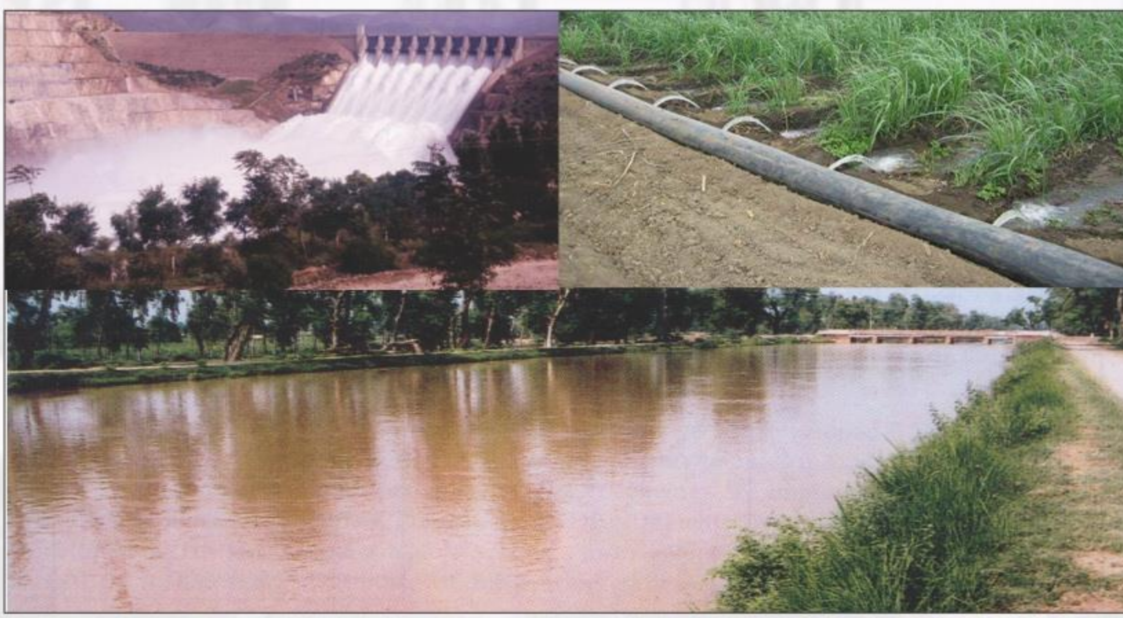
Managing Editors
Iqrar Ahmad Khan & Muhammad Farooq

Irrigation & Drainage Practices for Agriculture

Muhammad Rafiq Choudhry



University of Agriculture, Faisalabad, Pakistan

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Foreword

The digital age has its preferences. The reading time has been encroached upon by a watching time. The access to information is easy and a plenty where Wikipedia has emerged as the most powerful encyclopedia ever. Yet, a book is a book! We wish to promote the habit of reading books. Finding books is not difficult or expensive (www.pdfdrive.com) but a local context and indigenous experiences could be missing.

The University of Agriculture, Faisalabad (UAF) has achieved global rankings of its flagship programs and acceptance as a leader in the field of agriculture and allied sciences. A competent faculty, the stimulating ecosystem and its learning environment have attracted increasing attention. Publication of books is an important KPI for any institution of higher learning. Hence, UAF has embarked upon an ambitious 'books project' to provide reference texts and to occupy our space as a knowledge powerhouse. It is intended that the UAF books shall be made available in both paper and electronic versions for a wider reach and affordability.

UAF offers more than 160 degree programs where agriculture remains our priority. There are about 20 institutions other than UAF in Pakistan offering similar degree programs. Yet, there is no strong history of indigenously produced text/reference books that students and scholars could access. The last major effort dates back to the early 1990's when a USAID funded TIPAN project produced a few multiauthor text books. Those books are now obsoleted but still in demand because of lack of alternatives. The knowledge explosion simply demands that we undertake and expand the process anew.

Considering the significance of this project, I have personally overseen the entire process of short listing of the topics, assemblage of authors, review of contents and editorial work of 29 books being written in the first phase of this project. Each book has editor(s) who worked with a group of authors writing chapters of their choice and expertise. The draft texts were peer reviewed and language corrected as much as possible. There was a considerable consultation and revision undertaken before the final drafts were accepted for formatting and printing process.

This series of books cover a very broad range of subjects from theoretical physics and electronic image processing to hard core agricultural subjects and public policy. It is my considered opinion that the books produced here will find a wide acceptance across the country and overseas. That will serve a very important purpose of improving quality of teaching and learning. The reference texts will also be equally valued by the researchers and enthusiastic practitioners. Hopefully, this is a beginning of unleashing the knowledge potential of UAF which shall be continued. It is my dream to open a bookshop at UAF like the ones that we find in highly ranked universities across the globe.

This book is a comprehensive presentation on various aspects of irrigation and drainage including water resources, system management, flow measurement, drainage of agricultural lands, water management techniques applied to irrigation and water distribution system. It will be a good source for the students, teaching faculty, professionals, policy makers, planners and the farmers engaged in the fields of water recourses, irrigation, drainage, water management and delivery.

Before concluding, I wish to record my appreciation for my coworker Dr. Muhammad Farooq who worked skillfully and tirelessly towards achieving a daunting task. Equally important was the contribution of the authors and editors of this book. I also acknowledge the financial support for this project provided by the USDA endowment fund available to UAF.

Iqrar Ahmad Khan

Preface

This book on “Irrigation and Drainage Practices for Agriculture” has been specially designed and written to cater the academic and professional needs of the students of agriculture and engineering degree programs at various institutions. It would also provide a knowledge base for the engineers, agriculturists, researchers and experts dealing with water sector. Water being the most important single factor among a variety of crop inputs, a sound knowledge of its availability, management and use is indispensable for successful irrigated agriculture. As a professional in agriculture and irrigation engineering, one should understand the overall country’s irrigation system, sources of water and their availability against the requirements, methods to measure and apply irrigation water, drainage of excess water and leveling techniques for efficient application, in addition to learning the agronomic and other plant growth practices,

Groundwater contributes, on the average, more than 40% of the crop water needs in Pakistan. Therefore, pumping of groundwater and its contribution to agriculture has become an integral part of agricultural production enterprise. Consequently, understanding of pumping systems is essential for achieving potential production in agriculture through augmented groundwater supplies.

Water availability in Pakistan is quite constrained to meet the crop water needs, therefore, its efficient use must be guaranteed through accurate assessment of required quantities of irrigation water for the crop, application at appropriate time using efficient irrigation methods. Therefore, chapters on water measurement, irrigation scheduling and methods of irrigation have also been included in the book.

Efficient management of water at the farm cannot be accomplished without precisely leveling the fields and improving water conveyance and application systems. Laser leveling of fields ensures uniform application and availability of water to the crop, uniform crop stand and enhanced crop yield. In fact, no good water management can be achieved without precision land leveling. Therefore, surveying and land leveling techniques are also considered integral part of the agricultural production technologies.

If irrigation return flow and excess water from seepage is permitted to percolate to groundwater, it may result in rising watertable and consequent waterlogging, leading to the reduction of the crop production potential of fertile land. Drainage of agricultural land provides desirable soil-water plant environment to support desirable plant growth. In fact, there is no successful irrigation without drainage. Therefore, a chapter on drainage systems has been included to familiarize the students with the types of materials and methods of surface and subsurface land drainage.

Although rainfall provides the basic crop water needs in rainfed agriculture and exerts boosting impact on crop yields in irrigated agriculture, yet its role in developing water resources of the country cannot be overlooked. Conservation of this precious resource can make a remarkable difference in the production potential and economy of the country. Thus, knowledge of soil and water conservation techniques will certainly be indispensable to understand and resolve the water management issues of both irrigated and arid agriculture.

Whereas the book is focused to strengthen the knowledge base for the professionals in the fields of agriculture, agricultural engineering, irrigation and drainage, other professionals such as water management, renewable energy, pumps and well technology may also be benefited. Watercourse rehabilitation and lining are important components of Water Management. As the book is considered to meet the needs of water management specialists in carrying out the field activities of water management, it has also been equipped with a chapter on watercourse design, construction and maintenance. Thus, the book contains sufficient information to fulfill the practical requirements of students and specialists engaged in agriculture, water management, irrigation and other engineering disciplines dealing with irrigation system management.

The book is well supported with solved examples in pumping systems, irrigation scheduling, surveying and land leveling and watercourse design. However, in some areas such as groundwater flow modeling and pumping system design, it may involve complex mathematical considerations that may not be understood by the professionals in agriculture. Therefore, such complexities of engineering are out of the scope of this book. Only applied knowledge of irrigation and drainage, required to support irrigated agricultural practices, have been included in this book. The students desiring additional information and analytical considerations, are referred to consult other books on the specialized subjects.

The first edition of the book comprising 90 pages, was published in 1990, which was primarily, a collection of lecture notes on various topics of irrigation and drainage practices. The book has been revised and updated with major improvements in 1995, 1996, 1998, 2001, 2005, 2006 and 2008, which comprised review and improvement of text, addition of scientific information and induction of more examples, photographs and references. The current addition of 2017 has been revised to include updated information on irrigation management, international and national level trans-boundary issues as well as water treaties and accords relevant to the water resources of the country.

M. Rafiq Choudhry

Chapter 1

Water Resources

1.1. Indus Basin Water Resources

1.1.1. Indus Basin River System

The total watershed area of Indus Basin is 944,000 km^2. Out of this, 16% area lies in Pakistan and remaining in India, Afghanistan and Tibet. The total command area of Indus Basin is 114 mha, out of which about 52.48% is in Pakistan, 34.35% in India, 6.83% in China and 6.3% in Afghanistan (Malik et al. 2010). Indus river system primarily comprises of Indus River and its 27 tributaries. The major tributaries include Jhelum, Chenab, Ravi, Sutlej and Beas that formed the bases of river system contributing to Indus Basin Irrigation System and Indus Water Treaty (1960). The minor tributaries include river Gilgit, Swat, Kunar, Kabul, Bara, Soan, Kurram, Tochi, Tank, and Gomal Zhob.

Major component of inflow received by the Indus river system comes from glacier melts and snowfall. The five eastern tributaries namely, Jhelum, Chenab, Ravi, Sutlej and Beas are fed by monsoon rains in addition to the glacier and snowmelt. The discharge of the river system reaches maximum in April to June due to melting of snow and glacier, while the monsoon rains concentrate during July to September that cause occasional floods in the plains. The river system of Indus Basin in Pakistan is shown in Fig. 1.1.

Glaciers in Pakistan constitute a part of large collection of glaciated ice found in the world outside the north and south polar regions. In Pakistan, these glaciers are mainly located in three mountaneous ranges, namely Karakoram, Himalya and Hindukush. There are about 15 major glaciers in Pakistan each covering an area of 53 to 685 sq. kilometers. Some of the important ones include Siachen, Batoro, Bifo, Hisper, Panmah, Chongo Lungmah and Batura. These glaciers are sources of fresh water to 60 large and small rivers in Pakistan. The climate changes taking place in these glaciers can greatly affect the melting of water, thereby affecting water supplies for irrigated agriculture, drinking water, hydro-electric power and ecological habitats of the country.

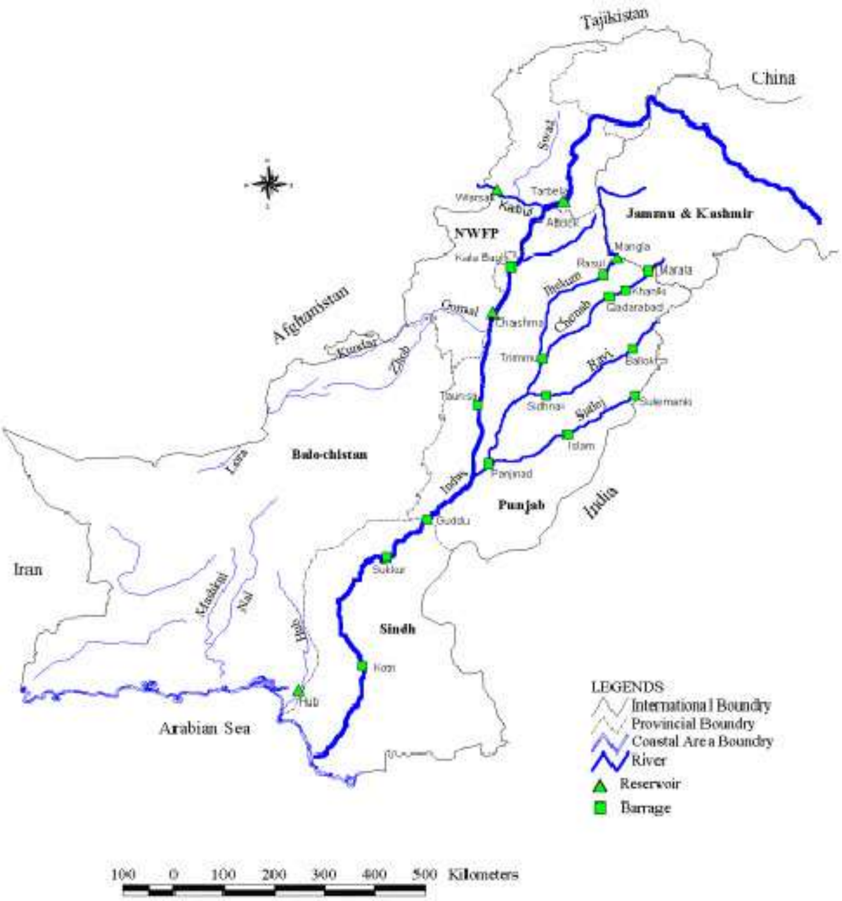

Fig.1.1 Indus Basin River System of Pakistan

1.1.2. Trans-boundary Water Issues

Water is the most fundamental source of life for the mankind. With the rapid growth in human population, shortage of water has become the inevitable consequence, especially in the third world countries. This in turn, made the water as a source of conflict between the sharing stake holders that may be the neighboring countries, states, provinces or other regional groups. At global level, rivers crossing the geographical boundaries between the neighboring countries have been a source of conflict, known as trans-boundary problems between neighboring countries originating from sharing of river water. Thus, a trans-boundary river may be defined as a river that crosses at least one international political border, or provincial / regional border within a nation. The hydrologic and political effects of rivers that cross significant boundaries may be positive in that they carry a significant amount of sediments, which may help in building land but may raise the height of riverbeds and thereby causing flooding. On the other hand, in many cases, international conventions governing water sharing have led to complex political disputes as discussed ahead.

1.1.2.1. Trans-boundary Water Issues at Global Level

Issues related to the trans-boundary river water have been existing between many countries of the world, which have been resolved through treaties and agreements from time to time. Table 1.1 shows some of the trans-boundary rivers of the world and the sharing countries. Some of the examples have been discussed here. India and Bangladesh share 54 common rivers, of which agreement has been reached only on sharing of water of Ganges river. The India-Bangladesh treaty on the sharing of the Ganges water was signed on December 12, 1996 and was based on a formula of the flows measured at Farakka, during the lean season each year, from 1 January to 31 May. The 30-year treaty is renewable by mutual consent.

The Nile river draws its water from 3 tributary rivers namely, White Nile, Blue Nile and Atbara, which flow from North Western Ethiopia to the Nile in East Sudan. In the past, it has influenced the Interstate politics among the sharing countries as it is the only reliable source of water in the region. Political stability and water security has forced the sharing countries to enter into mutually accepted agreements. A few such agreements during the colonial times, included between Egypt and Britain in 1829, Britain and Ethiopia in1902, Britain and Congo in1906, Egypt and Sudan in 1959 followed by Nile River Cooperative Framework in 1997 and finally the Entebbe Agreement in 2011, which restructured allocations and control over Nile water resources among the sharing countries.

Table 1.1 Trans-boundary Rivers of the World

S. No.	River	Length (km)	Sharing countries
1	Nile	6853	Rawanda, Barundi, Uganda, Congo, Tanzania, Kenya, Ethiopia, Eritrea, Sudan, South Sudan and Egypt.
2	Mekong	4350	China, Myanmar, Laos, Thailand, Cambodia and Vietnam
3	Ganges	2525	India and Bangladesh
4	Colorado	2333	USA and Mexico
5	Brahmaputra	2900	India Bangladesh and China
6	Indus	3200	Pakistan and India

May it be the Nile or the Amazon, the Euphrates or the Tigris rivers, the Mediterranean or the Pacific, or the Indus river, water has been instrumental in the survival of the mankind. In the subcontinent, 3 out of 7 South Asian countries, namely Pakistan, Bangladesh and Nepal are involved in water sharing conflicts with India. These conflicts have many times pushed the two nations to the state of war between them. Thus, a workable agreement between the sharing countries is indispensable for peaceful utilization of the water resources.

1.1.2.2. Trans-boundary Water Issues Between Pakistan and India

Historically, there have been numerous international conflicts between India and Pakistan emerging out of sharing of the water of Indus river system. Boundaries of the two neighboring states were drawn right across the Indus Basin, primarily based on the population density disregarding the river system crossing the boundary. Thus Pakistan, by virtue of its natural geographical handicap, formed the lower riparian and India as upper riparian. Consequently, a justifiable water distribution was necessary for viable co-existence of the two nations.

However, soon after independence, India in 1948 stopped the supply of water from the two headwork under its control i.e. Madhopur headworks on the Ravi river and Ferozpur headworks on the Sutlaj river. Shortage of water in the respective commands became so acute and intolerable that it deprived about 5.5 per cent of Pakistan's arable land from irrigation water putting tremendous strain on Pakistan's agriculture. The problem remained unresolved even after several dialogues between the two countries. Finally, then President of the International Bank of Reconstruction and Development (IBRD) offered his services to find a workable solution to the problem leading to the Indus Water Treaty of 1960.

1.1.3. Indus Water Treaty 1960

The water of the Indus river system originate in Tibet and Himalayan mountains of the state of Jammu and Kashmir, flowing through the plains of Punjab and Sindh and finally drain into the Arabian sea. Geographically, the 2-state partition was such that most of the watersheds of rivers of the Indus Basin were in India and occupied kashmir, while majority of the commanded area existed in Pakistan. At the time of partition, some of the headworks of Indus and its tributaries went under Indian control including Modhupur headwork on river Ravi and Ferozpur headwork on river Sutlej. These head works were serving the Upper Bari Doab canal and Dipalpur canal, which had 90% of the commanded area in Pakistan. Soon after partition in 1947, India being the Upper riparian, created problems for Pakistan by closing the Ferozpur head works, which stopped the flow of water to the relevant command area in southern Punjab of Pakistan.

Dialogues between Pakistan and India over sharing of the Indus river water were carried out and agreements between the two states were signed in December 1947 and May 1948, but these did not sustain practically for longer time. Pakistan was facing hard time in meeting the water requirements of about 0.65mha irrigated area. Consequently, the World Bank was approached to find a feasible solution through an agreement called the Indus Water Treaty of 1960.

The Indus Water Treaty is a water sharing agreement over water of Indus river system between India and Pakistan mediated by the International Bank for Reconstruction and Development, presently called the World Bank. The negotiations on the subject continued for 9 years. Finally, the treaty was signed in Karachi on September 19, 1960 by the President of Pakistan and Prime Minister of India duly witnessed by the World Bank, which attempted to resolve the issue with optimum utilization of water of the Indus system of rivers based on principles of equality. Both the countries agreed to exchange flow data and resolve issues through permanent Indus Commissions of the two countries.

The Indus river system catering the water needs of the Indus Basin, comprises 6 rivers namely, Indus, Jhelum, Chenab, Ravi, Sutlej and Beas. According to the treaty, 3 eastern rivers, namely Ravi, Sutlej and Beas with mean annual flow of 33 maf, were allocated for exclusive use by India before entering Pakistan, while the water of three western rivers; Indus, Jhelum and Chenab with mean annual flow of 139 maf were allocated to Pakistan. In addition to the allocation of rivers to each country, Pakistan received one time financial compensation in the form of two dams (Terbela and Mangla), 8 link canals, 5 barrages and one syphon for the loss of water from eastern rivers. The financial support for the project was provided through the Indus Basin Development Fund (IBDF) created by the World Bank and other donor countries including UK, Germany, Australia, New Zealand and Canada and was administered by the IBRD.

The treaty comprised 12 articles and 8 annexes. The treaty, in principle, fixed the rights and obligation of India and Pakistan in relation to each other. Resultantly, India has been annually using an average of 33 maf of water from the eastern rivers and has built several dams and barrages to supply water to the Indian Punjab province and its neighboring states. In addition, India has also been drawing limited water from the western rivers allocated to Pakistan, which is violation of the treaty by India. Some of the articles and annexes are presented below to understand the bases of the violations of the treaty by India and consequent complaints lodged by Pakistan.

Article II: All water of the eastern rivers (Ravi, Sutlej and Beas) shall be available for the unrestricted use of India. Pakistan shall be under obligation to let flow all water of the eastern rivers, and shall not permit any interference with the water of these rivers except for domestic use and non-consumptive use i.e. navigation, floating of timber or other property, flood protection or flood control, fishing or fish culture and wildlife.

Article III: India shall nor store any water or construct any storage works on the western rivers except as provided in annexes D and E of the treaty.

Article III, Annex D: All water of western rivers (Indus, Jhelum and Chenab) shall be available for unrestricted use of Pakistan. India shall be under obligation to let flow all the water of western rivers, and shall not permit any interference with the water of these rivers except for domestic use and non-consumptive use, limited agriculture use and limited utilization for generation of hydro-electric power.

Article IV: India shall not increase the catchments area, beyond the area on the effective data of any natural or artificial drainage or drain which crossed into Pakistan, and shall not undertake such construction or remodeling of any drainage or drain whose crossing might use material damage in Pakistan or entail the construction of a new drain or enlargement of an existing drainage or drain in Pakistan.

India being an upper riparian in the use of water of the Indus Basin violated the Indus Water Treaty in many of the cases. Two of these cases (Baglihar Dam and Kishenganga dam) are given below.

1.1.3.1. Baglihar Dam

The Baglihar dam, which is also known as Baglihar Hydroelectric Power Project, is a **run-of-the-river** power project on the Chenab river in the southern district of the Indian occupied state of Jammu and Kashmir. It is a gravity dam and is constructed over Chenab river. The height of the dam is 144 m and extends over a length of 117 m. The elevation of crest is 844.5 m. The live capacity of the dam is 1,50,00,000 m^3 and is capable of producing 900 MW hydropower. This project was conceived in 1992, approved in 1996, construction began in 1999 and was opened in 2008. The first phase of the Baglihar Dam was completed in 2004 and the second phase completed on 10 October 2008.

Fig.1.2 Baglihar Dam and Chenab River System

The Indus Water Treaty (1960) contained provisions for India to establish river-run power projects with limited reservoir capacity and flow control needed for feasible power generation. Availing this provision, India planned for 8 run-of-the-river projects, Baglihar and Kishanganga Hydroelectric projects were two of them. In 1999, Pakistan claimed that design parameters of Baglihar project had violated the Indus Water Treaty of 1960, as it obstructed the flow in the river Chenab. Pakistan demanded to change some of the design parameters. During 1999-2004 India and Pakistan held several rounds of talks on modifying the design of projects, but could not reach at any agreement.

After failure of talks on January 18, 2005, Pakistan approached the World Bank with some objections to the construction of Baglihar dam. Consequently, in May 2005 the World Bank appointed Professor Raymond Lafitte, a Swiss civil engineer, to study the case critically. Lafitte declared his final verdict on February 12, 2007 in which he upheld some minor objections of Pakistan, declaring that storage capacity of the dam be reduced by 13.5%, height of dam structure be reduced by 1.5 m (i.e. from 4.5

m to 3 m) and power intake tunnels be raised by 3 m, thereby limiting some flow control capabilities of the earlier design. However, he rejected Pakistan’s objections on height and gated control of spillway. Both parties (India and Pakistan) had already agreed that they will abide by the final verdict. Finally, on June 1, 2010, India and Pakistan resolved the issue relating to the initial filling of Baglihar dam in Jammu and Kashmir with the neighboring country deciding not to raise the matter further.

1.1.3.2. Kishanganga Hydroelectric Project

The Kishanganga Hydroelectric dam is a concrete faced rock fill dam that is located on river Neelum, which is tributary to the river Jhelhum in the area of Jammu & Kashmir. It is also considered run-of- the-river hydroelectric project designed to divert water from Neelum (Kishenganga) river to power plant in the Jhelum river basin. Its construction started in 2007 that was stopped by the international court because of the protest lodged by Pakistan in 2011. The height of dam is 37 mand has been designed for installed capacity of 330 MW.

Pakistan is constructing the Neelum–Jhelum Hydropower Plant downstream of the Kishanganga location on the same Neelum river. Both the plants are expected to operate by diverting river water to the power station before it is discharged into the river. The Kishanganga Project will divert a good portion of the river Neelum from Pakistan, which would reduce the power generation at the Neelum–Jhelum Hydropower Plant. India maintains that the project will divert only 10% of the river's flow while actual withdrawal has been estimated as high as 33% that would minimize the flow to the river Jhelum as both the projects are diverting river water.

Pakistan protested against the construction of Kishanganga plant and appealed to the international court at Hague complaining that the Kishanganga Hydroelectric Plant is violating the Indus River Treaty by increasing the catchment of the Jhelum river and depriving Pakistan of its water rights. Consequently, it would detain and reduce the flow of the Neelum/ Jhelum rivers. In June 2011, the CoA visited both the Kishanganga and Neelum–Jhelum projects. The Hague court gave a partial award in February 2013 and final award in December 2013, ruling that India could divert a minimum amount of water for power generation or non-consumptive use of water.

1.1.4. Water Apportionment Accord 1991

Apart from the trans-boundary issues between the two neighboring countries (India and Pakistan) disputes over water sharing of the Indus river system among various consumers existed even during pre-partition period. Reported incidents include protests lodged relevant to the Sutlej Valley and Sukkur Barrage projects during 1920, Protest of Bahawalpur state against the allocation of water to non-riparian areas and the protest of the Government of Sindh against the development projects of the province of Punjab in 1939.

To resolve the issues related to the apportionment of water of the Indus river system, a number of committees and commissions were constituted from time to time, which included Water Allocation Committee of 1968, 1970 and 1977. The major issues were related to the water allocation, reservoir release patterns, drawdown levels and use of groundwater in relation to surface water deliveries. However, no sustainable solution acceptable to all the stakeholders was reached and implemented. Therefore, disputes of water sharing between the provinces of Punjab and Sindh and to some extent with other provinces continued.

Finally, on March 16, 1991 an agreement to share water of the Indus river system was reached between the 4 provinces of Pakistan, which is referred as “Water Apportionment Accord 1991”. It was based on the existing and future water needs of the provinces, and unanimously reached between the Prime Minister of Pakistan and the Chief Ministers of the four provinces resolving the long outstanding dispute. The accord provided the following workable features:

1) It attempted to ensure the annual availability of canal water to each province under the umbrella of the Federal Government.
2) It also apportioned the expected flood water supplies from future storage facilities.
3) The accord concluded the seasonal and annual water allocations to the sharing provinces as given in Table1.2.

Table 1.2 Water Allocation in Accordance with Apportionment Accord 1991

S. No.	Province	Kharif/Summer (maf)	Rabi/ Winter (maf)	Annual (maf)
1	Punjab	37.07	18.87	55.94
2	Sindh	33.94	14.82	48.76
3	KPK (Commanded Area)	3.48	2.30	5.78
4	KPK (Civil Canals)	1.80	1.20	3.00
5	Balochistan	2.85	1.02	3.87
Total Allocation (maf)		77.34	37.01	114.35

Indus River System Authority (IRSA) was created in 1992 to implement the Water Apportionment Accord agreed among the provinces in 1991. At the time of the Accord the Indus Basin system consisted of the Tarbela reservoir on the Indus, Mangla reservoir on the river Jhelum, the network of link canals constructed under the Indus Replacement Works as a part of the Indus Water Treaty, and the system of barrages to divert water into the canals, some of which had existed since the 19th century. Since that time, IRSA has functioned effectively to allocate available supplies with dispute resolution among the provinces of Pakistan.

1.2. Irrigation of Agricultural Land

Irrigation is generally defined as ‘the artificial application of water to soil through a manually, electrically or mechanically managed system, for supplying moisture essential for plant growth. However, in the broader sense, it may be defined as the application of water to the soil for any number of the following purposes:

- To supply the moisture essential for plant growth.
- To transport nutrients from soil to the plant.
- To provide crop insurance against short duration droughts.
- To cool the soil and the atmosphere, thereby making more favorable environment for plant growth.
- To protect the crop from hazards of frost in winter and dehydration in hot weather.
- To dilute or washout salts in the root zone of salt affected soils.
- To soften tillage pan and soil clods for facilitating fine seedbed preparation and planting operation.
- To encourage plant root development.
- To encourage biological activities in the root zone, which are favorable for plant growth.

1.2.1. Importance of Irrigation

Pakistan is primarily an agricultural country where agriculture contributes 19 % of GDP and more than 43% of labor force. Under arid and semi-arid climatic conditions of the country, where average annual rainfall ranges from 254 to 356 mm (10 to 14 inches) against a potential demand of about 1778 mm (70 inches) of water for agriculture, irrigation is considered essential for crop production. Increasing pressure of population demands increased agricultural production for food security. This necessitates multiple cropping and better crop yields through improved uses of fertilizers and crop varieties, which cannot be practiced successfully without providing requisite irrigation water. Water stands at the top most important factors of plant growth. In fact, there can be no life without water.

1.2.2. Sources of Irrigation Water in Pakistan

Primarily, there are three sources of irrigation water, namely, surface water (canal water), groundwater (pumped water) and rainfall. Each one of these has specific importance and implication on production potential of crops.

1.2.2.1. Surface Water

The major sources of surface water contributing to the Indus Basin irrigation system, are the snow melts and precipitation over the mountainous regions. Runoff water through streams and rivers is stored in the reservoirs or is diverted directly through canal systems to the fields for irrigation. The Indus Basin irrigation system primarily comprised of 6 rivers namely, Indus, Jhelum, Chenab, Ravi, Sutlej and Beas. In Pakistan, however, the surface water resources, because of Indus Water Treaty (1960), were limited to the water of the Indus, Jhelum and Chenab rivers. Since, the eastern 3 rivers namely, Ravi, Sutlej and Beas were given to India. Consequently, Pakistan was deprived of the eastern rivers namely, Ravi and Sutlej. Therefore, water from western rivers has been diverted to the eastern rivers through 8 link canals to supply water to the areas previously irrigated by Ravi and Sutlej rivers.

The irrigation water in the Pakistan's Indus Basin Irrigation System (IBIS) is conveyed through 48 canals having a total length of 56,073 km and 137,000 main watercourses having a length of 1.6 million km. The system is further supported with farm channels and field ditches leading the water to the irrigated fields. Previously, the total number of watercources was reported as 107,000. The canal system supplies water to a network of branch canals, distributaries, minors, watercourses and field channels.

The 77-years record of the Indus river (1922-23 to 1999-2000) indicates that the watersheds of the Indus river yield about 138.7 maf of water annually (Ahmad 2002). According to Bandaragoda and Rehman (1995), the total annual flow into the river system of the Indus Basin is estimated to be 139 maf, out of which, 104 maf are diverted to the canal irrigation system to irrigate over 16 million ha of land. About 77.4% of the total irrigated area falls in Punjab, 2.8% in KPK, and 19.8% in Sindh and Balochistan. A significant portion of annual inflow is lost through seepage in the conveyance system (canals, distributaries, minors and watercourses) allowing only 43 maf to reach the field inlets and only 31 maf are available for crop use in the field as shown in the flow chart (Fig.1.3).

Referring to Fig.1.3, out of the total estimated 139 maf annual flow to the system, about 35 maf annually escape to the sea unutilized, particularly during floods, and therefore, does not become part of the canal irrigation system. Out of this, 10 maf are claimed to be required for harbor development, fish culture and protection of natural habitats below the Kotri Barrage. The remaining 25 maf can be beneficially conserved and stored in proposed storage dams such as Kalabagh, Bhasha and Dhasu etc. Thus, the development of storage reservoirs is the prime need of the country to overcome the floods during summer and deficits during winter season.

Considering the temporal distribution of water availability in the irrigation system, about 84% of the total annual river flow occurs during Kharif season and about 16% during Rabi season giving a Rabi:Kharif Ratio of 1:5.2 (Ahmad and Chaudhary 1988). The crop water requirements on the other hand occur at the ratio of 3:5 between Rabi and Kharif. Thus, there exists incompatibility between the water availability and requirements, particularly, during Rabi season, which has partially been regulated through managed releases from dams and reservoirs. It shows that storing of water during high flows and releases during low flows through dams and reservoirs is indispensable for sustainable agriculture. Presently, there are 3 major reservoirs in Pakistan namely, Tarbela, Mangla and Chashma having a total capacity of about 16 maf, which is continuously reducing because of sediment inflow.

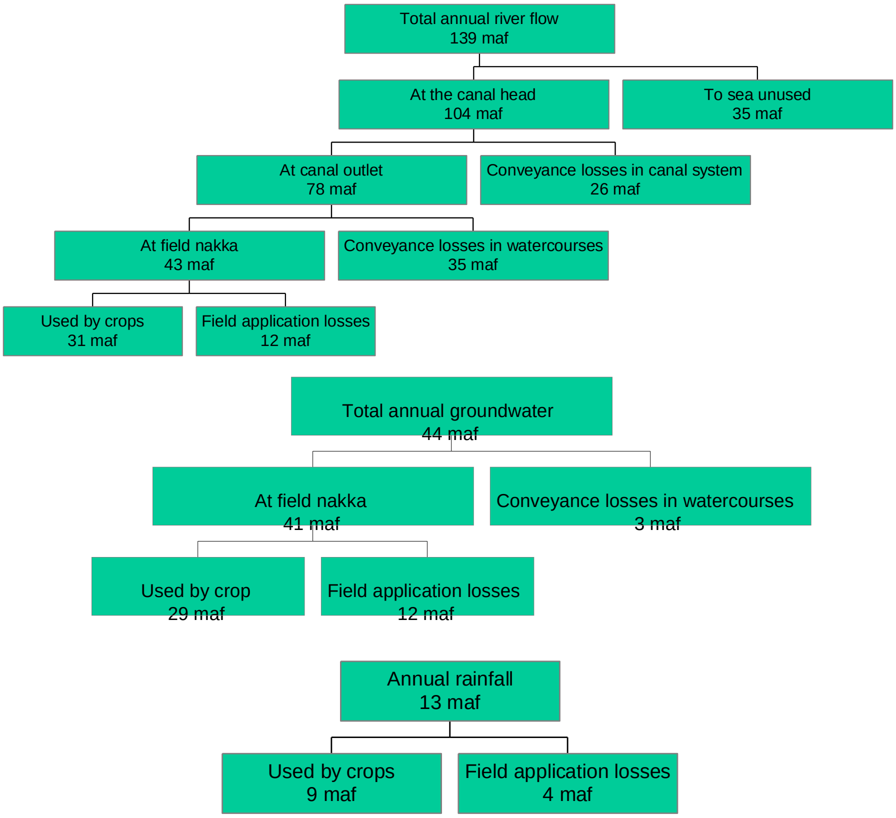

Fig.1.3 Flow Chart of Indus Basin Irrigation System

Source: Bandaragoda and Rehman (1995)

1.2.2.2. Groundwater

The groundwater has been in use since ancient times. Groundwater, which meets more than 40 to 50% of the irrigation water requirements, is obtained by pumping groundwater from the alluvial aquifers of the Indus Basin. Pakistan is very fortunate to possess a rich source of groundwater, particularly in the provinces of the Punjab and Sindh. According to the NESPAK (1991), about 48.5 maf of water are being pumped for irrigation, human consumption and industrial uses. More than 90% of the extracted groundwater is used for irrigation purposes and the remaining less than 10% is used for domestic, industrial and livestock needs.

