

Full Supply Level

The design water surface elevation should be high enough at field take-out points to provide required flow onto the field. A minimum working head of 10 to 15 cm should be provided at each turnout for the highest field elevation it commands.

Side-slope for Earthen Channels

Earthen watercourses should be designed to have stable side slopes. Side slopes for design flow depths that are not greater than one meter are given in Table 8.5

Table 8.5. Permissible side slope (Z) in earthen channels.

	Excavated section		Fill Section	
	Permissible Z	Recommended Z	Permissible Z	Recommended Z
1. Loam Silt loams, silty clays and clays	1:1	1:1	1:1	1.5:1
2. Sandy loams	1.1	1:1	1.5:1	2:1
3. Loamy sands and sand	1.5:1	2:1	2:1	3:1

Source: On Farm Water Management Field Manual Govt. of Pakistan.

For large canals the side slope (Z) for different types of soil are given in Table 8.6.

Table 8.6. Permissible side slope (Z) in large earthen canals (Chow, 1959).

Material	Permissible Z
1. Rock	Near vertical
2. Muck and peat soils	¼:1
3. Stiff clay or earth with concrete lining	½:1 to 1:1
4. Earth with stone lining or earth for large channels	1:1
5. Firm clay or earth for small ditches	1 ½:1
6. Loose sandy, earth	2:1
7. Sandy loam of porous clay	3:1

In may unlined earthen canals side slopes of 1.5:1 have been used , however, side slopes as steep as 1: 1 can be been used when the channels runs through cohesive materials. In lined canals the side slopes are generally steeper than in unlined canals.

8.5 Watercourses Seepage losses

The seepage loss rate in earthen watercourses depends primarily on soil texture and density. Course textured soils, such as sands have higher rate than fine textured soil. Similarly, loose un-compacted soils due to rodent and insect activity, plant root disturbance as well as berm cutting for turnout etc have higher seepage rate than dense compacted soils. If the estimated losses are too high, better earthen material can be brought in or the watercourse lined to reduced seepage. The estimated seepage losses are given in Table 8.7.

Table 8.7. Expected seepage losses in small earthen watercourses.

	Seepage losses per 1000 m in percent		Seepage losses per 1000 m in Liter /s	
Material	Q = 10 l/s	Q = 300 l/s	Q = 10 l/s	Q = 300 l/s
1.Fined textured soils (clay and clay loam's)	11	2	2	4.0
2. Medium textured soils (loams and silts)	20	2.5	2.6	7.5
3. Coarse Textured Soils (Sandy loam's)	40	4.8	4.5	12.5

Source: On Farm Water Management Field Manual Govt. of Pakistan.

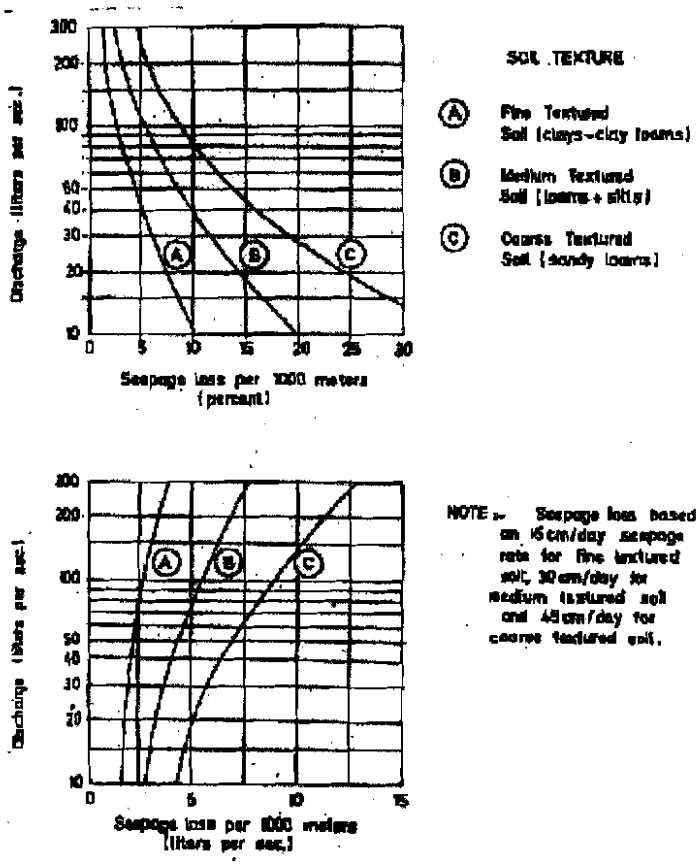


Figure 8.3 Seepage losses from small irrigation canal.

Surface Roughness (n)
Manning n depends on several factors
Large boulder, stone and gravel usually collect at the bottom of channel and result in high value of n at low stage and relatively low value at higher stage.
Vegetation: The estimated value of n should take into account the effect of vegetation in retarding flow and increasing n. In general, the relative importance of vegetation on n is a strong function of the depth of flow and the height, density, distribution and type of vegetation.

Channel Irregularity: The variation in the channel cross-section, shape, and wetted perimeter along the longitudinal axis of the channel. In general, gradual variation have a rather insignificant effect on n, change while abrupt changes can result in a much higher value of n than would be expected.

Obstructions: The presence of obstruction such as fallen trees, debris flows and log or debris jams can have significant impact on the value of n.

Channel Alignment: While curves of large radius without frequent changes in the direction of curvatures offer relatively little resistance, severe meandering with curves of small radius will significantly increase the value of n.

Sedimentation and Scouring; In general, active sedimentation and scouring yield channel variation which results in an increase value of n .

Stage and Discharge: The n value for most channels tends to decrease with an increase in the stage and discharge.

Table 8.8. Basic n values suggested by U.S Soil Conservation Services in 1963.

Channels character	Basic n value
Channels in earth	0.02
Channels cut into rock	0.025
Channels in fine gravel	0.024
Channels in coarse grave	0.028

Berm Width or Bank Width

The berm width of the watercourse should be equal to the flow depth, but not less than 0.30 meter. For lined section, the berm width includes the lining width plus earthen width. Berm width in large canal is shown in Table 8.9

Table 8.9:Berm Widths for large canal.

Full supply discharge, cfs	Berm Widths, ft.
0 – 100	10
101 – 200	15
201 – 500	20
501 – 1000	30
1001 – 2000	40
2001 – 6000	50
6001 – 9000	55
9001 – 11000	60
11001 – 14000	65
14001 – 18000	70
18001 – above	75

8.6 Design of lined Channels

Lined channels are built for five primary reasons:

- To permit the transmission of water at high velocities through area of deep or difficult excavation in a cost effective fashion.
- To permit the transmission of water at high cost at reduced construction cost.
- To decrease canal seepage, thus conserving water and reducing the water logging of lands adjacent to the canal.

- To reduce to annual costs of operation and maintenance.
- To ensure the stability of the channel section.

Best Hydraulic Section

It has been found that the conveyance of channel increases as the hydraulic radius increases or the wetted perimeter decreases. Among all the possible channels cross-sections, the best hydraulic section is a semicircle, for a given area, it has minimum wetted perimeter. However, it should be noted that the best hydraulic section is not always the most economical section. In practice the following factors must be considered:

The best hydraulic section minimize the area required to convey a specified flow; however, the area which must be excavated to achieve the flow area required by the best hydraulic section may be significantly larger if the overburden must be considered.

It may not be possible to construct a stable best hydraulic section in the available natural material. If the channel must be lined the cost of the lining must be comparable with the cost of excavation. The cost of the excavation depends not only on the amount of material, which must be removed but also on the ease of access to the site and the cost of disposing of the material removed. The slope of the channel in many cases must also be considered a variable since it is not necessarily completely defined by topographic consideration.

Table 8.10. Geometric elements of best hydraulic sections.

Cross sections	Area (A)	Wetted perimeters (P)	Hydraulic Radius (R)	Top Width (T)	Hydraulic Depth (D)
Trapezoidal : half of hexagon	1.73 Y^2	3.46 Y	0.500 Y	2.31 Y	0.750 Y
Rectangular: half of a square	2 Y^2	4 Y	0.500 Y	2 Y	Y
Triangular	2 Y^2	2.83 Y	0.354 Y	2 Y	0.500 Y
Semicircle	0.500 π Y^2	Π Y	0.500 Y	2 Y	0.250 ϑ Y
Parabola T = 2 $\sqrt{2}$ Y	1.89 Y^2	3.77 Y	0.500 Y	2.83 Y	0.667 Y

8.7 Design Criteria of Irrigation Canal

The purpose of the irrigation channel is to carry the water from Karez, river diversion, spring or Tube well to the farmers' fields. The design objectives include: achieving conveyance efficiency (low water losses), preventing silting or erosion and the Full Supply level in the in the watercourse high enough to provide proper working head to all the command area and low to prevent the submergence of outlet.

Capacity Requirements

The design capacity of the irrigation channel should be measured with the help of a cutthroat flume or current meter or float method. The capacity of the new irrigation channel will depends upon the command area to be irrigated and type of crops expected to be grown in the area. The water allowance can be assumed 0.5, 0.7 and 1.0 L/S/ha for Northern, central and western Afghanistan respectively.

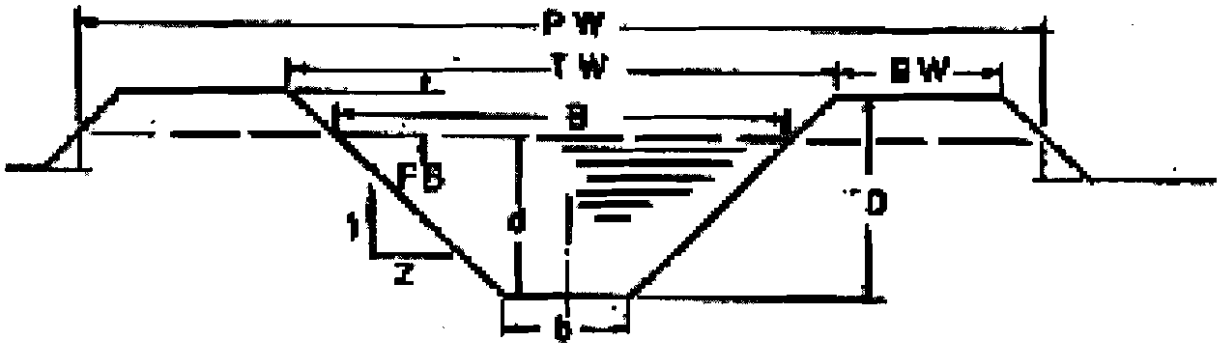


Figure 8.4 Typical cross-section of a small irrigation channel

b = Bottom width
d = Flow depth
B = width at water surface = b + 2 Zd
D = Total depth of the canal = d + FB
FB = Free board
Z = Side slope
FSL = Full Supply level
TW = Top width of the channel = b + 2

B. Permissible Velocity

The permissible velocity in a channel depends on its cross-section and longitudinal gradient. The longitudinal gradient adopted shall be such that as to produce a velocity as high as possible without scouring bed and sides of the canal and sacrificing command. High velocity has the following advantages: Reduces cross-sectional area and such has less excavation. Reduced transmission losses, low silt deposition and periodical removal. Retards weed growth. The maximum permissible velocity for various soils is given in Table 8.8

Manning's n

Manning n for small lined and unlined irrigation channel is given in Table 8.11

Table 8.11. Roughness coefficient n for small irrigation channels.

S .No	Unlined earthen channel	Minimum	Maximum	Recommended
1	New straight and uniform	0.02	0.025	0.025
2	Aged and vegetated with a. Short grasses b. Long grasses	 0.03 0.05	 0.040 0.080	0.035
	Lined irrigation Channels			
1	Concrete	0.012	0.018	0.014
2	Brick plastered	0.012	0.018	0.013
3	Unplastered	0.016	0.020	0.018

Field Survey

First conduct a closed circuit bench mark (BM) starting at a benchmark on the outlet assumed to be 10-meter elevation. One Pucca BM should be established such as stones, pump bases, tube well structures etc at each 300m to facilitate further surveying. All surveys should be started and closed on appoint of known elevation. If the error of closure is not within specified limits, the survey must be repeated. The permissible deviation is 0.05 cm per point.

Obtain the Full Supply Level (FSL), crest elevation of the outlets.

Conduct a profile survey of bed elevation and average field elevations of the main and all branches of the main irrigation channel.

The elevation of the highest fields to be served by each turnout is needed.

Prepare a profile map of the main and branch watercourses by plotting the profile, the location and elevation of all the existing water control structures. Locate all the turnouts on the profile maps.

FSL

Find the maximum FSL to able to serve the highest field.

Required FSL at Turnout.

Add 0.15 m to the highest field elevation to provide the working head needed to move water from the watercourse to the field. If the highest field is located down stream add a minimum of 0.01 m for each 100 m of distance.

Water Surface Profile

Water surface profile should be plotted on the map.

Select the size and shape of the watercourse and check the hydraulics. Manning's equation is generally used for flow characteristics.

The design velocity should be less than erosive velocity.

Sedimentation may occur if the velocity is too low.

It may not be practical to change the flow depth if it varies few cm.

Bottom width or flow depth can be adjusted to compensate for a change in slope.

The design of lined irrigation canal is very simple, since there is no sever restriction on higher or lower velocity, however, the velocity in the canal should be 0.5 to 2.0 m/s. Following design procedure may be adopted.

Estimate n for the given lining material.

Compute as per Manning's equation , section factor

$AR^{2/3} = (n\ Q) / S^{0.5}$

Assume the shape of the lined section as trapezoidal, with suitable side slope and bed width

From Figure 8.5 $AR^{2/3}/b^{8/3}$ and y/b , the value of y can be determined.

For best hydraulic cross-section, channel parameters can be used from Table 8.10.

Make a sketch providing the entire dimension.

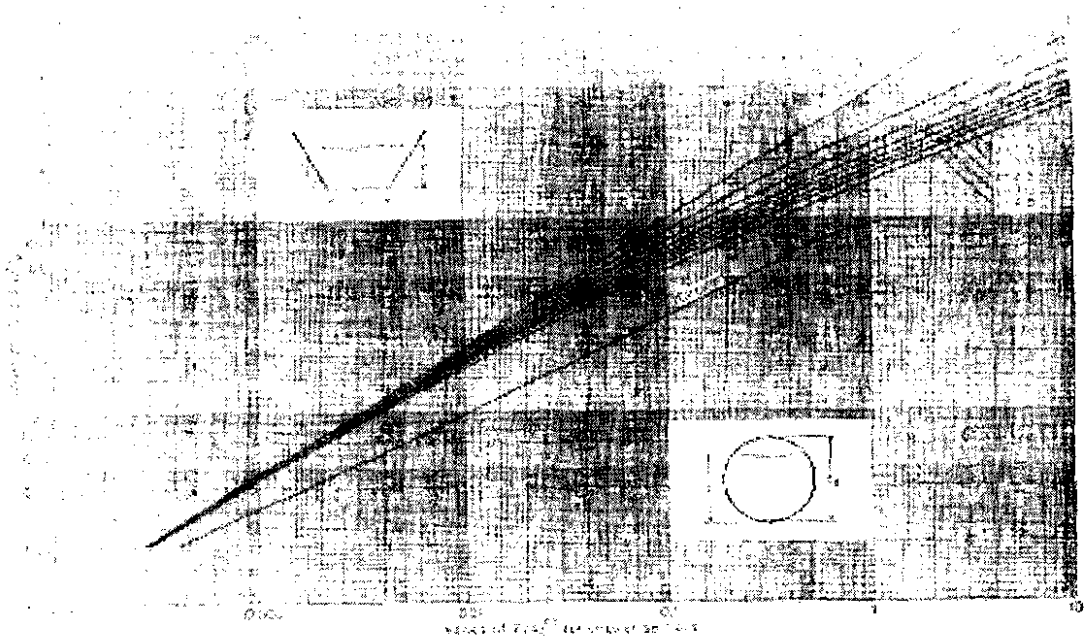


Figure 8.5 Curves for normal flow depth determination.

Manning equation for uniform flow is the most commonly used relationship for determining discharge:

$Q = (1/n)\ A\ R^{2/3}\ S^{1/2}$

Where Q = flow rate in m^3/s,

n = Manning's roughness coefficient

A = Cross-sectional area in m^2

S = bed slope for uniform flow (energy grade line)

For Trapezoidal section channel

$A = d\ (B + Z\ d)$

R = Area / Wetted perimeter

$R = \{ d (B + Zd) / [B + 2d (Z^2 + 1)^{1/2}]\}$

Where B = Bottom width in m

d = flow depth in m

Z = Side slope

$d = \{[(n Q) / S^{1/2}]^{3/5} [B+2d (Z^2 + 1)]^{1/2} \}^{2/5} / (B+Z d)$

Following equation is used as an iterative solution for d by using the old values d' in the right hand side to compute an improved values of d

(d – d') /d [allowable error in depth

The discharge at a given cross-section in a channel is dependent on the shape of the watercourse. The cross-section with a minimum wetted perimeter is the hydraulically optimum cross-section or best hydraulic section. The following equation will give the hydraulically optimum cross-section.

$d = 1.892 (n Q/S^{0.5})^{0.375} \{ [1/(2 (Z^2 + 1)^{0.5})]^{0.375} \}$

$B = d (2 (Z^2 + 1)^{0.5} - 2 Z$

8.8 Design Procedure for Lined Canal

Estimate Q, S and n for specified lining material;

Compute value of section factor

$AR^{2/3} = [n Q] / S^{0.5}$

Solve section factor equation for y given appropriate expression for A and R (Figure 8.5) for section factor equation solution techniques. This step may require assumptions regarding side slope, bottom widths etc.

If the best hydraulic section is required , complete channel parameters from Table 8.10.; otherwise compute channel parameters, using y from step 3.

Check
1. Minimum permissible velocity if carries silt and for vegetation.
2. Froude Number.

Estimate required height of lining above water surface and freeboard .
Summarize results with dimension sketches.

Example

Design a lined water course in Ghazni to carry a discharge of 20 l/s , longitudinal slope of the channel is 0.004. Double brick lining is recommended with rectangular section and n = 0.014 for brick lining.

Solution :

For best hydraulic section B = 2 d

$AR^{2/3} = (nQ) / S^{0.5} = 0.014 \times 20 \times 10^{-3} / 0.004 = 0.004427$

$A = B \times d = 2d \times d = 2d^2$

$P = B + 2d = 2d + 2d = 4\,d$

$R = A/P = 2d^2 / (4d)$

$AR^{2/3} = [\,2d^2\,]\,[\,2d^2 / (4d)\,]^{2/3} = 0.004427$

d 8/3 = 0.004427

d = 0.12 m

B = 2 d = 0.24 m

Total depth = d + freeboard = 0.12+ 0.15 = 0.27 m

Example

A lined channel of trapezoidal cross section is to be designed to carry 400 cfs (11 m3/s) of water. The lining of the canal is to be float –finished concrete and the longitudinal slope of the canal is 0.0016. Determine channel proportions.

Step 1. Estimate Manning n from Table 4.8; n = 0.015

Step 2. Section Factor $AR^{2/3} = [\,n\,Q\,] / S^{0.5} = 0.015 * 400/ 1.49\,(0.0016)^{0.5} = 100.7$

Step 3. Assume B = 20 ft (6.1 m) and Z = 2 and solve for y Figure 8.5

y = 2.5 ft (0.76m)

Step 4 Since the best hydraulic section is not required, this step is omitted

Step 5. Check minimum permissible velocity and Froude Number

$A = (B + ZY)\,y = (20 + 2(2.5))2.5 = 62.5\ ft^2 = 5.8\ m^2$

Velocity V = Q/A = 400/62.5 = 6.4 ft /s (2.0 m/s)

This velocity should prevent vegetative growth and sedimentation.

$F = V/(gD)^{0.5} = 0.78$ (sub-critical flow.

Step 6. Estimate height of lining above water surface (Figure 2)

Height of lining above the water surface = 1.2 ft (0.37 m)

Height of canal bank above the water surface = 2.9 ft (0.88 m).

Design of Large Capacity Lined and Unlined Canals

Non-alluvial soils are hard soils e.g loam, clay, moorum, boulder, etc. They are stable for the purpose of canal design and there is no silt problem. The only criterion in the channel design of these channels is the adoption of non-scouring velocity close the critical velocity of the soils. Table 8.12 gives the values of critical velocities

Table 8.12: Critical velocities for different soils.

S. No	Description of the Soils	Critical Velocity m/s
1	Earth	0.3-0.6
2	Ordinary moorum	0.6-0.9
3	Hard moorum	1.2
4	Boulders	1.5-1.8
5	Soft rocks	1.8-2.4
6	Hard rock	> 3.0

Table 8.13: Values of Manning's Co-Efficient, n for various types of linings

S.No.	Type of Lining	n
1	Portland cement concrete lining	0.014
2	Asphalt concrete lining (machine placed)	0.014
3	Brick lining covered with cement plaster	0.014
4	Soil cement, well finished	0.015
5	Soil cement rough as a grave surface	0.016
6	Pre-cast concrete-block lining	0.015 – 0.017
7	Brick lining with exposed brick surface (design value) Actual measured value of 'n' on the same lining after deterioration	0.0146 0.018 – 0.02
8	Shotcrete lining, smoothed	0.016
9	Shotcrete lining (average)	0.017
10	Compacted earth lining small canals	0.025
11	Compacted earth linings, large canals	0.0225 – 0.020

Table 8.14: Radii of Curves for Canals

Unlined Canals		Lined Canals	
Discharge Cfs	Radii Min. ft.	Discharge cfs.	Radii Min ft.
250 & above	5000	1000 & above	3000
Between 250 to 100	3000	Between 1000 to 700	2500
Between 100 to 50	1800	Between 700 to 500	2000
Between 50 to 10	1000	Between 500 to 200	1500
		Between 200 to 150	1000

Example

Design an irrigation channel to carry a discharge of 14.16 cumecs with a velocity of 0.775 m/s, assume side slope 1:1; bed slope 1 in 5000 and Manning's n = 0.0225

Solution:

From equation $Q = A \times V$

$A = Q/V = 14.16/0.775 = 18.27\ m^2$.

$A = (B + D)\,D = 18.27\ m^2$

Wetted perimeter

$$P = B + 2\,D\,\sqrt{2}$$

Hydraulic Radius = $R = 18.27 / (B + 2\sqrt{2}\,D)$

Using manning equation another value of R is found out

$$V = 1/n\ R^{2/3}\ S^{1/2}$$

$$0.775 = 1/0.0225\ R^{2/3} \times (1/5000)^{0.5}\ ;\ R = 1.37\ m$$

From equations

$$1.37 = 18.27 / (B + 2\sqrt{2}\,D)$$

$$B = 18.27/1.37 - 2\sqrt{2}\,D = 13.336 - 2.8328\,D$$

Substituting the value of B in equation [1]

$$18.27 = (13.336 - 2.828\,D)\,D + D^2$$

$$D^2 - 13.336\,D + 18.27 = 0$$

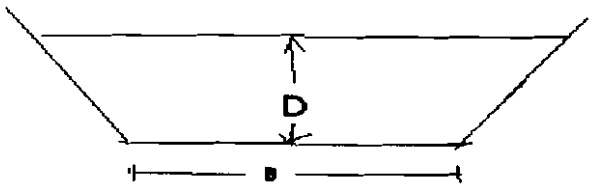
It is a quadratic equation in D , solving for D

D = 5.476 and 1.828

Taking D = 5.476

$B = 13.336 - 2.828 \times 5.476 = -2.15$ m which is not possible
Taking $D = 1.828$ m
$B = 13.336 - 2.828 \times 1.828 = 8.17$ m
$A = (8.17 + 1.828)\ 1.828 = 18.3\ m^2$, hence it is correct.

Example 8.2 Design of Large Capacity Canal
Design an unlined irrigation canal to carry a discharge of 14.16 cumecs with velocity of 0.775 m/s, Assume the side slope of 1:1 and; bed slope 1:5000, n = 0.0225.
Solution:
From equation
$Q = A \times V$
$A = Q/V = 14.14/0.775 = 18.27\ m^2$
$A = (B + D) \times D = 18.27 = BD + D^2$ (i)
$P = B + 2D[(2)^{1/2}]$



Hydraulic mean radius
$R = 18.27/(B + 2D(2)^{1/2})$ (ii)
Using Manning's equation

$V = 1/n\ R^{2/3}\ S^{1/2}$
$R^{2/3} = 0.755 \times 0.0225 \times (5000)^{1/2}$
$R = 1.37$ m (iii)

Equating (ii) and (iii)
$1.37 = 18.27/(B + 2\ (2)^{1/2} \times D$
$B = 18.27/1.37 - 2\ (2)^{1/2}\ D$

$B = 13.336 - 2.828\ D$

Substituting the value of B in equation (i)

$18.27 = (13.336 - 2.828\ D)D + D^2$

$18.27 = -1.828\ D^2 + 13.336\ D$

$1.828\ D2 - 13.336\ D + 18.27 = 0$

Quadratic equation

$D = 13.336 \pm [\ (13.336)^2 - 4 \times 1.828 \times 18.27]^{1/2} / (2 \times 1.828)$

$D = 5.476$ m , $D = 1.826$ m

$D = 5.476$ m

$B = 13.336 - 2.828 \times 5.476$

$B = -2.15$ Which is not possible

$B\ 13.336 - 2.828 \times 1.828 = 8.17$ m

A $((8.17 + 18.28)\ 1.828 = 18.3$

Hence it is correct

$D = 1.83$ m

$B = 8.2$ m

Free Board = $(CY)^{1/2}$

$F = (\ C\ Y\ )^{0.5}$

Where F = Freeboard in feet

Y = design depth of flow, feet

C = coefficient which varies from 1.5 at $Q = 20\ ft^3/s$ ($0.57\ m^3/s$) to 2.5 for $Q = 3000\ ft^3/s$ ($85\ m^3/s$)

Let $C = 2.0$

$F = (2 \times 6)^{1/2} = 3.5$ feet = 1.0 m

Total Depth = 1.83 + 1.0 = 2.83 m

Balancing Depth

Balancing depth is that depth of a canal section, such that the quantity of excavation from the bed is equal to earth filling required for making banks. A canal running in balancing depth is very economical in construction.

h = Height of the bank above the canal bed level (d + F.B)

d = canal full supply level

b = design bed with

y = balancing depth

z = bed cutting slope

n = bank filling slope

t = bank with at top

Area of cutting = (b + zy) y

Area of filling = 2 [t + n (h-y)] (h-y)

For the balancing depth

Area of cutting = Area of filling
Usual value of Z and n are 1 and 1.5 corresponding to cutting slope of 1:1 and filling slope 1.5:1. Putting these values
$(b+y)y = 2(h-y)[t + 1.5(h-y)]$
On simplification, we get
$y^2 - y(b/2 + 3h + t) + h(t + 3/2\,h) = 0$

Example
A canal has a bed width of 10 m, F.S. depth of 1.5 m, bank width 2m, cutting slope 1:1, and filling slope 1.5:1 and free board 0.5 m. Calculate balancing depth.
Solution:
H = 2.0 m (d + F.B)
B = 10 m
t = 2 m
Using the equation 1
$y^2 - y(10/2 + 3x\,2 + 2) + 2(2 + 3/2 \times 2) = 0$
$y^2 - 13y + 10 = 0$
$y = +13 \pm \sqrt{(13^2 - 4 \times 10)}/2$
$y = 13 \pm 11.35/2 = 0.82$ m

Check
Cutting = $(b + zy)y = (10 + 0.82)\,0.82 = 9\ m^2$.
Filling = $2[2 + 1.5(2.0 - 0.82)](2.0 - 0.82) = 9\ m^2$.

Minimum working head
Working head at field = 7.5 cm
Working head at outlet = 15 cm
Working head from:
Major distributory to a minor = 30 cm
Branch to major distributory = 50 cm
Main branch to branch = 50 cm
Main canal to main branch = 60 cm
An off take is of a channel is placed upstream of a contour point in the parent channel as for as possible.

8.9 Lining Material

An ideal canal lining should be impervious, strong, durable, and adaptable to the shape of the canal prism, inexpensive and easy to construct. Concrete, earth and asphalt have been used in many forms as lining material. Soil sealants have also been used with some success as an inexpensive method of control of seepage from existing canals but could not be used as a permanent canal lining. One of the classifications of the various types of lining may be as under.

A- Rigid and Semi-rigid linings
  1 Portland Cement concrete
  2 Asphalt Lining
  3 Brick Masonry

B- Earth lining
  1 Compacted Earth (Thin) Lining
  2 Compacted Earth (Thick) Lining
  3 Bentonite Lining

C- Membrane Linings
  1 Buried Asphalt Membrane
  2 Plastic Film

D- Soil Sealants
  1 Chemical Sealants

9 PREDICTING DESIGN RUNOFF

Runoff is defined as that portion of the rainfall which is neither infiltrated into the ground, stored on surface, nor evaporated, but which flows over the land. Predictions and estimations of runoff are of primary importance in the design of conservation structures since the flowing water is both a cause of erosion and source of water supply. Runoff peak rate and volume are described in this module.

Most soil conservation engineering is done on small watersheds, since one of the principles of conservation is to control the flow of water as close as possible to the place where it starts. This is sometime referred to as "upstream engineering". In the wet season of monsoon, the emphasis is on safe disposal of the runoff in the humid and sub-humid zones, i.e., in a manner which will cause minimum soil erosion. In semi-arid and arid zones, the emphasis is more on retaining the runoff for agricultural purposes as characteristic of the runoff; for design purposes, is the peak rate of discharge. In the semi-arid and arid regions, peak runoff rate is also important in some instances, but it is always necessary to estimate volume of runoff, durations and frequencies.

The predictions of surface runoff are complicated. These vary from simplistic models relating runoff to catchment area and return period, to highly complex mathematical models.

The mathematical model provide better estimates and predictions, but requires a very good bank of data (climatological, hydrological, geological, agricultural, etc.) which is often not available, which field studies are time consuming, laborious and expensive, and require the services of highly trained professionals. The field data and results are generally site-specific, with limited relevance to other areas.

This conclusion can be most discouraging to the conservation engineer/scientist, especially those working in Pakistan's Barani areas. They normally turn to the older literature which will give them simple, empirical rules of thumb, however incorrect these may be for their specific site. They will then try to allow for the uncertainty factor by a degree of over design. This process can be refined over the years by close observation of the results in the field, with corrections for future designs. The problem of over design, however, is not only one of economics; there is sometimes a question as to which are the 'safe' line of action of computations. For example, a structure may be over designed for strength, but undersigned for safety because of the excessive weight while designing concrete pipe structures.

Objectives

The module objectives are to:

* define and describe method for predicting peak runoff rate of small watersheds and volume runoff of a rain storm;
* identify field data limitations for predicting design runoff;
* develop skills in the use of rainfall and runoff data and information to improve predictions for subsequent design;
* Explain computational procedures for predicting runoff;
* upgrade computational procedures through corrective feed back.

Approach

A practical and simplified approach is adopted to describe the prediction methods for estimation of peak runoff rate and volume runoff for barani lands and mountains in Pakistan. The purpose is to assist soil and water conservationist design conservation measures. Also to understand limitations in data availability and chances of error in borrowing coefficients from other locations/countries. It will also remind scientists of the limitations regarding non-availability of data and inspire urgent research studies.

Climatic, hydrological and biological data available from local sources have been used to calibrate or adjust some of the borrowed data. The purpose is to initiate an effort based on local conditions and constraints.

9.1 Peak Runoff Rate

In Barani lands, it is necessary to estimate the peak runoff rates for designing spillways, weirs, culverts, waterways and other outlets which will carry the runoff at the height of the monsoon season when the soil storage capacity is saturated.

Structures and waterways are planned to carry maximum runoff, which can be expected in a specified recurrence interval. The recommended peak runoff frequencies for various types of structures are given in Table 9.1.

There are three main methods commonly used in predicting the peak runoff rate: a) Rational Method; b) Cook's Method, and c) Hydrologic Soil-Cover Complex Method. The assumptions made in the derivation of these methods are:

* rainfall occurs at uniform intensity for a duration at least equal to the time of concentration of the catchment.
* rainfall occurs on a uniform intensity over the entire catchment.

The basic assumptions in the derivation of these three methods are same. Therefore, selection of a method depends on: a) preference of the user; b) availability of data required for a prediction method; and c) reliability of predictions.

The soil conservationists in Pakistan are primarily using Rational Method for predicting peak runoff rate of small catchments. As the assumptions are same in all the three methods, the selection of a particular method will depend largely on the availability of data and users' preference. Considering these aspects, the Rational method is selected for predicting peak runoff rate.

9.2 Rational method

The Rational Method is a time honoured procedure developed by Ramser. This is simplest and possibly the most consistent one in its ability to adjust with the new developments in evaluating catchment characteristics. The method is expressed by the following equation (Schwab et al. 1956).

$$Q = 0.003 * CIA$$

where

Q	=	peak runoff rate for design purpose, m^3s^{-1};
C	=	runoff coefficient representing ratio of runoff to rainfall;
I	=	design rainfall intensity; mm h^{-1};
A	=	catchment area, ha.

Runoff Coefficient

The value of `C' is dependent on the intensity and amount of rainfall, surface cover and gradient, and antecedent soil moisture content. The values of `C' estimated from small plots are normally higher than actual field conditions. Ahmad et al. (1990) presented rainfall-runoff models based on the field plot data collected in loess soils of Fatehjang Target Area. Three surface gradients namely 1,5 and 10% and two surface cover conditions of fallow and cropped were used to collect runoff volume and rainfall data of individual storms during 1983-88. Plot size of 2m x 5m with tow replications were used. Later Ahmad and Aslam (1994) presented rainfall-runoff models based on the field data of small watersheds of 16 ha each. The slope of these watersheds rages 10-30% and vegetal cover is predominantly of scrub species namely Olea ferruginea, Dodonaea Viscosa, Gvmnosporia rovleana, Carissa opaca, and accasia species. Similar trend of `C' value was observed. Singh et al. (1990) presented values of `C' for varying vegetative cover, slope and soil texture. Different sets of `C' values are given for cultivated land, pasture land, and forest land.

Based on the available data in Pakistan `C' values given by Singh et al. (1990) were adjusted for Pakistani conditions. The values of `C' are presented in Table 9.2.

Time of Concentration

The time of concentration of a catchment is the time required for water to flow from the most remote point of the area to the outlet once the soil becomes saturated and minor depressions filled. It is assumed that, when the duration of a storm equals the time of

concentration, all parts of the watershed are contributing simultaneously to the runoff rate at the outlet. Kirpich (1940) described a method of computing the time of concentration.

$$Tc = 0.02\, L^{0.77}\, S^{-0.385}$$

Tc = time of concentration, min;
L = maximum length of flow, m;
S = catchment gradient in m^{-1} on the difference in elevation between the outlet and the most remote point divided by the length, L.

Hydrologists are not in agreement as to the best procedure for computing the time of concentration.

The WRRI-NARC maintained 3 gauging stations in small watersheds of about 16 ha each. The hydrolgraphs were used to estimate time of concentration. The recommended average velocities for estimating time of concentration are given in Table 9.4. This information can also be used in determining time of concentration.

9.3 Design Intensity of Rainfall

The design intensity of rainfall for any location can be estimated using relationship.

$$I = kT^{a} / (Tc + b)^{n}$$

where

I = intensity of rainfall for a given recurrence interval; mm ha^{-1};
T = recurrence interval;
Tc = time of concentration;
a,b,n,k = coefficients for a given location.

The values of maximum rainfall for 24 hours duration and design rainfall intensity for return period of 30 years are available in Pakistan (Table 9.3). If time of concentration is assumed one hour then design rainfall intensity can be used for recurrence interval of 30 years. The design rainfall is presented for four classes considering the 24 hours rainfall and one hour maximum rainfall intensity. The suggested values of design rainfall intensity are more appropriate because these are measured values (PMD 1971). Estimation of design intensities requires calibration of Eq. under Pakistani conditions which requires much more data than available today. The researchers are encouraged to collect data in collaboration with Soil Conservation Directorate.

9.4 Predicting Runoff Volume

The predictions of surface runoff volume are needed to estimate amount of water flowing from the small catchments. The volume predictions help to estimate amount of soil loss

or amount of water available for domestic, stock water or irrigation purposes. The total volume of runoff is also of primary interest in the design of flood control reservoirs.

The Soil Conservation Service of the United States in 1964 presented a method for predicting runoff. This method was developed using long term storm flow records of agricultural watersheds in the US. The relationship presented by SCS (1964) is as under:

$$Dr = (Dsr - 0.2\,d)^2 / Dsr + 0.8\,d$$

where

Dr = depth of surface runoff, mm;
Dsr = depth of storm rainfall, mm;
d = maximum potential difference between rainfall and runoff in mm, starting at the time of the storm's beginning.

The relationship developed for USA in the SCS method is difficult to extrapolate and adjust to the conditions prevailing in Pakistan. However, if data of `Dsr' and `d' are available Eq.9.4 can be used for predicting runoff volumes.

Ahmad et al. (1990) presented models for estimation of runoff depth based on the storm rainfall depth. These models were developed for cropped and fallow conditions using three surface gradients namely 1,5 and 10% slope. The relationship used is as under:

$$Dr = a + b*Dsr$$

Values of `a' and `b' for loess soils of Attock district are given in Table 9.5.
Ahmad and Aslam (1994) presented models for runoff estimation of degraded scrub forests at Satrameel with slope exceeding 30% .

$$Dr = -5.04 + 0.61 *Dsr$$

Runoff volume can be estimated by multiplying `Dr' with area of catchment in ha. The relationship is as under:

$$Vr = 10.0 *Dr*A$$

Where

Vr = volume of runoff, m^3;
Dr = depth of runoff, mm;
A = area of catchment, ha.

The data regarding rainfall in Barani lands are presented in Figures 9.1, 9.2 and 9.3 representing percent of annual rainfall received during July to September, maximum rainfall of 24 hours duration, and average rainfall intensity, respectively.

Solved Examples

Compute peak runoff rate of a cultivated field of 10 ha size with surface gradient of 5% and design rainfall intensity of 100 mm h^{-1}. The soil texture is silt-loam.

Solution

From Table 9.2 find value of `C' using:

- cultivated field
- silt-loam soil or medium soil class
- slope 5%

$C = 0.40$

Then $Q = 0.003 * CIA$

$Q = 0.003 * 0.4*100*10$

$= 1.2\ m^3s$

$= 42.3\ ft^3/s$

Example

Compute time of concentration for a field having maximum length of flow of 500 m and area of 10 ha. The gradient is 5%.

Solution

The time of concentration is computed using relationship of :

$T_c = 0.02 * L^{0.77} S^{-0.385}$

$= 0.02 * 500^{0.77} * .05^{-0.385}$

$= 7.6$ minutes

Using Table 9.4 the velocity for 5% slope is 1.2 m S^{-1}

$T_c = L/V$

$= 500/1.2 = 417$ seconds

$= 7.0$ minutes

The time of concentration computed from both the methods is almost same. This shows that any method can be used to estimate time of concentration.

Example

Estimate runoff volume of storm specifications given in above example. If rainfall amount is 250 mm and area is within loess soils. The field is cropped.

Solution

The values of `a' and `b' from Table 9.5 are:

$a = -5.77$

$b = 0.62$

The depth of surface runoff is computed as :

$D_r = a+b *D_{sr}$

$= -5.77 + 0.62 *250$

$= 149$ mm

The volume runoff is:

$V_r = 10 * D_r * A$

$= \quad 10 * 149 * 10$
$= \quad 14900\ m^3$

Table 9.1: Recommended maximum runoff frequencies for various types of structures.

Type of Structure	Frequency (year)
➢ Storage and diversion dams having permanent spillways	50-100
➢ Earth fill dams - storage having natural spillways	25-50
➢ Stock water dams	50
➢ Small permanent masonry gully control structures.	10-15
➢ Terrace outlets and vegetated waterways	10
➢ Field diversions	***15***

Table 9.2 Values of runoff coefficient "C" for use in the rational method.

Vegetative Cover and Slope	Soil Classes		
	Light	Medium	Fine
1. Cultivated Land			
0 to 5%	0.3	0.4	0.5
5 to 10%	0.4	0.5	0.6
10 to 30%	0.5	0.6	0.7
2. Pasture Land			
0 to 5%	0.1	0.2	0.3
5 to 10%	0.15	0.3	0.4
10 to 30%	0.2	0.4	0.5
3. Forest Land			
0 to 5%	0.1	0.2	0.3
5 to 10%	0.25	0.3	0.4
10 to 30%	0.3	0.4	0.5

Table 9.3 Design rainfall intensities for selected areas.

Location Return period (years)	5	10	25	100
	Design Rainfall Intensity (mm/hour)			
Pallanddri - AJK	100	125	150	190
Haripur - NWFP	95	115	140	180
Chkwal - Punjab	65	80	100	130
Jhelum - Punjab	50	60	75	95
Barkhan - Balochistan	30	45	50	65
Zhob - Balochistan	30	35	45	55
Loralai - Balochistan	30	35	40	50
Bostan - Balochistan	25	30	35	45
Quetta - Balochistan	25	25	30	40
Khuzdar - Balochistan	25	30	35	42
Las Bela - Balochistan	35	45	55	75
Saruna - Balochistan	40	50	65	85

Source:
Punjab 1 in 25 year Data ADAB Training Modules - NARC
Other return period Punjab data estimated from 1 in 25 year Punjab data using same ratios as Balochistan data
Balochistan, Halcrow Flood Estimation Manual for BMIADP

Table 9.4: Recommended average velocities for use in estimating time of concentration.

Average channel slope (%) from remote point of catchments to the outlet	Velocity m S^{-1}
1-2	0.6
2-4	0.9
4-6	1.2
6-10	1.5

Table 9.5: Coefficients of runoff model for Attock district (Ahmad et al. 1990).

Surface Cover	Surface Gradient	Linear Model Coefficients a	b
Fallow and unploughed	1	- 5.34	0.61
	5	-5.52	0.67
	10	-5.19	0.71
Cropped	1	-6.57	0.57
	5	-5.77	0.62
	10	-5.19	0.65

Table 9.6: Runoff from catchments

Average annual rainfall(mm)	Annual Evaporation (mm)	Loams	Sandy clay	clay
		Annual Runnoff in %		
over 1100	1300	13	13	20
901- 1100	1300	11	13	18
501- 900	1500	7	9	8
401- 500	1500	4	8	6
250- 400	1500	2	3	5

10 HYDRAULIC STRUCTURES

10.1 Types of Cross Drainage Structures

Aqueduct:
When the canal bed is above the maximum level of the water surface in the natural drain, then the canal is carried over the drain in RCC flume or RCC pipe or a steel pipe (depending on the canal discharge) supported on piers. The flow in the aqueduct is and the drain is an open channel flow. In the aqueduct, since the canal is flumed, a well-designed transition is at he entry and exit is provided.

Siphon Aqueduct
When the maximum water level is above the bed of the canal drain while then the canal is carried unflumed, over the drain, while the bed of drain is lowered and the drain passes underneath the canal through RCC barrels of square section or RCC pipe. While the flow in the canal is open channel, the flow in the drain through the barrels. No transition is provided for the canal.

Supper passage
When the FSL in the canal is below the bed of the drain, then the canal continues to flow unflumed and the natural drain water is carried out in RCC flume over the canal. This is opposite to the aqueduct.

Siphon Supper-passage
This is opposite to a siphon-aqueduct. When the bed of the natural drain is below the FSL of the canal, the canal water is siphoned below the bed of the drain through RCC pipes or barrels or just be dropping the bed of the canal such that the water level u/s and d/s in the canal are touching the sides of the RCC flume carrying the drain water so that the flow in the canal is under pressure.

Level Crossing
When the bed of the drain and the canal are almost at the same level, then the water of the two are allowed to mix and the canal supplies are regulated through a regulator. The design of the regulator is on the same line as that for a weir for surface flow conditions.

Drainage inlet
Instead of providing a structure to carry the water across the canal, the surface runoff due to rainfall is allowed to flow into the canal at suitable place. Such a situation is arises when the general direction of flow of the canal is perpendicular to the ground slope. Thus the surface runoff gets obstructed and should be allowed in the canal. This arrangement is economical since surface runoff occurs only a few times during the year, and the discharge is not high. Runoff water is collected outside the canal and when the level outside touches the calculated marks the gates are opened and water flows into the canal.

The canal is designed to carry this extra discharge.

Drainage Outlet

This is the same type of structure as an inlet built on the opposite bank of the canal slightly d/s of the drainage inlet. It discharges the extra water that has entered the canal through the inlet into an escape channel. The crest of the weir is kept slightly above the FSL so that if and when extra water enters the anal it automatically spills over the weir.

Tail Escape

This last structure on a minor or distributary and does not come under the definition of cross drainage work

Design Procedure for an Aqueduct

The primary consideration in the in the design of an aqueduct carrying a canal water is the minimum loss of head across the flume or in other words, the difference of water level u/s and d/s or the heading up or afflux at u/s is minimum. A drop in water level d/s would mean less command area. An excessive afflux at u/s would mean higher banks and higher crest of the regulating structures of the off taking braches and distributaries.

Head Loss = C $[V^2 - V_a^2/2g]$

Where V = Velocity in flume

Va = velocity in canal

With warped wall transition the C ranges from 0.1 to 0.4

Head loss at entrance = k $V^2/2g$, k varies from 0.5 for an entrance with sharp edge to 0.08 for a streamlined like the bell mouth entrance.

Total head loss (H) = $h_{ent} + h_2 + h_{exit}$ will depend on the shape of the inlet and exit and friction losses.

h_2 = loss due to friction can be computed by slope of the energy line

$S = [V \times n/R^{2/3}]$, n = 0.012 for steel pipe and concrete conduits, n = 0.014 for monolithic concrete conduit. For a canal with discharge less than 8.49 m3/s the H= 0.15 m is commonly used for design purposes.

Velocity A maximum velocity up to 1.5 m/s can be used. A flume ratio of 40-60% can be used.

Bed slope : The slope for the open flume aqueduct shall be parallel to the slope of the energy line.

Free board : Free board (height of the structures above the full supply level of the canal) are as follows:

Capacity (m3/s)	0-2.83	2.83-7.08	7.08-14.15	> 14.15
Free board (m)	0.30	0.45	0.60	0.80

10.2 Hydraulic Design of Siphon

A siphon aqueduct or siphon supper passage carries the discharge of one drain beneath the bed of another. For this purpose one or several pipes, masonry or RCC pipes may be used. Normally RCC pipes or barrels are used for both strength and economy.

The velocity in the barrel is greater than in the canal u/s and d/s. This is to achieve economy and also to prevent accumulation of silt in the siphon. The velocity in the siphon should be from 2.5 to 3.00 m/s to avoid siltation and should be less than 1.53 ms/.

Loss of head at inlet or outlet (h1)

$h1 = f1\ V^2/2g + (V^2 - V_a{}^{2)} / 2g$

V = Velocity in the pipe , Va = Velocity of approach in the canal

f1 = 0.084 for bell mouth and f1 = 0.505 for circular or cylindrical entrance with sharp edges.

For a good transition with a bell mouth the total head loss at entrance or exit becomes

$h1 = 0.084\ V^2/2g + 0.1\ (V^2 - V_a{}^{2)} / 2g$

Loss of head due to Skin friction

S can be found from Manning's formula

$S = [\ V \times n/R^{2/3}]$, n = 0.012 for steel pipe and concrete conduits, n = 0.014 for monolithic concrete conduit. Head loss h_2 = S x length of barrel.

Slope of siphon : These depends upon the specific situation and topography. However, the inclined portions of the siphon may have slope not greater than 2:1 followed by horizontal portion.

Uplift pressure on Siphon Structure: There are two way in which uplift pressures assumes significance and have to be given due consideration.

Uplift pressure acting on the roof of the barrel when the canal or the drain passing over the siphon is dry and siphon is running ful.

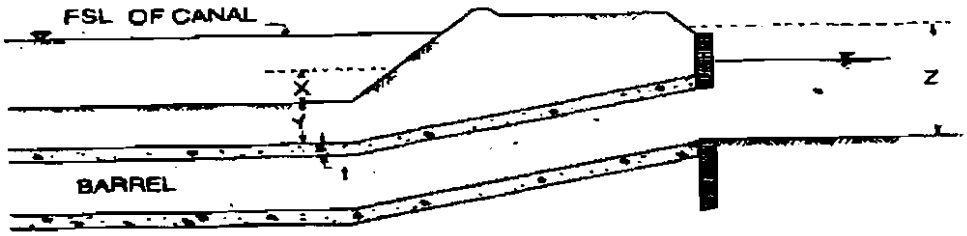


Figure 10.1 Uplift pressures on siphon structure.

The total uplift pressure on the roof
$= \gamma (x + y + t)$ lbs /ft^2

$= \gamma (x + y + t) - (\gamma_1 t + \gamma_2 y)$

where γ_1 = specific weight of concrete ;γ_2 = specific weight of earth
and γ specific weight of water

Loss of head due to bend (h3)

$H3 = F1\ V^2/2g$

According to Gibson $F1 = 0.0000676\ \theta^{2.17}$
Weishach gives F in term of R and D as follows:
$F1 = [\ 0.13 + 0.16\ (D/R)^{3.5}\ ]\ \delta/90$
Head loss = $k\ V^2/2g$; $k = (\delta/90)^{1/2}$

Total head loss = h1(entrance) + h2 (friction losses) + h3 (bend losses) + h1 (exit)

Design Steps for Siphon
The data given should be maximum discharge to pass through the siphon, maximum permissible heading up, bed level and water level.
Select a maximum permissible velocity of say 2.44 m/s
As a first approximation, determine the area of the siphon
Determine the number of pipes from the total sectional area.
$N \times \pi D2/4 \geq A$; N should be a whole number.
Determine the exact area, corresponding to N.
In order to determine the head loss, first determine the length of the barrel and bends by drawing the canal or drain cross-section and then determine the total head loss.
If the value of H is less than the permissible value, the calculated values of N and diameter or sizes are correct. If not then repeat the calculation.
For new condition check the value of velocity and it should be around 2.44 m/s.

10.3 Culvert Design
Highways, roads or canals traveling the land cut across individual watershed. To allow the flow from each watershed across the embankment culverts are built at the lowest point of the valleys.
The fundamental objective of hydraulic design of culvert is to determine the most economic size a which the design discharge is passed without exceeding the allowable headwater elevation, the major components of a culvert are its inlet, culvert size, and its outlet with exit energy dissipater.

10.4 Types of Control

Inlet Control

If the discharge of a culvert is dependent only on the headwater above the invert at entrance, the size of the pipe and geometric of the entrance. We find that the slope, length, and roughness of the culvert pipe do not influence the discharge

Outlet control

When the discharge is dependent on all hydraulics variables of the structures. These include the slope S_0, length L, diameter D, and roughness n, tail water depth TW, head water depth HW.

Optimum Slope

$S_{optimum} = 111\ [n^2 / 3\sqrt{D}]$

D in feet and Slope in ft/ft

Optimum discharge

$Q_{optimimum)} = 5.27\ D^{5/2}$

Q in cfs and D in feet

10.5 Design of Drop Structures

The longitudinal slopes of the canals are fixed as per sediment transport capacity for non-silting and non-scouring velocity. It may happened that the topography of the land on which the canal is to be laid is steeper than the designed bed slopes of the canal, resulting an excessive filling and high embankment on which canal is to be laid. This results in an increase in the cost of construction and, also the differential head, the difference between the ground water table and the water level in the canal may cause excessive seepage which results in water logging. To avoid this the, the bed of the canal is given a sudden drop or fall at a suitable place, so that it may run partly in excavation and partly in filling, depending on the command area. The structure build to safe guard the drop is called a canal fall structure, which is integral part of the canal system.

Kinds of Drops

- Vertical Drops
- Inclined Drops
- Piped Drops

Location of the Falls

Suitable site will be where the depth of excavation becomes less than balancing depth. Balancing depth is that depth of a canal section, such that the quantity of excavation from the bed is equal to the earth filling.

Falls should be located downstream of the outlets .

The site should be selected keeping in mind the requirements for a road crossing, as bridge combined with a fall offers an economical structure

A regulator may be combined with a fall d/s of the off-taking channel.

Vertical Drop Falls

The cistern element is that portion of the d/s floor, which is depressed below the normal bed level, to provide a cushion of water to take the impact of falling water.

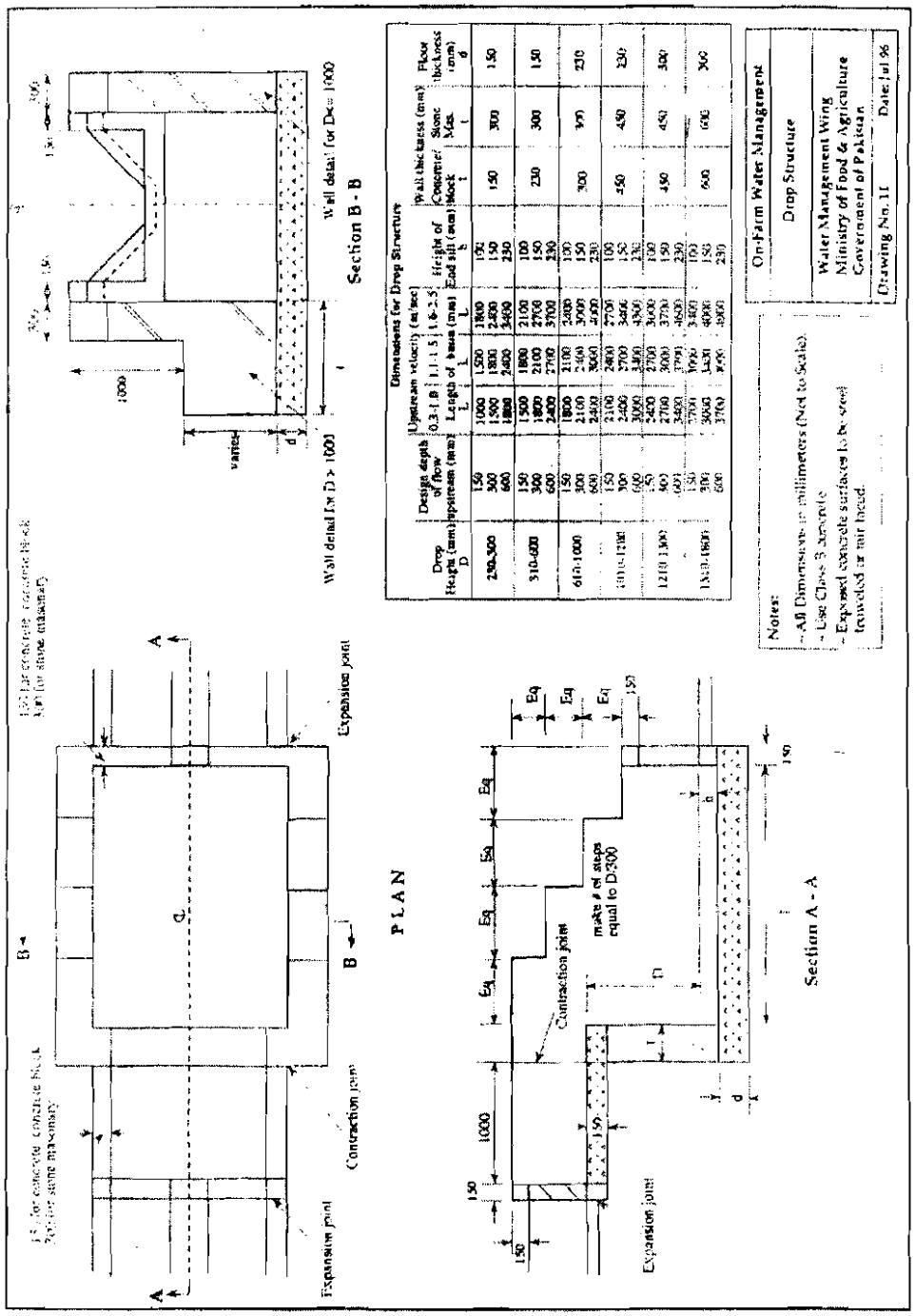


Figure 10.2. Design of vertical fall drops.

1. Eichvery Formula

$L_c = 3 X \sqrt{H_L} . E$

$X = [1/6] L_c$

Lc = Cistern length

X = Depth below bed level

$E = H + V_a^2/2g$,

H = Depth of water upstream in the canal

2. Blench formula

$(X + D2) = 2Dc + ¼ [H_L - 3/8 Dc]$

In no case should HL be less than 3/8 Dc, where Dc is the critical depth.

3. Dyas formula

$X = HL \times D^{1/3}$

Bahadarabad Irrigating Research Institute Formula

$Lc = 5 \sqrt{(E + H_L)}$

$X = [1/6] \sqrt{(E H_L)}$

Mushtaq and Uppal

Lc = 6 D2

X = 0.4 Dc

Sarda Type Fall

The length of the crest is kept equal to bed width of the canal. For channel carrying a discharge of more than 14.15 m^3/s, the crest width

$B = \sqrt{(H + d)}$ and for discharge less than 14.15 m^3/s $B = \sqrt{d}$,

H = Depth of water over the crest,

d = height of the crest above the d/s bed

Height above the u/s bed

$Q = C (2g)^{0.5} L H^{3/2} [H/B]^{1/6}$

C = 0.45 for the crest of trapezoidal shape and 0.415 for rectangular shape

Example

A canal is crossing a stream under siphonic condition. Design a siphon with the following data:

Stream

Discharge Q	=	400 cfs
Highest Flood Level	=	475.58 (at the point of crossing)
Bed Width, B	=	100 ft.

Canal

Q	=	100 cfs.
V	=	2.5 ft/sec
n	=	0.0225 (unlined)
F.S.L	=	478.70
Bed level	=	474.70
B.W.	=	10 ft.
Full Supply Depth	=	4 ft.
Free Board	=	1.5 ft.
Hydraulic Slope	=	1:4000

Solution

To determine which type of cross drainage structure would be suitable at this place, consider the following:

Canal discharge = 100 cfs

Stream discharge= 400 cfs

Stream discharge is greater than the canal discharge.

Highest Flood Level (H.F.L.)of the stream at the point of crossing in 475.58 whereas the bed level of canal is 474.70. Difference = 0.88 ft.

The difference is too small as free board for an aqueduct.

In view of the above, the canal will cross the stream through a siphon.

Length of siphon	=	Bed Width of Stream + 10% +Side Slopes
	=	100 + 10% + 2x45
	=	200 ft.
Discharge of siphon	=	100 + 10%
	=	110 cfs
Assume velocity in the barrels	=	4 ft./sec (For self-cleaning purposes)

Design of concrete square barrels

A	=	Q/V
	=	$110/4 = 27.5\ ft^2$
Let side of barrel	=	a
Area of barrel	=	a x a
a^2	=	11.0
a	=	5.25 ft.

Barrel will be pre-cast, concrete sections.

Scour

Depth of scour, R	=	$0.9(q^2/f)^{1/3}$
Assume, f	=	1.0
	=	$0.9\ (4^2/1.0)^{1/3}$ q = Q/B = 400/100 = 4
	=	2.27 ft.
Max Scour Depth	=	2 R
2 R	=	2 x 2.27 = 4.54 ft.

Barrel will be laid in the following shape at a level at least 4.54 ft. below the bed level.
Head Loss = Inlet loss + Friction loss through the barrel + Loss at bends + Loss at exit.
Loss at Inlet, hi

hi	=	$0.5\ V^2/2g$		
V	=	4 ft./sec		
hi	=	$0.5\ (4^2/2 \times 32.2)$	=	0.12 ft.

Friction loss, hf
The loss of head due to friction can be measured by the slope of the energy line or the water surface in the barrel. This is calculated by Manning's formula. Total head loss due to friction would be equal to :

hf	=	S x Length of the barrel.	
n	=	0.014	
Area of barrel, A	=	$27.5\ ft^2$	
Wetted Perimeter, P	=	4x5.25	= 21 ft
Hydraulic Mean Depth, R	=	A/P	
	=	27.5/21	=1.31
Velocity, V	=	$(1.486/n) \times R^{2/3} \times S^{1/2}$	
V	=	$(1.486/0.014) \times 1.31^{2/3} \times S^{1/2}$	
S	=	0.03	

Loss due to friction, hf = S x Length of the barrel = 0.03 x 200 = 6 ft.
Loss of head at bends, h_b

h_b	=	$2(V^2.\ f/2g)$	
where f	=	$\sin^2 \delta/2 + 2 \sin^4 \delta/2$	
δ	=	11.3^o	
$\delta/2$	=	5.65^o	
f	=	$(\sin 5.65)^2 + 2\ (\sin 5.65)^4$	
	=	0.009992	
h_b	=	$2 \times 4^2/2g \times f$	= 0.005 ft

Loss of head at Exit, h_e

h_e	=	$0.5/2g\ (V^2 - V_1^2)$	
Where V	=	Vel. In barrel	
V_1	=	Vel. In canal	
h_e	=	0.5/2g (4-2.5)	= 0.012 ft.

Total Head Loss	=	$h_i + h_f + h_b + h_e$	
	=	0.12+6+0.005+1.012	= 6.14 ft.

F.S.L. of canal on the downstream of the siphon

	=	F.S.L. – Head Loss
	=	478.70 – 6.14
	=	472.56

10.6 Practical Design Examples

Discharge Calculation of Channel

C.C.A = 330 Acres.
Take Max. = 6 feet.

$$\text{Discharge (Q)} = \frac{6 \times 330 \times 43560}{365 \times 3600 \times 24} = 2.73 \text{ Cusecs.}$$

Add 33 % losses. = 0.90 Cusecs.
Total = 4.63. Cusecs.
Assume:-

B = 2.5 feet
d = 1.5 feet
A = Bx D 2.5 x 1.5 = 3.75 Cusecs.
P = 5.5 feet.
$R^{2/3}$ = $(A/P)^{2/3}$ = $(3.75/5.5)^{0.66}$ =. 0.775

n = 0.025
$S^{1/2}$ = $(1/2000)^{1/2}$ = 0.02236

V = $1.49 \times R^{2/3} \times S^{1/2}$ =
Q = AV = 3.75 x 1.032
Q = 3.87 Cusecs
Hence the Design is O.K.

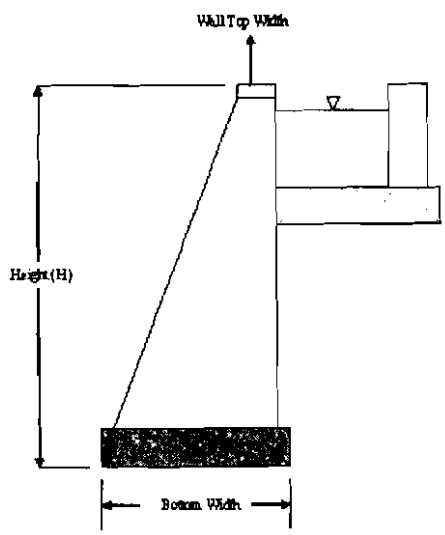


Figure 10.3 Design of Retaining Wall

H	= 7 feet.
Top width	= 1.5 feet.
Bottom Width.	= 4.75 feet.
Soil.	=120. P.C.F.
Stone	= 150 P.C.F
Φ	= 30°
W1	= 1575 Lbs.
W2	=1.181. 25 Lbs.
W	= W1 + W2 = 2756.25
X1	=3. feet.
X2	= 1.5 feet.

P $= \left(\frac{r H^2}{2} (1 - \sin \Phi)\right)/(1 + \sin \Phi)$

P $= \frac{120 \times 7 \times 7}{2} \left(\frac{1 - \sin 30^o}{1 \times \sin 30}\right) = 980$ Lbs.

Mo $= P (H/3) - 980 \times \frac{7}{3} = 2286.67$

Mr $= W1\ X1 + W2\ X2$

$= (1575 \times 3) + 1181.25 \times 1.5 = 6496.87$ Lbs/ft.

F.O.S against overturning.

$\frac{Mr.}{Mo} = \frac{6496.87}{2286.67} = 2.84$ which is greater than 2 hence safe.

F.O.S. AGAINST SLIDING

$\frac{u\,W}{P} = \frac{0.55 \times 2756.25}{980.} = 1.55$ which is greater than 1.5 hence O.K.

10.7 Design of Retaining Wall

H	=	6	Ft
Angle of internal	=	30	
Friction	=	0.55	
Top width	=	1.5	Ft
Bottom width	=	4.5	Ft
W	=	2700	LBS
P	=	$0.5*rH^2 (1-\sin\Phi) / (1+\sin\Phi) = 891$	
Over turning moment	=	1782	LBS Ft
Resisting moment	=	6412.5	LBS Ft
F.O.S against sliding	=	w/p*u	
	=	1.67 (greater than 1.5 hence O.K)	
F.O.S against overturning	=	Resisting moment/over turning	
	=	3.5 greater than 2 hence ok	

Design of R/Wall

H	=	8	Ft
Angle of internal	=	30	
Friction	=	0.55	
Top width	=	2	Ft
Bottom width	=	5.5	Ft
W	=	4500	LBS
P	=	$0.5*rH^2*(1-\sin\Phi)/(1+\sin\Phi) = 1584$	
Over turning moment	=	4224	LBS Ft
Resisting moment	=	13250	LBS Ft
F.O.S against sliding	=	w/p*u	
	=	1.56	
		greater than 1.5 hence O.K	
F.O.S against overturning	=	Resisting moment/over turning	
	=	3.13 (greater than 2 hence ok)	

Design of R/Wall

H	=	14	Ft
Angle of internal	=	30	
Friction	=	0.55	
Top width	=	2	Ft
Bottom width	=	9.75	Ft
W	=	12337.5	LBS
P	=	$0.5*rH^2*(1-\sin\Phi)/(1+\sin\Phi) = 4581$	
Over turning moment	=	22638	LBS Ft
Resisting moment	=	78793.75	LBS Ft
F.O.S against sliding	=	w/p*u	
	=	1.52	
		greater than 1.5 hence O.K	
F.O.S against overturning	=	Resisting moment/over turning	
	=	3.48 (greater than 2 hence ok)	

Design of R/Wall

H	=	6	Ft
Angle of internal	=	30	
Friction	=	0.55	
Top width	=	1.5	Ft
Bottom width	=	4.5	Ft
W	=	2700	LBS
P	=	$0.5*rH^2*(1-\sin\Phi)/(1+\sin\Phi) = 891$	
Over turning moment	=	1782	LBS Ft
Resisting moment	=	6412.5	LBS Ft

F.O.S against sliding	=	w/p*u
	=	1.67
		greater than 1.5 hence O.K
F.O.S against overturning	=	Resisting moment/over turning
	=	3.5 (greater than 2 hence ok)

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H	=	8	Ft
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W	=	4500	LBS
P	=	$0.5*rH^2*(1-\sin\Phi)/(1+\sin\Phi) = 1584$	
Over turning moment	=	4224	LBS Ft
Resisting moment	=	13250	LBS Ft
F.O.S against sliding	=	w/p*u	
	=	1.56	
		greater than 1.5 hence O.K	
F.O.S against overturning	=	Resisting moment/over turning	
	=	3.13 (greater than 2 hence ok)	

Design of R/Wall

H	=	14	Ft
Angle of internal	=	30	
Friction	=	0.55	
Top width	=	2	Ft
Bottom width	=	9.75	Ft
W	=	12337.5	LBS
P	=	$0.5*rH^2*(1-\sin\Phi)/(1+\sin\Phi) = 4581$	
Over turning moment	=	22638	LBS Ft
Resisting moment	=	78793.75	LBS Ft
F.O.S against sliding	=	uW/P	
	=	1.52	
		greater than 1.5 hence O.K	
F.O.S against overturning	=	Resisting moment/over turning	
	=	3.48 (greater than 2 hence ok)	

Design of R/Wall

H	=	12'
Top width	=	2'
Bottom width	=	6.76
Soil	=	
Stone	=	
ϕ	=	30°
W1	=	3600
W2	=	4275.50
W	=	W1 +W2 = 7875
M1	=	5.75
X2	=	3.17
P	=	$0.5*rH^2*(1-\text{Sin}\,\Phi)/(1+\text{Sin}\,\Phi)$

$$P = \frac{120\,(12 \times 12)}{2}\,\frac{(1-\text{Sin}\,30^o)}{1+\text{Sin}\,30^o}$$

$$P = P\,(H/3) = 2880 \times \frac{12}{3} = 11520 \text{ Lbs/in.}$$

$$Mr = W1\,X1 + W2\,X2 = 34251.25$$

F.O.S. against over turning

$$\frac{Mr}{Mo} = \frac{34251.25}{11520} = 2.970 \qquad 2 \text{ Hence O.K}$$

F.O.S. against sliding.

$$uW/P = 0.55\,\frac{(7875)}{2880} = 1.55$$

+ 1.55 @ Hence O.K

Design of R/Wall

H	= 6 ft.
Top width	= 1.5ft.
Bottom	= 3.ft
Soil	= 120. P.C.F.
Stone.	= 150 P.C.F.
ϕ	= 30°
W1	= 1350
W2	=675
W	= W1 X W2 = 2025

X1 = 2.25
X2 = 1
P = $0.5*rH^2*(1-\sin\Phi)/(1+\sin\Phi)$

$$= \frac{120}{2} \times 6 \times 6 \frac{(1-\sin 30)}{1+\sin 30} = 72$$

Mo = P (H/3) = 1440
Mr = W1 X1 + W2 X2 = 3712.5
F.O.S against over-turning

$$\frac{Mr}{Mr} = \frac{}{Mo} = 2.58 \quad 2 \text{ Hence safe.}$$

F.O.S. against sliding.

$$\frac{uW}{P} = 0.55 \frac{2025}{720} = 1.55$$

= 1.55 1.5 Hence safe.

Design of Retaining Wall

H = 7 feet.
Top width = 1.5 feet.
Bottom Width. = 4.75 feet.
Soil. =120. P.C.F.
Stone = 150 P.C.F
Φ = 30°
W1 = 1575 Lbs.
W2 =1.181. 25 Lbs.
W = W1 + W2 – = 2756.25
X1 =3. feet.
X2 = 1.5 feet.
P = $0.5*rH^2*(1-\sin\Phi)/(1+\sin\Phi)$

$$P = \frac{120}{2} \times 7 \times 7 \left(\frac{1-\sin 30^o}{1 \times \sin 30}\right) = 980 \text{ Lbs.}$$

$$M = P(H/3) - 980 \times \frac{7}{3} = 2286.67$$

Mr = W1 X 1 W2 x 2
= (1575 x 3) + 1181. 25x1.5 = 6496. 87 Lbs/ft.

F.O.S against overturning.

$\frac{Mr.}{Mo} = \frac{6496.87}{2286.67} = 2.84$ which is greater than 2 hence safe.

F.O.S. AGAINST SLIDING

$\frac{u\,W}{P} = \frac{0.55 \times 2756.25}{980.} = 1.55$ which is greater than 1.5 hence O.K.

Design of Super Passage.

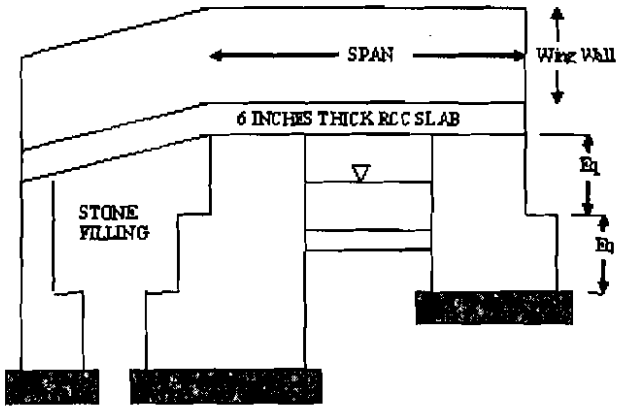


Figure 10.4. Example of design of a super passage

Assume 2.5 ft: x 3.5 Clear section and 6' thick RCC. Slab.

Fy = 40 K.S.I.
Fc" = 3 K S.I.
Clear width = 3.5 feet.
Effective width = 5.75 feet.

Loads.

a) D.L $= \frac{6}{12} \times 150$ = 75 P.S.F.

b) L.L. = = 200 P.S.F.

Impact. = 25 % of L.L = 50 P.S.F.

Total L.L = 250 P.S.F.

Factored load. = 1.4 (D.L) + 1.7 (L.L)

$= 1.4\ (75) + 1.7\ (25) = 530$ P.S.F.

Mu $= \frac{W1^2}{8} = 530 \times 5.75 \times 5.75$

$= 2190.39$ Lbs/ft.

Using # bars and ¾" Clear cover

D = 5"

Φ = 0.90

$\frac{M\mu}{\Phi\ bd^2} = \frac{2190.39 \times 12}{0.9 \times 12 \times 5 \times 5} = 97.35$ P.S.I.

As 97.35 < 200

Hence minimum steel is used.

i.e.

Ast = e min bd

$= 0.005 \times 12 \times 5 = 0.3\ in^2$

Design of Super Passage.

Assume 2.5 ft: x 3.5 ft clear section and 6" thick R.C.C. slab.

Fy	=	40 K.S.I.
Fc	=	3 K.S.I.
Clear width	=	3.5 Feet.
Effective width	=	5.75 feet
Loads.		
i) D.L.	=	$\frac{6}{12} \times 150 = 75$ P.S.F
ii. L.L.	=	200 P.S.F.
Impact.	=	25 25 % L.L. = 50 P.S.F.
Total L.L.	=	250 P.S.F.
Factored load	=	1.4 (D.L.) + 1.7 (L.L)
	=	1.4 (75) + 1.7 (25) = 530 P.S.F.

$M\mu = \frac{WL^2}{8} = \frac{530 \times 5.75 \times 5.75}{8}$

= 2190.39 Lbs/ft:

Using = 4 bars and ¾" clear cover

d = 5"

$\frac{M\mu}{\Phi\ bd^2} = \frac{2190.39 \times 12}{0.9 \times 12 \times 5 \times 5}$

= 97.35 P.S.I. as 97.35 < 200 Hence minimum steel is used.

i.e.

Abst = e min bd

= 0.005 x 12 x 5 = 0.3 in^2

Wo. of bars = $\frac{0.3}{0.196}$ = 1.53

Spacing = $\frac{12}{1.53}$ = 7.84" 7.5" C/C

Design of Super Passage.

Assume 2.5 ft. 3.5 ft Clear Section.
and 6" thick R.C.C. Slab.

Fy	=	40 K.S.I.
Fc	=	3 K.S.I.
Clear width	=	3.5ft.
Effective width	=	5.75 ft.
Loads		
i) D.L	=	$\frac{6 \times 150}{12}$ = 75 P.S.F.
ii) LL.	=	200 P.S.F.
Impact.	=	25 % L.L 50 P.S.F.
Total L.L.	=	250 P.S.F.
Factored load	=	1.4 (D.L) + 1.7 (L.L)
	=	1.4 (75) + 1.7 (L.L) (25)
	=	530 P.S.F.
Mμ	=	$\frac{WL^2}{8}$ = 530 x 5. 75 x 5.75
		= 2190.39 Lbs/ft.

Using # 4 bars and ¾" clear cover

d	=	5"
Φ	=	0.90
$\frac{M\mu}{\Phi bd^2}$	=	$\frac{2190.39 \times 12}{0.9 \times 12 \times 5 \times 5}$

= 97.35 P.S.F. as 97. 35 / 200 Hence minimum steel is used.

i.e.

As = min bd

= 0.005 x 12 x 5 = 0.3 in^2

No of bars = $\frac{0.3}{0.196}$ = 1.53

Spacing = $\frac{12}{1.53}$ = 7.84 7.56" C/C

D_n steel

ASD = 0.002 A = 0.002 x b x h

= 0.002 x 12 x 6 = 0.144 in^2

Using # 3 bars.

No. of bars = $\frac{0.144}{0.11}$ = 1.31

Spacing = $\frac{12}{1.31}$ = 9.16 9" C/C

Design of Super Passage.

Assume 2.5 ft. x 3.5 ft clear Section.

and 6" thick R.C.C. Slab.

Fy	=	40 K.S.I.
Fc	=	3 K.S.I.
Clear width	=	3.5 ft.
Effective width	=	5.75 ft.

Loads

i) D.L = $\frac{6}{12}$ x 150 = 75 P.S.F.

ii) LL. = 200 P.S.F.

Impact. = 25 L.L. = 50 P.S.F.

Total L.L. = 250 P.S.F.

Factored load = 1.4 (D.L) + 1.7 (L.L)

= 1.4 (75) + 1.7 (L.L) (25)

= 530 P.S.F.

$M\mu$ = $\frac{WL^2}{8}$ = 530 x 5. 75 x 5.75

= 2190.39 Lbs/ft.

Using # 4 bars and ¾" clear cover

d = 5"

Φ = 0.90

$\frac{M\mu}{\Phi bd^2}$ = $\frac{2190.39 \times 12}{0.9 \times 12 \times 5 \times 5}$

= 97.35 P.S.I. as 97. 35/ 200 Hence minimum steel

i.e.

As = min b d

= 0.005 x 12 x 5 = 0.3 in^2

No of bars = $\frac{0.3}{0.196}$ = 1.53

Spacing = $\frac{12}{1.53}$ = 7.84 7.5' C/C

$D_n$steel

ASD = 0.002 Ag = 0.002 x b x h

=0.002 x 12 x 6 = 0. 1.444 in^2

Using # 3 bars.

No. of bars = $\frac{0.144}{0.11}$ = 1.31

Spacing = $\frac{12}{1.31}$ = 9.16 9" C/C.

Design of Super Passage.

Assume 2.5 ft: x 3.5 Clear section and 6' thick RCC. Slab.

Fy = 40 K.S.I.

Fc" = 3 K S.I.

Clear width = 3.5 feet.

Effective width = 5.75 feet.

Loads.

a) D.L = $\frac{6}{12}$ x 150 = 75 P.S.F.

b) L.L. = 200 P.S.F.

Impact. = 25 % of L.L = 50 P.S.F.

Total L.L = 250 P.S.F.

Factored load. = 1.4 (D.L) + 1.7 (L.L)

= 1.4 (75) + 1.7 (25) = 530 P.S.F.

$M\mu = \frac{WL^2}{8} = \frac{530 \times 5.75 \times 5.75}{8}$

= 2190. 39 Lbs/ft.

Using # bars and ¾" Clear cover

D = 5"

Φ = 0.90

$\frac{Mu}{\Phi bd^2} = \frac{2190.39 \times 12}{0.9 \times 12 \times 5 \times 5}$ = 97.35 P.S.I.

As 97.35 < 200 Hence minimum steel is used.

Ast = e min bd

= 0.005x12x5 = 0.3 in^2

Design of Super Passage

Assume 2.5 ft: x 3.5 ft: Clear Section,
And 6" thick R.C.C.Slab.

FY	=	40	k.S.I.
FC	=	3	k.S.I.
Clear width	=	3.5	Feet.
Effective Width	=	5.75	Feet.

Loads.

1). D.L. = $\frac{6}{12}$X150 =75 P.S.F.

II) L.L. = 200 P.S.F.

Impact = 25% L.L = 50 P.S.F.

TOTAL L.L = 250 P.S.F.

Factored load = 1.4 (D.L) + 1.7 (L.L)

= 1.4 (75) + 1.7 (25) = 530 P.C.F.

$M\mu = \frac{WL^2}{8} = \frac{530X5.75X5.75}{8}$ =2190.39 Lbs/ ft:

Using = 4 bars and ¾" clear cover

D = 5"

Φ = 0.90

$\frac{M\mu}{\Phi bd^2} = \frac{2190.39x12}{0.9x12x5x5}$

= 97.35 P.S.I as 97.35 <200 Hence minimum steel is used.

i.e.

Ast = e min bxd

= 0.005 x12x5 = 0.3 in^2

No. of bars = $\frac{0.3}{0.196}$ = 1.53

Spacing = $\frac{12}{1.53}$ = 7.84 7.5" C/C

$\frac{D_n \text{ Steel}}{ASD}$ = 0.002 Ag = 0.002xbxn

= 0.002X12X6 = 0.144 in^2

Using = 3 bars.

No of bars = $\frac{0.144}{0.11}$ = 1.31

Spacing = $\frac{12}{1.31}$ = 9.16 9" C/C

10.8 Design of Aqueduct.

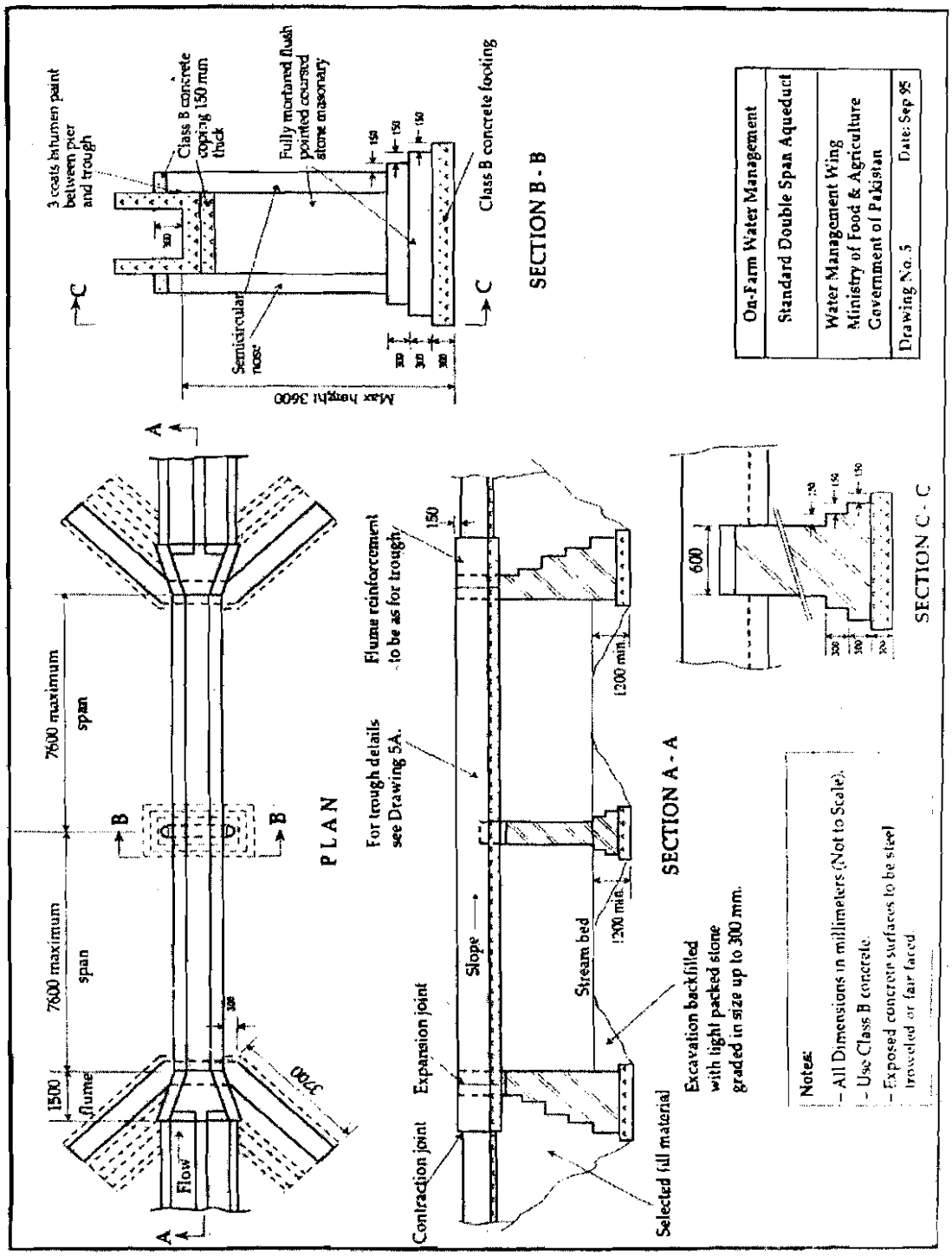


Figure 10.5. design of aquaduct.

Slab Design.

Fy = 40,000 F.S.I.

Fc = 3,000 P.S.I.

Clear span = 2.5 Feet.

Effective span = $2.5' + 2\ (\frac{75}{100} x\ 0.75) = 3.63$ ft;

Loads.

a) Slab = 0.5 x 1.50 = 75 P.S.I.

b) Water = 2 x 62.4 = 124.8 P.S.I

c) Silt = 0.5 x 110 = 55.0 P.S.I

Total dead load: = 254.8 P.S.I

Live load. = 150.0 P.S.I

Impact. = 1.4 *254.8) r 1.7 (287.5) = 675.47 P.S.I

Mμ = Mμ L^2 (Fixed end moment)

= 675.47 x $(3.63)^2$ = 741.72 Lbs/ft:

Using = 4 bars and ¾" clear cover.

d = 6 - ¾ -4 (4) = 5 inches.

Φ = 0.90

$\frac{M\mu}{\Phi\ bd^2}$ = $\frac{M\mu}{}$ = 741.72 x 12

Ru = 164.83 P.S.I

e = $\frac{1}{m}$ (1 - / 1-2 $\frac{m\ Ru}{fy}$)

e = $\frac{1}{15.68}$ (1 - / $\frac{1\text{-}2 \times 15.68 \times 164.83}{40{,}000}$)

e = $\frac{1}{15.68}$ (1 – 0.9332)

e = 0.00426 which is less than e min.

Therefore using e min = 0.005

Ast = e b d

Ast = 0.005 x 12 x 5 = 0.3 in^3

Use = A bars.

No. of bars. = $\frac{0.3}{0.196}$ = 1.53 bars.

Per ft: width.

Spacing = $\frac{12}{1.53}$ = 7.5 C/C

Distribution steel = 0.002 bd = 0.002 x 12 x 6 = 0.144 in^2

Using = 3 bars.

No of bars / ft. = $\frac{0.144}{0.11}$ = 1.31 bars

Spacing = $\frac{12}{1.31}$ = 9.16" C/C, Say 9" C/C

Design of Beam.

Clear span = 30 feet.

Effective span = $30 + 2\ (\frac{75}{100} \times 1.5 \times 2) = 32.25$ feet.

a) Dead Load

i) Slab = 1.25 x 0.5 x 150 = 93.75 Lbs/ft:.

ii) Silt Load = 1.25 x 6.5 x 110 = 68.75 Lbs/ft:

iii. Assume beam load = 2 x 1 x 150 = 300 Lbs/ft.

iv Total dead load = 462.5 Lbs/ft:

b) Live load.

i) Live load = 100 Lbs/ft:

ii) Live load = 1.5 x 1.5 x 62.4 = 1.40.4 Lbs/ft:

iii) Impact Load = 25% of live load = 60 Lbs/ft:

Total live load = 300 Lbs/ft:

Factor load. = Wu = 1.7 x 300 + 1.4 x 462.5 = 1203 Lbs/ft:

$M\mu = \frac{1203 \times (32.25)^2}{8} = 156399.39$

$M\mu = \frac{M\mu}{\Phi} = \frac{M\mu}{0.9} = 173777$ Lbs/ft:

Use = b = 10"

D = 24"

$\frac{M\mu}{\Phi\ bd^2} = Ru = \frac{173777 \times 12}{12 \times (24)\ (24)} = 302$

$e = \frac{1}{m}\ (1 - \sqrt{1 - 2\ \frac{m\ Ru}{fy}}$

$e = \frac{1}{15.68}\ (1 - \sqrt{1 - 2m \times \frac{302}{40000}}$

Ast = e x b x d

= 0.00081 x 12 x 24 = 2.3328 in^2

Use = A bars.

Use = Use 4 Bars.

No. of Bars. = $\frac{2.3328}{0.196}$ = 11.9 bars. Say 12 bars

$\frac{D_n Steel}{ASD}$ = 0.002 Ag = 0.002 x b x h

= 0.002 x 12 x 6 = 0.144 in^2

Using = 3 bars

No. of bars. = $\frac{0.144}{0.11}$ = 1.31

Spacing. = $\frac{12}{131}$ = 9.16 (9" C/C)

Design of Aqueduct. (Slab Design)

Fy = 40,000 F.S.I.

Fc = 3,000 P.S.I.

Clear span = 2.5 Feet.

Effective span = $2.5' + 2\ (\frac{75}{100} \times 0.75) = 3.63$ ft;

Loads.

a) Slab = 0.5 x I.50 = 75 P.S.I.

b) Water = 2 x 62.4 = I24.8 P.S.I

c) Silt = 0.5 x 110 = 55.0 P.S.I

Total dead load: = 254.8 P.S.I

Live load. = 150.0 P.S.I

Impact. = 1.4 *254.8) r 1.7 (287.5) = 675.47 P.S.I

$M\mu$ = $\frac{M\mu\ L^2}{12}$ (Fixed end moment)

= $675.47 \times (3.63)^2$ = 741.72 Lbs/ft:

Using = 4 bars and ¾" clear cover.

d = 6 - ¾ -4 (4) = 5 inches.

Φ = 0.90

$\frac{M\mu}{\Phi\ bd^2}$ = $M\mu$ = 741.72 x 12

Ru = 164.83 P.S.I

e = $\frac{1}{m}\ (1 - / 1\text{-}2\ \frac{m\ Ru}{fy})$

e = $\frac{1}{15.68}\ (1 - / \frac{1\text{-}2 \times 15.68 \times 164.83}{40{,}000})$

e = $\frac{1}{15.68}\ (1 - 0.9332)$

e = 0.00426 which is less than e min.

Therefore using e min = 0.005

Ast = e b d

Ast = 0.005 x 12 x 5 = 0.3 in^3

Use = A bars.

No. of bars. = $\frac{0.3}{0.196}$ = 1.53 bars.

Per ft: width.

Spacing = $\frac{12}{1.53}$ = 7.5 C/C

Distribution steel = 0.002 bd = 0.002 x 12 x 6 = 0.144 in^2

Using = 3 bars.

Nor or bars / ft. = $\frac{0.144}{0.11}$ = 1.31 bars

Spacing = $\frac{12}{1.31}$ = 9.16" C/C, Say 9" C/C

Design of Beam.

Clear span = 30 feet.

Effective span = $30 + 2\ (\frac{75}{100} \times 1.5 \times 2)$ = 32.25 feet.

a) Dead Load

i) Slab = 1.25 x 0.5 x 150 = 93.75 Lbs/ft:.

ii) Silt Load = 1.25 x 6.5 x 110 = 68.75 Lbs/ft:

iii. Assume beam load = 2 x 1 x 150 = 300 Lbs/ft.

iv Total dead load = 462.5 Lbs/ft:

b) Live load.

i) Live load = 100 Lbs/ft:

ii) Live load = 1.5 x 1.5 x 62.4 = 1.40.4 Lbs/ft:

iii) Impact Load = 25% of live load = 60 Lbs/ft:

Total live load = 300 Lbs/ft:

Factor load. = Wu = 1.7 x 300 + 1.4 x 462.5 = 1203 Lbs/ft:

Mu = $\frac{1203 \times (32.25)^2}{8}$ = 156399.39

Mn = $\frac{Mu}{0.9}$ = 173777 Lbs/ft:

Use = b = 10"

D = 24"

$\frac{Mn}{Bd}^2$ = Ru = 173777 x ° 12 = 302

12x(24) (24)

e = $\frac{1}{m}$ (1 - / 1-2 $\frac{m\,Ru}{fy}$

e = $\frac{1}{15.68}$ (1 - / 1-2m x $\frac{302}{40000}$

Ast = e x b x d

= 0.00081 x 12 x 24 = 2.3328 in^2

Use = A bars.

Use = Use 4 Bars.

No. of Bars. = $\frac{2.3328}{0.196}$ = 11.9 bars. Say 12 bars

$D_n$steel

ASD = 0.002 Ag = 0.002 x b x h

= 0.002 x 12 x 6 = 0.144 in^2

Using = 3 bars

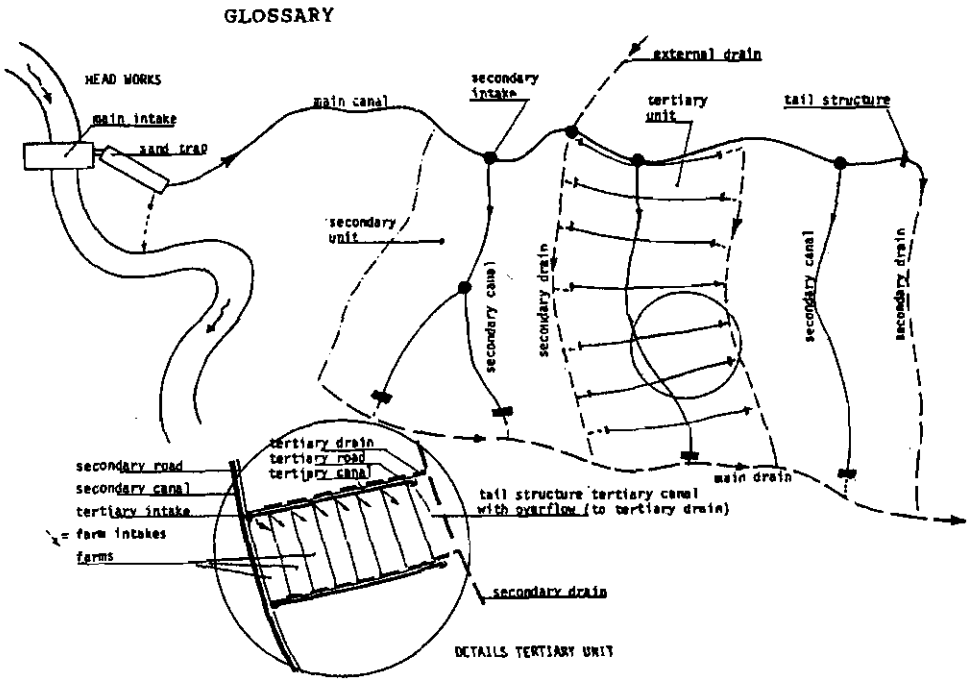
No. of bars. = $\frac{0.144}{0.11}$ = 1.31

Spacing. = 12/1.31 = 9.16 (9" C/C)

11 WATER DISTRIBUTION METHODS

11.1 Components of an irrigation system

Before we discuss water distribution in detail let us first look at a schematic diagram of a typical irrigation system and a definition of irrigation. An irrigation system normally looks as follows:



The names given to canals and structures differ from one region to another.

Main canals are also termed:	feeder/conveyance/trunk canals,
secondary canals:	branch/lateral/distributary canals,
tertiary canals:	supply/delivery/farm/terminal/sector canals or water courses,
tertiary unit:	farm block/users' unit/chak/sector,
intake structure:	turn-out/head structure.

Often irrigation schemes are divided into ***major*** (or ***main***) ***systems*** and ***minor systems***.

Major systems include:	main and secondary irrigation canals + structures, including tertiary intakes, main and secondary roads (not shown) + structures, main and secondary drains + structures.
Minor systems include:	tertiary irrigation canals, roads and drains + structures (not the tertiary intakes).

Normally an Irrigation Authority is responsible for the operation and maintenance (O+M) of the main system while the farmers in a tertiary unit are responsible for the operation and maintenance in their unit.

Figure 11.1. Schematic layout of an irrigation system and the different terms used.

Definition of Irrigation
Irrigation can be defined as the provision of measures that enables users to adequately supply their crops with water which is collected elsewhere.

This definition highlights three important features in irrigation:

- Water is collected conveyed and distributed;
- The flow should meet the irrigation requirements of the crops;
- The farmer should be able to handle the flow.

In practice, the first two items are often emphasized in the design of irrigation systems, but the third tends to be ignored. Not surprisingly, therefore, problems of water distribution are frequently reported in projects where many small holders have to cooperate in what is termed as tertiary units, farm blocks, and minor units.

11.2 Water Distribution Methods

The main distribution methods are:
Continuous flow, also termed as proportional flow. Water flows continuously in the entire canal system and is proportionally distributed to the irrigated farm areas. The flow is adjusted to match the changing crop requirements during the irrigation season.

Fixed flow. Similar to continuous flow but the flow size is kept constant throughout the season.

Rotational or on-off systems. Farm fields receive water in turn at a preset time and at a preset rate. Rotation is also applied in canal systems. In turn they convey either full flow or no flow at all.

On demand. Essentially the farmer is free to use water as he sees fit. Consequently there is a continuous but fluctuating flow in all canals. To limit these fluctuations the free of the farmer is restricted in one way or another.

In the *delivery on request* or *semi demand* systems, a verbal or written request for water is made a few days, sometimes a week, in advance. Then the irrigation authority draws a water distribution program for the next period. To obtain a more or less constant flow in the canal system, the applications are not always met exactly on the day requested for.

In the *free delivery system* the farmer may take water without prior notice. Usually the flow is metered and paid for by volume. The method requires large capacity canals and complicated structures for water regulation. The domestic water supply systems in pipes and taps are one such example of on demand water supply systems.
Upstream and downstream control

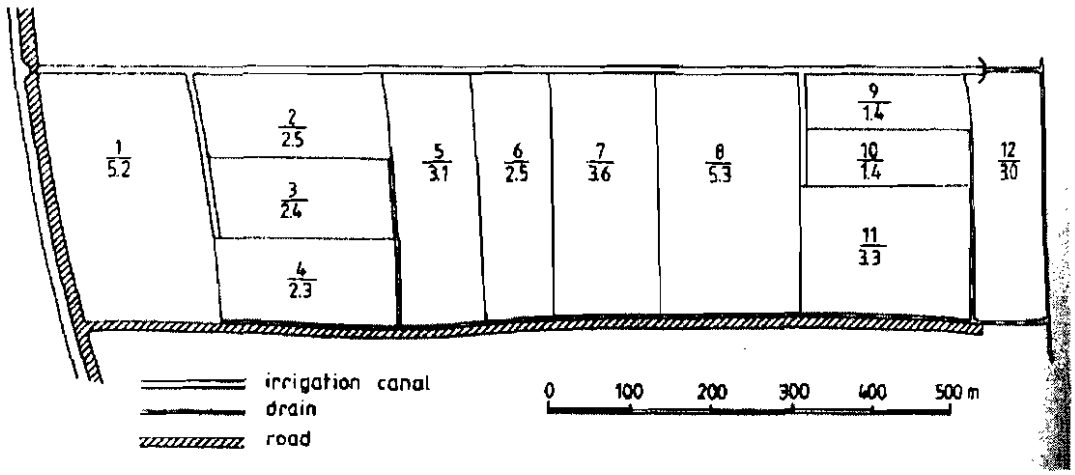
In methods such as continuous flow, rotational systems and semi demand, the irrigation authority eventually decides on the flows to be given. In these systems there usually is an upstream control of the flow: at the flow regulating structures the upstream water level is set and controlled and so are the flows through the structures. In a free delivery system, where the setting of the farm gates controls the flow in the entire scheme, there is downstream control. Note that downstream control can also be used in combination with distribution methods other than free delivery. Figure 2 shows the type of deliveries to the tertiary off takes in an irrigation system in response to the type of water allocation.

Water distribution within a tertiary unit

Except for the free delivery systems, the users within a tertiary unit generally irrigate their farms/fields in turns in a rotation. Some examples of rotation are the weekly warabandi inn irrigation systems in Pakistan and the family based rotation schedules in the traditional irrigation systems in Afghanistan and Pakistan.

In order to understand water distribution in a tertiary unit we use an example in which the rotation schedule will be calculated. We will make use of the following data:

Available Data: Maize is grown on 3 ha fields; night irrigation is practiced; Irrigation interval is 12 days and farmers irrigate one-by-one; Gross irrigation requirement is 8.5 mm/day.



Tertiary unit with farms of different sizes.

Solution: For each irrigation the farmer requires:

$12 \times 8.5 \times 10^{-3} \times 3 \times 10^{4} = 3060\ m^3$

If each farmer irrigates during 24 hours the best flow rate is

$3060/24 = 128\ m^3/hour = 35.4\ l/s$.

The irrigated area of the tertiary unit is $3 \times 122 = 36$ ha.

Table 11.1. Irrigation times for the tertiary unit farms.

		Theor. Flow 35.4 l/s		Increased flows 39 l/s				Increased flows 42.5 l/s			
Farm no.	Area (ha)	t (h) (3)	t (d) (4)	T1 (5)	T2 (6)	t (h) (7)	t (d) (8)	T1 (9)	T2 (10)	t (h) (11)	t (d) (12)
1	5.2	41.6	1.73	37.8	4.2	42	1.75	34.7	1.3	36	1.5
2	2.5	20	0.83	18.0	0	18	0.75	16.7	1.3	18	0.75
3	2.4	19.2	0.80	17.4	0.6	18	0.75	16.0	2.0	18	0.75
4	2.3	18.4	0.77	16.7	1.3	18	0.75	15.3	2.7	18	0.75
5	3.1	24.8	1.03	22.5	1.5	24	1	20.7	3.3	24	1
6	2.5	20	0.83	18.0	0	18	0.75	16.7	1.3	18	0.75
7	3.6	28.8	1.20	26.1	3.9	30	1.25	24.0	0	24	1
8	5.3	42.4	1.77	38.5	3.5	42	1.75	35.3	0.7	36	1.5
9	1.4	11.2	0.47	10.2	1.8	12	0.5	9.3	2.7	12	0.5
10	1.4	11.2	0.47	10.2	1.8	12	0.5	9.3	2.7	12	0.5
11	3.3	26.4	1.10	24.0	0	24	1	22.0	2.0	24	1
12	3.0	24	1.00	21.8	2.2	24	1	20.0	4.0	24	1
		288.0	12.00			282	11.75			264	11.00

11.3 Water Division Structures

In this section the most common types of water division structures will be discussed in terms of their general hydraulic characteristics.

Types of bifurcations

Division: bifurcating canals with capacities of the same order of magnitude and the same function in the system.

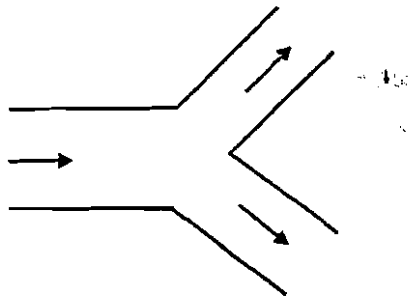


Figure 11.2. Flow division

Off takes: Smaller-lower order canals branching off from larger ones. Here the function of the off taking canal differs from the ongoing one.

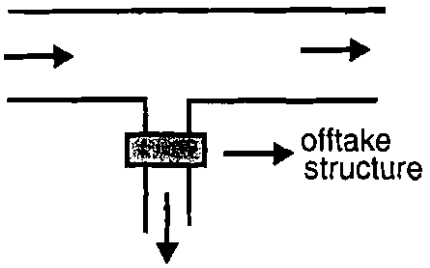


Figure 11.3 .Canal offtake

At the point of bifurcation the division of water IS realized by hydraulic structures. In case of a free off take Figure 11.4. the flow through the off take structure will change with different water levels in the parent canal. The rate of change will depend on the shape of the off taking structure.

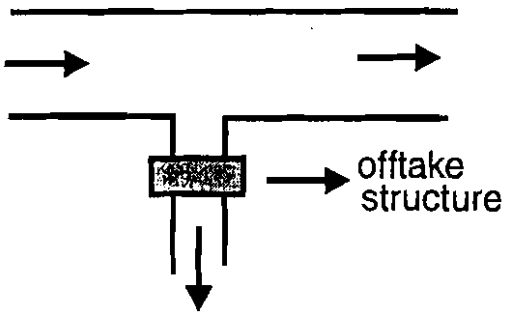


Figure 11.4. Free off take.

A free off take is usually only acceptable when the upstream water level remains constant. With changing flows and subsequently changing water levels in the parent canal, there often a need for water level control in order to create sufficient head for the off take structure and to avoid frequent resetting of the off take gate. This control can be achieved by a check structure or a cross regulator in the patent canal. Figure 11.5.

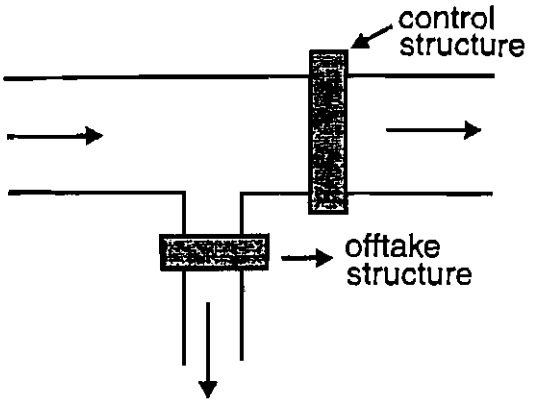


Figure 11.5. Controlled off take.

11.4 Types of Structures

A hydraulic structure at a bifurcation point in an irrigation system can be used for one or more of the following purposes:

- Flow regulation
- Controlling upstream water levels;
- Controlling downstream water levels;
- Measuring flows.

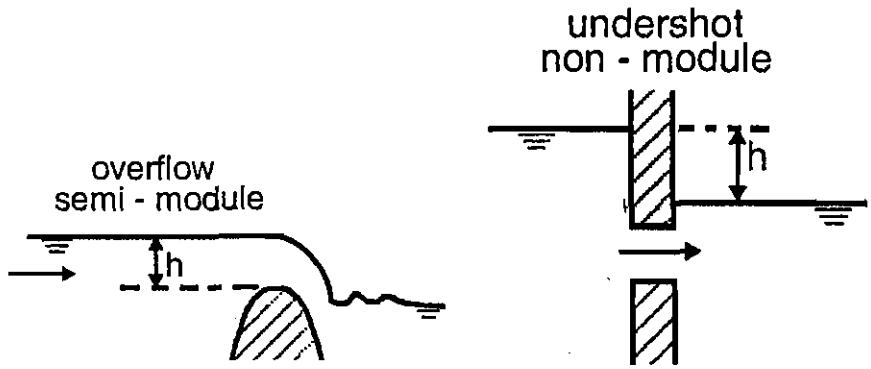
As\will be discussed in the following the suitability to meet these purposes depends on the hydraulic properties of these structures. A structure can further be classified by different boundary conditions in terms of upstream and downstream water levels:

Modular: Changes in either upstream or the downstream water levels do not affect the flow rate. In theory this situation can not exist. In practice, however, structures have been developed approximating a modular flow within a limited range of upstream water levels: step wise distributors such as the Neyrpic modules.
Semi-modular: the flow is affected only by the upstream water level.
Non-modular: the flow is affected by both the upstream and downstream water levels.

To determine the discharge in the first case no, in the second case one (upstream) and in the third case two (up-and downstream) water level gauge readings are needed. In general, preference should be given to modular or semi-modular structures. Moreover downstream users can manipulate the flow in case of non-modular structures.

Hydraulic structures might also be divided into:

Overflow structures
Undershot structures.



Overflow structures
These are normally used under semi-modular conditions. The general formula is:

$Q = c.h^{1.5}$

In which:

Q= Flow
c= constant
h= head

In figure 11.6 and 11.7 the most common types of overflow structures are presented with their modular limits (submergence ratio not to be exceeded to obtain modular flow, sediment passing capacities and application.

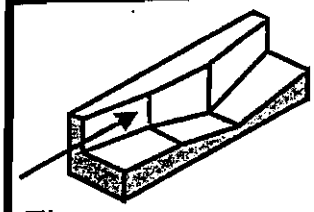
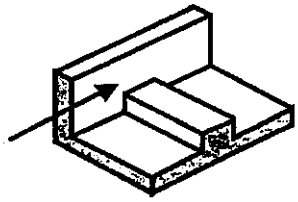
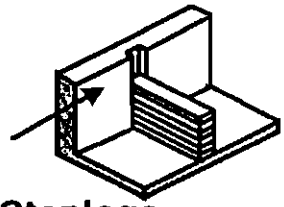
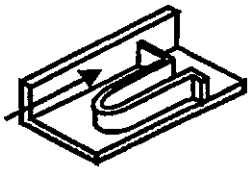
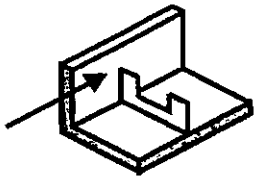
Type of structure	Modular limit	Sediment passing capacity	Application
 Flume	0.5 - 0.8	good	measurements
 Broad Crested Weir	0.4 - 0.95	fair	water level control + measurements
 Stoplogs	0.4 - 0.95	poor	water level control
 Duck Bill Weir	0.4 - 0.95	poor	water level control
 Sharp Crested Weir	headloss h + 0.05 m	very poor	measurements

Figure 11.6. Fixed overflow structures.

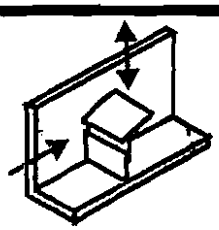
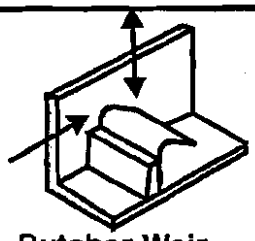
Type of structure	Modular limit	Sediment passing capacity	Application
 Romijn Weir	0.3	depending on movable or fixed undershot gate	flow regulation +measurements
 Butcher Weir	0.7	- do -	- do -

Figure 11.7 Adjustable overflow structures (Movable weirs)

The weir type structures, compared with flumes, have often structural advantages (more compact, simpler construction, cheaper). In case of silt carrying water, however, flumes have the advantage of higher silt carrying capacities. Rectangular weirs could be broad, short or sharp crested. The advantage of broad crested weirs and flumes is their small head loss.

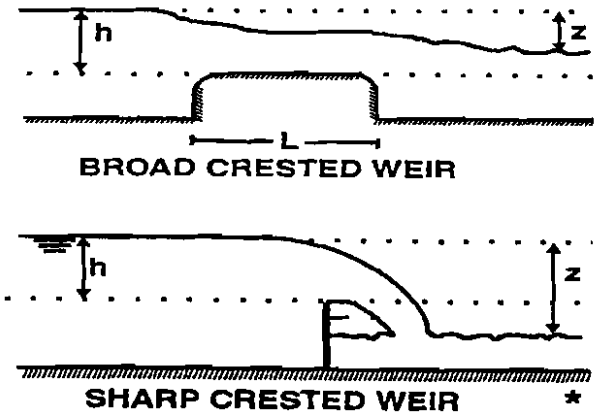


Figure 11.8. Broad and sharp crested weirs.

Undershot structures

These types of structures are generally used under semi-module as well as non-modular conditions. The orifice can be circular or rectangular ands fixed as well as adjustable (gated). The general formula is:

$$Q = c.h^{0.5}$$

In which:

Q= Flow
c= constant
h= head

In figure 11.9, A, B and C the most common undershot structures as used in irrigation are presented.

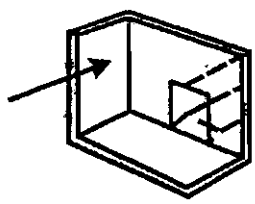
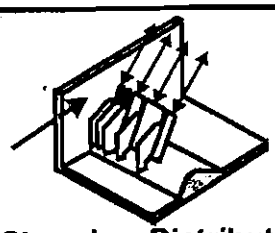
Type of structure	Modular limit	Sediment passing capacity	Application
 Fixed Outlet	variable	good	modular flow within limited range of upstream head - eg . see Mahbub et al. 1951 -
 Stepwise Distributor	0.6	fair	flow regulation +measurements

Figure 11.9A. Fixed undershot structures.

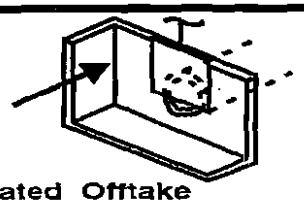
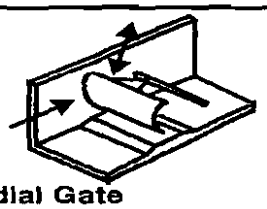
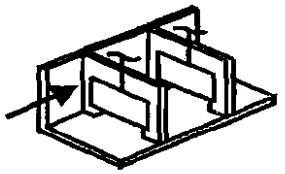
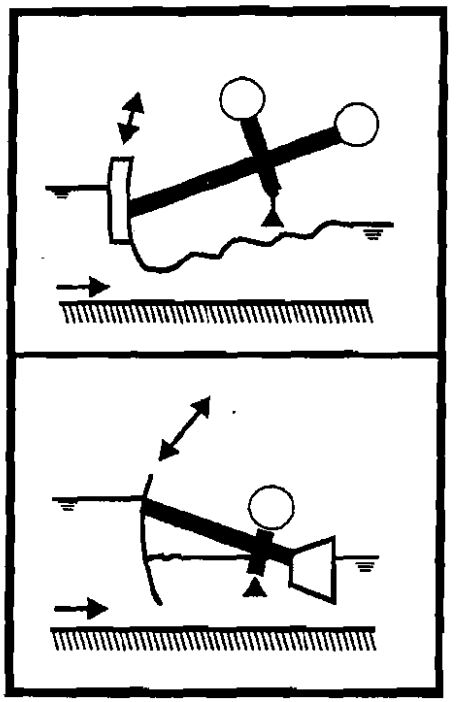
Type of structure	Modular limit	Sediment passing capacity	Application
 Gated Offtake	variable	poor	flow regulation
 Radial Gate	variable	very good	flow regulation or waterlevel control
 Constant Head Orifice	very good	poor	flow regulation +measurements

Figure 11.9B. Adjustable undershot structures.



Automatic upstream water level control

Automatic downstream water level control

Figure 11.9C. Automatic undershot structures.

11.5 Sensitivity and Hydraulic Flexibility

Two important hydraulic concepts explain the operational implications of selecting a certain type of stricture: the Sensitivity S and Hydraulic Flexibility F.

Sensitivity S
The discharge through a structure is directly related to the upstream head in case of semi-modular flow conditions and with the head loss in case of non-modular conditions. This can generally be expressed as:

$$Q = c.h^u$$

The sensitivity S of a structure depends on the power u and the head h. it is commonly expressed as the fractional change of discharge caused by the unit rise of the upstream head:

$$S = \frac{u}{h}\Delta h.$$

Summarizing the most common values for u are:

Overflow structures	u = 1.5
Undershot structures	u = 0.5
Canals	u = 1.6-1.8.

Sensitivity requirements depend on the purpose of the structures:
To minimize the upstream head fluctuations, the sensitivity S should be high. In other words the structure should have the highest possible factor u/h:
u large: weir or flume (u = 1.5)
h small: weir with long crest.

To minimize the fluctuations of discharge through the structure, caused by varying upstream water levels. In this case the factor u/h should be as small as possible (undershot type: u small = 0.5 and h as large as possible, entrance as narrow as possible).

To measure discharges.
Here also the sensitivity should be small (small variation in Q should result in relatively large variation in h to enable accurate reading).

Hydraulic Flexibility F

The flow at a bifurcation will be divided by a certain ratio. Changes in oncoming flows will result in changes in water level at the bifurcation. The relative change in distribution will depend on the hydraulic properties of the structures. This can be defined by the hydraulic flexibility F. The hydraulic flexibility is an important tool to visualize the generation of flow changes through a system. It is expressed as the ratio between relative change of the off taking flow and the relative change of the on going flow (or the ratio between sensitivities of the off taking and the ongoing structures (S_o/S_s).

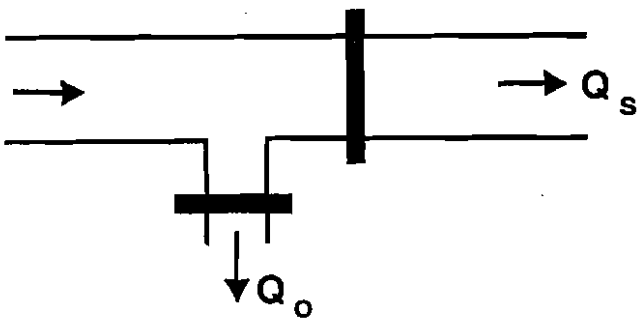


Figure 11.10. Bifurcation

The hydraulic flexibility could be expressed as:

$$F = S_o/S_s = u_o\, h_s/u_s\, h_o$$

11.6 Operational Characteristics

From an operational point of view five types of structures can be distinguished. In sequence of sophistication:

Fixed (fixed weirs and orifices)

The flow passing through depends on the shape of the opening and water levels upstream (and downstream when non-modular) of the structure. These structures are often used in systems with proportional fixed distribution of water. No adjustments are possible.

On-Off (shutter gates)

The structure is normally equipped which can either be in open or closed position.

Stepwise Adjustable (stoplogs and stepwise distributors)

The flow is regulated in steps.

Gradually Adjustable (gated undershot structures and movable weirs)

The flow can be regulated by changing the opening either by hand or mechanically

Automatic (automatic upstream and downstream water level control structures)
Most of these structures react actuated by floats, on changing water levels.

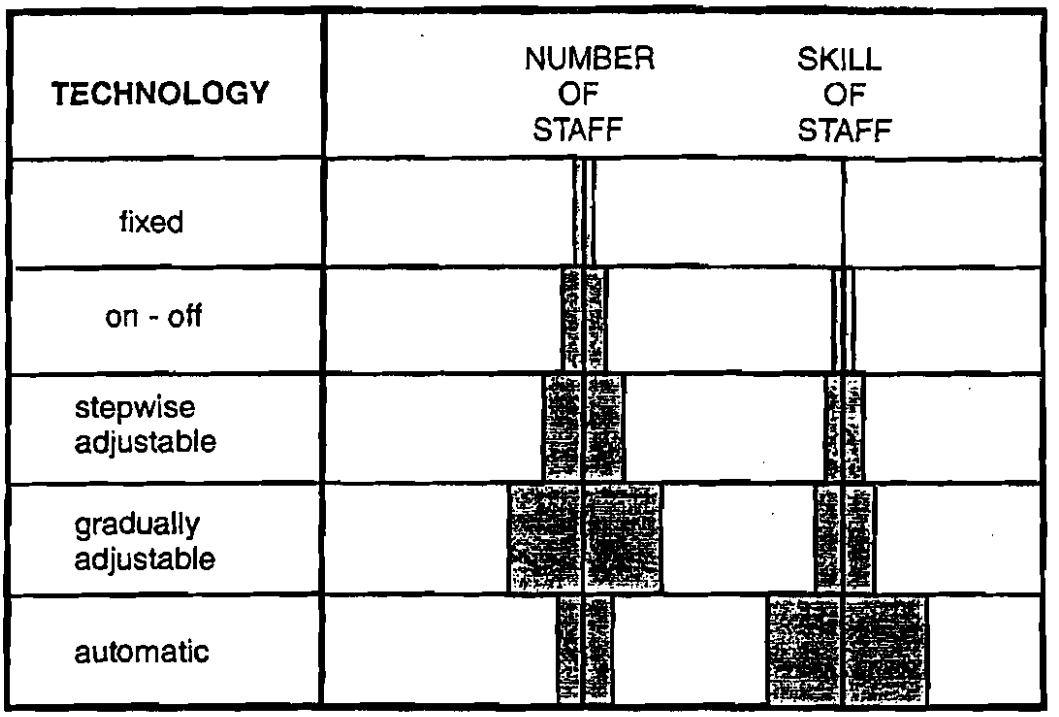


Figure 11.11. Operational characteristics.

12 SURFACE IRRIGATION METHODS

An adequate water supply is important for plant growth. When rainfall is not sufficient, the plants must receive additional water from irrigation. Various methods can be used to supply irrigation water to the plants. Each method has its advantages and disadvantages. These should be taken into account when choosing the method which is best suited to the local circumstances.

12.1 Basin Irrigation

Basins are flat areas of land, surrounded by low bunds. In general, the basin method is suitable for crops that are unaffected by standing in water for long periods (12-24 hours). Large application gifts (50 120 mm) are normally given and crops will be irrigated over long interval period (10-20 days).

Suitable Crops

Crops that are suited to basin irrigation include:

- Pastures, e.g. alfalfa, clover;
- Trees, e.g. citrus, banana;
- Crops which are broadcast, such as cereals;
- To some extent row crops such as tobacco.

Basin irrigation is generally not suited to crops which cannot stand in wet or waterlogged conditions for periods longer than 24 hours. These are usually root and tuber crops such as potatoes, cassava, beet and carrots which require loose, well-drained soils.

Suitable Land Slopes:

The flatter the land surface, the easier it is to construct basins. On flat land only minor leveling may be required to obtain level basins. It is also possible to construct basins on sloping land, even when the slope is quite steep. Level basins can be constructed like the steps of a staircase and these are called terraces.

Suitable Soils

Which soils are suitable for basin irrigation depends on the crop grown. Although most other crops can be grown on clays, loamy soils are preferred for basin irrigation so that water logging (permanent saturation of the soil) can be avoided. Coarse sands are not recommended for basin irrigation as, due to the high infiltration rate, percolation losses can be high. Also soils which form a hard crust when dry (capping) are not suitable.

Basin Layout

Basin layout not only refers to the shape and size of basins but also to the shape and size of the bunds. What is the shape of the basin: square, rectangular or irregular? What is the

size of the basin: 10, 100, 1000 or 10000 m^2? How high should the bund be: 10, 50 or 100 cm? What is the shape of the bund? These aspects are discussed in the following sections.

Shape and Size of Basins

The shape and size of basins are mainly determined by the land slope, the soil type, the available stream size (the water flow to the basin), the required depth of the irrigation application and farming practices.

Basin Width

The main limitation on the width of a basin is the land slope. If the land slope is steep, the basin should be narrow; otherwise too much earth movement will be needed to obtain level basins. Table 12.1 provides some guidance on the maximum width of basins or terraces, depending on the land slope.

Three other factors which may affect basin width are:

- depth of fertile soil,
- method of basin construction,
- agricultural practices.

If the topsoil is shallow, there is a danger of exposing the infertile subsoil when the terraces are excavated. This can be avoided by reducing the width of basins and thus limiting the depth of excavation.

Table 12.1 Approximate values for the maximum basin or terrace Width (m)

Slope %	Maximum width (m)	
	average	Range
0.2	45	35-55
0.3	37	30-45
0.4	32	25-40
0.5	28	20-35
0.6	25	20-30
0.8	22	15-30
1.0	20	15-25
1.2	17	10-20
1.5	13	10-20
2.0	10	5-15
3.0	7	5-10

Basins can be quite narrow if they are constructed by hand labour but will need to be wider if machines are used so that the machines can easily be moved around.

If hand or animal powered tillage is used then basins can be much narrower than if machines are used for cultivation. If machines are used then it is important to make sure that basin widths are some multiple of the width of the machines for efficient mechanization.

Basin Size

The size of basins depends not only on the slope but also on the soil type and the available water flow to the basins. The relationship between soil type, stream size and size of the basin is given in Table 12.2. Values are based on practical experience, and have been adjusted in particular to suit small-scale irrigation conditions.

Table 12.2 Suggested maximum basin areas (m2) for various soil types and available stream sizes (l/sec)

Stream size (l/sec)	Sand	Sandy loam	Clay loam	Clay
5	35	100	200	350
10	65	200	400	650
15	100	300	600	1000
30	200	600	1200	2000
60	400	1200	2400	4000
90	600	1800	3600	6000

In summary

Basins should be small if the:

1. slope of the land is steep
2. soil is sandy
3. stream size to the basin is small
4. required depth of the irrigation application is small
5. field preparation is done by hand or animal traction.

Basins can be large if the:

1. slope of the land is gentle or flat
2. soil is clay
3. stream size to the basin is large
4. required depth of the irrigation application is large
5. field preparation is mechanized.

Irrigating Basins
There are two methods to supply irrigation water to basins: the direct method and the cascade method.

The Direct Method
Irrigation water is led directly from the field channel into the basin through siphons, spiles or bund breaks. Figure 12.1 shows that "Basin a" is irrigated first, then "Basin b" is irrigated and so on. This method can be used for most crop types and is suitable for most soils.