In 2003, about 629,000 private tubewells in addition to more than 2500 public tubewells including SCARP tubewells were reported to be pumping groundwater to supplement surface water supplies (Qureshi et al. 2003). Historically, on the average, about 10,000 tubewells have been added to the system annually. The tubewell number has therefore, increased to more than 750,000 by the year 2016. According to Bandaragoda and Rehman (1995), the public and private tubewells contributed about 44 maf to the field outlets, which allowed about 29 maf of ground water for crop use (Fig.1.3). As the tubewell number has increased, their contribution to the irrigation system is currently estimated more than 50 maf. Constrained canal water supplies cannot alone meet the requirements of irrigated agriculture under rising population pressure. Consequently, the increased use of groundwater would result in increasing number of tubewells as long as the groundwater resources can support.

The major sources of recharge to the groundwater reservoir of the Indus Basin include the seepage from canal irrigation network (canals, watercourses, farm channels and irrigation return flow) and rainfall. The return flow from field irrigation has been estimated about 24% of the total annual recharge to groundwater (FDP 1992). For the past decade, the groundwater has been reported to be declining due to over pumping and consequent undermining of groundwater.

1.2.3. Rainfall

The third major source of water for agricultural purpose is rainfall, which is neither sufficient nor reliable. Rainfall in Pakistan is markedly variable in magnitude, time of occurrence and its aerial distribution. Practically, whole of Pakistan's productive land lies in arid to semi-arid zone. Almost two-thirds of the rainfall is concentrated in the three summer months (July – September). The mean annual precipitation ranges from less than 100 mm in parts of the lower Indus plain to over 750 mm near the foothills in the Upper Indus Plain. The total water generated by rainfall is estimated as 32 Billion m^3 out of which 10 to 20% is contributed to agriculture. The Indus Basin can be categorized in rainfall zones in terms of the aerial distribution as given in Table 1.3.

Table 1.3 Aerial Distribution of Rainfall in Pakistan

Rainfall Zone	Percent of land	Average Annual Rainfall
Arid	67	Less than 10”
Semi Arid	24	10-20”
Humid	5.5	20-30”
Para Humid (Very Wet)	3.5	30”

There are two major sources of rainfall in Pakistan; the Monsoons and the Western Disturbances. The relative contribution of rainfall in most of the canal commands is low when compared with the two other sources of irrigation water i.e.' canal water and groundwater. More than 60% of the Kharif season rainfall is concentrated in the month of July for almost all of the canal commands. The Monsoons originate in the Bay of Bengal and usually reach Pakistan, after passing over India, in early July and continue till September.

According to Bandaragoda and Rehman (1995), out of 13 maf annual rainfall, about 9 maf of rainfall are estimated to reach the irrigation system for crop use. Thus, a total of 69 maf water is available from all sources (canal water, groundwarter and rainfall), which meets only 64% of the annual irrigation requirements.

1.3. Pakistan's Indus Basin Irrigation System

After the Indus Water Treaty, the availability of water to Pakistan has been limited to the western three rivers; Indus, Jhelum and Chenab only. The total inflow to the Indus Basin river system (Fig. 1.3) is 139 maf. The river Indus alone contributes 65% of the total river flow, while the share of river Jhelum and Chenab is 17 and 19%, respectively. The months of peak flow are June and August during the Monsoon season.

The Indus river system is the prime source of irrigation water in Pakistan. It originates in Tibet from the glacial streams of the Himalayas and enters Pakistan in the northeast. It runs generally south-westward the entire length of Pakistan, which is about 2,900 km, and finally empties into the Arabian Sea. Outside the Indus Basin, the water resources potential in the province of Balochistan is estimated as 0.9 maf of groundwater. Out of this about 0.5 maf is currently being exploited while the remaining 0.4 maf remains exploitable for irrigated agriculture.

The total area of Pakistan is about 196 million acres out of which, the cultivated area is 26% (21 mha or 51.8 million acres). About 77% of the cultivated area (16.2 mha or 40 ma) is irrigated. The Pakistan's Indus Basin irrigation system (Fig. 1.4) comprises river Indus and its tributaries comprising Jhelum, Chenab, Kabul, Kurram, Soan, Haro and many other small rivers. In addition, many hill torrents such as D.G. Khan, D.I. Khan and Kirther range hill torrents occasionally drain into the Indus Basin water system. The Indus Basin irrigation system of Pakistan comprises 3 major dams, 12 link canals, 16 barrages, 2 head works, 2 siphons across major rivers and 44 main canal systems (23 in the Punjab, 14 in Sindh, 5 in KPK and 2 in Balochistan). The link canals have the capability of transferring at least 20 maf of water from western rivers (Indus, Jhelum and Chenab) to the 2 eastern rivers (Ravi and Sutlej) to support their command areas as these 2 rivers have been put to the discretion of India through Indus Water Treaty of 1960.

The annual operating capacity of the canals of the Pakistan's irrigation system is 16 mha meters through 137000 canal outlets or turnouts. The total length of canals and distributaries is about 63000 km. On the average, these canals annually draw about 12.9 mha meters (104 maf) at canal heads. The canal system comprises 2 types of canals namely, perennial canals and non-perennial canals. The perennial canals deliver water throughout the year, which command nearly 8.6 mha of cultivable area. The non-perennial canals deliver water only during kharif season and command nearly 5.4 mha of land. About 15.2 mha (37.5 ma) of land which is about 48.6% of culturable land, is considered culturable waste. This area remains out of cultivation due to shortage of water in the country, and therefore, can be utilized beneficially if requisite water is made available. In addition to the weir controlled canal system, many hill torrents such as D.G. Khan, D.I. Khan and Kirther range hill torrents occasionally drain into the Indus Basin water system to support its irrigation capacity.

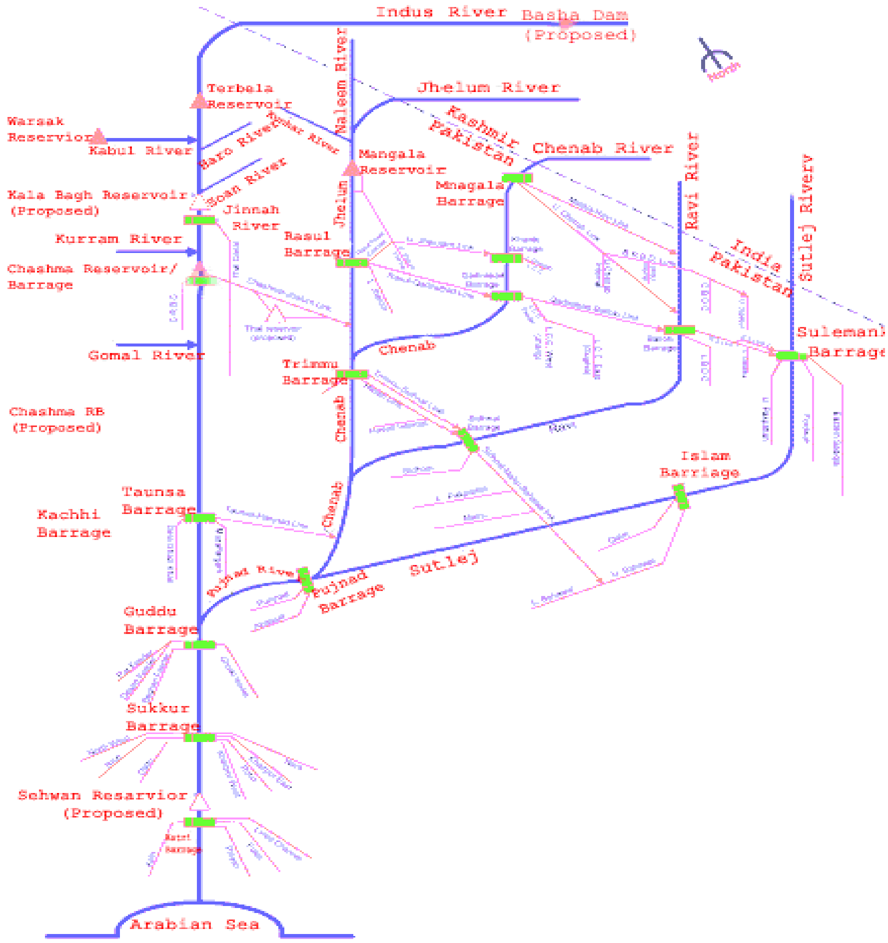

Fig. 1.4 Pakistan's Irrigation System Showing Rivers Dams and Barrages

1.3.1. Historical Developments in Indus Basin Irrigation System

Historically, the irrigation in the Indus Basin started with the human settlement, primarily along the banks of the rivers (riverain areas), where irrigation of the fields was carried out through inundation canals, which were functional only during periods of high river flow. These provided irrigation water for kharif (Summer) season and some moisture during Rabi (winter) season. The weir controlled irrigation system was initiated in 1859 with the completion of Upper Bari Doab canal (UBDC) from Madhupur headworks on rive Ravi. Last inundation canals were connected to the weir controlled system in 1962 with the completion of Guddu Barrage on river Indus. It was followed by Sirhind canal from Rupar headworks on river Sutlej in 1872 and Sidhnai canal from Sidhnai barrage on river Ravi in 1886. Further development of canal system included the following:

- The Lower Chenab Canal fron Khanki headworks on river Chenab in 1892.

- The Lower Jhelum Canal from Rasul headworks on river Jhelum in 1901.
- Lower and Upper Swat Canals from river Kabul and Paharpur canal completed during 1885-1914.

During early 20th century, it was realized that the water supplied by different river systems was not proportionate to the command areas. For example, supplies from river Ravi was not sufficient to meet the requirements of Bari Doab, while river Jhelum had surplus water for the irrigated areas of Chaj Doab. Therefore, it was desired to redistribute water of rivers to achieve more uniformity in the Indus Basin. Consequently, the Triple Canal Project linking Jhelum, Chenab and Ravi rivers was completed during 1907–1915. The project permitted the transfer of water from river Jhelum to Chenab and from Chenab to Ravi. This concept of linking rivers from surplus to deficient supply, provided basis of resolution for water of the Indus system in Indus Water Treaty. The chronological further developments in the canal network took place as under:

- Sutlej Valey project, comprising 4 barrages and 2 canals, was completed in 1933.
- Haveli and Rangpur canals from Trimmu headworks on Chenab river were constructed in 1939.
- Thal canal from Kalabagh headworks on Indus, were completed in 1947.
- International trans-boundary issue was created by India in 1948 when two headworks on Ravi and Sutlej rivers were closed and water supplies were stopped to the respective command areas in Pakistan.
- The dispute was finally resolved through Indus Water Treaty in 1960 under aegis of the World Bank, which allocated 3 eastern rivers to India and 3 western rivers to Pakistan along with a reconstruction and settlement plan comprising 2 major dams, 5 barrages, one siphon and 8 link canals in favor of Pakistan to compensate for loss of water for the command areas of eastern rivers. The reconstruction completed during 1960-1976.
- After India-Pakistan partition, Kotri, Taunsa and Guddu barrages were completed on the river Indus to replace inundation canals.

Historical developments in the irrigation system before the introduction of wear controlled canal system in1872, can be summarize as under:

- In the year 1633, Moghal Empror constructed a canal (*Shah Nehr*) from river Ravi to Shalimar Gardens, which was later named as Hasli canal.
- In 1817, British Army Engineers initiated construction and improvement of irrigation canal in the subcontinent. The first canal undertaken was Western Jumna canal.
- Construction of “Sirhind” canal from Sutlej river was initiated in 1832, which put to operation in 1872. This was the first canal in the subcontinent provided with a permanent masonary head work.
- Construction of Bari Doab canal was started in 1851, which was completed in 1859.

1.4. Barrages Headworks and Canals

A barrage is an artificial barrier or a control structure built across the river to prevent flooding, aid irrigation or navigation system or to generate electricity by tidal power. It is a low head diversion dam to raise the level of water and divert it through the canals that originate at the barrage. A headwork is a water diversion structure provided over a main or branch canal to raise the level of water in the canal to divert water to the branch canal or a distributary.

The barrages and headworks of the Indus Basin irrigation system located at various rivers being operated and managed in Pakistan to support the irrigated agricultural areas are given in Table 1.4. The link canals along with the rivers, which have been linked through these canals, are given in Table 1.5.

Table 1.4 Barrages and Headworks at Pakistan's Irrigation System

S. No.	River/ Location	Barrages and Headworks
1	Indus	Chashma, Taunsa
2	Jhelum	Rasul
3	Chenab	Marala, Khanki, Qadirabad and Trimmu
4	Ravi	Balloki and Sidhnai
5	Sutlej	Sulaimanki and Islam
6	Confluence of 5 Rivers	Panjnad

Table 1.5 Link Canals of Pakistan's Irrigation System

S. No.	Rivers Linked	Link Canal Systems
1	Indus – Jhelum	Chashma – Jhelum Link (C-J Link)
2	Indus – Chenab	Taunsa – Panjnad Link (T-P Link)
3	Jhelum – Chenaqb	Rasul – Qadirabad Link (R-Q Link)
4	Chenab – Ravi	Marala – Ravi Link (M-R Link)
5	Chenab – Ravi – Sutlej	Bambanwala –Ravi – Bedian Link (BRB Link)
6	Chenab – Ravi	Upper Chenab – Balloki Link (C-B Link)
7	Chenab – Ravi	Qadirabad – Balloki Link (Q-B Link)
8	Chenab – Ravi	Trimmu – Sidhnai Link (T-S Link)
9	Ravi – Sutlej	Balloki – Sulaimanki Link (B-S Link)
10	Ravi – Sutlej	Sidhnai – Malsi Link (SM Link)

1.4.1. Types of Canals

From operational point of view the canals may be classified as inundation, perennial and non-perennial as detailed below. The design principles and characteristics in terms of cross section slope and velocity remain the same to carry a given flow rate.

1.4.1.1. Inundation Canals

These canals operate only during raised level of water in the rivers (flood conditions). Therefore, these canals receive flood water and supply uncertain quantity of water to the fields. These canals essentially operated before the era of weir controlled system, introduced by British regime.

1.4.1.2. Perennial Canals

These canals originate from headworks, barrages or dams and continue flowing on regular basis all the year round. Examples include Lower Bari and Upper Bari Doab canals, Lower and Upper Chenab canals and Upper Jhelum canal.

1.4.1.3. Non-Perennial Canals

Non-perennial canals run only a part of the year during monsoon. The example includes Hakra canal.

1.5. Operational Issues of Pakistan's Irrigation System

The Indus Basin irrigation system has been operating for more than a century. Historically, the system has faced a number of changes in flow regimes including flood water from rivers, inundation canal irrigation to wear controlled system of canals supported by head works, barrages and dams. The system has been further influenced by human settlement patters ranging from nomadic mode and forced settlement patterns to properly organized villages and canal commands with introduction of water rights, water distribution policies at all levels of the system and development of planning and management infrastructures including provincial irrigation departments and finally Provincial Irrigation and Drainage Authorities (PIDAs). These developments and improvements in the irrigation system certainly brought radical changes in the operational performance, use of irrigation water and progressive improvements in the social as well as financial life of the end users / farmers. The productive potential of land and water resources also enhanced in terms of crop yields, quality and varieties of agricultural products with scientific research inputs and technical knowledge based advancements in the field of agriculture.

For sustainability of a system, proper maintenance and up gradation along with a balance between the demand and supply, are pre-requisites that must be guaranteed in order to avoid overstressing of the system. Consistently increasing population of the region certainly, created more demand for food and other agricultural products, which the system could not supply because of poor maintenance, lack of operational safeguards and financial and management lapses. The irrigation system in Pakistan has been facing a number of problems, which still remain as a challenge to resolve for the scientists, engineers, planners, policy makers and managers. Major operational problem of the irrigation system in Pakistan can be summarized as:

- Inefficiencies in the conveyance, distribution and application of water.
- Inadequacy of available water supplies.
- Inequity of distribution of irrigation water at all levels of the system.
- Inadequate and diminishing storage reservoir facilities.
- Unreliability of water supplies.
- Increased cropping intensities induced imbalance between supply and demand of water.
- Lack of knowledge of farmers on improved technologies on irrigation scheduling.
- Insufficient training opportunities for farmers on advanced technologies.
- Discrepancy among canal and tubewell water charges.
- Lack of legal framework of groundwater exploitation and development.
- Over drafting of groundwater from sweet water aquifer.
- Lack of Farmers' participation in operational activities of irrigation system.
- Inefficient water distribution system at watercourse command.
- Poor cleaning and maintenance of watercourses.
- Inadequate on-farm and off-farm drainage facilities.

1.5.1. Inefficiencies in Conveyance and Application

Efficiency is defined as the ratio between input and output, which may be expressed as a fraction (Input/Output) or as %age when multiplied by 100. Inefficiency indicates the degree of losses in the system, which is equal to 100-percent efficiency.

After Indus Water Treaty (1960), major rivers contributing to the Pakistan's irrigation system include Indus, Jhelum and Chenab rivers, which contribute a total annual inflow of 139 maf to the canal system as measured at the rim stations (Kalabagh on Indus river, Mangla on Jhelum and Marala on Chenab river). Out of this, an annual average of 35 maf is lost to the Arabian Sea without utilization for irrigation. Thus, only 104 maf (i.e. 139 – 35 = 104), is conveyed through the canal system (Main canals, branch canals, distributaries, minors and watercourses), where it faces high degree of seepage losses and ultimately only 43 maf reached at the field inlets.

1.5.2. Conveyance Efficiency

Conveyance efficiency means the efficiency of the canal irrigation system in conveying water from river to the fields or between any two conveyance components of the irrigation system. Thus, referring to Fig.1.3, the conveyance efficiency of the canal system from canal head to the irrigated fields remains = (43/ 104) × 100 = 41.3%, which gives the extent of water losses or inefficiency in the given component of canal irrigation system = 100 – 43 = 58.7%

1.5.3. Application Efficiency

Application efficiency means efficiency of the system during application of irrigation water to the fields. It is defined as the ratio between the water stored in the root zone of crop to the amount of water applied to the field. Research data has shown that about 15% of water is lost during conveyance through canals and 40% through watercourses, while 25% is lost during field application.

1.5.4. Inadequacy of Available Water Supplies

Inadequacy means the available water supplies are not enough to meet the crop water requirement. The existing canal irrigation system meets only 40% of the crop water demand. The remaining 40 to 50% of the demand is met from groundwater and 10 to 20% from rainfal (Choudhry 1987). Thus, the water supplies of the irrigation system in Pakistan are iadequate to meet the potential demand of agriculture sector.

1.5.5. Inequity of Distribution

Equitable distribution does not mean equal distribution to all the share holders but in accordance with their rights as determined by the warabandi rules, which may be based on cropped area or on the land holding. Under the existing water distribution at the watercourse command (warabandi) however, the actual distribution of water among the share holders is not in accordance with the relevant land holding because of water losses during conveyance in the watercourse. Therefore, the 1st farmer (Head farmer) gets the maximum and continues decreasing for the subsequent farmers and least is available with the last farmer. Therefore, inequity of distribution among the share holders exists in the irrigation system of the country.

1.5.6. Unreliability of Water Supplies

The water supplies can be considered reliable when farmer knows what amount at a given time he will get to irrigate his fields. Unreliability means, the water supplies by the irrigation system are not reliable as these mostly depend on the climatic changes of snow melt, rainfall and releases from the reservoirs. Water supplies during summer season may be 5 times that in the winter season and monsoon rains may exceed the crop water requirement, while in dry part of the summer season, the crops may face higher degree of water stress.

1.5.7. Lack of Farmers' Awareness

The farmers are generally not aware of the water availability, system performance as well as the advanced irrigation technologies. Awareness of the farmers regarding the extent of losses in the system and how to save these losses would help the farmers to use the irrigation water more efficiently. Thus, lack of farmers' awareness is one of the issues, which must be addressed through capacity building and field demonstrations to the farming community.

1.5.8. Lack of Farmers Participation

Participation of farmers in the improvement activities of the irrigation system and requisite investments, would give a sense of ownership to the farmers, which would encourage the farmers to safeguard the system and maintain it properly. Cost sharing of farmers in the watercourse improvement program is a good example of participation. Development projects without farmers' participation have been observed to fail badly. Review of operational issues of Fourth Drainage Project indicated that the project could not achieve its objectives primarily because of lack of farmer participation.

1.5.9. Increased Intensity of Cropping

Cropping intensity is the percentage of the area or land holding put under crop on seasonal or annual basis. The Indus Basin irrigation system was designed based on 75% annual cropping intensity, which has increases to more than 170%. Therefore, the increased cropping intensity is putting more pressure on the system in terms of increased water demand to achieve potential production. Thus, additional efforts are needed to make the system more efficient that may meet the increasing demand for water.

1.5.10. Discrepancy between Canal and Tubewell Water Charges

The water charges means the cost per unit of water delivered. In Pakistan, the tubewell water is more than 20 times costlier as compared to the canal water. In addition, the tubewells are generally owned by the bigger and rich farmers. As canal water meets only 40% of the crop requirements, the farmer is forced to buy costlier tubewell water, which tends to put extra financial pressure on the small farmers.

1.5.11. Issues of Water Charging

Previously, the canal water charges (Abiana) were based on the cropped area, which could be manipulated by the booking clerk (Patwari) and other managers collecting water charges for the Irrigation Department. In addition, the efficient farmer who cropped more area with the same water, was charged more as compared to the inefficient farmer who planted less area with the same water, which means that the efficient farmer was punished for his efficiency. Since 2003, the water charging system has been changed to area based, eliminating the interference of the booking clerk. This system eliminated some of the difficulties of the farmers.

1.5.12. Issues of Groundwater Development

Groundwater is also a natural resource like canal water but it is being pumped as a free commodity i.e. it is not being charged like canal water. Resultantly, any farmer can pump as much as he needs, which is encouraging over pumping leading to undermining of groundwater resource. In addition, the resourceful farmers are taking the share of smaller farmers. Thus, a legal framework is needed to optimally manage and control groundwater pumping for sustainability of the resource.

1.5.13. Inadequate and Diminishing Water Reservoirs

Existing water reservoirs are neither adequate nor sustainable. There are only 2 major reservoirs Tarbela and Mangla, which were constructed as a part of Indus Water Treaty 1960. The total capacity of 3 reservoirs (Tarbela, Mangla and Chashma) is about 16 maf. The capacity is depleting at a rate of about 0.1 maf per year. After the commissioning of Terbela Dam in 1974, no major water reservoir has been added to the irrigation system in Pakistan. Addition of more reservoirs is the priority need of the country to meet the deficits and regularize the irrigation water supplies. Water reservoirs are needed to generate electricity to overcome power shortage in the country.

1.6. Components of Irrigation System

The flow line diagram of the Indus Basin irrigation system (Fig. 1.5) shows various components of the system comprising watershed, river, dam, main canal, branch canal, distributary and a watercourse. Each of these components is described below.

1.6.1. Watershed

Generally, a watershed may be defined as an area draining into a river or a river system through a network of rills and rivulets. Thus, watershed or catchment is an area, which receives precipitation, snow melt and/or spring flow, and contributes to the formation of a network of natural channels with runoff leading to the development of rivers.

The concept of watershed originates from surface hydrology where a river is assumed to be affected primarily by its surface drainage area. In fact, both the surface and subsurface hydrology define a river, and therefore, the importance of subsurface hydrology should not be overlooked.

Thus, a watershed forms the origin of a river irrigation system. It is used to define the surface water drainage boundary as both the surface and subsurface hydrology contribute to define a watershed in the form of surface runoff, interflow and subsurface flow.

The rate of runoff in a watershed depends on the vegetations, land topography, degree and length of slope, shape and infiltration characteristics of the watershed. The sediments in canal irrigation water, originating from the watershed as a result of erosion, cause numerous problems such as loss of storage capacity of reservoirs and conveyance system and increased maintenance problems. Appropriate management of watershed can play an important role in controlling sediment inflow into the irrigation system. Fig. 1.6 shows a typical watershed.

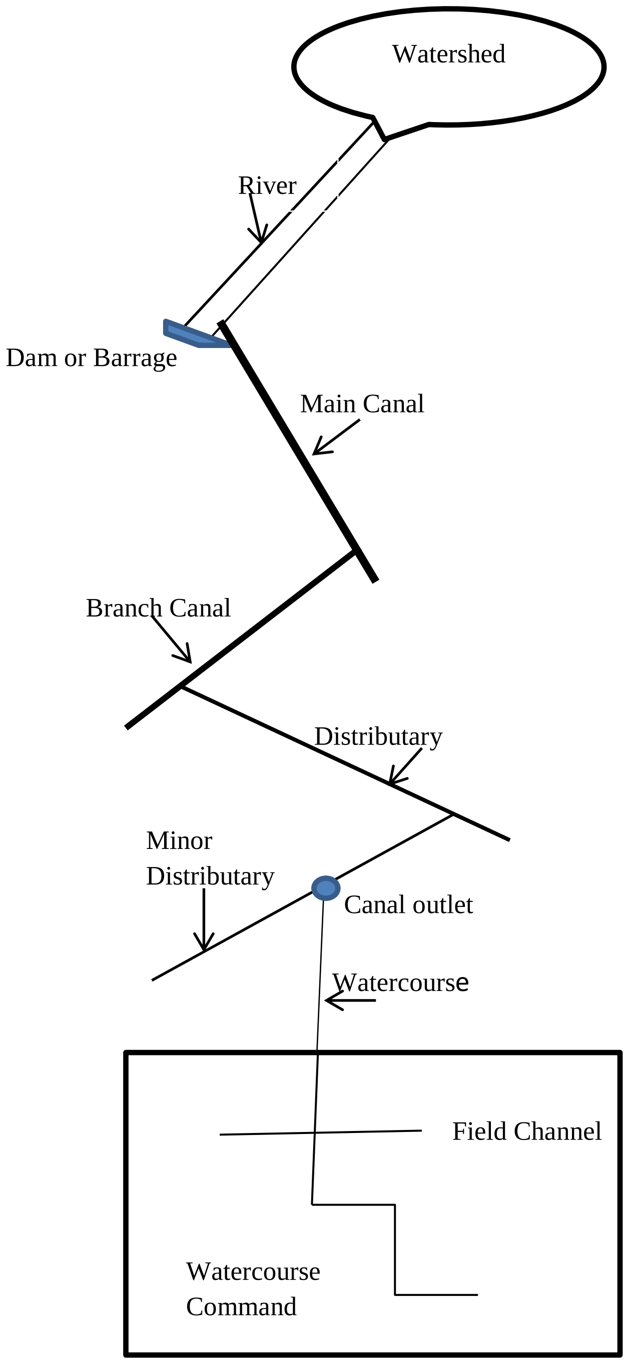

Fig. 1.5 Components of a Typical Irrigation System in the Indus Basin

Fig. 1.6 A Typical Natural Watershed

1.6.2. Rivers

A river is a natural stream, which develops because of runoff produced by the watershed. It receives water from watershed and conveys to the downstream system components through dams and barrages. Rivers Indus, Jhelum and Chenab are some of the examples. Fig. 1.7 shows a typical river or a natural stream and Fig. 1.8 shows a section of river Indus near Bisham Karakoram.

Fig. 1.7 A River or a Natural Stream

Fig. 1.8 River Indus near Bisham Karakoram

1.6.3. Dams

A dam is a control structure built across a river performing the following functions:

(a) Storage of water for regulating the flow regimes during various cropping seasons.

(b) Initiation of irrigation canals to facilitate water supplies to irrigated land.

(c) Raise the head of water for operating irrigation canals.

(d) Hydropower generation to provide electricity for domestic and industrial use.

(e) Provide a check against emerging flood by storing flood water.

Major dams of Pakistan's irrigation system include Tarbela, Mangla and Chashma, though Chashma also serves as a barrage with limited storage capacity. Relatively, smaller dams/ reservoirs include Warsak in KPK, Rawal in Punjab and Mirani in Balochistan province. Fig. 1.9 shows Mangla Dam in Pakistan and Fig. 1.10 shows arched concrete Alwero Dam at Gambela, Ethiopia.

1.6.4. Barrages

A barrage is also a control structure, which is built across the river, mainly to raise the level of water and to create a limited temporary storage for facilitating canal diversion and operation. A barrage diverts river water to the canals through gate operation. The examples include Trimu, Taunsa, Rohri and Sukkur Barrages. Fig. 1.11 shows the Taunsa Barrage of Punjab's irrigation system. A complete list of barrages and Headworks of the Pakistan's irrigation system has already been given in Table 1.4.

Fig. 1.9 Mangla Dam in Pakistan

Fig. 1.10 Alwero Dam at Ehiopia

Fig. 1.11 Taunsa Barrage on River Indus

1.6.5. Link Canal

It is a special canal constructed to link two rivers in order to divert some of the flow from one river to the other one. There are 12 such canals in the Pakistan's Irrigation System under Indus Basin Development Plan. Examples include Chashma-Jelhum link and Balloki-Sulamanki link etc.

1.6.6. Head works

A headwork is a diversion provided over a main canal to raise the water level in the canal and to divert the water to branch canals or distributaries. The examples include Head Faqirian at Fig.1.12 (Kahalwn and Kamper 2004)

Fig.1.12 Head Faqirian in Punjab

1.6.7. Main Canals

Main canal receives its water supply directly from the river through a dam or a barrage. Direct irrigation is usually not carried out from the main canal. Therefore, there are no outlets on the main canals. The examples include Upper Jhelum, lower Jhelum, Upper Chenab, Lower Chenab, Upper Bari Doab and Lower Bari Doab etc. Fig. 1.13 shows a typical Irrigation Canal in Sargodha (Kahalwn and Kamper, 2004). The area irrigated by one canal is called a canal command.

Fig. 1.13 A Typical Main Irrigation Canal in Sargodha

1.6.8. Branch canal

Branch canal take off from the main canal through diversion structures such as head works and convey water to different parts of the irrigated areas through distributaries. The branch canals usually are not provided with outlets for delivery of water to the fields except in special cases. These include Gugera and Rakh branch canals etc.

1.6.9. Major Distributary

Major distributaries (usually called distributaries or 'rajbah') take off from branch canals and sometimes from main canals and supply water to minor distributaries or directly to the main watercourses through canal outlets. The examples of major distributaries are Niazbeg, Shahkot, Pakpattran and Nasrana etc.

1.6.10. Minor Distributary

Minor distributaries (usually called minors) takeoff from major distributaries and rarely from branch canals. These supply water to watercourses for irrigation. As an example, Kamogil, Jallake, Roda and Thatti are the minors of Niazbeg distributary. The design discharge of Niazbeg distributary is 261 cusecs (7.36 m^3 /S) and those of minors range from 10 to 31 cusecs.

1.6.11. Watercourses

A watercourse serving the watercourse command, also called as main watercourse, is defined as any irrigation channel that receives water from a distributary or a minor through a canal outlet “Mogha” and leads to the farms and fields in accordance with a water distribution (warabandi) schedule. It is also called as “Sarkari or Main Khal” as shown in Fig.1.14. It is operated and managed by the Provincial Irrigation Department (PID) but constructed and maintained by the farmers or Water User Association. One or more watercourses, based on the size of irrigated area, may serve each village. On an average, a watercourse commands about 400 acres of land and is shared by about 50 farmers according to a rotation called ‘warabandi’. Each watercourse receives water from canal outlet whose location and size are approved by the PID and delivers to the farms through watercourse outlets and diversions. These diversions are provided with control structures commonly known as concrete “Nacca”. Water flows continuously in a watercourse as long as there is water in the minor of distributary.

1.6.12. Field Channels

Field channels, also called as Farm Channels or Farmer Branches, carry water from the main watercourse to the farms or individual fields. These channels originate from the main watercourse and command individual farmer’s fields at his farm. The main watercourse is a community watercourse that is shared by a number of farmers, while the Field Channels are planned, constructed, operated and maintained by the farmers themselves individually. Generally, the size of field channels is the same as that of main watercourse because they have to carry full flow received from the main watercourse. Water supply from the main watercourse to a given farm is permitted by the Irrigation Department at specific location and time in accordance with the rotation (“warabandi”) system. Water is diverted from main watercourse to the farm/ field channels through farm outlets or “Field Nacca”.

1.7. Irrigation System Management

The irrigation system, as described above, comprises a number of components from rivers to the watercourses (Rivers, Dams, Link Canals, Barrages/ Headworks, Main and Branch canals, distributaries and minors). Their planning, financing, construction, maintenance and management at different levels are exercised by different authorities. Federal Irrigation Management institutions include:

- Ministry of Water and Power (MoW&P)
- Indus River System Authority (IRSA)
- Federal Flood Commission (FFC)
- Planning Commission of Paskistan (PCP)
- Water and Power Development Authority (WAPDA)

Fig..1.14 A lined Main Watercourse Originating from Canal Outlet

Irrigation system management at provincial level is exercised by the following institutions:

- Provincial Irrigation Departments (PIDs)
- Provincial Irrigation and Drainage Authorities (PIDAs)

1.7.1. Federal Level Management

The irrigation system in Pakistan is primarily managed and operated by the Irrigation Department at the provisional government level. At federal level, however, Ministry of Water and Power exists to administer, plan and develop water resources through Water and Power Development Authority (WAPDA) that was established in 1958. The responsibilities of large-scale construction and operation of water resources facilities at federal level such as storage dams, barrages, and link canals lies with WAPDA. It is also responsible for planning and execution of groundwater development and management schemes such as Pipe Drainage, Tubewell Drainage and Reclamation Projects. These projects after completion are transferred to the Provisional Irrigation Departments for operation and maintenance. The examples include Salinity Control and Reclamation Projects (SCARPs), Mardan SCARP and Fourth Drainage Project Faisalabad etc. The Provisional Irrigation Department (PID) undertakes construction, operation and maintenance of irrigation facilities extending from barrages downward to main canals and to canal outlets feeding the main watercourses.

The release of water from dams and barrages to the main irrigation system is monitored, assessed and controlled by Indus River System Authority (IRSA) and received by the provincial irrigation departments for further operation and maintenance. The IRSA also manages to resolve disputes among the provinces pertaining to their shares of water and temporal releases. IRSA was established in 1992 after Water Apportionment Accord of 1991 through an act. It constitutes 5 members – one representative of the federal government and four other, representing one from each province. The primary function of IRSA is to monitor, regulate and distribute the available river water resources of the country among the provinces. Monitoring of water resources withdrawn by each canal system is done through "telemetry system". The data collected through "telemetry system" is transferred to central unit of IRSA for analysis. Each province prepares its indents for release and sends it to IRSA. The statements of withdrawals are also prepared and verified by the provinces. The other federal level institutions dealing with water sector include Planning Commission of Pakistan and Federal Flood Commission.

1.7.2. Provincial Level Management

(a) Provincial Irrigation Department

The Provincial Irrigation Departments exercise their management over the irrigation system below dams; comprising barrages, head works, main canals, distributaries, minor and main watercourses. Once the water is delivered to a given farm or farm outlet at specified location, the farmer has complete autonomy of use wherever he chooses to apply the available water to any of his fields /crops. The On-Farm irrigation system below canal outlet comprising main watercourses, is managed by the Provincial Irrigation Department (PID) but constructed and maintained by the cultivators or shareholders of the command. The field watercourses or farmers' branches are constructed and maintained by the farmers themselves. Usually, the size of a field watercourses is the same as that of main watercourse because the field watercourses have to accommodate the same flow rate as that delivered by the main watercourses.