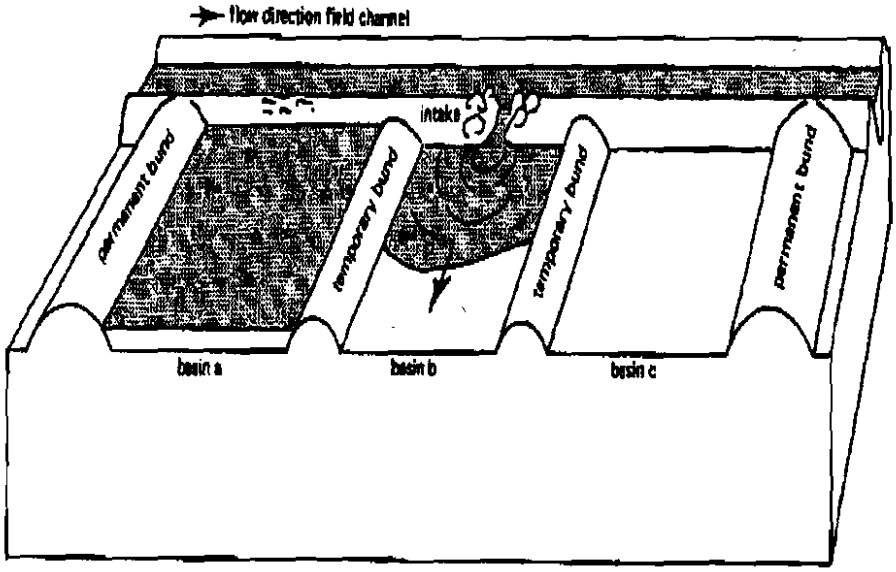


Figure 12.1. Direct Method of Water Supply

The Cascade Method
On sloping land, where terraces are used, the irrigation water is supplied to the highest terrace, and then allowed to flow to a lower terrace and so on.

12.2 Border Irrigation
Borders are usually long, uniformly graded strips of land, separated by earth bunds. In Contrast to basin irrigation these bunds are not to contain the water for ponding but to guide it as it flows down the field (Figure 12.2).

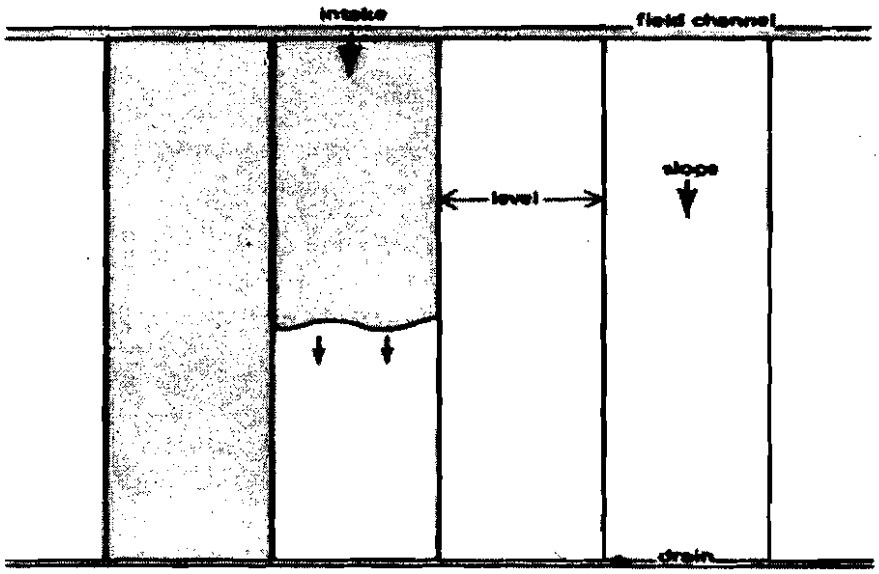


Figure 12.2. Border Irrigation

When to Use Border Irrigation

Border irrigation is generally best suited to the larger mechanized farms as it is designed to produce long uninterrupted field lengths for ease of machine operations. Borders can be up to 800 m or more in length and 3-30 m wide depending on a variety of factors. It is less suited to small-scale farms involving hand labour or animal-powered cultivation methods.

Suitable slopes: Border slopes should be uniform, with a minimum slope of 0.05% to provide adequate drainage and a maximum slope of 2% to limit problems of soil erosion.
Suitable soils: Deep homogenous loam or clay soils with medium infiltration rates are preferred. Heavy, clay soils can be difficult to irrigate with border irrigation because of the time needed to infiltrate sufficient water into the soil. Basin irrigation is preferable in such circumstances.

Suitable crops: Close growing crops such as pasture or alfalfa are preferred.

Border Layout

The dimensions and shape of borders are influenced in much the same way as basins and furrows by the soil type, stream size, slope, irrigation depth and other factors such as farming practices and field or farm size.

Many of the comments made about basins and furrows are generally applicable to borders also and so do not require repetition here. Table 12.3 provides a guideline to determine maximum border dimensions. It must, however, be stressed that this table is for general guidance only as the values are based on field experience and not on any scientific relationships.

Table 12.3: Suggested Maximum Border Lengths and Widths

Soil type	Border Slope (%)	Unit flow per metre width (l/sec)	Border Width (m)	Border Length (m)
SAND	0.2-0.4	10-15	12-30	60-90
Infiltration rate greater than 25 mm/h	0.4-0.6	8-10	9-12	60-90
	0.6-1.0	5-8	6-9	75
LOAM	0.2-0.4	5-7	12-30	90-250
Infiltration rate of 10 to 25 mm/h	0.4-0.6	4-6	6-12	90-180
	0.6-1.0	2-4	6	90
CLAY	0.2-0.4	3-4	12-30	180-300
Infiltration rate less than 10 mm/h	0.4-0.6	2-3	6-12	90-180
	0.6-1.0	1-2	6	90

Note: The flow is given per metre width of the border. Thus the total flow into a border is equal to the unit flow multiplied by border width (in metres).

Irrigating Borders

Borders are irrigated by diverting a stream of water from the channel to the upper end of the border. The water flows down the slope. When the desired amount of water has been delivered to the border, the stream is turned off. This may occur before the water has reached the end of the border. There are no specific rules controlling this decision.

However, if the flow is stopped too soon there may not be enough water in the border to complete the irrigation at the far end. If it is left running for too long, then water may run off the end of the border and be lost in the drainage system.

As a guideline, the inflow to the border can be stopped as follows:

- On clay soils, the inflow is stopped when the irrigation water covers 60% of the border. If, for example, the border is 100 m long a stick is placed 60 m from the farm channel. When the water front reaches the stick, the inflow is stopped.
- On loamy soils it is stopped when 70 to 80% of the border is covered with water.
- On sandy soils the irrigation water must cover the entire border before the flow is stopped.

However, these are only guidelines. Realistic rules can only be established locally when testing the system.

12.3 Furrow Irrigation

Furrows are small channels, which carry water down the land slope between the crop rows. The crop is usually grown on top or half way down the ridges between the furrows. This method is suitable for row crops that cannot stand in water for long periods. The method allows applying light irrigation (30-50 mm) and can be laid out in sloping fields along the contour. Considerable water savings can be obtained if layout is well done.

Figure 12.3. Furrow Irrigation

When to Use Furrow Irrigation

Furrow irrigation is suitable for a wide range of soil types, crops and land slopes, as indicated below.

Suitable Crops

Furrow irrigation is suitable for many crops, especially row crops. Crops that would be damaged if water covered their stem or crown should be irrigated by furrows.

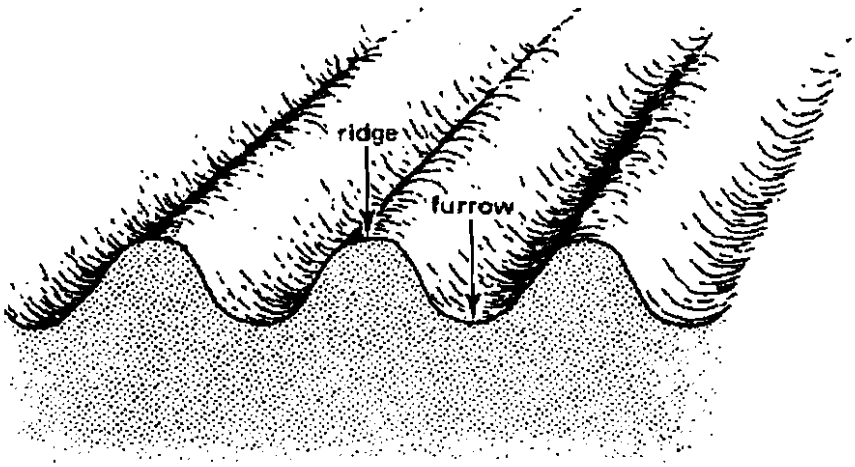


Figure 12.4 Top view and cross-section of furrows and ridges

Furrow irrigation is also suited to the growing of tree crops. In the early stages of tree planting, one furrow alongside the tree row may be sufficient but as the trees develop then two or more furrows can be constructed to provide sufficient water. Sometimes a special zig-zag system is used to improve the spread of water.

Corrugation irrigation, frequently mentioned in literature, is a special type of furrow irrigation, used for broadcast crops. Corrugations are small hills pressed into the soil surface. The application of this method is limited and is not included in this manual.
In summary, the following crops can be Irrigated by furrow irrigation:

- row crops such as maize, sunflower, sugarcane, soybean;
- crops that would be damaged by inundation, such as tomatoes, vegetables, potatoes, beans;
- fruit trees such as citrus, grape;
- broadcast crops (corrugation method) such as wheat.

Suitable Sopes
Uniform flat or gentle slopes are preferred for furrow irrigation. These should not exceed 0.5%. Usually a gentle furrow slope is provided up to 0.05% to assist drainage following irrigation or excessive rainfall with high intensity.

On undulating land furrows should follow the land contours. However, this can be a difficult operation requiring very careful setting out of the contours before cutting.
Suitable soils

Furrows can be used on most soil types. However, as with all surface irrigation methods, very coarse sands are not recommended as percolation losses can be high. Soils that crust easily are especially suited to furrow irrigation because the water does not flow over the ridge, and so the soil in which the plants grow remains friable.

Furrow Layout
This section deals with the shape, length and spacing of furrows. Generally, the shape, length and spacing are determined by the natural circumstances, i.e. slope, soil type and available stream size. However, other factors may influence the design of a furrow system, such as the irrigation depth, farming practice and the field length.

Furrow Length
Furrows must be on consonance with the slope, the soil type, the stream size, the irrigation depth, the cultivation practice and the field length. The impact of these factors on the furrow length is discussed below.

Slope
Although furrows can be longer when the land slope is steeper, the maximum recommended furrow slope is 0.5% to avoid soil erosion. Furrows can also be level and are thus very similar to long narrow basins. However a minimum grade of 0.05% is recommended so that effective drainage can occur following irrigation or excessive rainfall. If the land slope is steeper than 0.5% then furrows can be set at an angle to the main slope or even along the contour to keep furrow slopes within the recommended limits. Furrows can be set in this way when the main land slope does not exceed 3%. Beyond this there is a major risk of soil erosion following a breach in the furrow system.

On steep land, terraces can also be constructed (see Basin Irrigation) and furrows cultivated along the terraces.

Soil Type
In sandy soils water infiltrates rapidly. Furrows should be short (less than 110 a), so that water will reach the downstream end without excessive percolation losses.
In clay soils, the infiltration rate is much lower than in sandy soils. Furrows can be much longer on clayey than on sandy soils.

Stream Size
Normally stream sizes up to 0.5 l/sec will provide an adequate irrigation provided the furrows are not too long. When larger stream sizes are available, water will move rapidly down the furrows and so generally furrows can be longer. The maximum stream size that will not cause erosion will obviously depend on the furrow slope; in any case, it is advised not to use stream sizes larger than 3.0 l/sec (see Table 12.4).

Irrigation Depth
Applying larger irrigation depths usually means that furrows can be longer as there is more time available for water to flow down the furrows and infiltrate.

Cultivation Practice

When the farming is mechanized, furrows should be made as long as possible to facilitate the work. Short furrows require a lot of attention as the flow must be changed frequently from one furrow to the next. However, short furrows can usually be irrigated more efficiently than long ones as it is much easier to keep the percolation losses low.

Field length

It may be more practical to make the furrow length equal to the length of the field, instead of the ideal length, when this would result In a small piece of land left over. Equally the length of field may be much less than the maximum furrow length. This is not usually a problem and furrow lengths are made to fit the field boundaries.

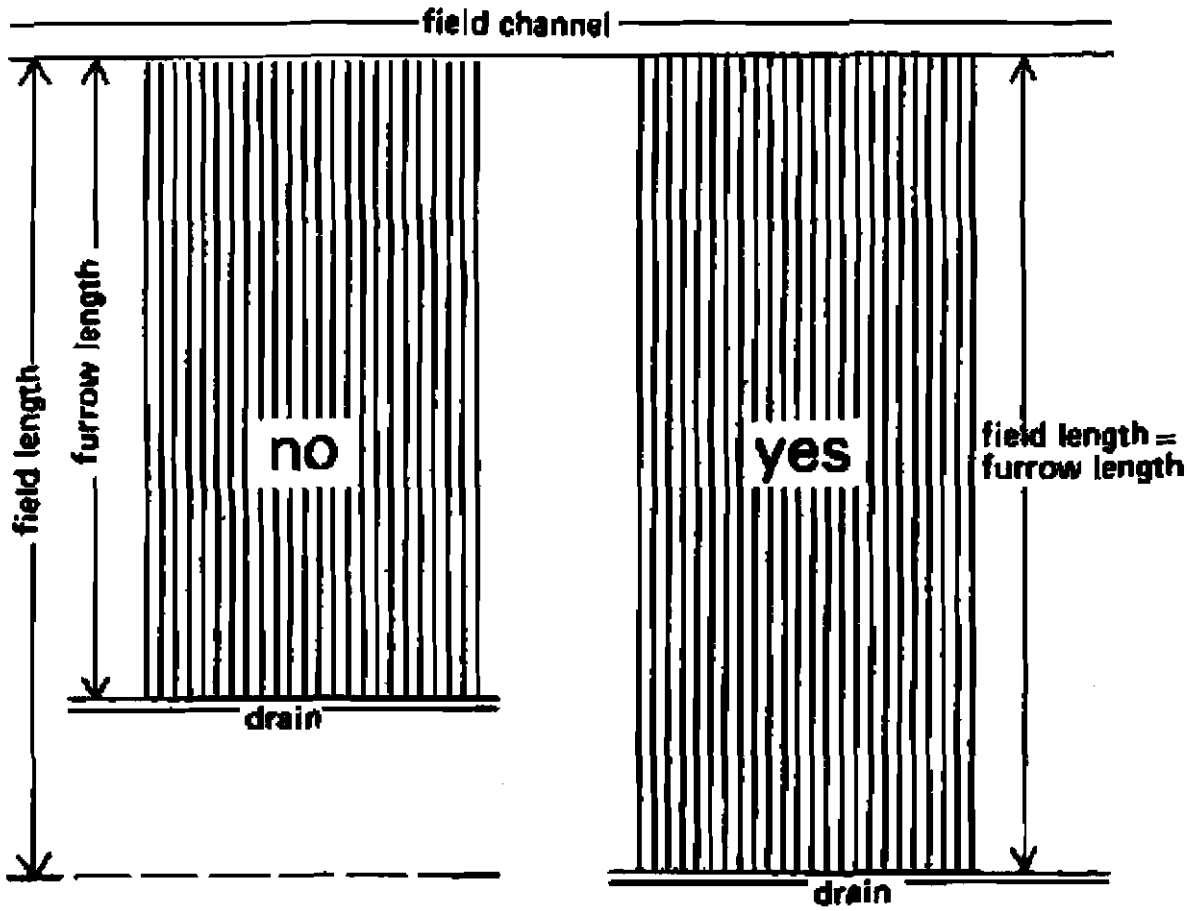


Figure 12.5 Field length and furrow length

Table 12.4 gives some practical values of maximum furrow lengths under small-scale irrigation conditions. The values shown in Table 12.4 are lower than those generally given in irrigation handbooks. These higher values are appropriate under larger scale, fully mechanized conditions.

Table 12.4 Practical Values of Maximum Furrow Lengths (m) Depending on Slope, Soil Type, Stream Size and Net Irrigation Depth

Furrow slope (%)	Maximum stream size (l/s) per furrow	Clay		Loam		Sand	
		Net irrigation depth (mm)					
		50	75	50	75	50	75
0.0	3.0	100	150	60	90	30	45
0.1	3.0	120	170	90	125	45	60
0.2	2.5	130	180	110	150	60	95
0.3	2.0	150	200	130	170	75	110
0.5	1.2	150	200	130	170	75	110

This table only provides approximate Information relating furrow slope, soil type, stream size and irrigation depth to furrow lengths. This should only be used as a guide as the data are based primarily on field experience and not on any scientific relationships. Maximum values of furrow length are given for reasonably efficient irrigation. However, furrow lengths can be even shorter than those given in the table and in general this will help to improve irrigation efficiency. Only by Installing a furrow system, following the guidelines, and then evaluating its performance can an appropriate system be developed for a given locality.

Furrow Shape

The shape of furrows is influenced by the soil type and the stream size.

Soil type: In sandy soils, water moves faster vertically than sideways (= lateral). Narrow, deep V-shaped furrows are desirable to reduce the soil area through which water percolates. However, sandy soils are less stable, and tend to collapse, which may reduce the irrigation efficiency.

In clay soils, there is much more lateral movement of water and the infiltration rate is much less than for sandy soils. Thus a wide, shallow furrow is desirable to obtain a large wetted area (Figure 12.7) to encourage infiltration.

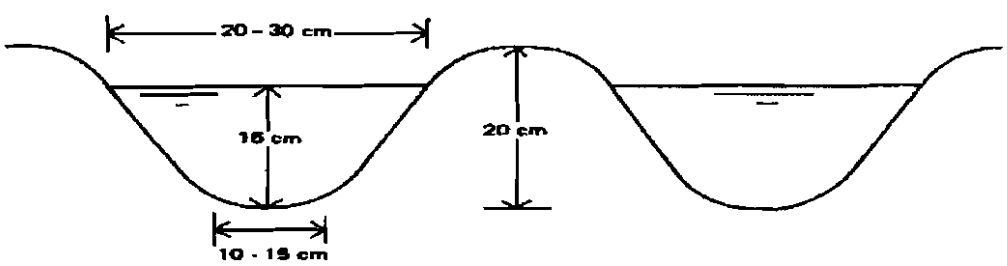


Figure 12.6. A Deep, Narrow Furrow on a Sandy Soil

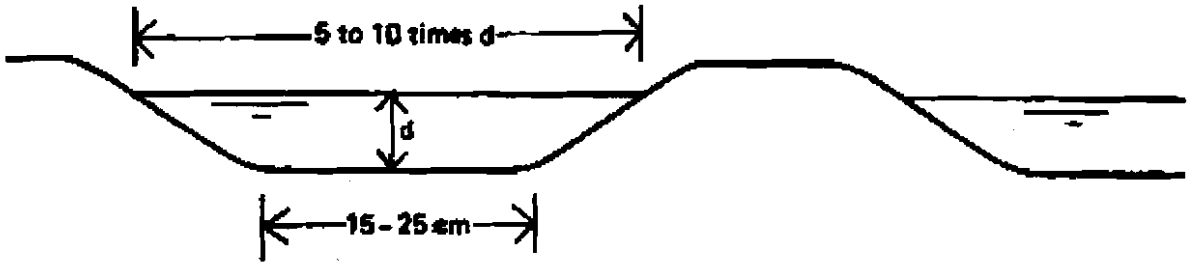


Figure 12.7. A Wide, Shallow Furrow on a Clay Soil

Stream Size
In general, the larger the stream size the larger the furrow must be to contain the flow.

Furrow spacing
The spacing of furrows is influenced by the soil type and the cultivation practice.

Soil type
As a rule, for sandy soils the spacing should be between 30 and 60 cm, i.e. 30 cm for coarse sand and 60 cm for fine sand.

On clay soils, the spacing between two adjacent furrows should be 75-150 cm. On clay soils, double-ridged furrows - sometimes called beds - can also be used. Their advantage is that more plant rows are possible on each ridge, facilitating manual weeding. The ridge can be slightly rounded at the top to drain off water that would otherwise tend to pond on the ridge surface during heavy rainfall (Figure 12.8).

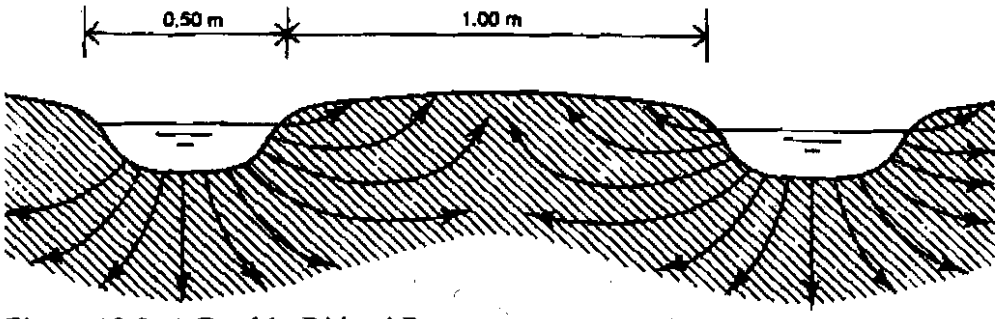


Figure 12.8. A Double-Ridged Furrow

Cultivation Practice
In mechanized farming a compromise is required between the machinery available to cut furrows and the ideal spacing for crops. Mechanical equipment will result in less work if a standard width between the furrows is maintained, even when the crops grown normally require a different planting distance. This way the spacing of the tool attachment does not need to be changed when the equipment is moved from one crop to another. However, care is needed to ensure that the standard spacing provide adequate lateral wetting on all soil types.

Irrigating Furrows

Water is supplied to each furrow from the field canal, using siphons or spiles. Sometimes, instead of the field canal with siphons or spiles, a gated pipe is used. Depending on the available flow in the farm channel, several furrows can be irrigated at the same time.

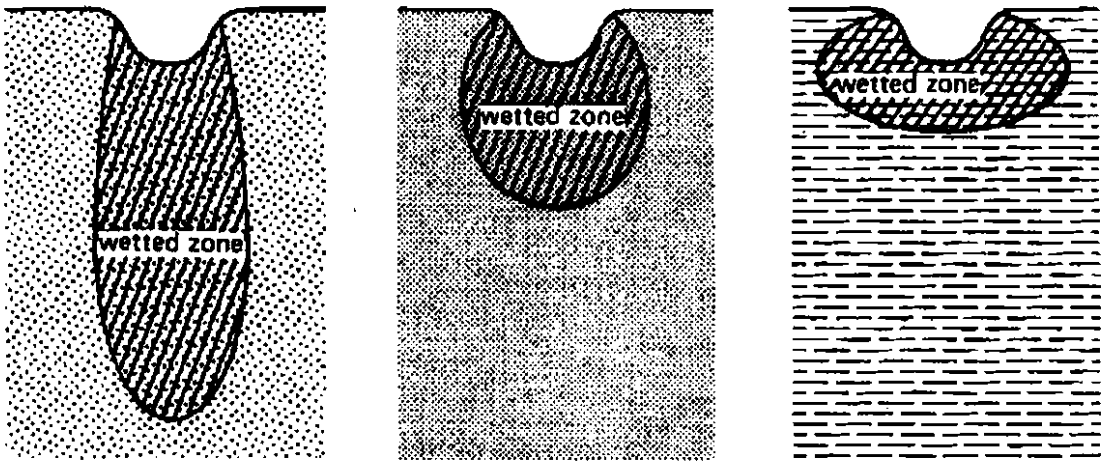
When there is a water shortage, it is possible to limit the amount of irrigation water applied by using 'alternate furrow irrigation'. This involves irrigating alternate furrows rather than every furrow.

Runoff at the ends of furrows can be a problem on sloping land. This can be as much as 30 percent of the inflow, even under good conditions. Therefore a shallow drain should always be made at the end of the field, to remove excess water. When no drain is made, plants may be damaged by waterlogging. Light vegetation allowed to grown in the drain can prevent erosion. Excessive runoff can be prevented by reducing the inflow once the irrigation water has reached the end of the furrows. This is called cut-back irrigation. It may also be possible to reuse runoff water further down the farm.

Wetting Patterns

In order to obtain a uniformly wetted rootzone, furrows should be properly spaced, have a uniform slope and the irrigation water should be applied rapidly.

As the root zone in the ridge must be wetted from the furrows, the downward movement of water in the soil is less important than the lateral (or sideways) water movement. Both lateral and downward movement of water depends on soil type as can be seen in Figure 12.9.



A - SAND B - LOAM C - CLAY

Figure 12.9 Different wetting patterns in furrows, depending on the soil type

Ideal Wetting Pattern

In an ideal situation adjacent wetting patterns overlap each other, and there is an upward movement of water (capillary rise) that wets the entire ridge (see Figure 12.10), thus supplying the root zone with water.

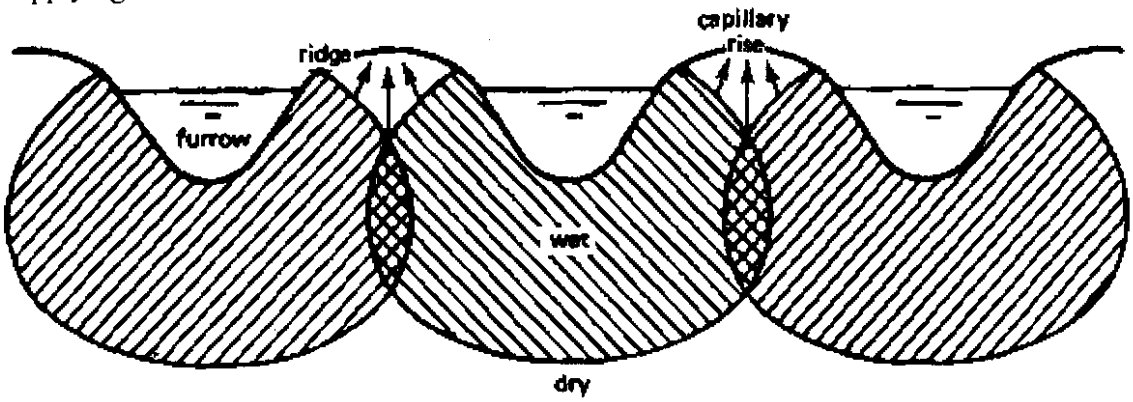


Figure 12.10 Ideal wetting pattern

To obtain a uniform water distribution along the furrow length, it is very important to have a uniform slope and a large enough stream size so that water advances rapidly down the furrow. In this way large percolation losses at the head of the furrow can be avoided. The quarter time rule is used to determine the time required for water to travel from the farm channel to the end of the furrow, in order to minimize percolation losses.

Poor Wetting Patterns

Poor wetting patterns can be caused by:

- unfavorable natural conditions, e.g. a compacted layer, different soil types, uneven slope;
- poor layout, e.g. a furrow spacing too wide;
- poor management: supplying a stream size that is too large or too small, stopping the Inflow too soon.

Unfavorable Natural Conditions

Compacted soil layers or different soil types have the same effect on furrow irrigation as they have on basin irrigation. The solution to the problem is also similar.

An uneven slope can result in uneven wetting along the furrow. Water flows fast down the steep slopes and slowly down the flatter slopes. This affects the time available for infiltration and results in poor water distribution. The problem can be overcome by regarding the land to a uniform slope.

Poor layout

If the furrow spacing is too wide (Figure 12.11) then the root zone will not be adequately wetted. The spacing of furrows needs careful selection to ensure adequate wetting of the entire root zone (Figure 12.11).

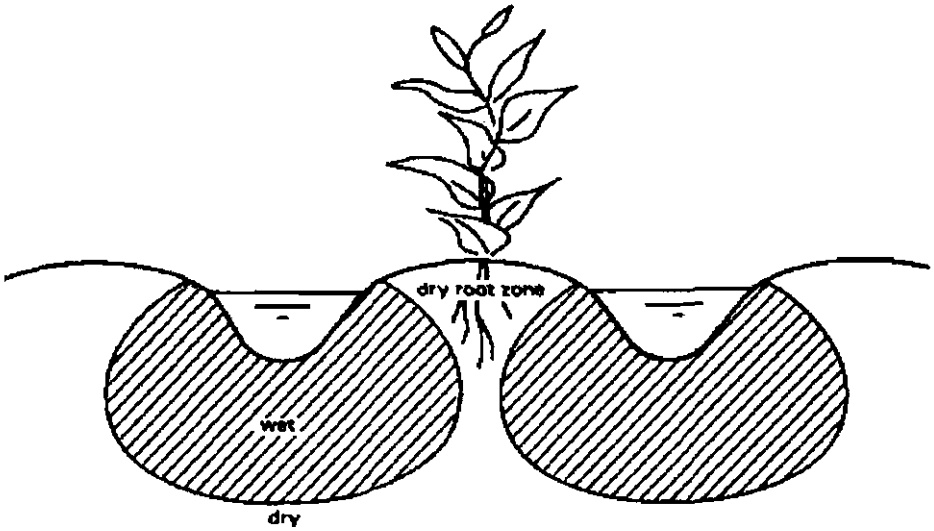


Figure 12.11 The spacing between two adjacent furrows is too wide

Poor management

A stream size that is too small (Figure 12.12) will result in inadequate wetting of the ridges. Even if the plants are located at the sides of the ridge, not enough water will be available. A small stream size will also result in poor water distribution along the length of the furrow. The advance will be slow and too much water will be lost through deep percolation at the head of the furrow.

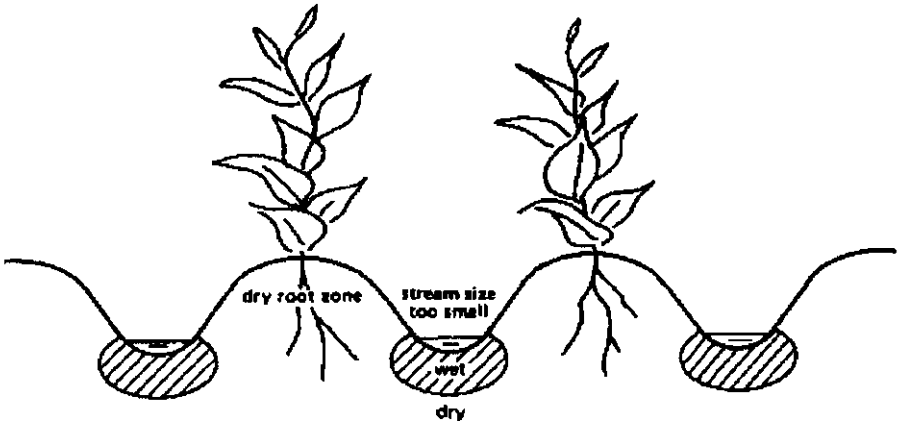


Figure 12.12 Stream size is too small to wet the ridge

If the stream size is too large on flat slopes, overtopping of the ridge may occur (Figure 12.13). On steeper slopes with too large a stream size, erosion of the bed and sides of the furrow may take place (Figure 12.13).

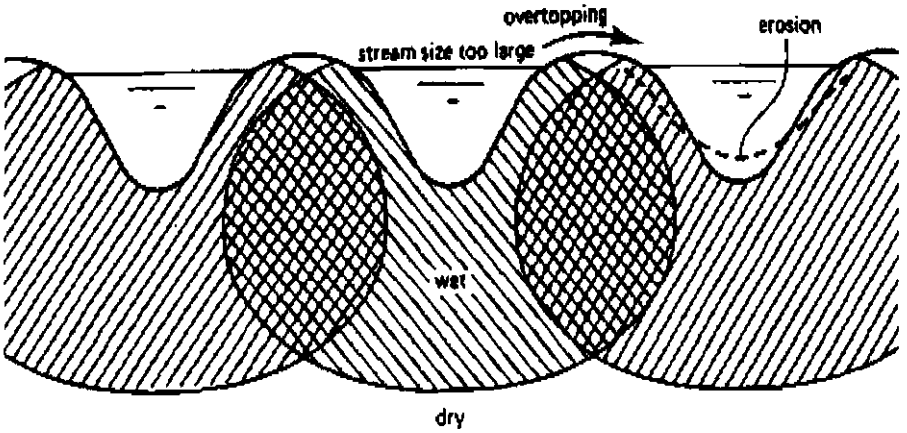


Figure 12.13 Stream size too large causing overtopping or erosion

A common management fault is to stop the inflow too soon. This is usually done to reduce runoff, but it results in a poor water distribution and the plants in particular at the end of the furrow do not get enough water. If the Inflow of irrigation water is not stopped soon enough, the runoff is excessive and plants at the end of the furrow may drown when an adequate drainage system to evacuate excess water is not provided.

Planting Techniques

The location of plants In a furrow system is not fixed but depends on the natural circumstances. A few examples will be mentioned.

- In areas with heavy rainfall, the plants should stand on top of the ridge in order to prevent damage as a result of waterlogging (Figure 12.14).
- If water is scarce, the plants may he put in the furrow itself, to benefit more from the limited water (Figure 12.15).
- As salts tend to accumulate in the highest point, a crop on saline soils should be planted away from the top of the ridge. Usually it is planted in two rows at the sides (Figure 12.16). However, it is important to make sure there is no danger of waterlogging.
- For winter and early spring crops in colder areas, the seeds may be planted on the sunny side of the ridge (Figure 12.17). In hotter areas, seeds may be planted on the shady side of the ridge, to protect them from the sun.

Figure 12.14 Protection against waterlogging

Figure 12.15 Protection against water scarcity

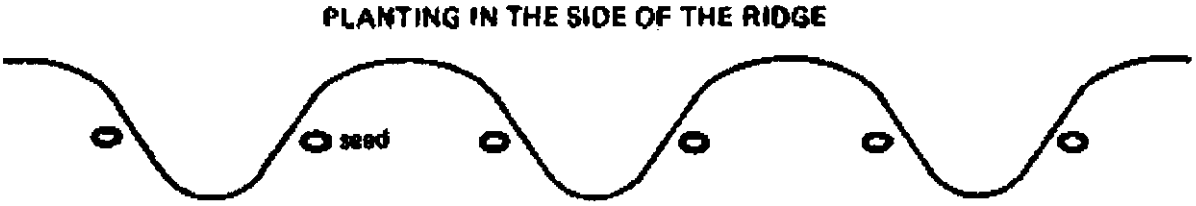


Figure 12.16 Protection against accumulation of salt

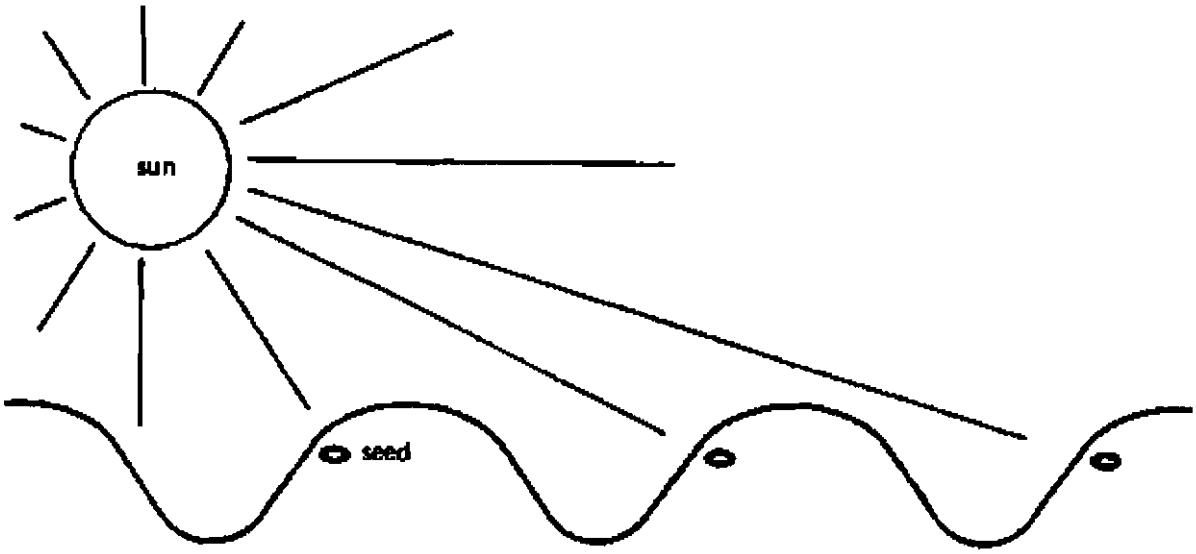


Figure 12.17 Winter and early spring crops: seeds planted on the sunny side of the ridge

13 SPRINKLER IRRIGATION

In sprinkling irrigation, water is sprayed into the air through a sprinkler nozzle and allowed to fall on the land surface in a uniform pattern at a rate less than the infiltration rate of the soil. The water application to the fields is somewhat similar to rainfall. Sprinkler irrigation has been in use for more than three decades in many parts of the world particularly in United States of America and the continental Europe. Sprinkler irrigation is very popular in Israel as about 95 per cent of the total irrigated area is under sprinkler. Use of sprinklers is gradually paining momentum in mid-east countries, such as Tunisia, Libya and Turkey.

Initially, the sprinklers were used on high value plantation corps such as tea, coffee, cardamom and orchards. Their use in gaining popularity on food crops, orchards, cotton and vegetables in areas where sprinkling is economically justifiable and technically feasible.

In order to boost food production for the growing population, irrigation supplies are needed to be augmented considerably to cover a large area of reclaimed land and at the same time, more scientific and economical use of water by sprinkling irrigation is likely to attract more attention in the near future. This method prevents soil deterioration in the form of scouring, packing and crusting of soil, rise in water table, disposal of fertile topsoil, reduction of soil permeability etc. Moreover the method is best suited for irregular topography, slipping gourds, small sized plots and fields containing crops with different water requirements. It can serve a large number of farmers with the same supply line.

13.1 Advantages of Sprinkler Irrigation

Sprinkler irritation system has the following advantages:

Seepage losses eliminated: Loss of water, such can be as high as 35 per cent, occurring as seepage loss in earthen channels during conveyance of water is eliminated. This loss of water in conveyance can be made to disappear in surface irrigation also by either using lined channels or underground pipe lines.

Much less water needed: Only that much water as is needed by the crop root zone is required to be supplied. This results in considerable saving in the quantity of water used for raising corps. The saving can be up to 65 per cent of water required for flood irrigation. This for a given quantity of water, sprinkler irrigation method can irrigate much larger area as compared to surface irrigation.

No leveling required: For uniform water application, no land leveling is required. This has an added advantage in the top fertile layer is not disturbed and lost which is inevitable with leveling operation if surface irrigation method is adopted.

Maximum land use: There is no loss of production area in making levees. This may be as high as 16 per cent, which is lost either in making earthen conveyance channels or irrigation levees. Whole of this area can be brought under cultivation, if sprinkler irrigation is practiced. Thus the land can be fully utilized for cultivation of crops.

Suited to coarse soil: Sprinkler is very much suited to irrigate coarse soil especially which stream size available is too small (less than 900 liters per minute).

Elimination of run-off and percolation losses: Sprinkler can apply irrigation at a rate, which is less than the infiltration rate of the soil. This helps in total elimination of run-off losses. There is a price control over the depth of application also. Sprinkler systems can apply any depth of application, thus eliminating deep percolation losses, which may be more predominant in light soil under surface irrigation. Many a time (as while seeds germinate) only light irrigation is required which can be effectively applied with sprinklers whereas; surface irrigation system can be designed only for a given depth of irrigation. Thus shallow irrigation, required by a crop in this early stage of growth, can be easily applied with sprinkler, which may be practically impossible with surface irrigation.

Accurate moisture control: Accurate and correct amount of moisture can be applied through the relation of timing, intensity and spread of irrigation water. Over-irrigation is completely eliminated.

Application of Fertilizers: Fertilizers and other such treatments like soil amendments can be applied in solution form along with irrigation water from the sprinklers. These materials are distributed uniformly and can be leached to the desired depth.

Maximum return: Maximum return of the precious inputs viz., seeds, fertilizer, labor and land investment is ensured. Early crop ripening brings better prices and hence higher profits from the produce.

Multiple crops: Crops tremendously benefit from right quantity and the right mode of watering. Double or even triple harvests are possible. Gentle penetration: penetration, as the root structure of the crop demands for complete nourishment, results from very fine droplets of water which gently settle on the soil and are immediately absorbed. Light irrigation needed for seedlings and young plants is possible.

Soil erosion eliminated: Fine sprays, gentler than even rain, just settle on the upper most layer of the soil and get immediately absorbed thus avoiding washing away of the top soil. Likewise fertilizers and soil nutrients present in the soil are also not worked away.

Crop damage reduced: Crop damage form frost can be reduced. Crops grown are immune to the risk of failure.

Portable system of irrigation: The entire piping system being light, cab be dismantled, reassembled and moved from one location to any other by a single man,. Locking and unlocking of pipes, values and fittings is just a matter of minutes.

Uniform water application: Very even rate of water application is achieved. The sprinkler nozzle gives uniformity of rain (+10%).

Danger of water logging eliminated: Contrary to surface irrigation wherein 70%of the water applied percolates to the water table resulting in creation of water logging conditions and adverse effects on the crop yield, sprinkler irrigation does not tend to raise water table.

Mechanical cultivation: Absence filed channels and furrows makes possible use of mechanical cultivation equipemn6t to prepare the soil for sowing or planting and to help maintain the soil in good condition up to harvesting.

Tillage cost reduced: As the soil does not tend to clod under sprinkler irrigation, a good tilth is easier to obtain. At least one dise harrow is saved.

Leaches down salts: Slats leach down in basic soils by sprinkler irrigation. Accordingly lands, where slats are brought to surface-by-surface irrigation, are brought to full productively by this method.

Low labour requirement: Labour requirement is reduced due to:Unskilled labour easily becomes, proficient in the simple and repetitive takes involved. It is one-man job to dismantle and move the equipmen6t, shift from one location to the other, the entire system being extremely portable. It does not require same attention as the surface irrigation requires.
Weeds and pests controlled: Insects and pests are discouraged due to frequent sprinkler irrigations. Weeds grown, when row crops are young, are easily eliminated.

Early production: New lands become productive earlier and a cover crop cab quickly be established even before clearing has been completed.

Assist germination: Seed germination is assured as the soil after sowing is kept in the condition conducive to assist germination and the emergence of the plant is aided by gentle application of water.

Higher crop yields and quantity: Both the yield and quantity of crop is higher under sprinkler irrigation than surface irrigation with the same conditions of soil fertility and climate. Soil fertility is improved as microorganism work to their fullest extent.

Supplemental to rainfall: Sprinkler irrigation is particularly well adapted to the areas where it is supplemental to a reasonably adequate rainfall. In the periods of inadequate rainfall, sprinklers can be quickly transported and set up to provide relief.

Special uses of sprinkler:
The inherent advantages of sprinklers have several other jobs as described briefly in the following.

Control of alkali: Sprinkler could be used; the control of soil alkali. White alkali is controlled and can be directly leached. Black alkali decomposed of sodium and potassium carbonates are insoluble and must be changed to soluble salts by addition of soil amendments such as gypsum and sulphur. The alkali soils have a very low infiltration rate due to dispersion of organic and inorganic matter and formation of a jelly mass and structure less soil-cementing agent. Therefore, when leaching alkali soil, the rate of application of water should be slightly slower than the infiltration rate of the soil so that the soluble salts percolate down the soil and gradually improve the soil condition by leaching away the slats in successive irrigations.

Irrigation with high slat content water. It is a common practice in sprinkler irrigation to saturate only the root of the crop in each irrigation. However, when using water with high salt content, this practice must be modified, as otherwise the salts will get accumulated in the root zone, and become toxic. When using waters of high slat content, the accumulated salts re1uired for this leaching may be 1.5 to 2.5 times the normal set, depending upon the soil conditions and the rate of water application.

Frost protection. Sprinkler irrigation can protect crop against frost damage when the temperature drops below freezing point. Water emits heat when it starts freezing on plants. This beat is diffused in plant foliage and surrounding air. The heat emitted by the water is absorbed by the plant and keeps the plant form freezing. Sensitive crops such as clergy, tomatoes, potatoes, straw berries and grapes, which are rqui5red to be protected from frost and low temperatures. For protecting, water must be sprayed continuously on plants and on the entire area to be protected. The system designed for frost port6ection does not require high unifomt9iy of application. Wider spacing can be used to achieve approximately 2.5 mm per hour water application rates consistent with good coverage, but not high uniformity. The low application rate is required as the frost may last a few days continually. High application rates could waterlog the soil, especially with a pre-irrigation. To obtain low application rates the discharge form a two nozzle sprinkler head can be reduced by plugging one of the nozzles.

Crop Cooling. Different crops have different critical temperatures above which the yield is influenced adversely. The design of sprinklers for cooling is similar to that of frost protecting, except that the water for cooling does not have to be applied continuously. Grape cooling has been found fairly effective by alternating 15 minutes on and 15 minutes off during a 6 hour cooling period. It is still a subject of research to know about the duration and quantity of sprinkling under different conditions of crops and climate.

Aeration irrigation. In aeration irrigation low application rates are combined with small size of the water drops. In this way the soil structure is not broken up and the soil air is never enti4rely excluded. Thus aeration irrigation has resulted in improved soil structure and elimination of soil compaction due to large size drop impact. The application rates are very slow; approximately y one half of the normal infiltration rate of the soil combined with good nozzle pressure to breakup the drops. The application rates vary form 7.5 mm/hr on light soils, 5 mm/hr on medium textured soil and 2.5 to 3.75 mm/hr on heavy soils at nozzle pressures of 3.35 to 3.5 kg/cm2. The system requires about double the number of sets as the application rate is cut to half. The results achieved in western countries are encouraging and has off set the added equipment cost and the additional labour required.

Fertilizer application. Soluble chemical fertilizers can be injected into the sprinkler systems and applied to the corp. The fertilizer is easily place at the desired depth in a soluble and readily available form to the plants, without any danger of being leached way,. The two operations, name irrigation and fertilizer applications, are done simultaneously thus saving the labour required for fertilizer applications.

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The equipment of fertilizer application is relatively cheap and simple and can be fabricated locally. The fertilizer injector can be made from a sealed tank or oil barrel and some tubing s and co0nnencteions. The venturi creates differential pressures and allows the fertilizer solution to flow in the main water line.

Fertilizer is also introduced into the system form the suction side of the pump through a pipe and regulated by a valve. Another pipe is connected from the discharge side of the pump to the fertilizer container for filling it with water. This system is relatively easier, but pump impellers are likely to be corroded due to fertilizer solution unless the impellers are made of corrosion resistance materials.

Limitation of Sprinkler irrigation

There are situations, which limit the use of sprinkler irrigation are as follows:

Poor uniformity is achieved when wind velocity is more than 16 km per hour.

Under conditions of high temperature, high wind velocity and low humidity, there could be considerable (10 to 15 per cent) evaporation losses form a spray while on its way from nozzle to ground surface. Sometimes these losses can be reduced if irrigation is made during night hours.

Initial cost of sprinkler set is high. In situation where surface irrigation could also be a feasible alternative, economy and performance should be comrade for both systems on long-term basis.

Sprinklers are not suited for crops requiring frequent large depth of irrigation, as in the case of paddy.

Soil born organisms are sometimes transported; to foliage through splashing form soil, which may result in spreading of some infectious diseases.

Sprinkler system requires continuous supply of water till required application has been made. In case of canal irrigation where supply is made to farmers on fixed hours turn basis, use of this system becomes more and more difficult.

Water has to be free from sand and other insoluble impurities; otherwise the pumping set will get damaged. Manural value of canal water is absent. It requires large power for its operation.

Table 13.1 Sprinkler versus Surface Irrigation system.

S.No	Surface Irrigation	Sprinkler Irrigation
1	Land surface has essentially to be leveled	More efficient water application for irrigation in valley and basin lands where leveling is difficult to be carried out.
2	Mechanized smoothening of individual fields is economically justified if the area is about 4 hectares.	Suitable for small land holdings.
3	Installation costs and power requirements are less.	Higher installation costs and greater power requirements
4	Uniformity in water application is difficult to obtain.	80% to 90% uniformity in water application rate is possible.
5	Results in water logging.	If the area is waterlogged, land can be brought back to full productivity. Seed rooting in waterlogged soil is prevented.
6	60% to 70% of the water applied by surface irrigation is lost in evaporation and deep percolation.	Water losses are much less. Percolation loss is practically negligible.
7	More water is required for irrigation because application rate and uniformity cannot be assured and losses are more.	Less water is required for irrigating a given area because application rates and uniformity of water application can be assured and losses are less.
8	Considerable movement of top soil to fill low spots in the fields takes place which increases erosion	Fertile top soil remains undisturbed. Fields having any slope can be irrigated and as such soil erosion is considerably reduced.
9	7% to 10% of the land comes under irrigation canals, water courses, etc. and is lost to cultivation.	All the land is available for cultivation and plant densities can be increased.
10	Furrows, ditches, water courses interfere with the operation of mechanized equipment	Mechanical cultivation equipment is more easily employed because of absence of ditches, furrows, water courses etc.
11	Good soil conditions are difficult to be maintained due to the movement of the finer particles into the voids of the coarser particles of the soil	Good soil condition right up to the harvest time is easily maintained.
12	Soluble fertilizers are carried by irrigation water to the low level spots in the filed and as such uniformity cannot be maintained.	Soluble fertilizer and soil amendments are easily applied with irrigation water.
13	Soil tends to clod, puddle making tillage operation not so easy.	Good tilth is much easier to obtain as the soil does not tend to clod. At least one disc harrow is saved.
14	Flooding reduces infiltration rate.	Water can be applied at a rate to suit soil infiltration rate and as such light, medium or heavy soils are irrigated with equal success.
15	Great deal of preparatory work is required to bring new land under cultivation.	Earlier production is rendered possible. A cover crop can be quickly established even before clearing has been completed.
16	Harmful ditch weeds are common.	Absence of ditches eliminates the growth of ditch weeds.

13.2 Limitation of Sprinkler System

High Capital Cost

Since the system has to be privately owned, many small farmers cannot afford it.

Operational Cost

Except for running cost of pump and cost of electricity, the running cost of the system is not high.

Operational Difficulties

The system is simple and except for running, repairs and maintenance of pumps and tractor(if used) , there is no major problem.

Windy conditions

Windy conditions disturb spray pattern of sprinklers and create irregaular irrigation of the fields. The , however, can be adjusted to a large extent by manipulation of pumping pressure.

Disadvantages of Sprinkler System

Sprinkler irrigation is not well adapted to soils having very low intake rates.

In this method evaporation losses are large, as compared to other system s, as sprinklers wet the entire soil area as well as crop leaves and plants etc.

High and continuous energy requirement for operation is required.

Sprinkler water containing salts or silt burn the plants and leaves, which may result in ultimate death of the plants;

Certain diseases spread easily of crops or fruit trees due to sprinkler.

13.3 Components of the Sprinkler System

Every type of sprinkler system consists of three main components, namely, a pumping plant, a network of pipelines and sprinklers. They also require a variety of accessories like fertilizer applicator and valves.

Pumping Plant

A pumping plant is required to lift water from the source which may be a river, a canal, a reservoir or a lakes and to pump it in the network of the pipelines at the required pressure. At certain location, a booster pump may also be required which may be located somewhere in the middle, to boost the pressure to the required level. In case the water source is located at higher elevation the pump may not be required to lift water but may be used as booster pump to maintain the required pressure.

Centrifugal or turbine pumps are usually used in this system. Centrifugal pump is used where the depth of the lift, i.e. the difference of level of pump inlet and water surface in the water source is less than 15 ft. however, if this difference is more than 15 ft. or where water surface of the source of water fluctuates widely, the use of a turbine pump is recommended.

The pump driving unit may be electric motor or an internal combustion engine which operates on diesel oil, gasoline or gas. Solar electric mat also be used where feasible. Where electricity is easily and cheaply available, electric motor is the best option.

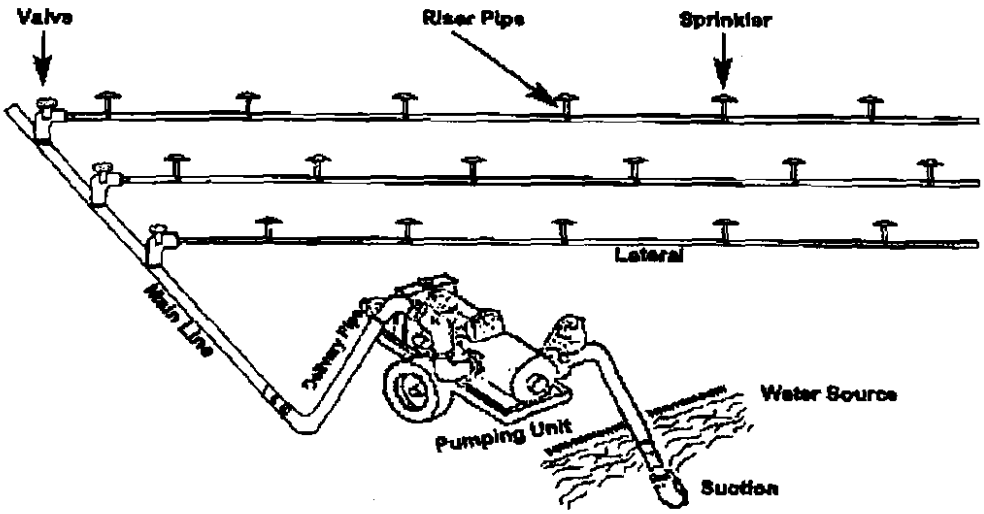


Figure 13.1 Typical Sprinkler irrigation system.

Pipe Network

Pipe work consists of main pipeline and lateral pipelines. Both type of pipelines can be permanent of portable depending upon the farm and type of use.

Main Pipelines

In the same way as in case of main canals of surface irrigation system, main pipelines are used to carry the bulk of the irrigation water to the fields to be irrigated. In case of farms with fixed boundaries or grassy plots like parks of golf courses, main pipelines are usually laid permanently. A permanent pipeline is usually buried deep in the ground so that they do not hinder operation of farm machinery specially plough or tractor. A portable pipeline is laid on the farm as and when required . steel, asbestos, PVC pipes are generally used for permanent pipelines .Portable pipelines should be lighter so that they could be easily moved. Usually aluminum, PVC pipes are used. Therefore , portable pipes are less expansive as compared to the permanent pipelines.

Lateral Pipelines

Lateral pipelines carry water from the main pipeline to the sprinkler. Each lateral, usually serve 1 to 30 sprinklers. Laterals are usually potable and are of aluminum pipes of length of 15,20 or40 ft. each length is fitted with quick coupling, which can be easily coupled of uncoupled.

Sprinkler Heads

Sprinklers are available in different types and shapes. However, the common feature of each type is that water from the sprinkler nozzle of perforated pipe sprays at high speed. Sprinklers are of two types, fixed or rotating type. Most of the sprinklers used these days are of rotating type. The various types of sprinklers are described below

13.4 Type of Fixed Sprinklers

These are fixed head sprinklers, perforated pipes and nozzle lines.

Fixed Head Sprinklers

Fixed head sprinklers are commonly used to irrigate small lawns and gardens. There is no moving part so these are simple and cheap. They operate at low-pressure head; therefore, they can be spaced closely and are mostly used with permanent system. Generally these sprinklers operate with spacing upto20x20 ft. they are suitable for irrigating parks, lawns, grasses, golf courses etc.

Perforated Lateral Pipelines

These sprinklers are used only for soils which have high water intake requiring smaller pressure than rotating sprinklers. A number of holes are bored in standing laterals pipes, indifferent arrangement and in different diameters. Because of the small diameter hole they get often clogged and have to be flushed. These sprinklers are suitable for orchard of small vegetable fields.

Nozzle Lines

A nozzle line is small pipe of galvanized steel with a single row of small nozzles spaced at regular intervals of 2 ft. to 3ft. along the entire pipe length. The pipes are supported overhead on posts and can be rotated to irrigate a strip of land along either side of the pipe length.

Types of Rotating Sprinklers

Rotating sprinklers are usually of three types, namely; Rapid whirling; giant of gun machine; and slowly rotating type.

Rapid Whirling Sprinkler (Reaction Rotating)

In this type the nozzle or nozzle of the sprinkler are so oriented that the reaction from the jet caused them to whirl. These sprinklers do not cover large areas due to rapid rotation, however, they are suitable for under tree sprinkling and for small fields. These types are not very popular.

Giant of Gun Sprinkler (Boom Type Sprinkler)

Giant of gun sprinklers are usually fixed to long rotating arms. Most gun sprinklers rotate by means of " rocker arm drive". and are usually used to irrigate irregular fields with obstruction

c. Slowly Rotating Sprinklers (slow rotating impact drive Type)

Most of the agricultural sprinklers are of slowly rotating impact drive sprinklers. They come in a variety of size and kinds ranging from small, single nozzle sprinklers, which are designed for operation at low pressure, to large multiple nozzle high-pressure sprinklers. The jet of water from the sprinkler sprays through a considerable distance. It is caused by the combination of pressure and slow reaction. The effective diameter of the area wetter by a sprinkler depends on the pressure at the nozzle and the angle at which the jet of water leaves it. The advantage of rotating sprinklers is that they can be adapted to all types of soils and crops.

High-pressure sprinklers are widely used. The characteristics of this type of sprinklers are: its wetting diameter ranges between 100 ft. and 300 ft. however, in high wind velocity, the distribution pattern gets disturbed. This sprinkler is suitable for tobaccos, track crops, corn, citrus and under tree crops, under low wind conditions. Large volume sprinkler are suitable to irrigate sugarcane, bananas and other dense tropical crops.

13.5 Types of Sprinkler System

Sprinkler system are classified according to the way the system is installed and operated. This includes whether the sprinkler head is operated individually or operated as a group; the sprinkler lateral is moved manually ; rotated round a central pivot or moved continuously along a canal or a ditch to irrigate big rectangular area or big areas by fixed system without moving it. Choice of a system and operation is dependent on the area to be irrigated, topography of the area and the source of water.

The system can be of the following types:

Permanent System
Semi-Permanent System
Portable System
Manually moved System
Mechanically Moved System
    Side Move System
    End Tow System
    Gun and Boom System
Continuous Move System
Central-Pivot System
Traveling Gun System
Traveling Lateral System
Semi-Permanent System
    Sprinkler Hop System
    Pipe System
    Hose Pull System

Permanent System

Permanent system refers to the system in which either the pipe works are permanently buried below the land surface with only sprinklers and portions of risers lift above the grounds, or the system in which pipes are laid above the ground on the field at the start of the irrigation season and then left there during the entire season. At the end of the season whole equipment is rolled up and stored. In buried system, asbestos cement , plastic and coated steel pipes are used. These systems are used in citrus plantations, orchards and other field crops. Such systems require heavy initial cost but require only a few skilled operators to irrigate even large areas.

Semi-Permanent System

A semi-permanent or semi-portable sprinkler system is the most widely used at present. It has the advantage of both portable and permanent equipment. Most semi-permanent system have permanent main line and sub-mains with portable sprinkler laterals . such system are used for irrigating field, fruit plantations and vegetable gardens. These are of the following types:

i. Sprinkler Hop System
ii. Hose Pull System

Sprinkler Hop System

In this system, sprinklers are placed only at alternate positions along the lateral. After irrigation of an area has been completed, the sprinklers are shut and new set of sprinklers are started to operate. The sprinklers can be operated and shut automatically. Since, this system can be operated automatically, it is suitable where night irrigation is required.

Hose Pull System

In this system both the mainline and laterals are permanently installed either underground or on ground surface. Small plastic hoses are connected to the laterals to supply irrigation water to one or two rotary sprinklers. Usually, the maximum the hose length used in this type is about 150 ft. to avoid higher friction losses in the pipe. The two sprinklers are placed between the tree rows and irrigate for the whole day. Plastic hoses reduce the number of laterals and provide flexibility in operation. The only drawback is that due to long exposure of the pipelines to bright sunlight or rough handling of plastic pipes, these pipes can get damaged, which means extra expenditure.

Portable System

A portable system usually has portable mains, portable laterals and sometimes-portable pump. They are either manually moved system or mechanically moved system. Gated pipe system is an example of manually moved system. The mechanically moved system works best on level or fairly uniform sloping fields. This type has lowest per hectare cost of any sprinkler system although the labour and operating costs are higher than other types of the system. Portable systems are:

- Manually Moved System
- Gated Pipe System
- Mechanically Moved System
- Slide Roll System and End Tow System

Side Roll System

A side or wheel moved system has wheels mounted on the laterals pipes, with the pipe serving as the axis of the wheel. The laterals are usually 30 to 40ft. in diameter aluminum pipes. The laterals as a unit can be moved either manually or by tractors. The wheel diameters are usually 3-to5 ft. so that the pipe remains well above the crops. The sprinklers are usually spaced 40 to 50 ft. this system is best suited to rectangular fields. It is suitable for close growing crops and low growing row crops and small grains

End Tow System

Usually a pair of wheels mounted on a simple U-fame is clamped to each section of pipe and pulled by tractor to the new position. The system is well suited to row crops. It is the last expansive of the mechanically moved system. The system is, however, not preferred because it gets damaged during towing.

Continuous Move System

This system consists of a central pivot in the form of a frame which supports a continuously rotating lateral. The fixed end of the lateral called the 'pivot point' is connected to the water supply. The lateral consists of a series of spans, ranging in length from 10 to 200 ft. and is supported at a height of about 10 ft. above the ground level.
Generally, the system is designed to complete the full revolution in 24 hours. Central pivot system is best suited to sandy soils, which absorbs water quickly and can support the heavy wheel loads from the towers. One major constraint is that the system irrigates circular areas. The corners of the fields are left unirrigated , which have to be irrigated by other means. The system is also very expensive and requires full workshop facility on the field for repairs.