The PID also administers water distribution and resolution of conflicts among water users. Water distribution among the share holder/ water users is implemented through Warabandi schedule issued by the PID that predefines the location and time of each user's turn. Whenever a distributary is in operation, a watercourse continues receiving an authorized share of discharge at almost constant rate round the clock depending on the flow in the distributary. However, the maintenance condition of the watercourse may alter the received flow rate. A poorly maintained watercourse results in reduced discharge at the outlet. A properly cleaned and maintained watercourse is in the interest of the farmers. The farmers receive water in accordance with a predetermined water distribution system on weekly or 10.5 days rotation cycle.

Punjab's irrigation system consists of 22 main canal systems and 13 barrages/head works. The aggregate length of canals is 23,000 miles with off-taking capacity of 1,30,000 cusecs. It serves 26.78 ma of culture-able land with the existing average cropping intensity of 130%. The river supplies are distributed to agricultural fields through a network of main canals, branch canals, distributaries, minors and watercourses.

(b) Provincial Irrigation and Drainage Authorities (PIDA's)

Historically the irrigation system in Pakistan has been managed by the Provincial Irrigation & Power Departments (PIPDs) of the Punjab, Sindh, KPK and Balochistan under the existing rules & regulations. These century old irrigation & drainage systems posed such a large number of operational, maintenance, institutional, financial & management problems that reforms were considered indispensable. The factors responsible for such deterioration as described by PIDA, 2007 included:

- Overall deterioration of system management.
- General lack of agency responsiveness.
- Inequitable distribution of irrigation water.
- Lower irrigation efficiency.
- Escalating gap between revenues & expenditures.
- Environmental degradation.
- Lack of farmers' participation in decision making & management.
- Lack of farmers' trust in the operation of irrigation system.

In order to overcome the problems mentioned above and to ensure participation of the water users, the Provincial Irrigation & Drainage Authorities were established in all the provinces of country which were named as:

1) PIDA (Punjab Irrigation & Drainage Authority)
2) SIDA (Sindh Irrigation & Drainage Authority)
3) NIDA (KPK / KPK Irrigation & Drainage Authority)
4) BIDA (Balouchistan Irrigation & Drainage Authority)

The reforms involve management transfer of irrigation & drainage systems to PIDAs assisted by the Area Water Boards (AWB) & Farmer Organizations (FO's).

(c) Punjab Irrigation and Drainage Authority

The Government of the Punjab established PIDA as autonomous body under the Act 1997 to take over the functions of the irrigation department pertaining to irrigation, drainage & flood control. Under this act, PIDA has been vested with control over all rivers, canals, drainages, streams, hill torrents, springs, and reservoirs & groundwater resources of the province. Consequently, PIDA has been assigned the following responsibilities and functions to perform:

- To perform all the duties & functions of the Punjab Irrigation Department.
- To plan, design, construct, operate & maintain the irrigation, drainage & flood control infrastructure located within the territorial jurisdiction of PIDA.
- Effective & efficient utilization of the irrigation water & its disposal.

- To introduce the concept of participatory management through the pilot AWB & FO`s and to adopt & implement policies aimed at promoting growth & development of FOs & monitoring of their planning and performance.
- To take measures for reducing operation and maintenance (O&M) expenditure & for improving maintenance activities.
- To take measures to improve assessment & collection of water rates & drainage charge.
- To promote economy in the use of water.
- To make the authority financially self-sustaining regarding the O&M cost of canal irrigation and drainage within a period of 7 to 10 years.

It is expected that ultimately, PIDA would be responsible for regulating the functions of AWBs & FOs including the regulation of canal system all over the province in accordance with the PIDA act. It will also be responsible to manage its own finances including the collection of charges from the clients such as the Govt. of Punjab for flood control & other public services, corporations, district councils and from AWBs; for negotiating transfer payments & subsidies from the Govt. of Punjab. The PIDA comprises the following management organizations:

(i) Area Water Boards

The Area Water Boards are formed to assume the responsibilities of managing and progressively financing the O&M cost of irrigation and drainage network within its jurisdiction. The AWBs are responsible for management of the canal commands & its branch systems from Barrage to District head drainage & flood control infrastructure. As provided in the act 1997, the AWBs are expected to perform the following functions:

- Approve & monitor the operation & maintenance work plan of FOs.
- Recommend the schemes for annual development programs.
- Approve rotational program of the water distribution.
- Check against the water thefts & other offences.
- Promote economy of water use under irrigation laws and to ensure equitable distribution of water & resolve disputes.
- Supervise assessment of the water rates.
- Monitor expenditures and budget allocations.
- Encourage participation of water users in irrigation management.
- Assist the authority & the Govt. in the formation, promotion & development of FOs & monitor their works.
- Maintain an inventory of FOs, WUAs, distributaries, drainage systems and other assets in the area of operation.

In Punjab, the first AWB was set up on the Lower Chenab Canal East (LCC East) circle Faisalabad that commanded about 1.6 ma. The second AWB has been established at lower Bari Doab Canal (LBDC) circle Sahiwal that commands about 0.07 ma of land.

(ii) Farmer Organizations

The management of the system at the distributary level is the responsibility of Farmer Organizations, which are formed, owned & controlled by the farmers. Thus, FOs are responsible for managing the distributaries, minors & subdivisions that fall into branch drains. It is also responsible to manage fresh groundwater tubewells, on- farm tile drains & off- farm sub-drains. The membership of the FOs is available to all farmers that are not gender specific. The primary functions of Farmer Organizations are as under:

- To manage, operate & maintain the irrigation infrastructure including any hydraulic structures located on it, for which it has been established.
- To obtain irrigation water supplies from the Authority or relevant Area Water Board & its head regulator and consequently, pay the agreed amount to the AWB concerned or the authority in the manner agreed between the FO's & AWB concerned or between FO's concerned & the authority.
- To supply the irrigation water equitably to the farmers & other water users within its area.
- To assess and collect he water rates & other irrigation charges to be collected from the water users.
- To levy upon & collect charges for additional services rendered by the FO's.
- To collect surcharge from water users & drainage beneficiaries in case of default in payment of their dues.
- To settle water disputes relating to farmer or other water users of the area.
- FOs are constituted among the elected farmer organizations at disributary level.
- FOs will exercise the powers of the Divisional Canal Offices (Ex. Engineer) under canal & drainage act 1873.

(iii) Composition of PIDA Management:

One of the purposes of the PIDA is to de-centralize powers vested in the officers at district & provincial levels & to involve directly the irrigators through participatory management for operation & maintenance of irrigation & drainage infrastructure. Finally, this will lead to an autonomous body having complete autonomy of revenue collection & its spending with proper accountability. The composition of the authority comprises the Minister of Irrigation Dept., Chairman Planning &Development, Secretary Finance, Secretary Irrigation&Planning, Secretary Agriculture, Managing Director PIDA & 6 farmer representatives.

1.7.3. Canal System Regulation

The canal water supplies are managed through conjunctive operation of link canals, reservoirs, barrages, headworks and canals depending on the river flow and time based share of each canal system. Diversion to various canal systems is based on the agreed accords, water rights and the seasonal needs. The historic water rights of a canal were established on the first come first serve basis. The water supply to the systems built later, were limited to what was left in the river after meeting the demand of canals built earlier.

The quantity of water released to any canal is according to the estimates of Director Regulations of PIPD on 10 daily basis for whole of the Rabi and Kharif seasons in advance. The water distribution schedule is then sent to the Executive Engineer located at the Barrage. The Executive Engineer estimates the daily requirements of the canal command by adding up the indents of distributaries submitted by the Sub Divisional Officer (SDO), which are in turn prepared by adding authorized full discharges of all the off taking distributaries considering the seepage losses taking place in the main or branch canals of the sub division duly sanctioned by the Superintending Engineer (SE). The indents submitted by the SDOs are based on the estimates considering the velocity in the main canal @ 4.8 km/h and in a branch canal @3.2 km/h. Thus, the indents specify both the gauge and discharge at the indenting site. Keeping in view the indents of canals and probable availability of water, the Executive Engineer sends the daily indents to the director regulations. Further, the director attempts to run the canals according to the indents. However, the actual releases depend on the available water supplies that may be less or more than the demand. The water rights are so rigidly followed that they cannot be transferred to another canal even if a canal system or a part of this does not need water.

The irrigation canals in Pakistan are designed on the basis of regime theory having minimum control over structures with limited flexibility in discharge rates. The canals are run on continuous basis supplying water to diversion structures depending on the availability of water in the system. Therefore, each outlet of the distributary/ minor continuously receives a flow rate according to its capacity. However, the tail reach outlets receive less than its rated discharge as compared to head reach outlets because of upstream diversions and seepage losses, which cause inequitable distribution among the outlets. These diversions are however, not based on crop water demand.

1.7.4. Canal Water Allocation / Water Allowance

The water allowance (WA) for a given canal is the number of cusecs required at the outlet to irrigate one thousand acres of land. The water allowance for early canal system was designed keeping in view the average values of cropping intensity and the area to be irrigated (Ahmad and Chaudhry 1988). On the average a canal command in the Punjab province receives about 3 cusecs of water per 1000 acres. In general, these design water allowances were low and the water supplies were not enough to satisfy the crop water requirements. Consequently, the salinity problems in these commands emerged. In some of the command areas where designed water allowance was higher than the crop water requirements, it resulted in waterlogging and salinity problems. The examples include D.G. Khan, Muzaffar Garh canal where water allowance is 7.8 cusecs/1000 acres and Thatta canal with a water allowance of 12 cusecs/ 1000 acres. Thus, excessive allocation of water tends to encourage waterlogging and lower allocation of water results in salinity problems, both reducing the production potential of land. In addition, the inequity of water allocation among various commands also caused low production opportunities, wastage of water and disputes leading to a vicious circle among the provinces.

Table-1.6 Water Allowance for Major Canals in Pakistan

Canal	WA (Cusecs per 1000 acres)	Canal	WA (Cusecs per 1000 acres)	Canal	WA
Central Bari Doab	3.22	Sidhnai	3.00	Lower Chenab	3.17
Haveli	3.00	Upper Chenab	2.73	Fordwah	3.60
Lower Jhelum	2.84	Pakpattan	3.60	Lower Bari Doab	3.00
Upper Jhelum	3.03	Panjnad	4.20	CRBC	8.00
D.G. Khan	6.36	Muzaffargarh	7.80	Lower Swat	14.00
Bahawal	4.25	Punjnad	4.20	Thatta	12.00
Abbasia	4.25	Sadiqia	3.60	Thal	3.18

Source: National Water Policy Draft Final Report (2002)

The Assessment of water allowance (WA) or Outlet Capacity (cusecs / 1000 acres) for the canal outlet utilized the Full Supply Factor (FSF) and Kharif Intensity at a given canal system, which were defined by the following relations:

Kharif Intensity = Area irrigated in Kharif / Total CCA (1.1)

and FSF = Area to be irrigated / Authorized full discharge (1.2)

$$\text{Outlet Capacity} = \frac{1000\ ac \times Kharif\ Intensity}{FSF\ X\ 100} \quad (1.3)$$

$$Outlet\ Capacity = \frac{1000ac \times Kharif\ Intensity \times Full\ Discharge}{Area\ to\ be\ ittigated\ ac \times 100} \quad (1.4)$$

The overall average water allowance in Pakistan has been assessed as 3.49 cusecs/1000 acres, which is much less than the average water allowances for other countries e.g India has average water allowance of 4.86 cusecs/1000 acres, Egypt has 5.31 cusecs/1000 acres and Mexico has 10 cusecs/1000 acres (Tarar 1997). Limited water resources against more water demand situation in the country requires rationalization of water allowance to equitably distribute the water resources among various competing users. The water allowances for some of the canal commands of Pakistan are presented in Table 1.6 that shows a wide variation in water allocation / water allowance ranging from 2.73 cusecs per 1000 acres at Upper Chenab to 14 cusecs per 1000 acres at Lower Swat canals.

1.7.5. Constraints of Existing Water Allocation

Enormous water losses during conveyance and inadequate available irrigation water supplies tend to increase the competition among the users and thereby negatively influence the water allocation. Inequitable distribution of water supply results in water deficiencies in tail reaches of canals, while upper reaches take excess supply. Such a distribution offsets the water allocation for some of the commands. Thus, existing water allocation for different canals, distributaries and watercourse commands not only introduces inequity of distribution in the system but also gives rise to lack of trust among the users.

Inequitable water allocation at watercourse commands also influences the water availability among the shareholders. Earlier water allocations at the Indus Basin irrigation system were based on average annual cropping intensity of 75%. The annual average cropping intensities have now increased to more than 150% (NESPAK 1991). Even at the design intensities, the canals are generally unable to deliver peak monthly crop water requirements. Since, in practice, the intensities are much higher than those used in the design, the water deficits are expected to increase further.

Cropping patterns have also changed from low water requiring crops (barley and sorghum) to higher water demanding crops such as wheat, cotton, rice and sugarcane etc. Increasing area under major crops indicates increasing crop water requirements. Increased use of fertilizers and high yielding varieties accompanied by increasing leaching requirements to reclaim saline soils and relatively lower delivery capability of canals put further constraints on the irrigation water supplies, which need to be rationalized in terms of canal water allowances.

Hafiz (1960) on the basis of lysimeter studies recommended that in order to sustain agricultural production, a water allowance of one cusec for 150 acres as compared to the existing allowance of one cusec for 330 acres is desirable. A water allowance of 3 cusecs was considered as equivalent to an application of 0.55 m depth of water per acre at an outlet command. The lower water allowances had resulted in reduced crop yields and higher salinity, as the quantity of applied water was not sufficient to flush out the salts.

1.8. Warabandi System

Warabandi is a local word in Punjabi language, which means fixing the turn of irrigation water for the share holders or water users of a watercourse command i.e. on-farm water distribution system. It is a rotational method for equitable distribution of the available water supply in an irrigation channels (main watercourse) by turns fixed according to a predetermined schedule specifying the day, time and duration of supply to each irrigator in proportion to the size of his land holding in the outlet command (Singh 1981; Malhotra 1982). Warabandi deals with the water distribution in the last component of irrigation system i.e. watercourse command.

The primary objectives of warabadni system are higher water use efficiency and equity in water distribution. Higher water use efficiency is achieved through the imposition of water scarcity on each farm or shareholder's land. The equity in distribution is attempted through supplying equal share of available water per unit land holding among all users.

In a watercourse command, warabandi proceeds from head to tail. Each farmer receives water in accordance with a predetermined water distribution system on weekly or 10.5 days rotation. The rotation time once started continues even when the distributary ceases to deliver water or there is no water in the watercourse. Thus, each land owner is entitled to receive the entire water in a watercourse in accordance with his land holding for a predetermined period of time (about 16-20 minutes/acre) on a specific week-day and at a specified time including night time. There are two major types of warabandi, i.e. fixed or 'pacca' warabandi and flexible or 'kacha' warabandi as summarized below.

1.8.1. Fixed Rotation or "Pacca Warabandi"

It is developed by the irrigation department and provides for each farmer, his fixed share of water and time of delivery to irrigate his land. The irrigation turn, thus fixed, cannot be altered or exchanged among the farmers. However, in practice the farmers tend to borrow or lend or exchange their irrigation turns to satisfy their field requirements, thus inducing flexibility in the water supplies, which is otherwise not permissible under the existing rules.

1.8.2. Flexible Rotation or "Kacha Warabandi"

In this system, a known amount of water is distributed by the farmers among themselves on the basis of area solely on their mutual agreement without formal involvement of any govt. agency. As long as the water users of the command can amicably and collectively manage the distribution, there is no need for official intervention. However, kacha warabandi system has become increasingly unpopular among majority of the farmers due to unfair exploitation by the influential farmers. Disputes arising in Kacha warabandi are resolved by the irrigation department administration. Although, the kacha warabandi is still in operation as the preferred method in southern Punjab and province of Sindh because this system favors the large land owner in these areas, yet in central Punjab, the majority of the watercourse commands have fixed warabandi, administered by the Provincial Irrigation Department.

1.8.3. Problems and Constraints of Warabandi

The warabandi system has been in operation for about one hundred years but still there are several unsolved problems pertaining to the design and operation as summarized below:

- A major problem is the conveyance loss in the watercourses, which are not accounted for in warabandi system. As the length of watercourse proceeds, the cumulative conveyance loss continues to increase from head to tail. Consequently, the cultivators near the head of watercourse are maximum gainers and those near the tail are the maximum losers.

- Shareholders receive proportionate running time but not necessarily a proportionate quantity of water. Practically nothing has been done to solve this problem.
- The system does not provide flexibility to accommodate farming and crop needs. It is the farmer who has to adjust his cropping pattern and cropped acreage accordingly.
- The canal system originally designed for 75% cropping intensity cannot cater the needs of 100-170% cropping intensity. Therefore, the farmers have to explore alternate water sources such as groundwater even if the water quality is undesirable.
- Decreasing land holdings present more difficulties in implementing improved irrigation practices.
- The cropped area charging system presents a number of operational problems such as the crop raised by using tubewells water is charged as canal water.
- Induces malpractices of ‘Abiana’ (i.e. water charge) assessment and collection.
- Encourages inefficient use of water by over irrigation practices.
- The present system lacks consideration of rainfall or droughts and continues to deliver the same amount of water as scheduled.
- The existing time allocation per acre (approximately 17-20 minutes per acre) is not enough to accomplish irrigation of even one acre field during a given irrigation turn. This forces the farmers to adopt illegal means of water bargaining.
- The water allocation to a farmer is based on the area of the farm and does not consider the location of the farm along the watercourse. Consequently, the water losses occurring during the conveyance creates inequity in the system.

1.8.4. Recommended Improvements in Warabandi System

Based on the research carried out during the past few decades, the following improvements in warabandi have been suggested.

- Equitable water distribution models should be used to develop equitable water allocation among the share holders at a watercourse command.
- The transitional water losses in the watercourses should be assessed and reflected in the distribution roster for proportionate time allocation to each successive water user.
- The filling and draining time as influenced by the watercourse improvements/lining activities should be re-evaluated and reflected in warabandi schedule.

- The cropped-area based water charging system should be replaced by time-area based systems to remove the bureaucratic bottlenecks and to improve the water use efficiency.
- The beneficiaries should be involved in decision making process of water distribution at watercourse command level.
- Increasing the duration of turn cycle from one week to two weeks may enable a farmer to accomplish irrigation of larger fields during a given turn without trading options.
- Induce flexibility of use in the warabandi system by permitting exchange of water turns. This will facilitate the farmers to accomplish irrigation within allocated time.
- Update the century-old warabandi rules to be consistent with the present day needs.

1.8.5. Roster of Turns

The rotation of turn begins at the head farm, i.e. the fields closest to the canal outlet (mogha) usually on Monday at 6:00 am, and the turn is passed on to the next farmer on completion of a farmer's turn. Usually, the rotation of turn for a complete pacca warabandi is of seven days but it may be 10½ days duration at some places. By a shift in turn, once a year, usually in April, the night irrigators become day irrigators and vice versa by rotating the roster by 12 hours.

After the completion of allocated time of a cultivator, the second cultivator located at some distance downstream requires time to fill the empty watercourse between the two diversion points. This time is called Filling Time or 'Bharai'. Therefore, filling time is added to the second and subsequent shareholders at the rate of 5 minutes per 64 meters length of watercourse.

When water reaches the last farmer at the tail, he receives the total stored quantity of water present in the watercourse, in addition to his net allocated time. This water being additional supply is subtracted from his gross allocated time at the rate of 3 minutes per 64 meters length of watercourse. This is called Draining Time or 'Nikas'. After the last farmer has taken his turn of irrigation, the cycle is completed and the turn is shifted to the first farmer at the head to start a new weekly cycle of irrigation turn.

In a weekly rotation, the total Filling Time is debited to the common pool time of 168 hours (one week) and credited to the individual account of each farmer. The discount value of cumulative 'Bharai' time is the 'Nikas' time which is also credited to the common pool. After considering allowance for cumulative 'Bharai' and 'Nikas' times, the flow time for a unit area i.e. the Unit Flow Time (UFT) in hours per acre is given by the equation (5).

$$\text{UFT}\left(\frac{\text{hr}}{\text{acre}}\right)=\frac{168\ (\text{h})-\text{Total fill time (h)}+\text{Total Drain time (h)}}{\text{Total area (acre)}} \qquad (1.5)$$

The UFT thus determined, applies uniformly to the whole watercourse command. This is the basic unit time e.g 17 minutes/acre, which is used to determine Turn Time (TT) allowable to the each farmer.

The Draining Time at a specified rate is computed for the whole length of water course and subtracted from the turn time of the last farmer that had been computed by multiplying UFT with the size of his holding. Draining Time is not permissible to any farmer except the last farmer in the "warabandi" schedule.

The TT for an individual farmer (hours for the land holding) in a weekly rotation cycle is determined using the equation (6).

$$\mathrm{TT\ (hr)} = (\mathrm{UFT} \times \text{land holding}) + \text{filling time} - \text{drainage time} \qquad (1.6)$$

Filling time is generally zero in case of the last farmer and draining time is zero for all the farmers except the last one. Therefore, the filling time and draining time for a given farmer are computed in accordance with the length of run from previous outlet to his outlet and the total length of watercourse, respectively at specified rates as given below.

Filling Time for a farmer = 5 min × Number of acres (linear run) along WC

Draining Time for last farmer = 3 min × No of acres (linear run) along total WC

1.9. Water Charging System "ABIANA"

Water is the most important input factor for crop production. Its major users include agriculture, domestic users, industry and animals. As it is a scarce resource, its optimal use cannot be overlooked. It must be managed to be equitably available to all the competing users for meeting their requirements in optimal manner. Although, it is a natural resource, yet it cannot be made available to the consumers free of cost because of the costs associated with its storage, distribution, flow and utilization facilities for beneficial use. The Indus Basin irrigation system in Pakistan, therefore, involves a large infrastructure comprising, dams, barrages, head works and control structures for managing the surface and sub surface water to its potential users. The cost of the irrigation and drainage infrastructure is enormously high, which cannot be recovered from its users. Since the birth of the irrigation system, a nominal charge is however, levied upon its users by the service providers, which is very small as compared to the cost of the infrastructure. The water charging systems introduced so far are discussed below.

1.9.1. Cropped Area Based Water Charging System

It is a canal water revenue collection system based on the number of acres of a particular crop sown in a given season. It is an older system which has been practiced for about a century since the introduction of the weir controlled irrigation. As the charging was done on the basis of area put under crop, an efficient farmer bringing more area under cultivation was charged more than an inefficient farmer bringing less area under crop with the same quantity of water delivered. The Booking Clerk "Patwari" was the central character of this system who was, though instrumental in collecting field data, yet he was responsible for numerous problems of revenue assessment and collection under this system. Erroneous recording of the type of crop and cropped area against illegal monitory benefits was usually practiced, which was causing financial loss to the Government as well as illegal deals with the farmers. The rates of cropped area based water charges (Rs/ha) prevailing till during 2003 for major crops are given below in Table 1.7. In general, the charges were higher for high water consuming crops and lower for low water consuming crops and no water charge was levied on fallow land.

Table 1.7 Cropped -Area Based Water Rates for Major Crops in Pakistan 2002-03

Crop	Rs/ac	Rs /ha
Garden Approved	139.50	344.5
General Garden and Vegetables	115.20	284.5
Water Mellon and spices	77.50	192.0
Oil Seed	64.30	159.0
Sugarcane	177.20	437.7
Cotton and Tobacco	93.00	229.7
Rice	88.50	218.6
Wheat	59.80	147.7
Maize	53.14	131.3
Millet and Pulses	44.30	109.4
Fodders	37.60	92.9
Forrest	62.00	153.1
Fish Farm (0.04 Cusecs)	581.30	1435.8

1.9.2. Flat Rate Water Charging System

The annual expenditure on management and maintenance of irrigation system in the province of the Punjab had been estimated in 2003 to be over Rs. 5 billion. On the other hand, the revenue collected through water charges remained much below the expenditure, forcing the Government to provide large subsidies to the canal water supplies. Thus, a substantial amount of funds was spent on the collection of water charges. Therefore, an appropriate water charging policy was desired, not only to make the collection of water charges more efficient, but also to recover the cost more effectively and eliminate the problems associated with erroneous recording and reporting of cropped area by the booking clerk / “Patwari”. Thus, on June 10, 2003, the Government of Punjab decided to change the canal water charging policy from Cropped- Area based to Area- based flat rate charging. Thus, it is a recently introduced canal water revenue collection system in which the farmers are charged a fixed amount on seasonal basis per unit of their land regardless of the type and area of crop sown. Under the flat rate system, the water rates for different cropping seasons (Rabi, and Kharif) and types of irrigation supplies regardless of the cropped area are given in Table 1.8, Consequently, water charges were assessed on land holding and “Patwari” was not involved in “Abiana” assessment.

Table 1.8 Flat Rates of Water Charging 2006

Canal Command	Cropping Season/ Type of Irrigation Supplies	Flat Water Charging Rate	
	Type of Irrigated Command	Rs./ac or Rs./Season	Rs./ha
Perennial			
	Winter /Rabi Season Crops	Rs 50/ac	123.5
	Summer / Kharif Season Crops	Rs 85.4/ac	211.0
	Sanctioned gardens	Rs 250/ac	617.5
	Fish Farms of 0.4 cusecs supply	Rs 8500/crop season	
	Fish Farms of 0.04 cusecs supply	Rs 850/ crop season	
NonPerennial			
	Kharif Only	Rs 85 /ac	210
	Sanctioned gardens	Rs 250/ac	617.5
	Fish Farms of 0.4 cusecs supply	Rs 8500/ crop season	
	Fish Farms of 0.04 cusecs supply	Rs 850/ crop season	

1.10. On-Farm Water Management

Following the Indus Waters Treaty (1960) between India and Pakistan, which settled the division of the water of the Indus Basin rivers between the two countries in the 1960s and early 1970s strategies and investment in irrigation and drainage, Pakistan Government focused on the following three activities:

(a) Construction of major storages, barrages and link canals, financed by the IBDF, to compensate for loss of surface supplies to India.

(b) Control of waterlogging and salinity through drainage (public pipe drainage and tubewell drainage projects).

(c) Expanding water supplies by new storages and public tubewell schemes.

During the 1970s, however, the strategic direction of the Government changed to improve and manage the existing water resources because of (a) the high unit costs of additional stored water, (b) the poor performance of the surface irrigation system (especially inadequate maintenance leading to operational problems); and (c) sustainability problems with the public sector tubewell fields.

Through the government's Revised Action Programme for Irrigated Agriculture of 1979 (RAP) the emphasis switched to improving system efficiency through rehabilitation and upgrading, and to small-scale physical investments (private tubewells, watercourse improvements and soil reclamation). Large and long gestation projects for drainage and surface water reservoirs were deferred. The new strategy focused on better utilization of existing infrastructure, especially at the watercourse level, the "saving" of surface water "losses" from channels, and improved management of water from the irrigation command level down to the farm. The four projects evaluated in this report were the first projects, out of ten, supported by the World Bank Group under these new priorities. The projects are the first and second On-farm Water Management projects, the Command Water Management Project and the Irrigation Systems Rehabilitation Project.

Management of a given system generally means to take decisions for improving or using the system to improve its efficiency to its highest possible extent. Water management implies to the management of water where ever it exists whether as surface water, groundwater or rainfall runoff etc. The On-Farm Water Management (OFWM) as understood and practiced in Pakistan means the management of water at the farm below canal outlet, which is a tertiary component of the irrigation system.

1.10.1. Need for On-Farm Water Management

The primary and secondary canal irrigation systems comprising main canals, branch canals, distributaries and minor canals are designed, constructed and managed by the Provincial Irrigation Departments, whereas the construction and maintenance of the tertiary component of the system (i.e. watercourses) is the responsibility of farmers. Because of the inadequacy of knowledge, lack of cooperation and motivation of the farmers, and partly due to ignorance by the provincial irrigation department, the tertiary channels (watercourses) became badly deteriorated, resulting in excessive conveyance and application water losses (Fig.1.15). From management point of view, the tertiary component of the Indus Basin Irrigation system remained poorly maintained because of lack of organization among the beneficiaries, lack of resources and absence of a systematic cleaning and maintenance program.

1.10.2. On-Farm Water Management Projects

In order to assess the performance of the system at farm level, research studies were conducted jointly by the WAPDA and research team from Colorado State University, USA, primarily at MONA Reclamation Project with technical input from the University of Agriculture Faisalabad.

Fig. 1.15 Examples of Unimproved and Poorly Maintained Watercourses

It was found that the water losses in the watercourses (including main watercourses and farmer branches) ranged from 30 to 50%. A high degree of water loss (about 25%) also took place during application of irrigation to the fields (Clyma et al. 1975).

Use of traditional technologies for conveyance and application of water, land leveling and crop production had created a number of system oriented and management problems in the irrigation practices leading to a variety of inefficiencies in the irrigation system. Identification of these problems at the field level and carrying out research during early 1970's to find possible solutions led to the development and implementation of On-Farm Water Management projects (OFWM) in Pakistan. During these field oriented research activities, technologies of watercourse improvement/lining, precision land leveling, and irrigation scheduling were developed to improve the irrigation efficiency and increasing the agricultural production.

Realizing the importance of irrigation water and tremendous water losses during conveyance and application at the farm level, a series of water management projects were developed as given in Table 1.9.

Table 1.9 On-Farm Water Management Projects in Pakistan

Projects	Financial Assistance	Period	Scope
Pilot OFWM	USAID	1976-80	Selected areas of four provinces
OFWM – I	World Bank and IFAD	1981-86	Selected areas of four provinces
OFWM-I	Asian Development Bank (ADB)	1981-88	Thal Punjab and Parpur in KPK.
OFWM –II	World Bank	1986-91	Selected areas of four provinces
OFWM –II	Asian Development Bank (ADB)	1988-93	Dera Ghazi Khan, Punjab
OFWM-III	World Bank/ IDA	1991-96	All four provinces, and FATA
OECF OFWM	OECF Japan	-----	All four provinces, and FATA
National OFWM Program	Government of Pakistan	1997-2007	All four provinces, and FATA

1.10.3. Projects Completed 1976 - 2017

From 1976 to 2017, the Punjab Agriculture Department, Water Management Directorate has completed 47 projects focused at water management at commands of tertiary and secondary components of the irrigation system. Major objectives of these projects were to increase overall irrigation efficiency through increasing delivery efficiency, adopting improved irrigation practices, promoting crop diversification, effective application of non-water inputs, improvement of community watercourses, precision land leveling of farmers' fields, and adoption of advanced irrigation agronomic techniques. The completed projects are listed below:

- Up-Gradation of Water Management Training Institute
- Water Management Spatial Database System
- OFWM-I (World Bank Assisted)
- OFWM-II (World Bank Assisted)
- Second OFWM Project DG Khan (ADB Assisted)
- OFWM-III Project (World Bank Assisted)
- Third Punjab OFWM Project D.G. Khan & Bahawalpur (ADB Assisted)
- OFWM-III (Japan Assisted)
- OFWM Component of Chashma Right Bank Project (CRBCIP) (ADB Assisted)
- OFWM Component of Bahawalpur Rural Development Project (BRDP) (ADB Assisted)
- OFWM Component of National Drainage Program (NDP) (IDA/OECF/ADB Assisted)
- OFWM Component of Punjab Private Sector Groundwater Development Project (IDA/ World Bank Assisted)
- Greater Thal Canal Command Area Development Project Phase-I
- Punjab Irrigated-Agriculture Productivity Improvement Project -Phase I
- Pilot Testing of Solar Water Pumps
- Pilot Project for Promotion of Cotton Cultivation in Thal Region with Drip Irrigation
- Water Conservation and Productivity Enhancement through High Efficiency (Pressurized Irrigation Systems)- The Punjab Component
- National Project to Stimulate the Adaptation of Permanent Raised Bed for Maize, Wheat & Cotton Wheat Farming System in Pakistan. (Punjab Component)
- National Program for Improvement of Watercourses in Pakistan (The Punjab Component)
- Strengthening of LASER Land Leveling Services in Punjab
- Watercourse Improvement (District Government)
- Accelerated Improvement of Watercourses
- OFWM Component of Crop Maximization Program
- Barani Village Development Program Rawalpindi Division (BVDP)
- Improvement of Turbine Tube well and Kaha Sultan Watercourses in Dera Ghazi Khan and Rajanpur Districts
- D.G. Khan Rural Development Program (DGKRDP)
- Drought Emergency Recovery Program (DERA)
- Khushal Pakistan Program (KPP)
- Fordwah Eastern Sadiqia South Drainage Project
- OFWM Second SCARP Transition Project Sheikupura (World Bank Assisted)
- Second Barani Area Development Project Rawalpindi (ADB Assisted)
- Strengthening of Training Program under OFWM Project (World Bank Assisted)

- OFWM Private Tube well Development Project D.G. Khan (World Bank Assisted)
- Command Area Development, Fateh Pur Lift Irrigation Scheme, District Khushab
- OFWM SCARP Khushab Project(ADB Assisted)
- Construction of Watercourses in Pirowali Nankana Sahib, District Sheikhupura
- Provision of Laser Land Leveling Equipment Entire Punjab
- OFWM Gujranwala Project (ADB Assisted)
- Command Water Management Project (OFWM Component U.S. AID/ IDA Assisted)
- Demonstration and Training in Water Lifting Devices
- SCARP Transition Pilot Project, Khan Ka Dogran (IDA Assisted)
- OFWM-IV Drainage Project (World Bank Assisted)
- Thal OFWM Project (ADB Assisted)
- Watercourse Improvement in Sheikhupura (U.K.Grant)
- OFWM, Bahawalpur & R.Y.Khan (Local Funded)
- OFWM Scrap-VI Project R.Y. Khan (IDA Assisted)

1.10.4. On-Going Water Management Projects 2017-18

The water management projects currently implemented in the province of Punjab include:

- Rainwater Management in Cotton Fields to Minimize Impacts of Climate Change.
- Promotion of High Value Agriculture through Provision of Climate Smart Technology Package.
- Punjab Irrigated-Agriculture Productivity Improvement Project (PIPIP Phase II).
- Optimizing Watercourse Conveyance Efficiency through Enhancing Lining Length.
- Provision of LASER Land Levelers to Farmers/Service Providers on Subsidized Cost.

1.10.5. Sources of Water Losses

The sources of water losses in the irrigation system of Pakistan include:

- Seepage through the bed and banks of watercourses.
- Leakage through the cracks and junctions of diversion points.
- Irregular profile and poor alignment of watercourses.
- Mole holes in the watercourses.
- Crossing by animals and human beings.

- Over spilling and breaches in the watercourses and field boundaries.
- Roots of trees and grasses on the watercourse banks.
- Excessive water storage and evaporation losses at junctions.
- Over irrigation due to unlevel fields.
- Leakage through deteriorated control structures.
- Borrowing of soil for closing of water diversions by spade.
- Cutting of watercourse banks by adjoining farmers.
- Excessive irrigation return flow.
- Poor assessment of "How much to apply" and "when to apply" an irrigation.

1.10.6. OFWM Components / Activities

Major activities of OFWM included:

- Watercourse improvement and lining (Examples Fig. 1.16- 1.18)
- Precision Land leveling / Laser Land Leveling
- Irrigation agronomy and Irrigation Scheduling
- Crop Rotation and Crop Inputs
- Land Development and Preparation
- Mechanization of Agriculture
- Development of Storage Tanks
- Sailaba and Rod Kohi irrigation
- Installation of Cooperative Tubewells
- Micro irrigation systems (Sprinkler and Drip Irrigation)
- Hydra Ram pumps
- In-Service Staff Training
- Strengthening of Water User Associations
- Farm Demonstration and Farmer Training Centers
- Formation and Operation of Water User Associations

Fig. 1.16 Renovated but Unlined Watercourse

Fig,1.17 A Precast Slab Lined WC

Fig.1.18 A Brick Lined Rectangular Watercourse

1.10.7. Objectives of Water Management Projects

Although, numerous water management programs were implemented in different provinces of the country from 1976 to 2008 under a variety of management situations, yet the primary objectives of the programs as recorded by the Directorate of Water Management Punjab (2000), included the following:

- To ensure efficient conveyance an application of irrigation water.
- Promote improved On-Farm Water Management practices for increasing agricultural productivity.
- Mobilize farming communities to share investment costs.
- Capacity building for operation and maintenance of irrigatrion facilities.

1.10.8. Functions of Water Management Programs

The Directorate of Water Management Punjab (2000) summarized the following functions:

- Organization of Water User Associations
- Renovation/ Rehabilitation of watercourses
- Promotion of Laser Leveling, Furrow-Bed irrigation and Zero Tillage Technologies
- Installation of Community Tubewells
- Construction of Water Storage Tanks
- Installation of Micro Irrigation schemes i.e. Sprinkler, Drip and Lift Irrigation
- Construction of On-Farm Drainage Schemes
- Implementation of Participatory Irrigation Management (PIM) at distributary level
- Training of manpower in water management

- Coordination with research, extension and field for transfer of technology

1.10.9. Performance of Water Management Programs

Province wise performance in watercourse improvement and lining, from 1976 to 1993, is summarized in Table 1.10

Table 1.10 Province Wise Performance of Water Management Projects in Pakistan (1976 - 1993)

Province	Total Watercourses	Completed Watercurses	Percent Improved
Punjab	55000	14125	26
Sindh	40000	3774	9
KPK	10000	2361	24
Balochistan	2000	686	34
Pakistan	107000	20946	19

Source: National Water Management Program

1.11. Hill Torrents and Rod Kohi Irrigation in Pakistan

Pakistan covers about 80 mha of land, out of which about 40% is suitable for agriculture and forestry. The cultivated land comprises about 20.2 mha, of which 15.3 mha is irrigated through canal irrigation system while the remaining is under rain-fed or "barani" cultivation. Such rain-fed areas receive various degrees of flood flows called hill torrents emerging from different hill ranges. Thus, a hill torrent is a non-perennial stream descending from a hilly area having a very high ratio of normal or minimum flow to its peak, which is of very short duration and generally impregnated with high silt charge. Matloob (2003) defines Hill Torrent as "the rainwater collected from different mountainous/hilly areas through a large number of smaller gorges in a bigger storm that eventually flows in the form of floods to the plains".

Hill Torrents drain upland areas having steep slopes and generally have little base flow, rising rapidly following a rainfall in the catchment area thus, resulting in high peak flows with high silt loads. Runoff water is utilized for flood irrigation through temporary structures with high risk opportunities of success or failure depending upon the magnitude and frequency of rainfall and the strength of structures built for the purpose.

1.11.1.

Conventional water resources of river flow and groundwater are being harnessed to support agriculture. However, water supply from these resources can meet only less than 65% of the requirements in irrigated areas. In barani areas, the situation is more grave than irrigated areas. The water available from rains can meet only one-fifth to one-fourth of the crop requirements. The major exploitable potential source of water in barani areas is hill torrent flow, which offers great prospects for agricultural development. If left unmanaged, a large amount of un-conserved flood flow annually may result in great loss of good quality water and serious damage to the crops, livestock, houses and infrastructure.

The watershed commands for hill torrents in various parts of the country drain about 55% of total area. If properly managed and used, hill torrents can not only cater the agricultural needs of large tracts of barani land, but also can help to safeguard the people’s lives and properties. Harnessing of flow from hill torrents can also play a great role in socio-economic development of the areas.

1.11.2. Characteristics of Hill Torrents

Matloob (2003) summarized the following characteristics of Hill Torrent.

- The hill torrents flow though the natural channels, which usually have no vegetation but steep slopes resulting in silt loaded high velocity flows.
- Hill Torrents have little or no base flow with high peaks, unpredictable and uncontrolled flows leading to difficult management options.
- Hill Torrents bring a large amount of sediments called “Mat” and deposit in the plains and thus, may create a hard layer of stones on the upper surface of land, which may reduce its workability for agriculture.
- Hill torrents often change their course through plain areas and are thus, likely to cause a variety of damages such as human beings, livestock, crops, houses, roads, canal system and electric poles etc.
- Hill torrents are unreliable; therefore, the farmers can’t plan cropping system in time.
- Due to high velocity, the bunds may be washed away, which are needed to be rebuilt before the occurrence of the next hill torrent. Such practices are quite laborious and uneconomical for the farmers.
- The water from the hill torrents cannot be stored in dams/reservoirs due to bigger stones and high silt contents in it. These stones may damage the embankments of dams/reservoirs and along with silt deposition reduces their capacity.
- Hill torrent may create traffic hazards for longer periods of time.
- The farmers can’t cultivate their land for many years due to the erosive actions of hill torrents.
- The hill torrents may compel the inhabitants to migrate to the other places.
- Due to high velocity, the upper fertile soil may be eroded and thus a productive land may be lost.
- Water diversion from Hill Torrents requires heavy diversion structures causing enormous economic and management pressures on the farmers.

1.11.3. Rod Kohi Irrigation

The word “Rod Kohi” is a combination of two words “Rod” meaning stream and “Kohi” means Hill. Therefore, Rod Kohi means stream, which originates from hills. Such a stream is also called Hill Torrent. Thus, Rod Kohi Irrigation is the system under which irrigation is accomplished with the water available from Rod Kohi or Hill Torrent.

Rod Kohi irrigation is practiced in Dera Ismail Khan, Bannu, Karak and Kohat districts of KPK and Dera Ghazi Khan district of the Punjab and Kachhi plains of Balochistan provinces. Some of the famous hill torrents include Kaura, Vehowa, Sanger, Sori Sakhi Sarwar, Vidor, Mithawan, Kaha, Chachar, Pitok and Zangi hill torrents. There is a great potential for improving the rod kohi irrigation system. The Provincial Irrigation and Power Department, particularly in Balochistan, is managing the flood diversions by constructing head works, diversion structures and weirs.

The flows from hill torrents are diverted by constructing temporary dikes “bunds” usually 1 to 2 m high across the bed of the torrent to store and raise water level, which is led into embanked fields. Once a set of embanked fields are filled, water is allowed to percolate slowly into the soil. After these fields are filled, temporary bunds are breached to allow the water to be diverted into the downstream areas to fill another set of fields. This system of diversion continues till the flood flows are completely exhausted or all the fields commanded by the hill torrent are filled. In Punjab, the rod kohi irrigated fields are constructed and maintained through the Kamara system and administered under the Minor Canal Act 1905 by the Torrent Officer. Typical Rod Kohi Channel or Hill Torrent at foot hills and at downstream of hills are shown in Fig. 1.19 a, and b, respectively.

Fig. 1.19a Hill Torrent at Foot Hills of D.G. Khan

Source: Matloob (2003)

Fig. 1.19b Hill Torrent / Rodkohi Stream at Downstream of Hills

Source: Matloob (2003)

1.11.4. Major Hill Torrent Areas

There are a number of hill torrents which have been grouped into various subbasins as given in Table 1.11. Accordingly, various hill torrent areas of Pakistan constitute about 1.50 mha of culturable land and gross potential of 2.34 mha meters. Enormous land and water potential exists in most of the hill torrent areas. Therefore, management of hill torrents can bring vast areas under irrigated agriculture, which are presently lying uncultivated and barren because of unavailability of irrigation water.

1.11.5. Hill Torrent Structures

For harnessing of hill torrents, the following structures may be constructed to suit the local geophysical, as well as traditional practices, water rights and warabandi etc.

- Dispersion structures
- Diversion structures
- Detention dams
- Storage dams
- Channels

The detention dams are useful for the augmentation of groundwater which may be exploited by tubewells. These dams may help recharging aquifers of the area for sustained tubewell water supplies. Excess water from hill torrents can be stored in storage dams during high flood periods. Presently, Manchhar and Kinghar lakes are serving the purpose of such reservoirs.

Table 1.11 Potential of Hill Torrents in Pakistan

Hill Torrent	Catchment Area (km^2)	Annual Culturable Area (mha)	Potential Runoff (mha m)
Indus Basin	10200	0.20	0.52
Makran Coastal area	122507	0.20	0.12
Kharan Desert	122030	0.10	0.05
KPK Hill Torrents			
Kabul River Basin	15015	0.20	0.24
D.I. Khan Bannu and Kohat	25650	0.10	0.26
Hazara	2932	0.10	0.03
Sindh Hill Torrents			
Khirther Range	16058	0.04	0.16
Karachi Area	3056	0.01	0.01
Sehwan and Petaro Area	13200	0.07	0.01
Punjab Hill Torrents			
Pothohar Area	19600	0.10	0.20
DG Khan	23732	0.14	0.32
FATA Hill Torrents	27200	0.24	0.42
Total	493000	1.50	2.34

Channelization of hill torrents may save enormous quantities of flows by preventing overspills and seepage losses. Uncontrolled hill torrents also bring quite large silt load which tend to reduce the channel capacity at downstream reaches. Therefore, provision of various structural improvements is necessary for harvesting the benefits of such flood flows. Fig 1.20 shows an example of the Diversion structure in Rodkohi irrigation system.

1.11.6. Hill Irrigation in Northern Areas of KPK

Hill Irrigation is primarily the contour irrigation practiced in the northern areas of KPK province including Chitral, Swat, Dir and Hazara districrs. Under the Hill Irrigation system, water from the source is diverted into contour channels across the hill off-taking from the source such as streams or springs. These irrigation schemes are smaller in size and operated directly by a group of beneficiaries through social organizations.

The Provincial Irrigation and Power Department of KPK and a number of other organizations such as Agha Khan Rural Support Program, Canadian Aid, Netherland Aid ODA UK have promoted and developed these irrigation schemes to help the farmers of the area. The provincial Government has developed about 27 lift pump schemes in Malakand Agency, Mardan, Peshawar, Bannu, Kohat and Swat districts, which are capable of irrigating about 7800 ha of land.

Hill Irrigation schemes are relatively costlier to develop and maintain than the other irrigated schemes because of the topographic conditions and distant location in remote areas. However, these are used to support high value upland crops such as fruits.

1.11.7. Karez Irrigation in Balochistan

Karezes are the infiltration galleries constructed to tap the groundwater through interlinked dug holes. These are small perennial irrigation schemes in the upland areas of Balochistan province developed from perennial springs. It is a traditional means of tapping groundwater by tunneling into the valley alluvium at a slope, which ends at the seepage face of the hill. Presently, the use of karezes is declining, because of the difficulty in maintenance and declining groundwater levels. During 1980's, the area irrigated by karezes was estimated as 11% of the total irrigated area of Balochistan Province.

1.12. Water Harvesting

Water scarcity is a challenge to the irrigators of arid regions, which necessitates conservation of each drop of water from any water resource that may be subjected to loss of water if not collected and utilized. Water harvesting is, therefore, collection of runoff and its utilization for productive purposes. As the population is increasing, scarcity of water for irrigation, livestock, industry, house hold purposes and human consumption, is becoming a greater challenge for the mankind. Consequently, people are increasingly becoming aware of the importance of water to their survival and consequences of its limited water supply. In arid and semi-arid regions of the world including Pakistan, conservation of water resources has become a priority issue.

Conservation of rain water, hill torrent flows, groundwater pumping and recovery of water losses in conveyance system as well as irrigation return flow, all lead to the harvesting of water. Collection of runoff from roof tops, runoff from local catchments and seasonal floodwater from local streams, are some of the examples of water harvesting. Water, thus harvested, can be utilized for irrigation, drinking water, groundwater recharge and enhancing water supplies for livestock, industry and domestic purposes. At many places, the water collected is just redirected to a deep pit with percolation, which is example of recharging groundwater. The harvested water can be used as drinking water in addition to the purposes given above. Therefore, water harvesting can be undertaken through a variety of ways given below.

- Recovery of runoff from roof tops

- Runoff from local catchments
- Seasonal flood water from local streams
- Conserving water through watershed management

Water harvesting can generally be categorized as (i) Rainwater Harvesting (ii) Floodwater Harvesting as summarized below.

1.12.1.1. Rainwater Harvesting

Rainwater harvesting can be defined as the accumulation of rainwater from surfaces where the rain falls and storing of rainwater at appropriate storage facility for subsequent utilization for useful purposes before it may be wasted, polluted or lost through seepage. Harvested rain water in urban areas can be used for domestic purposed such as garden irrigation, washing clothes and utensils, livestock consumption, and indoor heating for houses etc. In rural areas, it may be accumulated to irrigate crops as shown in Fig. 1.21. The harvesting of rainwater can further be utilized to increase groundwater recharge, reduce storm water runoff, urban floods and overloading of sewage systems in the urban areas. In the coastal areas rainwater can be used as a recharge source to reduce sea water intrusion.

Fig. 1.21 Example of Rainwater Harvesting

1.12.2. Floodwater Harvesting

Flood water harvesting results from accumulation of flood water at appropriate storage point. Runoff may be harvested from roofs, ground surfaces, farms as well as from intermittent or ephemeral watercourses. Flood water harvesting techniques which harvest runoff from roofs or ground surfaces means capturing rain where it falls or capturing the run off in villages, towns or agricultural farms. Storing flood water in the reservoirs of dams, is also an example of flood water harvesting. Under the flood water situations, measures should be taken to keep that water clean by not allowing polluting activities to take place in the catchment.

In general, rainwater harvesting is the activity of direct collection of rainwater. The rainwater collected can be stored for direct use or can be recharged into the groundwater. Rain is the first form of water in the hydrological cycle and therefore, it is a primary source of water. Rivers, lakes and groundwater are all secondary sources of water. In present day activities, we depend entirely on such secondary sources of water.

1.12.3. Advantages of Rainwater Harvesting

- Rainwater harvesting provides an independent source of water supply at the farm or house hold storage facilities.
- The stored rainwater provides water during drought periods.
- It can help in mitigating flooding of low-lying areas.
- It helps to sustain ground water levels if recharged. The underground water reservoirs can be increased, which can be utilized during water stress periods.
- It is a good source of potable water as rainwater is substantially free of salts.
- Soil erosion can be reduced to some extent by harvesting the rain water.
- When collected, rainwater percolates into the soil, forcing salts down and away from the root zone area. Thus, rainwater harvesting can reduce salt accumulation in the soil.
- It allows for greater root growth and water uptake, which increases the drought tolerance of plants.
- The traditional methods of water harvesting can be highly beneficial for sustainable agriculture in rain-fed areas and will help in mitigating the current water scarcity problems.
- The water harvesting practices can boost agriculture production in the rain -fed areas of the country.

1.12.4. Rainfall Harvesting System Capacity

The capacity of a rainwater harvesting system depends on the depth of rainfall received, size of contributing area (roof size), storage capacity, and the demand for water. Table 1.12 shows the gallons of water produced annually from different sizes of roof and depth of rainfall. This may facilitate the readers to assess the annual rainfall yield under a given set of roof area and rainfall amount. The contributing area of a house with sloping roof, is based on the house’s footprint only, neglecting the slope of the roof.

Table1.12 Annual Rainfall Yield for Various Roof Sizes and Rainfall Amounts

Roof Size (ft²)	Rainfall (inches)								
	0.85	1.0	2.0	4.0	5.0	10.0	15.0	20.0	24.0
	Annual Rainfall Yield (Gallons of Water)								
1000	1240	5562	11124	22247	2809	5618	8427	11236	13483
1100	1354	6618	11236	22472	3090	6180	9270	12360	14832
1200	1369	6674	11348	22697	3370	6741	10110	13483	16180
1300	1383	7730	11461	22921	3652	7304	10956	14607	17528
1400	1397	7787	11573	33146	3838	7865	11514	15730	18876
1500	4	843	685	3371	4214	8427	12642	16854	20225
1600	425	899	797	3596	4495	8989	13485	17978	21573
1700	439	955	809	3821	4775	9550	14325	19101	22921
1800	453	1011	821	4046	5057	10113	15171	20225	24270
1900	467	1967	1033	4271	5337	10674	16011	21348	25618
2000	481	1123	1145	4496	5618	11236	16854	22472	26966

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Chapter 2

Measurement of Irrigation Water

This chapter summarizes the importance of water measurement in irrigation system management and efficient use of water for agricultural, domestic and industrial purposes. Units of measurement, devices and methods of measurement and their descriptions supported with solved examples have also been included. It also includes interpretation of collected discharge data to estimate average flow rates, which would further facilitate, the irrigation specialists, agronomists and engineers to assess accurately the amount of water applied to the field.

2.1. Importance of Water Measurement

Accurate measurement of irrigation water permits more efficient use of this valuable natural resource. Systematic water measurements, properly recorded, interpreted and used, constitute the foundation upon which increasing efficiencies of water conveyance, application and use must be based.

Present-day knowledge of soil-moisture-plant relations permits irrigation system to be designed for applying water in correct quantities when needed. The water application rates are based on the soil intake rates, thereby obtaining maximum efficiency of water use and increased crop yields. Water measurement prevents poor crop growth resulting from insufficient water application and reduces chances of water logging problems because of over-irrigation. Therefore, water measurement forms key component of irrigation scheduling and efficient use of water resources in the field.

Limited availability of surface water and increased cost of groundwater pumping require that water be used economically and efficiently. Water measurement is also important component of evaluation of existing irrigation systems and designing new projects. Measured diversion of design flow rates to the main canals, branch canals, distributaries and watercourses, through control structures such as dams, barrages, headwords and canal outlets, not only provides sound basis of water resources planning and judicious distribution but also overcomes any disputes among the farmers. Due to limited scope of the book, however, the water measurement technologies used at dams, barrages, and headworks have not been included. The presented water measurement devices primarily define the principles of operation and estimation of flow rate that may be applied to fields, watercourses, distributaries or higher level measurements. Accuracy of water measurement is of prime importance, in the design and operation of any water distribution system. The utility of flow measurement in the open channels at tertiary component of irrigation system may be summarized as:

- To determine the flow rates available in the irrigation channels
- To evaluate the depth of application of irrigation to a specified field
- To evaluate and improve application efficiency
- To determine conveyance and overall efficiency of irrigation system
- To evaluate water losses during conveyance

US Bureau of Reclamation (2001) explained that good water management requires accurate water measurement and some benefits of water measurement include:

- Accurate accounting and good records help allocate equitable shares of water between competitive uses both on and off the farm.
- Good water measurement practices facilitate accurate and equitable distribution of water within district or farm, resulting in fewer problems and easier operation.
- Accurate water measurement provides the on-farm irrigation decision-maker with the information needed to achieve the best use of the irrigation water applied while typically minimizing negative environmental impacts.
- Installing canal flow measuring structures reduces the need for time-consuming current metering. Without these structures, current metering is frequently needed after making changes of delivery and to make seasonal corrections for changes of boundary resistance caused by weed growths or changes of sectional shape by bank slumping and sediment deposits.
- Instituting accurate and convenient water measurement methods improves the evaluation of seepage losses in unlined channels. Thus, better determinations of the cost benefits of proposed canal and ditch improvements are possible.
- Permanent water measurement devices can also form the basis for future improvements, such as remote flow monitoring and canal operation automation.

2.2. Units of Water Measurements

Water can be measured either at rest or in motion. Water at rest, i.e. in reservoirs, ponds and lakes is measured in units of volume such as liters, cubic meters, cubic feet, acre feet, acre inches, hectare-cm, hectare-m, and second-foot-days etc. Measurement of water in motion, i.e. flowing in rivers, canals, pipelines, field channels and channel structures, is expressed in the units of volume per unit time (flow rate) such as gallons per minute, cusecs, liters per second, cubic meters per second and cubic meters per day etc. (USBR 2001).

2.2.1. Volume Units

Volume of water can be measured in a number of units but the commonly used in the field are defined as follows.

Hectare-meter

A volume of water to cover an area of one hectare (10,000 m^2) to a depth of one meter.

$$1 \text{ hectare-meter} = 10{,}000 \text{ m}^2 \times 1 \text{ m} = 10{,}000 \text{ m}^3 \quad (2.1)$$

Acre-foot

A volume required to cover an area of one acre to a depth of one foot.

$$1 \text{ acre-foot} = 43560 \text{ ft}^2 \times 1 \text{ ft} = 43560 \text{ ft}^3 \quad (2.2)$$

Acre-inch

A volume of water required to cover an area of one acre to a depth of one inch, is defined as one acre-inch.

$$\text{One acre inch} = 43560 \text{ ft}^2 \times 1/12 \text{ ft} = 3630 \text{ ft}^3 \quad (2.3)$$

Second feet day (SFD)

If the water flowing at a rate of one cubic foot per second (CFS) is collected for 24 h, the volume of water will be one SFD.

$$\text{One SFD} = 1 \text{ ft}^3/\text{sec} \times 24 \times 3600 \text{ sec} = 86400 \text{ ft}^3 \quad (2.4)$$

2.2.2. Flow Rate or Discharge Units

Flow rate or discharge can be measured in terms of volume per unit time. There are many units of measuring discharge, but the commonly used ones are given below:

(i) Liters per second (lps): If a volume of one liter of water is passing through a given cross section in one second, the discharge is called one liter per second, commonly abbreviated as lps.

(ii) Cubic feet per second (Cusec): If one cubic foot volume of water is passing though a cross section of channel in one second, the discharge is called one cubic foot per second (1 ft^3/sec) or one cusec, which can be equated in terms of liters per second as:

$$1 \text{ ft}^3/\text{sec} = 28.3 \text{ lps} \quad (2.5)$$

(iii) Cubic meter per second (Cumec): If one cubic meter volume of water is passing through a cross section of channel in each second, the discharge will be one cubic meter per second (1m^3/s) or one cumec. Commonly used conversions in water measurement are summarized in Table 2.1.

Table 2.1 Conversion Factors for Units Used in Irrigation and Drainage

Type of Unit	To Convert		Multiply by
	From	To	
Length Units			
	Miles	Yards	1760
	Yards	Feet	3
	Feet (ft)	Inches (in)	12
	Inches	'Sooter'	8
	Killometers (km)	Meters (m)	1000
	Meters (m)	Centimeters (cm)	100
	Centimeters (cm)	Millimeters (ml)	10
	Meters (m)	Feet (ft)	3.28
	Feet (ft)	Centimeters (cm)	30.47
	Inches (in)	Centimeters (cm)	2.54
	Inches (in)	Millimeters (mm)	25.4
Area Units			
	Acre (ac)	Square feet (ft^2)	43560
	Hectare (ha)	Square meters (m^2)	10,000
	Hectares (ha)	Acres (ac)	2.417
	Square meters (m^2)	Square centimeter (cm^2)	10000
	Square Centimeter (cm^2)	Square inches (m^2)	0.155
Volume Units			
	Cu meter (m^3)	Cu feet (ft^3)	35.315
	Cubic meter (m^3)	Liters (l)	1000
	Liters (l)	Cu feet (ft^3)	0.03531
	Cubic feet (ft^3)	Liters (l)	28.3
	Gallon	Liters (l)	4.5
	Acre-feet	Cubic meters (m^3)	1233.47
	Million acre feet (maf)	Million cubic meters (m m^3)	1233.47
	Million hectare meters (m ha m)	Million acre feet (maf)	8.107
	Cubic foot of water (ft^3)	Pounds (lbs)	62.4
Flow Rate/ Discharge Rate			
	Liters per second (lps)	IMP Gallons per minute (gpm)	13.199
	Liters per second (lps)	Cubic feet per second (ft^3/sec)	0.0353
	Cubic Feet per Second	Liters per Second (lps)	28,3
Energy			
	Watts (w)	Foot Pounds per Second (ft-lb/sec)	0.7376
	Watts (w)	Horse Power (Hp)	0.00134
	Killo Watts (kw)	Horse Power (Hp)	1.341
	Horse Power (Hp)	Killo Watts (kw)	0.746
Pressure			
	1 Atmosphere	Pounds per sq. inch	14.03
	1bar	Centimeter of water	1032

2.3. Methods of Water Measurement

Several methods are available for measuring irrigation water. These can be grouped into four categories:

- Weighing method
- Volumetric method
- Velocity-area method
- Measuring devices (orifices, weirs, flumes etc.)

2.3.1. Weighing Method

Measurement of quantity of water or any fluid can be made in terms of weight of fluid (W) collected in a given time period (t), which can be converted to the volume (V) by dividing with specific gravity (G) or unit weight (μ) at given temperature. For example, in case of water:

$$\text{Volume of water} = \frac{\text{Total weight of water}}{\text{Unit weight of water}} \tag{2.6}$$

OR

$$V_w = W_w / \mu_w \tag{2.7}$$

Where:

V_w = Volume of water

W_w = Weight of water

μ_w = Unit weight of water

$$\text{Flow rate } (Q) = V_w / t \tag{2.8}$$

The volumetric measurement or flow measurements in terms of weight are considered very accurate and can be used in hydraulic laboratories for calibrating other devices and instruments used for measuring flow rate. However, its use is limited to laboratory determinations and is seldom used in the field irrigation measurements.

2.3.2. Volumetric Method

A simple method of measuring small irrigation streams is to collect the flow in a reservoir of known volume for a measured period of time. The time required to fill the container is recorded with a stopwatch. The rate of flow is measured as:

$$\text{Discharge}\left(\frac{\text{liters}}{\text{sec}}\right) = \frac{\text{Volume of reservoir (liters)}}{\text{Total time required to fill (sec)}} \tag{2.9}$$

Where:

Volume of water in reservoir = width × length × depth of water

Volumetric method is the most accurate one for application in field measurement. However, it is cumbersome and usually requires special arrangements of reservoirs to record the width, length and depth etc. that may not be available instantly.

2.3.3. Velocity- Area Method

The rate of flow passing through a section of a pipe or open channel is determined by multiplying the cross sectional area of the flow section at right angle to the direction of flow by the average velocity of water. Methods to measure cross sectional area and velocity in a channel are explained below.

2.3.3.1. Measurement of Cross Sectional Area

For the purpose of irrigation, water may be flowing through open channel or a pipe. The methods of measuring cross sectional area of pipe as well as that of watercourse are explained below.

(a) Cross Sectional Area of Pipe

Pipes are usually circular in shape, therefore the cross sectional area of circular pipe is given by the equation:

$$\text{Cross Sectional Area (A)} = \pi d^2/4 \text{ or } \pi r^2 \qquad (2.10)$$

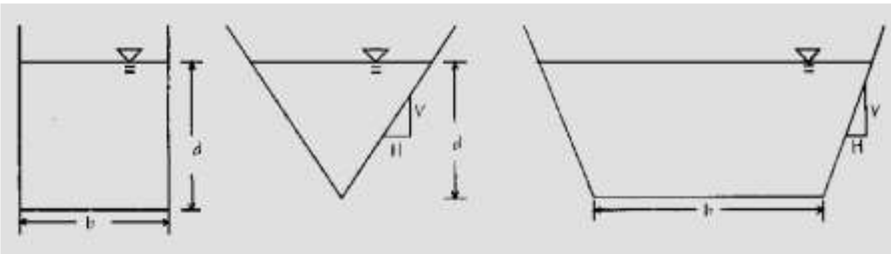
Where: d = internal diameter of pipe and

r = internal radius of pipe

(b) Cross Sectional Area of Open Channel

An open channel conveying water may have a regular or an irregularly shaped cross section. Regular cross sections having fixed shape and dimension are generally found in lined channels where as the unlined channels have irregular sections because they are liable to transition due to forces causing erosion and deposition.

(b1) Regular cross section: The most commonly used regular cross-sections include rectangular, triangular and trapezoidal as shown in Fig. 2.1



Rectangular Triangular Trapezoidal

Fig.2.1 Channel Cross Sectional Shapes

The description of variables in Fig. 2.1 is as under:

b = Bottom width of the cross-section

d = Flow depth at the cross-section

Z= Side slope = Horizontal / Vertical = H / V

B = Top width of triangular or trapezoidal section

A = Cross sectional area as detailed below.

For Rectangular Cross Section A = b d (2.11)

Triangular Cross Section $A = zd^2$ (2.12)

Trapezoidal Cross Section $A = bd + zd^2$ (2.13)

(b2) Irregular Cross Section: There is no fixed bottom width, flow depth and side slope in irregular cross-sections. The cross-sectional area of irregular section can be estimated as explained below:

Method I

Extend a meter rod or a scale or tape across the top of water surface in the channel. Divide and mark the top width in convenient number of sections such as b1, b2, b3, b4 and b5 as shown in Fig. 2.2 that may be numbered as the sections 1, 2, 3, 4 and 5, respectively.

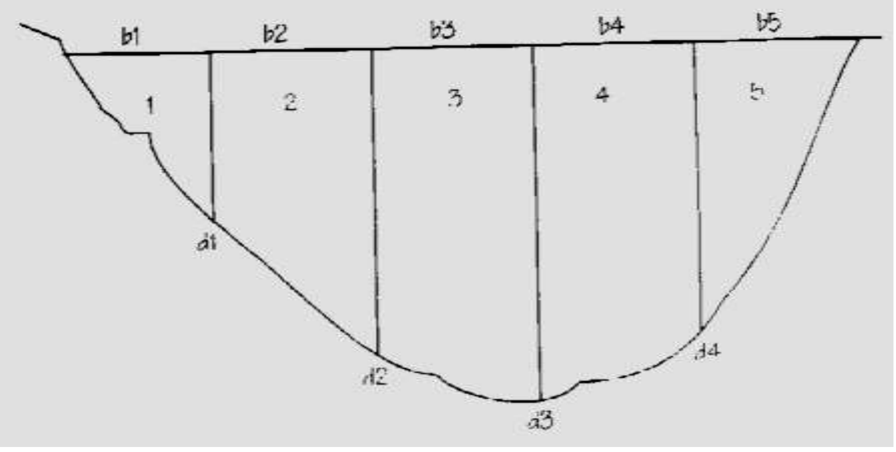


Fig. 2.2 Irregular Cross Section of an Irrigation Channel

Measure the depth of channel for the extended meter rod or tape at marked points of sections b1, b2, b3, b4 and b5, and denote the depths as d1, d2, d3, d4 and d5 with initial depth (d0) and last depth (d5) as zero. The average depth for each section 1,2,3,4 and 5 will be given by D1, D2, D3, D4 and D5. Record the data and apply procedure of estimating the cross sectional area A1, A2, A3, A4 and A5 for each subsection 1,2,3,4,5, respectively, to arrive at the total cross sectional area of the channel by summation as shown in Table 2.2.

Table 2.2 Estimation of Cross Sectional Area of Unlined Channel

Subsection	Top width	Average depth	Area (width × depth)
1	b1	D1 = (d+d1)/2	A1 = (b1 × D1)
2	b2	D2= (d1+d2)/2	A2 = (b2 × D2)
3	b3	D3= (d2+d3)/2	A3 = (b3 × D3)
4	b4	D4= (d3+d4)/2	A4 = (b4 × D4)
5	b5	D5= (d4+d5)/2	A5 = (b5 × D5)

$$\text{Total cross-sectional area (A)} = A1 + A2 + A3 + A4 + A5 \qquad (2.14)$$

Method II

The area of an irregular cross section can also be estimated by plotting the dimensions of 'b' and 'd' in each sub section to some predetermined scale on a graph paper. Count the number of small squares or divisions and multiply with the area represented by each small division to find the area of a cross section as given in Fig. 2.3.

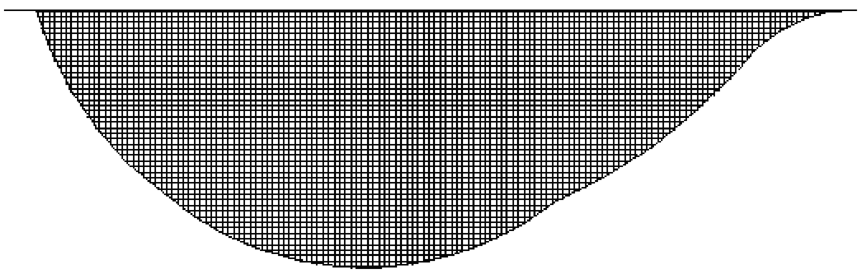


Fig.2.3 Graphical solution of estimating cross sectional area of watercourse

Scale: 1 mm: 0.1 m or 1 cm: 1 m i.e.

1 mm²	(on graph paper)	= 0.01 m² (in the channel)
1 cm²	(on graph paper)	= 1 m² (in the channel)

Note the number of cm^2 on graph paper (as counted on the graph paper). It will give equivalent cross sectional area of the channel in m^2. Smaller the scale used on graph paper, more accurate would be the estimated area.

2.3.3.2. Measurement of Average Velocity

In an open channel, the velocity of water increases from bottom to top of the channel. At the bottom of channel, the velocity approaches zero because of the friction offered by the surface resistance of the channel bed. At the surface open to atmosphere, the velocity may decrease to some extent because of the atmospheric air. Therefore, the maximum velocity in an open channel would exist just below the exposed surface.

Two procedures used to measure velocity under velocity-area method include Float method and Current Meter method depending on the size of open channel as explained below:

(i) Float Method

It is a method of flow measurement, which consists of noting the average rate of movement of a floating body and the average cross-sectional area of channel measured at right angle to the direction of flow. The product of average velocity and average cross sectional area gives the discharge. The procedure followed in this method would be as under:

A straight section of the channel about 15 to 30 m long with uniform cross-section is selected. Measurements of depth and width are made at several locations within the trial section, to arrive at the average cross-sectional area. The average velocity in the channel with float is measured as explained below.

A string is stretched across the section at right angle to the direction of flow. The float is placed a short distance upstream from the trial section. The float must remain up right and $2/3^{rd}$ of body dipped in water. A floating leaf of a ball would not qualify as float because it would tend to overestimate the velocity of water. The time taken by the float to pass from the upper end to the lower end of the selected section, is recorded. Several trials are made to get the average time of travel. To determine the velocity of water, the length of trial section is divided by the average time taken by the float. Since the velocity of water at the surface is greater than the average velocity of the stream, a ball is not used for velocity estimation. It is necessary to correct the observed measurement by multiplying with a constant factor, which is usually assumed to be 0.85.

(ii) Current Meter Method

The velocity of water in a stream or river may be measured directly with a current meter, which may be two general types.

(i) Cup type current meter with vertical axis of rotation (Fig. 2.4)

(ii) Propeller type current meter with horizontal axis (Fig. 2.5).

The cup type meter is more sensitive to disturbances in water. The propeller type current meter has been used for higher ranges of velocities, (i.e. 20-30 m/s) than the cup type currentmeters (10-15 ft/s). For field observations, the cup type has generally been found superior to the propeller type. However, for pipe flow measurement and laboratory observations, the propeller meter has been found to be more useful. The discharge is estimated by multiplying the mean velocity of water by the area of cross-section of the stream.

The cup type current meter is a smaller instrument containing a revolving wheel carrying a set of vanes that is rotated by the movement of water (water currents). It is of two types (a) price current meter (b) pygmy current meter. Both types have similar features except the size of wheel. The pygmy wheel is 2 inches in diameter compared with 5 inches diameter wheel in price meter. The pygmy has no provision for cable suspension. The cup wheel with balancing weight is mounted on a vertical rod with a heavier base that is placed in the channel to measure the velocity of water. The number of revolutions and elapsed time is noted to find the number of clicks of wheel per unit time, which may be read against a calibration curve provided with the meter to measure velocity of water. The use of pygmy meter is limited to velocities up to 3-4 feet per second, which mostly prevail in smaller channels (watercourses). The price current meter is suited to canals and bigger channels having velocities greater than 4 m/s.