13.6 Application Rate of Sprinkler

$$Ar = 60\ Q/a$$

Ar = Application rate of sprinkler in mm/h
And a = wetted area of the sprinkler in m^2 and Q is the sprinkler discharge in L/min.
For set move or permanent system
The application rate is given by the equation

$$Ar = 60\ Q/LS$$

L = the distance between the lateral in m and
S = spacing between adjacent sprinkler lines in m^2
For the design example of a gun sprinkler system refer to Annexure F.

14 TRICKLE IRRIGATION SYSTEM

Trickle irrigation is a pressurized irrigation system, which is widely practiced in several parts of the world. In this method, water is applied slowly and frequently to the base or root zone of each plant through a network of small diameter plastic pipes and mechanical devices, called emitters. The method is suitable in water short areas specially arid or semi-arid areas and is particularly used for orchards and vegetables.

14.1 Advantages of Trickle Irrigation System

The advantages of the system are:

- Suitable for undulating terrain,
- Less labour requirement and lower operation cost,
- Very high efficiency of water use,
- Satisfactory use of poor quality water,
- Higher efficiency in the use of fertilizers,
- Maximum crop yield,
- Decreased tillage,
- Improved plant protection and less disease ,
- High quality produce,
- Less weed growth, and
- Low-pressure requirement, etc.

Limitation of Trickle Irrigation System

- Small diameter pipes and emitters get clogged frequently.
- High initial cost to be borne by the farmer as compared to surface irrigation system.
- High skill required to design install and operate the system.
- Components of Trickle Irrigation System
- Drip irrigation system basically consist of a pump and overhead tank system , a network of pipelines of various sizes, and emitters (or drippers, tricklers etc.).

Pump and Overhead Tank System

Water from the source, which can be a well, canal, river, reservoir etc, is pumped into an overhead tank of a suitable size by pump. Usually, centrifugal pump is used to pump water to the tank. The tank is placed at a suitable elevation so that required is maintained in the pipeline s and emitters.

14.2 Components of Trickle Irrigation System

i. Main Pipe

The pipeline through which water flows from the tank to the system onto the field is called main pipeline. The main pipeline is usually buried underground and is generally made of rigid material like steel, cement concrete of asbestos cement. It supplies water

from the source to the main control point at the field. Flow through the main pipe is controlled by valves, discharge and pressure meters, automation equipment and controls, filters and dissolved fertilizer applicators. All these are called control heads. If the field/farm is not very big, one such control head controls the entire farm operation. If the farm is big, then more than one and sometimes several such control heads are provided.

ii. Sub Main (Secondary) Pipe

From the main pipe several sub-mains take off to distribute water to various parts of the field or sub-units of the farm. A valve on the sub main controls each sub-unit. Sub-mains are generally, rigid black polyethylene pipe usually laid on the ground. Diameter of the sub mains usually lie between 1.5 to 3.5 inches with the pressure rating of 4 ATM.

Manifold (or Header) Pipe

Manifold pipe is a rigid on flexible pipe generally of 1 to 3 inches in diameter. It distributes water from sub mains to a number of laterals. A single valve controls the flow.

Lateral (Pipe)

Laterals supply water to the plants through emitter. Lateral is flexible (soft) polyethylene or PVC pipe laid on top of the ground and carry emitters. The diameter of laterals usually lies between ½ to 1 inch. A typical drip irrigation system may gave length upto3, 000 ft./ha. Of laterals. Laterals lines are usually placed one per tree or plant row.

Emitter (Trickle)

An emitter discharges a small uniform flow of water drop by drop at a constant rate. Ideally emitter should have relatively large flow cross section so that along with supply so required amount of water at slow rate at atmospheric pressure, it should have capability to flush or clean itself. Emitters should be inexpensive compact and capable of self cleansing or flushing. Emitters are classified according t the mechanism, each emitter uses to dissipate pressure. They are long path emitters, orifice emitters, vortex emitters, flushing emitters, compensating emitters, multi outlet emitters, etc.

14.3 Design Criteria of Drip Irrigation System

The most import component of a drip system is emitter. The efficiency of the system depends on the emitter selection. Therefore, the first step in the design procedure is the emitter selection. In a typical system, the number of emitters lies between 100and 1000 per hectare. They should be:

- Capable of supplying water at low flow rates at atmospheric pressure
- Capable of wetting large percentage of area
- Low in cost
- Capable of self cleansing
- Simple in design, installation and maintenance
- Less sensitive to line variations

Component		Main parts	Function
		Fertilizer tank with its regulator.	To add fertilizer to irrigation water and to mix it with water. The inlet of water in the fertilizer tank is extended upto the bottom of the tank so as to stir the solution well. The fertilizer dispenser injects fixed amount of fertilizer to the tank, according to the need of the corp.
		Venturi unit	To create a pressure drop at the constrained section of the throat in order to drain fertilizer solution and irrigation water to the heart.
2. Heart		Filter tank	The fertilizer enriched water is led by means of tubing from the head to the filter tank. This supplies the dirt free waster to the tail.
3. Tail	(i)	Main line	The main line is 2 to 4 cm in diameter tubing and conveys water from the heart to the laterals.
	(ii)	Laterals.	The laterals are plastic pipes which are fitted to the main pipe line.
	(iii)	Nozzles	Nozzles are provided to the laterals to regulate the supply of water to the plants; the water trickles drop by drop. Each nozzle discharges approximately the same amount of water.

Description of Drip Irrigation System

As the very name indicates, it is the method of applying water in the form of drops directly to the plants. A typical drip irrigation system is shown in Figs. 14.1 and 14.2. A head is installed at each plot. The head tank is kept outside the field at an elevated platform to maintain a pressure head of 3 to 5 m. the location of the tank should be such that the main pipe is conveniently taken in the middle of the field (See Fig. 14.2).The size

of the tank depends upon the area of the field to be irrigated in one filling of the tank, and normally it may be a cube of side 3 meters.

The water conduction lines (main and the laterals), in drip irrigation system, are generally made of plastic materials. The basic considerations in favor of plastic conduits are:

- Cheapness.
- Marked resistance of plastic materials to chemicals.
- Resistant to sunlight.
- Black texture to avoid algae growth.
- Resistant to insects.
- Standard dimensions under a variety of climatic conditions.

Poly-Vinyl Chloride (PVC) and poly propylene are the two most common types of tubing used in drip irrigation. PVC conduits are suitable for buried water conduction

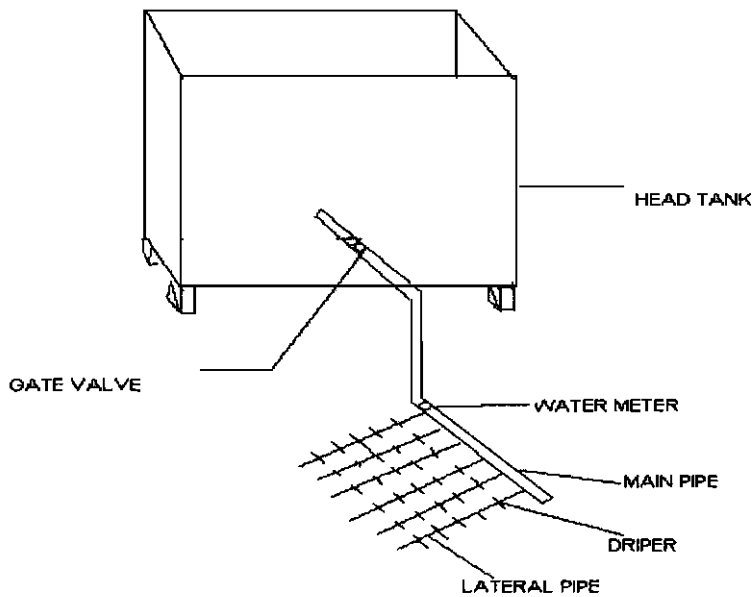


Figure 14.1 Components of a trickle irrigation system

lines. PVC and polyethylene are, in general, suitable for the main and lateral or trickle lines and polypropylene for the nozzles. The pipes are so dimensioned as to make it possible to join them by inserting them, one into another, without special connections.

The main line is 2 to 4 cm in diameter and the laterals, 0-5 to 2 cm. the laterals are fitted to the main pipe line at a distance equal to row to row spacing of a particular crop and it is 75 cm for equal to row to row spacing of a particular crop and it is 75 cm for most of the corps. The length and diameter of the laterals is generally an economic consideration. Usual diameter is 12 mm to 16 mm and length 40 m. A 12 mm diameter lateral of 40 m length is sufficient considering a pressure loss of 10 per cent and nozzles discharging 4.5 litres per hour spaced one m along the laterals. A 16 mm diameter tubing of 40m length is essential with nozzles discharging 10 litres per hour. The laterals can be adjusted in the field to run along the row corps irrespective of the variable spacing in between.

Holes are provided n the laterals at a distance depending upon the spacing of the crops. The holes are sometimes enveloped with sockets in order to avoid direct spouting because of which the water would be wasted away from the plant root zone. Emitter tubes, instead of holes, are provided in some designs for obtaining uniform flow distribution throughout the system. The discharge through the outlet openings would be 2 to 10 litres per hour depending upon the crop requirements. Nozzles are also sometimes provided at the end of the emitter tubes to alter the discharge. Two mm diameter nozzle is considered suitable. The nozzle is designed to supply the desired quantity of water to each plant, without getting clogged. There are three typical designs of the nozzles.

Type A. It is a molded plastic nozzle consisting of a 5 cm long cylindrical tube containing a threaded core 1 mm vide and 2 mm long threads. The movement of water along the elongated path results in a reduction of pressure and the water trickles out of the nozzle at a rate of 4.5 litres per hour.

Type B. This plastic nozzle consists of three parts. The central component is a cylinder 2 to 3 cm in length. At its upper end a 'heart' is threaded and the space between the external thread of the 'heart' and the internal thread of the cylinder forms a spiral pathway through which the water flows. As the pathway is relatively shorter than type A nozzle, the discharge is more ie. 9 litres per hour. The lower part of the nozzle serves for attaching it to the lateral line.

Type C. It is a short plastic nozzle with a heart similar to the one in type B, but the path for water is a narrow alit along the heart. The discharge is about 11 litres per hour.

The spacing between the two consecutive nozzles in governed by the following factors:

- Type of crop.
- Growth stage of the crops.
- Type of soil.
- Crop row interval or planting distance ; and
- Agro-technical practices (cultivation etc.) and economic aspects (equipment cost).

In the case of close spaced crops, the nozzles are located at a close interval of 0.5 to 1 m. In the case of fruit trees or plantation, high discharge nozzles, three or four in number are located on each tree. If three in number, one is adjacent to the tree and the other two at 0.5 m before and behind the tree. Four nozzles of 4.5 litres per hour capacity may be provided at 0.5 m on the four sides of the providing a spacing of 1 to 1.5 .

Advantages and limitations of drip irrigation system.

Advantages

Drip irrigation has number of advantages over the other methods ; the important ones are discussed below:

(1) ***Water saving.*** Drip irrigation is a method of watering plants frequently and with volumes of water approaching the consumptive use of the plants. Thus it minimizes such conventional losses as deep percolation, run off or conveyance loss and soil-water evaporation. Evaporation low which occurs during the application of water in sprinkler irrigation cannot occur in this method. Even the weed growth is less because the soil between the crop plants is dry. These conventional losses may amount to 75 pre cent of the water diverted to the field. This amount of water is saved in the only and in quantity which is barely needed by the plants for their growth or consumptive use. Thus, on an average, the water required by drip irrigation is only 20 to 30 per cent of that required by conventional methods like flooding and furrow methods and there is net saving of 70 to 80 per cent. The saving for sugarcane crop is about 40 to 50 per cent. The saving for this corp is less because water has to be applied along the entire length of the row of crops instead of at selective points in other crops.

(2) ***Uniformity of irrigation.*** The uniformity of irrigation, in this method, is very high i.e. of the order of 90 per cent as compared to other irrigation methods. In wild flooding or furrow flowing, uniformity can be as low as 25 per cent and in sprinkles irrigation; it may be upto 50 per cent. Drip irrigation avoids over-irrigation that needs to be given in other methods in order to provide sufficient moisture even at the deficient area.

(3) ***Crop yields.*** This system gives higher yield of crops such as potato, beet root, chilly, lady fingers etc. the yield is more by about 15 to 30 per cent than by the control methods. For other crops, the yield is at least as good as in the other methods of irrigation. Drip irrigation promotes faster establishment of young trees advancing bearing age of the plants, aid sound germination of seeds increasing the population per unit area, and prevents thicker growth of weeds reducing the sharing problem of soil nutrients and moisture. These conditions are probably responsible for higher yield.

(4) ***Soil and water salinity.*** The suitability of agriculture in not merely based on the soil salinity or salts present in the irrigation water; rather it is the relative concentration of salts that is the deciding factor for their suitability. Since moisture is always present in the root zone more or less at the field capacity, in drip irrigation the relative concentration

will be less. But in other methods of irrigation, the soil moisture will deplete even by 50 per cent before the next irrigation is given and hence the salt concentration will be more.

Thus the waters or soils, containing salts, which shall have to be declared unsuitable for agriculture with other methods of irrigation, can be utilized for crop growth when irrigated be drip, method.

(5) ***Labour cost.*** The labour requirement is more that in fixed sprinkler system. However it is less when compared to the movable sprinkler system, basin flooding or furrow flowing ; because the drip method does not require grading or leveling of the land and making of channels or furrows, which are essential in other methods of surface irrigation.

A more detailed comparison between furrow irrigation, sprinkler irrigation and drip irrigation methods is given in Table 14.1.

Limitations

Like any other system, drip irrigation method has the following limitations:

(1) *Application of fertilizers* : Application of insoluble or slightly soluble fertilizers, such as super phosphate or sulphate of potash, is not readily possible through the flow system in drip irrigation. These have to be applied on the surface, which is satisfactory.

(2) *Accumulation of salts* : Use of highly saline water or on soils containing high levels of salts, there will be an accumulation of

Table 14.1 Comparison of Furrow, Sprinkler and Drip Irrigation Methods

S. No.	Characteristie	Furrow irrigation	Sprinkler irrigation	Drip irrigation
1.	Water requirement	Higher water requirement as flow in furrows entail considerable water losses to the extent of 55 per cent, occur.	Low water requirement, sixty-five per cent of that required in surface irrigation methods. Total loss is estimated to be about 18 per cent.	Least water requirement, since water delivered directly to the plant. Water consumption is about 20-30 per cent of the conventional methods.
2.	Evaporation loss	Less water lost by evaporation.	Higher evaporation loss.	Evaporation loss almost negligible.
3.	Suitability for soils	Suitable for all soils except for very light soils having high permeability.	Suitable for all types of soils, especially for shallow coarse textured, highly permeable soils requiring light irrigation at frequent intervals.	Suitable for all types of soils, especially for coarse textured soils with scarce water resources. Crops can be grown over saline soils also.
4.	Soil surface wetting	Wets 20-50 per cent of irrigated soil surface.	Entire surface area is wetted.	Least wetting of soil surface due to direct water supply to the plants.
5.	Surface temperature and its effect on crop period.	Soil temperature is reduced owing to flooding of entire furrows line. This delays the maturity of crop and hence the crop and hence the crop period is extended.	Entire surface temperature is reduced, resulting in comparatively longer crop period due to delayed maturity.	Surface temperature is kept high, the moisture availability hewing restricted to root zone. This hastens the ripening or maturity of the produce and hence the crop period is reduced.
6.	Rate of growth	Comparatively low rate of vegetative growth.	Better and quicker germination of seeds. Plants grow more vigorously. Improved response of crops as compared to surface irrigation.	Highest rat of vegetative growth. Much quicker germination of seeds and rate of plant development..
7.	Suitability for crops	Suitable for all cultivated crops planted in rows and trunk crops. Not suitable for shallow rooted crops requiring light irrigation. Also not suitable for seed germination.	Widespread application. Especially suitable for tea and coffee gardens, young seedlings and plants.	Suitable for close growing corps such as vegetables and also for orchards.
8.	Adjustment of water application	Regulation of water application is almost absent.	Sprinkler pressure controls the amount of water applied and the rate of rotation of the sprinkler.	Adjustment of water application very easy by using nozzles of different sizes.
9.	Automation of water supply	Automation is absent. Higher labour cost is involved in guiding and distributing water to the furrows.	Complete automation is rare.	Economical and convenient water control. Automatic valve at head terminates supply after the desired supply is discharged. Water motor is also installed in the head to know the discharge.

Sr.No	Characteristic	Furrow irrigation	Sprinkler irrigation	Drip irrigation
10.	Uniformity of irrigation	Non-uniform distribution of water along the length of the furrow. The uniformity of irrigation may be as low as 25 per cent. Natural conditions such as texture and structure of the soil are the deciding factors of the uniformity over which the irrigation is has no or little control.	The uniformity of irrigation may be about 50 per cent. Intensity of wind is the controlling factor.	The uniformity of irrigation in the drip irrigation is above 90 per cent.
11.	Fertilizer application	Localized application of fertilizer is absent.	Fertilizer is applied through irrigation water by means of fertilizer applicator. Localized application is absent.	Fertilizer application is localized, as it is delivered to the plant root only through irrigation water. Thus the fertilizer application is best regulated, besides minimizing its requirement; however application of insoluble and slightly soluble fertilizer such as super phosphate or sulphate of potash through the flow is not readily possible in sprinkler and drip irrigation systems.
12.	System and components parts	Furrows change with the type of crops grown. Furrows made on gradient involve skill and labour. However no piping system is involved.	Sprinkling system may be stationary or portable. The system consists of main pipes and laterals besides risers. It has movable components like nozzles, etc. The risers constitute obstruction to cultivation operations. However the spacing of laterals and nozzles is easy to vary.	The drip irrigation system is generally stationary. The pipe lines are made from plastic materials and thus cheaper. The layout is free from any movable component. The spacing of laterals and nozzles is easy to vary to suit crop and soil requirement. Risers are absent and as such there is no obstruction to cultivation operations by machinery.
13.	Effect of wind velocity	In level furrow method, strong wind (10 to 12 km per hour), blowing in opposite direction interferes with the application of irrigation water.	Strong wind interferes with application of water, particularly when it flows in the opposite direction. High wind velocity also causes high evapo-transpiration.	Not much affected by wind conditions.
14.	Soil erosion	Soil erosion near upper end of the field results in furrows of long length.	Soils erosion is controlled.	No soil or tail water loss.

Sr.No	Characteristic	Furrow irrigation	Sprinkler irrigation	Drip irrigation
15.	Weed control	Wild weed and pest growth.	Weed and pest growth, and diseases are less. Cost of their removal is less.	Weed, pest and insect troubles are least since only selected points on the land are dry. Hence the problems of weed eontrol and the preservation of soil nutrients for the erops are easy.
16.	Percolation of water and leaching of salts and nutrients	Percolation losses arc very high, especially in sandy soils. Thus the salt nutrients and fertilizers are leached down to greater depths and are not available for plant growth.	Percolation of water and leaching down of salts, fertilizers etc. is less. Leaching takes place in basic soils only.	Minimum pereolation reduces leaching down of fertilizers and nutrients.
17.	Land leveling requirement	Leveling of fields is very essential in straight graded furrows.	No land leveling is required.	Land leveling is not required.
18.	Moisture distribution	Moisture distribution is normal to the furrow shape.	Moisture distribution is two dimensional.	Moisture distribution is two dimensional for soils of low permeability and for soils with an impervious subsoil layer. The moisture distribution in soils of high permeability (sandy soils) is mainly vertical i.e. unidirectional.
19.	Land utilization	About 7.5 per cent of land is lost to field channels.	More land is available for cultivation as water courses are not required.	Makes available maximum land for cultivation.
20.	Crop yield	The yield is less as compared to other methods.	Higher yield, though less than drip irrigation system.	Higher yield if crops in this system are possible such as tomato, beet root, lady finger, etc. and for other corps, the yield will be at least as good as in other methods of irrigation.
21.	Cost	Lowest initial investment but high operation labour charges.	High initial cost and operation charges. Steel or light weight aluminum pipes are costly as they have to withstand high operating pumping pressures.	Low initial and operating charges as compared to sprinkler irrigation. However the plastic pipes have low durability and therefore need frequent replacement (4.5 yards)

salts at the edge of the wetted zone (See Fig. 14.2). If the system is not in use and there is a heavy rainfall, there is a possibility of water moving backward towards the dripper. This will push the accumulated salts at the edge of the wetted zone towards the rood zone and the crop would be badly affected. A practical remedy for this problem is to operate the drip system even during rain.

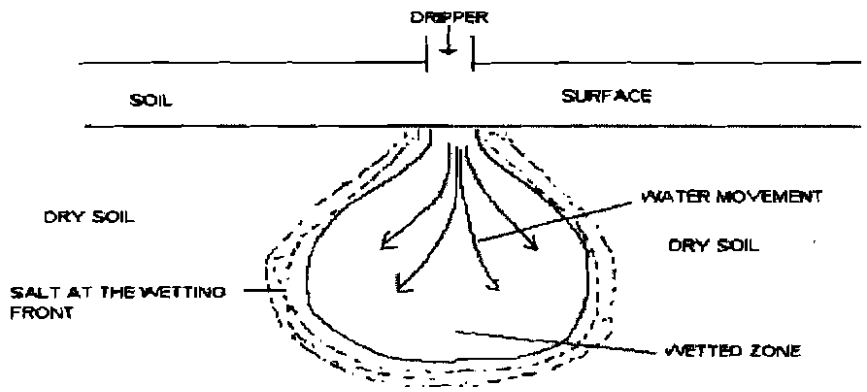


Figure 14.2 Salt accumulation .

3) *Dripper or nozzle blockage*: This is a major problem is drip irrigation system. The diameter of the nozzles or outlet holes is about 0.5 to 2 mm and there could be blockade due to soil or other materials. The remedy is to filter the water and use sediment free water. The holes can also be covered with sockets.

(4) *Shallow root zones*: Plantation or trees grown with drip irrigation may develop shallow confined root zones. Therefore they can extract nutrients from less area and the anchorage of the roots will be weak. Thus there is a possibility of toppling of the trees under windy weather. The remedy lies in supplementing these trees with fertilizers. To minims the toppling of the trees, the drippers should be placed each side of the butt at some distance rather than at the base of the tree.

(5) *High cost*: The cost of the drip irrigation system, which includes mainly the purchase of P.V.C or polyethylene or alkaethylene pipes is about Rs.10,000 to Rs.12,500 per

hectare. These pipes are serviceable for about 5 years. Thus there is an annual expenditure of Rs.2,000 to Rs.2,500 per hectare towards the cost of the pipes. Due to such high cost, drip irrigation system may not get much appreciation from the farmers. The remedy lies in subsidizing the cost of pipes or by effecting from a higher income that would result, from raising cash crops like cotton, tobacco and banana.

The cost can also be reduced further by catering to the adjacent area to the unit by rotating the main line through 180 degrees.

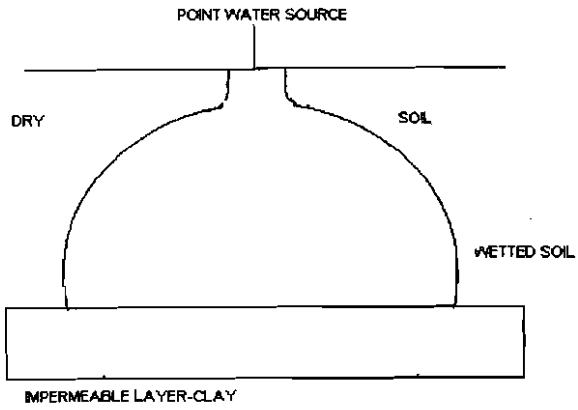
In the case of orchards, the initial cost and operation charges are lesser with wider spacing than with fixed sprinkler system.

Suitability of Drip Irrigation

Drip irrigation is suitable for almost all row crops and orchards. It is also adaptable for all types of soils and even to highly sloping terrains.

Crops which can be raised: Row crops like tomato, chilly, brinjal, radish, cauliflower, beet root, lady fingers, sugarcane, banana, grapes, tobacco etc. can be grown with drip irrigation method. Of course, paddy crop can not be cultivated with this method since it requires standing water.

Type of soils: This method can be adapted to highly permeable soils like sandy soils, sandy loam, gravelly silty clay or to shallow soils underlain with impermeable strata. Irrigation is applied at shorter intervals in sandy soils since its water bolding capacity is low. The water travels vertically much faster in this type of soil and the soil wetting pattern is elongated (see Fig 14.3). In clayey soil, the wetting front is two dimensional i.e. it advances equally in both directions. In soils underlain with impermeable layer, water is forced to travel laterally due to the obstruction. Different wetting patterns in these types of soils are shown in Fig. 14.3.



SOIL WITH AN IMPERMEABLE SUB-SOIL LAYER

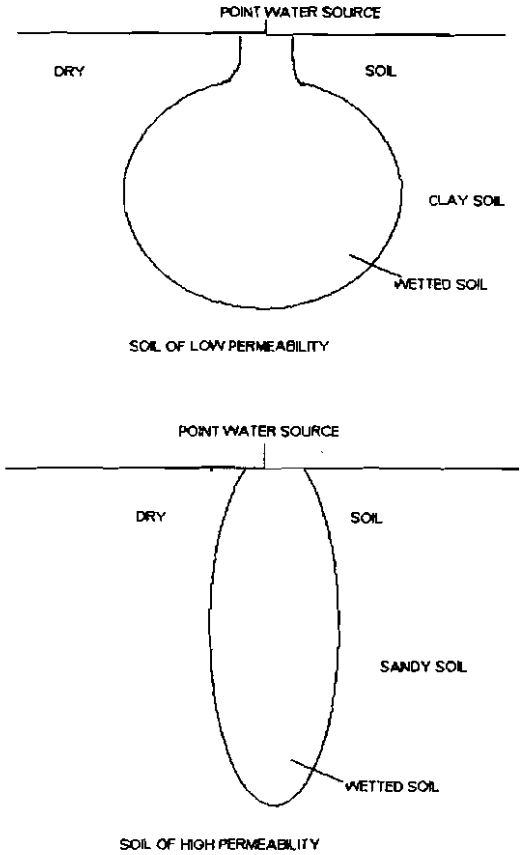


Figure 14.3 Wetting pattern in different types of soils.

Types of terrain: Drip irrigation is suitable for all types of terrains; flat, plateau and steep. In hilly terrains, irrigation water can be applied to the plants without causing wasteful run-off or destructive erosion. This method is ideally suited to hilly areas with thin layer of soil because the plant develops its root system in the top soil only, when irrigated by this method.

Water conduction in the tubing network is as simple computation as a simple water supply distribution system.

The flow in the main pipe line is considered to be laminar flow or in the transition zone between the laminar and a turbulent flow i.e. functioning as a pipe with a smooth surface and the flow having a Reynolds number less than 2000. Darcy Weisbach formula bolds good for the hydraulic computation of the pipeline.

By adjusting the number of drops per minute, the number of hours necessary to discharge the required litres of water needed for each plant is determined.

As an alternative, the flow pressure a available at a hole in the pipe line of the drip system can be calculated as excess of the head loss that has occurred due to friction. The discharge through the hole or nozzle is computed assuming it as an orifice. Knowing the number of nozzles or holes in a lateral and the number of laterals in the main pipe line, the discharge to be allowed through the control valve in determined. Then from the discharge and water requirement of the crop, the duration of irrigation can be worked out.

For optimum yield of the crop, uniform application of water at the optimum level all over the field is necessary. Uniformity can be achieved by the following methods:

(1) *By adjusting the emitters by trial and error.* Adjustment of one emitter upsets another and in practice, it is very laborious especially when there are large numbers of outlets.

(2) *By doubling the pipes*: Smaller inner tubes having orifices at larger intervals are enclosed in outer tubes with orifices at closer intervals. This system provides better uniformity, but is costlier.

(3) *By changing the grades*: To get constant discharge, the pressure or head causing flow should be the same fro a given size of the opening. Thus the loss of pressure in the downstream direction can be compensated by a falling grade. This method also can not be readily adopted in practice.

(4) *By varying the size of the outlet opening*: To get a constant discharge, the size of the opening should be increased towards the downstream side; if the openings are spaced at equal intervals. The diameter of the opening can be worked out from the Darcy-Weisbach equation. This is also not practicable.

(5) *By varying the size and spacing of the opening*: The head loss increase with the increase in the length of the pipe. If the outlet openings are provided at closer intervals towards the downstream direction, then the rate of head loss per unit length would be lower and hence the rate of diminution of the residual pressure also would be lower. This means that the size of the opening should be increased less abruptly for obtaining the same discharge in all the openings.

(6) *By varying the size of the pipe*: Determination of size of each opening is obviously a tedious process and there is difficulty in manufacture.

In practice, instead of changing the size of each and every opening for maintaining a constant discharge, a series of openings in a segment of length can be considered as a unit. Using the average velocity in that pipe length, the size of the opening is determined and the same size opening is adopted in that segment. This procedure is applied for different segments of the entire pipe length. The above approximate design is found to introduce a maximum error of 10 per cent which is satisfactory for all practical purposes. Thus a uniformity of flow distribution above 90 per cent can be achieved in drip irrigation.

For design example of a trickle irrigation system, refer to Annexure F.

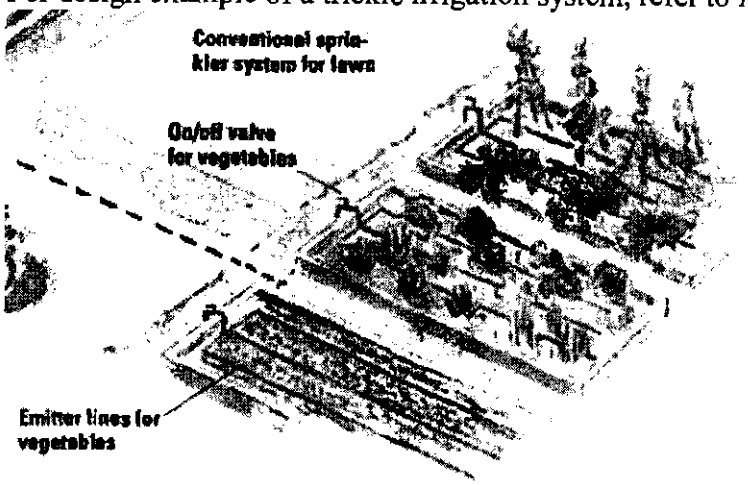


Figure 14.4. An example emitter type drip irrigation system

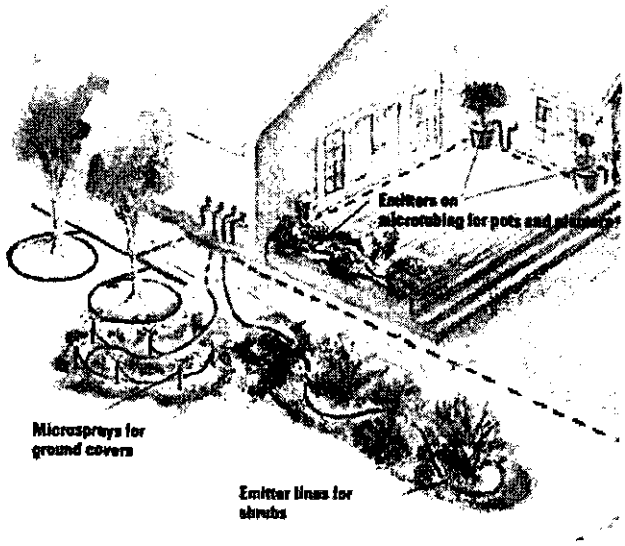


Figure 14.5. An example micro spray type drip irrigation system

15 GROUND WATER FOR IRRIGATION

Ground water is an important source for irrigation. A physiographic unit containing one large permeable formation or several connected or interrelated formations, which contain water and called as ground water basin. Basin boundaries may be natural streams, rivers, mountain ranges etc. In the present days, basin wide planning and development of ground water for irrigation has assumed special importance since over development of ground water in one part of the basin may affect the whole basin adversely.

Definitions
Some of the terms commonly used in connection with ground water are defined below:

Aquifer: These are defined as permeable formations, which contain and permit appreciable movement of water through them under ordinary field conditions. They are mostly sand and gravel formations.

Aquiclude: The formations which though contain water yet are incapable of transmitting it through them, are called aquiclude. Clay formations are the example.

Acquitard: The saturated bed formations which yield very small quantity of water but through which appreciable leakage of water is possible are called acquitard.

Aquifuge:The formations which neither contain water in them nor permit transmission of water through them are called aquifuge. Rock formations are the example of aquifuge.

Type of Acquifers: Aquifers are mainly of two types, viz:

a. Unconfined aquifers
b. Confined aquifers

a.Unconfined Aquifers
The top most water bearing strata, which has no confining impervious layer at the top is called unconfined aquifer. The water table in this aquifer may have different depths below the natural surface at different places.

In certain locations in unconfined aquifers, due to existence of impervious layer of small a real extent, water gets trapped above it. The upper most surface of this trapped water is called perched water table. The level of the perched water is always above the water table in the aquifer. If the perched water finds an outlet at the lower end, it may flow through it and is called spring. The discharge of spring rises and falls seasonally. In dry season it reduces considerably.

b.Confined Aquifer

When an aquifer is confined on the top by an impervious layer or by an aquiclude. It is called a confined aquifer. The water in this aquifer remains under pressure at all points. These are analogous to pipe lines flowing under pressure at all points. Since water is under pressure, there is no water table in the confined aquifer.

In conditions as depicted below, if a hole is drilled up to the confined aquifer, water will rise corresponding to the pressure in the confined aquifer. At a place where the pressure is more than the depth of the over burden, water will start flowing through the hole under pressure. These conditions are called artesian conditions and the aquifer is some time called as artesian aquifer.

15.1 Methods of Extracting Ground water

The various methods through which groundwater is extracted for irrigation purposes are:

a. Karzes (Kanat)
b. Infiltration Galleries
c. Wells
   - Open Wells
   - Tube Wells

a. Karezes (Kanat) Irrigation

Karezes are one of the oldest method of extraction of ground water in certain parts of the world, specially Iran and Balochistan. Karez is a horizontal unlined tunnel excavated in permeable water bearing alluvial fans to collect and convey water to the fields. In Balochistan both in Pakistan and in Iran, alluvial fans are commonly found at the foothills. These fans are traditionally the major source of ground water for irrigation in those areas. The ground water from these fans is collected by excavating unlined tunnel through which water gets infiltrated and is conveyed to long distances through gravity. Since the tunnels are unlined, infiltration of water takes place from the whole section of the tunnel. The tunnel are provided with a slight longitudinal slopes so that they may permit flow of water in them by gravity. For construction and for access to the various sections of the tunnels, vertical shafts are excavated at regular intervals, all along there alignments. At the mouth of a karez, an open watercourse is usually constructed, which conveys water to the fields. Schematic cross sections of a typical karez and watercourse at its mouth is shown below.

b.Infiltration Galleries

An infiltration gallery is a horizontal permeable structure for intercepting and conveying water to the fields by gravity. This is one of the oldest forms of collection of water for irrigation and water supply. Such galleries had found wide spread use in Europe and USA. In Pakistan, such galleries were constructed at few places in Balochistan and NWFP. However, they have never been used on large scale as a major source of irrigation water.

A gallery is usually, constructed in horseshoe section and is located in a water bearing formation of a river or a stream. They are constructed along or across the the section of a perennial river. The invert level of the gallery is placed usually 10 to 15 ft. The galleries are backfilled to allow water to infiltrate into them. For collection of water in large quantities several lateral galleries may be laid perpendicular to the main gallery. The galleries both main and laterals, are laid with some horizontal slope so that the infiltrating water is conveyed through them by gravity. To excavate and to facilitate inspection of these gallery is kept such that velocity of flow in the gallery is kept around 3 ft./sec.

c. wells

Wells the major source of obtaining ground water for irrigation in Pakistan. Wells are hole dug in the ground with their beds well below the ground water level. Water from these wells is lifted either manly or by means of pumps. Wells can be open wells or tube wells. In the following chapter these are discussed in detail.

15.2 Types of Pumps

Centrifugal pumps

These pumps are the most important class of pumps at present. These are very extensively used as compared to other classes of pumps. The maximum suction had under which the pump can practically worked effectively as about 20-25 ft.

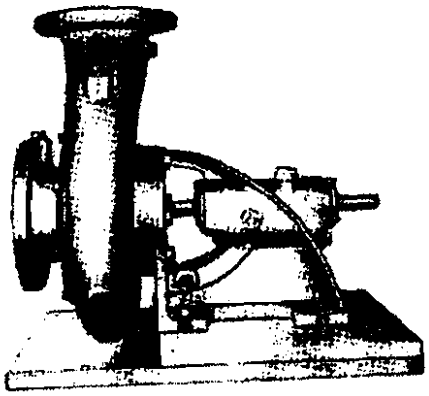


Figure 15.1 Centrifugal pump.

Turbine pump

These are used under situation when ground water is very deep, the installation of a conventional centrifugal pump in a sump becomes difficult and expensive therefore turbine pumps are used in such places. Such pumps are of very compact design and can be lowered in the casing and set at a place which is deeper than the expected depression of water during pumping. The top 50-100 feet of the bore hole and casing pipe is generally kept larger than the remaining bore, so as to accommodate the pump in the

casing itself. The usual sizes of the housing pipe range from 6-10 inches. A maximum number of 20 stages can be added to the pump. The pump is driven by vertical shaft type electric motor, coupled directly and placed to the top of the line shaft at the ground level.

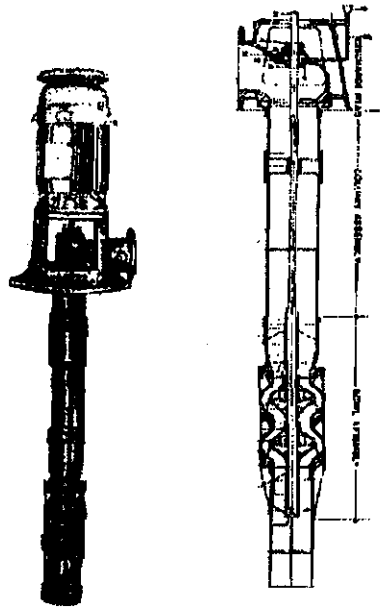


Figure 15.2. Turbine pump.

Submersible pump

A new type of centrifugal pump has been developed in which the motor is coupled directly to the impellor and the whole set as submerge in water. These are called submersible pump.The pump is a vertical centrifugal pump with radial are mixed flow impellors. All metallic bearings are water lubricated and protected against ingress of sand.

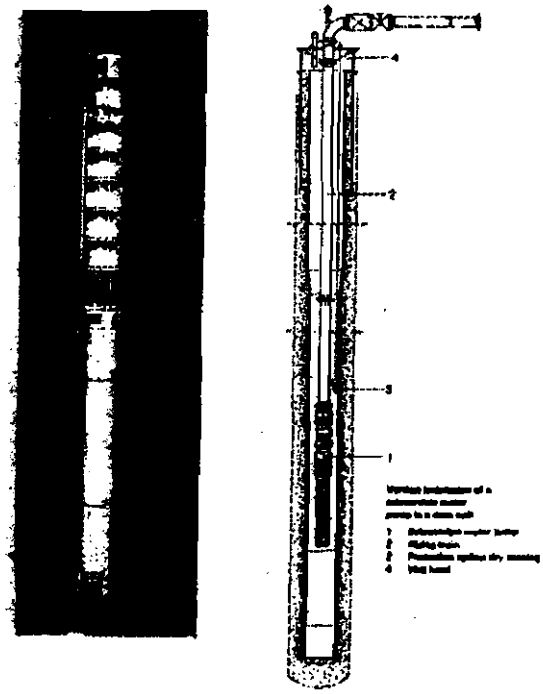


Figure 15.3. Submersible pump.

15.3 Pump Power Requirement

The unit of power is foot lbs per second. If a cubic foot of water weighing 62.5 lbs to be lifted through a height of 1 foot in 1 second, the power needed is 62.5 ft. lbs/second. The unit of power commonly in use is horse power, which is consumed while 550 pounds (Lbs) through one ft in 1 second or 33000 lbs through 1 ft in 1 minute. These are usually expressed 550 ft-lb/sec or 33000 ft-lb/minute. Thus a force of 1 horse power can lift 1 cubic ft of water through a height, h. the lift in feet:

$$H = \frac{\text{Horse Power}}{\text{Weight of 1 cubic ft of water}}$$

Or $$h = \frac{550}{62.5} = 8.8\,ft$$

Or $$1H.P = \frac{62.5X8.8}{550}$$

It means that when 1 cubic foot of water is lifted through 8.8 ft, the power consumed is 1 horse power. This question is used to determine the theoretical horse power (HP) to raise water to different heights. Based upon this question, the theoretical horse power required to raise water to different heights is given in the following table. It may however be noted that no machine is 100% efficient. There is always some loss of power, and for this reason the actual efficiency of a pumping set is used to determine its actual Horse Power.

Table 15.1: Horsepower required to lift a certain discharge.

Gallons per minute	Lift in feet								
	30	50	60	90	100	150	200	300	400
10	0.075	0.12	0.15	0.22	0.25	0.37	0.50	0.75	1.00
50	0.375	0.62	0.75	1.12	1.25	1.87	2.50	3.75	5.00
100	0.75	1.25	1.5	2.25	2.50	3.75	5.00	7.5	10.00
200	1.5	2.50	3.00	4.50	5.00	7.50	10.0	15.0	20.00
300	2.25	3.75	4.5	6.75	7.50	11.25	15.00	22.5	30.00
400	3.00	5.00	6.00	10.00	10.0	15.00	20.00	30.0	40.00
500	3.75	6.25	7.5	12.00	12.5	18.75	25.00	37.5	50.00

Efficiency of a Pumping Plant

If a pump is discharging Q cubic feet per second with a lift of h feet, then it is consuming a water horse power WHP, equal to

$$WH.P = \frac{62.5 * Q * h}{550} = \frac{Qh}{8.8}$$

For withdrawing a certain volume of water, a machine or a pump operated by some source of energy is needed. The actual power applied to the shaft to the pump is the break horse power (BHP). This is equal to

$$BHP = \frac{WHP}{PumpEfficiency}$$

or $$Efficiency = \frac{WHP}{BHP}$$

Head Loss in Pipes and Fittings

In order to determine the total head required to determine the required Horse power of a pump for a scheme the head losses in the pipes and other fittings such as valves, elbows

etc needs to be considered and added to the total required head in the above equations. The following table gives the head losses in meters, due to friction per 1000 m of 15 years old ordinary steel pipes.

Table 15.2 Head losses in meters, due to friction per 1000 m of 15 years old ordinary steel pipes

		Nominal Diameter of Pipe									
Rated Flow (m^3/hr)	L/s	¾ in. (1.90 cm)	1 in. (2.54 cm)	2 in. (5.08 cm)	2½ in. (6.35 cm)	3 in. (7.62 cm)	4 in. (10.61 cm)	5 in. (12.70 cm)	6 in. (15.24 cm)	8 in. (20.32 cm)	10 in. (25.40 cm)
0.25	0.069	10	2.4								
0.5	0.139	36	9								
0.75	0.208	82	18								
1.00	0.278	145	33	1.2	0.4						
1.5	0.417	300	68	2.5	0.8						
2	0.556		110	4.0	1.5	0.6	0.1				
3	0.834			8.9	3.0	1.2	0.3				
4	1.11			15	5.0	2.0	0.5	0.2			
5	1.39			22	7.5	3.0	0.7	0.3			
6	1.67			31	10	4.2	1.0	0.4			
8	1.95			54	18	7.3	1.8	0.6	0.3		
10	2.78			80	27	11	2.7	0.9	0.4		
12	3.34			110	32	15	3.8	1.1	0.5		
14	3.89			150	38	20	5.0	1.3	0.7	0.2	
16	4.45			190	65	26	6.5	2.2	0.9	0.2	
18	5.00			240	80	32	8.0	2.7	1.1	0.3	
20	5.56			265	95	38	9.0	3.2	1.3	0.3	
25	6.95			430	150	60	14	5.0	2.0	0.5	0.2
30	8.34					80	20	7.0	2.8	0.7	0.2
35	9.73					108	27	9.5	3.7	1.0	0.3
40	11.1						34	12	4.8	1.2	0.4
45	12.5						43	14	5.8	1.5	0.5
50	13.9						50	18	7.2	1.8	0.6
55	15.3						60	22	8.8	2.4	0.8
60	16.7						70	25	10	2.8	0.8
65	18.1						84	30	12	3.0	1.0
70	19.5						98	34	13	3.4	1.2
80	22.2							42	17	4.3	1.5
90	25.0							54	22	5.5	1.9
100	27.8							64	26	6.6	2.3

The following figure presents friction losses caused by elbows, valves and other fittings in pipe systems.

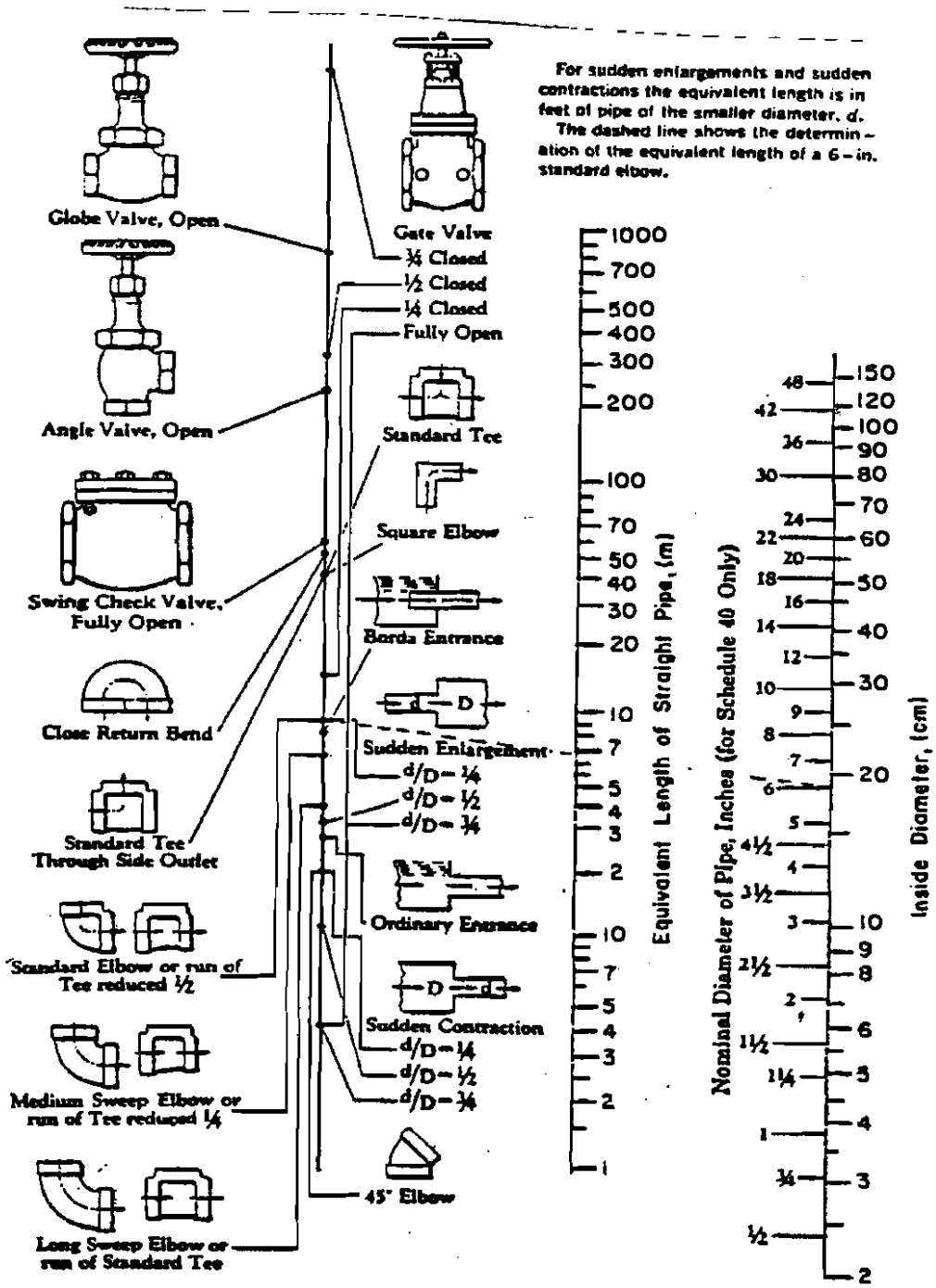


Figure 15.4. Head loss in pipe fittings

16 DESIGN OF DAMS

Dams form part of a controlled irrigation system but they also have other roles to play viz:

- Flood control
- Hydroelectric power generation
- Soil conservation
- As compared to a barrage which only diverts water into canals for irrigation. Other differences between a dam and a barrage are:
- A dam stores flood runoff whereas there is no storage reservoir in a barrage. The Chasma barrage on the Indus is an exception which stores 0.4 maf of water due to its specific location.

A dam is a high structure, whereas a barrage is a low structure in which the depth of water does not exceed 30-35 ft. (A barrage may be called a low diversion dam).
A dam requires a solid rock foundation (except earth dams) and dependable shoulders, whereas a barrage can be founded on permeable alluvium.

This chapter deals briefly with various types of storage dams and the requirements in site selection. A few examples of design are included at the end.

16.1 Classification of dams

Dams may be classified on the basis of the type of construction or construction material as follows.

Gravity Dam – It is a structure built of solid masonry or concrete which resists the water pressure and other imposed loadings mainly by its weight. It is usually built straight in plant or in a series of straight lines taking advantage of topography or it may sometimes be curved in plan and is called a curved gravity dam. Although this curvature offers extra strength the section is designed as for a straight gravity dam. Examples of straight gravity dams are Warsak dam in Pakistan, Grand Coulee in USA, and Bhakra dam in India. Rawal dam in Pakistan and Shasta dam in USA are examples of curved gravity dam.

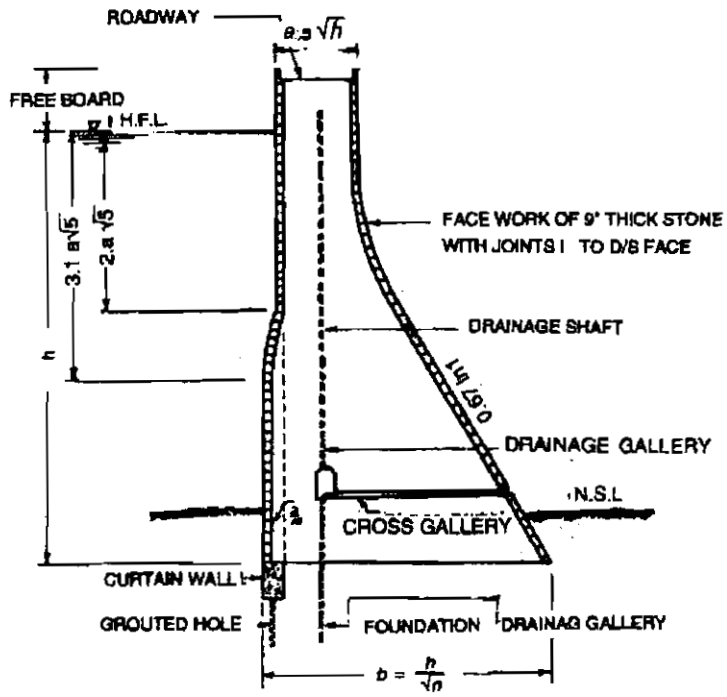


Figure 16.1 Cross section of gravity dam.

2. **Arch Dam**: It is a structure of solid concrete or masonry curved u/s in plan, which resists a small part of water load by its own weight, and by transmitting the remainder to the canyon walls by arch action. Hoover dam in USA is an example.

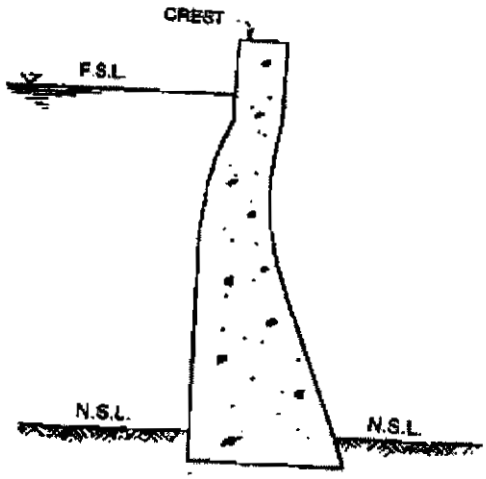


Figure 16.2 Arc dam.

3.**Buttrees Dams**: They consist of two structural elements:
The sloping deck which takes the water pressure.
The buttresses which support the deck slab.

They can be further classified according to the type of the deck slab:

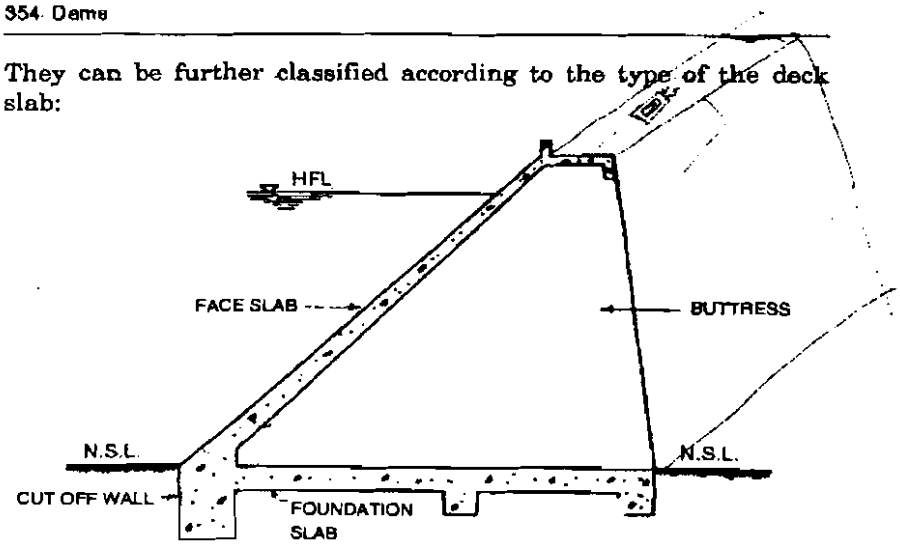


Figure 16.3 Buttress dam.

4.**Earth Dam:** It is a structure earth, graded and compacted to resist seepage and sliding. The line of seepage through the dam, also called the phreatic line, should not cut the d/s slope, otherwise this point of intersection becomes the point of infinite exit gradient and failure would start from this point. The phreatic line should cut the base of the dam and seepage may be lead out of the body of the dam through filters. They may be classified into the following types on the basis of material used:

Rock fill Dam
Rockfill, i.e. big size stones dumped in the form of a nearly trapezoidal profile
A thick layer (about 5 ft) of dry rubble masonry (hand packed) on the u/s slope to form the support forth concrete slab.
A concrete slab of a suitable thickness to rest on the rubber masonry with proper leak proof expansion joints to stop seepage.
A cutoff in continuation of the slab is provided to check any seepage through the foundation..

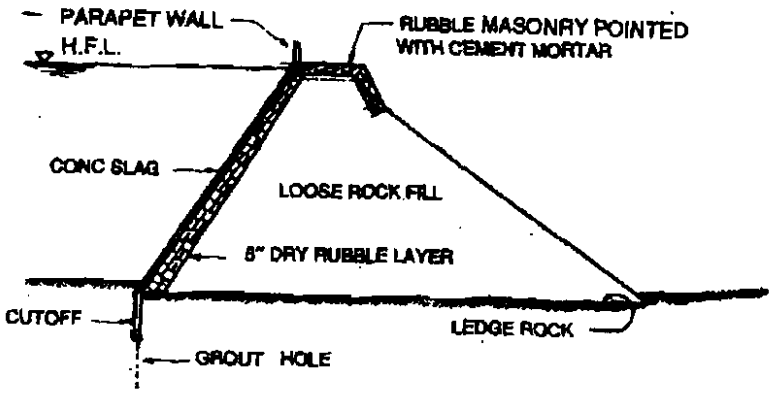


Figure 16.4: Rockfill dam

Timber or Steel dam

Usually these of lessor hight and are built of timber or steel. They are very similar buttress dam the only difference being that instead of RCC slab, a steel dick is used.

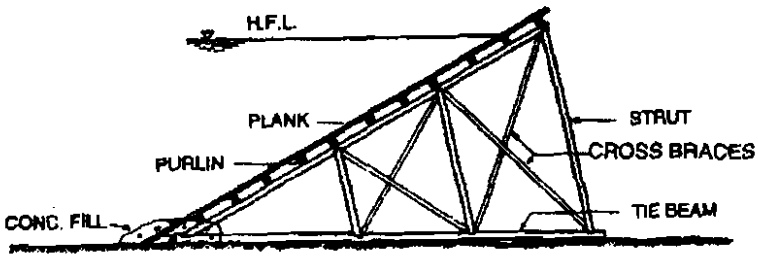


Figure 16.5: Buttress type Timber Dam

Factors in selecting the type of a dam:

The following factors may be kept in mind for the selection of the type of dam:

- Objectives
- Site conditions
- Hydraulic factors
- Climatic effects.
- Traffic requirements

Objectives: The following objectives should be kept in mind when selecting the type of dam:

- Safety
- Economy
- Efficiency
- Appearance

Site conditions: The following have to be looked into:

Foundation conditions. Collect data regarding permissible foundation stresses, percolation rates, excavation requirements, foundation settlements and the effectiveness of grouting. A hard rock foundation, without geological faults and which can be consolidated by grouting is fit for any type of dam and any height. For poorer foundation conditions, dams of lesser height of gravity or buttress type may be considered. Earth Dams may be built safely and economically on practically all types of foundations of earth or rock. Gravity and arch dams require sound foundations while an arch dam in addition requires strong abutments. Buttress dams may be built on foundations which are not suitable for gravity dams since slight settlement do not matter in their construction.

Topography- When the foundation are suitable for a gravity ore arch dam, the topography determines whether a straight gravity dam or one with a broken axis or a curved gravity dam or an arch dam is to be built. For arch dams the canyon should be well defined, the desirable ratio of crest length to the height of the dam being about 5:1.

Accessibility of dam site: Accessibility is important when selecting a type of dam dependent upon the availability of construction materials. If a railway or road has to be built then the cost of the dam will obviously increase.

Construction materials: If the construction materials say, aggregate, sand, stone, or suitable earth (clay) are available nearby, for concrete masonry and earth dams respectively then the cost will determine the type of the dam. If all these materials are to be hauled from long distances then a buttress dam may be more desirable as it requires less material than the other types.

Forces to be considered:
Before proceeding with the gravity method of analysis and determining the various stability factors, the following forces are taken into account for the non-overflow section:

- Weight of the dam
- Water pressure
- Uplift pressure
- Earthquake
- Temperature stresses
- Silt pressure
- Ice pressure
- Wave action

Silt pressure: The elevation of silt deposit due to silting up of the reservoir depends upon various factors like the watershed management and the amount of silt carried by the stream, as well as the pattern of the silt settlement in the reservoir and the existence of any reservoir u/s. Once the silt elevation (during the useful life of dam) is fixed in the

design, the silt pressure is computed by Rankin's formula for earth pressure.

$$P_s = \frac{\gamma_s \; h_s^2}{2} \left(\frac{1-\sin\theta}{1+\sin\theta} \right)$$

Where P_s = Total silt pressure in lbs in addition to water pressure which is calculated as for the case when there is no silt

γ_s = Submerged weight of silt in lbs/cu.ft

h_s = Height of silt in ft

θ = Angle of repose of silt

Ice pressure against dams: When a reservoir is completely frozen over and a rise of temperature occurs, the ice expands and if restrained, exerts pressure on the dam.

Wave action: This is to be taken into account for providing the free board and for computing the wave pressures acting on the dam.

Molitors's formula gives the wave height:

$$h_w = 0.17\sqrt{VF} + 2.5 - (F)^{1/4}$$

Where h_w = height of the wave from trough to the crest

V = velocity of wind in miles/hour

F = Fetch or straight length of water subject to wind action in miles.

The actual free board to be provided is 1.5 h_w. The wave pressure is given by

$$P_w = 125\, h_w^2$$

Where P_w = Wave pressure/unit length of dam in lbs.

The center of application of this force is above the still water surface. And acts at 0.375 h_w.

16.2 Earth Dam Design

The design of earth dams has been put on highly scientific lines with the development of soil mechanics. In olden days, experience and thumb rules used to govern the design.

As a result of extensive development of water resources all over the world the number of dam sites favorable for masonry and concrete dams are constantly decreasing and particularly in Pakistan there are a very few. Thus earth dams have assumed more importance and are coming into wider use. It will not be possible to discuss the design in great detail for which the references given at the end may be consulted.

Design Criteria

From a study of the failure of earth dams, the following criteria are laid down:

- Sufficient free board should be provided to avoid overtopping.
- The free surface line also called phreatic line should not meet the d/s slope, as this will cause infinite exit gradients and wash away the soil particles. Moreover the seepage loss should be at a minimum.
- The u/s and d/s slopes should be safe against slipping under different conditions including sudden draw down.
- There should be no chance for the water to have free passage through the dam or foundations.
- The u/s and d/s faces and the crest should be protected against wave action, frost and rain respectively.