Fig.2.4 A Standard cup type current meter

Components of Current Meter

A standard cup type current meter has the following components:

- Vanes to keep the wheel heading towards currents.
- A cable or a rod for handling the meter.
- Weights for sinking the price meter and maintain its stability along currents.
- Electric circuit for signaling the number of revolutions by sounding clicks.
- Round wading base in case of rod.

Use of Current Meter for Flow Measurement

The instrument may be suspended by a cable for measurements in deep streams or attached to an upright rod fixed to a strong base for measurement in shallow streams. The number of revolutions of the wheel in a given time interval is obtained and the corresponding velocity is reckoned from a calibration table or graph of the instrument. Current meter measurements in canals and streams are generally made at metering bridges, cableways, or at other structures giving convenient access to the stream. The channel at the measuring section should be straight, with a fairly regular cross-section.

When the mean velocity of a stream is determined with a current meter, the cross-section of flow channel is divided into a number of sub-sections as shown in Fig. 2.2. Separate measurements are made for each sub-section. The width of sub-section may vary from 1 to 6 m, depending on the size of the stream and the precision desired. Greater the number of sub-sections, greater would be the precision of area as well as of discharge measurement. Not more than 10% discharge should occur in one section. It has been found that the average of the readings, taken at 0.2 and 0.8 m of the depth below the surface of water, is an accurate estimate of the mean velocity in a vertical plane. For sub-sections, at the ends of the cross section (triangular sections) such as (1) and (5) in Fig. 2.2, the average velocity in stream, not over 1.5 feet in depth, is at about 0.6 of the depth from the surface. In streams over 1.5 feet in depth, the average velocity is represented by the average of velocities at 0.2 and 0.8 of depth (Israelsen and Hansen 1990). Only one measurement of velocity at 0.6 of depth is sufficient. The discharge for each sub-area is measured by multiplying the cross-sectional area and velocity of each sub-area. The total discharge is obtained by summing over the discharges of all sub-areas for the given channel cross section.

Another method of the determining average velocity in a stream is the integration method in which current meter is raised and lowered at the constant rate from bottom to the top of the stream. As the water velocity varies with depth, each depth will influence the resulting number of revolutions differently in a given period of time.

Fig. 2.5 Propeller type current meter

2.3.4. Measurement of Discharge

Discharge or flow rate can be determined by multiplying the velocity of water with the cross sectional area of the channel as given in Equation 2.15.

$$Q = A \times V \tag{2.15}$$

Where:

Q = discharge rate, m^3/sec

A = area of cross-section, m^2

V = velocity of flow, m/sec

Thus, the discharge through the first section is

$$Q_1 = A_1V_1 \quad (2.16)$$

Similarly, at section 2, 3, 4 and 5, the discharge Q is given by the equation:

$$Q_2 = A_2V_2 \quad Q_3 = A_3V_3 \quad Q_4 = A_4V_4 \text{ and } Q_5 = A_5V_5$$

In case, more than one channels are contributing to the major channel, the total discharge of the major channel is given by the equation. Thus, total discharge through the channel is given by the equation:

$$Q = Q_1 + Q_2 + Q_3 + Q_4 \quad (2.17)$$

2.3.5. Flow Measuring Devices / Structures

A control structure or a flow measuring device measures the discharge by establishing a relationship between the flow depth and discharge, which is obtained by changing the flow regime from sub critical to super critical as it passes through critical flow condition. At critical flow condition, there is a unique relationship between flow depth and discharge, which for rectangular channels, can be expressed as follows:

$$q = (gy^3)^{1/2} \quad (2.18)$$

where:

q = discharge per unit width of channel

g = acceleration due to gravity

y = flow depth

All discharge measuring structures in open channels such as flumes and weirs employ this principle of hydraulic control. A weir maintains hydraulic control i.e depth-discharge relationship, by contacting the flow in vertical plane and a flume obtains the control by contacting the flow in horizontal plane. The above given current meter is also considered a flow measuring device.

Modular flow (Free flow)

A flow over a weir or through a flume is modular when it is independent of variation in tail water level, which is in sub-critical state. To achieve this condition, the tail water head i.e. down streams head (h2 or hb) must not rise beyond a certain %age of upstream head (h1 or ha). For weirs and flumes, the minimum required differential head (dh) to operate in modular flow condition, can be expressed as a fraction of the upstream head called Submergence Ratio as defined below:

Submergence Ratio = h_2/h or h_b / h_a (2.19)

Where:

h_1 or h_a = Upstream head

h_2 or h_b = Downstream head

Submergence ratio is the modular limit at which the real discharge deviates maximum by 1% from the discharge calculated by depth-discharge relationship. The submergence ratio of a broad crested weirs and flumes can be as high as 0.95 and for a sharp crested weirs and throatless flumes, it will be lowered by 1, e.g for cut-throat flume the modular limit is 0.65. Thus, for a cut throat flume, free flow condition is defined by hb / ha < 0.65 and the submerged flow condition is given by hb / ha > 0.65. Greater the ratio, greater would be the submergence.

In case of sharp crested weir, the tail water level must remain at least 8 cm below the crest level for modular limit and accuracy of discharge. Weirs maintain hydraulic control i.e depth-discharge relationship by contacting the flow in vertical plane and flumes (such as cutthroat flume) obtain the control by contacting the flow either in horizontal plane or in the vertical plane such as BCW flume.

In farm irrigation practices, the most commonly used devices for measuring flow of water are (flow meters for pipe flow) and orifices, weirs and flumes for open channel flow. In addition, specialized methods such as co-ordinate method may be used to measure the discharge of the tubewell and siphons for open channel. The choice among these measuring devices depends on the expected flow rates, available head and specific site conditions. Some of these devices and the procedures of measuring discharge are described below:

2.3.5.1. Orifices

An orifice is an opening, usually round or rectangular in a plate or bulk head, the top of which is well below the upstream water level and through which water flows. An orifice may be circular, rectangular as shown in Fig.2.6.a and 2.6.b, respectively. It may be used for measuring flow from a reservoir, a water course or through a pipe. The main feature of orifice flow is that most of the potential energy of water is converted into kinetic energy of the free jet issued through the orifice.

Under free flow conditions, the downstream water level remains above lower crest of orifice and therefore, discharges entirely into air. Under submerged conditions, the downstream water level is above the lower crest of orifice. Submergence is generally defined as the ratio between the downstream head (hb) and upstream head (ha) across the orifice. The submerged flow will prevail if the downstream head affects the upstream head. The loss of head is equivalent to the difference between the upstream and downstream elevations of water. During free flow condition, the upstream head is not affected by the downstream head. Therefore, for free flow measurement, only the upstream head is needed.

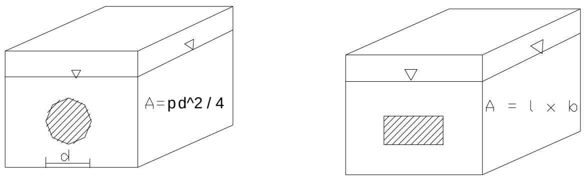
The discharge (Q) through an orifice can also be measured using equation 2.15. The velocity (V) through an orifice can be measured by the equation 2.20.

$$V = \sqrt{2gh} \tag{2.20}$$

Where:

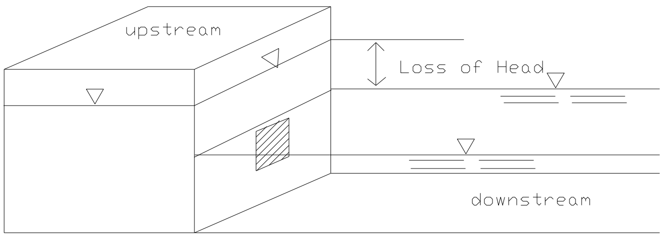
g = Acceleration due to gravity, 9.81 m^2/s

h = Head of water from the center of the orifice

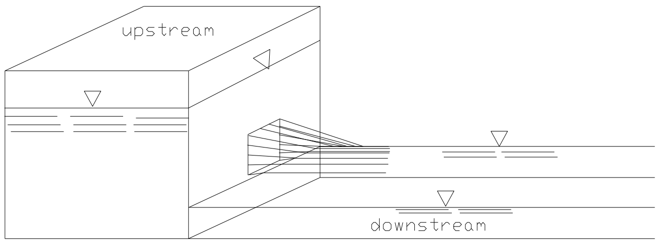


(a) (b)

Fig 2.6 Cross sections of Orifice (a) Circular and (b) Rectangular



(a) Submerged Flow



(b) Free Flow

Fig 2.7 Flow through the Orifice

Thus, for an orifice, the theoretical discharge may be measured as:

$$Q = A\sqrt{2gh} \tag{2.21}$$

Owing to the frictional resistance of orifice, the actual velocity (Va) is less than the theoretical velocity (V). Moreover, due to contraction of discharging jet, the actual area (Aa), is less than the theoretical area (A). Thus,

$$Va = Cv \times V$$

Aa = Cc × A

Qa = Cd × Q

Where

Cv = coefficient of velocity

Cc = Coefficient of contraction

Cd = Coefficient of discharge = Cv × Cc

Thus, considering the friction and jet contraction, the actual discharge (Qa) through a freely flowing small orifice can be measured using the equation 2.22:

$$Q_a = C_d \times A \times \sqrt{2gh} \tag{2.22}$$

The coefficient of discharge ranges from 0.6 to 0.8 depending on the position of orifice in relation to the sides and bottom of the water reservoir or channel and also on the degree of roughness of the edges of orifice. For freely discharging orifices, the head (h) is measured from center of orifice. The head – discharge relation for submerged orifice is given by the equation 2.23:

$$Q_a = C_d \times A \times \sqrt{2g(h_1 - h_{2)}} \tag{2.23}$$

(i) Small Orifice

A small orifice is the one whose dimensions are small in comparison with the head of water above its center, so that at any point in its are the head may be taken as equal to that at its center without significant error.

(ii) Large Orifice

An orifice is considered to be large, if the available head is less than 5 times the height of the orifice. In this case:

$$Q_a = \frac{2}{3} \times C_d \times b \times \sqrt{2g}\sqrt{(h_1^{3/2} - h_2^{3/2})} \tag{2.24}$$

Where

b = width of orifice

h_1 = head from top of orifice to the upstream surface of water

h_2 = head from crest of orifice to the surface of water for free flowing orifice

Well maintained circular orifices can produce discharge accuracies within 1% if properly machined and installed. Rectangular orifice flowing less than full, acts as a weir. The following examples demonstrate the procedure of determining discharge with given set of measurements/data.

Example 2.1

A rectangular orifice 1.5 m wide and 1.0 m deep is discharging water freely from a tank. If the water level in the tank is 3.0 m above the top edge of the orifice and Cd=0.60, find the discharge through the orifice .

Given Data

Width	=	1.5 m
Depth (d)	=	1.0 m
h_1	=	3 m
$h_2 = h_1 + d$	=	3 +1 = 4m
Cd	=	0.6
Height of orifice	=	1m

As the orifice is discharging freely,

Max. available head above center of orifice = 3 + 0.5 = 3.5 m

Required

Discharge (Q) = ?

Solution

Since the available head (3.5 m) is less than 5 times the height of orifice (i.e. 5 × 1 = 5m), the orifice is the large one. Therefore, for large orifice:

$$Q_a = \frac{2}{3} \times C_d \times b \times \sqrt{2g}\sqrt{(h_1^{3/2} - h_2^{3/2})} \qquad (2.25)$$

$$Q = 2/3 \times 0.6 \times 1.5 \times \sqrt{2 \times 9.81}\left((4)^{3/2} - (3)^{3/2}\right)$$

$$Q = 7.45\ m^3/sec.$$

Example 2.2

A rectangular orifice 1.5 m wide and 1 m deep is discharging water freely from a tank. If the water level in the tank is 7 m above the tope edge of the orifice and Cd = 0, find the discharge through the orifice.

Given Data

Maximum available head above the center of orifice is = 7+0.5= 7.5. Since the maximum available head above the bottom of orifice is greater than 5 times the height of orifice (5 ×1 = 5m), the orifice is considered to be a small one.

For small orifice:

$$Q = C_d \times b \times d \times \sqrt{2gh} \qquad (2.26)$$

$$Q = 0.6 \times 1.5 \times 1 \sqrt{2 \times 9.8 \times 7.5}$$

$$Q = 0.6 \times 1.5 \times 2.12$$

$$Q = 10.91\ m^3/s$$

Advantages of orifice

- Cheaper than weirs

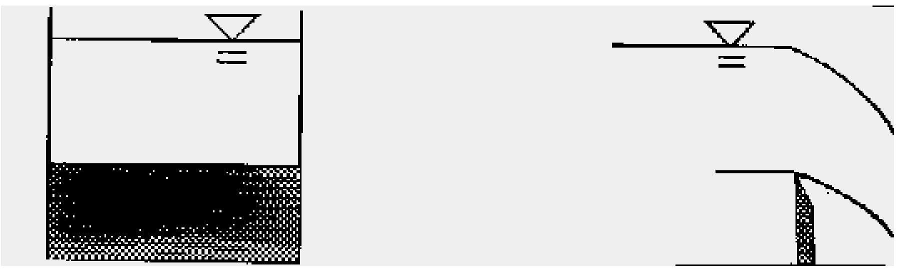
- Submerged orifice requires small loss of head, which makes it suitable for use in areas having low gradient land

Disadvantages of orifice

- The head loss due to friction is higher than that in flumes
- Tends to collect floating debris, sand and silt on the upstream side of the orifice, thus, preventing accurate measurement

2.3.5.2. Weirs

A weir is an obstruction in the channel that causes the water to rise behind the weir and then to flow over it as shown in Fig. 2.8. It maintains the hydraulic control i.e depth-discharge relationship by contacting the flow in vertical plane. Weirs are used to measure the flow in irrigation channels, or canal outlets.



Front View Side View

Fig. 2.8 Sharp crested weir installed in a rectangular channel

Weir Crest

The top of the weir over which water flows is called the crest. The sheet of water flowing over a weir is known as nappe or vein. The depth of water flowing over the weir crest measured at some point in the pond is called head.

Advantages of Weirs

- The advantages of a weir for flow measurement include:
- Greater accuracy of discharge measurement
- Simplicity and ease of construction
- Non-obstruction to floating material
- Durability

Disadvantages of Weirs

- Requirement of considerable loss of head makes their use limited in areas having low topgrahic slope.
- Deposition of gravel, sand and silt upstream of the weir prevents accurate measurement.

Types of Weirs

Weirs are classified on the basis of shape and crest.

i) Classification Based on Shape of Crest

(a) Sharp Crested Weirs

A weir having a sharp edged or crest such that the overflowing sheet of water has the minimum surface contact with the crest (Fig. 2.8). A sharp crested weir consists of a vertical plate with a sharp crest over which the flow take place normal to the direction of plate. The flow contracts in a vertical plane as it passes over the plate. Resultantly, the flow regime changes from sub critical (upstream of weir) to supercritical (downstream of the weir). A properly designed and installed weir is a very accurate discharge measuring device for open channels. Since weirs are placed normal to the direction, the weirs are used for discharged regulations as well as for discharged measurements in the canal irrigation system.

Weirs require larger energy head to permit a given discharge as compared to flumes, and as such the flow depth upstream of a weir is increased. Consequently, the weir should not be installed very close to the turn out as the effect of increasing head may extend to the turn out and reduce the discharge. Weirs are also not favored by the farmers as those require substantial labor for cleaning the sediments that may settle upstream of the weir structure. Lower flow velocity upstream of a weir, may cause the sediments to settle in the channel.

A sharp crested weir is an overflowing controlled structure whose thickness of crest in direction of flow, is equal to or less than 2 mm. The crest surface and sides of notch have plain surfaces, which make 90° intersection with the upstream face of the weir. The downstream edge of weir should be beveled at 45° with the weir surface. The weir plate is thicker than 2 mm.

Sharp crested weir provides opportunity of accurate discharge measurement, if the downstream water level is lower than the weir crest to ensure the ventilation of air pocket beneath the overflow jet. Thus, the sharp crested weir requires a substantial loss of head to obtain free flow which is a major disadvantage. The working head over weir is not measured at the weir but at 3-4 times at upstream in the channel. In case, the approach channel is sufficiently large i.e approach velocity is negligible, and weir is fully contracted, the shape of the approach channel may not influence the discharge. Thus, the fully contracted weir may be used with large non-rectangular approach channels.

Rectangular Sharp Crested Weir

A rectangular notch symmetrically located in a thin metallic plate with a sharp crest, which is placed perpendicular to the flow direction, is defined as Rectangular Sharp Crested Weir. It is classified in following three types:

Contracted Weirs

A weir which has an approach channel whose bed and valves are sufficiently away from the weir crest and sides. Under these condtions, the channel boundries have no significant influence on the contraction of nappie. It doesn't extend the full width of approach channel.

Suppressed Weir

It is a weir, which extends across the full width of a rectangular approach channel. Consequently, channel boundaries may significantly influence the contraction of nappie.

Partially Contracted Weir

A rectangular weir where the bed and walls of approach channel are not significantly away from the weir crest i.e the contraction of which are not fully developed due to proximity of walls and bottom of approach channel.

Triangular Sharp Crested Weir

It comprises a V-shaped notch in a vertical thin plate, which is placed perpendicular to the side and bottom of the straight channel. It is very accurate discharge measuring device suitable for wide range of flow. It is also called Thompson Weir. A triangular sharp crested weir may be partially or fully contracted.

Limits of Application for Triangular Notch

Commonly used sizes of triangular sharp crested weir (fully contracted) are 90, 45 and 25°. The dimension across the top of the weir are twice, equal and half of the vertical depth, respectively.

- Tail water level should remain below the vertex of notch.
- Angle of fully contracted triangular weir may range from 25-100°.
- Partially contracted notches have 90° angle only.

(b) Cippoletti Weir

A cippoliti weir is a fully contacted sharp crested weir with a trapezoidal controlled section. The crest is horizontal and side slopes outward with an inclination to 1:4 side slope (i.e.1 horizontal to 4 vertical). Cippoletti assumed that the decreased discharge over the contracted rectangular weir due to side contraction could be compensated by the increase of discharge because of inclination of the side of the controlled section. It may however, be noted that the accuracy of measurement obtained with Cippoletti weir is not as great as that obtained with rectangular or V-notch sharp crested weir. The discharge measurement equation for a Cippoletti weir is the same as for the rectangular fully contracted weir. The value of coefficient of discharge (C_d) usually equals 0.63.

Application Limits for Cippoletti Weir

- The height of weir crest above the bottom of approach channel should be at least twice the head over the crest with the minimum of 0.3.
- The distance between the sides of trapezoidal control section and the approach channel should be at least twice the head over the crest with a minimum of 0.3.

- The upstream head over the crest of weir (h1) should not be less than 0.06 m.
- To enable the aeration of the nappe, the tail water level should be at least 0.05 m below the crest level.

© Broad Crested Weirs

The weir having a broad crest is called broad crested weir. In this case, the falling nappe is supported by the weir over its crest in the longitudinal direction (Fig. 2.9)

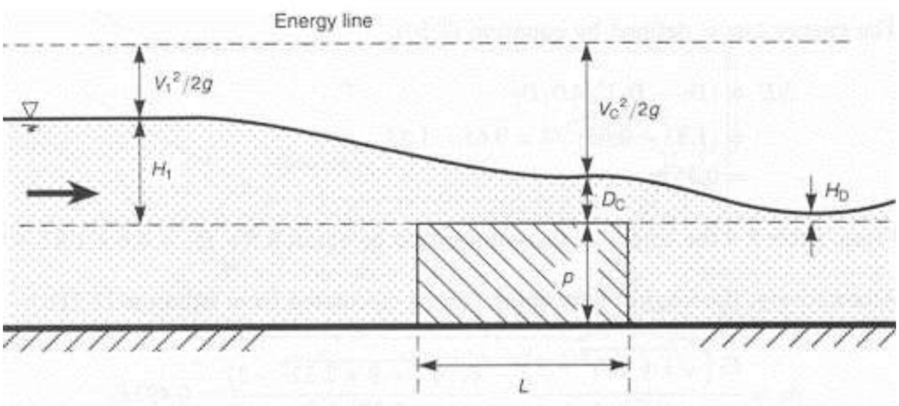


Fig.2.9 Specifications and flow pattern over broad crested wier

Broad crested weirs are generally constructed from reinforced concrete, which usually extend over full width of the channel. They are used to measure the discharge of rivers and canals and are much more suited than the sharp crested weirs. As the discharge measurement is based on the critical depth, the broad crested weir has the advantage that it operates effectively with higher downstream water levels than a sharp crested weir. Only rectangular broad crested weirs will be considered here, although there are a variety of possible shapes: triangular, trapezoidal and round crested all being quite common. However, it needs to be calibrated either in the field by river gauging, that can be accomplished using a propeller type current meter or by means of a scaled-down model in the laboratory.

Head-discharge Relationship

When the length (L), of the crest is greater than about three times the upstream head, the weir is broad enough for the flow to pass through critical depth close to its downstream edge. Applying the continuity equation to the section on the weir crest where the flow is at critical depth gives:

$$Q = Ac \times Vc$$

Now assuming that the breadth of the weir (b) spans over the full width (B) of the channel and that the cross-sectional area of flow is rectangular, then:

$$Ac = b \times Dc \quad \text{and} \quad Vc = (g \times Dc)^{1/2}$$

Thus, the continuity equation can be written as:

$$Q = b \times Dc \times (g \times Dc)^{1/2} \quad (2.27)$$

$$Q = \sqrt{g} \times b \times Dc^{\frac{3}{2}} \quad (2.28)$$

However, equation 1 does not provide a very practical means of calculating Q. It is much easier to use a stilling well located just upstream of the weir to measure the head of water, H1, above the crest than to attempt to measure the critical depth on the crest itself. In order to eliminate Dc from the equation, we can use the fact that in a rectangular channel.

$$Dc = \frac{2}{3} Ec \quad (2.29)$$

Using the weir crest as the datum level and assuming no loss of energy, the specific energy at an upstream section (Section 1) equals that at the critical section. Therfore:

$$\frac{V_1{}^2}{2g} + H_1 = \frac{Vc^2}{2g} + Dc = Ec \quad (2.30)$$

$$Ec = H_1 + \frac{V_1{}^2}{2g} \quad (2.31)$$

$$Dc = \frac{2}{3} Ec \quad (2.32)$$

$$Dc = \frac{2}{3}(H_1 + \frac{V_1{}^2}{2g}) \quad (2.33)$$

Substitute this expression into Equation 2.28, it gives:

$$Q = 1.705 \times b \times (H_1 + \frac{V_1{}^2}{2g})^{\frac{3}{2}} \quad (2.34)$$

The term $\frac{V_1^2}{2g}$ in the above equation is the velocity head of the approaching flow. As with the rectangular sharp crested weir, the problem arises that the velocity of approach, V_1 cannot be calculated until Q is known, and Q cannot be calculated until V_1 is known. A way around this is to involve an iterative procedure, but in practice, it is often found that the velocity head is so small as to be negligible. Alternatively, a coefficient of discharge C, can be introduced into the equation to allow for the velocity of approach, non-parallel streamlines over the crest, and energy losses. C varies between about 1.4 and 2.1 according to the shape of the weir and the discharge, but frequently has a value of about 1.6. Thus:

$$Q = C \times b \times H_1{}^{\frac{3}{2}} \quad (2.35)$$

The broad crested weir will cease to operate according to the above equations, if a backwater from further downstream causes the weir to submerge. Above equations can be applied until the head of water above the crest on the downstream side of the weir, HD, exceeds the critical depth on the crest. This is often expressed as the submergence ratio, HD/H1. The weir will operate satisfactorily up to a submergence ratio of about 0.66, that is when HD = 0.66H1. For sharp crested weirs, the head-discharge relationship becomes inaccurate at a submergence ratio of 0.22. Thus, the broad crested weir has a wider operating range.

Minimum Height of Broad Crested Weir

Minimum height of a broad crested weir (p) can be obtained by applying the energy equation to two sections (Fig. 2.9a). In this case the bottom of the channel is used as the datum level. Assuming that the channel is horizontal over this relatively short distance, that both cross-sectional areas of flow are rectangular and that there is no loss of energy then:

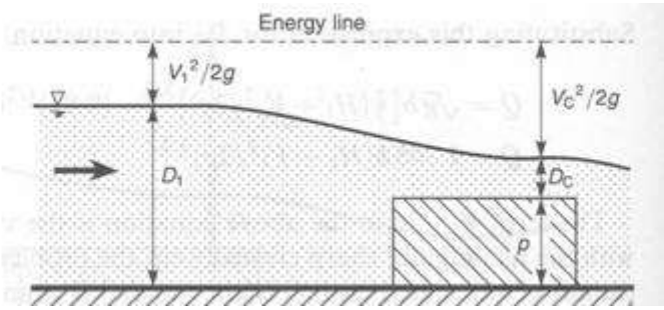


Fig. 2.9a Energy considerations in broad crested weir

$$\frac{V_1^2}{2g} + D_1 = \frac{V_C^2}{2g} + D_C + p \tag{2.36}$$

$$V_1 = \frac{Q}{A_1} \quad D_C = \left(\frac{Q^2}{gB^2}\right)^{\frac{1}{3}} \tag{2.37}$$

and

$$V_C = (gD_C)^{\frac{1}{2}} \tag{2.38}$$

Where:

D_1= Depth of channel upstream of weir

V_1 = Velocity of water in the channel upstream of weir

D_c = Critical depth of flow above the the weir

Vc = Critical velocity above the weir

p = Height of weir above datum

Q = Discharge over the weir

G = Acceleration constant due to gravity

B = Width of channel

Thus, the above given equations 2.36 can be solved for p when Q and D_1 are known. Alternatively, the depth D_1, upstream of the weir can be calculated if Q and p are known. The procedure to find the minimum height of a broad crested weir can be demonstrated by the example 2.3

Example 2.3

Water flows along a rectangular channel at a depth 1.3 m when the discharge is 8.74 m^3/s. The channel width (B) is 5.5 m, the same as the weir (b). Ignoring energy losses, what is the minimum height (p) of a broad crested weir if it is to function with critical depth on the crest?

$$V_1 = Q/A = 8.74 / (1.3 \times 5.5) = 1.222 \text{ m/s}$$

$$D_C = \left(\frac{Q^2}{gB^2}\right)^{\frac{1}{3}} = ((8.74)2/(9.81 \times 5.52))1/3$$

$$= 0.636 \text{ m}$$

$$V_C = (gD_C)^{\frac{1}{2}} = (9.81 \times 0.636)1/2$$

$$= 2.498 \text{ m/s}$$

Substitute these values into the above given equation 2.36 and then solve for p

$$(1.222)^2/19.62 + 1.300 = (2.498)^2/19.62 + 0.636 + p$$

$$0.0761 + 1.300 = 0.318 + 0.636 + p$$

$$p = 0.422 \text{ m}$$

Thus, the weir should have a height of 0.422 m measured from the bed level.

(ii) Classification Based on Length of Crest

Suppressed Weir

When the length of the crest of the weir is the same as the width of the channel, the weir is said to be a suppressed weir. The width of the nappe for suppressed weir is equal to the length of the crest. Thus, in this case, the effect of the sides or ends of the weir on contraction of nappe is eliminated or suppressed (Fig. 2.10). Therefore, suppressed weir is also called weir without end contraction.

Contracted Weir

When the crest length of a weir is less than the width of channels, there will be a lateral contraction of the nappe so that its length is less than the crest length. Such type of weir has end contraction and is known as the contracted weir (Fig. 2.11). The width of nappe is thus reduced by the amount of contraction at each end. Such a weir may be one end contracted or two ends contracted depending on whether one or both ends have reduced width of crest, respectively.

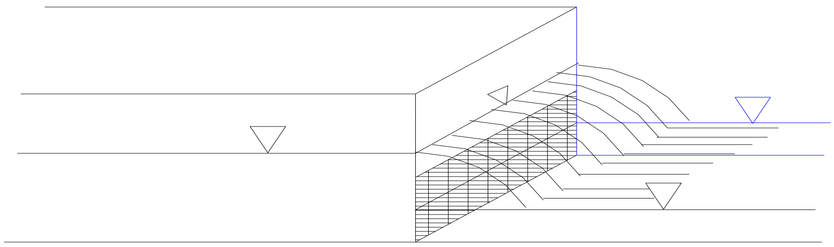


Fig.2.10 Suppressed Rectangular Weir

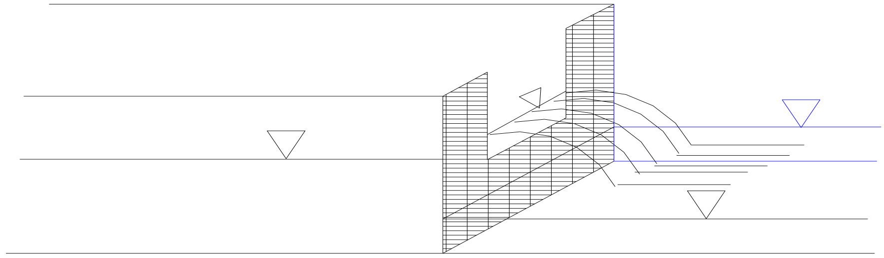


Fig.2.11 Two end contracted weir

iii) Classification based on the shape of weir

(a) Rectangular Weir

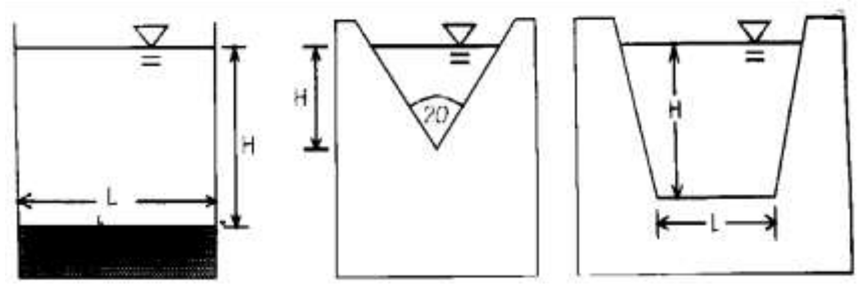
A weir having rectangular cross-section is called rectangular weir. It is used to measure comparatively large discharges. It may be sharp crested or broad crested. It has horizontal crest and vertical sides (Fig. 2.12).

(b) Triangular Weir

A weir having triangular cross-section is called triangular weir. The triangular weir is particularly useful where the discharge is to vary over a large range and the same accuracy is desired for both small and large discharges (Fig. 2.12).

(c) Trapezoidal Weir

A weir having trapezoidal cross-section is called trapezoidal weir and may be regarded as a combination of a rectangular and a triangular weir as shown in Fig. 2.12.



Rectangular Weir Triangular Weir Trapezoidal

Fig.2.12 Types of weir cross section

Discharge Computation

Formulae for discharge computation for various types of weirs are given below.

a) Rectangular Weir

General Formula

$$Q = \frac{2}{3} C_d \sqrt{2g}\, LH^{3/2} \tag{2.39}$$

Where:

Q = Discharge, m^3/sec

L = Crest length, m

H = head over the weir, m

g = acceleration due to gravity, 9.8 m s^{-2}

Cd = coefficient of discharge

Francis Formula

Suppressed Rectangular Weir

$$Q = 0.0184\, LH^{3/2} \tag{2.40}$$

Where:

Q = discharge, liters/sec

L = length of crest, cm

H = head over the weir, cm

Contracted Rectangular Weir

$$Q = 0.0184\,(L - 0.1nH)H^{3/2} \tag{2.41}$$

Where:

n = number of end contractions

b) Triangular Weir

$$Q = (\frac{8}{15}\ C_d \times \sqrt{2g} \tan\theta)\ H^{5/2} \tag{2.42}$$

For θ = 90° in triangular weir, θ/2 = 45°

As tan 45 = 1, eq. 26 can be written as:

$$Q = \frac{8}{15}\ C_d \sqrt{2g}\ H^{5/2} \tag{2.43}$$

Where:

Q = Discharge, m^3/sec

Cd = Coefficient of Discharge

H = head, m

c) Trapezoidal Weir

Discharge through the trapezoidal weir is equal to the sum of the discharges through rectangular and triangular portions, thus

$$Q = \frac{2}{3}\ C_d \sqrt{2g}\ LH^{3/2} + (\frac{8}{15}\ C_d \sqrt{2g} \tan\theta)\ H^{5/2} \tag{2.44}$$

The coefficients of discharge (C_d) involved in the equation (2.44) for the two portions of the weir may be slightly different.

Example 2.4

Water flows over a suppressed rectangular weir 1m wide at a head of 15 cm. Find the discharge through the weir if the coefficient of discharge is 0.62.

L = 1 m

H = 15 cm = 0.15 m

Cd = 0.62

The discharge (Q) is given by the equation 2.45

$$Q = \frac{2}{3}\ C_d \sqrt{2g}\ LH^{3/2} \tag{2.45}$$

Q = 2/3 x 0.62 x $\sqrt{2 \times 9.81} \times 1(0.15)^{\frac{3}{2}}$

Q = 2/3 x 0.62 x 4.429 x 0.058

Q = 0.106 m^3/sec

Example 2.5

Using Francis formula, compute the discharge over a rectangular weir, 45 cm long with a head of 12 cm under the following conditions:

- with no end-contraction
- with one end-contraction

- with two end-contractions

Given:

L = 45 cm

H = 12 cm

Required:

Discharge (Q) = ?

Solution:

i) with no end-contraction

$$\boldsymbol{Q = 0.0184\ LH^{3/2}}$$

= 0.0184 × 45 × (12)3/2

= 0.0184 × 45 × 41.569

= 34.4 liters/sec

ii) with one end-contraction

$$\mathrm{Q = 0.0184\ (L - 0.1nH)H^{3/2}}$$

= 0.0184 (45 – 0.1 x 12) (12)3/2

= 0.0184 (43.8) 41.5569

= 33.5 liters/sec

iii) with two ends-contractions

$$\mathrm{Q = 0.0184\ (L - 0.1nH)H^{3/2}}$$

= 0.0184 (45 – 02 × 12) 123/2

= 0.0184 (42.6) 123/2

Q = 32.6 liters/sec

Example 2.6

A suppressed rectangular weir has a width of 60 cm at its crest. The head of water flowing over the crest is 30 cm. Determine its discharge using Francis formula and compare it with discharge determined by General formula. Assume Cd = 0.63

Given:

L = 60 cm

H = 30 cm

Required:

Q = ?

Solution

i) Using Francis formula

$$Q = 0.0184\ LH^{3/2}$$

$= 0.0184 \times 60 \times (30)1.5$

$= 183.3$ liters/sec

ii) Using general formula

Assuming Cd = 0.63

$$Q = \frac{2}{3}\ C_d\sqrt{2g}\ LH^{3/2} \qquad (2.46)$$

Substituting the values, we get:

$Q = 2/3\ (0.63)\ \sqrt{2 \times 9.81} \times 0.6\ (0.3)1.5$

$Q = 0.1843\ m^3/sec$

Q = 184.3 lps

Hence the discharge computed with both the formulae are comparable.

Example 2.7

Find out the depth of water through the right-angled triangular weir, if the discharge through the weir is 0.106 m^3/sec and coefficient of discharge is 0.59.

Given:

2θ	=	90°	$\theta = 45°$
Q	=	1.06 m^3/sec	
Cd	=	0.59	

Required:

H	=	?

Solution

$$Q = (\frac{8}{15}\ C_d \times \sqrt{2g}\tan\theta)\ H^{5/2}$$

As tan 45° = 1 the formula changes to

$$Q = \frac{8}{15}\ C_d\sqrt{2g}\ H^{5/2}$$

$0.106 = 8/15 \text{ x } 0.59\ \sqrt{2 \times 9.81}\ H^{5/2}$

$H^{5/2} = \frac{0.106 \times 15}{\sqrt{x} \times 9.8 \times 8 \times 0.59}$

$H^{5/2} = 0.076 \qquad H = 0.0762/5$

H = 0.357 m

Example 2.8

A rectangular weir 8 m long is to be built across a rectangular channel to discharge a flow of 9 m^3/sec. If the depth of water on the upstream side of the weir is 2 m, what should be the height of weir. Adopt Cd = 0.62, and neglect end contractions.

Given:

L = 8 m

Q = 9 m^3/sec

d = 2.0 m

Cd = 0.62

Required:

H = ?

Solution

$$Q = \frac{2}{3} C_d \sqrt{2g}\, LH^{3/2}$$

$9 = 2/3 \times 0.62 \times 8 \sqrt{2 \times 9.81}\ H^{3/2}$

$9 = 14.646\ H^{3/2}$

$H^{3/2} = 0.14$

$H = (0.614)^{2/3}$

H = 0.722 m

Height of weir = 2 - 0.722 = 1.277 m

Example 2.9

A 30 m long weir is divided into 10 equal bays by vertical posts each 0.6 m wide. Using Francis formula, calculate the discharge over the weir under an effective head of 1 meter.

Given:

Diagram = 30 m = 3000 cm

L = 1 m = 100 m

With of each post = 0.6 m = 60 cm

Effective length (L) = 3000 – (9 × 60) = 3000 – 540 = 2460 cm

Number of end contractions (n) = 2 × 10 = 20

Required:

Q = ?

Solution

From Equation (13)

$$Q = 0.0184\ (L - 0.1nH)H^{3/2}$$

$$Q = 0.0184\ (2460 - 0.1 \times 20 \times 100)\ (100)^{3/2}$$

$$= (0.0184) \times (2260) \times (1000)$$

$$Q = 41584 \text{ liters/sec or } 41.584 \text{ m}^3\text{/sec}$$

Example 2.10

A trapezoidal weir has a bottom width (sharp crested) of 60 cm and side slope of 45° with horizontal. If the head of water flowing over the crest is 30 cm, determine the discharge if Cd = 0.63.

Given data

L = 60 cm = 0.6 m

θ = 45° or tan θ = 1

H = 30 cm = 0.3 m

Cd = 0.63

Required:

Q = ?

Solution

Discharge (Q) for a trapezoidal weir is given by the equation:

$$Q = \frac{2}{3}\ C_d\sqrt{2g}\ LH^{3/2} + \ (\frac{8}{15}\ C_d\sqrt{2g}\tan\theta)\ H^{5/2}$$

Q = (0.67 × 0.63 × 4.429 × 0.6) × (0.3)1.5 + 0.533 × 0.63 × 4.429 × (0.3)2.5

Q = 1.121 × (0.3)1.5 × 1.48 × (0.3)2.5

Q = 1.21 × 0.164 × 1.48 × 0.049

Q = 0.0134 m^3/sec

2.3.5.3. Flumes

A flume is a specially shaped flow measuring devices that provides a hydraulic control i.e depth-discharge relationship, by contacting the flow in horizontal phase or in vertical phase, and therefore, is used to measure the rate of flow of water in the open channels. There are mainly four types of flumes.

1) Parshall flume
2) Cut throat flume
3) Broad crested weir flume
4) Double throated flume

Characteristics of Flumes

All flumes have a converging followed by a diverging section. In addition, some flumes have a straight section called throat located between these two sections. These include Parshall flume and cut-throat flume. In the Parshall flumes, the slope of the bed is different in the 3 sections, while in the cut-throat flume, the bed slope is horizontal in all the 3 sections. The cut-throat has a zero or no throat width compared to the partial flumes. Flumes are installed parallel to the direction of flow in an open channel. Consequently, the flumes can't be used for discharge regulation. They can be used only for discharge measurement.

a) Parshall Flume

Parshall flume was designed by R. Parshall in 1953 as shown in Fig. 2.13. It comprises a converging section, a throat and a diverging section. The discharge is obtained by measuring the loss in head caused by forcing a stream of water through a converged section and throat of the flume with a depressed bottom. The disadvantages of weirs and submerged orifices are largely overcome by the parshall flume. It operates successfully with less loss of head than that required for weirs. The accuracy of flow measurement is ordinarily within 5% of the real discharge. Flumes ranging from 2.54 to 3.03 cm (1 inch to 10 feet) throat width are used to measure discharge from.038 to 56,60 liters per second (5 gpm to 200 cusecs).

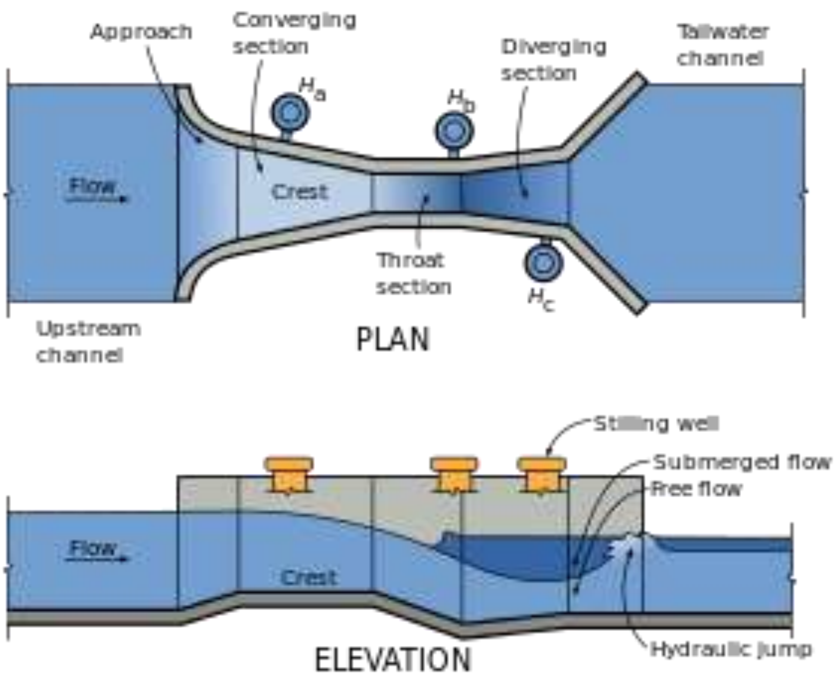


Fig. 2.13 Parshall Flume

The large sizes of flumes have an approach floor and wing walls at the upstream end. The floor of the converging section is level, both longitudinally and transversely. The floor of the throat inclines downward, and the floor of the diverging section slopes upward. The size of a flume is designated by the width of the throat (w) and length (L). The detailed specifications of a typical Parshall Flume are shown in Fig. 2.14.

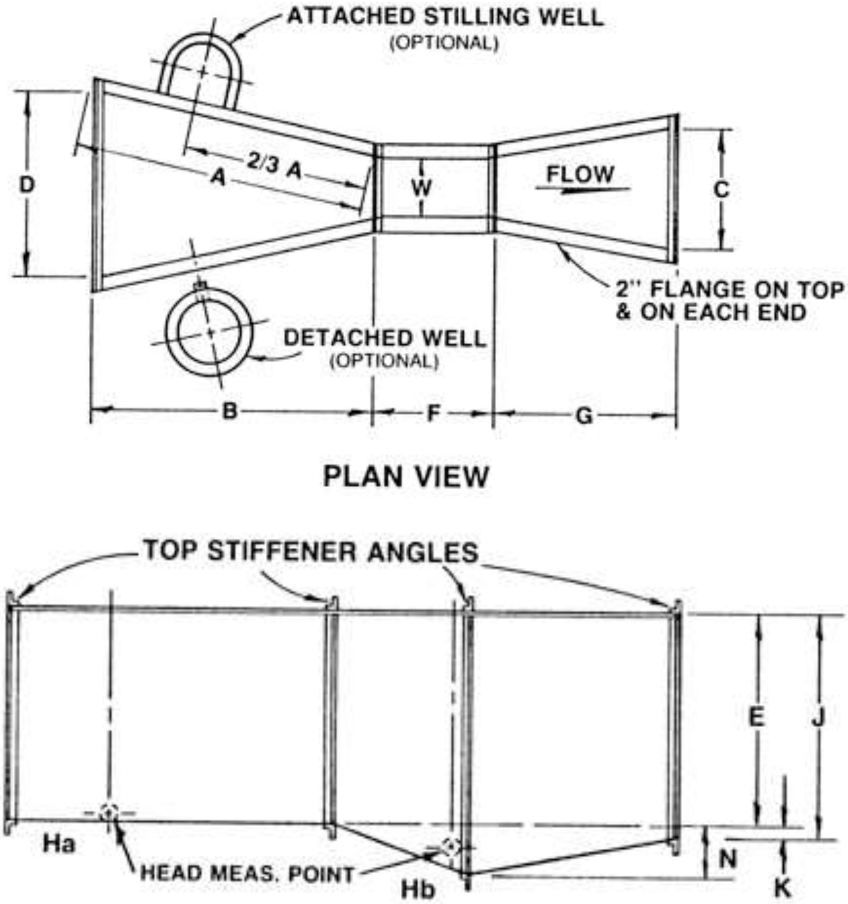


Fig.2.14 Design Specifications of a Parshall Flume

Discharge Measurement with Parshall Flume

Only one head measurement (ha) is required for free flow conditions, which exists when the head at downstream gauge is less than 60% of that at the upstream gauge i.e. Hb/Ha ≤ 60. At a higher ratio of head values (Hb/Ha ≥ 60), the flow is considered submerged. An important characteristic of flumes is less head loss as compared to the higher crested weirs. Submergence limit is defined as that point of submergence at which real discharge deviates by 10% from that indicated by the observed flow.

Fairly accurate measurements can be made with a submergence of 90% with Parshall flume.

Selection of Parshall Flume

The successful operation of the Parshall flume depends largely upon the correct selection of size and proper setting of the flume (free flow or submerged). Whenever possible, the selection and setting should be made such that free conditions may always exist. For economy, the smallest flume that will satisfy the requirements should be selected. Under the existing discharge rates and sizes of the watercourses in Pakistan, the guide lines given in Table 2.3 may help in selecting the right size of flume.

Table 2.3 Guideline for Selection of Parshall Flume

Throat width (inch)	Upper limits of discharge (cfs)
2	0.46
3	1.12
6	2.90
9	5.00

Advantages of Parshall Flume

- The Parshall flume can operate with relatively smaller head loss as compared to weirs.
- It has the capability of making good measurements with no submergence, moderate submergence or even with considerable submergence downstream.
- Its velocity of flow is sufficiently high to virtually eliminate the sediment deposition within the structure during operation.

Disadvantages of Parshall Flume

- The Parshall flumes are usually more expensive than weirs and orifices.
- It is relatively difficult to construct and install as compared to other flumes because of uneven floor.
- They require a solid, water tight foundation.

b) Cut Throat Flume

The cut-throat flume (Fig. 2.15) was developed in 1973 at Colorado State University of United States of America (Skogerboe et al. 1973). As the name indicates, the throat length, existing in Parshall Flume, has been removed in Cut-Throat Flume to give a throat cross section. In addition, the bottom of the flume is flat instead of depressed bottom as in case of Parshall Flume. This flume can successfully measure the discharge in unlined watercourses under both the free and submerged flow conditions with an accuracy of 95%.

Fig. 2.15 A Cut-throat Flume

Design Specifications of Cut Throat Flume

The size of the cut throat flume is specified by the internal width of throat and the total length (including converging and diverging sections). It is available in many sizes such as 4” × 3 ft, 8” × 3ft and 12” × 3ft etc. It comprises a converging section (1 ft long) and a diverging section (2 feet long) with walls at right angle to flat base. The depth of flume varies from 1.5 to 2 feet to accommodate flow depth of the existing watercourses. Vertical scales, at specified locations in each of the converging and diverging sections, are installed to facilitate measurement of upstream head (h_a) and downstream head (h_b). The zero graduation of the head scale should be carefully installed to indicate true inside bottom of the flume. A stilling well is provided outside the flunme at the downstream scale location as well as at the upstream scale location to facilitate accurate measurement of the head, free of wave influence. Detailed design specifications of a cut throat flume are given in Fig. 2.16 (Skogerboe et al. 1973).

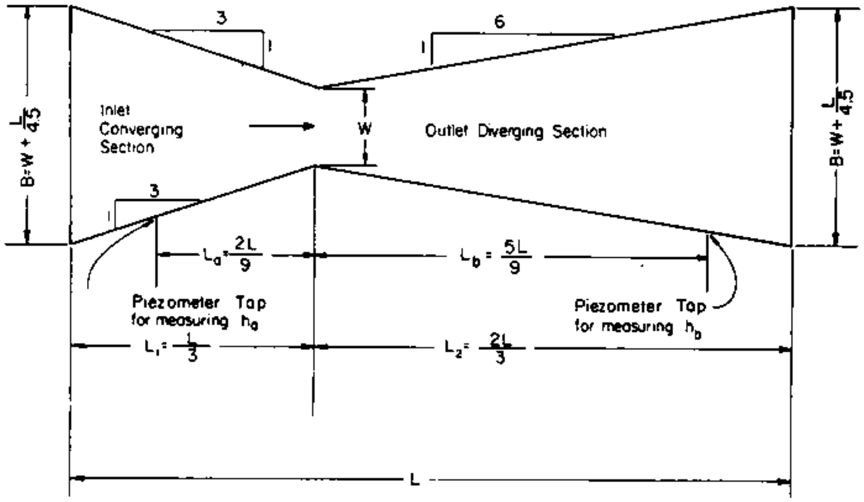


Fig. 2.16 Design Specifications of Cut- Throat Flume

Advantages

The cut throat flume has the following beneficial characteristics.

1) Easy to construct and install because of flat bottom.
2) Higher accuracy of flow measurement ($\geq$ 95%) under both free and submerged flow conditions.
3) Head loss through the flume is lower than other measuring devices such as parhsall flumes and weirs. It is therefore a suitable water measuring device in low gradient channels in the Indus Basin.

Selection of Cut Throat Flume

Discharge measurement with a flume depends mainly on the size of throat. A given flume size can measure discharge of a specified range. Therefore, to measure discharge of a given watercourse, proper size of cut throat flume should be selected. As the flow rate of a watercourse generally ranges from 1 cfs to 4 cfs, a flume size of 8” x 3’ would generally be appropriate. A smaller throated flume may rise the upstream head excessively, resulting in undesirable upstream storage and loss of head and flow rate. A bigger sized flume may not give a measurable head difference and may even lead to excessive submergence. Therefore, selection of appropriate size of the flume is important to accurately measure the discharge.

Judgment of the approximate flow rate to be measured, is required for selection of a flume to meet specific measuring requirement. Discharge Rating Curves or Measurement Tables, can be used to find the range of discharge that a given flume size can measure accurately. However, for convenience, the following Table 2.4 can help in selecting the right size of the cut-throat flume.

Table 2.4 Guidelines for Selection of a Cutthroat Flume

Throat width (inch)	Upper limit of discharge (cfs)
4	3.1
8	6.3
12	9.5
16	12.7

Flume dimensions and angles must be checked thoroughly to select the right flume. Throat size is the most sensitive and important measurement. A flume should be rejected if the throat width varies by more than ±1/16” from standard dimension. The other dimensions may not be allowed to vary by more than ± 1/8”. Check all the welded joints to ensure that there is no bulging, dents or leakage through the device.

Orientation of Cutthroat Flume for leveling

If the flume has to be installed in a channel with silt loaded water flowing through it, the spirt level position may not be visible when placed at the bottom floor of the flume. Therefore, orientation of flume for leveling is required before placing in the flowing watercourse. For orientation, make sure that two longitudinal and transverse locations on the flume top are parallel in horizontal plane with two similar locations on the flume bottom have been fixed. To achieve this, place the wooden level (at least 6” long) in transverse and longitudinal directions on the bottom or floor of the flume, as close to the flume throat as possible, preferably on the converging section of the flume, and bring the bubble to the center level position. Find the same transverse level position somewhere on top of the flume either in the throat region, or on the cross piece at the start of the converging section or on the cross piece at he end of the diverging section. Mark this position for subsequent leveling in flume in flowing water. Likewise, for longitudinally leveling, place the level on top of walls of converging or diverging section of the flume. Wherever the bubble comes in the center, mark that position on the top of the walls.

Always use these two marked positions to level the flume if it has to be installed in the flowing water, otherwise use the floor or bottom of the flume to ensure that the flume is installed in a level position. Check occasionally these predetermined positions with reference to the flume bottom, as the flume may become deformed after a long interval of use. Installation of the flume in a watercourse before water reaches the installation / measurement location, does not need orientation.

Installation of flume and Flow Measurement Procedure

Following steps should be taken to successfully install the flume in the watercourse for discharge measurement.

1) Select a straight section of the watercourse to install the flume. Avoid installation at a turning or diversion point for accurate measurement.

2) Place the flume in the centre of the channel. It should be placed parallel to the direction of flow of water in the channel.
3) Avoid excessive submergence as submergence tends to introduce errors of observation and measurement of discharge. Most of the channels in Pakistan have flat gradient beds with very little free board. With the cut throat flume good discharge measurements can be made even under submerged flow conditions. However, submergence should not excess 90%. Where conditions permit free flow measurement should be encouraged.
4) Level the flume in both the longitudinal and transverse, directions by placing the spirit level at points determined by orientation of leveling.
5) Seal the sides of the flume with soil such that the level of flume is not disturbed. This would direct all the channel flow through the flume.
6) Check the bottom and sides of the flume for any leakage. In case leakage is identified, it should be overcome by placing soil on upstream side.
7) Check to make sure if holes through stilling well are open and there is no mud in stilling well.
8) Soon after installation, the water level in the watercourse on upstream of flume starts rising. Due to upstream storage temporarily, unsteady state of flow occurs. Therefore, before recording reading, wait for some time (about half an hour) till the flow becomes steady.
9) Walk along the watercourse upstream of watercourse to see if there is any overtopping which should be overcome for satisfactory discharge measurement.
10) Record readings of ha and hb in accordance with the submergence condition on observation sheet such as given in the Table 2.5. The observed head values are recorded with an elapsed time of 5, 10, 15, 20, 30, 60 minutes, which may be continued over the desired period of time (e.g. a day) if so desired.
11) Determine submergence (hb / ha). For free flow condition, only ha values are needed to determine the discharge, whereas under submerged conditions values of both ha and hb are needed as shown in Table 2.5.
12) Use appropriate depth discharge table to find the flow rate against observed head values.

Submergence

In flow measurement, submergence is defined by the ratio of downstream head to the upstream head (hb/ha) exceeds 0.60. With cut-throat flume, the flow is considered to be submerged if hb / ha ≥ 0.65. Higher the ratio, greater is the submergence. The submergence primarily shows the effect of change in downstream head to the upstream head.

Free flow

In case the ratio h_b/h_a, is less than 0.65, the flow is considered free. Practically, if ha is not affected by the change in hb, it is considered as free flow. Table 2.5 shows example data recorded during flow measurement with cut-throat flume. Discharge may be found using appropriate flow rating table.

Table 2.5 Data Recording Sheet for Discharge Measurement with Cut-throat Flume

Clock time	h_a(ft)	h_b(ft)	h_b/h_a	Flow condition	Discharge (cusecs)
1000	1.20	0.96	0.80	Submerged 0.8 > 0.65	
1005	1.65	0.99	0.60	Free 0.60<0.65	
1010	1.80	1.00	0.56	Free 0.60<0.65	
1015	1.90	1.02	0.53	Free 0.53<0.65	
1020	2.00	1.05	0.52	Free 0.52<0.65	

c) Broad Crested Weir Flume (BCW flume)

Unlike Parshall and cut-throat flumes, the control section in the broad crested flume is provided by raising the bed of the flume like a weir instead of providing a contraction. The whole length of flume has same width. (Ahmad 1991). The water flows through the flume in the same way as over a broad crested weir.

Interpretation of Discharge Data

As Table 2.5 yields discharge rate against the clock time, these data are plotted on a simple graph as shown in Fig. 2.17. During irrigation process, as the water reaches the flow measuring device, it starts storing behind the device to develop head, enough to cause the flow through the device (at constricted width or over the raised bed of the flume such as a weir). After initiation, the flow rate will increase for a certain period of time till it shows a constant rate with clock time, which is a steady state condition. During steady state, the curve remains horizontal. The area under the curve gives the volume of water released through the flume, whereas the shaded area above the increasing discharge curve, shows the dead storage behind the flume. These volumes (released and stored) can be determined by the graphical method. The flow rate measured by the cut-throat flume is the rate shown by horizontal line on the graph, which is achieved under steady state conditions.

After the flume is removed, the amount of water stored behind the flume is also released. It is, therefore, recommended that the flow measurement must be completed within the warabandi allocated time of the farmer so that he may recover the volume of water detained by the flume. In case, the flume is installed in one farmer's time and removed during other farmer's time, the share of first farmer's water would be received by the subsequent farmer, which would benefit the second farmer at the cost of first farmer's right of turn.

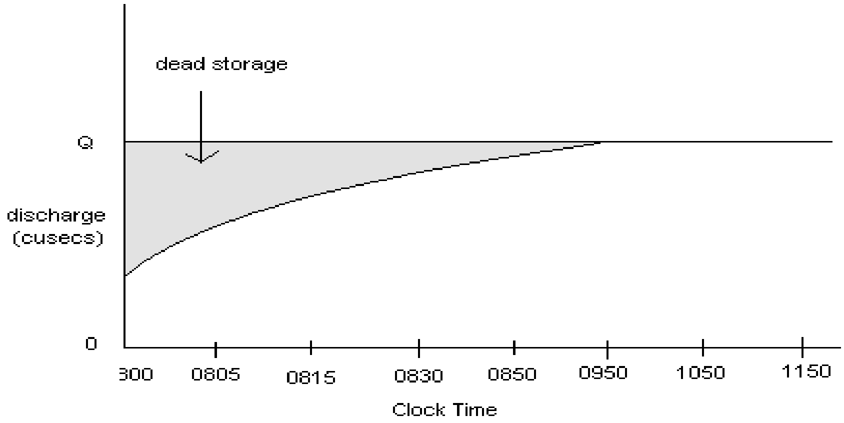


Fig.2.17 Discharge-Time Curve

2.3.5.4. Measuring Tubewell Discharge with Purdue Coordinate Method

This method estimates the tubewell discharge by measuring the fall of water in a given length of a stream of water flowing freely from the end of a round horizontal and straight pipe for a distance of at least 6 times the diameter of the pipe. The method is equally good for both the full as well as partially flowing pipe. The measuring point is the average location at which the horizontal jet touches the vertical ordinate at a pre-specified abscissa (distance from the end of the pipe) as shown in Fig. 2.18. In order to measure the discharge, the following procedure is adopted:

- Check the level of discharge pipe with spirit level. The pipe must be horizontal or at 0% slope.
- Use a carpenter's square or folding rule or a calibrated right angle rule and measure the vertical distance of water jet (cm) from the top of the pipe to the top of falling stream, which is called the Y- coordinate.
- Measure the horizontal distance (cm) of the water jet from beginning of jet to the end of the falling jet, which is called the X-Coordinate.
- Measure the inside diameter of the pipe (cm).

With the above measured values, the discharge of the pipe is determined using equation 2.47 for full flowing pipe.

$$Q = 0.0172\, D^2\, X / Y^{1/2} \qquad (2.47)$$

Where:

Q = Discharge (lps)

D = Inside diameter of pipe (cm)

X = X – Coordinate (cm)

Y = Y- Coordinate (cm)

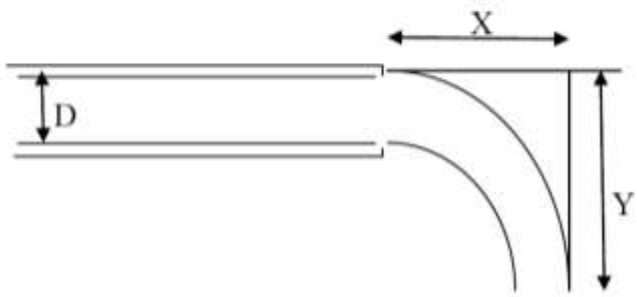


Fig. 2.18 Discharge Measurement by Purdue Coordinate Method

Example 2.11

Refering to Fig. 2.18 if X=30 cm, Y= 36 cm and D = 12.7 cm, using equation 2.47, the discharge (Q) can be calculated as:

$$Q = 0.0172\ (12.7)^2\ (30) / (36)^{1/2}$$

$$Q = 14.0 \text{ lps}$$

References

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Chapter 3

Irrigation Methods

An adequate water supply is important for plant growth. When rainfall is not sufficient, the plants must receive additional water from irrigation for their survival. Various methods can be used to supply irrigation water to the plants. Each method has its advantages, disadvantages and performance efficiency, which should be taken into account when choosing the method of application best suited to the local circumstances.

An irrigation system as a whole comprises four subsystems, namely (i) water supply (ii) water delivery (iii) water application, and (iv) excess water removal from soil (Walker and Skogerboe 1987). For successful operation, each subsystem must perform efficiently. Both the irrigation and drainage are considered complementary to each other. The present chapter on irrigation methods primarily deals with the application and use of irrigation water to the crops.

Irrigation water may be applied to the crops in many ways depending on the source of water supply, soil, topography, crops, climate and practices and preferences of the farmer. Generally, water may be applied by spreading it on the field surface by gravity, beneath the soil surface through perforated pipes, by spraying it under pressure through sprinklers or by applying it drop by drop through drippers. Each of these activities can be accomplished by using different irrigation techniques and associated equipment.

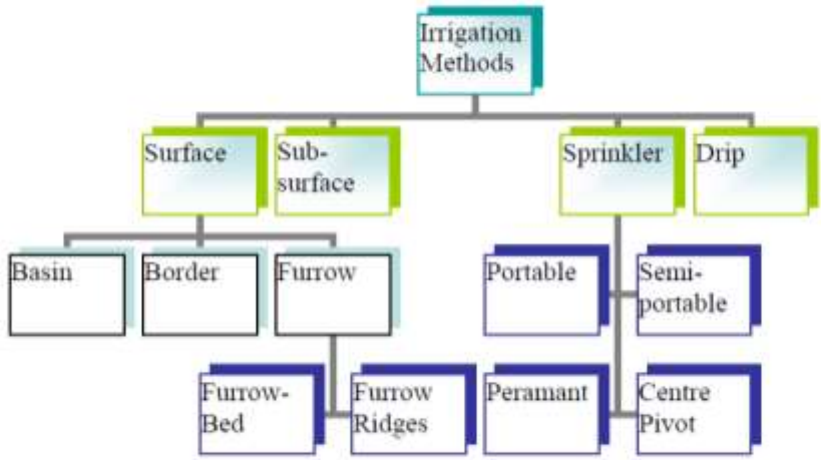


Fig. 3.1 Methods of Irrigation Application

For sustainable irrigated agriculture, it is important that the selected irrigation method provides sufficient control over the uniformity, adequacy and accuracy of application of water to efficiently meet the crop water requirements involving least hazard of salinity and waterlogging. While selecting and designing an irrigation system, ease of application, application of the measured amount at required time with highest efficiency and economic use of irrigation water must be considered. Fig. 3.1 gives the classification of these irrigation methods. Each of these irrigation methods has been described below.

3.1. Surface Irrigation (Gravity Irrigation)

Surface irrigation is the application of water by gravity flow to the surface of the field. In the surface methods of irrigation, water is applied directly to the soil surface from a channel located at the upper reach of the field. The driving force in such irrigation system is gravity and hence alternate name is gravity irrigation or gravity flooding. Two general requirements of prime importance are to obtain high efficiency and uniform distribution of water over the field. Once distributed over the surface of the field, water enters the soil and is redistributed by the forces of gravity and capillary to be available for plant use through root system. In this method, either the entire field is flooded (basin irrigation) or the water is fed into small channels (furrows) or strips of land (borders). Therefore, the surface irrigation is further subdivided into a number of techniques depending on the topography, slope and surface shaping of the field as summarized below.

3.1.1. Basin irrigation

A basin is a flat area of land, level in all directions and surrounded by low bunds, which prevent the water from flowing out to the adjacent fields. This is the most common, simplest and oldest method of controlled irrigation in Pakistan. There have been many variations since its use. However, in general, it involves dividing the field into smaller unit areas so that each has a nearly level surface (Fig 3.2). Bunds or ridges are constructed around the field farming basin within which are constructed around the field forming basin filled to the desired depth can be controlled. The basins are filled to the desired depth and the water is retained until it infiltrates into the soil. When irrigating rice or ponding water for leaching salts from the soil the depth of water may be maintained for considerable period of time by allowing water to continue to flow into the basins.

3.1.1.1. Suitability of Basin Irrigation

Basin irrigation is suited to many different soils but most commonly used for rice grown on flat lands or in terraces on hill sides (Jensen 1980). Trees can also be grown in basins. In general, the basin method is suitable for crops that remain unaffected by standing water for relatively longer periods (e.g. 12-24 h). The most important soil characteristic influencing the design of basins is the water infiltration rate and the size of the available stream, which determine the area that can be enclosed in each basin. Basin may vary in size from 1 square meter used for growing vegetables to as much as ½ acre (or 4 "kanal" area) for the production of rice and other grain crops. One "kanal" equals 20 "merla" or 5440 square feet area.

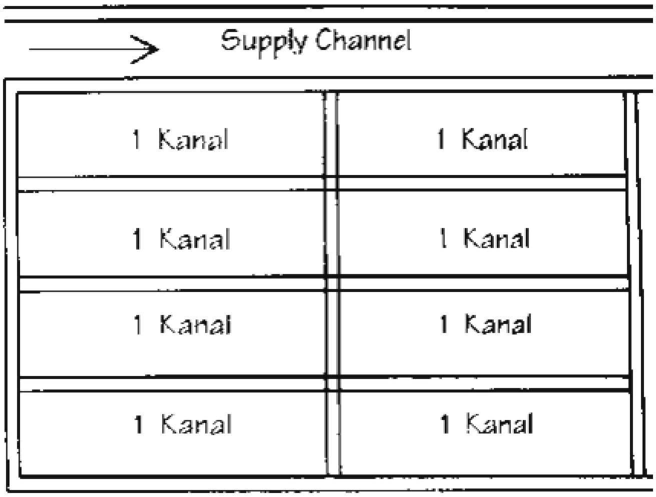


Fig. 3.2 A Typical Basin Irrigation system

In Pakistan, under Basin irrigation, one acre of land is usually divided into two, four or 8 parts to be irrigated by a common channel passing through the center as shown in Fig. 3.2. This is mainly due to smaller sizes of stream available. Sandy soils require smaller basins as compared to the clayey soils. The objective in selecting the basin size is to enable the irrigator to flood the entire area in a reasonable length of time, so that the desired depth of water can be applied with a high degree of uniformity over the entire basin. The basin method of irrigation was constrained primarily by smaller flow rates such as from a dug well in the past. As the flow rates increased by installing tubewells, the basin system has been mostly replaced by the border irrigation system as explained below. Rice, cotton, small grain, maize, groundnuts, gram, lucern, pasture and many other field crops are suited to this system of irrigation. It is seldom used for crops which are sensitive to wet soil conditions around the stem. The primary requirement in basin irrigation is a field level in both directions so that irrigation water applied to basin must be stored in filtrate into the soil.

3.1.1.2. Advantages of Basin Irrigation

Adequate control of water, uniform distribution, high application efficiency with proper design and operation is obtained by this method. It is useful for leaching of excess salts and conservation of water. It is suited to smaller flow rates and smaller land holdings.

3.1.1.3. Disadvantages of Basin Irrigation

i) The levees interfere with the movement of farm machinery equipment. This method is therefore not suited to mechanized farming.

ii) It is sometimes difficult to drain excess water in clayey soils.

iii) Considerable land is occupied by the levees and ditches, reducing the area available for crop production.

iv) Excessive loss of water through additional water channels and over irrigation practice.

v) Because of smaller units, mostly, the corners are left unplowed and unplanted.

vi) Basin method utilizes greater time and energy of farm power per unit area.

vii) It results in lower land and water productivity.

vii) Basin irrigation requires precise land leveling, which increases cost of production.

3.1.2. Border irrigation

This method comprises narrow and longer strips of land and makes use of parallel ridges to guide a sheet of flowing water as it moves down the slope. Thus, borders are long, sloping or level strips of land separated by bunds as shown in Fig. 3.3. Irrigation water can be fed to the borders in several ways such as opening up the irrigation channel bank, using small pipe outlets or gates or by means of siphons. During irrigation, a sheet of water flows down the slope of the border, guided by the bunds.

A border may be level or graded depending on the topographic conditions of the field, particularly in longitudinal direction. The slope in transverse direction is zero in both the cases. In a graded border, the land is divided into a number of long parallel uniformly graded strips of land called borders that are separated by low ridges. The border needs to be laser leveled for better advance rate and irrigation performance. The essential feature is to provide such a land surface that water can flow down the field with uniform depth. Each strip, confined by the border ridges, is irrigated independently for a specified period for given flow rate and depth to be applied.

The width of border strip depends on the size of irrigation stream, amount of cross slope, kind and width of farm machinery and the desired accuracy of land leveling. Length of border strips should be limited to 400 meters. Longer strips are advantageous as they reduce the cost of water conveyance system. Length affected by the infiltration rate of soil, depth of water applied surface roughness and texture of soil.

Under graded border, care should be taken to design the border such that minimum required infiltrated depth is achieved at upstream end of border and a control over excessive accumulation of water at downstream end to avoid undesirable inundation of crop. A border width of 30 meters and length of 200 meters has been found ideal under irrigated conditions of the Punjab.

Fig.3.3 Border Irrigation System

3.1.2.1. Suitability of Border Irrigation

- Border irrigation is suited to most soils where depth and topography permit required land leveling at a reasonable cost and without reduction in soil productivity because of movement of upper fertile depth of soil during leveling.
- It is more suitable to soils having moderate infiltration rates such as loamy soils.
- It is not used in coarse sandy soils having high infiltration rates as the advace rate of irrigation water is too slow to yield efficient irrigation.
- It is also not suited to soils having a very low infiltration rates as the required depth of water does not get the opportunity to infiltrate.
- Level borders are suitable to irrigate all close growing crops like wheat, barley, fodder crops as well as to irrigate, rice, cotton and sugarcane. Graded border is, however, not suitable for crops such as rice, which require standing water during most part of its growing season.
- The length and slope of graded border may be limited by the depth of fertile soil.
- Deeper soils may permit greater length and slope of the border.

3.1.2.2. Advantages of Border Irrigation

- Border dikes can be constructed economically with simple farm implements.

- Uniform distribution and high application efficiency are possible if system is properly designed and managed.
- Large irrigation streams can be effectively used.
- Operation of the system is simple.
- Adequate surface drainage can be provided at the down stream if outlets are available.
- Suitable for mechanized farming.
- Saving of land and water due to reduction of water channels and permanent dikes as compared to Basin irrigation.

3.1.2.3. Disadvantages of Border Irrigation

- This method requires proper land leveling and uniform gentle slope in the direction of irrigation.
- Usually large irrigation streams are required.
- Land is wasted under ridges. However, they are temporarily constructed to guide flow of water.
- Time taken to irrigate increases with length of border. Therefore, excessively long borders cannot be accommodated for smaller stream flows.