The following are recommended values:

For dams ≥ 200 ft 1, 0 ft above the top of spillway gates for dams ≤ 200 ft above the top of spillway gates for the spillway crests a minimum of 6 ft should be provided

Control of phreaticline and seepage control:

- The phreatic line can be made to cut the base of the dam by the following methods. The seepage through the body of the dam and foundations can be reduced by the following methods:
- Provision of core wall.
- Grouting the foundation
- Cutt-off trenches

Sheet pile cut off in place of cut off trenches, are used for foundation of silt, fine sand and gravel, but in case of material resistant to penetration, sheet piles become uneconomical

Stability of slopes: The body of the dam and the foundation are subjected to shear stresses due to their own weight, by the seepage forces originating from pore pressure, from criteria for stability of slopes is to keep these shear stresses within the tolerable limits. A stable slope should resist deformation within the limits of tolerable bertical settlement and horizontal displacement. The failure of the slope occurs by way of sliding of a zone of an outer portion of an embankment which is separated from the remainder of the main body and is called shear failure. The slope shall be stable when the restoring moment set up by the self weight of the soil mass is more than the shearing moment (or disturbing moment) generated along the slip surface of the slope. Normally a factor of safety of 1.6 is taken.

$$\text{Factor of safety} = \frac{\textit{restoring moment}}{\textit{disturbing moment}} = 1.6$$

17 TRAINING OF RIVERS

17.1 River Training Works

Rivers flowing in alluvial plains meander in wide widths. River discharges also vary widely. In certain years, the flood season discharge may be as high as 500 times the discharge of low flow season. Therefore, rivers carry different sediment loads at different times of the year according to variations in discharge. Due to non-uniform variations in discharge and sediment load, the rivers enlarge and contract their channel width and gradually change their course form time to time.

When a barrage has been constructed across the river, the river is forced or guided to flow through it.

Moreover, fluming of the waterway of a barrage results in afflux, which may sometime be several feet. The effect of afflux is felt upto several miles upstream of barrage. High afflux may submerge large tracts of costly land and properties along the river, if these are not protected.

Therefore, to ensure that the river flows through the barrage and to protect the submergence of large tracts of costly lands and properties on the upstream, certain works upstream of a barrage are constructed. These works are called as river training works. The training works are: guide banks; marginal banks; spurs; pitched island; bank protection works etc. These are described below.

17.2 Guide Banks

The works in the river required to guide the passage of river water through a weir or a barrage are called gu7ide banks. The guide banks consist of two heavily built set of embankments with heavily protected curved ends in the shape of bell mouth. The width of waterway is kept equal to the width of the structure form abutment to abutment. The riverside slopes of the guide banks are pitched with stone and are provided with stone apron.

Layout of Guide Bank

Guide banks are constructed in a pair and symmetrical in plan. A symmetrical layout of the guide banks in plan is very essential to direct the flow through the barrage as gently as possible. The curve at upper end is carried well behind the bank, to an angle of 120 to 140 degree and with a radius of curvature equal to 200 to 1200 ft. depending upon the probable estimated velocity that is likely to occur near it. The radius of curvature of the downstream end of the bank may be 300 to 750 ft. and angel of the downstream curve as 45 to 60 degree. To ensure gentle entry and passage of water through the guide banks, they are laid out in such a way that they diverge from the barrage towards upstream and have longer length on the upstream and shorter length on to 1.25 L to 1.50 L whereas on the downstream it is kept equal to 0.25 to 0.5 L, where L is equal to the total width of the structure form abutment to abutment.

Section of Guide Bank

The guide banks are usually constructed with a top width of 30 ft. and side slopes of 2:1. The height of the banks are kept in such a way that a free board of 4 to 10 ft. is provided throughout the length of the guide bank above the highest flood level in the river. The slope on the river side is protect heavily by stone pitching. Moreover, the pitching at the impermeable head should be 25% extra. The following table gives the thickness of pitching:-

Table 17.1 The thickness of pitching

Sand Classification	Thickness of stone pitching, in.				
	Fall per mile in inches				
	3	9	12	18	24
Very Coarse	16	19	22	25	28
Coarse	22	25	28	31	34
Medium	28	31	34	37	40
Fine	34	37	40	43	46
Very Fine	40	43	46	49	52

Apron of Guide Bank

On the river side, the guide bank is proved with dry pitched stone floor. This floor is called apron. The apron is provided in sufficient width and thickness and is laid at the deepest bed level. The apron protects the guide bank from the effects of scouring. In case of scouring in font of the guide bank, the stones of the apron would fall into the scour hole thereby covering up the slope of the hole. The falling of the apron into the scour hole, which is called launching of the apron, prevents the scour in the river bed to travel backwards and to reach closer to the guide banks. However, in spite of it, if the scour travels backwards and closer to the guide bank, more and more part of the apron would launch. Therefore, the apron is proved with a width equal to not less than 1.5 times the deepest scour, which is expected to take place. The scour is calculated by Lacey's scour depth formula. For determining the deepest scour, it is multiplied by a factor.

$$R = 0.9\,(q^2/f)^{1/3}$$

Where R = scour depth, fit.
q = discharge per unit width of river, cfs per ft.
F = Lacey's silt factor

Impermeable Head of Guide Bank

On the upstream end, the head of the guide bank is provided with central angle of 120 to 140 degree. The radius of curvature is kept between 200 to 1200 ft. depending upon the slope of the river. This is called impregnable head. On the downstream end, the head is provided with a central angle of 45 to 60 degree. The radius of curvature depends upon the slope of the river.

The pitched apron of the impregnable head should be carried round the curve The curved portion should be carried to the full height of the bank. The side pitching of the impregnable head should be provided with 20°/o extra thickness. The width of the apron at the impregnable head should be equal to twice the deepest scour. It maybe remembered that at other parts of the guide bank, the width of apron is kept 1.5 times the deepest scour.

Illustrative Example

Design a guide bank required for a barrage across a river.

Given			
Max. Flood	600,000 cfs		
Slope	9 in.per mile		
River bed material	Medium sands		
Silt factor, f	1.00		
Bed Level	EL 622		
High Flood Level	EL 648/645		
Widht of the barrage (about to abutment, L)	2623 ft.		
Discharge per unit width, q	228.74 cfs/ft.		
Solution			
Dimension fo the Guide Bank			
Total length 1.75 L	1.75 x 2623	=	4590 ft
Upstream length 1.5 L	1.50 x 2623	=	3935 ft
Downstream length 0.25 L	0.25 x 2623	=	656 ft
Assume free board	4 ft		
Top level of guide bank	645 + 4	=	649
Assume top width of the guide bank		=	30 ft
Top width at impregnable head		=	60 ft
Thickness of pitching (for fine sand			
And river slope 9 in/mile) (table 17.1)	37 in. say		3.25 ft
Thickness of pitching at impregnable head	1.25 x 3	=	3.75
Side slope	2:1		
Depth of Scour, R	$0.9(q^2/f)^{1/3}$	=	33.66ft
Width of apron	1.50 x 33.66	=	51ft
Width of apron around the impregnable head	2.0 x 33.66	=	67 ft

Marginal Bank

With the construction of a weir or a barrage, the general level of water in the river upstream is raised by a few feet in all seasons. This has been called afflux and has been described earlier. This rise of water level is felt upto several miles upstream. If us rise is

left unattended, it will apart from threatening the safety of the structure, will submerge large areas upstream. Therefore, to protect properties from submergence and destruction, a set of embankments are constructed on both side of the river upstream of the barrage. These embankments are called marginal banks. Since the marginal banks are permanent structures these are aligned in such a way that they do to interfere with the river meander. They are tied or joint to the high grounds upstream so that the river flows within the two marginal banks and is not allowed t out flank the barrage and also not submerged areas upstream.

Marginal banks are usually provided with a top width of 20 ft. with river side slope of 2:1 to 5:1 and land side slope of 4:1 to 6:1. The river side slope is protected with stone itching and flexible stone apron.

17.3 Spurs

Spurs are heavily fortified embankments constructed in the river bed transverse to the river banks. Their main function is to control the river channel so that it may run along a specified course and provide protection to the river banks from scouring by keeping the river course away from it

Spurs may be aligned in such a way that they may either hold, repel or attract a river course. Thus they may be called a holding spur, a repelling spur or an attracting spur as per the function they perform.

Holding Spurs

Spurs constructed to hold the rivet current should he short in length and should be constructed perpendicular to the bank. Such spurs arc utilized to hold the channel in position and to protect the banks. To attract the river current towards the bank, spurs should be constructed N

Attracting Spurs

To attracting the river current towards the bank, spurs should be constructed pointing towards upstream of the flow. The angle of inclination may vary from 600 to 8O attracts rivet flow towards the banks from which the spur is originating. Such spurs i constructed very strong as they have to bear the fill frontal attack of the river flow

Repelling Spurs

Spurs constructed to repel the river current should be aligned pointing toward downstream. The angle of inclination with the normal to the bank varies from 1I to 600. The head of the spur should be heavily protected to resist swirling as of the current. The distance between the two spurs is kept I to 2 times the length of the spur. On bends, the distance between spurs is kept more in case of con banks and small for concave banks.

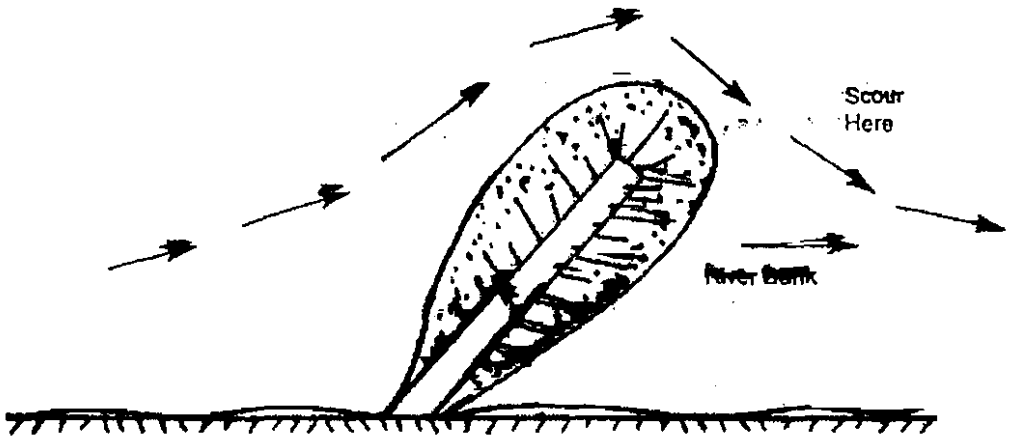


Figure 17.1 Repelling spur.

Classes of Spurs
The spurs can be divided into two classes according to the construction viz impermeable spurs and permeable spurs. These are described below

Impermeable Spurs
Impermeable spurs as the name implies are these spurs which do not allow to seep through them. Common type of impermeable spurs are solid stone, e core with stone pitching etc.

The different types of impermeable spurs commonly used in Pakistan are bar spur hockey spur, inverted hockey spurs, T-Head spur and sloping spur. The main difference between the various types of spurs lies in the shape of the spur head.

Bar Spur
It is the simplest type of spur. It is made of stones placed in the river bed and projecting into the river transverse to the flow.

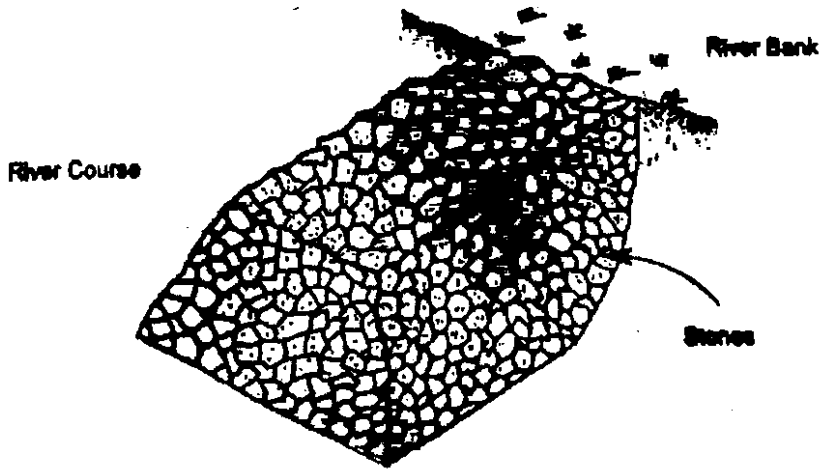


Figure 17.2 Bar spur.

Hockey Spur
As the name implies, this type of spur has a shape of a hockey stick with stone pitching on the concave face and to some length on the back. It attracts the river, and is helpful for bank protection.

river, and is helpful for bank protection.

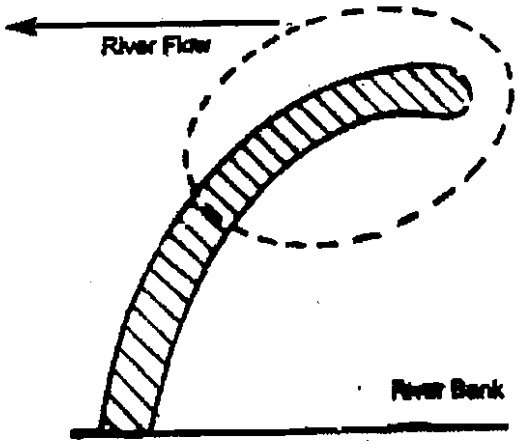


Figure 17.3 Hockey spur.

Inverted Hockey Spur
It has the inverted image of hockey spur and is like an inverted hockey stick. Its convex face is armored with stone, It repels the river flow.

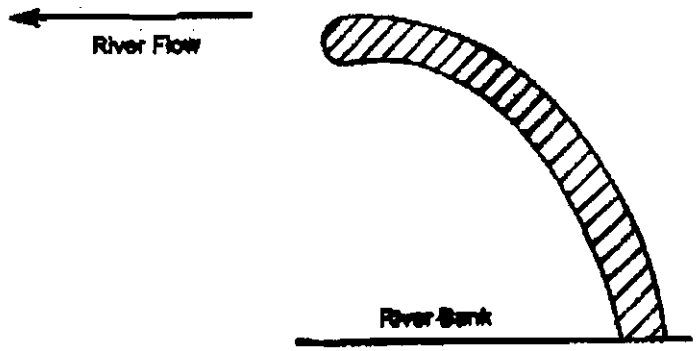


Figure 17.4 Inverted hokey spur.

T-Head Spur

This spur has the shape of T. The head is generally placed parallel with flow with about 1/3 length on the upstream and 2/3 length on the down stream of the flow.

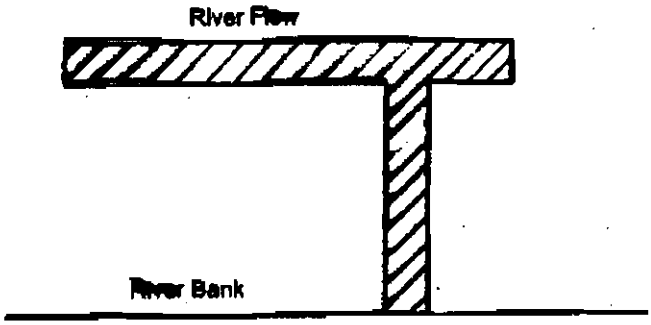


Figure 17.5 T-head spur.

Sloping Spur

It is a wedge shape ramp of solid stone with its height decreasing form the bank to the end a t aslope varying from 1:6 to 1:10. The side slope is generally 1:2.

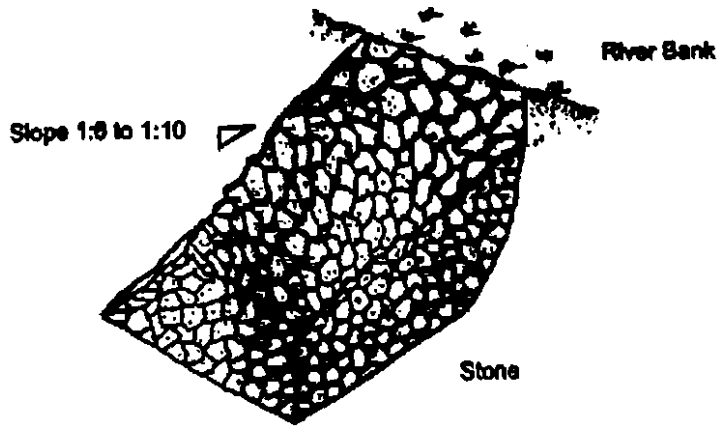


Figure 17.6 Sloping spur.

Components of Impermeable Spur

The components of impermeable spur are the head and shank. The shank consists of earth embankment, with pitching and stone apron. The earth embankment is provided with top width of 30 ft. and side slopes of 1:2. A minimum free board of 4 to 5 ft. above the highest flood level is provided. The stone pitching on the slopes protects the earthen embankment from erosion by the fast river current. The thickness of the stone pitching is kept the same as in the case of pitching for the guide banks. To protect the stone pitching from being undermined by scouring at its toe, a horizontal apron is provided in the same way as in the case of guide banks.

Permeable Spur

Permeable spur is that which allows water to seep through it However, as the silt laden water passes through the void in the material of such spur, its velocity is laden considerably, which allows water to drop its entire silt load in the pores of the spur gradually making it fully impermeable Common types of permeable spur are brushwood spur, wire crated stone spur, tree spur, hurdle dykes, etc.

Tree Spur

It is made of trees. Trees 20 to 40 ft. in height and which trunks varying between I to 4 ft. in girth may be cut and fastened to strong iron or wooden poles on the river banks by steel ropes. The trees ate hung upside down along the banks of the river. The leaves and branches slow down the velocity of water thereby inducing siltation.

In smaller rivers, tree spurs can be constructed by stretching a steel wire rope ½" – 1.5" in diameter across the river and anchored at its ends. At intermediate points it is supported by steel rails or by wooden poles. Trees 20 to 40 ft. in length are cut lauded on boats and tied to it either by coir ropes or by steel wires.

The spurs so constructed gives very satisfactory results. At places, where river brings a lot of silt, a solid earthen barrier can be constructed in a matter of weeks.

Weir Crated Stone Spur

This type of spur is made up of loose stones placed in wire crates. The wire crates are usually 2 to 3 ft in size and their top is kept about 3 ft. above the flood level.

Pitched Island

It is an artificially created island in the river. It is made of earth with pitched side slopes and stone apron. Sometimes an existing natural island in the river is reshaped and pitched and provided with aprons. This is called pitched island.

Pitched islands cause redistribution of tractive force, velocity and sediment charge of the river flow. The tractive force near a pitched island begins to increase rapidly after the construction of island, thereby resulting deep scour around the island thus drawing the river channel near it arid holding it there permanently Therefore, pitched islands are used, to correct river approach conditions by attracting river course above the weir and holding it there. It decreases the intensity of flow along marginal banks and guide banks thereby protecting them from damage. It also deepens the river channel, which becomes helpful for silt distribution in front of head regulators of off-taking channels and for navigation purposes. Pitched is a particularly suitable in deep channels. In shallow channels or channels with flashy floods, they are not suitable.

18 SALINITY AND WATERLOGGING

18.1 Salinity

Salinity refers to a build up of salts in soil, eventually to toxic levels for plants. Salt in soils decreases the osmotic potential of the soil so that plants can't take up water from it. When soils are salty, the soil has greater concentrations of solute than does the root, so plants can't get water from soil. The salts can also be directly toxic, but plant troubles usually result primarily from inability to take up water from salty soils. Problems with salinity are most commonly associated with excessive water application, rather than with too little.

All irrigation water contains dissolved salts derived as it passed over and through the land, and rain water also contains some salts. These salts are generally in very low concentration in the water itself. However, evaporation of water from the dry surface of the soil leaves the salts behind. Perhaps you have seen the whitish salt crust that can result on the surface of soils?

Salinity is especially likely to become a problem on poorly drained soils when the groundwater is within 3 m or less of the surface (depending on the soil type). In such cases, water rises to the surface by capillary action, rather than percolating down through the entire soil profile, and then evaporates from the soil surface.

Salinity is a worldwide problem, particularly acute in semi-arid areas which use lots of irrigation water, are poorly drained, and never get well flushed. Globally, something on the order of 20% of the world's irrigated acreage is estimated to be affected by salinity, with salt concentrations high enough n about 10% of irrigated acreage to decrease yields significantly.

Salinity obviously reduces crop productivity. In the USA, salinity may be lowering crop yields on as much as 25-30% of the nation's irrigated lands. In Mexico, salinity is estimated to be reducing grain yields by about 1 million tons per year, or enough to feed nearly million people. In extreme cases, land is actually being abandoned because it is too salty to farm profitably.

What happens when you irrigate?

Irrigation inevitably leads to the salinization of soils and waters. In the United States yield reductions due to salinity occur on an estimated 30% of all irrigated land. World wide, crop production is limited by the effects of salinity on about 50% of the irrigated land area. In many countries irrigated agriculture has caused environmental disturbances such as waterlogging, salinization, and depletion and pollution of water supplies. Concern is mounting about the sustainability of irrigated agriculture.

Where does all the salt come from?
Application of irrigation water results in the addition of soluble salts such as sodium, calcium, magnesium, potassium, sulfate, and chloride dissolved from geologic materials with which the waters have been in contact. Evaporation and transpiration (plant uptake) of irrigation water eventually cause excessive amounts of salts to accumulate in soils unless adequate leaching and drainage are provided. Excessive soil salinity reduces yields by lowering plant stand and growth rate. Also, excess sodium under conditions of low salinity and especially high pH can promote slaking of aggregates, swelling and dispersion of soil clays, degrading soil structure and impeding water and root penetration. Some trace constituents, such as boron, are directly toxic to plants.

Indicators of salinity
Die back of mature trees; wetness (groundwater seepage); patchy crop and pasture establishment; reduced yields; legumes perform poorly or fail; loss of clover/medics; sea barley grass; bare areas; halophytes; salt crystals; white salt crust.

18.2 Effects of Salinity
Over the course of history, thriving civilizations declined in part due to their inability to sustain food production on lands that had been salinized. It is estimated that 10 million hectares are now being lost every year as a result of salinity and/or waterlogging. Many of these problems are caused by excessive use of water for irrigation due to inefficient irrigation distribution systems, poor on-farm management practices, and inappropriate management of drainage water. Inefficient on-farm irrigation practices cause local salinity problems. Local problems increase as a result of poor on-farm drainage. Excessive irrigation increases salt loading in water tables and downstream aquifers which causes regional salinization. Lack of local and regional drainage systems results in lands being put out of agricultural production.

Remedial Measures
The "treatment" for salinization is to flush the soil with lots of water. Subsurface drainage can be used to minimize salinity by ensuring that a minimum quantity of water, "the leaching fraction" drain through the soil profile.. Drainage is aimed at ensuring that water table is maintained deep enough to prevent significant upward movement of salts by capillary flow
Shortage of irrigation and associated build of soil salinity can be prevented by the occasional application of heavy watering which flush accumulated salts down below the root zone.
Where irrigation water is saline, it can sometimes be mixed with good quality water or the two sources may be used alternately.
With careful management different crop sensitivities can be matched to availaiablity and quality of water to produce a viable cropping pattern.
When impeding layers exist in an otherwise permeable upper soil profile, shallow sub-soiling to just below the restrictive layer should be carried out. This will allow water

applied for reclamation or for normal irrigation to wash the accumulated salts down the topsoil.

Other salinity management techniques are:

- Seed placement
- Residues Management
- More frequent irrigation
- Pre-plant irrigation
- Changing surface irrigation method
- Chemical amendments

Table 18.1: EC_e values of soil salinity classes

Class	EC_e (dS/m)	Comments
Non-saline	<2	Salinity has no significant effect
Slightly saline	2-4	Yields of very sensitive crops may be affected
Moderately saline	4-8	Yields of many crops affected
Very saline	8-16	Only tolerant crops yield satisfactorily
Highly saline	>16	Only a few very tolerant crops yield satisfactorily

(Adapted from Taylor [1996])

Measurement of Salinity

Salinity is described in terms of electrical conductivity (EC). The higher the EC value, the more saline the soil or water.

Pure water without any salt is a fairly good insulator. If you throw a hair dryer into the bath, it is the salt in the water that will conduct the electricity. The more salt in the water, the better it will conduct electricity.

Soil or water EC can be easily measured with a hand held meter. To read the EC of a soil, a mixture of 1 part soil to 5 parts water is made and the reading from the meter is expressed as $EC_{1:5}$.

A salinity meter can measure the electrical conductivity of water
Photo: Dean Morris

The common unit for Electrical conductivity is **deciSiemens/metre (dS/m)**.

As a guide for soils:

sandy or loamy soils are saline if $EC_{1:5}$ is above 0.4 dS/m

clay soils are saline if $EC_{1:5}$ is above 0.7 dS/m

Measuring salinity in this way is not very accurate because it doesn't take into account soil texture, so in the laboratory electrical conductivity is measured by making a saturated extract of soil and water. The EC value is then expressed as EC_e or EC_{se}. The important salinity values for soil in terms of EC_e are shown in Table 18.1 above.

Classification of irrigation water based on EC

Class	EC (dS/m)
Ustable	0-1.5
Mageinal	1.5-2.7
Hazardoues	>2.7

The salinity of water is often measured using different units (Table 18.2).

Table 18.2: Units used for salinity (from Irrigation Salinity Note No. 7 March 1999, Salt Action)

1 dS/m=1mS/cm=1000EC (uS/cm)=640ppm
1 deciSiemen per metre (dS/m)
= 1 milliSiemen per centimetre (mS/cm)
= 1000 EC (microSiemens per centimetre)
= 640 parts per million in water (mg/litre)

So, to convert ...

from this unit	To this unit	Do this
EC (uS/cm)	dS/m	divide by 1000
ppm (mg/l)	dS/m	divide by 640
dS/m	EC	multiply by 1000
dS/m	ppm	multiply by 640

Water salinity is shown in mg/L in some publications.
So water of 8dS/m = 5120mg/L

18.3 Waterlogging

Another problem associated with excessive irrigation on poorly drained soils is waterlogging. This occurs (as is common for salinity) in poorly drained soils where water can't penetrate deeply. For example, there may be an impermeable clay layer below the soil. It also occurs on areas that are poorly drained topographically. What happens is that the irrigation water (and/or seepage from canals) eventually raises the water table in the ground -- the upper level of the groundwater -- from beneath. Growers don't generally realize that waterlogging is happening until it is too late -- tests for water in soil are apparently very expensive.

The raised water table results in the soils becoming waterlogged. When soils are water logged, air spaces in the soil are filled with water, and plant roots essentially suffocate -- lack oxygen. Waterlogging also damages soil structure.

Worldwide, about 10% of all irrigated land suffers from water logging. As a result, productivity has fallen about 20% in this area of cropland.

Both waterlogging and salinization could be reduced if the efficiency of irrigation systems could be improved, and more appropriate crops (less water hungry) could be grown in arid and semi-arid regions. In addition, increasing the cost of water to more closely reflect its true value would encourage its conservation, rather than using

incentives that essentially encourage wasting water, as some water rights laws in the US do. When a soil is waterlogged, most of the pore space is filled with water. Gases (eg oxygen) move about 10,000 times more slowly through water than air. Therefore when a soil is waterlogged, oxygen cannot enter the soil quick enough and its soil supply declines, while the carbon dioxide and other gases increases because it cannot escape. The rate of oxygen decline depends on the demand of the plant which in turn depends on the amount of organic matter in the soil, and soil temperatures.

The cause of injury to the plant may be an accumulation of carbon dioxide, a deficiency of oxygen, the formation of reduced compounds of iron and manganese, or reduction of nitrates to nitrites and of sulphates to sulphides. Root growth decreases and most of the processes that are essential to meeting the nitrogen and mineral-nutrient needs of plants are brought to a stop (Bear 1965).

Indicators of waterlogging
Continuous wetness; presence of weeds such as rushes *(Juncus* spp); water buttons *(Cotula* spp); sea barley grass; patchy or stunted crop and pasture growth; yellowing or reddening of leaves; excessive growth of algae; "rotten egg gas" smell from soil (anaerobic); presence of dull yellow mottles (minor waterlogging) or blue-grey mottles (strong waterlogging) in the soil profile.

The depth of water table which adversely affects the growth of different crops is given below.

Crop	Depth of water table (m)
Wheat	0.9 to 1.2
Cotton	1.5 to 1.8
Rice	0.5
Sugarcane	0.3
Fodder crops	1.2

18.4 Causes of Waterlogging
Inadequate surface drainage
When the surface drainage is not adequate the heavy preipation in the area is not drained off quickly and the rain water remains stagnenet over the area from considerable time. This gives rise to heavy percolation and water table rise in the area.

Seepage from canals
In nature, the watertable is in state of equibilrum. The amount of inflow is practically equal to the amount of outflow. This equilibrium is upset by the construction of a new canal system as a new constant source of inflow due to seepage is introduced.. The water table in the area therefore rises

Over irrigation of fields
When the irrigation water applied to the field is in excess of the requirement of the crop, deep percolation takes place which causes ground water recharge.

Obstruction of natural drainage
If a natural drainage is obstructed by irrigation channel, rail or road embankment, it will not be able to pass the rain water of the catmint. There will be flooding of land and consequent water logging.

Construction of a water reservoir
Similar to seepage from a canal, the seepage from the reservoir augment the ater table and may cause water logging.

Natural obstruction to the flow of ground water
Sometimes subsoil does not permit free flow of subsoil water due to some natural obstruction. This may accelerate the process of rising of water table.

18.5 Remedial Measures

Efficient drainage
An efficient drainage system which permits a quick flow of rain water in short period helps to reduce waterlogging.

Reducing percolation from canals
Measures for substantial reduction of percolation losses from canals have to be undertaken. This can be achieved in the following ways.
Lining of irrigation channels
Lowering of full supply level in the channels
Construction of interceptor drains

Economic use of irrigation
Cultivators should be educated on economic use of water and to avoid wastage.

Lining of canals and watercourses
The losses by percolation from the watercourses could be 20-30% or above. Their lining further checks the inflow of water to subsoil.

Removing obstruction in natural drainage
Drainage crossing with road, railways and canals should be remodeled to make it more efficient.

Lowering of groundwater by pumping
The surplus ground water which cause undesirable rise in water table can be pumped out by installing shallow wells and deep wells.

Changes in cropping pattern
A change in cropping pattern may minimize the damage to plant life.

Adoption of high efficenciy irrigation methods (sprinkler and trickle irrigation)
The reduce percolation losses as only predetermined amount of water is applied to the land.

18.6 Salinity Waterlogging Interaction

Although waterlogging does not directly cause salinity it can exacerbate the problem. It impedes plant growth, reduces water use which can cause increased recharge and reduces plant cover which promotes capillary rise in discharge areas. A combination of salinity and waterlogging is very damaging to plant growth. Waterlogging can significantly reduce crop yields or kill plants on soils containing only small amounts of salt.

Drainage solutions for land salinity/rising water tables and sometimes waterlogging is being increasingly used as an attempted cure. This is not surprising as drainage is an engineering approach that has, in some cases, improved saline land and reduced the effects of excess water but in a number of cases may cause other problems further downslope. It does not address the long term issue of excess water in the system and in many cases just shifts the problem elsewhere.

19 CANAL MAINTENANCE

After design and construction of a canal irrigation system, its maintenance is the routine work and continuous process. Maintenance must be done to keep irrigation schemes working properly. Maintenance of work includes cleaning and repairing rain cuts and rodent's holes, removing floating debris and other unwanted materials, repairing canals and drains, repairing structures and canal linings and cleaning, lubricating and painting gates, and replacing parts of structures and machines. Maintenance program may be different in different canal water supply schemes and at different places.

Where maintenance is neglected, minor jobs turn into major jobs and maintenance cost becomes very high. Canals and drains often become choked with weeds and silt and gates do not work because they are broken or rusted through lack of attention. There are many reasons why maintenance is not done. Sometimes irrigators and canal operators do not understand its importance and so have no interest. On many larger schemes very little money is made available for maintenance and so there is often not enough equipment, men and materials to do the work.

Maintenance requirements will differ from place to place and schemes to schemes. The most common problems and ways of solving them mentioned by (Melvyn Kay, UK) are as follows:

19.1 Unlined Canals

There are four main problems

1. Silting
2. Weed growth
3. Burrowing animals
4. Bank Erosion

Silting

This is a very common problem and is mainly due to silty water being taken from a river. The silt settles on the bed of the canal and reduces the flow capacity. Siltation can be a problem on new canal system when they are operating at very low flows in the early stages of development. Some canals can become completely blocked when the silt content of the water is very high.

Silt can be removed by hand but this usually limited to watercourses and farm channels and their laterals. To make sure that the correct shape of channel is excavated a profile board of the correct shape can be used. This is inserted into the channel at regular intervals during excavation. The labourers can then see whether or not more excavation is needed. Canals must be emptied and allowed to dry out for several days before excavation begins. This not only makes the work easier but it also reduces the risk of infection from water borne diseases. The two most important diseases are

Schistosomiasis and Malaria. Each of these diseases has a special life cycle which involves water. It is estimated that more than 200 million people are suffering from Schistosomiasis and 500 million from the Malaria in the world.

The most common machines used for clearing silt are draglines and hydraulic excavators. These machines work from the canal bank top and are usually crawler mounted for easy working in muddy conditions. They have a reach of 10-15 meters. Small canals can be cleaned by one machine working from one of the bank tops. Larger canals usually require two machines working on opposite banks. Although the excavators can work while the canal is flowing it is difficult to see how well the work is proceeding. Silt may also be carried further down stream by the flow. Excavation is best done at the end of the irrigation season when the canal can be drained down. The silt is dumped outside the canal, left to dry and spread by bulldozer. Up to 300 meters of canal can be cleared each day depending on the working conditions. Where as one person can desilt 150-100 cubic foot during one day manually in watercourses.

Usually water flow is stopped in the canals during the month of January. The last two oe three weeks are best suited for desilting. Similarly during monsoon season the farmers can desilt the part of the watercourses carrying water to their vicinity. One acre-foot water can be saved by desilting one watercourse. This saving increases the area under cultivation and production.

The equipment used in desilting should be well maintained in advance. The maintenance work should be performed at daylight. Proper management of light is necessary if work done at night. Desilting begins from outlet to the tail end of watercourse. It takes two to three days for long and a few hours for short watercourses.

Weed Growth

Silting encourages weed growth. In some cases the weeds become so dense they almost stop the water flow. There are many types of water weed. Some grow on canal banks close to the water. Others grow in water and are either submerged or float on the surface. These weeds are removed by cutting or by excavation when silt is being removed. They can be cut manually or by machine. Hand tools can be used like scythes and rakes. The right tools will mean that the work can be done quickly but large numbers of labourers are needed. One man can usually clear from 50 –150 m of farm channel each day. Weeds are best cut near the base of the stem so that roots are left undisturbed. Regular cutting in this way will weaken and kill the plants.

Chemicals are available which can kill aquatic weeds. However, they can be expensive and great care is needed when they are used. Some of the chemicals are harmful to people and animals and also to crops. They should not be used in places where people regularly uses canal the canals for drinking and bathing. Water weeds are not always a nuisance. These can be cut and used as roofing material for houses.

Burrowing Animals
Leakage through canal banks can be caused by small burrowing animals or by rotting of plants not removed during construction. Small holes can be plugged with soil using spades and small hand compactors. Larger holes may need machines to excavate out the damaged section of bank and replace it with new compacted soil. Chemicals can be used to kill rodents that make holes in the banks. Rats and scorpentines make holes in the banks of canals.

Erosion of Banks
Canal banks cab be eroded by rainfall, wave action, animal using the canals for drinking and heavy vehicles using them as roads. The worst damage is usually done in wet weather. Gullying from the rainfall is probably the most common erosion problem. Repairs are best done at the end of rainy season when holes can be filled with soil and compacted either by hand or by machine. Where gulling leads to excessive seepage and a risk of breaching the bank immediate repairs are needed. Erosion can be greatley reduced by growing grass on the banks. This provides protection for the soil and also slow down the water running off the side slopes. Cattle damage can br reduced by stopping vehicles using the canal banks for access during wet weather.

At the location of fall structures, erosion occurs nearby the side walls and cistern and usually where the width of the structure is minimum. So protection should be provided in these parts of the structures where velocity of water flow is maximum.

Such places should be avoided for swimming, bathing, and children's water games if small hollows or big pits are formed in the bed of the channel and canals and animals excreta is mixing somewhere in the water. Scouring of bed should be controlled by putting stones etc in this type of sections.

19.2 Lined Canals
These are usually built to reduce maintenance problems. They will do so providing they have been built properly. After sometime it may be necessary to replace sections of cracked concrete lining and damaged joints. This can be expensive and requires skilled masons to do the work well. Normally silt and weed growth are not a problem as settling basins and silt trapes are used to get rid of silt. If silting does occur then weeds may also grow. This can only be removed by hand because machines would damage the lining.

Similarly if the canal lining is of brick masonry, abnormal settlement or cracks may develop with time or plaster in cement mortar layed over the bricks may crack. This failure is usually local and can be repaired easily.

19.3 Ponds and mini dams

Maintenance and proper care is needed for these structures. All the waterways bringing water to the inlets of these structures should be monitored and inspected for repair and desilting. The work should be completed on time to store maximum amount of water for irrigation purposes. The capacity of ponds or reservoirs is reduced due to accumulation of sand silt and clay. The water then rises up in the structure and damages the structure due to seepage in the side walls or bunds. The grassy waterways should be kept in the designed limits to control sediment entry into the small dams.

These structures should be free from weeds and water loving plants. The grass should be left on the banks to control bank erosion. Deep rooted trees, herbs and shrubs should be removed from the banks to reduce seepage rates and store maximum amounts without wastage. The shape and structure of grass waterways should not change and proper slope be maintained for easy crossing of the farm machinery. Animals should be kept away from the banks of reservoirs to control bank erosion from their feet and weight for the provision of water to agricultural lands during drought.

Drains

These are at the end of the schemes and tend to be neglected. Silt is not usually a problem because most of the flow is from rainwater and excess irrigation. However weeds growth can be very heavy. For practical purposes maintenance of open drains is very similar to that of unlined canals. When clearing drains it is best to work upstream from the outfall so that water can flow easily out of the scheme.

19.4 Breaches

These are breaks which occur in channel banks causing water to spill from the channel in an uncontrolled manner. In canals they occur when banks have not been properly compacted during construction, when they have been eroded and not properly maintained; and when water levels rise because of misuse and water flows over the bank top. Breaches on large canals can be very dangerous. People can be drowned and a great damage done by the sudden release of water. Repairs are often done in emergency using a bulldozer to push up soil to close the gap in the bank. This is a bad practice because it leaves the bank very weak. The soil is not compacted and it is sitting on a lose muddy foundation. The best method is to drain out canal; remove all the mud and debris from the damaged section and rebuild the damaged section with new compacted soil.

Care is needed to compact the soil where it ties into the existing bank. Breaches in small channels such as farm channels and watercourses can be repaired by hand. If the flow is not too high (50 l/sector less) the breach can be repaired while the water is flowing. For larger flow it will be necessary to drain down the channel. In drains, breaches are usually deliberate. Irrigators sometimes open the protecting bank around the drains to release flood water or excess irrigation water when the small drain inlets cannot cope with the flow. This can cause severe gully erosion and the material wash down can block water

flow and cause flooding. Irrigators should not be allowed to continue such damaging practices. Canal and drain structures must also be carefully maintained for the system to work properly.

Water control gates
Most irrigation schemes use moving gates to control the distribution of water. These need regular attention if they are to continue to function. Many of the more modern gates are supplied with handbook by the manufacturer giving details of greasing, oiling, general cleaning and checking movement over full range of travel. Water seals are usually inspected when the canal is empty and they are placed if worn or perished. Gates are usually made of steel and this requires regular painting to avoid rust. Downstream protection of structures is important.

19.5 Rehabilitation and Improvement
Rehabilitation refers to the system or sub-system wise reconstruction or restoration of facilities and /or structures from a deteriorated condition to a serviceable condition that meets their entire original functions as well as meeting all performance standards.

Improvements refer to the relocation or addition of a new facility those results in more efficient canal operation for better distribution of water to the farmers. Under the above definitions the need for rehabilitation would only occur as result of a deferred maintenance program. Differed maintenance could be the direct result of a government policy or the direct result of inadequate budgets, poor management, and improper repairs.

These two types of improvement programs require the expenditure of funds that must be separated from the regular operation and maintenance funds. These programs should not be confused with replacement. Replacement refers to the periodic replacement of worn out equipment and facilities that have a more or less fixed life span (e.g, the replacement of gates, gate hoists, vehicles, broken section of lining etc.). Rehabilitation and improvements are combined under the term capital improvements with regard to funding and budgets, for accounting purposes. This is accomplished by including a capital improvement program that is funded from a separate budget. The ADP budgets should be used only for capital improvements and PC-1 proforma must be prepared. Maintenance program include the following:

- Routine
- Preventive
- Seasonal
- Annual
- Unscheduled
- Emergency

Each type of these maintenance programs has its own way of performing it, and the details can be seen in "Operation and Maintenance Manual for Canal Systems in N-W.F.P".

20 REFERENCES

Ahmad Bashir, Hussain, Z and Longinire I. 1993. Farm Management Hand Book. ASSP/USAID International Consulting Division, Washington, D.C., USA.

Ahmad, N. 2003. Tahafaz-I-Aab and Abpashi. Urdu Language Pamphlet. Dept. of Water Management. N-W.F.P. Agricultural University.

Ahmad, S. and M. Aslam. 1994. Development of resource management strategies to raise productivity of eroded areas and watersheds. Final Report of PAK-SS-PR-11, A Project Funded by US National Research Council, PARC, WRRI-NARC.

Ahmad, S., M. Khan, and Z. Ikram. 1990. Soil and water conservation and integrated land use in Pothwar, Pakistan. Proceedings of International Seminar on "Soil Physics:

Applications under Stress Environments", PARC, p.301-313.

Ahmad, S., 1993. Water Use and Crop Production in Pakistan, PARC, Islamabad,

Ali, Iqbal., 1993. Irrigation and Hydraulic Structures (Theory, Design and Practices). Institute of Environmental Engineering & Research. NED University of Engineering and Technology, Karachi, Pakistan.

American Society of Agricultural Engineers, 1985. R264.2, Minimum requirements of the design, installation, and performance of sprinkler irrigation equipment. In 1985 Agricultural Engineers Yearbook.

American Society of Agricultural Engineers, 1985. ASAE Engineering Practice, ASAE EP 405

Balochistan Minor Irrigation and Agricultural Project, Jan 1993. Design Manual Perennial Irrigation Canals and Canal Structures. Quetta, Pakistan.

Barron. AR., 1978. Handbook of Dam Engineering, McGraw Hill, New York.

Bhargava, B.S. 1980. Minor Irrigation Development Administration, Ashish

Bos, M. C., Replogle, J. A. & Clemmens, A. 1. 1984. Flow Measuring Flumes for Open Channel Systems. John Wiley & Sons, New York.

Brouwer. C et al, Irrigation Water Management Training manual No.3 "Irrigation Water needs" FAO of the United Nations 1985-1987.

Brouwer. C et al, Irrigation Water Management Training manual No.2 "Elements of topographic surveying" FAO of the United Nations 1985-1986.

Brouwer.C , Heibloem .M & Geoffeau, Irrigation Water Management Training manual No.1 "Introduction to Irrigation" FAO of the United Nations 1985-1986.

Byerlee Derek and Hussain Tariq, 1992. Farming System of Pakistan Vanguard Books (Pvt) Ltd, 45-Mall, Lahore.

Chow, V.T. Open channel hydraulics, McGraw Hill Book Co., Inc.; New York N.Y., 1959.Company Ltd. Ram Nagar, New Delhi-110055.

Department of Agriculture, ARI, Saryab, Rani Bagh, Ziarat, Balochistan, 1995. Quetta.

Department of Agriculture, Irrigated Crop Production Manual, 1992. Ministry of Agriculture, Bangladesh.

Department of Agronomy, University of Agriculture, Faisalabad (Pakistan).

Doorenbos. J et al, guidelines for predicting crop water requirements, "Crop water Requirements FAO Irrigation and drainage paper 24" FAO of the United Nations Rome,1992.

FAO 33, Yield Response to Water, Irrigation and Drainage Paper, Rome, 1979.

FAQ, 1975. Irrigation and Drainage Paper No. 26, 1 & 2. Small Hydraulic Structures. Food and Agriculture Organization of the United Nations, Rome.

Framiji, K.K., 1969. Irrigation and drainage in the world (A Global Review) Second Edition Vol II. ICID, 48 Nyaya Marg. Chana Kapuri New Delhi-21 India.

Govt. of N-W.F.P. "Operation and Maintenance Manual for Canal Systems in N-W.F.P".1992. Irrigation Department.

Handbook of water measurement, USBR (1975)

Henderson, F. M. 1966. Open Channel Flow. MacMillian Co., New York.

Horst L., 1998, The Dilemmas of Water Division, Considerations and Criteria for Irrigation System Design, International Water Management Institute.

Janke, W., Mumtaz Ahmad, and other, 1991. Bringing New Technology to Barani Farmers, PARC, Islamabad.

Justins, J.D., Hinds, J. and Greager, 1945. W.P. Engineering for Dams, John Wiley and Sons, New York.

Kay, Melvyn. 1983. Surface Irrigation Systems and Practice. Cranfield Institute of Technology, Cranfield Press. England UK.

Keller, J., and R.D. Bliesner. 1990. Sprinkle and Trickle Irrigation. Published by Van Nostrand Reinhold, New York.

Khoso, A. W, Crops of Sindh 1992, Fifteen Editions, Sindh Agricultural University Tandojam.

King, H. W and Brater, E. F. 1963. Handbook of Hydraulics. 5th Edition. McGraw-Hill Book Co., New York.

Kirpich, P.Z. 1940. Time of concentration of small agricultural watersheds. Civil Eng.

Lieftinck, P., Sadove, AR. and Creyke. T.C., 1968. Water and Power Resources of West Pakistan, Vol. III published for the World Bank by John Hopkins Press, Baltimore.

Malik, B.A. 1986. Pakistan limiting water resources and growing demand. International Commission on Irrigation and drainage. Lahore

Manual-Corps of Engineers, 1960. Design Stability of Earth and Rock-fill Dams, U.S. Army Engineers

Meijer, T. K. E. 1991. Design of Smallholders' Irrigation Systems. Department of Irrigation and Soil and water conservation. Wageningen Agricultural University, The Netherlands

P A T A Project, Irrigated Agronomy Training, Saidu Sharif.

PARC/CI:M:MYT , NARC, AERU (P ARC), Increasing wheat productivity, 1986. Islamabad~ AARI, Faisalabad.

PMD. 1971. Monthly and monsoon normal isohyetal maps of the Northern West Pakistan. Directorate of Meteorological Services, Karachi.

Portland Cement Association. 1964. Handbook of Concrete Culvert Pipe Hydraulics. Portland Cement Association. Chicago, IL.

Rashid A 1996. et al "Soil Science", National Book Foundation, Islamabad.

Rao, J.V. 1960. A Short Course on Dams Engineering, I. I. T. Kharagpur.

Rao J.V., 1960. An Analytical Approach for Testing the Stability of Slopes Subject to Efflux of Seepage, Journal of Scientific Research, I .I. T., Kharagpur.

Sain Kanwar, 1962. Modern Trends in the Construction of Dams and Power Houses, Atma Ram and Sons, Delhi.

Schwab, G.D., R.K. Frevert., T.W. Administer. and K.K. Barnes. 1956. Soil and water conservation engineering. John Wiley and Sons, Inc., New York.

Siddiqui, I.H., 2003. Irrigation and Drainage Engineering, Royal Book Company, Karachi.

SCS.1964. Engineering field manual for conservation practices. US Dpt. of Agri.Soil Conservation Service, USA.

Skogerboe, G. V., Ray S. Bennett, Wynn R. Walker, 1990. Technical Bulletin 120, Colorado State University Experiment Station, Fort Collins, Colorado.

Sharma,R.K. and T.K.Sharma. (1993) Canal Structures (Including River Engineering). Text Book of Irrigation Engineering, Vol.III. Oxford and IBH Publishing Co.Pvt. Ltd. New Delhi.

Sharma,S.K. 1987. Principles and Practices of Irrigation Engineering. India.
Sherrad, J.L., Woodward, R.J., Gizienski and Clevenger, WA., 1963.Earth and Earth Rock Darns, John Wiley and Sons, New York.

Singh, G., C. Venkataramanan., G. Sastry. and B.P. Joshi. 1990. Manual of soil and water conservation practices. Oxford and IBH Publishing Co. Pvt. Ltd., India.

Soil Survey of Pakistan, 1985. Atlas of Pakistan, Islamabad.

Soil Conservation Service, 1984. Trickle Irrigation. U.S. Department of Agriculture, Soil Conservation Service, National Engineering Handbook.

Technical Assistance Bureau of Agency for International Development, Guide for Field Crop Production in the Tropics and the Sub-tropics, Washington, D.C 20523, USA. 1974.

U.S. Bureau of Reclamation. 1977., Design of Small Dams, Ministry of Interior, Washington,

USBR Publication. Design of Sinai! Cairn! Structures. U.S. Department of Interior, Bureau of Reclamation, Engineering Research Center, Denver, CO.

USBR, 1967. Design Standards No. 3 - Canals and Related Structures. U.S. Department of Interior, Bureau of Reclamation, Engineering Research Center, Denver, Co.

Withers, B., and S. Vipond, 1980. Irrigation Design and Practices. Cornell University Press, Ithaca, N.Y.

ANNEXURE A: DISCHARGE TABLES

Table A1. Suppressed rectangular weir discharge values with different widths.

SUPPRESSED RECTANGULAR WEIR

Q - Liters per second L- Length in meter $Q=1.83865LH^{3/2}$

Head (cm)	0.2	0.3	0.5	0.75	1.00	Head	1.30	1.50	1.75	2.00	2.50	2.75	3.00
0.5	0.1	0.2	0.3	0.5	0.7	1.0	2.4	2.8	3.2	3.7	5.1	4.6	5.5
1.0	0.3	0.6	0.9	1.4	1.8	2.0	6.8	7.8	9.1	10.4	14.3	13.0	15.6
1.5	0.5	1.0	1.7	2.5	3.4	3.0	12.4	14.3	16.7	19.1	26.3	23.9	28.7
2.0	0.8	1.6	2.6	3.9	5.2	4.0	19.1	22.1	25.7	29.4	40.5	36.8	44.1
2.5	1.1	2.2	3.6	5.5	7.3	5.0	26.7	30.8	36.0	41.1	56.5	51.4	61.7
3.0	1.4	2.9	4.8	7.2	9.6	6.0	35.1	40.5	47.3	54.0	74.3	67.6	81.1
3.5	1.8	3.6	6.0	9.0	12.0	7.0	44.3	51.1	59.6	68.1	93.6	85.1	102.2
4.0	2.2	4.4	7.4	11.0	14.7	8.0	54.1	62.4	72.8	83.2	114.4	104.0	124.8
4.5	2.6	5.3	8.8	13.2	17.6	9.0	64.5	74.5	86.9	99.3	136.5	124.1	148.9
5.0	3.1	6.2	10.3	15.4	20.6	10.0	75.6	87.2	101.8	116.3	159.9	145.4	174.4
5.5		7.1	11.9	17.8	23.7	11.0	87.2	100.6	117.4	134.2	184.5	167.7	201.2
6.0		8.1	13.5	20.3	27.0	12.0	99.4	114.6	133.8	152.9	210.2	191.1	229.3
6.5		9.1	15.2	22.9	30.5	13.0	112.0	129.3	150.8	172.4	237.0	215.5	258.5
7.0		10.2	17.0	25.5	34.1	14.0	125.2	144.5	168.6	192.6	264.9	240.8	288.9
7.5		11.3	18.9	28.3	37.8	15.0	138.9	160.2	186.9	213.6	293.7	267.0	320.4
8.0		12.5	20.8	31.2	41.6	16.0	153.0	176.5	205.9	235.3	323.6	294.2	353.0
8.5		13.7	22.8	34.2	45.6	17.0	167.5	193.3	225.5	257.8	354.4	322.2	386.6
9.0		14.9	24.8	37.2	49.6	18.0	182.5	210.6	245.7	280.8	386.1	351.0	421.2
9.5		16.2	26.9	40.4	53.8	19.0	198.0	228.4	266.5	304.6	418.8	380.7	456.8
10.0		17.4	29.1	43.6	58.1	20.0	213.8	246.7	287.8	328.9	452.2	411.1	493.4
10.5			31.3	46.9	62.6	21.0	230.0	265.4	309.6	353.9	486.6	442.4	530.8
11.0			33.5	50.3	67.1	22.0	246.6	284.6	332.0	379.5	521.8	474.3	569.2
11.5			35.9	53.8	71.7	23.0	263.7	304.2	354.9	405.6	557.7	507.0	608.4
12.0			38.2	57.3	76.4	24.0	281.0	324.3	378.3	432.4	594.5	540.5	648.5
12.5			40.6	60.9	81.3	25.0	298.8	344.7	402.2	459.7	632.0	574.6	689.5
13.0			43.1	64.6	86.2	26.0	316.9	365.6	426.6	487.5	670.3	609.4	731.3
13.5			45.6	68.4	91.2	27.0	335.3	386.9	451.4	515.9	709.4	644.9	773.9
14.0			48.2	72.2	96.3	28.0	354.1	408.6	476.7	544.8	749.2	681.0	817.3
14.5			50.8	76.1	101.5	29.0	373.3	430.7	502.5	574.3	789.6	717.9	861.4
15.0			53.4	80.1	106.8	30.0	392.8	453.2	528.7	604.2	830.8	755.3	906.4
15.5			56.1	84.2	112.2	31.0	412.6	476.0	555.4	634.7	872.7	793.4	952.1
16.0			58.8	88.3	117.7	32.0	432.7	499.2	582.5	665.7	915.3	832.1	998.5
16.5			61.6	92.4	123.2	33.0	453.1	522.8	610.0	697.1	958.5	871.4	1045.7
17.0				96.7	128.9	34.0	473.9	546.8	637.9	729.0	1002.4	911.3	1093.6
17.5				101.0	134.6	35.0	494.9	571.1	666.3	761.4	1047.0	951.8	1142.1
18.0				105.3	140.4	36.0	516.3	595.7	695.0	794.3	1092.2	992.9	1191.4
18.5				109.7	146.3	37.0	538.0	620.7	724.2	827.6	1138.0	1034.5	1241.4
19.0				114.2	152.3	38.0	559.9	646.0	753.7	861.4	1184.4	1076.7	1292.1
19.5				118.7	158.3	39.0	582.2	671.7	783.7	895.6	1231.5	1119.5	1343.4
20.0				123.3	164.5	40.0		697.7	814.0	930.3	1279.2	1162.9	1395.4
20.5				128.0	170.7	41.0		724.0	844.7	965.4	1327.4	1206.7	1448.1
21.0				132.7	176.9	42.0		750.7	875.8	1000.9	1376.3	1251.2	1501.4
21.5				137.5	183.3	43.0		777.7	907.3	1036.9	1425.7	1296.1	1555.3
22.0				142.3	189.7	44.0		805.0	939.1	1073.3	1475.7	1341.6	1609.9
22.5				147.2	196.2	45.0		832.5	971.3	1110.1	1526.3	1387.6	1665.1
23.0				152.1	202.8	46.0			1003.9	1147.3	1577.5	1434.1	1720.9
23.5				157.1	209.5	47.0			1036.8	1184.9	1629.2	1481.1	1777.3
24.0				162.1	216.2	48.0			1070.0	1222.9	1681.5	1528.6	1834.3
24.5				167.2	223.0	49.0			1103.6	1261.3	1734.3	1576.6	1892.0
25.0					229.8	50.0			1137.6	1300.1	1787.7	1625.2	1950.2
25.5					236.8	51.0				1339.3	1841.6	1674.1	2009.0
26.0					243.8	52.0				1378.9	1896.0	1723.6	2068.4
26.5					250.8	53.0				1418.9	1950.9	1773.6	2128.3
27.0					258.0	54.0				1459.2	2006.4	1824.0	2188.8
27.5					265.2	55.0				1499.9	2062.4	1874.9	2249.9
28.0					272.4	56.0				1541.0	2118.9	1926.3	2311.5
28.5					279.7	57.0				1582.5	2175.9	1978.1	2373.7
29.0					287.1	58.0				1624.3	2233.4	2030.4	2436.5
29.5					294.6	59.0				1666.5	2291.4	2083.1	2499.8

Table A2. Discharge values of the contracted rectangular weirs with different widths.

STANDARD CONTRACTED RECTANGULAR WEIR

Q - Liters per second L- Length in meter $Q=1.83865(L-0.2H)H^{3/2}$

Head (cm)	0.15	0.3	0.5	0.75	1.00	1.30	1.50	1.75	2.00	2.50	2.75	3.00
1	0.3	0.5	0.9	1.4	1.8	2.4	2.8	3.2	3.7	4.6	5.1	5.5
2	0.8	1.5	2.6	3.9	5.2	6.7	7.8	9.1	10.4	13.0	14.3	15.6
3	1.4	2.8	4.7	7.1	9.5	12.4	14.3	16.7	19.1	23.8	26.2	28.6
4	2.1	4.3	7.2	10.9	14.6	19.0	21.9	25.6	29.3	36.7	40.3	44.0
5	2.9	6.0	10.1	15.2	20.4	26.5	30.6	35.8	40.9	51.2	56.3	61.5
6	3.7	7.8	13.2	19.9	26.7	34.8	40.2	47.0	53.7	67.2	74.0	80.7
7	4.6	9.7	16.5	25.1	33.6	43.8	50.6	59.1	67.6	84.7	93.2	101.7
8		11.8	20.1	30.5	40.9	53.4	61.7	72.1	82.5	103.3	113.7	124.1
9		14.0	23.9	36.3	48.7	63.6	73.6	86.0	98.4	123.2	135.6	148.0
10		16.3	27.9	42.4	57.0	74.4	86.1	100.6	115.1	144.2	158.7	173.3
11			32.1	48.8	65.6	85.7	99.1	115.9	132.7	166.2	183.0	199.8
12			36.4	55.5	74.6	97.5	112.8	131.9	151.0	189.2	208.4	227.5
13			40.9	62.4	83.9	109.8	127.0	148.6	170.1	213.2	234.8	256.3
14			45.5	69.5	93.6	122.5	141.8	165.9	189.9	238.1	262.2	286.2
15			50.2	76.9	103.6	135.7	157.0	183.7	210.4	263.8	290.5	317.2
16			55.1	84.5	113.9	149.2	172.7	202.2	231.6	290.4	319.8	349.3
17			60.1	92.3	124.5	163.2	188.9	221.2	253.4	317.8	350.0	382.2
18				100.3	135.4	177.5	205.6	240.7	275.8	346.0	381.1	416.2
19				108.4	146.5	192.2	222.6	260.7	298.8	374.9	413.0	451.0
20				116.8	157.9	207.2	240.1	281.2	322.3	404.6	445.7	486.8
21				125.3	169.5	222.6	258.0	302.2	346.5	434.9	479.2	523.4
22				133.9	181.4	238.3	276.2	323.7	371.1	466.0	513.4	560.8
23				142.8	193.5	254.3	294.9	345.6	396.3	497.7	548.4	599.1
24					205.8	270.7	313.9	367.9	422.0	530.1	584.1	638.2
25					218.3	287.3	333.3	390.7	448.2	563.1	620.5	678.0
26					231.1	304.2	353.0	413.9	474.8	596.7	657.7	718.6
27					244.0	321.4	373.0	437.5	502.0	631.0	695.4	759.9
28					257.2	338.9	393.4	461.5	529.6	665.8	733.9	802.0
29					270.5	356.6	414.1	485.8	557.6	701.2	773.0	844.8
30					284.0	374.6	435.1	510.6	586.1	737.2	812.7	888.2
31					297.7	392.9	456.4	535.7	615.0	773.7	853.0	932.4
32					311.5	411.4	477.9	561.2	644.4	810.8	894.0	977.2
33					325.5	430.1	499.8	587.0	674.1	848.4	935.5	1022.7
34					339.7	449.1	522.0	613.1	704.2	886.5	977.6	1068.8
35					354.1	468.3	544.4	639.6	734.8	925.1	1020.3	1115.5
36					368.6	487.7	567.1	666.4	765.7	964.3	1063.6	1162.9
37					383.2	507.3	590.1	693.5	797.0	1003.9	1107.4	1210.8
38					398.0	527.2	613.3	721.0	828.7	1044.0	1151.7	1259.4
39					412.9	547.2	636.8	748.7	860.7	1084.6	1196.6	1308.5
40					427.9	567.5	660.5	776.8	893.1	1125.7	1241.9	1358.2
41					443.1	587.9	684.5	805.1	925.8	1167.2	1287.8	1408.5
42					458.4	608.6	708.7	833.8	958.9	1209.1	1334.2	1459.4
43					473.9	629.4	733.1	862.7	992.3	1251.5	1381.1	1510.7
44					489.4	650.4	757.7	891.9	1026.0	1294.4	1428.5	1562.7
45						671.6	782.6	921.4	1060.1	1337.6	1476.4	1615.1
46						693.0	807.7	951.1	1094.5	1381.3	1524.7	1668.1
47						714.5	833.0	981.1	1129.2	1425.4	1573.5	1721.6
48						736.2	858.5	1011.3	1164.2	1469.9	1622.8	1775.7
49						758.0	884.2	1041.8	1199.5	1514.8	1672.5	1830.2
50						780.1	910.1	1072.6	1235.1	1560.1	1722.7	1885.2
51						802.3	936.2	1103.6	1271.0	1605.8	1773.3	1940.7
52						824.6	962.5	1134.8	1307.2	1651.9	1824.3	1996.7
53						847.1	989.0	1166.3	1343.7	1698.4	1875.7	2053.1
54						869.7	1015.6	1198.0	1380.4	1745.2	1927.6	2110.0
55						892.5	1042.5	1229.9	1417.4	1792.4	1979.9	2167.4
56						915.4	1069.5	1262.1	1454.7	1840.0	2032.6	2225.2
57						938.4	1096.7	1294.5	1492.3	1887.9	2085.7	2283.5
58								1327.1	1530.1	1936.2	2139.2	2342.3
59								1359.9	1568.2	1984.8	2193.1	2401.4

Table A3. Discharge values of standard Cipolletti weir for different widths.

STANDARD CIPOLLETTI WEIR

Q - Liters per second L- Length in meter $Q=1.85908\ L\ H^{3/2}$

Head (cm)	0.2	0.3	0.5	0.75	1.00	1.30	1.50	1.75	2.00	2.50	2.75	3.00
1	0.3	0.6	0.9	1.4	1.9	2.4	2.8	3.3	3.7	4.6	5.1	5.6
2	0.8	1.6	2.6	3.9	5.3	6.8	7.9	9.2	10.5	13.1	14.5	15.8
3	1.4	2.9	4.8	7.2	9.7	12.6	14.5	16.9	19.3	24.2	26.6	29.0
4	2.2	4.5	7.4	11.2	14.9	19.3	22.3	26.0	29.7	37.2	40.9	44.6
5	3.1	6.2	10.4	15.6	20.8	27.0	31.2	36.4	41.6	52.0	57.2	62.4
6	4.1	8.2	13.7	20.5	27.3	35.5	41.0	47.8	54.6	68.3	75.1	82.0
7		10.3	17.2	25.8	34.4	44.8	51.6	60.3	68.9	86.1	94.7	103.3
8		12.6	21.0	31.5	42.1	54.7	63.1	73.6	84.1	105.2	115.7	126.2
9		15.1	25.1	37.6	50.2	65.3	75.3	87.8	100.4	125.5	138.0	150.6
10		17.6	29.4	44.1	58.8	76.4	88.2	102.9	117.6	147.0	161.7	176.4
11			33.9	50.9	67.8	88.2	101.7	118.7	135.6	169.6	186.5	203.5
12			38.6	58.0	77.3	100.5	115.9	135.2	154.6	193.2	212.5	231.8
13			43.6	65.4	87.1	113.3	130.7	152.5	174.3	217.8	239.6	261.4
14			48.7	73.0	97.4	126.6	146.1	170.4	194.8	243.5	267.8	292.2
15			54.0	81.0	108.0	140.4	162.0	189.0	216.0	270.0	297.0	324.0
16			59.5	89.2	119.0	154.7	178.5	208.2	238.0	297.5	327.2	356.9
17				97.7	130.3	169.4	195.5	228.0	260.6	325.8	358.3	390.9
18				106.5	142.0	184.6	213.0	248.5	283.9	354.9	390.4	425.9
19				115.5	154.0	200.2	231.0	269.4	307.9	384.9	423.4	461.9
20				124.7	166.3	216.2	249.4	291.0	332.6	415.7	457.3	498.8
21					178.9	232.6	268.4	313.1	357.8	447.3	492.0	536.7
22					191.8	249.4	287.8	335.7	383.7	479.6	527.6	575.5
23					205.1	266.6	307.6	358.9	410.1	512.7	563.9	615.2
24					218.6	284.2	327.9	382.5	437.2	546.5	601.1	655.7
25					232.4	302.1	348.6	406.7	464.8	581.0	639.1	697.2
26					246.5	320.4	369.7	431.3	492.9	616.2	677.8	739.4
27						339.1	391.2	456.4	521.6	652.1	717.3	782.5
28						358.1	413.2	482.0	550.9	688.6	757.5	826.3
29						377.4	435.5	508.1	580.7	725.8	798.4	871.0
30						397.1	458.2	534.6	611.0	763.7	840.1	916.4
31							481.3	561.5	641.8	802.2	882.4	962.6
32							504.8	588.9	673.1	841.3	925.5	1009.6
33							528.6	616.7	704.9	881.1	969.2	1057.3
34							552.9	645.0	737.1	921.4	1013.6	1105.7
35							577.4	673.7	769.9	962.4	1058.6	1154.8
36							602.3	702.7	803.1	1003.9	1104.3	1204.7
37								732.2	836.8	1046.0	1150.6	1255.2
38								762.1	871.0	1088.7	1197.6	1306.5
39								792.4	905.6	1132.0	1245.2	1358.4
40								823.0	940.6	1175.8	1293.4	1410.9
41								854.1	976.1	1220.2	1342.2	1464.2
42								885.5	1012.1	1265.1	1391.6	1518.1
43								917.4	1048.4	1310.5	1441.6	1572.6
44								949.5	1085.2	1356.5	1492.1	1627.8
45									1122.4	1403.0	1543.3	1683.6
46									1160.0	1450.0	1595.0	1740.0
47									1198.0	1497.6	1647.3	1797.1
48									1236.5	1545.6	1700.2	1854.7
49									1275.3	1594.2	1753.6	1913.0
50									1314.6	1643.2	1807.5	1971.9
51									1354.2	1692.8	1862.0	2031.3
52									1394.2	1742.8	1917.1	2091.3
53									1434.6	1793.3	1972.6	2152.0
54									1475.4	1844.3	2028.7	2213.1
55									1516.6	1895.8	2085.3	2274.9
56									1558.2	1947.7	2142.5	2337.2
57									1600.1	2000.1	2200.1	2400.1
58									1642.4	2053.0	2258.3	2463.5

Table A4. Discharge values of 90 degree V-Notch (Triangular weir)

90 ° V-NOTCH
$Q=1.38H^{5/2}$

Head (m)	$Q(m^3/s)$	Head (m)	$Q(m^3/s)$	Head (m)	$Q(m^3/s)$
0.050	0.0008	0.155	0.0131	0.260	0.0476
0.550	0.3096	0.160	0.0141	0.265	0.0499
0.600	0.3848	0.165	0.0153	0.270	0.0523
0.065	0.0015	0.170	0.0164	0.275	0.0547
0.070	0.0018	0.175	0.0177	0.280	0.0572
0.075	0.0021	0.180	0.0190	0.285	0.0598
0.080	0.0025	0.185	0.0203	0.290	0.0625
0.085	0.0029	0.190	0.0217	0.295	0.0652
0.090	0.0034	0.195	0.0232	0.300	0.0680
0.095	0.0038	0.200	0.0247	0.305	0.0709
0.100	0.0044	0.205	0.0263	0.310	0.0738
0.105	0.0049	0.210	0.0279	0.315	0.0769
0.110	0.0055	0.215	0.0296	0.320	0.0799
0.115	0.0062	0.220	0.0313	0.325	0.0831
0.120	0.0069	0.225	0.0331	0.330	0.0863
0.125	0.0076	0.230	0.0350	0.335	0.0896
0.130	0.0084	0.235	0.0369	0.340	0.0930
0.135	0.0092	0.240	0.0389	0.345	0.0965
0.140	0.0101	0.245	0.0410		
0.145	0.0110	0.250	0.0431		
0.150	0.0120	0.255	0.0453		

Table A5 Cutthroat flume discharge values under free flow condition.

	Submergence	S < 0.55	S < 0.55	S < 0.58	S < 0.58	S < 0.63	S < 0.63	S < 0.67	S < 0.67
	Flume width	W = 2 in	W = 2 in	W = 4 in	W = 4 in	W = 6 in	W = 6 in	W = 8 in	W = 8 in
Hu (cm)	Hu (ft)	Qf (cfs)	Qf(l/s)	Qf (cfs)	Qf(l/s)	Qf (cfs)	Qf(l/s)	Qf (cfs)	Qf(l/s)
5.00	0.16	0.023	0.66	0.050	1.43	0.078	2.20	0.105	2.98
5.20	0.17	0.025	0.71	0.054	1.54	0.083	2.36	0.113	3.20
5.40	0.18	0.027	0.77	0.058	1.65	0.089	2.53	0.121	3.43
5.60	0.18	0.029	0.82	0.062	1.76	0.095	2.70	0.130	3.67
5.80	0.19	0.031	0.87	0.066	1.88	0.102	2.88	0.138	3.91
6.00	0.20	0.033	0.93	0.071	2.00	0.108	3.07	0.147	4.16
6.20	0.20	0.035	0.99	0.075	2.12	0.115	3.26	0.156	4.42
6.40	0.21	0.037	1.05	0.079	2.25	0.122	3.45	0.165	4.68
6.60	0.22	0.039	1.11	0.084	2.38	0.129	3.65	0.175	4.95
6.80	0.22	0.041	1.17	0.089	2.52	0.136	3.86	0.185	5.23
7.00	0.23	0.044	1.24	0.094	2.65	0.144	4.07	0.195	5.51
7.20	0.24	0.046	1.30	0.099	2.79	0.151	4.28	0.205	5.80
7.40	0.24	0.048	1.37	0.104	2.94	0.159	4.51	0.215	6.10
7.60	0.25	0.051	1.44	0.109	3.09	0.167	4.73	0.226	6.41
7.80	0.26	0.053	1.51	0.114	3.24	0.175	4.96	0.237	6.72
8.00	0.26	0.056	1.58	0.120	3.39	0.184	5.20	0.248	7.04
8.20	0.27	0.058	1.66	0.125	3.55	0.192	5.44	0.260	7.36
8.40	0.28	0.061	1.73	0.131	3.71	0.201	5.68	0.272	7.69
8.60	0.28	0.064	1.81	0.137	3.88	0.210	5.93	0.284	8.03
8.80	0.29	0.067	1.89	0.143	4.04	0.219	6.19	0.296	8.37
9.00	0.30	0.069	1.97	0.149	4.21	0.228	6.45	0.308	8.72
9.20	0.30	0.072	2.05	0.155	4.39	0.237	6.71	0.321	9.08
9.40	0.31	0.075	2.13	0.161	4.56	0.247	6.98	0.334	9.45
9.60	0.31	0.078	2.22	0.168	4.74	0.256	7.26	0.347	9.82
9.80	0.32	0.081	2.30	0.174	4.93	0.266	7.54	0.360	10.19
10.00	0.33	0.084	2.39	0.181	5.11	0.276	7.82	0.373	10.58
10.20	0.33	0.088	2.48	0.187	5.30	0.286	8.11	0.387	10.96
10.40	0.34	0.091	2.57	0.194	5.50	0.297	8.41	0.401	11.36
10.60	0.35	0.094	2.66	0.201	5.69	0.307	8.71	0.415	11.76
10.80	0.35	0.097	2.75	0.208	5.89	0.318	9.01	0.430	12.17
11.00	0.36	0.101	2.85	0.215	6.10	0.329	9.32	0.444	12.59
11.20	0.37	0.104	2.95	0.222	6.30	0.340	9.63	0.459	13.01
11.40	0.37	0.107	3.04	0.230	6.51	0.351	9.95	0.474	13.43
11.60	0.38	0.111	3.14	0.237	6.72	0.363	10.27	0.490	13.87
11.80	0.39	0.115	3.24	0.245	6.94	0.374	10.60	0.505	14.31
12.00	0.39	0.118	3.35	0.253	7.15	0.386	10.93	0.521	14.75
12.20	0.40	0.122	3.45	0.260	7.37	0.398	11.26	0.537	15.21
12.40	0.41	0.126	3.55	0.268	7.60	0.410	11.61	0.553	15.66
12.60	0.41	0.129	3.66	0.276	7.83	0.422	11.95	0.569	16.13
12.80	0.42	0.133	3.77	0.284	8.06	0.434	12.30	0.586	16.60
13.00	0.43	0.137	3.88	0.293	8.29	0.447	12.66	0.603	17.07
13.20	0.43	0.141	3.99	0.301	8.52	0.460	13.02	0.620	17.56

		S < 0.55	S < 0.55	S < 0.58	S < 0.58	S < 0.63	S < 0.63	S < 0.67	S < 0.67
		W = 2 in	W = 2 in	W = 4 in	W = 4 in	W = 6 in	W = 6 in	W = 8 in	W = 8 in
Hu (cm)	Hu (ft)	Qf (cfs)	Qf(l/s)	Qf (cfs)	Qf(l/s)	Qf (cfs)	Qf(l/s)	Qf (cfs)	Qf(l/s)
13.40	0.44	0.145	4.10	0.309	8.76	0.472	13.38	0.637	18.05
13.60	0.45	0.149	4.22	0.318	9.01	0.485	13.75	0.655	18.54
13.80	0.45	0.153	4.33	0.327	9.25	0.499	14.12	0.672	19.04
14.00	0.46	0.157	4.45	0.335	9.50	0.512	14.50	0.690	19.55
14.20	0.47	0.161	4.57	0.344	9.75	0.525	14.88	0.708	20.06
14.40	0.47	0.165	4.69	0.353	10.01	0.539	15.27	0.727	20.58
14.60	0.48	0.170	4.81	0.362	10.26	0.553	15.66	0.745	21.11
14.80	0.49	0.174	4.93	0.372	10.52	0.567	16.05	0.764	21.64
15.00	0.49	0.178	5.05	0.381	10.79	0.581	16.45	0.783	22.17
15.20	0.50	0.183	5.18	0.390	11.05	0.595	16.86	0.802	22.72
15.40	0.51	0.187	5.30	0.400	11.32	0.610	17.26	0.822	23.27
15.60	0.51	0.192	5.43	0.409	11.59	0.624	17.68	0.841	23.82
15.80	0.52	0.196	5.56	0.419	11.87	0.639	18.10	0.861	24.38
16.00	0.52	0.201	5.69	0.429	12.15	0.654	18.52	0.881	24.95
16.20	0.53	0.206	5.82	0.439	12.43	0.669	18.94	0.901	25.52
16.40	0.54	0.210	5.96	0.449	12.71	0.684	19.38	0.922	26.10
16.60	0.54	0.215	6.09	0.459	13.00	0.700	19.81	0.942	26.68
16.80	0.55	0.220	6.23	0.469	13.29	0.715	20.25	0.963	27.27
17.00	0.56	0.225	6.37	0.479	13.58	0.731	20.69	0.984	27.87
17.20	0.56	0.230	6.51	0.490	13.87	0.747	21.14	1.005	28.47
17.40	0.57	0.235	6.65	0.500	14.17	0.763	21.60	1.027	29.08
17.60	0.58	0.240	6.79	0.511	14.47	0.779	22.05	1.048	29.69
17.80	0.58	0.245	6.93	0.522	14.78	0.795	22.51	1.070	30.31
18.00	0.59	0.250	7.08	0.533	15.08	0.811	22.98	1.092	30.93
18.20	0.60	0.255	7.22	0.544	15.39	0.828	23.45	1.115	31.56
18.40	0.60	0.260	7.37	0.555	15.71	0.845	23.93	1.137	32.20
18.60	0.61	0.265	7.52	0.566	16.02	0.862	24.40	1.160	32.84
18.80	0.62	0.271	7.67	0.577	16.34	0.879	24.89	1.183	33.49
19.00	0.62	0.276	7.82	0.588	16.66	0.896	25.37	1.206	34.14
19.20	0.63	0.281	7.97	0.600	16.99	0.913	25.87	1.229	34.80
19.40	0.64	0.287	8.13	0.611	17.31	0.931	26.36	1.252	35.47
19.60	0.64	0.292	8.28	0.623	17.64	0.949	26.86	1.276	36.14
19.80	0.65	0.298	8.44	0.635	17.98	0.966	27.37	1.300	36.82
20.00	0.66	0.304	8.60	0.647	18.31	0.984	27.88	1.324	37.50
20.20	0.66	0.309	8.76	0.659	18.65	1.002	28.39	1.348	38.18
20.40	0.67	0.315	8.92	0.671	18.99	1.021	28.91	1.373	38.88
20.60	0.68	0.321	9.08	0.683	19.34	1.039	29.43	1.397	39.58
20.80	0.68	0.326	9.24	0.695	19.68	1.058	29.95	1.422	40.28
21.00	0.69	0.332	9.41	0.707	20.03	1.076	30.48	1.447	40.99
21.20	0.70	0.338	9.57	0.720	20.38	1.095	31.02	1.473	41.71
21.40	0.70	0.344	9.74	0.732	20.74	1.114	31.56	1.498	42.43
21.60	0.71	0.350	9.91	0.745	21.10	1.133	32.10	1.524	43.15
21.80	0.72	0.356	10.08	0.758	21.46	1.153	32.65	1.550	43.89

		S < 0.55	S < 0.55	S < 0.58	S < 0.58	S < 0.63	S < 0.63	S < 0.67	S < 0.67
		W = 2 in	W = 2 in	W = 4 in	W = 4 in	W = 6 in	W = 6 in	W = 8 in	W = 8 in
Hu (cm)	Hu (ft)	Qf (cfs)	Qf(l/s)	Qf (cfs)	Qf(l/s)	Qf (cfs)	Qf(l/s)	Qf (cfs)	Qf(l/s)
22.00	0.72	0.362	10.25	0.771	21.82	1.172	33.20	1.576	44.63
22.20	0.73	0.368	10.42	0.784	22.19	1.192	33.75	1.602	45.37
22.40	0.73	0.374	10.60	0.797	22.56	1.212	34.31	1.628	46.12
22.60	0.74	0.380	10.77	0.810	22.93	1.232	34.88	1.655	46.87
22.80	0.75	0.387	10.95	0.823	23.30	1.252	35.44	1.682	47.63
23.00	0.75	0.393	11.13	0.836	23.68	1.272	36.02	1.709	48.40
23.20	0.76	0.399	11.31	0.850	24.06	1.292	36.59	1.736	49.17
23.40	0.77	0.406	11.49	0.863	24.45	1.313	37.17	1.764	49.95
23.60	0.77	0.412	11.67	0.877	24.83	1.333	37.76	1.791	50.73
23.80	0.78	0.419	11.85	0.891	25.22	1.354	38.35	1.819	51.52
24.00	0.79	0.425	12.04	0.904	25.61	1.375	38.94	1.847	52.31
24.20	0.79	0.432	12.22	0.918	26.01	1.396	39.54	1.875	53.11
24.40	0.80	0.438	12.41	0.932	26.40	1.417	40.14	1.904	53.91
24.60	0.81	0.445	12.60	0.946	26.80	1.439	40.74	1.932	54.72
24.80	0.81	0.452	12.79	0.961	27.20	1.460	41.35	1.961	55.54
25.00	0.82	0.458	12.98	0.975	27.61	1.482	41.96	1.990	56.36
25.20	0.83	0.465	13.17	0.989	28.02	1.504	42.58	2.019	57.18
25.40	0.83	0.472	13.37	1.004	28.43	1.526	43.20	2.049	58.02
25.60	0.84	0.479	13.56	1.018	28.84	1.548	43.83	2.078	58.85
25.80	0.85	0.486	13.76	1.033	29.26	1.570	44.46	2.108	59.70
26.00	0.85	0.493	13.96	1.048	29.68	1.592	45.09	2.138	60.54
26.20	0.86	0.500	14.15	1.063	30.10	1.615	45.73	2.168	61.40
26.40	0.87	0.507	14.35	1.078	30.52	1.637	46.37	2.198	62.25
26.60	0.87	0.514	14.56	1.093	30.95	1.660	47.02	2.229	63.12
26.80	0.88	0.521	14.76	1.108	31.38	1.683	47.67	2.259	63.99
27.00	0.89	0.528	14.96	1.123	31.81	1.706	48.32	2.290	64.86
27.20	0.89	0.536	15.17	1.139	32.24	1.730	48.98	2.321	65.74
27.40	0.90	0.543	15.38	1.154	32.68	1.753	49.64	2.353	66.63
27.60	0.91	0.550	15.58	1.170	33.12	1.776	50.31	2.384	67.52
27.80	0.91	0.558	15.79	1.185	33.57	1.800	50.98	2.416	68.41
28.00	0.92	0.565	16.00	1.201	34.01	1.824	51.65	2.448	69.32
28.20	0.93	0.573	16.21	1.217	34.46	1.848	52.33	2.480	70.22
28.40	0.93	0.580	16.43	1.233	34.91	1.872	53.01	2.512	71.14
28.60	0.94	0.588	16.64	1.249	35.36	1.896	53.70	2.544	72.05
28.80	0.94	0.595	16.86	1.265	35.82	1.921	54.39	2.577	72.98
29.00	0.95	0.603	17.07	1.281	36.28	1.945	55.09	2.610	73.90
29.20	0.96	0.611	17.29	1.297	36.74	1.970	55.78	2.643	74.84
29.40	0.96	0.618	17.51	1.314	37.21	1.995	56.49	2.676	75.78
29.60	0.97	0.626	17.73	1.330	37.67	2.019	57.19	2.709	76.72
29.80	0.98	0.634	17.95	1.347	38.14	2.045	57.90	2.743	77.67
30.00	0.98	0.642	18.18	1.364	38.62	2.070	58.62	2.776	78.62
30.20	0.99	0.650	18.40	1.380	39.09	2.095	59.34	2.810	79.58
30.40	1.00	0.658	18.63	1.397	39.57	2.121	60.06	2.844	80.55

		S < 0.55	S < 0.55	S < 0.58	S < 0.58	S < 0.63	S < 0.63	S < 0.67	S < 0.67
		W = 2 in	W = 2 in	W = 4 in	W = 4 in	W = 6 in	W = 6 in	W = 8 in	W = 8 in
Hu (cm)	Hu (ft)	Qf (cfs)	Qf(l/s)	Qf (cfs)	Qf(l/s)	Qf (cfs)	Qf(l/s)	Qf (cfs)	Qf(l/s)
30.60	1.00	0.666	18.86	1.414	40.05	2.146	60.78	2.878	81.52
30.80	1.01	0.674	19.08	1.431	40.53	2.172	61.51	2.913	82.49
31.00	1.02	0.682	19.31	1.448	41.02	2.198	62.25	2.948	83.47
31.20	1.02	0.690	19.54	1.466	41.50	2.224	62.99	2.982	84.46
31.40	1.03	0.698	19.78	1.483	42.00	2.250	63.73	3.017	85.45
31.60	1.04	0.707	20.01	1.500	42.49	2.277	64.47	3.053	86.45
31.80	1.04	0.715	20.24	1.518	42.99	2.303	65.22	3.088	87.45
32.00	1.05	0.723	20.48	1.535	43.48	2.330	65.98	3.123	88.46
32.20	1.06	0.732	20.72	1.553	43.99	2.356	66.74	3.159	89.47
32.40	1.06	0.740	20.95	1.571	44.49	2.383	67.50	3.195	90.49
32.60	1.07	0.748	21.19	1.589	45.00	2.410	68.26	3.231	91.51
32.80	1.08	0.757	21.43	1.607	45.51	2.438	69.03	3.268	92.54
33.00	1.08	0.765	21.68	1.625	46.02	2.465	69.81	3.304	93.57
33.20	1.09	0.774	21.92	1.643	46.53	2.492	70.58	3.341	94.61
33.40	1.10	0.783	22.16	1.661	47.05	2.520	71.37	3.378	95.65
33.60	1.10	0.791	22.41	1.680	47.57	2.548	72.15	3.415	96.70
33.80	1.11	0.800	22.66	1.698	48.09	2.576	72.94	3.452	97.75
34.00	1.12	0.809	22.91	1.717	48.62	2.604	73.73	3.489	98.81
34.20	1.12	0.818	23.16	1.735	49.14	2.632	74.53	3.527	99.88
34.40	1.13	0.826	23.41	1.754	49.67	2.660	75.33	3.564	100.94
34.60	1.14	0.835	23.66	1.773	50.21	2.688	76.14	3.602	102.02
34.80	1.14	0.844	23.91	1.792	50.74	2.717	76.94	3.640	103.10
35.00	1.15	0.853	24.17	1.811	51.28	2.746	77.76	3.679	104.18
35.20	1.15	0.862	24.42	1.830	51.82	2.774	78.57	3.717	105.27
35.40	1.16	0.871	24.68	1.849	52.36	2.803	79.39	3.756	106.37
35.60	1.17	0.881	24.94	1.868	52.91	2.833	80.22	3.795	107.47
35.80	1.17	0.890	25.20	1.888	53.46	2.862	81.05	3.834	108.57
36.00	1.18	0.899	25.46	1.907	54.01	2.891	81.88	3.873	109.68
36.20	1.19	0.908	25.72	1.927	54.56	2.921	82.71	3.912	110.80
36.40	1.19	0.917	25.98	1.946	55.12	2.950	83.55	3.952	111.92
36.60	1.20	0.927	26.25	1.966	55.68	2.980	84.40	3.992	113.04
36.80	1.21	0.936	26.51	1.986	56.24	3.010	85.24	4.032	114.17
37.00	1.21	0.946	26.78	2.006	56.80	3.040	86.09	4.072	115.31
37.20	1.22	0.955	27.05	2.026	57.37	3.070	86.95	4.112	116.45
37.40	1.23	0.965	27.31	2.046	57.94	3.101	87.81	4.152	117.60
37.60	1.23	0.974	27.59	2.066	58.51	3.131	88.67	4.193	118.75
37.80	1.24	0.984	27.86	2.086	59.08	3.162	89.54	4.234	119.90
38.00	1.25	0.993	28.13	2.107	59.66	3.192	90.41	4.275	121.06
38.20	1.25	1.003	28.40	2.127	60.24	3.223	91.28	4.316	122.23
38.40	1.26	1.013	28.68	2.148	60.82	3.254	92.16	4.357	123.40
38.60	1.27	1.022	28.96	2.168	61.40	3.285	93.04	4.399	124.58
38.80	1.27	1.032	29.23	2.189	61.99	3.317	93.93	4.441	125.76
39.00	1.28	1.042	29.51	2.210	62.58	3.348	94.82	4.483	126.94

Table A6: Discharge Correction factors for different degrees of submerged flow conditions of Cutthroat flumes.

	W = 2 IN	W = 4 IN	W = 6 IN	W = 8 IN
S	Qs/Qf	Qs/Qf	Qs/Qf	Qs/Qf
0.560	0.997			
0.570	0.995			
0.580	0.993			
0.590	0.990			
0.600	0.986	0.999		
0.610	0.982	0.998		
0.620	0.978	0.996		
0.630	0.973	0.994		
0.640	0.967	0.992	0.996	
0.650	0.962	0.989	0.995	
0.660	0.955	0.986	0.994	
0.670	0.948	0.982	0.992	
0.680	0.941	0.978	0.990	
0.690	0.933	0.973	0.988	0.999
0.700	0.925	0.967	0.985	0.998
0.710	0.915	0.961	0.981	0.997
0.720	0.906	0.955	0.977	0.995
0.730	0.896	0.948	0.972	0.992
0.740	0.885	0.940	0.967	0.989
0.750	0.873	0.931	0.961	0.986
0.760	0.861	0.922	0.954	0.981
0.770	0.848	0.912	0.947	0.976
0.780	0.834	0.902	0.939	0.971
0.790	0.820	0.890	0.930	0.964
0.800	0.804	0.878	0.920	0.957
0.810	0.788	0.865	0.909	0.949
0.820	0.771	0.851	0.898	0.940
0.830	0.753	0.835	0.885	0.931
0.840	0.734	0.819	0.872	0.920
0.850	0.714	0.802	0.857	0.908
0.855	0.704	0.793	0.849	0.901
0.860	0.693	0.783	0.841	0.894
0.865	0.682	0.773	0.832	0.887
0.870	0.670	0.763	0.824	0.880
0.875	0.658	0.752	0.814	0.872
0.880	0.646	0.741	0.804	0.863
0.885	0.633	0.730	0.794	0.855
0.890	0.620	0.718	0.784	0.845
0.895	0.607	0.706	0.773	0.836
0.900	0.593	0.693	0.761	0.825
0.905	0.579	0.679	0.749	0.814
0.910	0.564	0.665	0.736	0.803
0.915	0.548	0.651	0.722	0.791
0.920	0.532	0.635	0.708	0.778
0.925	0.516	0.619	0.693	0.764
0.930	0.498	0.602	0.677	0.750
0.935	0.480	0.584	0.660	0.734
0.940	0.461	0.565	0.642	0.717
0.945	0.441	0.545	0.623	0.699
0.950	0.420	0.524	0.602	0.680
0.955	0.398	0.501	0.580	0.659
0.960	0.375	0.477	0.556	0.636
0.965	0.349	0.450	0.529	0.610
0.970	0.322	0.421	0.500	0.581
0.975	0.293	0.389	0.467	0.549
0.980	0.260	0.353	0.429	0.511
0.985	0.223	0.310	0.385	0.465
0.990	0.179	0.259	0.329	0.408
0.995	0.123	0.189	0.252	0.324

Note: S =submergence ratio = Hu/Hd
Qf = free flow, Qs =submerged flow

ANNEXURE B: DESIGN OF LINED PARABOLIC CANALS

Table B1. Hydraulic design tables for parabolic lined section for discharge of 20 L/S.

Energy Slope ---	Manning's n	Flow Depth, y (m)	Area, A (m^2)	Flow Velocity (m/s)	Free Board, FB (m)	Total Depth, (m)	Top Width, TW (m)	Bank Width, BW (m)	Pad Width (m)
0.0001	0.014	0.25	0.11	0.18	0.15	0.40	1.12	0.3	1.72
0.0002	0.014	0.22	0.09	0.23	0.15	0.37	1.03	0.3	1.63
0.0004	0.014	0.19	0.07	0.30	0.15	0.34	0.96	0.3	1.56
0.0006	0.014	0.18	0.06	0.34	0.15	0.33	0.92	0.3	1.52
0.0008	0.014	0.17	0.05	0.38	0.15	0.32	0.89	0.3	1.49
0.0010	0.014	0.16	0.05	0.42	0.15	0.31	0.87	0.3	1.47
0.0012	0.014	0.15	0.04	0.45	0.15	0.30	0.86	0.3	1.46
0.0014	0.014	0.15	0.04	0.47	0.15	0.30	0.85	0.3	1.45
0.0016	0.014	0.15	0.04	0.50	0.15	0.30	0.84	0.3	1.44
0.0018	0.014	0.14	0.04	0.52	0.15	0.29	0.83	0.3	1.43
0.0020	0.014	0.14	0.04	0.54	0.15	0.29	0.82	0.3	1.42
0.0022	0.014	0.14	0.04	0.56	0.15	0.29	0.81	0.3	1.41
0.0024	0.014	0.14	0.03	0.58	0.15	0.29	0.81	0.3	1.41
0.0026	0.014	0.13	0.03	0.60	0.15	0.28	0.80	0.3	1.40
0.0028	0.014	0.13	0.03	0.61	0.15	0.28	0.80	0.3	1.40
0.0030	0.014	0.13	0.03	0.63	0.15	0.28	0.79	0.3	1.39
0.0032	0.014	0.13	0.03	0.65	0.15	0.28	0.79	0.3	1.39
0.0034	0.014	0.13	0.03	0.66	0.15	0.28	0.78	0.3	1.38
0.0036	0.014	0.13	0.03	0.68	0.15	0.28	0.78	0.3	1.38
0.0038	0.014	0.12	0.03	0.69	0.15	0.27	0.78	0.3	1.38
0.0040	0.014	0.12	0.03	0.70	0.15	0.27	0.77	0.3	1.37
0.0042	0.014	0.12	0.03	0.72	0.15	0.27	0.77	0.3	1.37
0.0044	0.014	0.12	0.03	0.73	0.15	0.27	0.77	0.3	1.37
0.0046	0.014	0.12	0.03	0.74	0.15	0.27	0.76	0.3	1.36
0.0048	0.014	0.12	0.03	0.75	0.15	0.27	0.76	0.3	1.36
0.0050	0.014	0.12	0.03	0.76	0.15	0.27	0.76	0.3	1.36
0.0052	0.014	0.12	0.03	0.78	0.15	0.27	0.76	0.3	1.36
0.0054	0.014	0.12	0.03	0.79	0.15	0.27	0.75	0.3	1.35
0.0056	0.014	0.12	0.03	0.80	0.15	0.27	0.75	0.3	1.35
0.0058	0.014	0.11	0.02	0.81	0.15	0.26	0.75	0.3	1.35
0.0060	0.014	0.11	0.02	0.82	0.15	0.26	0.75	0.3	1.35
0.0062	0.014	0.11	0.02	0.83	0.15	0.26	0.74	0.3	1.34
0.0064	0.014	0.11	0.02	0.84	0.15	0.26	0.74	0.3	1.34
0.0066	0.014	0.11	0.02	0.85	0.15	0.26	0.74	0.3	1.34
0.0068	0.014	0.11	0.02	0.86	0.15	0.26	0.74	0.3	1.34
0.0070	0.014	0.11	0.02	0.87	0.15	0.26	0.74	0.3	1.34
0.0072	0.014	0.11	0.02	0.88	0.15	0.26	0.74	0.3	1.34
0.0074	0.014	0.11	0.02	0.88	0.15	0.26	0.73	0.3	1.33

Table B2. Hydraulic design tables for parabolic lined section for discharge of 30 L/S.

Energy Slope ---	Manning's n	Flow Depth, y (m)	Area, A (m^2)	Flow Velocity (m/s)	Free Board, FB (m)	Total Depth, (m)	Top Width, TW (m)	Bank Width BW (m)	Pad Width (m)
0.0001	0.014	0.29	0.15	0.19	0.15	0.44	1.23	0.3	1.83
0.0002	0.014	0.25	0.12	0.25	0.15	0.40	1.13	0.3	1.73
0.0004	0.014	0.22	0.09	0.33	0.15	0.37	1.05	0.3	1.65
0.0006	0.014	0.20	0.08	0.38	0.15	0.35	1.00	0.3	1.60
0.0008	0.014	0.19	0.07	0.43	0.15	0.34	0.97	0.3	1.57
0.0010	0.014	0.19	0.06	0.46	0.15	0.34	0.95	0.3	1.55
0.0012	0.014	0.18	0.06	0.50	0.15	0.33	0.93	0.3	1.53
0.0014	0.014	0.17	0.06	0.52	0.15	0.32	0.92	0.3	1.52
0.0016	0.014	0.17	0.05	0.55	0.15	0.32	0.90	0.3	1.50
0.0018	0.014	0.17	0.05	0.58	0.15	0.32	0.89	0.3	1.49
0.0020	0.014	0.16	0.05	0.60	0.15	0.31	0.88	0.3	1.48
0.0022	0.014	0.16	0.05	0.62	0.15	0.31	0.88	0.3	1.48
0.0024	0.014	0.16	0.05	0.64	0.15	0.31	0.87	0.3	1.47
0.0026	0.014	0.15	0.05	0.66	0.15	0.30	0.86	0.3	1.46
0.0028	0.014	0.15	0.04	0.68	0.15	0.30	0.86	0.3	1.46
0.0030	0.014	0.15	0.04	0.70	0.15	0.30	0.85	0.3	1.45
0.0032	0.014	0.15	0.04	0.72	0.15	0.30	0.85	0.3	1.45
0.0034	0.014	0.15	0.04	0.73	0.15	0.30	0.84	0.3	1.44
0.0036	0.014	0.15	0.04	0.75	0.15	0.30	0.84	0.3	1.44
0.0038	0.014	0.14	0.04	0.76	0.15	0.29	0.83	0.3	1.43
0.0040	0.014	0.14	0.04	0.78	0.15	0.29	0.83	0.3	1.43
0.0042	0.014	0.14	0.04	0.79	0.15	0.29	0.83	0.3	1.43
0.0044	0.014	0.14	0.04	0.81	0.15	0.29	0.82	0.3	1.42
0.0046	0.014	0.14	0.04	0.82	0.15	0.29	0.82	0.3	1.42
0.0048	0.014	0.14	0.04	0.83	0.15	0.29	0.82	0.3	1.42
0.0050	0.014	0.14	0.04	0.85	0.15	0.29	0.81	0.3	1.41
0.0052	0.014	0.14	0.03	0.86	0.15	0.29	0.81	0.3	1.41
0.0054	0.014	0.14	0.03	0.87	0.15	0.29	0.81	0.3	1.41
0.0056	0.014	0.13	0.03	0.88	0.15	0.28	0.80	0.3	1.40
0.0058	0.014	0.13	0.03	0.89	0.15	0.28	0.80	0.3	1.40
0.0060	0.014	0.13	0.03	0.91	0.15	0.28	0.80	0.3	1.40
0.0062	0.014	0.13	0.03	0.92	0.15	0.28	0.80	0.3	1.40
0.0064	0.014	0.13	0.03	0.93	0.15	0.28	0.79	0.3	1.39
0.0066	0.014	0.13	0.03	0.94	0.15	0.28	0.79	0.3	1.39
0.0068	0.014	0.13	0.03	0.95	0.15	0.28	0.79	0.3	1.39
0.0070	0.014	0.13	0.03	0.96	0.15	0.28	0.79	0.3	1.39
0.0072	0.014	0.13	0.03	0.97	0.15	0.28	0.79	0.3	1.39
0.0074	0.014	0.13	0.03	0.98	0.15	0.28	0.78	0.3	1.38

Table B3. Hydraulic design tables for parabolic lined section for Discharge of 40 L/S.

Energy Slope ---	Manning's n	Flow Depth y (m)	Area, A (m^2)	Flow Velocity (m/s)	Free Board, FB (m)	Total Depth, (m)	Top Width, TW (m)	Bank Width BW (m)	Pad Width (m)
0.0001	0.014	0.32	0.19	0.21	0.15	0.47	1.32	0.3	1.92
0.0002	0.014	0.28	0.15	0.27	0.15	0.43	1.21	0.3	1.81
0.0004	0.014	0.25	0.11	0.35	0.15	0.40	1.12	0.3	1.72
0.0006	0.014	0.23	0.10	0.41	0.15	0.38	1.07	0.3	1.67
0.0008	0.014	0.22	0.09	0.46	0.15	0.37	1.03	0.3	1.63
0.0010	0.014	0.21	0.08	0.50	0.15	0.36	1.01	0.3	1.61
0.0012	0.014	0.20	0.08	0.53	0.15	0.35	0.99	0.3	1.59
0.0014	0.014	0.19	0.07	0.56	0.15	0.34	0.97	0.3	1.57
0.0016	0.014	0.19	0.07	0.59	0.15	0.34	0.96	0.3	1.56
0.0018	0.014	0.18	0.06	0.62	0.15	0.33	0.95	0.3	1.55
0.0020	0.014	0.18	0.06	0.64	0.15	0.33	0.94	0.3	1.54
0.0022	0.014	0.18	0.06	0.67	0.15	0.33	0.93	0.3	1.53
0.0024	0.014	0.18	0.06	0.69	0.15	0.33	0.92	0.3	1.52
0.0026	0.014	0.17	0.06	0.71	0.15	0.32	0.91	0.3	1.51
0.0028	0.014	0.17	0.05	0.73	0.15	0.32	0.91	0.3	1.51
0.0030	0.014	0.17	0.05	0.75	0.15	0.32	0.90	0.3	1.50
0.0032	0.014	0.17	0.05	0.77	0.15	0.32	0.89	0.3	1.49
0.0034	0.014	0.16	0.05	0.79	0.15	0.31	0.89	0.3	1.49
0.0036	0.014	0.16	0.05	0.80	0.15	0.31	0.88	0.3	1.48
0.0038	0.014	0.16	0.05	0.82	0.15	0.31	0.88	0.3	1.48
0.0040	0.014	0.16	0.05	0.84	0.15	0.31	0.87	0.3	1.47
0.0042	0.014	0.16	0.05	0.85	0.15	0.31	0.87	0.3	1.47
0.0044	0.014	0.16	0.05	0.87	0.15	0.31	0.87	0.3	1.47
0.0046	0.014	0.16	0.05	0.88	0.15	0.31	0.86	0.3	1.46
0.0048	0.014	0.15	0.04	0.89	0.15	0.30	0.86	0.3	1.46
0.0050	0.014	0.15	0.04	0.91	0.15	0.30	0.86	0.3	1.46
0.0052	0.014	0.15	0.04	0.92	0.15	0.30	0.85	0.3	1.45
0.0054	0.014	0.15	0.04	0.94	0.15	0.30	0.85	0.3	1.45
0.0056	0.014	0.15	0.04	0.95	0.15	0.30	0.85	0.3	1.45
0.0058	0.014	0.15	0.04	0.96	0.15	0.30	0.84	0.3	1.44
0.0060	0.014	0.15	0.04	0.97	0.15	0.30	0.84	0.3	1.44
0.0062	0.014	0.15	0.04	0.98	0.15	0.30	0.84	0.3	1.44
0.0064	0.014	0.15	0.04	1.00	0.15	0.30	0.84	0.3	1.44
0.0066	0.014	0.14	0.04	1.01	0.15	0.29	0.83	0.3	1.43
0.0068	0.014	0.14	0.04	1.02	0.15	0.29	0.83	0.3	1.43
0.0070	0.014	0.14	0.04	1.03	0.15	0.29	0.83	0.3	1.43
0.0072	0.014	0.14	0.04	1.04	0.15	0.29	0.83	0.3	1.43
0.0074	0.014	0.14	0.04	1.05	0.15	0.29	0.83	0.3	1.43

Table B4. Hydraulic design tables for parabolic lined section for Discharge of 50 L/S.

Energy Slope ---	Manning's n	Flow Depth y (m)	Area, A (m^2)	Flow Velocity (m/s)	Free Board, FB (m)	Total Depth, (m)	Top Width, TW (m)	Bank Width BW (m)	Pad Width (m)
0.0001	0.014	0.35	0.23	0.22	0.15	0.50	1.40	0.3	2.00
0.0002	0.014	0.30	0.17	0.29	0.15	0.45	1.28	0.3	1.88
0.0004	0.014	0.27	0.13	0.37	0.15	0.42	1.18	0.3	1.78
0.0006	0.014	0.25	0.12	0.43	0.15	0.40	1.12	0.3	1.72
0.0008	0.014	0.23	0.10	0.48	0.15	0.38	1.09	0.3	1.69
0.0010	0.014	0.22	0.10	0.53	0.15	0.37	1.06	0.3	1.66
0.0012	0.014	0.22	0.09	0.56	0.15	0.37	1.04	0.3	1.64
0.0014	0.014	0.21	0.08	0.60	0.15	0.36	1.02	0.3	1.62
0.0016	0.014	0.21	0.08	0.63	0.15	0.36	1.01	0.3	1.61
0.0018	0.014	0.20	0.08	0.65	0.15	0.35	0.99	0.3	1.59
0.0020	0.014	0.20	0.07	0.68	0.15	0.35	0.98	0.3	1.58
0.0022	0.014	0.19	0.07	0.71	0.15	0.34	0.97	0.3	1.57
0.0024	0.014	0.19	0.07	0.73	0.15	0.34	0.96	0.3	1.56
0.0026	0.014	0.19	0.07	0.75	0.15	0.34	0.96	0.3	1.56
0.0028	0.014	0.19	0.06	0.77	0.15	0.34	0.95	0.3	1.55
0.0030	0.014	0.18	0.06	0.79	0.15	0.33	0.94	0.3	1.54
0.0032	0.014	0.18	0.06	0.81	0.15	0.33	0.94	0.3	1.54
0.0034	0.014	0.18	0.06	0.83	0.15	0.33	0.93	0.3	1.53
0.0036	0.014	0.18	0.06	0.85	0.15	0.33	0.92	0.3	1.52
0.0038	0.014	0.17	0.06	0.87	0.15	0.32	0.92	0.3	1.52
0.0040	0.014	0.17	0.06	0.88	0.15	0.32	0.91	0.3	1.51
0.0042	0.014	0.17	0.06	0.90	0.15	0.32	0.91	0.3	1.51
0.0044	0.014	0.17	0.05	0.92	0.15	0.32	0.91	0.3	1.51
0.0046	0.014	0.17	0.05	0.93	0.15	0.32	0.90	0.3	1.50
0.0048	0.014	0.17	0.05	0.95	0.15	0.32	0.90	0.3	1.50
0.0050	0.014	0.17	0.05	0.96	0.15	0.32	0.89	0.3	1.49
0.0052	0.014	0.16	0.05	0.97	0.15	0.31	0.89	0.3	1.49
0.0054	0.014	0.16	0.05	0.99	0.15	0.31	0.89	0.3	1.49
0.0056	0.014	0.16	0.05	1.00	0.15	0.31	0.88	0.3	1.48
0.0058	0.014	0.16	0.05	1.02	0.15	0.31	0.88	0.3	1.48
0.0060	0.014	0.16	0.05	1.03	0.15	0.31	0.88	0.3	1.48
0.0062	0.014	0.16	0.05	1.04	0.15	0.31	0.88	0.3	1.48
0.0064	0.014	0.16	0.05	1.05	0.15	0.31	0.87	0.3	1.47
0.0066	0.014	0.16	0.05	1.07	0.15	0.31	0.87	0.3	1.47
0.0068	0.014	0.16	0.05	1.08	0.15	0.31	0.87	0.3	1.47
0.0070	0.014	0.16	0.05	1.09	0.15	0.31	0.87	0.3	1.47
0.0072	0.014	0.15	0.05	1.10	0.15	0.30	0.86	0.3	1.46
0.0074	0.014	0.15	0.04	1.11	0.15	0.30	0.86	0.3	1.46

Table B5. Hydraulic design tables for parabolic lined section for Discharge of 60 L/S.

Energy Slope ---	Manning's n	Flow Depth y (m)	Area, A (m^2)	Flow Velocity (m/s)	Free Board, FB (m)	Total Depth, (m)	Top Width, TW (m)	Bank Width BW (m)	Pad Width (m)
0.0001	0.014	0.37	0.26	0.23	0.15	0.52	1.47	0.3	2.07
0.0002	0.014	0.32	0.20	0.30	0.15	0.47	1.34	0.3	1.94
0.0004	0.014	0.29	0.15	0.39	0.15	0.44	1.23	0.3	1.83
0.0006	0.014	0.26	0.13	0.45	0.15	0.41	1.17	0.3	1.77
0.0008	0.014	0.25	0.12	0.51	0.15	0.40	1.13	0.3	1.73
0.0010	0.014	0.24	0.11	0.55	0.15	0.39	1.10	0.3	1.70
0.0012	0.014	0.23	0.10	0.59	0.15	0.38	1.08	0.3	1.68
0.0014	0.014	0.23	0.10	0.62	0.15	0.38	1.06	0.3	1.66
0.0016	0.014	0.22	0.09	0.66	0.15	0.37	1.05	0.3	1.65
0.0018	0.014	0.22	0.09	0.69	0.15	0.37	1.03	0.3	1.63
0.0020	0.014	0.21	0.08	0.71	0.15	0.36	1.02	0.3	1.62
0.0022	0.014	0.21	0.08	0.74	0.15	0.36	1.01	0.3	1.61
0.0024	0.014	0.20	0.08	0.76	0.15	0.35	1.00	0.3	1.60
0.0026	0.014	0.20	0.08	0.79	0.15	0.35	0.99	0.3	1.59
0.0028	0.014	0.20	0.07	0.81	0.15	0.35	0.99	0.3	1.59
0.0030	0.014	0.20	0.07	0.83	0.15	0.35	0.98	0.3	1.58
0.0032	0.014	0.19	0.07	0.85	0.15	0.34	0.97	0.3	1.57
0.0034	0.014	0.19	0.07	0.87	0.15	0.34	0.97	0.3	1.57
0.0036	0.014	0.19	0.07	0.89	0.15	0.34	0.96	0.3	1.56
0.0038	0.014	0.19	0.07	0.91	0.15	0.34	0.95	0.3	1.55
0.0040	0.014	0.19	0.06	0.92	0.15	0.34	0.95	0.3	1.55
0.0042	0.014	0.18	0.06	0.94	0.15	0.33	0.94	0.3	1.54
0.0044	0.014	0.18	0.06	0.96	0.15	0.33	0.94	0.3	1.54
0.0046	0.014	0.18	0.06	0.97	0.15	0.33	0.94	0.3	1.54
0.0048	0.014	0.18	0.06	0.99	0.15	0.33	0.93	0.3	1.53
0.0050	0.014	0.18	0.06	1.01	0.15	0.33	0.93	0.3	1.53
0.0052	0.014	0.18	0.06	1.02	0.15	0.33	0.92	0.3	1.52
0.0054	0.014	0.18	0.06	1.03	0.15	0.33	0.92	0.3	1.52
0.0056	0.014	0.17	0.06	1.05	0.15	0.32	0.92	0.3	1.52
0.0058	0.014	0.17	0.06	1.06	0.15	0.32	0.91	0.3	1.51
0.0060	0.014	0.17	0.06	1.08	0.15	0.32	0.91	0.3	1.51
0.0062	0.014	0.17	0.06	1.09	0.15	0.32	0.91	0.3	1.51
0.0064	0.014	0.17	0.05	1.10	0.15	0.32	0.90	0.3	1.50
0.0066	0.014	0.17	0.05	1.12	0.15	0.32	0.90	0.3	1.50
0.0068	0.014	0.17	0.05	1.13	0.15	0.32	0.90	0.3	1.50
0.0070	0.014	0.17	0.05	1.14	0.15	0.32	0.90	0.3	1.50
0.0072	0.014	0.17	0.05	1.15	0.15	0.32	0.89	0.3	1.49
0.0074	0.014	0.17	0.05	1.16	0.15	0.32	0.89	0.3	1.49

Table B6. Hydraulic design tables for parabolic lined section for Discharge of 100 L/S.

Energy Slope ---	Manning's n	Flow Depth y (m)	Area, A (m^2)	Flow Velocity (m/s)	Free Board, FB (m)	Total Depth, (m)	Top Width, TW (m)	Bank Width BW (m)	Pad Width (m)
0.0001	0.014	0.45	0.38	0.26	0.15	0.60	1.69	0.3	2.29
0.0002	0.014	0.39	0.29	0.34	0.15	0.54	1.54	0.3	2.14
0.0004	0.014	0.35	0.23	0.44	0.15	0.50	1.40	0.3	2.00
0.0006	0.014	0.32	0.19	0.52	0.15	0.47	1.33	0.3	1.93
0.0008	0.014	0.30	0.17	0.57	0.15	0.45	1.28	0.3	1.88
0.0010	0.014	0.29	0.16	0.62	0.15	0.44	1.25	0.3	1.85
0.0012	0.014	0.28	0.15	0.67	0.15	0.43	1.22	0.3	1.82
0.0014	0.014	0.27	0.14	0.71	0.15	0.42	1.20	0.3	1.80
0.0016	0.014	0.27	0.13	0.75	0.15	0.42	1.18	0.3	1.78
0.0018	0.014	0.26	0.13	0.78	0.15	0.41	1.16	0.3	1.76
0.0020	0.014	0.26	0.12	0.81	0.15	0.41	1.15	0.3	1.75
0.0022	0.014	0.25	0.12	0.84	0.15	0.40	1.13	0.3	1.73
0.0024	0.014	0.25	0.12	0.87	0.15	0.40	1.12	0.3	1.72
0.0026	0.014	0.24	0.11	0.89	0.15	0.39	1.11	0.3	1.71
0.0028	0.014	0.24	0.11	0.92	0.15	0.39	1.10	0.3	1.70
0.0030	0.014	0.24	0.11	0.94	0.15	0.39	1.09	0.3	1.69
0.0032	0.014	0.23	0.10	0.97	0.15	0.38	1.09	0.3	1.69
0.0034	0.014	0.23	0.10	0.99	0.15	0.38	1.08	0.3	1.68
0.0036	0.014	0.23	0.10	1.01	0.15	0.38	1.07	0.3	1.67
0.0038	0.014	0.23	0.10	1.03	0.15	0.38	1.07	0.3	1.67
0.0040	0.014	0.22	0.10	1.05	0.15	0.37	1.06	0.3	1.66
0.0042	0.014	0.22	0.09	1.07	0.15	0.37	1.05	0.3	1.65
0.0044	0.014	0.22	0.09	1.09	0.15	0.37	1.05	0.3	1.65
0.0046	0.014	0.22	0.09	1.11	0.15	0.37	1.04	0.3	1.64
0.0048	0.014	0.22	0.09	1.13	0.15	0.37	1.04	0.3	1.64
0.0050	0.014	0.22	0.09	1.14	0.15	0.37	1.03	0.3	1.63
0.0052	0.014	0.21	0.09	1.16	0.15	0.36	1.03	0.3	1.63
0.0054	0.014	0.21	0.09	1.18	0.15	0.36	1.02	0.3	1.62
0.0056	0.014	0.21	0.08	1.19	0.15	0.36	1.02	0.3	1.62
0.0058	0.014	0.21	0.08	1.21	0.15	0.36	1.02	0.3	1.62
0.0060	0.014	0.21	0.08	1.22	0.15	0.36	1.01	0.3	1.61
0.0062	0.014	0.21	0.08	1.24	0.15	0.36	1.01	0.3	1.61
0.0064	0.014	0.21	0.08	1.25	0.15	0.36	1.01	0.3	1.61
0.0066	0.014	0.20	0.08	1.27	0.15	0.35	1.00	0.3	1.60
0.0068	0.014	0.20	0.08	1.28	0.15	0.35	1.00	0.3	1.60
0.0070	0.014	0.20	0.08	1.30	0.15	0.35	1.00	0.3	1.60
0.0072	0.014	0.20	0.08	1.31	0.15	0.35	0.99	0.3	1.59
0.0074	0.014	0.20	0.08	1.32	0.15	0.35	0.99	0.3	1.59

Table B7. Hydraulic design tables for parabolic lined section for Discharge of 200 L/S.

Energy Slope ---	Manning's n	Flow Depth y (m)	Area, A (m^2)	Flow Velocity (m/s)	Free Board, FB (m)	Total Depth, (m)	Top Width, TW (m)	Bank Width BW (m)	Pad Width (m)
0.0001	0.014	0.58	0.64	0.31	0.15	0.73	2.07	0.3	2.67
0.0002	0.014	0.51	0.49	0.41	0.15	0.66	1.87	0.3	2.47
0.0004	0.014	0.45	0.38	0.53	0.15	0.60	1.69	0.3	2.29
0.0006	0.014	0.42	0.33	0.61	0.15	0.57	1.60	0.3	2.20
0.0008	0.014	0.39	0.29	0.68	0.15	0.54	1.54	0.3	2.14
0.0010	0.014	0.38	0.27	0.74	0.15	0.53	1.49	0.3	2.09
0.0012	0.014	0.36	0.25	0.80	0.15	0.51	1.46	0.3	2.06
0.0014	0.014	0.35	0.24	0.84	0.15	0.50	1.43	0.3	2.03
0.0016	0.014	0.35	0.23	0.89	0.15	0.50	1.40	0.3	2.00
0.0018	0.014	0.34	0.22	0.93	0.15	0.49	1.38	0.3	1.98
0.0020	0.014	0.33	0.21	0.96	0.15	0.48	1.36	0.3	1.96
0.0022	0.014	0.33	0.20	1.00	0.15	0.48	1.35	0.3	1.95
0.0024	0.014	0.32	0.19	1.03	0.15	0.47	1.33	0.3	1.93
0.0026	0.014	0.32	0.19	1.06	0.15	0.47	1.32	0.3	1.92
0.0028	0.014	0.31	0.18	1.09	0.15	0.46	1.31	0.3	1.91
0.0030	0.014	0.31	0.18	1.12	0.15	0.46	1.29	0.3	1.89
0.0032	0.014	0.30	0.17	1.15	0.15	0.45	1.28	0.3	1.88
0.0034	0.014	0.30	0.17	1.18	0.15	0.45	1.27	0.3	1.87
0.0036	0.014	0.30	0.17	1.20	0.15	0.45	1.26	0.3	1.86
0.0038	0.014	0.29	0.16	1.23	0.15	0.44	1.26	0.3	1.86
0.0040	0.014	0.29	0.16	1.25	0.15	0.44	1.25	0.3	1.85
0.0042	0.014	0.29	0.16	1.27	0.15	0.44	1.24	0.3	1.84
0.0044	0.014	0.29	0.15	1.29	0.15	0.44	1.23	0.3	1.83
0.0046	0.014	0.28	0.15	1.32	0.15	0.43	1.23	0.3	1.83
0.0048	0.014	0.28	0.15	1.34	0.15	0.43	1.22	0.3	1.82
0.0050	0.014	0.28	0.15	1.36	0.15	0.43	1.21	0.3	1.81
0.0052	0.014	0.28	0.15	1.38	0.15	0.43	1.21	0.3	1.81
0.0054	0.014	0.28	0.14	1.40	0.15	0.43	1.20	0.3	1.80
0.0056	0.014	0.27	0.14	1.42	0.15	0.42	1.20	0.3	1.80
0.0058	0.014	0.27	0.14	1.44	0.15	0.42	1.19	0.3	1.79
0.0060	0.014	0.27	0.14	1.45	0.15	0.42	1.19	0.3	1.79
0.0062	0.014	0.27	0.14	1.47	0.15	0.42	1.18	0.3	1.78
0.0064	0.014	0.27	0.13	1.49	0.15	0.42	1.18	0.3	1.78
0.0066	0.014	0.26	0.13	1.51	0.15	0.41	1.17	0.3	1.77
0.0068	0.014	0.26	0.13	1.52	0.15	0.41	1.17	0.3	1.77
0.0070	0.014	0.26	0.13	1.54	0.15	0.41	1.17	0.3	1.77
0.0072	0.014	0.26	0.13	1.56	0.15	0.41	1.16	0.3	1.76
0.0074	0.014	0.26	0.13	1.57	0.15	0.41	1.16	0.3	1.76

Table B8. Hydraulic design tables for parabolic lined section for Discharge of 300 L/S.

Energy Slope ---	Manning's n	Flow Depth y (m)	Area, A (m^2)	Flow Velocity (m/s)	Free Board, FB (m)	Total Depth, (m)	Top Width, TW (m)	Bank Width BW (m)	Pad Width (m)
0.0001	0.014	0.68	0.87	0.35	0.15	0.83	2.34	0.3	2.94
0.0002	0.014	0.59	0.67	0.45	0.15	0.74	2.11	0.3	2.71
0.0004	0.014	0.52	0.51	0.58	0.15	0.67	1.90	0.3	2.50
0.0006	0.014	0.48	0.44	0.68	0.15	0.63	1.79	0.3	2.39
0.0008	0.014	0.46	0.40	0.76	0.15	0.61	1.72	0.3	2.32
0.0010	0.014	0.44	0.36	0.82	0.15	0.59	1.67	0.3	2.27
0.0012	0.014	0.42	0.34	0.88	0.15	0.57	1.63	0.3	2.23
0.0014	0.014	0.41	0.32	0.93	0.15	0.56	1.59	0.3	2.19
0.0016	0.014	0.40	0.31	0.98	0.15	0.55	1.56	0.3	2.16
0.0018	0.014	0.39	0.29	1.02	0.15	0.54	1.54	0.3	2.14
0.0020	0.014	0.39	0.28	1.07	0.15	0.54	1.52	0.3	2.12
0.0022	0.014	0.38	0.27	1.11	0.15	0.53	1.50	0.3	2.10
0.0024	0.014	0.37	0.26	1.14	0.15	0.52	1.48	0.3	2.08
0.0026	0.014	0.37	0.25	1.18	0.15	0.52	1.46	0.3	2.06
0.0028	0.014	0.36	0.25	1.21	0.15	0.51	1.45	0.3	2.05
0.0030	0.014	0.36	0.24	1.24	0.15	0.51	1.44	0.3	2.04
0.0032	0.014	0.35	0.24	1.27	0.15	0.50	1.42	0.3	2.02
0.0034	0.014	0.35	0.23	1.30	0.15	0.50	1.41	0.3	2.01
0.0036	0.014	0.35	0.23	1.33	0.15	0.50	1.40	0.3	2.00
0.0038	0.014	0.34	0.22	1.36	0.15	0.49	1.39	0.3	1.99
0.0040	0.014	0.34	0.22	1.38	0.15	0.49	1.38	0.3	1.98
0.0042	0.014	0.34	0.21	1.41	0.15	0.49	1.37	0.3	1.97
0.0044	0.014	0.33	0.21	1.43	0.15	0.48	1.37	0.3	1.97
0.0046	0.014	0.33	0.21	1.46	0.15	0.48	1.36	0.3	1.96
0.0048	0.014	0.33	0.20	1.48	0.15	0.48	1.35	0.3	1.95
0.0050	0.014	0.32	0.20	1.50	0.15	0.47	1.34	0.3	1.94
0.0052	0.014	0.32	0.20	1.53	0.15	0.47	1.34	0.3	1.94
0.0054	0.014	0.32	0.19	1.55	0.15	0.47	1.33	0.3	1.93
0.0056	0.014	0.32	0.19	1.57	0.15	0.47	1.32	0.3	1.92
0.0058	0.014	0.32	0.19	1.59	0.15	0.47	1.32	0.3	1.92
0.0060	0.014	0.31	0.19	1.61	0.15	0.46	1.31	0.3	1.91
0.0062	0.014	0.31	0.18	1.63	0.15	0.46	1.31	0.3	1.91
0.0064	0.014	0.31	0.18	1.65	0.15	0.46	1.30	0.3	1.90
0.0066	0.014	0.31	0.18	1.67	0.15	0.46	1.30	0.3	1.90
0.0068	0.014	0.31	0.18	1.69	0.15	0.46	1.29	0.3	1.89
0.0070	0.014	0.31	0.18	1.71	0.15	0.46	1.29	0.3	1.89
0.0072	0.014	0.30	0.17	1.72	0.15	0.45	1.28	0.3	1.88

ANNEXURE C: HYDRAULIC DESIGN OF LINED TRAPEZOIDAL CANALS

Table C1. Hydraulic design tables for lined trapezoidal irrigation channels for discharges of 10, 20, and 30 L/S.