3.1.3. Furrow Irrigation

Furrows are small channels, which carry water down the land slope between the crop rows. Water infiltrates into the soil as it moves along the slope. The crop is usually grown on the ridges or raised beds between the furrows as shown in Fig. 3.4. Irrigation water flows from the irrigation channel into the furrows by opening up the bank of the channel, or by means of siphons or gated pipes.

The furrow method of irrigation is used in the irrigation of row crops with furrows developed between the crop rows in the planting and cultivating processes. The size and slope of the furrow depends upon the crop grown, equipment used and spacing between crop rows. Water infiltrates into the soil and spreads laterally to irrigate the areas between the furrows. The length of time, the water to flow in the furrows depends on the amount of water required to replenish the root zone and the infiltration rate of the soil. Both large and small irrigation streams can be used by adjusting the number of areas where surface drainage is necessary. Furrows can also be used to dispose the runoff from rain flooded field.

3.1.3.1. Suitability of Furrow Irrigation

Furrow irrigation can be used to irrigate all crops planted in rows, including orchards and vegetables. This method is suitable for irrigating maize, sorghum, sugarcane, tobacco, groundnut, potato and other vegetables (Fig. 3.4). This method is well suited to crops which may be injured by ponded surface water for longer periods of time, for example 12-24 hours (Jensen 2007). The examples include cotton and maize crops. It is suited to all soils except sandy due to high infiltration rate and lower capillary forces.

Fig. 3.4 Furrow Irrigation

3.1.3.2. Advantages of Furrow Irrigation

- In this method, water directly contacts only a part of the land surface and the remaining part of the ridge or bed receive moisture through capillary action, reducing the opportunities of crusting and evaporation losses.
- Earlier cultivation is possible.
- More area can be irrigated with given amount of water.
- No wastage of land under field ditches.
- Water saving up to 30% in the case of furrow-ridge irrigation and up to 50% in the case of furrow-bed irrigation as compare to border or basin irrigation can be achieved.

3.1.3.3. Disadvantages of Furrow Irrigation

- It requires land grading so that water can move through entire length of furrow without over spilling or ponding.
- It requires continual slope by removing low and high spots.
- It interferes farming equipment during hoeing and harvesting operations.
- It is not suited to close growing and grain crops.
- It is not suited to sandy soils due to unstability of ridges against water.

3.1.4. Corrugation Irrigation

It consists of running water in small furrows, called corrugations which direct the flow down the slope. In this method, more and smaller furrows are made for water control. This method minimizes the crusting effect and used for seed germination sometimes, which have been broadcast (Hagan et al. 1967).

3.2. Subsurface Irrigation

In this method of irrigation, water is applied below the ground surface by maintaining an artificial water table at some depth depending upon the soil texture and the depth of the plant roots. Water reaches the plant roots through capillary action. Water may be introduced through open ditches or underground pipe lines such as tile drains or mole drains. The depth of open ditches or tranches vary from 30 to 100 cm and are spread about 15 to 30 cm apart.

3.2.1. Suitability of Subsurface Irrigation

It is suited to soils having reasonably uniform texture and permeable enough for water to move rapidly both horizontally and vertically within and for some distance below the crop root zone. The soil profile must also contain barrier against excessive losses through deep percolation, either a nearly impermeable layer in the substratum when artificial water table can be maintained throughout the growing season. Topography must be smooth and level.

It can be used for soils having low water holding capacity and evaporation loses are minimum in this method. This method is used in limited areas because it requires favorable natural conditions. Some areas along the link canal or around reservoirs have natural sub-irrigation. Water having high salt contents cannot be used.

3.3. Siphon Irrigation

The word siphon, (or syphon), is used to refer to a wide variety of devices that involve the flow of liquids through tubes. However, in irrigation, it refers, particularly, to a tube in an inverted 'U' shape, which causes a liquid first to flow upward i.e. above the surface of the upstream water reservoir without any external pumping energy but powered by the fall of the liquid to the downstream end of the tube as it flows down the tube under the force of gravity and finally discharges at a level lower than the surface of the reservoir, which may be a field being irrigated or another water reservoir. The irrigation of field with siphon tubes is shown in Fig. 3.5

Fig. 3.5 Siphon Irrigation

The theory behind the flow through the siphon tube may be explained as follows: The siphon tube comprises 2 limbs with the downstream limb being longer than the upper or upstream limb. When the siphon tube is full of water, the volume as well as weight of water contained in the lower limb is greater than that of the upper limb. As the water in the lower limb moves downstream, it creates negative pressure in the top portion of the curved tube, which pulls the water in the lower limb to move upward.

The upper end of the tube being submerged in water with the positive head of water in the source reservoir or irrigation channel and being at higher elevation than the lower end or the exit point of the tube (i.e.the elevation of the water level in the field), the liquid in the lower limb of siphon exerts gravity force pulling the liquid down to the exit side of the siphon. The atmospheric pressure on the water in the source reservoir pushes the liquid from the upper reservoir into the reduced pressure zone at the top portion of the siphon, which continues to flow down the pipe under gravitational pull as it leaves the tube at the lower end to the field. The elevation difference between the water levels in the channel and the field is the effective head in the operation of the siphon tube as shown in Fig. 3.6

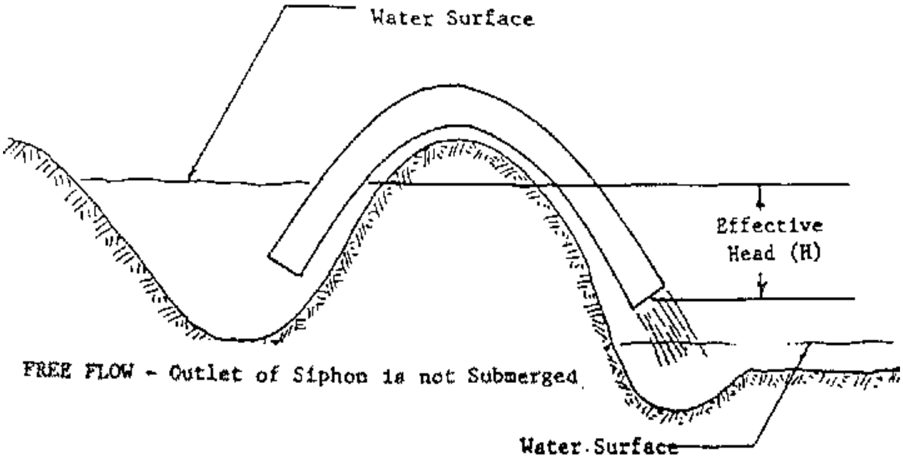


Fig. 3.6 Siphon Irrigation Principle

3.3.1. Advantages of Siphon Irrigation

The advantages of siphon irrigation include:

- Use of siphons in applying irrigation water to the field permits water diversion from irrigation channel to the field without cutting through the channel bank or the field dikes. It is more suited to the application of water to the furrows.
- Use of siphon permits measurement of the flow rate as well as thetotal amount of water applied to the field.

3.3.2. Limitations of Siphon Irrigation

The siphon irrigation system is necessarily use of siphon tubes to convey irrigation water from the irrigation channel to the field. Necessary conditions for successful operation of the siphon irrigation system include:

- The elevation of water at the source (reservoir or irrigation channel) should be higher than the elevation of water at the exit of the siphon tube.
- The upstream end (inlet) of the siphon tube must remain dipped in water for successful operation. Once the inlet of siphon is out of water or exposed to the atmosphere, air will enter the tube and flow of water through the siphon will stop.
- The siphon tube must be filled completely with water before putting it to action. The siphon will not operate if it is patially filled.
- The upstream limb of the siphon tube (from inlet to the top of the curvature) should be smaller than the lower limb (from top of the curvature to the exit of tube).

- For manual installation of the siphon tubes to irrigate a field, the diameter of the tube should not be greater than the palm of the irrigator i.e. < 3 inches to facilitate blocking of the lower end of the tube for retaining water of the tube while installing the siphon. However, larger diameter of tubes sizes can be used with flat rubber sheet instead of irrigator's palm.
- In order to avoid over spilling of irrigation channel, a balance between the inflow to the irrigation channel and outflow to the field must be maintained, which requires that the siphon size, flow rate and number of siphons must be selected to maintain a balance between the inflow and outflow.

3.4. Sprinkler Irrigation System

Sprinkler irrigation is a method of applying irrigation water under pressure to the crops similar to natural rainfall. Water is distributed through a system of pipes by pumping, which is then sprayed into the air through sprinklers so that it breaks up into small water drops falling to the cropped field. The pumping and piping systems, sprinklers and operating conditions must be designed to enable a uniform application of water. The spray is developed by flow of water under high pressure through small orifices or nozzles. With careful selection of nozzle sizes, operating pressures and sprinkler spacing, the amount of irrigation water required to fill the crop root zone can be applied uniformly at a rate to suit the infiltration rate of the soil, obtaining efficient irrigation (Fig. 3.7).

Fig. 3.7 Sprinkler Irrigation System

3.4.1. Suitability

The sprinkler irrigation system is a very suitable method for irrigation on undulating topography and on shallow soils. It is best suited to coarse sandy soils where other gravity irrigation methods cannot be used successfully because of high infiltration rate and low rate of advance. It is a controlled system of water application that permits measured quantities of water in shallow depths with higher efficiency and greater frequency of irrigation. The sprinkler irrigation system is also suitable in undulating terrain where land grading is expensive or technically not feasible because of the removal of fertile soil depth by earth movement. Sprinkler irrigation system can be used successfully in hilly regions where crops are grown on terraces in smaller land units and where the water supply may be available in limited flow rate. Other advantages of sprinkler irrigation system are given under section 3.3.5. Sprinkler irrigation system allows to apply water at desired location of crop without infiltration or loss during conveyance.

3.4.1.1. Suitability to Crops

Sprinkler irrigation is suited for most row, field and tree crops and water can be sprayed over or under the crop canopy. However, large sprinklers are not recommended for irrigation of delicate crops such as lettuce because the large water drops produced by the sprinklers may damage the crop.

3.4.1.2. Suitability to Topographic Slopes

Sprinkler irrigation is adaptable to any farmable slope, whether uniform or undulating. The lateral pipes supplying water to the sprinklers should always be laid out along the land contour whenever possible. This will minimize the pressure changes at the sprinklers and provide a uniform irrigation.

3.4.1.3. Suitability to Soils

Sprinklers are best suited to sandy soils with high infiltration rates although they are adaptable to most soils. The average application rate from the sprinklers (mm/h) is always chosen to be less than the basic infiltration rate of the soil (see Annex 2) so that surface ponding and runoff can be avoided.

Sprinklers are not suitable for soils which easily form a crust. If sprinkler irrigation is the only method available, then light fine sprays should be used. The larger sprinklers producing larger water droplets are to be avoided.

3.4.1.4. Suitability to Irrigation Water

A good clean supply of water, free of suspended sediments, is required to avoid problems of sprinkler nozzle blockage and spoiling the crop by coating it with sediment. The uniformity of sprinkler applications can be affected by wind and water pressure. Spray from sprinklers is easily blown about by even a gentle breeze and this can seriously reduce uniformity. To reduce the effects of wind the sprinklers can be positioned more closely together. Sprinklers will only work well at the right operating pressure recommended by the manufacturer. If the pressure is above or below this then the distribution will be affected. The most common problem is when the pressure is too low. This happens when pumps and pipes wear. Friction increases and so pressure at the sprinkler reduces. The result is that the water jet does not break up and all the water tends to fall in one area towards the outside of the wetted circle. If the pressure is too high then the distribution will also be poor. A fine spray develops which falls close to the sprinkler.

3.4.2. Components of Sprinkler Irrigation System

3.4.2.1. Pumping Unit

A pumping unit is required for lifting water from the water source and moving it under applied pressure through the distribution system i.e. main, sub main, laterals and through the sprinkler heads.The pumping unit usually consists of a centrifugal pump or turbine with a prime mover such as electric motor or a diesel engine. The sources of water for the sprinkler system may include a river, shallow well, pumped groundwater through a tubewell or turbine storage tank or an irrigation channel etc. The pumping units may comprise electric motors for fixed installations or diesel engines and pump installed on portable installations.

3.4.2.2. Main line and Sub Main

The main line and Sub main pipe lines may be fixed or portable to carry water from the source or pumping unit to various other parts of the system in the field. Permanent lines are generally buried into the ground below the working depth of the implements. In other cases they are temporarily installed, and can be moved from field to field. Light weight aluminum pipe with quick coupling systems are preferred for portable lines. The fixed pipes are generally of steel, plastic or poly venyl chloride (PVC) material. Sub mains carry water from main to lateral lines.

3.4.2.3. Lateral Lines

These pipes carry water from main or sub main pipe line to the sprinkler head through the riser pipes. They are portable and equipped with quick coupling devices. Commonly they are available in 3 to 4 m length with rubber gasket in the female portion of coupling to act as a water seal.

3.4.2.4. Sprinkler Heads

Sprinkler heads are used for spraying water on the field. Types of sprinkler head include: rotating head, fixed head and perforated heads. Fixed head type sprinkler heads are used in landscaping installations. Sprinkler heads can also be classified on basis of pressure.

1) Low pressure sprinkler (1.5 to 2.5 kg/cm^2)
2) Intermediate pressure sprinkler (2.5 to 5 kg/cm^2)
3) High pressure sprinkler (5 to 10 kg/cm^2)

3.4.3. Application Rate

This is the average rate at which water is sprayed onto the crops and is measured in mm/h or inches/h. The application rate depends on the size of sprinkler nozzles, the operating pressure and the distance between sprinklers. When selecting a sprinkler system it is important to make sure that the average application rate is less than the basic infiltration rate of the soil. This will allow all the applied water to be readily absorbed by the soil with no runoff.

3.4.4. Sprinkler Drop Sizes

As water sprays from a sprinkler it breaks up into small drops between 0.5 and 4.0 mm in size. The small drops fall close to the sprinkler whereas the larger ones fall close to the edge of the wetted circle. Large drops can damage delicate crops and soils and so in such conditions it is best to use the smaller sprinklers.

Drop size is also controlled by the pressure and size of nozzles. When the pressure is low, water drops tend to be much larger as the water jet does not break up easily. Thus, in order to avoid damage to crop and soil, smaller diameter nozzles operating at or above the normal recommended operating pressure, may be used.

3.4.5. Advantages of Sprinkler Irrigation

- Soluble fertilizers, herbicides and fungicides can be applied in the irrigation water more uniformly and economically. This facilitates uniform crop stand and effective pest control. Sprinkler systems are particularly suited for foliar application.
- Sprinkling of water can be used to protect crop against frost and high temperature.
- Labor costs are generally reduced as compared to other methods.
- More land is available for cropping and irrigation process does not interfere with farm machinery, particularly with portable sprinkling system.
- Sprinkler irrigation can be used for almost all crops except rice and on most soils except very fine textured soils (heavy clay) where the infiltration rates are less than 4 mm/h.
- Erosion of soil cover, which is common in surface irrigation, can be eliminated.
- Shallow soils, land with steep slope and soils involving extensive land preparation can be irrigated efficiently.
- This method is popular in regions of water scarcity and uneven topography.
- Portable sprinkling systems are field location friendly including single gun or boom sprinklers or traveling gun that can be periodically moved from one position to another by hand or mechanically during irrigation.

3.4.6. Disadvantages of Sprinkler Irrigation

- It is not useful in windy areas as the water drops get dislocated by wind.
- Ripening fruit must be protected from the spray.
- The unavoidable wetting of foliage in field crops results in increased sensitivity to diseases.
- A sustained water supply and pumping pressure are required for successful irrigation application, which may add to the operational cost.

- Water must be clean and free of debris, sand and large amount of dissolved salts.
- It requires high initial investment as compared to surface methods of irrigation.
- Power requirements are usually high since sprinklers operate at a pressure of 5 N/cm^2 to 900 N/cm^2.

3.4.7. Classification of Sprinkler Systems

Sprinkler systems are classified according to (i) whether the sprinkler heads are operated individually (gun or boom sprinkler) or as group along laterals (Side Role or Center Pivot system), (ii) how they are moved or cycled to irrigate the entire field (Portable Systems).

3.4.7.1. Rain Gun Sprinkler System

This system comprises single gun attached to the pumping system that can sprinkle at least one acre of land with one setting (Fig. 3.8).

Fig. 3.8 Single Gun Sprinkler System

According to mobility, the sprinkler system may be classified as portable, semi-portable and permanent as discussed below.

3.4.7.2. Portable System

It has portable main lines, lateral and pumping plant, risers and sprinkler heads. It can be moved from field to field in different settings. This type of system is placed on the field where irrigation application is desired. After completion of irrigation it can be completely moved from the field and therefore do not provide any hindrance to farm operation. However, labor requirements increase.

3.4.7.3. Semi-Portable System

Water course, pumping plant, main and sub mains are fixed only laterals along with risers and sprinkler heads are portable.

3.4.7.4. Permanent System

A permanent system consists of permanently laid main, sub mains, lateral and stationary water course and pumping plant.

3.4.7.5. Solid Set System

Solid set system remain in a single location during an irrigation season and supplied by a fixed network of pipes (Fig. 3.9). These systems irrigate the entire field with a single set of components, and therefore, are more costly than most of other systems. The labor and maintenance are however minimal. They are suitable for crops requiring minimum cultural practices. Solid set systems are generally designed to use low – flow medium pressure sprinklers. Spacing vary from 9 m x 9 m to 73 m x 73 m. Nozzle sizes can vary from 1.59 mm to 36 mm and pressure can vary from 30 psi to 85 psi (i.e. 205 kpa to 585 kpa).

Fig.3.9 Solid Set Sprinkler Irrigation

3.4.7.6. Side Role System

A side role or wheal move system has wheels mounted on the lateral pipe with length acting as the axle of the wheel. A lateral length upto 400 m is rolled forward and backward by applying power at the center or at the end while pipe remains in nearly horizontal straight line. The wheel diameter should be large enough to pass over the crop without damaging it (Fig. 3.10).

Fig. 3.10 Side Roll Sprinkler system

3.4.7.7. Center Pivot System

It consists of a number of sprinkler guns mounted on a trussed beam that rotates continuously around a pivot (center) supplied with pumping unit. It is a continuous move system. A center pivot system infact comprises a single lateral. The lateral pipe with a set of sprinklers is supported on a drive unit and suspended by trusses. The water is supplied from the source (a pump, a reservoir or a ditch) to the lateral through the pivot. An electric motor mounted on the drive unit rotates the lateral that is supported at the other end on wheels trailing in the field in circular track. A smaller unit of center pivot system at the university campus has the following design and operational specifications (Fig. 3.11).

Fig. 3.11 Centre Pivot Irrigation System

3.4.8. Sprinkler System Layout

The most common type of sprinkler system layout is shown in Fig. 3.11a. It consists of a system of aluminum or plastic pipes (laterals), which are moved by hand along the main line. The rotary sprinklers are usually spaced 9-24 m apart along the lateral, which are normally 5-12.5 cm in diameter. The lateral pipe is located in the field until the irrigation is complete. After completion of irrigation, the lateral is disconnected from the mainline and moved to the next location. It is re-assembled and connected to the mainline and the irrigation begins again. Under this layout, the laterals can be moved one to four times a day until the whole field is irrigated. More than one lateral can be used to irrigate larger areas in shorter time. An irrigation cycle consists of placement of 1st lateral to the last lateral to complete irrigation of the whole field.

The wetting pattern from a single sprinkler is not very uniform. In order to achieve a desired uniformity, sprinklers must be operated with an overlap of at least 65% of the wetted diameter. This determines the maximum spacing between sprinklers.

A common problem with sprinkler irrigation is the large labour force needed to move the pipes and sprinklers around the field. In some places such labour may not be available and may also be costly. To overcome this problem, the hand moved (lateral placement) system may be replaced by mobile systems such as side role, raingun or centre pivot.

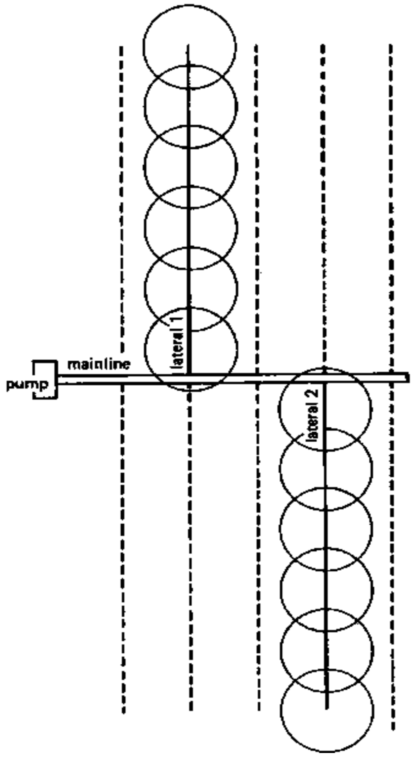


Fig. 3.11a Hand-moved Sprinkler System Using Two Laterals

3.5. Drip or Trickle or Micro Irrigation System

Drip irrigation, also known as Trickle irrigation or Micro irrigation, is an irrigation method, which saves water and fertilizer by allowing water to drip slowly to the roots of plants, either to the soil surface or directly to the root zone as subsurface flow, through a network of valves, pipes, tubing, and emitters supported by pumping system as shown in Fig.3.12. It is also called pressurized irrigation system as the water is delivered under low pressure.

Drip is one of the latest methods of irrigation that is becoming popular in areas with water scarcity and coarse soils having high infiltration rate. It minimizes conveyance and other conventional losses such as deep percolation, runoff and subsoil water through dippers. In this method, irrigation is accomplished by using small diameter plastic lateral lines with devices called emitters or drippers at selected spacing to deliver water drop by drop to the soil surface near the base of the plants. The system applies water slowly to keep the soil moist within the desired range of plant root system. The system operates at pressure of 10 to 15 psi. The emitters dissipate pressure energy from the distribution system by means of orifices, vortexes and tortuous or long flow paths thus allowing a limited volume of water to discharge. Most emitters are placed on ground but they can also be buried. The emitted water moves within soil system largely by unsaturated flow.

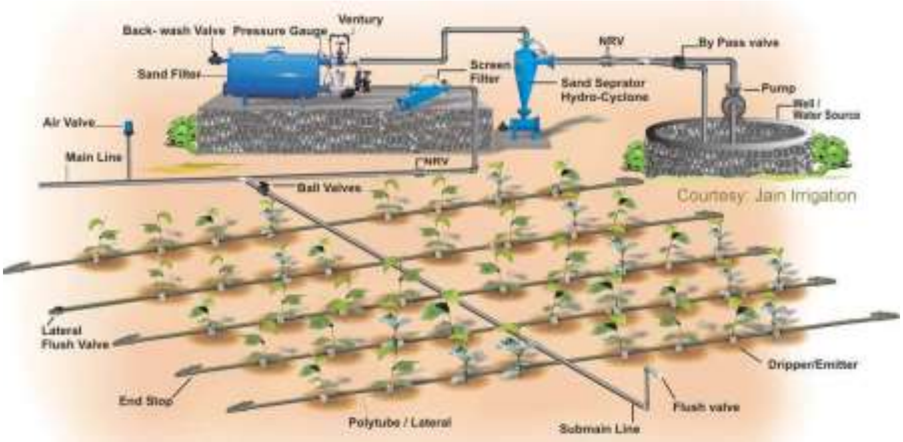


Fig. 3.12 A Typical Drip Irrigation System

3.5.1. Historical Developments

Drip irrigation has been used since ancient times when buried clay pots were filled with water, which gradually released water to infiltrate into the soil. Modern drip irrigation began its development in Afghanistan in 1866 where clay pipes were used to create combined irrigation and drainage systems. In 1913, Colorado State University demonstrated applying water to the root zone of plants through perforated pipes without raising the water table. Similar methodology of water application was introduced in Germany in the 1920s and porous canvas hose was used at Michigan State University in 1934, which was similar to the drip irrigation system. With the advent of plastic materials and various types of emitters, improvements in drip irrigation system started in the greenhouses of Europe and the United States.

The modern drip irrigation technology was developed in Israel in 1959 using plastic emitters to slow down the velocity of water inside the emitter. This method proved successful, which spread to Australia, North America, and South America by the late 1960s. The first drip tape, called Dew Hose, was developed in the United States, in the early 1960s. In 1989, Jain Irrigation Company in India introduced effective water-management through drip irrigation to the Indian agriculture.

Modern drip irrigation has become the world's most valued innovation in agriculture since the invention of the impact sprinkler in the 1930s, which offered the first practical alternative to surface irrigation. Drip irrigation may also use devices such as micro-spray heads, which spray water in a small area, instead of dripping emitters, and are generally used on trees and vine crops. Subsurface drip irrigation (SDI) uses permanently or temporarily buried dripper lines or drip tapes located at or below the plant roots. It is becoming popular for row crop irrigation, especially in areas where water supplies are limited or recycled water is used for irrigation. Factors including land topography, soil, water, crop and agro-climatic conditions are needed to determine the most suitable drip irrigation system and components to be used in a specific installation.

3.5.2. Principles of Drip Irrigation

In drip irrigation, the driving force of water movement is provided by an external energy source such as a pump or a raised reservoir. Water from the source is delivered through a closed pipe system under low pressure. It differs from surface irrigation technologies such as flood, basin, border and furrow irrigation, in which the driving force of water flow is gravity, and the delivery and application structures including canals, ditches, furrows, small ponds and basins, which are open to the atmosphere. The term trickle irrigation is generally used to describe irrigation methods in which small quantities of water are applied at short intervals directly to the soil. The uniqueness of drip irrigation is the partial wetting of the soil as water is applied by the emitters at the plant locations only, leaving the remaining area out of water.

In on-surface installation, each emitter moistens the adjacent surface area. The %age of the wetted surface area and soil volume depends on soil properties, initial moisture level of the soil, the applied water volume, emitter flow rate and the number of plants per acre, which are usually 6,000 to 7,000 per acre. In subsurface installations, the soil surface remains dry. The lateral movement of the water beneath the surface of a medium or heavy textured soil is more pronounced than in sandy soils as shown in Fig. 3.13. Whenever the dripper's flow rate exceeds the soil intake rate (infiltration rate) and its hydraulic conductivity, the water tends to flow on the soil surfaceas runoff and wets larger soil volume, which is undesirable. Therefore, efficient drip irrigation requires a flow rate less than the infiltration rate of soil.

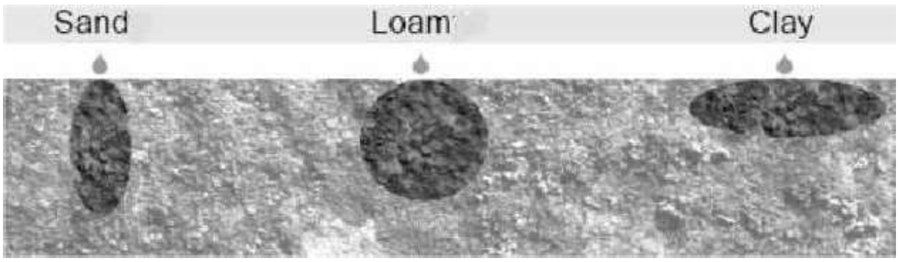


Fig. 3.13 Wetting Pattern in Sandy, loamy and Clayey Soils

Indicative values for the wetted diameter by a single dripper may be 30 cm in a light soil, 60 cm in a medium soil and 120 cm in a fine textured soil. Due to the partial wetting of the soil in drip irrigation, water has to be applied more frequently than with other irrigation methods that wet the entire area such as sprinkler and flood irrigation. Water is delivered from the emitter continuously in drops at one point, infiltrates into the soil and wets the root zone vertically by gravity and horizontally due to capillarity. The customary dripper flow-rate range is 2 to 4 liters per hour (lph).

3.5.3. Advantages

Drip irrigation significantly increases the efficiency of water application and improves the growing conditions of the irrigated crops. Appropriate water management can minimize water and nutrient losses beneath the root-zone. It has many advantages over other methods of irrigation as given below:

- Application of water precisely to the crop root zone without wetting the entire area between plants, reducing the total evaporative surface, saves considerable amount of water. Thus, water does not flow beyond the limits of the irrigated area as may happen with sprinkler or other irrigation methods.
- The germination and development of weeds is limited because a limited soil surface is wetted.
- Fertilizers can be injected into the irrigation water, and therefore, can be applied to the crop more efficiently.
- Drip irrigation can achieve more than 90% application efficiency, which is the highest among known irrigation methods.
- Frequent or daily application of water keep the soil water more diluted and leached to the outer limits of the wetting pattern, encouraging the use of saline water.
- The partial wetting of the soil surface does not interfere with other crop production activities in the field.
- Drip irrigation functions successfully on steeper slopes, shallow and compacted soils with low water infiltration rate and sandy soils with low water-holding capacity.
- This method results in substantial increase in yield per unit of water used, thereby increasing the water use efficiency.

3.5.4. Disadvantages

- The initial cost of drip irrigation equipment is high.
- Water must be relatively clear to overcome frequent clogging of emitters. The causes of clogging may be physical, chemical or biological factors.
- The root development remains limited due to the lack of moisture.
- Because of limited root system, nutrient availability may be limited as compared to gravity irrigation methods.
- There may be deposition of salts in root zone because there is no leaching of salts due to less moisture. Excess salt accumulation may occur at the soil surface or towards the fringes of wetted soil.

3.5.5. Limitations of Drip Irrigation System

Some of the disadvantages given above may also cause limited use of the system. Other limitations may include:

- The emitters are susceptible to clogging by solid particles, suspended organic matter and chemical precipitates formed in the water. Clogging may also occur by suction of soil particles and root intrusion into the dripper.

- Upward capillary movement of moisture from the wetted soil volume and evaporation from the soil surface may cause a high concentration of salts in the upper soil layer.
- The laterals, particularly the thin-walled tapes and the tiny drippers are prone to damage by rodents, rats and moles etc. Subsurface laterals and drippers may also be damaged by rodents.
- The frequent water applications to limited soil volume may lead to the development of restricted or shallow root systems.

3.5.6. Components of Drip Irrigation System

A drip irrigation system consists of water source, pumping unit, mixing chamber (fertilizer tank), filters, flow meter, main line, submain, lateral and emitters (Fig. 3.14). The main line delivers water to the submain and the submains into the laterals. The emitters, which are attached to the laterals, distribute water for irrigation. The main, submains and laterals are usually made of black-PVC (Poly Venyl Chloride) tubings. The emitters are also usually made of PVC material. These materials are preferred for drip system as it can withstand saline irrigation water and is also not affected by chemical fertilizers. Mixing chamber (Fertilizer Tank) is used to mix fertilizers or chemicals into the system for application to the crops.

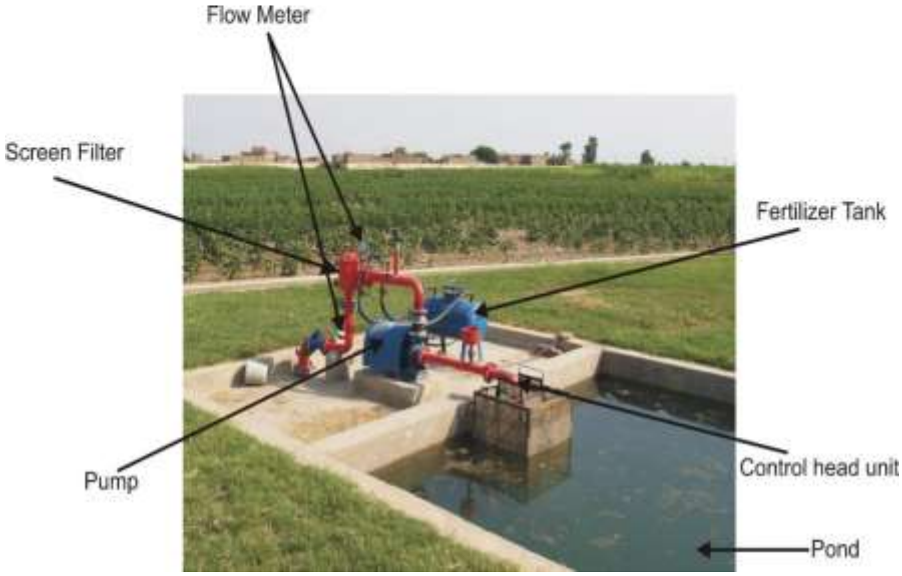


Fig.3.14 Water Supply System for DRIP Irrigation at NIAB Faisalabad

Functions performed by each system component are described below.

- The sources of water for drip irrigation system may include canals, wells, tubewells and water reservoir etc. The pump facilitates the flow of water from the source to the drippers under appropriate pressure and at appropriate discharge. If foreign particles from soil, leaves, algae, etc coming with water are not filtered, the drippers may be clogged that may result in obstructing the water supply to plants. In order to overcome this problem, sand filter is connected to the main line to stop impurities and to allow only clean water to move ahead in the system.

- Majority of the impurities are being filtered by the sand filter, while some minute sand particles and other impurities pass through the sand filter, which are further removed by the screen filter. Screen filter consists of screen strainer which filters physical impurities and allows only clean water to enter in the system.
- Maintaining normal operating pressure in the system is essential to ensure the uniformity of the irrigation. Thus, pressure gauges are provided in the drip irrigation system to indicate the operating pressure. The Ventury and Fertilizer Tank are provided to inject the fertilizer or chemical solution to the irrigation water for application to the crop with irrigation water, which helps to save fertilizer.
- Control valves are provided at the inlet of submain to control the flow rate and at the end of submain as flush valve to facilitate regular cleaning of main and submain. Air release valve is installed at the highest point on the mainline to release the entrapped air during start of the system and to break the vacuum during shut off.
- Main line is used to carry water from water source to submain. Filter is attached with the main line to provide clean/filtered water to submain. The Submain line is used to supply water to the laterals on one or both side of it. Laterals or Drip tapes are either line source tubing or drip laterals. These convey water from submain lines to root zone via drippers. They are spread in open field and their spacing is decided on the basis of row to row distance of crop. Water supplied by the laterals reaches the plant root zone through drippers. Type of crop, soil and crop water requirements are deciding factors for dripper spacing on the lateral. Fig. 3.15 shows installation of main line and laterals of drip irrigation system under Lower Bari Doab Canal (LBDC) project.
- Water Storage Tank is provided to store irrigation water that may be pumped to the drip pipe line through a pumping system. Under LBDC Project, a storage tank of 22 m × 22 m × 2 m size, was provided for a 10 acre area drip irrigation system to support cotton crop on beds as shown in Fig.3.16. A complete drip irrigation system with diesel engine pumping system, fertilizer tank, contrl valves and filters along with the storage tank is shown in Fig. 3.17.

Fig. 3.15 Installation of Drip System in progress (LBDC Project)

Fig.3.16 Water Storage Tank (20 × 20 × 2 m) for drip irrigation system

Fig. 3.17 Pumping System for Drip Irrigation Irrigation System

Drip irrigation system installed on flat sown cotton is given in Fig. 3.18 and cotton on beds in Fig. 3.19. Generally, one lateral is installed in the centre of the bed to serve two rows of cotton.

Fig. 3.18 Drip Irrigation for Cotton at NIAB Faisalabad, Pakstan

Fig. 3.19 Drip Irrigation with Furrow Bed Cotton at LBDCIP Chichawatni Site

3.5.7. Suitability

Although this method is adopted in areas where water is scarce and quality is marginal or soil is sandy, it is especially suited to fruit trees and other valuable trees and vine crops. It is adopted for crops where application at selected points is possible instead of covering the whole area as done in gravity methods. For dense crops, its application is restricted due to cost.