Watercourse Design Data

Discharge = 10 l/s Side slope = 1.0 Thickness = 0.08 m

Energy slope S	Bottom width b, m	Flow Depth d, m	Total depth D, m	Top width TW m	Vel. n=0.014 V m/s	1:2:4 mix. Cement bags/m	Berm = 0.15 m Sand m3/m	Gravel m3/m
0.0001	0.30	0.15	0.23	0.76	0.149	0.628	0.044	0.086
0.0002	0.30	0.12	0.23	0.76	0.194	0.628	0.044	0.086
0.0003	0.30	0.11	0.23	0.76	0.226	0.628	0.044	0.086
0.0004	0.30	0.09	0.15	0.80	0.251	0.615	0.043	0.086
0.0004	0.30	0.10	0.23	0.76	0.251	0.628	0.044	0.086
0.0005	0.45	0.08	0.15	0.75	0.273	0.590	0.041	0.083
0.0005	0.30	0.10	0.23	0.76	0.273	0.628	0.044	0.086
0.0006	0.40	0.08	0.15	0.70	0.293	0.565	0.040	0.079
0.0007	0.40	0.07	0.15	0.70	0.310	0.565	0.040	0.079
0.0008	0.35	0.08	0.15	0.65	0.326	0.540	0.038	0.076
0.0009	0.35	0.07	0.15	0.65	0.341	0.540	0.038	0.076
0.0010	0.30	0.08	0.15	0.60	0.354	0.515	0.036	0.072
0.0015	0.30	0.07	0.15	0.60	0.412	0.515	0.036	0.072
0.0020	0.30	0.06	0.15	0.60	0.459	0.515	0.036	0.072
0.0030	0.30	0.06	0.15	0.60	0.535	0.515	0.036	0.072
0.0040	0.30	0.05	0.15	0.60	0.596	0.515	0.036	0.072
0.0050	0.30	0.05	0.15	0.60	0.648	0.515	0.036	0.072
0.0060	0.30	0.05	0.15	0.60	0.694	0.515	0.036	0.072
0.0070	0.30	0.04	0.15	0.60	0.735	0.515	0.036	0.072

Watercourse Design Data

Discharge = 10 l/s Side slope = 0.667 Thickness = 0.08 m

Energy slope S	Bottom width b, m	Flow Depth d, m	Total depth D, m	Top width TW m	Vel. n=0.014 V m/s	1:2:4 mix. Cement bags/m	Berm = 0.15 m Sand m3/m	Gravel m3/m
0.0001	0.35	0.15	0.23	0.66	0.151	0.604	0.042	0.085
0.0002	0.30	0.13	0.23	0.61	0.196	0.579	0.041	0.081
0.0003	0.30	0.12	0.23	0.61	0.228	0.579	0.041	0.081
0.0004	0.30	0.11	0.23	0.61	0.255	0.579	0.041	0.081
0.0005	0.45	0.08	0.15	0.65	0.277	0.558	0.039	0.078
0.0006	0.45	0.08	0.15	0.65	0.296	0.558	0.039	0.078
0.0006	0.35	0.10	0.23	0.61	0.296	0.579	0.041	0.081
0.0007	0.40	0.08	0.15	0.60	0.314	0.533	0.037	0.075
0.0008	0.40	0.07	0.15	0.60	0.330	0.533	0.037	0.075
0.0009	0.35	0.08	0.15	0.55	0.345	0.508	0.036	0.071
0.0010	0.35	0.08	0.15	0.55	0.359	0.508	0.036	0.071
0.0015	0.30	0.07	0.15	0.50	0.418	0.483	0.034	0.068
0.0020	0.30	0.07	0.15	0.50	0.465	0.483	0.034	0.068
0.0030	0.30	0.06	0.15	0.50	0.542	0.483	0.034	0.068
0.0040	0.30	0.05	0.15	0.50	0.604	0.483	0.034	0.068
0.0050	0.30	0.05	0.15	0.50	0.656	0.483	0.034	0.068
0.0060	0.30	0.05	0.15	0.50	0.703	0.483	0.034	0.068
0.0070	0.30	0.05	0.15	0.50	0.744	0.483	0.034	0.068

Watercourse Design Data

Discharge = 20 l/s Side slope = 1.0 Thickness = 0.08 m

Energy slope S	Bottom width b, m	Flow Depth d, m	Total depth D, m	Top width TW m	Vel. n=0.014 V m/s	1:2:4 mix. Cement bags/m	Berm = 0.15 m Sand m3/m	Gravel m3/m
0.0001	0.30	0.22	0.30	0.90	0.178	0.728	0.051	0.102
0.0002	0.40	0.16	0.23	0.86	0.230	0.679	0.048	0.095
0.0003	0.35	0.15	0.23	0.81	0.268	0.653	0.046	0.092
0.0004	0.30	0.15	0.23	0.76	0.299	0.628	0.044	0.088
0.0005	0.30	0.14	0.23	0.76	0.325	0.628	0.044	0.088
0.0006	0.30	0.13	0.23	0.76	0.348	0.628	0.044	0.088
0.0007	0.30	0.13	0.23	0.76	0.369	0.628	0.044	0.088
0.0008	0.30	0.12	0.23	0.76	0.388	0.628	0.044	0.088
0.0009	0.30	0.12	0.23	0.76	0.405	0.628	0.044	0.088
0.0010	0.30	0.12	0.23	0.76	0.421	0.628	0.044	0.088
0.0015	0.50	0.08	0.15	0.80	0.491	0.615	0.043	0.086
0.0015	0.30	0.10	0.23	0.76	0.491	0.628	0.044	0.088
0.0020	0.45	0.08	0.15	0.75	0.546	0.590	0.041	0.083
0.0020	0.30	0.10	0.23	0.76	0.546	0.628	0.044	0.088
0.0030	0.35	0.08	0.15	0.65	0.636	0.540	0.038	0.076
0.0040	0.30	0.08	0.15	0.60	0.709	0.515	0.036	0.072
0.0050	0.30	0.07	0.15	0.60	0.770	0.515	0.036	0.072
0.0060	0.30	0.07	0.15	0.60	0.825	0.515	0.036	0.072

Watercourse Design Data

Discharge = 20 l/s Side slope = 0.667 Thickness = 0.08 m

Energy slope S	Bottom width b, m	Flow Depth d, m	Total depth D, m	Top width TW m	Vel. n=0.014 V m/s	1:2:4 mix. Cement bags/m	Berm = 0.15 m Sand m3/m	Gravel m3/m
0.0001	0.35	0.22	0.30	0.75	0.180	0.689	0.048	0.097
0.0002	0.45	0.16	0.23	0.76	0.233	0.655	0.046	0.092
0.0003	0.40	0.15	0.23	0.71	0.272	0.629	0.044	0.086
0.0004	0.35	0.15	0.23	0.66	0.303	0.604	0.042	0.085
0.0005	0.30	0.15	0.23	0.61	0.329	0.579	0.041	0.081
0.0006	0.30	0.14	0.23	0.61	0.352	0.579	0.041	0.081
0.0007	0.30	0.14	0.23	0.61	0.373	0.579	0.041	0.081
0.0008	0.30	0.13	0.23	0.61	0.393	0.579	0.041	0.081
0.0009	0.30	0.13	0.23	0.61	0.410	0.579	0.041	0.081
0.0010	0.30	0.12	0.23	0.61	0.427	0.579	0.041	0.081
0.0015	0.30	0.11	0.23	0.61	0.497	0.579	0.041	0.081
0.0020	0.45	0.08	0.15	0.65	0.553	0.558	0.039	0.078
0.0030	0.40	0.08	0.15	0.60	0.644	0.533	0.037	0.075
0.0040	0.35	0.08	0.15	0.55	0.718	0.508	0.036	0.071
0.0050	0.30	0.08	0.15	0.50	0.780	0.483	0.034	0.068
0.0060	0.30	0.07	0.15	0.50	0.836	0.483	0.034	0.068
0.0070	0.30	0.07	0.15	0.50	0.885	0.483	0.034	0.068

Watercourse Design Data

Discharge = 30 l/s Side slope = 1.0 Thickness = 0.08 m

Energy slope S	Bottom width b, m	Flow Depth d, m	Total depth D, m	Top width TW m	Vel. n=0.014 V m/s	1:2:4 mix. Cement bags/m	Berm = 0.15 m Sand m3/m	Gravel m3/m
0.0001	0.45	0.23	0.30	1.05	0.197	0.803	0.056	0.113
0.0002	0.30	0.22	0.30	0.90	0.255	0.728	0.051	0.102
0.0003	0.30	0.20	0.30	0.90	0.297	0.728	0.051	0.102
0.0004	0.45	0.16	0.23	0.91	0.331	0.704	0.049	0.099
0.0004	0.30	0.19	0.30	0.90	0.333	0.728	0.051	0.102
0.0005	0.40	0.15	0.23	0.86	0.360	0.679	0.048	0.095
0.0005	0.30	0.18	0.30	0.90	0.360	0.728	0.051	0.102
0.0006	0.35	0.16	0.23	0.81	0.385	0.653	0.046	0.092
0.0007	0.35	0.15	0.23	0.81	0.408	0.653	0.046	0.092
0.0008	0.30	0.16	0.23	0.76	0.429	0.628	0.044	0.088
0.0009	0.30	0.15	0.23	0.76	0.448	0.628	0.044	0.088
0.0010	0.30	0.15	0.23	0.76	0.466	0.628	0.044	0.088
0.0015	0.30	0.13	0.23	0.76	0.543	0.628	0.044	0.088
0.0020	0.30	0.12	0.23	0.76	0.605	0.628	0.044	0.088
0.0030	0.30	0.11	0.23	0.76	0.704	0.628	0.044	0.088
0.0040	0.45	0.08	0.15	0.75	0.784	0.590	0.041	0.083
0.0040	0.30	0.10	0.23	0.76	0.784	0.628	0.044	0.088
0.0050	0.40	0.08	0.15	0.70	0.853	0.565	0.040	0.079
0.0060	0.40	0.08	0.15	0.70	0.913	0.565	0.040	0.079

Watercourse Design Data

Discharge = 30 l/s Side slope = 0.667 Thickness = 0.08 m

Energy slope S	Bottom width b, m	Flow Depth d, m	Total depth D, m	Top width TW m	Vel. n=0.014 V m/s	1:2:4 mix. Cement bags/m	Berm = 0.15 m Sand m3/m	Gravel m3/m
0.0001	0.35	0.23	0.30	0.86	0.199	0.786	0.055	[illegible]
0.0002	0.40	0.22	0.30	0.80	0.258	0.714	0.050	[illegible]
0.0003	0.30	0.22	0.30	0.76	0.301	0.664	0.047	[illegible]
0.0004	0.30	0.21	0.30	0.70	0.335	0.664	0.047	[illegible]
0.0005	0.45	0.15	0.23	0.76	0.364	0.655	0.046	[illegible]
0.0006	0.40	0.16	0.23	0.71	0.390	0.629	0.044	[illegible]
0.0007	0.40	0.15	0.23	0.71	0.413	0.629	0.044	[illegible]
0.0008	0.35	0.15	0.23	0.66	0.434	0.604	0.042	[illegible]
0.0009	0.35	0.15	0.23	0.66	0.454	0.604	0.042	[illegible]
0.0010	0.30	0.16	0.23	0.61	0.472	0.579	0.041	[illegible]
0.0015	0.30	0.14	0.23	0.61	0.550	0.579	0.041	[illegible]
0.0020	0.30	0.13	0.23	0.61	0.613	0.579	0.041	[illegible]
0.0030	0.30	0.11	0.23	0.61	0.713	0.579	0.041	[illegible]
0.0040	0.50	0.08	0.15	0.70	0.794	0.583	0.041	[illegible]
0.0040	0.30	0.11	0.23	0.61	0.794	0.579	0.041	[illegible]
0.0050	0.45	0.08	0.15	0.61	0.864	0.558	0.039	[illegible]
0.0050	0.30	0.10	0.23	0.61	0.864	0.579	0.041	[illegible]
0.0060	0.40	0.08	0.15	0.60	0.925	0.533	0.037	[illegible]

Table C2. Hydraulic design tables for lined trapezoidal irrigation channels for discharges of 40, 50, and 60 L/S.

Watercourse Design Data

Discharge = 40 l/s Side slope = 1.0 Thickness = 0.08 m

Energy slope S	Bottom width b. m	Flow Depth d. m	Total depth D. m	Top width TW m	Vel. n=0.014 V m/s	1:2:4 mix. Cement bags/m	Berm = 0.15 m Sand m3/m	Gravel m3/m
0.0001	0.40	0.28	0.38	1.14	0.211	0.892	0.062	0.125
0.0002	0.45	0.22	0.30	1.05	0.274	0.803	0.056	0.113
0.0003	0.35	0.22	0.30	0.95	0.319	0.753	0.053	0.106
0.0004	0.30	0.22	0.30	0.90	0.355	0.728	0.051	0.102
0.0005	0.30	0.21	0.30	0.90	0.386	0.728	0.051	0.102
0.0006	0.50	0.15	0.23	0.96	0.414	0.729	0.051	0.102
0.0006	0.30	0.20	0.30	0.90	0.414	0.728	0.051	0.102
0.0007	0.45	0.16	0.23	0.91	0.438	0.704	0.049	0.099
0.0008	0.40	0.16	0.23	0.86	0.461	0.679	0.048	0.095
0.0009	0.40	0.15	0.23	0.86	0.482	0.679	0.048	0.095
0.0010	0.35	0.16	0.23	0.81	0.501	0.653	0.046	0.092
0.0015	0.30	0.15	0.23	0.76	0.583	0.628	0.044	0.088
0.0020	0.30	0.14	0.23	0.76	0.680	0.628	0.044	0.088
0.0030	0.30	0.13	0.23	0.76	0.757	0.628	0.044	0.088
0.0040	0.30	0.12	0.23	0.76	0.843	0.628	0.044	0.088
0.0050	0.30	0.11	0.23	0.76	0.916	0.628	0.044	0.088

Watercourse Design Data

Discharge = 40 l/s Side slope = 0.667 Thickness = 0.08 m

Energy slope S	Bottom width b. m	Flow Depth d. m	Total depth D. m	Top width TW m	Vel. n=0.014 V m/s	1:2:4 mix. Cement bags/m	Berm = 0.15 m Sand m3/m	Gravel m3/m
0.0001	0.50	0.28	0.38	1.01	0.234	0.861	0.060	0.121
0.0002	0.50	0.23	0.30	0.90	0.278	0.764	0.054	0.107
0.0002	0.35	0.27	0.38	0.86	0.276	0.786	0.055	0.110
0.0003	0.40	0.23	0.30	0.80	0.323	0.714	0.050	0.100
0.0004	0.35	0.22	0.30	0.75	0.360	0.689	0.048	0.097
0.0005	0.30	0.23	0.30	0.70	0.391	0.664	0.047	0.093
0.0006	0.30	0.22	0.30	0.70	0.419	0.664	0.047	0.093
0.0007	0.30	0.21	0.30	0.70	0.444	0.664	0.047	0.093
0.0008	0.45	0.16	0.23	0.76	0.467	0.655	0.046	0.092
0.0008	0.30	0.20	0.30	0.70	0.467	0.664	0.047	0.093
0.0009	0.30	0.19	0.30	0.70	0.488	0.664	0.047	0.093
0.0010	0.40	0.16	0.23	0.71	0.508	0.629	0.044	0.088
0.0015	0.35	0.15	0.23	0.66	0.591	0.604	0.042	0.085
0.0020	0.30	0.15	0.23	0.61	0.658	0.579	0.041	0.081
0.0030	0.30	0.14	0.23	0.61	0.766	0.579	0.041	0.081
0.0040	0.30	0.12	0.23	0.61	0.854	0.579	0.041	0.081
0.0050	0.30	0.12	0.23	0.61	0.928	0.579	0.041	0.081
0.0060	0.30	0.11	0.23	0.61	0.994	0.579	0.041	0.081

Watercourse Design Data

Discharge = 50 l/s Side slope = 1.0 Thickness = 0.08 m

Energy slope S	Bottom width b. m	Flow Depth d. m	Total depth D. m	Top width TW m	Vel. n=0.014 V m/s	1:2:4 mix. Cement bags/m	Berm = 0.15 m Sand m3/m	Gravel m3/m
0.0001	0.50	0.29	0.38	1.24	0.223	0.942	0.066	0.132
0.0002	0.35	0.28	0.38	1.11	0.290	0.867	0.061	0.121
0.0003	0.45	0.22	0.30	1.05	0.337	0.803	0.056	0.113
0.0004	0.40	0.22	0.30	1.00	0.376	0.778	0.055	0.109
0.0005	0.35	0.22	0.30	0.95	0.409	0.753	0.053	0.105
0.0006	0.30	0.22	0.30	0.90	0.437	0.728	0.051	0.102
0.0007	0.30	0.21	0.30	0.90	0.463	0.728	0.051	0.102
0.0008	0.30	0.20	0.30	0.90	0.487	0.728	0.051	0.102
0.0009	0.30	0.20	0.30	0.90	0.509	0.728	0.051	0.102
0.0010	0.45	0.16	0.23	0.91	0.530	0.704	0.049	0.099
0.0015	0.40	0.15	0.23	0.86	0.617	0.679	0.048	0.095
0.0020	0.35	0.15	0.23	0.81	0.687	0.653	0.046	0.092
0.0030	0.30	0.14	0.23	0.76	0.800	0.628	0.044	0.088
0.0040	0.30	0.13	0.23	0.76	0.891	0.628	0.044	0.088
0.0050	0.30	0.12	0.23	0.76	0.969	0.628	0.044	0.088

Watercourse Design Data

Discharge = 50 l/s Side slope = 0.667 Thickness = 0.08 m

Energy slope S	Bottom width b. m	Flow Depth d. m	Total depth D. m	Top width TW m	Vel. n=0.014 V m/s	1:2:4 mix. Cement bags/m	Berm = 0.15 m Sand m3/m	Gravel m3/m
0.0001	0.45	0.33	0.46	1.06	0.226	0.932	0.065	0.131
0.0002	0.40	0.29	0.38	0.91	0.293	0.811	0.057	0.114
0.0003	0.50	0.23	0.30	0.90	0.342	0.764	0.054	0.107
0.0003	0.35	0.27	0.38	0.86	0.342	0.786	0.055	0.110
0.0004	0.45	0.22	0.30	0.85	0.381	0.739	0.052	0.104
0.0004	0.30	0.27	0.38	0.81	0.381	0.760	0.053	0.107
0.0005	0.40	0.22	0.30	0.80	0.414	0.714	0.050	0.100
0.0006	0.35	0.23	0.30	0.75	0.443	0.689	0.048	0.097
0.0007	0.35	0.22	0.30	0.75	0.469	0.689	0.048	0.097
0.0008	0.30	0.23	0.30	0.70	0.494	0.664	0.047	0.093
0.0009	0.30	0.22	0.30	0.70	0.516	0.664	0.047	0.093
0.0010	0.30	0.21	0.30	0.70	0.537	0.664	0.047	0.093
0.0015	0.45	0.15	0.23	0.76	0.625	0.655	0.046	0.092
0.0020	0.35	0.16	0.23	0.66	0.696	0.604	0.042	0.085
0.0030	0.30	0.15	0.23	0.61	0.810	0.579	0.041	0.081
0.0040	0.30	0.14	0.23	0.61	0.903	0.579	0.041	0.081
0.0050	0.30	0.13	0.23	0.61	0.981	0.575	0.041	0.081
0.0060	0.30	0.13	0.23	0.61	1.051	0.579	0.041	0.081

Watercourse Design Data

Discharge = 60 l/s Side slope = 1.0 Thickness = 0.08 m

Energy slope S	Bottom width b. m	Flow Depth d. m	Total depth D. m	Top width TW m	Vel. n=0.014 V m/s	1:2:4 mix. Cement bags/m	Berm = 0.15 m Sand m3/m	Gravel m3/m
0.0001	0.40	0.34	0.46	1.32	0.234	1.006	0.070	0.141
0.0002	0.40	0.29	0.38	1.14	0.303	0.892	0.062	0.125
0.0003	0.35	0.27	0.38	1.11	0.353	0.867	0.061	0.121
0.0004	0.45	0.23	0.30	1.05	0.383	0.803	0.056	0.113
0.0005	0.40	0.23	0.30	1.00	0.428	0.778	0.055	0.109
0.0006	0.35	0.23	0.30	0.95	0.458	0.753	0.053	0.106
0.0007	0.35	0.22	0.30	0.95	0.485	0.753	0.053	0.105
0.0008	0.30	0.22	0.30	0.90	0.510	0.728	0.051	0.102
0.0009	0.30	0.22	0.30	0.90	0.533	0.728	0.051	0.102
0.0010	0.30	0.21	0.30	0.90	0.555	0.728	0.051	0.102
0.0015	0.45	0.16	0.23	0.91	0.646	0.704	0.049	0.099
0.0015	0.30	0.19	0.30	0.90	0.646	0.728	0.051	0.102
0.0020	0.40	0.15	0.23	0.86	0.719	0.679	0.048	0.095
0.0030	0.30	0.16	0.23	0.76	0.837	0.628	0.044	0.088
0.0040	0.30	0.15	0.23	0.76	0.933	0.628	0.044	0.088
0.0050	0.30	0.14	0.23	0.76	1.014	0.628	0.044	0.088

Watercourse Design Data

Discharge = 60 l/s Side slope = 0.667 Thickness = 0.08 m

Energy slope S	Bottom width b. m	Flow Depth d. m	Total depth D. m	Top width TW m	Vel. n=0.014 V m/s	1:2:4 mix. Cement bags/m	Berm = 0.15 m Sand m3/m	Gravel m3/m
0.0001	0.50	0.35	0.46	1.11	0.237	0.958	0.067	0.134
0.0002	0.35	0.34	0.46	0.96	0.307	0.892	0.062	0.124
0.0002	0.50	0.28	0.38	1.01	0.307	0.861	0.060	0.121
0.0003	0.40	0.28	0.38	0.91	0.358	0.811	0.057	0.114
0.0004	0.35	0.28	0.38	0.86	0.398	0.786	0.055	0.110
0.0005	0.50	0.22	0.30	0.90	0.433	0.764	0.054	0.107
0.0005	0.30	0.28	0.38	0.81	0.433	0.760	0.053	0.107
0.0006	0.45	0.22	0.30	0.85	0.464	0.739	0.052	0.104
0.0007	0.40	0.22	0.30	0.80	0.491	0.714	0.050	0.100
0.0008	0.40	0.22	0.30	0.80	0.517	0.714	0.050	0.100
0.0009	0.35	0.22	0.30	0.75	0.540	0.689	0.048	0.097
0.0010	0.35	0.22	0.30	0.75	0.562	0.689	0.048	0.097
0.0015	0.50	0.16	0.23	0.81	0.654	0.680	0.048	0.095
0.0015	0.30	0.21	0.30	0.70	0.654	0.664	0.047	0.093
0.0020	0.45	0.15	0.23	0.76	0.728	0.655	0.046	0.092
0.0020	0.30	0.19	0.30	0.70	0.728	0.664	0.047	0.093
0.0030	0.35	0.16	0.23	0.66	0.848	0.604	0.042	0.085
0.0040	0.30	0.16	0.23	0.61	0.945	0.579	0.041	0.081
0.0050	0.30	0.15	0.23	0.61	1.027	0.579	0.041	0.081
0.0060	0.30	0.14	0.23	0.61	1.100	0.579	0.041	0.081

Table C3. Hydraulic design tables for lined trapezoidal irrigation channels for discharges of 70, 80, and 100 L/S.

Watercourse Design Data

Discharge = 70 l/s Side slope = 1.0 Thickness = 0.08 m

Energy slope S	Bottom width b, m	Flow Depth d, m	Total depth D, m	Top width TW m	Vel. n=0.014 V m/s	1:2.4 max. Cement bags/m	Berm = 0.15 m Sand m3/m	Gravel m3/m
0.0001	0.50	0.34	0.46	1.62	0.243	1.054	0.074	0.148
0.0002	0.50	0.29	0.38	1.26	0.315	0.942	0.066	0.132
0.0002	0.30	0.34	0.46	1.22	0.315	0.955	0.067	0.134
0.0003	0.40	0.28	0.38	1.16	0.367	0.892	0.062	0.125
0.0004	0.35	0.27	0.38	1.11	0.409	0.867	0.061	0.121
0.0005	0.30	0.27	0.38	1.06	0.444	0.841	0.059	0.118
0.0006	0.45	0.22	0.30	1.05	0.476	0.803	0.056	0.113
0.0007	0.40	0.22	0.30	1.00	0.504	0.778	0.055	0.109
0.0008	0.40	0.22	0.30	1.00	0.530	0.778	0.055	0.109
0.0009	0.35	0.22	0.30	0.95	0.554	0.753	0.053	0.105
0.0010	0.35	0.22	0.30	0.95	0.576	0.753	0.053	0.105
0.0015	0.30	0.21	0.30	0.90	0.671	0.728	0.051	0.102
0.0020	0.50	0.15	0.23	0.86	0.747	0.729	0.051	0.102
0.0020	0.30	0.19	0.30	0.90	0.747	0.728	0.051	0.102
0.0030	0.40	0.15	0.23	0.86	0.870	0.679	0.048	0.095
0.0040	0.30	0.16	0.23	0.76	0.969	0.628	0.044	0.088
0.0050	0.30	0.15	0.23	0.76	1.054	0.628	0.044	0.088

Watercourse Design Data

Discharge = 70 l/s Side slope = 0.667 Thickness = 0.08 m

Energy slope S	Bottom width b, m	Flow Depth d, m	Total depth D, m	Top width TW m	Vel. n=0.014 V m/s	1:2.4 max. Cement bags/m	Berm = 0.15 m Sand m3/m	Gravel m3/m
0.0001	0.45	0.40	0.53	1.16	0.244	1.017	0.071	0.143
0.0002	0.40	0.35	0.46	1.01	0.319	0.907	0.064	0.127
0.0003	0.30	0.38	0.38	1.01	0.372	0.861	0.060	0.121
0.0003	0.35	0.33	0.46	0.96	0.372	0.882	0.062	0.124
0.0004	0.40	0.29	0.38	0.91	0.414	0.811	0.057	0.114
0.0005	0.35	0.29	0.38	0.86	0.450	0.786	0.055	0.110
0.0006	0.50	0.23	0.30	0.90	0.482	0.764	0.054	0.107
0.0006	0.35	0.27	0.38	0.86	0.482	0.786	0.055	0.110
0.0007	0.30	0.28	0.38	0.81	0.511	0.760	0.053	0.107
0.0007	0.50	0.22	0.30	0.90	0.511	0.764	0.054	0.107
0.0008	0.45	0.22	0.30	0.85	0.537	0.739	0.052	0.104
0.0009	0.40	0.23	0.30	0.80	0.561	0.714	0.050	0.100
0.0010	0.40	0.22	0.30	0.80	0.584	0.714	0.050	0.100
0.0015	0.30	0.23	0.30	0.70	0.680	0.664	0.047	0.093
0.0020	0.50	0.16	0.23	0.91	0.759	0.680	0.048	0.095
0.0020	0.30	0.21	0.30	0.70	0.757	0.664	0.047	0.093
0.0030	0.45	0.15	0.23	0.76	0.881	0.655	0.046	0.092
0.0030	0.30	0.19	0.30	0.70	0.881	0.664	0.047	0.093
0.0040	0.35	0.16	0.23	0.66	0.982	0.604	0.042	0.085
0.0050	0.35	0.15	0.23	0.66	1.067	0.604	0.042	0.085

Watercourse Design Data

Discharge = 80 l/s Side slope = 1.0 Thickness = 0.08 m

Energy slope S	Bottom width b, m	Flow Depth d, m	Total depth D, m	Top width TW m	Vel. n=0.014 V m/s	1:2.4 max. Cement bags/m	Berm = 0.15 m Sand m3/m	Gravel m3/m
0.0001	0.40	0.40	0.53	1.46	0.251	1.105	0.077	0.155
0.0002	0.40	0.33	0.46	1.32	0.326	1.005	0.070	0.141
0.0003	0.45	0.29	0.38	1.21	0.379	0.917	0.064	0.128
0.0004	0.40	0.28	0.38	1.16	0.423	0.892	0.062	0.125
0.0005	0.35	0.28	0.38	1.11	0.459	0.867	0.061	0.121
0.0006	0.50	0.23	0.30	1.10	0.492	0.828	0.058	0.116
0.0006	0.30	0.28	0.38	1.06	0.492	0.841	0.059	0.118
0.0007	0.50	0.22	0.30	1.10	0.521	0.828	0.058	0.116
0.0007	0.30	0.27	0.38	1.06	0.521	0.841	0.059	0.118
0.0008	0.45	0.22	0.30	1.05	0.548	0.803	0.056	0.113
0.0009	0.40	0.23	0.30	1.00	0.573	0.778	0.055	0.109
0.0010	0.40	0.22	0.30	1.00	0.596	0.778	0.055	0.109
0.0015	0.30	0.22	0.30	0.90	0.694	0.728	0.051	0.102
0.0020	0.30	0.21	0.30	0.90	0.773	0.728	0.051	0.102
0.0030	0.45	0.15	0.23	0.91	0.900	0.704	0.045	0.099
0.0040	0.35	0.16	0.23	0.81	1.002	0.653	0.046	0.092
0.0050	0.35	0.15	0.23	0.81	1.090	0.653	0.044	0.092

Watercourse Design Data

Discharge = 80 l/s Side slope = 0.667 Thickness = 0.08 m

Energy slope S	Bottom width b, m	Flow Depth d, m	Total depth D, m	Top width TW m	Vel. n=0.014 V m/s	1:2.4 max. Cement bags/m	Berm = 0.15 m Sand m3/m	Gravel m3/m
0.0001	0.40	0.45	0.61	1.21	0.255	1.068	0.076	0.153
0.0002	0.50	0.34	0.46	1.11	0.330	0.958	0.067	0.134
0.0002	0.35	0.40	0.53	1.06	0.330	0.967	0.068	0.135
0.0003	0.40	0.30	0.46	1.01	0.384	0.907	0.064	0.127
0.0004	0.50	0.28	0.38	1.01	0.428	0.861	0.060	0.121
0.0004	0.35	0.33	0.46	0.96	0.428	0.882	0.062	0.124
0.0005	0.45	0.27	0.38	0.96	0.465	0.856	0.059	0.119
0.0006	0.40	0.28	0.38	0.91	0.498	0.811	0.057	0.114
0.0007	0.35	0.28	0.38	0.86	0.528	0.786	0.055	0.110
0.0008	0.50	0.23	0.30	0.90	0.555	0.764	0.054	0.107
0.0008	0.35	0.27	0.38	0.86	0.555	0.786	0.055	0.110
0.0009	0.50	0.22	0.30	0.90	0.580	0.764	0.054	0.107
0.0009	0.30	0.28	0.38	0.81	0.580	0.760	0.053	0.107
0.0010	0.45	0.22	0.30	0.85	0.604	0.739	0.052	0.104
0.0010	0.30	0.27	0.38	0.81	0.604	0.760	0.053	0.107
0.0015	0.35	0.22	0.30	0.75	0.703	0.689	0.048	0.097
0.0020	0.30	0.23	0.30	0.70	0.783	0.664	0.047	0.093
0.0030	0.30	0.20	0.30	0.70	0.911	0.664	0.047	0.093
0.0040	0.40	0.16	0.23	0.71	1.015	0.629	0.044	0.088
0.0050	0.40	0.15	0.23	0.71	1.104	0.629	0.044	0.088

Watercourse Design Data

Discharge = 100 l/s Side slope = 1.0 Thickness = 0.08 m

Energy slope S	Bottom width b, m	Flow Depth d, m	Total depth D, m	Top width TW m	Vel. n=0.014 V m/s	1:2.4 max. Cement bags/m	Berm = 0.15 m Sand m3/m	Gravel m3/m
0.0001	0.40	0.45	0.61	1.62	0.266	1.218	0.085	0.171
0.0002	0.50	0.35	0.46	1.42	0.345	1.056	0.074	0.148
0.0002	0.53	0.34	0.46	1.45	0.345	1.071	0.075	0.150
0.0003	0.40	0.34	0.46	1.32	0.401	1.005	0.070	0.141
0.0004	0.30	0.35	0.46	1.22	0.447	0.954	0.067	0.134
0.0005	0.45	0.28	0.38	1.21	0.486	0.917	0.064	0.128
0.0006	0.40	0.28	0.38	1.16	0.520	0.892	0.062	0.125
0.0007	0.35	0.29	0.38	1.11	0.561	0.867	0.061	0.121
0.0008	0.35	0.28	0.38	1.11	0.579	0.867	0.061	0.121
0.0009	0.50	0.24	0.38	1.06	0.606	0.841	0.059	0.118
0.0010	0.50	0.22	0.30	1.10	0.630	0.828	0.058	0.116
0.0010	0.50	0.28	0.38	1.06	0.630	0.841	0.059	0.118
0.0015	0.40	0.22	0.30	1.00	0.734	0.778	0.055	0.109
0.0020	0.35	0.22	0.30	0.95	0.817	0.753	0.053	0.105
0.0030	0.30	0.21	0.30	0.90	0.951	0.728	0.051	0.102
0.0040	0.50	0.15	0.23	0.96	1.060	0.729	0.051	0.102
0.0040	0.30	0.19	0.30	0.90	1.060	0.728	0.051	0.102
0.0050	0.45	0.15	0.23	0.91	1.152	0.704	0.049	0.099
0.0050	0.30	0.18	0.30	0.90	1.152	0.728	0.051	0.102

Watercourse Design Data

Discharge = 100 l/s Side slope = 0.667 Thickness = 0.08 m

Energy slope S	Bottom width b, m	Flow Depth d, m	Total depth D, m	Top width TW m	Vel. n=0.014 V m/s	1:2.4 max. Cement bags/m	Berm = 0.15 m Sand m3/m	Gravel m3/m
0.0001	0.50	0.46	0.61	1.31	0.269	1.139	0.080	0.160
0.0002	0.45	0.40	0.53	1.16	0.349	1.017	0.071	0.143
0.0003	0.35	0.40	0.53	1.06	0.406	0.967	0.068	0.135
0.0004	0.45	0.33	0.46	1.06	0.453	0.932	0.065	0.131
0.0004	0.30	0.39	0.53	1.01	0.453	0.942	0.066	0.133
0.0005	0.40	0.33	0.46	1.01	0.492	0.907	0.064	0.127
0.0006	0.50	0.28	0.38	1.01	0.527	0.861	0.060	0.121
0.0006	0.35	0.33	0.46	0.96	0.527	0.882	0.062	0.124
0.0007	0.45	0.28	0.38	0.96	0.558	0.836	0.059	0.117
0.0008	0.40	0.29	0.38	0.91	0.587	0.811	0.057	0.114
0.0009	0.40	0.28	0.38	0.91	0.613	0.811	0.057	0.114
0.0010	0.40	0.27	0.38	0.91	0.638	0.811	0.057	0.114
0.0015	0.45	0.23	0.30	0.85	0.743	0.739	0.052	0.104
0.0015	0.30	0.28	0.38	0.81	0.743	0.760	0.053	0.107
0.0020	0.40	0.22	0.30	0.80	0.828	0.714	0.050	0.100
0.0030	0.35	0.21	0.30	0.75	0.964	0.689	0.048	0.097
0.0040	0.50	0.21	0.30	0.70	1.073	0.664	0.047	0.093
0.0050	0.30	0.20	0.30	0.70	1.167	0.664	0.047	0.093

Table C4. Hydraulic design tables for lined trapezoidal irrigation channels for discharges of 200, and 230 L/S.

Watercourse Design Data

Discharge = 200 l/s Side slope = 1.0

Energy slope S	Bottom width b. m	Flow Depth d. m	Total depth D. m	Top width TW m	Vel. n=0.014 V m/s
0.0001	0.55	0.57	0.76	2.07	0.316
0.0001	0.55	0.37	0.76	2.07	0.316
0.0002	0.45	0.51	0.69	1.83	0.410
0.0003	0.45	0.46	0.61	1.67	0.477
0.0004	0.55	0.40	0.53	1.61	0.531
0.0005	0.50	0.39	0.53	1.56	0.578
0.0006	0.45	0.39	0.33	1.51	0.619
0.0007	0.55	0.34	0.46	1.47	0.655
0.0008	0.50	0.35	0.46	1.42	0.689
0.0009	0.50	0.33	0.46	1.42	0.720
0.0010	0.45	0.34	0.46	1.37	0.749
0.0015	0.35	0.33	0.46	1.27	0.872
0.0020	0.45	0.28	0.38	1.21	0.972
0.0030	0.35	0.28	0.38	1.11	1.131
0.0040	0.30	0.28	0.38	1.06	1.260
0.0040	0.50	0.22	0.30	1.10	1.260

Watercourse Design Data

Discharge = 200 l/s Side slope = 0.667

Energy slope S	Bottom width b. m	Flow Depth d. m	Total depth D. m	Top width TW m	Vel. n=0.014 V m/s
0.0001	0.60	0.62	0.84	1.72	0.320
0.0002	0.60	0.51	0.69	1.52	0.415
0.0002	0.50	0.55	0.76	1.51	0.415
0.0003	0.60	0.46	0.61	1.41	0.483
0.0004	0.50	0.46	0.61	1.31	0.538
0.0005	0.45	0.45	0.61	1.26	0.585
0.0006	0.55	0.39	0.53	1.26	0.627
0.0006	0.40	0.45	0.61	1.21	0.627
0.0007	0.50	0.39	0.53	1.21	0.664
0.0008	0.45	0.40	0.53	1.16	0.698
0.0009	0.45	0.39	0.53	1.16	0.730
0.0010	0.40	0.40	0.53	1.11	0.759
0.0015	0.45	0.34	0.46	1.06	0.884
0.0020	0.40	0.33	0.46	1.01	0.984
0.0030	0.45	0.28	0.38	0.96	1.146
0.0040	0.40	0.27	0.38	0.91	1.276
0.0050	0.50	0.23	0.30	0.90	1.388
0.0050	0.35	0.27	0.38	0.86	1.388

Watercourse Design Data

Discharge = 230 l/s Side slope = 1.0

Energy slope S	Bottom width b. m	Flow Depth d. m	Total depth D. m	Top width TW m	Vel. n=0.014 V m/s
0.0001	0.50	0.62	0.84	2.18	0.327
0.0002	0.55	0.51	0.69	1.93	0.424
0.0003	0.60	0.45	0.61	1.82	0.494
0.0004	0.45	0.46	0.61	1.67	0.550
0.0005	0.60	0.39	0.53	1.66	0.598
0.0006	0.50	0.40	0.53	1.56	0.641
0.0007	0.45	0.40	0.53	1.51	0.679
0.0008	0.45	0.39	0.53	1.51	0.714
0.0009	0.40	0.39	0.53	1.46	0.746
0.0010	0.55	0.34	0.46	1.47	0.776
0.0015	0.40	0.34	0.46	1.32	0.903
0.0020	0.35	0.33	0.46	1.27	1.006
0.0030	0.40	0.29	0.38	1.16	1.171
0.0040	0.35	0.28	0.38	1.11	1.305

Watercourse Design Data

Discharge = 230 l/s Side slope = 0.667

Energy slope S	Bottom width b. m	Flow Depth d. m	Total depth D. m	Top width TW m	Vel. n=0.014 V m/s
0.0001	0.60	0.67	0.91	1.81	0.331
0.0002	0.60	0.55	0.76	1.61	0.430
0.0002	0.45	0.62	0.84	1.57	0.430
0.0002	0.60	0.55	0.76	1.61	0.430
0.0003	0.55	0.51	0.69	1.47	0.500
0.0004	0.60	0.46	0.61	1.41	0.557
0.0005	0.55	0.45	0.61	1.36	0.606
0.0006	0.50	0.44	0.61	1.31	0.649
0.0007	0.60	0.39	0.53	1.31	0.688
0.0008	0.55	0.39	0.53	1.26	0.723
0.0009	0.50	0.40	0.53	1.21	0.755
0.0010	0.50	0.39	0.53	1.21	0.786
0.0015	0.50	0.35	0.46	1.11	0.915
0.0020	0.45	0.34	0.46	1.06	1.019
0.0030	0.50	0.28	0.38	1.01	1.187
0.0030	0.35	0.34	0.46	0.96	1.187
0.0040	0.45	0.28	0.38	0.96	1.322
0.0050	0.40	0.27	0.38	0.91	1.437

Table C5. Hydraulic design tables for lined trapezoidal irrigation channels for discharges of 260, and 290 L/S.

Watercourse Design Data

Discharge = 260 l/s Side slope = 1.0

Energy slope S	Bottom width b. m	Flow Depth d. m	Total depth D. m	Top width TW m	Vel. n=0.014 V m/s
0.0001	0.45	0.68	0.91	2.27	0.337
0.0001	0.60	0.63	0.84	2.28	0.337
0.0002	0.50	0.56	0.76	2.02	0.438
0.0003	0.50	0.51	0.69	1.88	0.509
0.0004	0.55	0.46	0.61	1.77	0.567
0.0005	0.50	0.45	0.61	1.72	0.617
0.0006	0.60	0.40	0.53	1.66	0.661
0.0007	0.55	0.39	0.53	1.61	0.700
0.0008	0.50	0.40	0.53	1.56	0.736
0.0009	0.45	0.40	0.53	1.51	0.769
0.0010	0.60	0.35	0.46	1.52	0.800
0.0010	0.45	0.39	0.53	1.51	0.800
0.0015	0.50	0.34	0.46	1.42	0.931
0.0020	0.40	0.34	0.46	1.32	1.038
0.0030	0.50	0.28	0.38	1.26	1.208
0.0030	0.30	0.34	0.46	1.22	1.208
0.0040	0.40	0.28	0.38	1.16	1.346

Watercourse Design Data

Discharge = 260 l/s Side slope = 0.667

Energy slope S	Bottom width b. m	Flow Depth d. m	Total depth D. m	Top width TW m	Vel. n=0.014 V m/s
0.0001	0.55	0.73	0.99	1.87	0.342
0.0002	0.55	0.61	0.84	1.67	0.443
0.0003	0.50	0.57	0.76	1.51	0.516
0.0004	0.55	0.51	0.69	1.47	0.575
0.0005	0.60	0.46	0.61	1.41	0.625
0.0006	0.55	0.46	0.61	1.36	0.669
0.0007	0.50	0.46	0.61	1.31	0.709
0.0008	0.45	0.46	0.61	1.26	0.745
0.0009	0.60	0.39	0.53	1.31	0.779
0.0010	0.55	0.40	0.53	1.26	0.810
0.0015	0.45	0.39	0.53	1.16	0.943
0.0020	0.50	0.34	0.46	1.11	1.051
0.0030	0.40	0.34	0.46	1.01	1.223
0.0040	0.35	0.33	0.46	0.96	1.363
0.0040	0.50	0.28	0.38	1.01	1.363
0.0050	0.45	0.28	0.38	0.96	1.482

Watercourse Design Data

Discharge = 290 l/s Side slope = 1.0

Energy slope S	Bottom width b. m	Flow Depth d. m	Total depth D. m	Top width TW m	Vel. n=0.014 V m/s
0.0001	0.55	0.68	0.91	2.37	0.347
0.0002	0.60	0.56	0.76	2.12	0.450
0.0003	0.55	0.52	0.69	1.93	0.523
0.0004	0.45	0.52	0.69	1.83	0.583
0.0005	0.55	0.46	0.61	1.77	0.634
0.0006	0.50	0.45	0.61	1.72	0.679
0.0007	0.45	0.45	0.61	1.67	0.719
0.0008	0.60	0.39	0.53	1.66	0.756
0.0009	0.55	0.39	0.53	1.61	0.790
0.0010	0.50	0.40	0.53	1.56	0.822
0.0015	0.55	0.34	0.46	1.47	0.957
0.0020	0.45	0.34	0.46	1.37	1.066
0.0030	0.35	0.34	0.46	1.27	1.241
0.0030	0.55	0.28	0.38	1.31	1.241
0.0040	0.50	0.27	0.38	1.26	1.383

Watercourse Design Data

Discharge = 290 l/s Side slope = 0.667

Energy slope S	Bottom width b. m	Flow Depth d. m	Total depth D. m	Top width TW m	Vel. n=0.014 V m/s
0.0001	0.50	0.80	1.07	1.93	0.351
0.0002	0.60	0.63	0.84	1.72	0.455
0.0003	0.60	0.56	0.76	1.61	0.530
0.0004	0.60	0.52	0.69	1.52	0.591
0.0005	0.55	0.51	0.69	1.47	0.642
0.0006	0.50	0.50	0.69	1.42	0.688
0.0007	0.60	0.44	0.61	1.41	0.729
0.0008	0.55	0.45	0.61	1.36	0.766
0.0009	0.50	0.45	0.61	1.31	0.801
0.0010	0.45	0.46	0.61	1.26	0.833
0.0015	0.50	0.39	0.53	1.21	0.970
0.0020	0.45	0.38	0.53	1.16	1.080
0.0030	0.45	0.34	0.46	1.06	1.257
0.0040	0.40	0.33	0.46	1.01	1.401
0.0050	0.50	0.28	0.38	1.01	1.523

Table D1. Irrigation channels dimensions for Brick lining with 10 and 20 L/S.

Double Brick w/brick floor
Discharge = 10 l/s

Energy slope S	Bottom width b. m	Flow depth d. m	Vel. V m/s	Total depth D. m
0.0001	0.84	0.09	0.146	0.19
0.0001	0.48	0.14	0.146	0.27
0.0001	0.36	0.19	0.146	0.35
0.0002	0.72	0.08	0.189	0.19
0.0002	0.36	0.15	0.189	0.27
0.0003	0.60	0.08	0.221	0.19
0.0003	0.36	0.13	0.221	0.27
0.0004	0.60	0.08	0.246	0.19
0.0004	0.36	0.11	0.246	0.27
0.0005	0.48	0.08	0.267	0.19
0.0006	0.48	0.08	0.286	0.19
0.0007	0.36	0.09	0.303	0.19
0.0008	0.36	0.09	0.319	0.19
0.0009	0.36	0.09	0.333	0.19
0.0010	0.36	0.08	0.346	0.19
0.0015	0.36	0.07	0.403	0.19
0.0020	0.36	0.07	0.449	0.19
0.0030	0.36	0.06	0.523	0.19
0.0040	0.36	0.05	0.583	0.19
0.0050	0.36	0.05	0.634	0.19
0.0060	0.36	0.05	0.678	0.19
0.0070	0.36	0.04	0.719	0.19
0.0080	0.36	0.04	0.756	0.19
0.0090	0.36	0.04	0.790	0.19
0.0100	0.36	0.04	0.822	0.19
0.0110	0.36	0.04	0.851	0.19

Single Brick w/brick floor
Discharge = 10 l/s

Energy slope S	Bottom width b. m	Flow depth d. m	Vel. V m/s	Total depth D. m
0.0001	0.48	0.14	0.146	0.24
0.0001	0.36	0.19	0.146	0.32
0.0002	0.48	0.11	0.189	0.24
0.0003	0.36	0.13	0.221	0.24
0.0004	0.84	0.06	0.246	0.16
0.0004	0.36	0.11	0.246	0.24
0.0005	0.72	0.06	0.267	0.16
0.0005	0.36	0.11	0.267	0.24
0.0006	0.72	0.06	0.286	0.16
0.0006	0.36	0.10	0.286	0.24
0.0007	0.60	0.06	0.303	0.16
0.0007	0.36	0.06	0.303	0.24
0.0008	0.60	0.06	0.319	0.16
0.0008	0.36	0.09	0.319	0.24
0.0009	0.60	0.06	0.333	0.16
0.0009	0.36	0.09	0.333	0.24
0.0010	0.60	0.06	0.346	0.16
0.0010	0.36	0.08	0.346	0.24
0.0015	0.48	0.06	0.403	0.16
0.0020	0.48	0.05	0.449	0.16
0.0030	0.36	0.06	0.523	0.16
0.0040	0.36	0.05	0.583	0.16
0.0050	0.36	0.05	0.634	0.16
0.0060	0.36	0.05	0.678	0.16
0.0070	0.36	0.04	0.719	0.16
0.0080	0.36	0.04	0.756	0.16
0.0090	0.36	0.04	0.790	0.16
0.0100	0.36	0.04	0.822	0.16
0.0110	0.36	0.04	0.851	0.16

Double Brick w/brick floor
Discharge = 20 l/s

Energy slope S	Bottom width b. m	Flow depth d. m	Vel. V m/s	Total depth D. m
0.0001	0.72	0.17	0.174	0.27
0.0001	0.48	0.24	0.174	0.35
0.0002	0.60	0.15	0.225	0.27
0.0002	0.36	0.25	0.225	0.35
0.0003	0.48	0.16	0.262	0.27
0.0004	0.48	0.14	0.292	0.27
0.0005	0.84	0.09	0.318	0.19
0.0005	0.36	0.17	0.318	0.27
0.0006	0.72	0.09	0.340	0.19
0.0006	0.36	0.16	0.340	0.27
0.0007	0.72	0.09	0.360	0.19
0.0008	0.72	0.08	0.379	0.19
0.0008	0.36	0.15	0.379	0.27
0.0009	0.60	0.09	0.396	0.19
0.0010	0.60	0.09	0.412	0.19
0.0015	0.48	0.09	0.480	0.19
0.0020	0.48	0.08	0.534	0.19
0.0030	0.36	0.09	0.622	0.19
0.0040	0.36	0.08	0.693	0.19
0.0050	0.36	0.08	0.753	0.19
0.0060	0.36	0.07	0.807	0.19
0.0070	0.36	0.07	0.855	0.19
0.0080	0.36	0.07	0.899	0.19
0.0090	0.36	0.06	0.939	0.19
0.0100	0.36	0.06	0.977	0.19

Single Brick w/brick floor
Discharge = 20 l/s

Energy slope S	Bottom width b. m	Flow depth d. m	Vel. V m/s	Total depth D. m
0.0001	0.96	0.13	0.174	0.24
0.0001	0.60	0.19	0.174	0.32
0.0002	0.72	0.13	0.225	0.24
0.0002	0.48	0.19	0.225	0.32
0.0002	0.36	0.25	0.225	0.40
0.0003	0.60	0.13	0.262	0.24
0.0003	0.36	0.21	0.262	0.32
0.0004	0.48	0.14	0.292	0.24
0.0004	0.36	0.19	0.292	0.32
0.0005	0.48	0.13	0.318	0.24
0.0006	0.48	0.13	0.340	0.24
0.0007	0.48	0.12	0.360	0.24
0.0008	0.48	0.11	0.379	0.24
0.0009	0.36	0.14	0.396	0.24
0.0010	0.96	0.06	0.412	0.16
0.0010	0.36	0.14	0.412	0.24
0.0015	0.84	0.06	0.480	0.16
0.0015	0.36	0.12	0.480	0.24
0.0020	0.72	0.06	0.534	0.16
0.0020	0.36	0.11	0.534	0.24
0.0030	0.60	0.06	0.622	0.16
0.0040	0.60	0.06	0.693	0.16
0.0050	0.48	0.06	0.753	0.16
0.0060	0.48	0.06	0.807	0.16
0.0070	0.48	0.06	0.855	0.16
0.0080	0.48	0.05	0.899	0.16
0.0090	0.36	0.06	0.939	0.16
0.0100	0.36	0.06	0.977	0.16

Table D2. Irrigation channels dimensions for Brick lining with 50 and 60 L/S.

Hydraulic Tables for Rectangular Brick lined Channe

Table 37.1

Double Brick w/brick floor
Discharge = 50 l/s

Energy slope S	Bottom width b. m	Flow depth d. m	Vel. V m/s	Total depth D. m
0.0001	0.96	0.25	0.218	0.35
0.0001	0.72	0.32	0.218	0.43
0.0002	0.72	0.25	0.283	0.35
0.0002	0.60	0.29	0.283	0.43
0.0003	0.60	0.25	0.330	0.35
0.0003	0.48	0.32	0.330	0.43
0.0004	0.60	0.23	0.367	0.35
0.0004	0.48	0.28	0.367	0.43
0.0005	0.84	0.16	0.399	0.27
0.0005	0.60	0.21	0.399	0.35
0.0006	0.72	0.17	0.428	0.27
0.0006	0.48	0.24	0.428	0.35
0.0007	0.72	0.16	0.453	0.27
0.0007	0.48	0.23	0.453	0.35
0.0008	0.72	0.15	0.476	0.27
0.0008	0.48	0.22	0.476	0.35
0.0009	0.60	0.17	0.498	0.27
0.0009	0.48	0.21	0.498	0.35
0.0010	0.60	0.16	0.518	0.27
0.0010	0.48	0.20	0.518	0.35
0.0015	0.48	0.17	0.603	0.27
0.0015	0.36	0.23	0.603	0.35
0.0020	0.48	0.16	0.672	0.27
0.0030	0.84	0.09	0.782	0.19
0.0030	0.48	0.14	0.782	0.27
0.0040	0.72	0.09	0.871	0.19
0.0040	0.36	0.16	0.871	0.27
0.0050	0.72	0.08	0.947	0.19
0.0050	0.36	0.15	0.947	0.27
0.0060	0.60	0.09	1.014	0.19
0.0060	0.36	0.14	1.014	0.27
0.0070	0.60	0.09	1.075	0.19
0.0070	0.36	0.13	1.075	0.27
0.0080	0.60	0.08	1.130	0.19
0.0080	0.36	0.12	1.130	0.27
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

Single Brick w/brick floor
Discharge = 50 l/s

Energy slope S	Bottom width b. m	Flow depth d. m	Vel. V m/s	Total depth D. m
0.0001	0.84	0.28	0.218	0.40
0.0001	0.72	0.32	0.218	0.48
0.0002	0.84	0.22	0.283	0.32
0.0002	0.60	0.29	0.283	0.40
0.0003	0.72	0.21	0.330	0.32
0.0003	0.60	0.25	0.330	0.40
0.0004	0.72	0.19	0.367	0.32
0.0004	0.48	0.28	0.367	0.40
0.0005	0.96	0.14	0.399	0.24
0.0005	0.60	0.21	0.399	0.32
0.0006	0.96	0.13	0.428	0.24
0.0006	0.60	0.20	0.428	0.32
0.0007	0.84	0.14	0.453	0.24
0.0007	0.60	0.19	0.453	0.32
0.0008	0.84	0.13	0.476	0.24
0.0008	0.48	0.22	0.476	0.32
0.0009	0.84	0.13	0.498	0.24
0.0009	0.48	0.21	0.498	0.32
0.0010	0.72	0.14	0.518	0.24
0.0010	0.48	0.20	0.518	0.32
0.0015	0.60	0.14	0.603	0.24
0.0015	0.48	0.17	0.603	0.32
0.0020	0.60	0.13	0.672	0.24
0.0020	0.36	0.21	0.672	0.32
0.0030	0.48	0.14	0.782	0.24
0.0040	0.48	0.12	0.871	0.24
0.0050	0.48	0.11	0.947	0.24
0.0060	0.36	0.14	1.014	0.24
0.0070	0.36	0.13	1.075	0.24
0.0080	0.36	0.12	1.130	0.24
0.0090	0.36	0.12	1.181	0.24

Double Brick w/brick floor
Discharge = 60 l/s

Energy slope S	Bottom width b. m	Flow depth d. m	Vel. V m/s	Total depth D. m
0.0001	0.84	0.31	0.229	0.43
0.0001	0.72	0.36	0.229	0.51
0.0002	0.84	0.25	0.297	0.35
0.0002	0.72	0.28	0.297	0.43
0.0003	0.72	0.24	0.345	0.35
0.0003	0.60	0.29	0.345	0.43
0.0004	0.72	0.22	0.385	0.35
0.0004	0.60	0.26	0.385	0.43
0.0005	0.60	0.24	0.418	0.35
0.0005	0.48	0.30	0.418	0.43
0.0006	0.84	0.17	0.448	0.27
0.0006	0.60	0.22	0.448	0.35
0.0007	0.84	0.16	0.474	0.27
0.0007	0.60	0.21	0.474	0.35
0.0008	0.72	0.17	0.499	0.27
0.0008	0.48	0.25	0.499	0.35
0.0009	0.72	0.17	0.521	0.27
0.0009	0.48	0.24	0.521	0.35
0.0010	0.72	0.16	0.542	0.27
0.0010	0.48	0.23	0.542	0.35
0.0015	0.60	0.16	0.631	0.27
0.0015	0.48	0.20	0.631	0.35
0.0020	0.60	0.19	0.703	0.27
0.0020	0.36	0.24	0.703	0.35
0.0030	0.48	0.15	0.819	0.27
0.0040	0.48	0.14	0.912	0.27
0.0050	0.72	0.09	0.991	0.19
0.0050	0.36	0.17	0.991	0.27
0.0060	0.72	0.09	1.062	0.19
0.0060	0.36	0.16	1.062	0.27
0.0070	0.72	0.08	1.125	0.19
0.0070	0.36	0.15	1.125	0.27
0.0080	0.60	0.09	1.183	0.19
0.0080	0.36	0.14	1.183	0.27
0.0090	0.60	0.09	1.236	0.19
0.0090	0.36	0.14	1.236	0.27

Single Brick w/brick floor
Discharge = 60 l/s

Energy slope S	Bottom width b. m	Flow depth d. m	Vel. V m/s	Total depth D. m
0.0001	0.96	0.28	0.229	0.40
0.0001	0.84	0.31	0.229	0.48
0.0002	0.96	0.22	0.297	0.32
0.0002	0.72	0.28	0.297	0.40
0.0003	0.84	0.21	0.345	0.32
0.0003	0.60	0.29	0.345	0.40
0.0004	0.72	0.22	0.385	0.32
0.0004	0.60	0.26	0.385	0.40
0.0005	0.72	0.20	0.418	0.32
0.0005	0.48	0.30	0.418	0.40
0.0006	0.60	0.22	0.448	0.32
0.0006	0.48	0.28	0.448	0.40
0.0007	0.60	0.21	0.474	0.32
0.0008	0.60	0.20	0.499	0.32
0.0009	0.60	0.19	0.521	0.32
0.0010	0.84	0.14	0.542	0.24
0.0010	0.60	0.19	0.542	0.32
0.0015	0.72	0.14	0.631	0.24
0.0015	0.48	0.20	0.631	0.32
0.0020	0.72	0.13	0.703	0.24
0.0020	0.48	0.18	0.703	0.32
0.0030	0.60	0.13	0.819	0.24
0.0030	0.36	0.20	0.819	0.32
0.0040	0.48	0.14	0.912	0.24
0.0050	0.48	0.13	0.991	0.24
0.0060	0.48	0.12	1.062	0.24
0.0070	0.48	0.11	1.125	0.24
0.0080	0.36	0.14	1.183	0.24
0.0090	0.96	0.04	1.236	0.16

Table D3. Irrigation channels dimensions for Brick lining with 100 and 120 L/S.

Discharge = 100 l/s

Energy slope S	Bottom width b. m	Flow depth d. m	Vel. V m/s	Total depth D. m
0.0001	0.96	0.40	0.260	0.59
0.0001	0.84	0.46	0.260	0.67
0.0002	0.96	0.31	0.337	0.43
0.0002	0.84	0.35	0.337	0.51
0.0003	0.84	0.31	0.392	0.43
0.0003	0.72	0.35	0.392	0.51
0.0004	0.96	0.25	0.437	0.35
0.0004	0.72	0.32	0.437	0.43
0.0005	0.84	0.25	0.475	0.35
0.0005	0.72	0.29	0.475	0.43
0.0006	0.84	0.24	0.509	0.35
0.0006	0.72	0.27	0.509	0.43
0.0007	0.84	0.23	0.539	0.35
0.0007	0.60	0.31	0.539	0.43
0.0008	0.72	0.25	0.567	0.35
0.0008	0.60	0.29	0.567	0.43
0.0009	0.72	0.24	0.592	0.35
0.0009	0.60	0.28	0.592	0.43
0.0010	0.72	0.23	0.616	0.35
0.0010	0.60	0.27	0.616	0.43
0.0015	0.84	0.17	0.717	0.27
0.0015	0.60	0.23	0.717	0.35
0.0020	0.84	0.16	0.799	0.27
0.0020	0.60	0.21	0.799	0.35
0.0030	0.72	0.16	0.930	0.27
0.0030	0.48	0.22	0.930	0.35
0.0040	0.60	0.16	1.036	0.27
0.0040	0.48	0.20	1.036	0.35
0.0050	0.60	0.15	1.127	0.27
0.0050	0.36	0.25	1.127	0.35
0.0060	0.48	0.17	1.206	0.27
0.0060	0.36	0.23	1.206	0.35
0.0070	0.48	0.16	1.278	0.27
0.0070	0.36	0.22	1.278	0.35
0.0080	0.48	0.16	1.344	0.27

Discharge = 100 l/s

Energy slope S	Bottom width b. m	Flow depth d. m	Vel. V m/s	Total depth D. m
0.0001	0.96	0.40	0.260	0.56
0.0001	0.84	0.46	0.260	0.64
0.0002	0.84	0.35	0.337	0.48
0.0002	0.72	0.41	0.337	0.56
0.0003	0.96	0.27	0.392	0.40
0.0003	0.72	0.35	0.392	0.48
0.0004	0.84	0.28	0.437	0.40
0.0004	0.72	0.32	0.437	0.48
0.0005	0.72	0.29	0.475	0.40
0.0005	0.60	0.35	0.475	0.48
0.0006	0.96	0.21	0.509	0.32
0.0006	0.72	0.27	0.509	0.40
0.0007	0.96	0.20	0.539	0.32
0.0007	0.72	0.26	0.539	0.40
0.0008	0.84	0.22	0.567	0.32
0.0008	0.60	0.29	0.567	0.40
0.0009	0.84	0.21	0.592	0.32
0.0009	0.60	0.28	0.592	0.40
0.0010	0.84	0.20	0.616	0.32
0.0010	0.60	0.27	0.616	0.40
0.0015	0.72	0.20	0.717	0.32
0.0015	0.48	0.29	0.717	0.40
0.0020	0.60	0.21	0.799	0.32
0.0030	0.48	0.22	0.930	0.32
0.0040	0.48	0.20	1.036	0.32
0.0050	0.72	0.13	1.127	0.24
0.0050	0.48	0.19	1.127	0.32
0.0060	0.60	0.14	1.206	0.24
0.0060	0.48	0.17	1.206	0.32
0.0070	0.36	0.22	1.278	0.32
0.0080	0.36	0.21	1.344	0.32

Energy slope S	Bottom width b. m	Flow depth d. m	Vel. V m/s	Total depth D. m
0.0001	0.96	0.46	0.272	0.67
0.0001	0.84	0.53	0.272	0.75
0.0002	0.96	0.36	0.353	0.51
0.0002	0.84	0.41	0.353	0.59
0.0003	0.96	0.31	0.411	0.43
0.0003	0.84	0.35	0.411	0.51
0.0004	0.84	0.31	0.457	0.43
0.0004	0.72	0.36	0.457	0.51
0.0005	0.84	0.29	0.497	0.43
0.0005	0.72	0.34	0.497	0.51
0.0006	0.96	0.24	0.532	0.35
0.0006	0.72	0.31	0.532	0.43
0.0007	0.96	0.23	0.564	0.35
0.0007	0.72	0.30	0.564	0.43
0.0008	0.84	0.25	0.593	0.35
0.0008	0.72	0.28	0.593	0.43
0.0009	0.84	0.24	0.620	0.35
0.0009	0.60	0.32	0.620	0.43
0.0010	0.84	0.23	0.645	0.35
0.0010	0.60	0.31	0.645	0.43
0.0015	0.72	0.23	0.751	0.35
0.0015	0.60	0.27	0.751	0.43
0.0020	0.96	0.16	0.836	0.27
0.0020	0.60	0.24	0.836	0.35
0.0030	0.84	0.16	0.974	0.27
0.0030	0.60	0.21	0.974	0.35
0.0040	0.72	0.16	1.084	0.27
0.0040	0.48	0.23	1.084	0.35
0.0050	0.60	0.17	1.179	0.27
0.0050	0.48	0.21	1.179	0.35
0.0060	0.60	0.16	1.263	0.27
0.0060	0.48	0.20	1.263	0.35
0.0070	0.60	0.15	1.338	0.27
0.0070	0.36	0.25	1.338	0.35
0.0080	0.60	0.15	1.406	0.27
0.0080	0.36	0.24	1.406	0.35

Energy slope S	Bottom width b. m	Flow depth d. m	Vel. V m/s	Total depth D. m
0.0001	0.96	0.46	0.272	0.64
0.0001	0.84	0.53	0.272	0.72
0.0002	0.96	0.36	0.353	0.48
0.0002	0.84	0.41	0.353	0.56
0.0003	0.84	0.35	0.411	0.48
0.0003	0.72	0.41	0.411	0.56
0.0004	0.96	0.28	0.457	0.40
0.0004	0.84	0.31	0.457	0.48
0.0005	0.84	0.29	0.497	0.40
0.0005	0.72	0.34	0.497	0.48
0.0006	0.84	0.27	0.532	0.40
0.0006	0.72	0.31	0.532	0.48
0.0007	0.72	0.30	0.564	0.40
0.0007	0.60	0.36	0.564	0.48
0.0008	0.96	0.22	0.593	0.32
0.0008	0.72	0.28	0.593	0.40
0.0009	0.96	0.21	0.620	0.32
0.0009	0.72	0.27	0.620	0.40
0.0010	0.96	0.20	0.645	0.32
0.0010	0.72	0.26	0.645	0.40
0.0015	0.84	0.20	0.751	0.32
0.0015	0.60	0.27	0.751	0.40
0.0020	0.72	0.20	0.836	0.32
0.0020	0.48	0.30	0.836	0.40
0.0030	0.60	0.21	0.974	0.32
0.0040	0.60	0.19	1.084	0.32
0.0050	0.48	0.21	1.179	0.32
0.0060	0.48	0.20	1.263	0.32
0.0070	0.48	0.19	1.338	0.32
0.0080	0.72	0.13	1.406	0.24
0.0080	0.48	0.18	1.406	0.32

Table D4. Irrigation channels dimensions for Brick lining with 200and 230 L/S.

Hydraulic Tables for Rectangular Brick l

Double Brick w/brick floor Discharge = 200 l/s				
Energy slope S	Bottom width b. m	Flow depth d. m	Vel. V m/s	Total depth D. m
0.0001	0.96	0.68	0.309	0.91
0.0002	0.96	0.52	0.401	0.75
0.0002	0.84	0.60	0.401	0.83
0.0003	0.96	0.45	0.466	0.67
0.0003	0.84	0.51	0.466	0.75
0.0004	0.96	0.40	0.520	0.59
0.0004	0.84	0.46	0.520	0.67
0.0005	0.96	0.37	0.565	0.51
0.0005	0.84	0.42	0.565	0.59
0.0006	0.96	0.35	0.605	0.51
0.0006	0.84	0.39	0.605	0.59
0.0007	0.84	0.37	0.641	0.51
0.0007	0.72	0.43	0.641	0.59
0.0008	0.96	0.31	0.674	0.43
0.0008	0.84	0.35	0.674	0.51
0.0009	0.96	0.30	0.704	0.43
0.0009	0.84	0.34	0.704	0.51
0.0010	0.84	0.33	0.733	0.43
0.0010	0.72	0.38	0.733	0.51
0.0015	0.96	0.25	0.853	0.35
0.0015	0.72	0.33	0.853	0.43
0.0020	0.84	0.25	0.950	0.35
0.0020	0.72	0.29	0.950	0.43
0.0030	0.72	0.25	1.106	0.35
0.0030	0.60	0.30	1.106	0.43
0.0040	0.72	0.23	1.232	0.35
0.0040	0.60	0.27	1.232	0.43
0.0050	0.96	0.17	1.340	0.27
0.0050	0.60	0.25	1.340	0.35
0.0060	0.84	0.17	1.435	0.27
0.0060	0.60	0.23	1.435	0.35
0.0070	0.84	0.17	1.520	0.27

Single Brick w/brick floor Discharge = 200 l/s				
Energy slope S	Bottom width b. m	Flow depth d. m	Vel. V m/s	Total depth D. m
0.0001	0.96	0.68	0.309	0.96
0.0002	0.96	0.52	0.401	0.72
0.0002	0.84	0.60	0.401	0.80
0.0003	0.96	0.45	0.466	0.64
0.0003	0.84	0.51	0.466	0.72
0.0004	0.96	0.40	0.520	0.56
0.0004	0.84	0.46	0.520	0.64
0.0005	0.84	0.42	0.565	0.56
0.0005	0.72	0.49	0.565	0.72
0.0006	0.96	0.35	0.605	0.48
0.0006	0.84	0.39	0.605	0.56
0.0007	0.96	0.33	0.641	0.48
0.0007	0.84	0.37	0.641	0.56
0.0008	0.84	0.35	0.674	0.48
0.0008	0.72	0.41	0.674	0.56
0.0009	0.96	0.30	0.704	0.40
0.0009	0.84	0.34	0.704	0.48
0.0010	0.96	0.29	0.733	0.40
0.0010	0.84	0.33	0.733	0.48
0.0015	0.84	0.28	0.853	0.40
0.0015	0.72	0.33	0.853	0.48
0.0020	0.72	0.29	0.950	0.40
0.0020	0.60	0.35	0.950	0.48
0.0030	0.84	0.22	1.106	0.32
0.0030	0.60	0.30	1.106	0.40
0.0040	0.84	0.20	1.232	0.32
0.0040	0.60	0.27	1.232	0.40
0.0050	0.72	0.21	1.340	0.32
0.0050	0.60	0.25	1.340	0.40
0.0060	0.72	0.20	1.435	0.32
0.0060	0.48	0.29	1.435	0.40
0.0070	0.60	0.22	1.520	0.32

Double Brick w/brick floor Discharge = 230 l/s				
Energy slope S	Bottom width b. m	Flow depth d. m	Vel. V m/s	Total depth D. m
0.0001	0.96	0.76	0.320	1.07
0.0002	0.96	0.58	0.415	0.83
0.0002	0.84	0.67	0.415	0.91
0.0003	0.96	0.50	0.483	0.67
0.0003	0.84	0.57	0.483	0.83
0.0004	0.96	0.45	0.538	0.59
0.0004	0.84	0.51	0.538	0.75
0.0005	0.96	0.41	0.585	0.59
0.0005	0.84	0.47	0.585	0.67
0.0006	0.96	0.38	0.626	0.51
0.0006	0.84	0.44	0.626	0.59
0.0007	0.96	0.36	0.664	0.51
0.0007	0.84	0.41	0.664	0.59
0.0008	0.96	0.35	0.698	0.51
0.0008	0.84	0.39	0.698	0.59
0.0009	0.84	0.38	0.729	0.51
0.0009	0.72	0.44	0.729	0.59
0.0010	0.96	0.32	0.759	0.43
0.0010	0.84	0.36	0.759	0.51
0.0015	0.84	0.31	0.883	0.43
0.0015	0.72	0.36	0.883	0.51
0.0020	0.96	0.25	0.984	0.35
0.0020	0.72	0.32	0.984	0.43
0.0030	0.84	0.24	1.145	0.35
0.0030	0.72	0.28	1.145	0.43
0.0040	0.72	0.25	1.276	0.35
0.0040	0.60	0.30	1.276	0.43
0.0050	0.72	0.23	1.387	0.35
0.0050	0.60	0.28	1.387	0.43
0.0060	0.96	0.17	1.486	0.27
0.0060	0.72	0.22	1.486	0.35
0.0070	0.96	0.16	1.574	0.27

Single Brick w/brick floor Discharge = 230 l/s				
Energy slope S	Bottom width b. m	Flow depth d. m	Vel. V m/s	Total depth D. m
0.0001	0.96	0.76	0.320	1.04
0.0002	0.96	0.58	0.415	0.80
0.0003	0.96	0.50	0.483	0.72
0.0003	0.84	0.57	0.483	0.80
0.0004	0.96	0.45	0.538	0.64
0.0004	0.84	0.51	0.538	0.72
0.0005	0.96	0.41	0.585	0.56
0.0005	0.84	0.47	0.585	0.64
0.0006	0.96	0.38	0.626	0.56
0.0006	0.84	0.44	0.626	0.64
0.0007	0.96	0.36	0.664	0.48
0.0007	0.84	0.41	0.664	0.56
0.0008	0.96	0.35	0.698	0.48
0.0008	0.84	0.39	0.698	0.56
0.0009	0.96	0.33	0.729	0.48
0.0009	0.84	0.38	0.729	0.56
0.0010	0.84	0.36	0.759	0.48
0.0010	0.72	0.42	0.759	0.56
0.0015	0.72	0.36	0.883	0.48
0.0015	0.96	0.28	0.883	0.40
0.0020	0.84	0.28	0.984	0.40
0.0020	0.72	0.32	0.984	0.48
0.0030	0.96	0.22	1.145	0.32
0.0030	0.72	0.28	1.145	0.40
0.0040	0.84	0.22	1.276	0.32
0.0040	0.60	0.30	1.276	0.40
0.0050	0.84	0.20	1.387	0.32
0.0050	0.60	0.28	1.387	0.40
0.0060	0.72	0.22	1.486	0.32
0.0060	0.60	0.26	1.486	0.40
0.0070	0.72	0.21	1.574	0.32
0.0070	0.60	0.24	1.574	0.40

Table D5. Irrigation channels dimensions for Brick lining with 260 and 290 L/S.

Table 47.1

Double Brick w/brick floor Discharge = 260 l/s				
Energy slope S	Bottom width b. m	Flow depth d. m	Vel. V m/s	Total depth D. m
0.0001	0.96	0.84	0.330	1.15
0.0002	0.96	0.64	0.428	0.91
0.0002	0.84	0.74	0.428	0.99
0.0003	0.96	0.54	0.498	0.75
0.0004	0.96	0.49	0.555	0.67
0.0004	0.84	0.56	0.555	0.75
0.0005	0.96	0.45	0.603	0.67
0.0005	0.84	0.51	0.603	0.75
0.0006	0.96	0.42	0.646	0.59
0.0006	0.84	0.48	0.646	0.67
0.0007	0.96	0.40	0.684	0.59
0.0007	0.84	0.45	0.684	0.67
0.0008	0.96	0.38	0.720	0.51
0.0008	0.84	0.43	0.720	0.59
0.0009	0.96	0.36	0.752	0.51
0.0009	0.84	0.41	0.752	0.59
0.0010	0.96	0.35	0.782	0.51
0.0010	0.84	0.40	0.782	0.59
0.0015	0.96	0.30	0.911	0.43
0.0015	0.84	0.34	0.991	0.51
0.0020	0.84	0.31	1.015	0.43
0.0020	0.72	0.36	1.015	0.51
0.0030	0.96	0.24	1.181	0.35
0.0030	0.72	0.31	1.181	0.43
0.0040	0.84	0.24	1.316	0.35
0.0040	0.72	0.28	1.316	0.43
0.0050	0.72	0.25	1.431	0.35
0.0050	0.60	0.30	1.431	0.43
0.0060	0.72	0.24	1.532	0.35
0.0060	0.60	0.28	1.532	0.43
0.0070	0.72	0.23	1.623	0.35

Single Brick w/brick floor Discharge = 260 l/s				
Energy slope S	Bottom width b. m	Flow depth d. m	Vel. V m/s	Total depth D. m
0.0001	0.96	0.84	0.330	1.12
0.0002	0.96	0.64	0.428	0.88
0.0003	0.96	0.54	0.498	0.80
0.0003	0.84	0.63	0.498	0.88
0.0004	0.96	0.49	0.555	0.72
0.0004	0.84	0.56	0.555	0.80
0.0005	0.96	0.45	0.603	0.64
0.0005	0.84	0.51	0.603	0.72
0.0006	0.96	0.42	0.646	0.56
0.0006	0.84	0.48	0.646	0.64
0.0007	0.96	0.40	0.684	0.56
0.0007	0.84	0.45	0.684	0.64
0.0008	0.96	0.38	0.720	0.56
0.0008	0.84	0.43	0.720	0.64
0.0009	0.96	0.36	0.752	0.48
0.0009	0.84	0.41	0.752	0.56
0.0010	0.96	0.35	0.782	0.48
0.0010	0.84	0.40	0.782	0.56
0.0015	0.96	0.30	0.911	0.40
0.0015	0.84	0.34	0.911	0.48
0.0020	0.96	0.27	1.015	0.40
0.0020	0.84	0.31	1.015	0.48
0.0030	0.84	0.27	1.181	0.40
0.0030	0.72	0.31	1.181	0.48
0.0040	0.96	0.21	1.316	0.32
0.0040	0.72	0.28	1.316	0.40
0.0050	0.84	0.22	1.431	0.32
0.0050	0.60	0.30	1.431	0.40
0.0060	0.84	0.21	1.532	0.32
0.0060	0.60	0.28	1.532	0.40
0.0070	0.84	0.20	1.623	0.32

Double Brick w/brick floor Discharge = 290 l/s				
Energy slope S	Bottom width b. m	Flow depth d. m	Vel. V m/s	Total depth D. m
0.0001	0.96	0.91	0.339	1.23
0.0002	0.96	0.69	0.440	0.99
0.0003	0.96	0.59	0.512	0.83
0.0003	0.84	0.68	0.512	0.91
0.0004	0.96	0.53	0.570	0.75
0.0004	0.84	0.61	0.570	0.83
0.0005	0.84	0.56	0.620	0.75
0.0005	0.72	0.66	0.620	0.91
0.0006	0.96	0.46	0.664	0.67
0.0006	0.84	0.52	0.664	0.75
0.0007	0.96	0.43	0.703	0.59
0.0007	0.84	0.49	0.703	0.67
0.0008	0.96	0.41	0.739	0.59
0.0008	0.84	0.47	0.739	0.67
0.0009	0.96	0.39	0.773	0.59
0.0009	0.84	0.45	0.773	0.67
0.0010	0.96	0.38	0.804	0.51
0.0010	0.84	0.43	0.804	0.59
0.0015	0.96	0.33	0.936	0.43
0.0015	0.84	0.37	0.936	0.51
0.0020	0.96	0.29	1.043	0.43
0.0020	0.84	0.33	1.043	0.51
0.0030	0.84	0.29	1.214	0.43
0.0030	0.72	0.33	1.214	0.51
0.0040	0.96	0.23	1.352	0.35
0.0040	0.84	0.26	1.352	0.43
0.0050	0.84	0.24	1.470	0.35
0.0050	0.72	0.28	1.470	0.43
0.0060	0.84	0.22	1.574	0.35
0.0060	0.60	0.31	1.574	0.43
0.0070	0.72	0.24	1.668	0.35

Single Brick w/brick floor Discharge = 290 l/s				
Energy slope S	Bottom width b. m	Flow depth d. m	Vel. V m/s	Total depth D. m
0.0001	0.96	0.91	0.339	1.28
0.0002	0.96	0.69	0.440	0.96
0.0003	0.96	0.59	0.512	0.80
0.0003	0.84	0.68	0.512	0.96
0.0004	0.96	0.53	0.570	0.72
0.0004	0.84	0.61	0.570	0.88
0.0005	0.96	0.49	0.620	0.72
0.0005	0.84	0.56	0.620	0.80
0.0006	0.96	0.46	0.664	0.64
0.0006	0.84	0.52	0.664	0.72
0.0007	0.96	0.43	0.703	0.64
0.0007	0.84	0.49	0.703	0.72
0.0008	0.96	0.41	0.739	0.56
0.0008	0.84	0.47	0.739	0.64
0.0009	0.96	0.39	0.773	0.56
0.0009	0.84	0.45	0.773	0.64
0.0010	0.96	0.38	0.804	0.56
0.0010	0.84	0.43	0.804	0.64
0.0015	0.96	0.33	0.936	0.48
0.0015	0.84	0.37	0.936	0.56
0.0020	0.96	0.29	1.043	0.40
0.0020	0.84	0.33	1.043	0.48
0.0030	0.84	0.29	1.214	0.40
0.0030	0.72	0.33	1.214	0.48
0.0040	0.72	0.30	1.352	0.40
0.0040	0.60	0.36	1.352	0.48
0.0050	0.96	0.21	1.470	0.32
0.0050	0.72	0.28	1.470	0.40
0.0060	0.84	0.22	1.574	0.32
0.0060	0.72	0.26	1.574	0.40
0.0070	0.84	0.21	1.668	[illegible]

ANNEXURE E: DESIGN OF UNLINED TRAPEZOIDAL CANALS

Table E1. Unlined water course design for Q = 20L/s with vegetation.

Watercourse Design Data										
Discharge =20 l/sec			Side Slope = 1.5			Retardance Class – E (shorter than 5 cm)				
Energy slope S	Manning n	Vel. V m/s	Bottom width b. m	Flow depth d. m	Free board FB m	Total depth d. m	Top Width TW m	Bank width BW m	Pad width PW m	Vel. n=0.025 V m/s
0.0001	0.080	0.046	0.30	0.45	0.15	0.60	2.09	0.45	3.43	0.111
0.0002	0.075	0.063	0.30	0.37	0.15	0.52	1.86	0.37	3.05	0.143
0.0003	0.072	0.076	0.30	0.33	0.15	0.48	1.74	0.33	2.86	0.166
0.0004	0.070	0.086	0.30	0.31	0.15	0.46	1.67	0.31	2.73	0.185
0.0005	0.069	0.095	0.30	0.29	0.15	0.44	1.61	0.30	2.66	0.201
0.0006	0.068	0.103	0.30	0.27	0.15	0.42	1.57	0.30	2.62	0.215
0.0007	0.067	0.110	0.30	0.26	0.15	0.41	1.54	0.30	2.59	0.227
0.0008	0.066	0.116	0.30	0.25	0.15	0.40	1.51	0.30	2.56	0.239
0.0009	0.065	0.122	0.30	0.24	0.15	0.39	1.48	0.30	2.53	0.249
0.0010	0.065	0.128	0.30	0.24	0.15	0.39	1.46	0.30	2.51	0.259
0.0015	0.063	0.153	0.30	0.21	0.15	0.36	1.39	0.30	2.44	0.300
0.0020	0.061	0.173	0.30	0.20	0.15	0.35	1.34	0.30	2.39	0.333
0.0030	0.059	0.205	0.30	0.17	0.15	0.32	1.27	0.30	2.32	0.386
0.0040	0.058	0.232	0.30	0.16	0.15	0.31	1.23	0.30	2.28	0.428
0.0050	0.057	0.255	0.30	0.15	0.15	0.30	1.20	0.30	2.29	0.464
0.0060	0.056	0.275	0.30	0.14	0.15	0.29	1.18	0.30	2.23	0.495
0.0070	0.056	0.293	0.30	0.14	0.15	0.29	1.16	0.30	2.21	0.523
0.0080	0.055	0.309	0.30	0.13	0.15	0.28	1.14	0.30	2.19	0.549
0.0090	0.055	0.325	0.30	0.13	0.15	0.28	1.13	0.30	2.18	0.572

Table E2. Unlined water course design for Q = 20 L/S with no vegetation..

Watercourse Design Data										
Discharge = 20 l/sec			Side Slope = 1.5			Retardance Class – No vegetation				
Energy slope S	Manning n	Vel. V m/s	Bottom width b. m	Flow depth d. m	Free board FB m	Total depth d. m	Top Width TW m	Bank width BW m	Pad width PW m	Vel. n=0.025 V m/s
0.0001	0.032	0.093	0.30	0.29	0.15	0.44	1.63	0.30	2.68	0.111
0.0002	0.031	0.122	0.30	0.25	0.15	0.40	1.49	0.30	2.54	0.143
0.0003	0.031	0.144	0.30	0.22	0.15	0.37	1.41	0.30	2.46	0.166
0.0004	0.030	0.161	0.30	0.20	0.15	0.35	1.36	0.30	2.41	0.185
0.0005	0.030	0.176	0.30	0.19	0.15	0.34	1.33	0.30	2.38	0.201
0.0006	0.030	0.189	0.30	0.18	0.15	0.33	1.30	0.30	2.35	0.215
0.0007	0.030	0.200	0.30	0.18	0.15	0.33	1.28	0.30	2.33	0.227
0.0008	0.029	0.211	0.30	0.17	0.15	0.32	1.26	0.30	2.31	0.239
0.0009	0.029	0.221	0.30	0.17	0.15	0.32	1.25	0.30	2.30	0.249
0.0010	0.029	0.231	0.30	0.16	0.15	0.31	1.23	0.30	2.28	0.259
0.0015	0.029	0.270	0.30	0.14	0.15	0.29	1.18	0.30	2.23	0.300
0.0020	0.029	0.302	0.30	0.13	0.15	0.28	1.15	0.30	2.20	0.333
0.0030	0.028	0.353	0.30	0.12	0.15	0.27	1.11	0.30	2.16	0.386
0.0040	0.028	0.394	0.30	0.11	0.15	0.26	1.08	0.30	2.13	0.428
0.0050	0.028	0.428	0.30	0.10	0.15	0.25	1.06	0.30	2.11	0.464
0.0060	0.028	0.459	0.30	0.10	0.15	0.25	1.04	0.30	2.09	0.495
0.0070	0.028	0.486	0.30	0.09	0.15	0.24	1.03	0.30	2.08	0.523
0.0080	0.028	0.511	0.30	0.09	0.15	0.24	1.02	0.30	2.07	0.548
0.0090	0.027	0.534	0.30	0.09	0.15	0.24	1.01	0.30	2.06	0.572

Table E3. Unlined water course design for Q = 30L/s with vegetation.

Watercourse Design Data										
Discharge = 30 l/sec			Side Slope = 1.5			Retardance Class – E (shorter than 5 cm)				
Energy slope S	Manning n	Vel. V m/s	Bottom width b. m	Flow depth d. m	Free board FB m	Total depth d. m	Top Width TW m	Bank width BW m	Pad width PW m	Vel. n=0.025 V m/s
0.0001	0.070	0.057	0.30	0.50	0.17	0.67	2.31	0.50	3.82	0.123
0.0002	0.066	0.077	0.30	0.42	0.15	0.57	2.01	0.42	3.30	0.159
0.0003	0.064	0.092	0.30	0.38	0.15	0.53	1.88	0.38	3.08	0.185
0.0004	0.062	0.104	0.30	0.35	0.15	0.50	1.80	0.35	2.95	0.206
0.0005	0.061	0.115	0.30	0.33	0.15	0.48	1.74	0.33	2.84	0.223
0.0006	0.060	0.125	0.30	0.31	0.15	0.46	1.69	0.31	2.77	0.239
0.0007	0.059	0.133	0.30	0.30	0.15	0.45	1.65	0.30	2.70	0.253
0.0008	0.059	0.141	0.30	0.29	0.15	0.44	1.62	0.30	2.67	0.266
0.0009	0.058	0.148	0.30	0.28	0.15	0.43	1.59	0.30	2.64	0.278
0.0010	0.058	0.155	0.30	0.27	0.15	0.42	1.57	0.30	2.62	0.289
0.0015	0.056	0.185	0.30	0.24	0.15	0.39	1.48	0.30	2.53	0.335
0.0020	0.055	0.209	0.30	0.23	0.15	0.38	1.43	0.30	2.48	0.372
0.0030	0.053	0.248	0.30	0.20	0.15	0.35	1.35	0.30	2.40	0.431
0.0040	0.052	0.281	0.30	0.19	0.15	0.34	1.31	0.30	2.36	0.479
0.0050	0.051	0.308	0.30	0.17	0.15	0.32	1.27	0.30	2.32	0.519
0.0060	0.050	0.332	0.30	0.16	0.15	0.31	1.24	0.30	2.29	0.555
0.0070	0.050	0.354	0.30	0.16	0.15	0.31	1.22	0.30	2.27	0.587
0.0080	0.050	0.375	0.30	0.15	0.15	0.30	1.21	0.30	2.26	0.616
0.0090	0.049	0.393	0.30	0.15	0.15	0.30	1.19	0.30	2.24	0.642

Table E4. Unlined water course design for Q = 30L/s with no vegetation

Watercourse Design Data										
Discharge = 30 l/sec			Side Slope = 1.5			Retardance Class – No vegetation				
Energy slope S	Manning n	Vel. V m/s	Bottom width b. m	Flow depth d. m	Free board FB m	Total depth d. m	Top Width TW m	Bank width BW m	Pad width PW m	Vel. n=0.025 V m/s
0.0001	0.030	0.107	0.30	0.34	0.15	0.49	1.78	0.34	2.92	0.123
0.0002	0.029	0.141	0.30	0.29	0.15	0.44	1.62	0.30	2.67	0.159
0.0003	0.029	0.166	0.30	0.26	0.15	0.41	1.53	0.30	2.58	0.185
0.0004	0.029	0.186	0.30	0.24	0.15	0.39	1.48	0.30	2.53	0.205
0.0005	0.028	0.203	0.30	0.23	0.15	0.38	1.44	0.30	2.49	0.223
0.0006	0.028	0.218	0.30	0.22	0.15	0.37	1.41	0.30	2.46	0.239
0.0007	0.028	0.232	0.30	0.21	0.15	0.36	1.38	0.30	2.43	0.253
0.0008	0.028	0.244	0.30	0.20	0.15	0.35	1.36	0.30	2.41	0.266
0.0009	0.028	0.256	0.30	0.20	0.15	0.35	1.34	0.30	2.39	0.277
0.0010	0.028	0.267	0.30	0.19	0.15	0.34	1.32	0.30	2.37	0.289
0.0015	0.027	0.312	0.30	0.17	0.15	0.32	1.27	0.30	2.32	0.335
0.0020	0.027	0.349	0.30	0.16	0.15	0.31	1.23	0.30	2.28	0.372
0.0030	0.027	0.408	0.30	0.14	0.15	0.29	1.18	0.30	2.23	0.432
0.0040	0.027	0.456	0.30	0.13	0.15	0.28	1.15	0.30	2.20	0.479
0.0050	0.027	0.497	0.30	0.12	0.15	0.27	1.12	0.30	2.17	0.520
0.0060	0.026	0.533	0.30	0.12	0.15	0.27	1.10	0.30	2.15	0.555
0.0070	0.026	0.565	0.30	0.11	0.15	0.26	1.09	0.30	2.14	0.586
0.0080	0.026	0.594	0.30	0.11	0.15	0.26	1.08	0.30	2.13	0.615
0.0090	0.026	0.621	0.30	0.11	0.15	0.26	1.07	0.30	2.12	0.642

Table E5. Unlined water course design for Q = 40L/s with vegetation.

Watercourse Design Data										
Discharge = 40 l/sec			Side Slope = 1.5			Retardance Class – E (shorter than 5 cm)				
Energy slope S	Manning n	Vel. V m/s	Bottom width b. m	Flow depth d. m	Free board FB m	Total depth d. m	Top Width TW m	Bank width BW m	Pad width PW m	Vel. n=0.025 V m/s
0.0001	0.064	0.065	0.35	0.53	0.18	0.71	2.49	0.53	4.09	0.132
0.0002	0.061	0.088	0.30	0.46	0.15	0.61	2.14	0.46	3.51	0.171
0.0003	0.059	0.105	0.30	0.41	0.15	0.56	1.99	0.41	3.27	0.199
0.0004	0.057	0.119	0.30	0.38	0.15	0.53	1.90	0.38	3.12	0.221
0.0005	0.056	0.132	0.30	0.36	0.15	0.51	1.83	0.36	3.01	0.240
0.0006	0.055	0.142	0.30	0.34	0.15	0.49	1.78	0.34	2.92	0.257
0.0007	0.055	0.152	0.30	0.33	0.15	0.48	1.74	0.33	2.85	0.273
0.0008	0.054	0.161	0.30	0.32	0.15	0.47	1.71	0.32	2.79	0.286
0.0009	0.054	0.169	0.30	0.31	0.15	0.46	1.68	0.31	2.75	0.299
0.0010	0.053	0.177	0.30	0.30	0.15	0.45	1.65	0.30	2.70	0.311
0.0015	0.052	0.211	0.30	0.27	0.15	0.42	1.56	0.30	2.61	0.361
0.0020	0.051	0.238	0.30	0.25	0.15	0.40	1.50	0.30	2.55	0.402
0.0030	0.049	0.283	0.30	0.22	0.15	0.37	1.42	0.30	2.47	0.466
0.0040	0.048	0.320	0.30	0.21	0.15	0.36	1.37	0.30	2.42	0.518
0.0050	0.047	0.351	0.30	0.19	0.15	0.34	1.33	0.30	2.38	0.562
0.0060	0.047	0.379	0.30	0.18	0.15	0.33	1.30	0.30	2.35	0.600
0.0070	0.046	0.404	0.30	0.18	0.15	0.33	1.28	0.30	2.33	0.635
0.0080	0.046	0.427	0.30	0.17	0.15	0.32	1.26	0.30	2.31	0.667
0.0090	0.046	0.448	0.30	0.16	0.15	0.31	1.24	0.30	2.29	0.696

Table E6. Unlined water course design for Q = 40L/s with no vegetation.

Watercourse Design Data										
Discharge = 40 l/sec			Side Slope = 1.5			Retardance Class – No vegetation				
Energy slope S	Manning n	Vel. V m/s	Bottom width b. m	Flow depth d. m	Free board FB m	Total depth d. m	Top Width TW m	Bank width BW m	Pad width PW m	Vel. n=0.025 V m/s
0.0001	0.029	0.118	0.30	0.39	0.15	0.54	1.91	0.39	3.13	0.132
0.0002	0.028	0.156	0.30	0.33	0.15	0.48	1.73	0.33	2.83	0.171
0.0003	0.028	0.183	0.30	0.29	0.15	0.44	1.63	0.30	2.68	0.199
0.0004	0.028	0.205	0.30	0.27	0.15	0.42	1.57	0.30	2.62	0.221
0.0005	0.028	0.224	0.30	0.26	0.15	0.41	1.53	0.30	2.58	0.240
0.0006	0.027	0.241	0.30	0.25	0.15	0.40	1.49	0.30	2.54	0.257
0.0007	0.027	0.256	0.30	0.24	0.15	0.39	1.46	0.30	2.51	0.272
0.0008	0.027	0.270	0.30	0.23	0.15	0.38	1.44	0.30	2.49	0.286
0.0009	0.027	0.282	0.30	0.22	0.15	0.37	1.42	0.30	2.47	0.299
0.0010	0.027	0.294	0.30	0.22	0.15	0.37	1.40	0.30	2.45	0.311
0.0015	0.027	0.345	0.30	0.20	0.15	0.35	1.34	0.30	2.39	0.361
0.0020	0.026	0.386	0.30	0.18	0.15	0.33	1.29	0.30	2.34	0.402
0.0030	0.026	0.452	0.30	0.16	0.15	0.31	1.24	0.30	2.29	0.466
0.0040	0.026	0.505	0.30	0.15	0.15	0.30	1.20	0.30	2.25	0.518
0.0050	0.026	0.550	0.30	0.14	0.15	0.29	1.18	0.30	2.23	0.562
0.0060	0.026	0.590	0.30	0.13	0.15	0.28	1.15	0.30	2.20	0.600
0.0070	0.025	0.626	0.30	0.13	0.15	0.28	1.14	0.30	2.19	0.635
0.0080	0.025	0.659	0.30	0.12	0.15	0.27	1.12	0.30	2.17	0.667
0.0090	0.025	0.689	0.30	0.12	0.15	0.27	1.11	0.30	2.16	0.696

Table E7. Unlined water course design for Q = 100 l/s with vegetation.

Watercourse Design Data										
Discharge = 100 l/sec			Side Slope = 1.5		Retardance Class – E (shorter than 5 cm)					
Energy slope S	Manning n	Vel. V m/s	Bottom width b. m	Flow depth d. m	Free board FB m	Total depth d. m	Top Width TW m	Bank width BW m	Pad width PW m	Vel. n=0.025 V m/s
0.0001	0.050	0.099	0.40	0.70	0.23	0.93	3.20	0.70	5.30	0.166
0.0002	0.048	0.133	0.35	0.60	0.20	0.80	2.76	0.60	4.56	0.215
0.0003	0.046	0.158	0.35	0.54	0.18	0.72	2.52	0.54	4.15	0.250
0.0004	0.045	0.179	0.30	0.52	0.17	0.69	2.38	0.52	3.93	0.279
0.0005	0.045	0.197	0.30	0.49	0.16	0.65	2.26	0.49	3.74	0.304
0.0006	0.044	0.213	0.30	0.47	0.16	0.63	2.18	0.47	3.58	0.325
0.0007	0.044	0.227	0.30	0.45	0.15	0.60	2.10	0.45	3.46	0.344
0.0008	0.043	0.240	0.30	0.44	0.15	0.59	2.06	0.44	3.38	0.362
0.0009	0.043	0.253	0.30	0.42	0.15	0.57	2.02	0.42	3.32	0.378
0.0010	0.043	0.264	0.30	0.41	0.15	0.56	1.99	0.41	3.26	0.393
0.0015	0.041	0.314	0.30	0.37	0.15	0.52	1.87	0.37	3.06	0.457
0.0020	0.041	0.354	0.30	0.35	0.15	0.50	1.79	0.35	2.93	0.510
0.0030	0.040	0.420	0.30	0.31	0.15	0.46	1.68	0.31	2.75	0.592
0.0040	0.039	0.473	0.30	0.29	0.15	0.44	1.62	0.30	2.67	0.659
0.0050	0.039	0.519	0.30	0.27	0.15	0.42	1.57	0.30	2.62	0.716
0.0060	0.038	0.560	0.30	0.26	0.15	0.41	1.55	0.30	2.58	0.766
0.0070	0.038	0.597	0.30	0.25	0.15	0.40	1.50	0.30	2.55	0.811
0.0080	0.038	0.631	0.30	0.24	0.15	0.39	1.47	0.30	2.52	0.852
0.0090	0.037	0.662	0.30	0.23	0.15	0.38	1.45	0.30	2.50	0.889

Table E8. Unlined water course design for Q = 100 L/s with no vegetation.

Watercourse Design Data										
Discharge = 100 l/sec			Side Slope = 1.5		Retardance Class – No vegetation					
Energy slope S	Manning n	Vel. V m/s	Bottom width b. m	Flow depth d. m	Free board FB m	Total depth d. m	Top Width TW m	Bank width BW m	Pad width PW m	Vel. n=0.025 V m/s
0.0001	0.026	0.160	0.35	0.54	0.18	0.72	2.50	0.54	4.12	0.166
0.0002	0.026	0.211	0.30	0.47	0.16	0.63	2.18	0.47	3.59	0.215
0.0003	0.025	0.248	0.30	0.43	0.15	0.58	2.03	0.43	3.34	0.251
0.0004	0.025	0.278	0.30	0.40	0.15	0.55	1.95	0.40	3.20	0.279
0.0005	0.025	0.304	0.30	0.38	0.15	0.53	1.89	0.38	3.10	0.303
0.0006	0.025	0.325	0.30	0.36	0.15	0.51	1.84	0.36	3.02	0.325
0.0007	0.025	0.344	0.30	0.35	0.15	0.50	1.80	0.35	2.96	0.344
0.0008	0.025	0.362	0.30	0.34	0.15	0.49	1.77	0.34	2.90	0.362
0.0009	0.025	0.378	0.30	0.33	0.15	0.48	1.74	0.33	2.86	0.378
0.0010	0.025	0.393	0.30	0.32	0.15	0.47	1.72	0.32	2.82	0.393
0.0015	0.025	0.458	0.30	0.29	0.15	0.44	1.63	0.30	2.68	0.457
0.0020	0.025	0.509	0.30	0.28	0.15	0.43	1.58	0.30	2.63	0.509
0.0030	0.025	0.592	0.30	0.25	0.15	0.40	1.50	0.30	2.55	0.592
0.0040	0.025	0.659	0.30	0.23	0.15	0.38	1.45	0.30	2.50	0.659
0.0050	0.025	0.716	0.30	0.22	0.15	0.37	1.41	0.30	2.46	0.716
0.0060	0.025	0.766	0.30	0.21	0.15	0.36	1.38	0.30	2.43	0.766
0.0070	0.025	0.811	0.30	0.20	0.15	0.35	1.36	0.30	2.41	0.811
0.0080	0.025	0.852	0.30	0.20	0.15	0.35	1.34	0.30	2.39	0.852
0.0090	0.025	0.890	0.30	0.19	0.15	0.34	1.32	0.30	2.37	0.890

Table E9. Unlined water course design for Q = 140 L/s with vegetation.

Watercourse Design Data										
Discharge = 140 l/sec			Side Slope = 1.5			Retardance Class – E (shorter than 5 cm)				
Energy slope S	Manning n	Vel. V m/s	Bottom width b. m	Flow depth d. m	Free board FB m	Total depth d. m	Top Width TW m	Bank width BW m	Pad width PW m	Vel. n=0.025 V m/s
0.0001	0.046	0.114	0.45	0.77	0.26	1.02	3.52	0.77	5.82	0.181
0.0002	0.044	0.153	0.40	0.66	0.22	0.88	3.03	0.66	5.01	0.234
0.0003	0.043	0.182	0.35	0.61	0.20	0.81	2.78	0.61	4.61	0.273
0.0004	0.042	0.206	0.35	0.57	0.19	0.76	2.62	0.57	4.32	0.304
0.0005	0.041	0.226	0.35	0.54	0.18	0.71	2.49	0.54	4.10	0.330
0.0006	0.041	0.244	0.30	0.53	0.18	0.70	2.40	0.53	3.98	0.354
0.0007	0.041	0.261	0.30	0.51	0.17	0.68	2.33	0.51	3.85	0.375
0.0008	0.040	0.276	0.30	0.49	0.16	0.65	2.26	0.49	3.73	0.394
0.0009	0.040	0.290	0.30	0.48	0.16	0.63	2.20	0.48	3.63	0.411
0.0010	0.040	0.303	0.30	0.46	0.15	0.62	2.15	0.46	3.55	0.428
0.0015	0.039	0.360	0.30	0.42	0.15	0.57	2.01	0.42	3.30	0.498
0.0020	0.038	0.406	0.30	0.39	0.15	0.54	1.92	0.39	3.15	0.555
0.0030	0.037	0.481	0.30	0.35	0.15	0.50	1.81	0.35	2.96	0.646
0.0040	0.036	0.542	0.30	0.33	0.15	0.48	1.73	0.33	2.83	0.719
0.0050	0.036	0.594	0.30	0.31	0.15	0.46	1.68	0.31	2.74	0.781
0.0060	0.036	0.641	0.30	0.29	0.15	0.44	1.63	0.30	2.68	0.836
0.0070	0.035	0.683	0.30	0.28	0.15	0.43	1.60	0.30	2.65	0.885
0.0080	0.035	0.722	0.30	0.27	0.15	0.42	1.57	0.30	2.62	0.930
0.0090	0.035	0.757	0.30	0.27	0.15	0.42	1.55	0.30	2.60	0.971

Table E10. Unlined water course design for Q = 140 L/s with no vegetation.

Watercourse Design Data										
Discharge = 140 l/sec			Side Slope = 1.5			Retardance Class – No vegetation				
Energy slope S	Manning n	Vel. V m/s	Bottom width b. m	Flow depth d. m	Free board FB m	Total depth d. m	Top Width TW m	Bank width BW m	Pad width PW m	Vel. n=0.025 V m/s
0.0001	0.025	0.179	0.35	0.62	0.21	0.82	2.81	0.62	4.66	0.181
0.0002	0.025	0.234	0.30	0.54	0.18	0.72	2.46	0.54	4.07	0.234
0.0003	0.025	0.273	0.30	0.49	0.16	0.66	2.27	0.49	3.75	0.273
0.0004	0.025	0.304	0.30	0.46	0.15	0.62	2.15	0.46	3.54	0.304
0.0005	0.025	0.330	0.30	0.44	0.15	0.59	2.07	0.44	3.40	0.330
0.0006	0.025	0.354	0.30	0.42	0.15	0.57	2.02	0.42	3.32	0.354
0.0007	0.025	0.375	0.30	0.41	0.15	0.56	1.98	0.41	3.25	0.375
0.0008	0.025	0.394	0.30	0.40	0.15	0.55	1.94	0.40	3.18	0.394
0.0009	0.025	0.412	0.30	0.39	0.15	0.54	1.91	0.39	3.13	0.411
0.0010	0.025	0.428	0.30	0.38	0.15	0.53	1.88	0.38	3.09	0.428
0.0015	0.025	0.498	0.30	0.34	0.15	0.49	1.78	0.34	2.92	0.498
0.0020	0.025	0.555	0.30	0.32	0.15	0.47	1.72	0.32	2.81	0.555
0.0030	0.025	0.645	0.30	0.29	0.15	0.44	1.63	0.30	2.68	0.645
0.0040	0.025	0.719	0.30	0.27	0.15	0.42	1.57	0.30	2.62	0.718
0.0050	0.025	0.781	0.30	0.26	0.15	0.41	1.53	0.30	2.58	0.780
0.0060	0.025	0.836	0.30	0.25	0.15	0.40	1.50	0.30	2.55	0.835
0.0070	0.025	0.885	0.30	0.24	0.15	0.39	1.47	0.30	2.52	0.884
0.0080	0.025	0.930	0.30	0.23	0.15	0.38	1.45	0.30	2.50	0.929
0.0090	0.025	0.971	0.30	0.23	0.15	0.38	1.43	0.30	2.48	0.971

Table E11. Unlined water course design for Q = 200 L/s with vegetation.

Watercourse Design Data										
Discharge = 200 l/sec			Side Slope = 1.5			Retardance Class – E (shorter than 5 cm)				
Energy slope S	Manning n	Vel. V m/s	Bottom width b. m	Flow depth d. m	Free board FB m	Total depth d. m	Top Width TW m	Bank width BW m	Pad width PW m	Vel. n=0.025 V m/s
0.0001	0.043	0.132	0.50	0.85	0.28	1.13	3.90	0.85	6.45	0.197
0.0002	0.041	0.178	0.45	0.73	0.24	0.97	3.37	0.73	5.55	0.256
0.0003	0.040	0.211	0.40	0.67	0.22	0.90	3.09	0.67	5.11	0.298
0.0004	0.039	0.238	0.40	0.63	0.21	0.84	2.91	0.63	4.79	0.332
0.0005	0.038	0.262	0.35	0.61	0.20	0.81	2.78	0.61	4.60	0.361
0.0006	0.038	0.282	0.35	0.58	0.19	0.77	2.67	0.58	4.41	0.387
0.0007	0.038	0.301	0.35	0.56	0.19	0.74	2.58	0.56	4.26	0.409
0.0008	0.037	0.319	0.35	0.54	0.18	0.72	2.51	0.54	4.13	0.430
0.0009	0.037	0.335	0.30	0.54	0.18	0.72	2.46	0.54	4.07	0.450
0.0010	0.037	0.350	0.30	0.53	0.18	0.70	2.40	0.53	3.98	0.468
0.0015	0.036	0.415	0.30	0.48	0.16	0.63	2.20	0.48	3.63	0.545
0.0020	0.035	0.467	0.30	0.44	0.15	0.59	2.08	0.44	3.42	0.607
0.0030	0.035	0.553	0.30	0.40	0.15	0.55	1.95	0.40	3.21	0.706
0.0040	0.034	0.623	0.30	0.37	0.15	0.52	1.87	0.37	3.07	0.786
0.0050	0.034	0.683	0.30	0.35	0.15	0.50	1.81	0.35	2.97	0.855
0.0060	0.033	0.736	0.30	0.34	0.15	0.49	1.76	0.34	2.89	0.915
0.0070	0.033	0.784	0.30	0.32	0.15	0.47	1.72	0.32	2.82	0.969
0.0080	0.033	0.828	0.30	0.31	0.15	0.46	1.69	0.31	2.77	1.019
0.0090	0.033	0.869	0.30	0.30	0.15	0.45	1.66	0.30	2.72	1.064

Table E12. Unlined water course design for Q = 200 L/s with no vegetation.

Watercourse Design Data										
Discharge = 200 l/sec			Side Slope = 1.5			Retardance Class – No vegetation				
Energy slope S	Manning n	Vel. V m/s	Bottom width b. m	Flow depth d. m	Free board FB m	Total depth d. m	Top Width TW m	Bank width BW m	Pad width PW m	Vel. n=0.025 V m/s
0.0001	0.025	0.197	0.40	0.70	0.23	0.93	3.20	0.70	5.29	0.197
0.0002	0.025	0.256	0.35	0.61	0.20	0.82	2.81	0.61	4.65	0.256
0.0003	0.025	0.298	0.35	0.56	0.19	0.75	2.60	0.56	4.28	0.298
0.0004	0.025	0.332	0.30	0.54	0.18	0.72	2.47	0.54	4.09	0.332
0.0005	0.025	0.361	0.30	0.52	0.17	0.69	2.36	0.52	3.91	0.361
0.0006	0.025	0.387	0.30	0.50	0.17	0.66	2.28	0.50	3.77	0.387
0.0007	0.025	0.410	0.30	0.48	0.16	0.64	2.22	0.48	3.65	0.410
0.0008	0.025	0.431	0.30	0.47	0.16	0.62	2.16	0.47	3.56	0.431
0.0009	0.025	0.450	0.30	0.45	0.15	0.60	2.11	0.45	3.47	0.450
0.0010	0.025	0.468	0.30	0.44	0.15	0.59	2.08	0.44	3.41	0.468
0.0015	0.025	0.545	0.30	0.40	0.15	0.55	1.96	0.40	3.22	0.545
0.0020	0.025	0.607	0.30	0.38	0.15	0.53	1.89	0.38	3.10	0.607
0.0030	0.025	0.707	0.30	0.35	0.15	0.50	1.79	0.35	2.93	0.706
0.0040	0.025	0.787	0.30	0.32	0.15	0.47	1.72	0.32	2.8[illegible]	0.786
0.0050	0.025	0.855	0.30	0.31	0.15	0.46	1.67	0.31	2.74	0.855
0.0060	0.025	0.915	0.30	0.29	0.15	0.44	1.63	0.30	2.68	0.915
0.0070	0.025	0.969	0.30	0.28	0.15	0.43	1.60	0.30	2.65	0.969
0.0080	0.025	1.019	0.30	0.28	0.15	0.43	1.58	0.30	2.63	1.018
0.0090	0.025	1.064	0.30	0.27	0.15	0.42	1.55	0.30	2.60	1.064

Table E13. Unlined water course design for Q = 260 L/s with vegetation.

Watercourse Design Data										
Discharge = 260 l/sec			Side Slope = 1.5			Retardance Class – E (shorter than 5 cm)				
Energy slope S	Manning n	Vel. V m/s	Bottom width b. m	Flow depth d. m	Free board FB m	Total depth d. m	Top Width TW m	Bank width BW m	Pad width PW m	Vel. n=0.025 V m/s
0.0001	0.040	0.148	0.55	0.92	0.31	1.22	4.21	0.92	6.96	0.211
0.0002	0.039	0.198	0.50	0.78	0.26	1.05	3.64	0.78	5.99	0.273
0.0003	0.038	0.234	0.45	0.72	0.24	0.96	3.34	0.72	5.51	0.318
0.0004	0.037	0.264	0.40	0.69	0.23	0.92	3.15	0.69	5.21	0.355
0.0005	0.036	0.290	0.40	0.65	0.22	0.87	3.00	0.65	4.96	0.386
0.0006	0.036	0.313	0.40	0.62	0.21	0.83	2.89	0.62	4.76	0.413
0.0007	0.036	0.334	0.35	0.61	0.20	0.82	2.80	0.61	4.64	0.437
0.0008	0.036	0.353	0.35	0.59	0.20	0.79	2.72	0.59	4.51	0.460
0.0009	0.035	0.371	0.35	0.58	0.19	0.77	2.66	0.58	4.39	0.481
0.0010	0.035	0.388	0.35	0.56	0.19	0.75	2.60	0.56	4.29	0.500
0.0015	0.034	0.459	0.30	0.52	0.17	0.70	2.39	0.52	3.96	0.582
0.0020	0.034	0.517	0.30	0.49	0.16	0.65	2.25	0.49	3.71	0.648
0.0030	0.033	0.611	0.30	0.44	0.15	0.59	2.08	0.44	3.4.	0.754
0.0040	0.033	0.688	0.30	0.41	0.15	0.56	1.99	0.41	3.26	0.840
0.0050	0.032	0.754	0.30	0.39	0.15	0.54	1.92	0.39	3.15	0.913
0.0060	0.032	0.813	0.30	0.37	0.15	0.52	1.87	0.37	3.06	0.978
0.0070	0.032	0.865	0.30	0.36	0.15	0.51	1.83	0.36	2.99	1.036
0.0080	0.032	0.914	0.30	0.35	0.15	0.50	1.79	0.35	2.93	1.089
0.0090	0.031	0.959	0.30	0.34	0.15	0.49	1.76	0.34	2.88	1.138

Table E14. Unlined water course design for Q = 260 L/s with no vegetation.

Watercourse Design Data										
Discharge = 260 l/sec			Side Slope = 1.5			Retardance Class – No vegetation				
Energy slope S	Manning n	Vel. V m/s	Bottom width b. m	Flow depth d. m	Free board FB m	Total depth d. m	Top Width TW m	Bank width BW m	Pad width PW m	Vel. n=0.025 V m/s
0.0001	0.025	0.211	0.45	0.77	0.26	1.03	3.53	0.77	5.83	0.211
0.0002	0.025	0.273	0.40	0.67	0.22	0.90	3.10	0.67	5.12	0.273
0.0003	0.025	0.318	0.40	0.62	0.21	0.82	2.87	0.62	4.72	0.318
0.0004	0.025	0.354	0.35	0.59	0.20	0.79	2.72	0.59	4.50	0.355
0.0005	0.025	0.386	0.35	0.56	0.19	0.75	2.61	0.56	4.30	0.385
0.0006	0.025	0.413	0.35	0.54	0.18	0.72	2.52	0.54	4.14	0.413
0.0007	0.025	0.437	0.30	0.54	0.18	0.72	2.45	0.54	4.06	0.437
0.0008	0.025	0.460	0.30	0.52	0.17	0.70	2.39	0.52	3.95	0.460
0.0009	0.025	0.480	0.30	0.51	0.17	0.68	2.34	0.51	3.86	0.481
0.0010	0.025	0.500	0.30	0.50	0.17	0.66	2.29	0.50	3.78	0.500
0.0015	0.025	0.582	0.30	0.45	0.15	0.61	2.12	0.45	3.48	0.582
0.0020	0.025	0.649	0.30	0.43	0.15	0.58	2.03	0.43	3.33	0.648
0.0050	0.025	0.755	0.30	0.39	0.15	0.54	1.92	0.39	3.15	0.755
0.0040	0.025	0.841	0.30	0.36	0.15	0.51	1.84	0.36	3.02	0.840
0.0050	0.025	0.914	0.30	0.35	0.15	0.50	1.79	0.35	2.93	0.913
0.0060	0.025	0.978	0.30	0.33	0.15	0.48	1.75	0.33	2.86	0.978
0.0070	0.025	1.036	0.30	0.32	0.15	0.47	1.71	0.32	2.81	1.036
0.0080	0.025	1.089	0.30	0.31	0.15	0.46	1.68	0.31	2.76	1.089
0.0090	0.025	1.138	0.30	0.30	0.15	0.45	1.66	0.30	2.71	1.138

Table E15. Unlined water course design for Q = 290 L/s with vegetation.

Watercourse Design Data										
Discharge = 290 l/sec			Side Slope = 1.5			Retardance Class -- E (shorter than 5 cm)				
Energy slope S	Manning n	Vel. V m/s	Bottom width b. m	Flow depth d. m	Free board FB m	Total depth d. m	Top Width TW m	Bank width BW m	Pad width PW m	Vel. n=0.025 V m/s
0.0001	0.039	0.154	0.55	0.95	0.32	1.27	4.35	0.95	7.21	0.217
0.0002	0.038	0.207	0.50	0.82	0.27	1.09	3.76	0.82	6.21	0.281
0.0003	0.037	0.245	0.45	0.75	0.25	1.00	3.46	0.75	5.71	0.327
0.0004	0.036	0.276	0.45	0.70	0.23	0.93	3.25	0.70	5.35	0.364
0.0005	0.036	0.303	0.40	0.68	0.23	0.90	3.11	0.68	5.14	0.396
0.0006	0.035	0.327	0.40	0.65	0.22	0.86	2.99	0.65	4.93	0.424
0.0007	0.035	0.349	0.40	0.62	0.21	0.83	2.89	0.62	4.76	0.449
0.0008	0.035	0.368	0.35	0.62	0.21	0.82	2.82	0.62	4.67	0.473
0.0009	0.035	0.387	0.35	0.60	0.20	0.80	2.75	0.60	4.55	0.494
0.0010	0.034	0.404	0.35	0.58	0.19	0.78	2.69	0.58	4.44	0.514
0.0015	0.034	0.478	0.35	0.53	0.18	0.71	2.47	0.53	4.06	0.598
0.0020	0.033	0.538	0.30	0.51	0.17	0.68	2.33	0.51	3.85	0.666
0.0030	0.033	0.637	0.30	0.46	0.15	0.61	2.14	0.46	3.52	0.775
0.0040	0.032	0.717	0.30	0.43	0.15	0.58	2.04	0.43	3.34	0.864
0.0050	0.032	0.785	0.30	0.41	0.15	0.56	1.97	0.41	3.23	0.939
0.0060	0.031	0.846	0.30	0.39	0.15	0.54	1.92	0.39	3.14	1.006
0.0070	0.031	0.901	0.30	0.37	0.15	0.52	1.87	0.37	3.07	1.065
0.0080	0.031	0.952	0.30	0.36	0.15	0.51	1.84	0.36	3.01	1.120
0.0090	0.031	0.998	0.30	0.35	0.15	0.50	1.80	0.35	2.96	1.170

Table E16. Unlined water course design for Q = 290 L/s with no vegetation.

Watercourse Design Data										
Discharge = 290 l/sec			Side Slope = 1.5			Retardance Class -- No vegetation				
Energy slope S	Manning n	Vel. V m/s	Bottom width b. m	Flow depth d. m	Free board FB m	Total depth d. m	Top Width TW m	Bank width BW m	Pad width PW m	Vel. n=0.025 V m/s
0.0001	0.025	0.217	0.50	0.79	0.26	1.06	3.67	0.79	6.05	0.217
0.0002	0.025	0.281	0.40	0.71	0.24	0.94	3.23	0.71	5.35	0.281
0.0003	0.025	0.327	0.40	0.65	0.22	0.86	2.99	0.65	4.93	0.327
0.0004	0.025	0.364	0.35	0.62	0.21	0.83	2.83	0.62	4.70	0.364
0.0005	0.025	0.396	0.35	0.59	0.20	0.79	2.72	0.59	4.49	0.396
0.0006	0.025	0.424	0.35	0.57	0.19	0.76	2.62	0.57	4.33	0.424
0.0007	0.025	0.450	0.35	0.55	0.18	0.73	2.55	0.55	4.20	0.449
0.0008	0.025	0.473	0.35	0.53	0.18	0.71	2.48	0.53	4.08	0.472
0.0009	0.025	0.494	0.30	0.53	0.18	0.71	2.43	0.53	4.04	0.494
0.0010	0.025	0.514	0.30	0.52	0.17	0.70	2.39	0.52	3.95	0.514
0.0015	0.025	0.598	0.30	0.48	0.16	0.64	2.21	0.48	3.64	0.598
0.0020	0.025	0.666	0.30	0.45	0.15	0.60	2.09	0.45	3.44	0.666
0.0030	0.025	0.776	0.30	0.41	0.15	0.56	1.98	0.41	3.25	0.776
0.0040	0.025	0.864	0.30	0.38	0.15	0.53	1.90	0.38	3.12	0.864
0.0050	0.025	0.939	0.30	0.36	0.15	0.51	1.84	0.36	3.02	0.939
0.0060	0.025	1.006	0.30	0.35	0.15	0.50	1.80	0.35	2.95	1.005
0.0070	0.025	1.065	0.30	0.34	0.15	0.49	1.76	0.34	2.89	1.065
0.0080	0.025	1.120	0.30	0.33	0.15	0.48	1.73	0.33	2.84	1.119
0.0090	0.025	1.170	0.30	0.32	0.15	0.47	1.71	0.32	2.79	1.169

ANNEXURE F: EXAMPLE OF DESIGN OF SPRINKLER AND TRICKLE IRRIGATION SYSTEM AT KRAPPA, KARAK -NWFP

Gun Sprinkler System

I. Soil:-

i) Texture: Sandy loam

ii) Available water holding capacity (Wa) = 120 mm/mi)

iii) Steady State infiltration rate = 12-25 mm/hr

II. Crop:-

i) Type : Peanut

ii) Effective root depth (Z) = 0.75 m

iii) Consumptive use

(a) Peak = 7.6 mm/day.

(b) Seasonal water requirements = 610 mm.

iv) Management Allowed Deficit (MAD) = 40%

III. Climate:- Hot

IV. Water Supply :- Pond.

V. Sprinkler system components:-

Main pipes (PVC high pressure pipes), GI pipe risers, easy couplers, flexible pipe, gun sprinklers, pump with diesel engine.

VI. Design Calculation and lay out:-

1) Net depth to be applied per irrigation (dx)

dx = MAD/100 Wa Z

= 40/100 x 120mm/m x 0.75 m = 36 mm.

Gross depth of application = 36/Eff = 42 mm

Assume efficiency = 85%

2) Irrigation Interval (f)

f = 42/7.6 = 6 days.

3) Actual operating time (f - time off) = 6 days per week.

4) Operating hours = 16 hrs

5) ET = 7.6 mm/day

6) Sprinkler Brand Name : Bauer , No : R 25/SR 25 (Cone Nozzle), Made in Austria

7) Operating Pressure = 3 Bars = 31 m

8) Nozzle size = 20 mm

9) Capacity = 26.6 m3/h

10) Distance of throw = 39.5 m

11) Application rate = 5.4 mm/h

12) Nos of sprinkler set = 1

13) Maximum command area at one setting = 1 ha

14) Farm size = 5 ha

15) Maximum pipe length = 450 m

16) Head loss in pipe = 2.4 m/ 100 m for 76 mm (3 inches) pipe

17) For 450 m pipe head loss = 10.8 m

18) Elevation Difference = 15 m

19) Total required head = 10.8+31+15 = 56.8 m

Power Requirement:

Total Gross head = 56.8 m (217 feet)

then

WHP = rQh/550 = 62.4 x 0.265 x 217/550 = 6.5 hp.

let engine and pump efficiency = 50%

BHP = 6.5/.0.5 = 13 ~ .let it to be 16 hp

Trickle Irrigation System

I. **Crop:** Citrus

II. **Climate**:- Hot

III. **Water Supply** :- Storage tank.

IV. **Trickle system components:-**

Main and lateral pipes (LDPE), valves, emitters, filter

V. **Design lay out:-**

1. Area = 1 acre (0.4 ha)
2. Length main pipe = 30 m
3. Length single lateral = 150 m
4. No of laterals = 8
5. No. of trees per lateral = 41
6. Total No. of trees = 328

INSTALLATION COST OF A SMALL SCALE SPRINKLER AND TRICKLE IRRIGATION SYSTEM AT KRAPPA, KARAK -NWFP

S.No	Description of items	Unit Price (Rs)	Total Price (Rs)
a.	**SPRINKLER IRRIGATION SYSTEM**		
1	Engine 16 hp + pump	40,000/-	40,000
2	Gun sprinklers	5,000	10,000
3	Main Pipe 500 m	80 per m	40,000
4	Flexible pipe 40 m	200 per m	8,000
5	Easy couplers and GI pipe risers 12 Nos	1000/-	12,000
6	POL cost		15,000
7	TA		20,000
	Total Cost of the Sprinkler project		145,000
	Farmers contribution 10% of the material cost + labors		11,000
	Net Cost of Sprinkler project (less farmers contribution)		**134,000**
b.	**TRICKLE IRRIGATION SYSTEM**		
9	Main pipe (LDPE) 25 mm	40 per m	1200
10	Lateral pipe (LDPE) 12.5 mm	25 per m	30,000
11	Emitters	30 per emitter	10,500
12	Valve 25 mm	150 per valve	150
13	Valves 12.5 mm	100	800
14	Filter	2000	2000
	Total Cost of the Trickle project		44,650
	Farmers contribution 10% of the material cost + labors		4,500
	Net Cost of the Trickle project (less farmers contribution)		**40,150**
	Total Cost for Sprinkler and Trickle systems (to be provided by the PCRWR)		**1,74,150**
	Total Farmer's contribution		**15,500**
	TOTAL COST		**1,89,650**

ANNEXURE G: PHOTOGRAPHS OF FIELD EXCURSIONS

Pipe flow measurement techniques at AUP Farm

Discharge measurement with current meter at AUP Farm

Canal design exercise in the class room

Measurement of drainage collector discharge at Lower Swat Canal Mardan

Use of a canal fall as a trapezoidal weir for flow measurement at AUP Farm

Discharge measurement with cut throat flume.

Visit to a weir at Malakand

Trainees comparing the designs prepared by them with the reality in the field in Swat Valley

Visit to a Pre-cast Parabolic Canal segment factory in Swabi.

Measurement of seepage losses in canal by ponding test.

Survey exercise

Visit to a siphon along the Warsak Right Bank canal