3.5.8. Selection of Irrigation Method

The following factors are considered in selecting an appropriate method of irrigation.

- Topographic features such as surface irregularities, steepness of slope and changes in slope direction.
- Soil factors such as profile, water holding capacity, intake rate and depth.
- Crop factors such as growth period, water requirements, depth of root zone and growth characteristics.
- Geographic factors such as field shape, drainage, buildings, roads, railways and other obstructions.
- Water supply, quality, available flow rate, delivery schedule and location of water source.
- Labor requirements, cost and availability.

- Energy requirements, cost and availability.
- Availability of system components (whether imported or local).
- Farmer's preference and requirements.
- Climatic conditions under which the irrigation system has to operate.
- Soil type and water quality.

3.5.9. Distribution of Water in the Soil

The flow of water and its distribution within the soil by drip irrigation is different from that obtained with other irrigation techniques. Under Drip irrigation, water distribution within the soil follows a three-dimensional flow pattern, compared with the one-dimensional, vertical percolation pattern typical of flood and sprinkler irrigation that wet the entire soil surface area.

Water is applied from a point or line source. Point sources are discrete drippers; each of them wets a discrete volume of soil. Line sources are drip laterals in which the drippers are installed close to each other. The water flows along the lateral so that the wetted volumes formed by adjacent emitters, overlap and create a wetted strip. With on-surface drip irrigation, the wetted soil surface area is a small fraction of the total soil surface area. With subsurface drip irrigation, the wetting pattern is quite different as water moves downward, sideways and also upwards.

Fig. 3.20 presents water distribution pattern in the soil under drip (Fig. 3.20a) and SDI system (Fig. 3.20b).

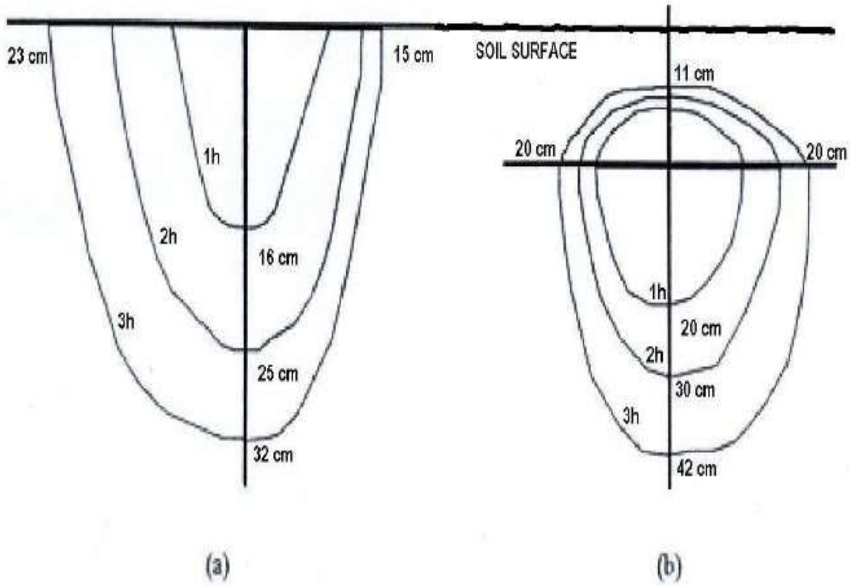


Fig. 3.20 Water distribution in the Soil (a) On-Surface Drip irrigation and (b) SDI

Two driving forces, namely gravity and capillary, simultaneously influence the flow of water in the soil. Gravity drives water downwards, while capillary force drives the water in all directions. The equilibrium between these two forces determines the distribution pattern of water within the soil. The water distribution pattern affects the spreading of the roots in the soil and also the distribution and accumulation of the dissolved chemicals - nutrients and salts in the soil.

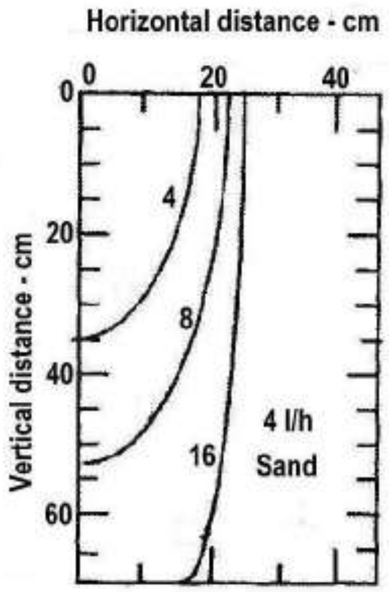
3.5.10. Factors Affecting Soil Wetting Patterns

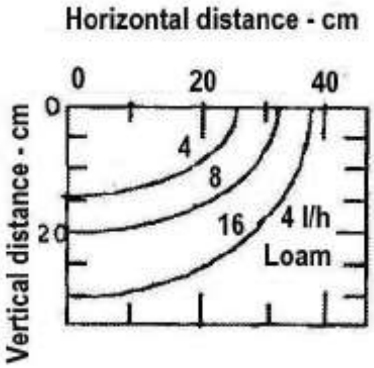
The main factors affecting the distribution pattern of water and solutes in the wetted soil volume with drip irrigation are listed below:

- Capillary forces are more pronounced in finer textured soils, while gravitational are more pronounced in medium and course textured soils. Therefore, the horizontal width of the wetted soil volume is greater than the vertical depth in finer textured soils and the wetted soil shapes like an onion.
- As the textures moves towards course materials, the vertical (gravitational) forces dominate the capillary forces and therefore, in medium textured soils, the wetted volume is pear-shaped, and in coarse textured sopils, the vertical water movement is more pronounced than the horizontal one so that the wetting volume resembles the shape of a carrot.
- The greatest wetted horizontal diameter by drippers of on-surface drip laterals is near the soil surface i.e., 10 – 30 cm deep.
- The greatest wetting horizontal diameter by drippers of subsurface drip laterals is at the depth of the lateral. The vertical dimension of wetted soil above the emitter in SDI is about ¼ of the wetted width in sandy soil and about ½ of the wetted width in silty and clayey soils.
- For the same application time-length and amount of water applied, a lower flow rate renders a narrow and deeper wetting pattern and a higher flow rate renders a wider and shallower wetting pattern.

3.5.11. Emitter Spacing

For the same application time-length and volume of water applied, narrow spacing with overlapping renders narrower and deeper wetting pattern. The wetted width by each dripper increases until adjacent circles overlap. After overlapping, most of the flow is directed downwards. Water distribution patterns for sandy and loam soils with a flow rate of 4 lph are shown in Fig. 3.21.





(a) Sandy Soil (b) Loamy Soil

Fig. 3.21 Single Dripper Water Distribution in Sandy and Loamy Soils

3.5.12. Root System Development under Drip Irrigation

It is well known that the water application regime and water distribution pattern in the soil affect the pattern of root system development. Each plant family has a typical root distribution pattern, stemming from the growing conditions in the plant's site of origin and its adaptation of the plant to the local growing environment. The root system pattern and soil properties are important factors in determining dripper spacing and the scheduling of the irrigation regime. Shallow and sparse root systems require a close dripper spacing and frequent water applications, while deep and branched root systems allow for wider spacing and larger intervals between irrigations. Frequent and small water applications by drip irrigation lead to the development of shallow and compact root systems. This increases crop sensitivity to heat spells and water stress. Large plants with shallow root systems are prone to uprooting by strong storms. On the other hand, because of the improved aeration and nutrition in the drip irrigated soil volume, the density of the active fine roots is significantly higher than the density of root systems that grow under sprinkler irrigation.

3.5.13. Design of Drip Irrigation System

The design of drip irrigation system is the integration of physical components into a system arrangement, which is able to meet the crop water requirement subject to the soil, water and equipment limitations. The design procedure is as follows:

3.5.13.1. Collection of Basic Information

Before designing a drip irrigation system, field survey is carried out to get information about the area where the drip system is to be installed, crop to be sown, soil type, water source and power source availability at the site.

3.5.13.2. Peak Water Requirement (PWR)

Peak water requirement is also called peak daily consumptive use. It is calculated using the equation 3.1.

$$\text{PWR (mm/day)} = \frac{Et_o \times K_c \times C_f}{Irrigation\ Efficiency} \qquad (3.1)$$

Where:

Et_o is the reference evapotranspiration

K_c is the crop coefficient

C_f is the canopy factor

As the Crop Water Requirement (CWR) is the product of Et_o, Kc and C_f, select the Et_o and Kc as design values for which the CWR is maximum.

3.5.13.3. Selection of Drippers/Emitters

Water from the laterals reaches the plant root zone through drippers, which are spaced in accordance with the type of crop, soil and crop water requirements. Water tends to move downward faster in sandy soil, which requires closely spaced drippers such as with 7.5 lph flow rate installed 0.25-0.30 m apart. In loamy soils, water moves downwards slowly and spreads more evenly, therefore, it is recommended to use drippers with flow rate of 4 lph and spaced 0.40-0.45 m apart. In clay soils, water will be absorbed very slowly, therefore, use 2 lph drippers with 0.45-0.60 m spacing. Under each type of soil, total number of emitters is calculated using the equation 3.2.

$$\text{Total No. of emitters (Nos.)} = \frac{\text{Total drip length (m)}}{\text{Emitter spacing (m)}} \qquad (3.2)$$

3.5.13.4. Selection of Laterals

Laterals convey water from submain lines to root zone through drippers. The spacing between laterals is determined by the distance between centers of crop rows. Length of run has a direct effect on the uniformity of each drip lateral. If laterals are too long, pressure loss causes a higher application rate at the beginning of the run than at the end. The total length of lateral which will be required to cover the whole field at a given lateral spacing is calculated using the equation 3.3.

$$\text{Total lateral length (m)} = \frac{Area\ (acres) \times 4047}{Lateral\ Spacing\ (m)} \qquad (3.3)$$

It is important to find out how long a lateral can be used on each side of the sub main so that variation in discharge due to friction loss is within allowable limit. The allowable limit for emitter flow variation is less than 10%, which can be extended to 20% depending on the crop. For 10% variation in discharge, a 20% variation in the available head is acceptable. Accordingly allowable length of lateral (L) can be calculated from flow Equation 3.4 (Hazen-Williams Equation using C = 150) as given below:

$$H_{l} = \frac{5.35\, Q^{1.852}\, L}{D^{4.871}} \quad (3.4)$$

Where:

H_l is pressure loss due to friction (m)

Q is total discharge of lateral (lps)

L is length of lateral (m)

D is inside diameter (cm)

3.5.13.5. Selection of Submain

Submain is a conduit which carries water from main line and delivers to the laterals. Standard Discharge Rate (SDR) in submain can be calculated as:

$$\text{SDR of submain (lph)} = \frac{\text{total flow in submain (lps)} \times 3600}{\text{submain length (m)}} \quad (3.5)$$

Size of submain may be selected from a graph against the values of SDR and submain length.

3.5.13.6. Selection of Main line

Selection of mainline involves determining diameter of main pipe line. It depends on flow rate, operating pressure and filed topography. The mainline size is selected so that allowable pressure variation due to friction loss in the pipe, remains within limit for the economic pipe sizing. Frictional head loss can be calculated using Hazen-Williams equation (3.6) as given below:

$$H_{l} = \frac{15.27\, Q^{1.852}\, L}{D^{4.871}} \quad (3.6)$$

Where:

Hl is pressure loss due to friction (m)

Q is total discharge of lateral (lps)

L is length of lateral (m)

D is inside diameter (cm)

3.5.13.7. Total Flow Rate

Once the lateral lines and emitters are selected, the total flow rate through the drip system for the design area can be calculated as follows:

$$\text{Total flow rate (LPH)} = \frac{\text{Total area (sq.m)} \times \text{Emitter flow (lph)}}{\text{Lateral spacing (m)} \times \text{Emitter spacing (m)}} \quad (3.7)$$

3.5.13.8. Calculation of Application Rate

Application rate is the flow per unit area. It can be calculated by the following two methods (Equation 3.8 and 3.9).

Method 1

$$\text{Application rate in mm/hr} = \frac{\text{Total Flow (lph)}}{\text{Total Area } (\text{m}^2)} \tag{3.8}$$

Method 2

$$\text{Application rate in mm/hr} = \frac{\text{Emitter Flow (lph)}}{\text{Lateral spacing (m)} \times \text{Emitterspacing (m)}} \tag{3.9}$$

3.5.13.9. Operation Time

Operation time means the time for which the drip system will operate to complete the irrigation of the design area. It can be calculated using peak water requirement and application rate as given below:

$$\text{Operation time in hrs/day} = \frac{\text{PWR (mm/day)}}{\text{Application rate (mm/hr)}} \tag{3.10}$$

3.5.13.10. Flow per Operation

Flow per operation is calculated as:

$$\text{Flow per operation in LPH} = \frac{\text{Total Flow (lph)}}{\text{Nos. of operation}} \tag{3.11}$$

3.5.14. Selection of Filters

Filtration requirement depends on size of flow path in the emitter, quality of water and flow in the mainline. For large systems, depending on water quality, different filters or combination of filters can be used. For large flow requirements, filters can be connected in parallel using manifolds so that pressure loss across the filter is within limit. Four types of filters are mainly available in different sizes as described below.

3.5.14.1. Screen (Mesh) Filter

Screen filters or strainers remove physical impurities escaping from sand filters. These are made of plastic or metal materials in various sizes for different flow rates ranging from 1 to 40 m^3/h.

3.5.14.2. S and (Media) Filter

Sand filters are connected to the main line in series with the screen filters. It is used to filter water with suspended particles and organic impurities such as soil particles, leaves, pieces of branches and algae etc. Either sand or gravel can be used as media for filtration. It is also called as depth filter.

3.5.14.3. Disc Filter

It is made of plastic material and has round discs with micro water path, stalked together in a cylinder so that impurities cannot pass through the discs.

3.5.14.4. Hydro-Cyclone

It is a static device that applies centrifugal force to a flowing liquid mixture. It converts the incoming liquid velocity into rotary motion. It is made of M.S. metal and has a conical shaped cylinder to give centrifugal action to the flow of water so that heavy impurities settle down. It is used in case of sandy water along with the screen filter, particularly where the water source is a dug well.

3.5.15. Selection of Pump / Total Head Requirement

Total Head (pressure) required at the inlet of the mainline or filter can be estimated as:

Total Head (m) = Operating pressure (m) + friction loss through mainline, submains and laterals (m) + fittings loss (m)+ Filter loss (m) + elevation difference (m) (3.12)

In case of centrifugal pump, the total head requirement can be estimated as:

Total Head (m) = Suction head (m) + Delivery head (m) + Operating pressure (m) + Mainline friction loss (m) + fittings loss (m) + Filter loss (m) + Elevation difference (m) (3.13)

3.5.16. Horse Power Requirement:

The horse power requirement (HP) for motor is calculated as follows:

$$\text{Horse Power (HP)} = \frac{\text{Flow (lps)} \times \text{Total head (m)}}{75 \times \text{Motor efficiency} \times \text{Pump efficiency}} \qquad (3.14)$$

3.5.17. System Maintenance

Efficient operation of the drip irrigation system depends on proper maintenance of the system. Plugging or clogging of emitters may occur because of physical, chemical or biological contaminants present in water and the flow line, which may be considered as a major maintenance problem with the drip irrigation system. For optimal performance, drip irrigation systems require routine system maintenance, which involves series of actions as detailed below.

3.5.17.1. Mechanical Filtration

Mechanical filtration methods, including settling basins, screens, centrifugal sand separators and cartridge and/or sand filters, can be used to reduce suspended particles. Filtration units may require the addition of booster pumps for proper backwashing and flushing operations.

3.5.17.2. Settling basins

Settling basins can remove suspended materials in water ranging from sand to silt. Settling basins must be constructed so that they can be cleaned of sediments at intervals, either mechanically or by water flushing.

3.5.17.3 Sand Separators

The operation of sand separator is based on the principle of centrifugal force. Dense sediments such as mineral solids are separated from the bulk water they accumulated and are swept out with a small portion of the water.

3.5.17.4 Screens

Screens are probably the most versatile of all components for removing foreign matters from water. They can be constructed in almost any shape, size and mesh except that there is a lower limit to the opening size. Numerous types of screens are available from giant rotating screens cleaned automatically with jets of water and/or brushes with small amount of water spillage, to simple screens which are placed in open channels of water and are cleaned by hand at suitable intervals.

3.5.17.5 Sand Filters

Sand filters encompass a wide range of both filtering and flow capacities. The flow capacity depends largely on the physical dimensions of the filter while the filtering capability depends upon the size of the sand grains, depth of the filter bed and flow of water per cubic meter of filtering material.

3.5.17.6 Cartridge Filters

Cartridge fiber filters serve almost the same purpose as sand filter, but can be made to filter out sediments finer than sand. They are made from variety of materials such as paper fiber and fiber glass. Cartridge filters tend to be more useful where the concentration of suspended solids is low. At high concentrations, the cartridge filters must be frequently cleaned because these are often the final filtering component before the water enters the system.

3.5.17.7 Flushing

Flushing individual laterals has been successful in some systems but flushing tends to be an attempt to cure than to prevent plugging Flushing can be done manually, lateral by lateral by opening the end of the line. Flushing valves are available that can be operated using the irrigation water pressure. Any number of laterals can be flushed from one flushing control valve, depending on the system capacity. Flushing velocity should not exceed 0.3 m/sec.

3.5.17.8 Chemical Treatment

Most surface water supplies for irrigation contain a wide variety of particulate matter ranging from very coarse floating organic materials to very fine mineral sediments. Filtration alone is usually not sufficient to prevent emitter clogging. Subsurface waters, namely deep well waters, are normally free of most of these particulate except the troublesome bacteria, predominantly the iron and sulfur bacteria.

Acids

Precipitation of calcium or magnesium carbonate can occur in drip lines and emitters if the water pH is over 7.5. Chemical precipitation can be reduced by adding acids, usually sulfuric or hyperchloric acid, to lower the pH. A reduced pH also aids in bacterial control with chlorination.

Algaecides

Algaecides are commonly used to prevent the growth of algae in water. The most common algaecides are copper sulfate, sulfuric acid and chlorine.

Bacteriacides

Bacterial slimes which form within closed pipelines as well as in open water are a hazard in themselves and as a potential coagulant of fine particles.

Chlorine

Chlorine is the most effective and inexpensive treatment for bacterial slimes. The chlorine is introduced into the system upstream from the filters and can be either sodium hyperchlorite or as chlorine gas. The gas treatment is usually more expensive, more difficult to handle and potentially more hazardous to the operator than sodium or calcium hyperchlorite at similar concentrations.

Oxidants

Sodium hyperchlorite acts as an oxidant for precipitation of troublesome iron in the water. It can be used deliberately to precipitate iron allowing it to settle out before the water is introduced into the system.

Flocculants

Flocculants are used largely in conjunction with settling basins. The flocculation of fine material into large aggregates permits many of them to settle out. With some materials, the floccules may be less dense than the individual particles in which cases the floccules are screened or filtered off.

3.6. Gated Pipe Irrigation

Traditionally, the furrow irrigation for row crops such as cotton, maize, potato and other vegetables is carried out by diverting irrigation water from the source to the furrows through unlined or earthen irrigation channels. More efficient application of water to the furrows, can however, be carried out through gated pipes, which comprise pipes having adjustable gates at appropriate distances allow water to flow from gated pipes to furrows.

Thus, gated pipe irrigation (Fig. 3.22) is an improvement on furrow irrigation, in which the conventional head ditch and siphons are replaced by pipes having gates for delivering irrigation water to the furrows. Irrigation water flows from gates, which are regularly spaced along the pipeline and may be adjusted to the desired rate of flow. Gated Pipes are efficient and advanced means of distributing water to row crops such as cotton, maize and vegetables. They function both as conveyance and distribution systems and consist of relatively large diameter aluminum or plastic pipes, with gates usually corresponding to the furrow spacing.

Currently, there are several types of gated pipes available in the market, which include aluminum, galvanized iron, canvas hose, vinyl plastic reinforced, butyl rubber with nylon reinforcement, black polyethylene and flexible polyvinyl chloride (Armin Company 1089; Hasting Company 1986). However, data on the design and performance can be obtained from the relevant manufacturers. Uniformity of flow is determined by setting the gates precisely, to deliver equal flow rates into furrows. Gated pipes are available in various sizes, capacities and materials with properly designed gates to accommodate different discharge rates and furrow spacing for different row crops.

Fig. 3.22 Gated Pipe Irrigation Method

Osman (2000) mentioned that good design of gated pipes with precision land-leveling improved the water distribution uniformity and saved irrigation water by 12 and 29% in cotton and wheat respectively. Fischbach and Somerhalder (1971) found that with gated pipe irrigation, higher application efficiency and water application efficiency of 91.9% can be achieved, which are influenced by the length and diameter of pipe, orifice diameter, orifice spacing, pressure head and number of outlets operating simultaneously.

Variations in pipe slope, diameter, number of gates, gate area and mean outflow affect uniformity of outflows. The parameters used to determine discharge from the gates along the gated pipes include inside pipe diameter, roughness, and outlet size, gate spacing, and total inflow rates. The friction losses through gated pipe system are computed based on full pipe flow. Most of the flow in gated pipes occurs at Reynolds number between 104 and 106, and Darcy-Weisbach formula is used to calculate the friction loss. Recommended velocities in gated pipes range from 5 ft/s (1.5m/s) to 8 ft/s (2.4 m/s). The gated pipe irrigation operates system under pressure head such as 50 cm or less.

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Chapter 4

Irrigation Scheduling

Irrigation scheduling is the process of predicting the plants' future needs for water, developing irrigation plans for the growing season of a crop or a cropping year and application of irrigation to the crop with required depth of water at right time. The irrigation scheduling allows determination of the current soil-moisture status and future crop water needs based on historical data or field measurement, and guides an irrigator to accomplish the irrigation of a cropped field with pre-defined moisture content in an efficient manner.

4.1. Components of Irrigation Scheduling

The primary objective of irrigation scheduling is the management of water so as to accomplish efficient use of available water supplies and to increase crop production. This would require assessment of needed amounts of irrigation water and their time and means of application to answer the following questions.

- How to apply irrigation?
- Where to apply irrigation?
- How much water to apply during irrigation?
- When to apply irrigation?

4.1.1. How to Apply Irrigation?

It refers to the method of irrigation such as basin, border, furrow, sprinkler or drip irrigation. Since each method has an application efficiency associated with it, which is used to determine the amount of water to be applied to meet the crop water requirement. Therefore, the question 'how to apply an irrigation may be considered built into 'how much to apply'. For example, drip irrigation allows an application efficiency of 85 to 90% as compared to about 60% in case of border irrigation. Whereas the water requirement for a given crop, soil and climate would remain the same under both the irrigation systems, the amount of water to be applied would differ because of the application efficiency associated with each application system. Thus, dividing the net moisture requirement at the root zone by the application efficiency (90% in drip irrigation or 60% in case of border irrigation) would determine different amounts to be applied under each irrigation system. Consequently, the depth of water applied to satisfy a given crop water requirement at the root zone would be less in case of drip irrigation as compared to border irrigation.

In addition, the flow regime of the applied amount has to confirm with the surface conditions of the field. Therefore, the method of application should be properly defined for scheduling irrigation. For example, furrow irrigation or gated pipe irrigation require development of furrows, while border irrigation needs borders to be developed as part of field preparation. Similarly, drip irrigation requires that the design flow rate must remain less than the infiltration rate of the soil and drip irrigation system may be applied to a row crop (on flat or bedded field) while border irrigation system may be used for both the row crop and the non-row crops. Thus, "How to apply irrigation" must also be a part of an Irrigation Schedule.

The crop water requirement at the root zone, remain unaffected no matter which method of irrigation (basin, border, furrow or sprinkler) is used by the farmer. However, the amount to be delivered, in one way or the other, is affected by the method of irrigation. For example, furrow and furrow-bed methods of irrigation, respectively, need 12% and 30% less amount of water as compared to the border irrigation. Sprinkler irrigation may also need less amount, particularly in shallow or sandy soils, and therefore, would require frequent irrigations. Thus, each method of irrigation would influence the needed amount of water as well as the frequency of irrigation to a varying degree.

4.1.1. Where to Apply Irrigation?

Time and amount of water availability at a farm is constrained by the warabandi system, which allows to apply irrigation to only limited cropped area. Therefore, under the given cropping pattern and the area under each crop, the irrigation schedule developed for a farm should indicate which field or a group of fields has to be irrigated during a particular irrigation turn. Thus, the decision on 'where to apply' will depend on the crop growth stage and a combination of the time to irrigate and the irrigation turn. Synchronization of irrigation plan with the availability of water (warabandi) is very important for practical irrigated agriculture.

Irrigation scheduling for individual field does not need to answer 'where to apply' but for many fields of the farm, this question must be answered. Thus, irrigation scheduling is also the management of irrigation water so that it is applied in right amount, at right time, and to the right field. Thus, irrigation scheduling primarily deals with the questions of 'how much to apply', when to apply and where to apply.

4.1.2. How Much Water to Apply?

It pertains to the estimation of the quantity of water that must be delivered to a given cropped field to satisfy the crop water requirements including the consideration for unavoidable losses and leaching requirements, if any. The amount of water to be applied is, in fact a function of consumptive use of the crop (water requirements at the root zone), application efficiency, water holding capacity of soil and the irrigation technique used during irrigation process.

For successful plant growth, both the over irrigation or under irrigation should be avoided. Poor irrigation practices create water logging under excessive application and reduction in crop yield and salinity under stressed conditions. Therefore, only required and measured amounts of irrigation should be applied to crops to replenish the moisture consumed in given period of time bringing the moisture level to field capacity. Irrigations must be scheduled and applied before the moisture contents reach permanent wilting point (PWP). Once the PWP has reached and plants have started wilting, further application of water (i.e. irrigation) may not ensure survival of the plants.

4.1.3. When to Apply Irrigation?

A successful irrigation scheduling means applying right amount of water at right time and maximizing irrigation efficiency by minimizing water losses. An irrigation system is usually designed to deliver a steady flow rate to an irrigated field at a rate sufficient to meet the peak irrigation requirements, particularly, in summer season. If the same amounts are continuously applied during the whole season disregarding the stage of crop, it will lead to excessive water application, particularly during early and late seasons. On the other hand, constrained water supplies during peak season may result in under irrigation and consequent reduction in crop yields.

Once irrigation has been accomplished, the evapotranspiration process continues to lose moisture from soil till the soil moisture reaches PWP. For survival of the crop, soil moisture should not be allowed to drop below the PWP. Preferably, it should not be allowed to reach the PWP because once the plants start wilting, recovery becomes difficult. Therefore, the subsequent irrigation needs to be applied when certain moisture (e.g. 10 to 20%) still remains to support the crop as a factor of safety. Hence a predefined soil moisture level such as Management Allowed Deficit (MAD), and the rate of moisture depletion (depending on the soil texture and weather conditions) would determine the time for next irrigation, which provides an answer to “when to apply an irrigation”. In fact irrigation scheduling can help avoiding both over and under irrigation consequences and facilitate achieving optimum applications with right amount and right time during the growing season.

4.2. Soil - Water – Plant Relationship

The relationship among soil, water and plant characteristics is based on those physical properties of soil, water and plant as one system that deals with storage and movement of water within the soil and to the plant roots in meeting the requirements of crops. Knowledge of soil-water-plant relationship is very important in irrigation scheduling.

4.2.1. Soil

It is porous medium that comprises solid particles (sand, silt, clay, colloids and organic matter), air and water and it can hold and transmit water in different forms such as liquid, solid or gas in vertical and horizontal directions. Soil characteristics include texture, structures, porosity, pore size distribution, organic matter and salinity etc. Various size ranges or groups of soil particles are called soil separates. Classification Systems of Soil Separates and the relevant particle size ranges as defined by the USDA, ASTM and International Society of Soil Science (ISSS) are given in Table 4.1.

Table 4.1 Classification Systems of Soil Separates by Various Standards

S. No.	Soil Separate	Particle Size Range		
		USDA	ISSS	ASTM
1	Very Coarse Sand	2.00 – 1.00	-	>2.00
2	Coarse Sand	1.00 – 0.50	2.0 – 0.20	2.00 -0.42
3	Medium Sand	0.50 – 2.50	-	-
4	Fine Sand	0.25 – 0.10	0.20 – 0.02	0.42 – 0.074
5	Very Fine Sand	0.10 – 0.05	-	-
6	Silt	0.05 – 0.002	0.02 – 0.002	0.074– 0.005
7	Clay	< 0.002	<0.002	0.005- 0.001
8	Colloids	-	-	< 0.001

The soil texture refers to the sizes of the particles where as soil structure refers to the arrangement of soil particles to form granules or remain structure less or single grained. In nature, however, the soil exists with various combination of the soil particles, which form various textural classes such as sand, sandy loam, sandy clay loam, sandy clay, silt loam, silt, clay loam, silty clay loam, silty clay and clay. Using particle size analysis, the procedure to determine the textural classes and their properties are given by the SCS handbook 15.

Structure of soil can be categorized as Single Grained, Massive, Structureless, Blocky, Granular, Prismatic or Columnar etc. The porosity, pore size distribution and hydraulic characteristics of soil such as infiltration, vertical and horizontal hydraulic conductivity and water storage capacity are primarily influenced by the texture and structure of soil, which form the bases of soil-water-plant relationship. The characteristics of liquid influencing its flow through soil may include viscosity, specific gravity and unit weight. The wetting, draining and drying of soil follow the hydraulic principles and the above given soil and water characteristics.

4.2.2. Soil Moisture and Soil Water

Soil moisture is that portion of water that is held in the soil pores by the forces of capillary, adhesion and cohesion. Therefore, soil moisture, can be classified as capillary moisture or hygroscopic moisture. It is present at negative pressure and occupies micro pores of soil mass above watertable.

Soil water, on the other hand, refers to the free form of water present in macro pores and can move under the force of gravity. The dividing line between soil moisture and soil water is watertable. The watertable is considered to be at zero atmospheric pressure. Below watertable, water is at positive pressure that increases with the depth of water. Above watertable, it is at negative pressure, which increases with the height above watertable. However, capillary fringe constitutes both soil moisture and soil water as the soil in this range is saturated, although it lies above watertable. The soil water and soil moisture are inter transferable forms under different hydraulic conditions, e.g. rising of soil water by capillary action changes it to the form of soil moisture and vice versa.

4.2.3. Types of Soil Moisture

Moist soil may contain moisture as hygroscopic or capillary in the micro pores or as water in the macro pores. These types of soil moisture are explained below.

4.2.3.1 Hygroscopic Moisture

Frevert et al. (1981) defined hygroscopic moisture as water held tightly to the surface of soil particles by forces of adsorption. Consequently, this type of moisture is adsorbed over the surface of soil particles in the form of a thin film and is held by the force of adhesion and thus, not available to the plants for growth.

4.2.3.2 Capillary moisture

This component of moisture is located within the capillaries of soil or inter-connected smaller pores of soil, and is held by the force of surface tension (Frevert et al. 1981). Smaller the pores size and moisture content, greater is the force of tension with which moisture is held in these pores. Capillary moisture cannot be drained by gravity or drainage system. This moisture makes the major components of moisture available to plants.

4.2.3.3 Gravitational Water

It occupies macro pores of soil and can flow under the force of gravity. The gravitational water present in the soil, as a result of irrigation or rainfall, cannot be held in the soil and moves down to join the groundwater under natural drainage process. This component of soil water is generally not available to the plants as it leaves the root zone within few days after irrigation.

4.2.3.4 Water Vapor

Water in the vapor form is present in the soil pores. During an irrigation process, as water infiltrates into the soil, first of all the micro pores are filled with water because of negative pressure or suction existing in the micro pores. Once the micro pores are saturated, water moves towards the macro pores. This is followed by the runoff on the surface of the field as well as gravitational movement of water to the lower layers of soil. Thus, during irrigation, as the water advances, part of the applied water keeps moving downward as gravitational water. This phenomenon can be seen when rain drops fall on a dry soil surface. Both the hygroscopic and capillary moisture are under negative pressure whereas gravitational water is under positive pressure in the soil.

4.2.4 Soil Moisture Limits for Plant Growth

When irrigation water is applied to the field, its movement in the soil continues in the form of gravitational water and capillary moisture till the soil gets saturated. At the same time, the plant roots extract water and transpire. Similarly, during the drying process the soil loses moisture. The moisture status in the soil with respect to its availability to the plants, defines certain limits of moisture, which may be defined as Field Capacity and Permanent wilting point as explained below.

4.2.4.1 Field Capacity (FC)

The soil moisture content at which all the gravitational water has been drained out of root zone. Usually, the soil moisture content at 1/3 atmosphere tension is considered to be the field capacity that is usually attained approximately 2 to 3 days after irrigation.

4.2.4.2 Permanent Wilting point (PWP)

The moisture content at which, the plant roots cease to extract soil moisture. Usually, the soil moisture contents at 15 atmospheres are considered to be thePWP.

4.2.4.3 Available Moisture (AM)

The difference in moisture content between FC and PWP is considered the available moisture (AM). This is the moisture, which plants can utilize without going through stress and is given by the equation 4.1.

$$\mathrm{AM} = \mathrm{FC} - \mathrm{PWP} \tag{4.1}$$

Where:

AM = Available moisture

FC = Field Capacity

PWP= Permanent Wilting Point

4.2.4.4 Net Available Moisture (NAM)

It is the difference in moisture content between the FC and pre defined level of moisture content when irrigation is to be applied under MAD (Fig. 4.1a) or between the FC and actual moisture content at the time of irrigation (Fig. 4.1b). It is equivalent to the amount of moisture consumed by the plant, which must be replenished during irrigation application as shown in Fig. 4.1 b.

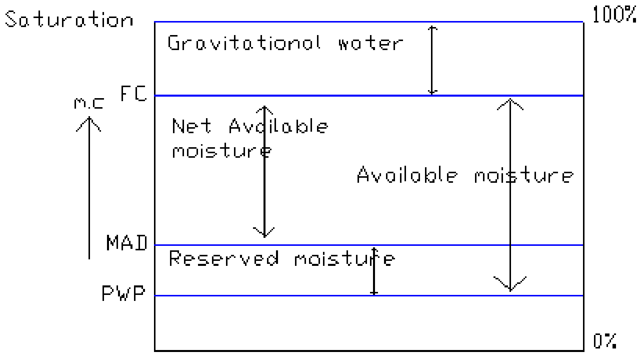


Fig. 4.1a Soil- Moisture- Relationship Under Management Allowed Deficit (MAD)

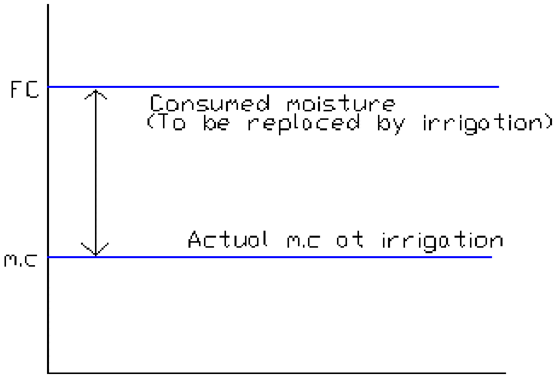


Fig. 4.1b Soil-Moisture Relationship Under Sampled M.C

4.2.4.5 Management Allowed Deficit (MAD)

As the name shows, it is the deficit in moisture content below field capacity that can be allowed before next irrigation. In other words, it is the precondition that is imposed on the maximum utilization of moisture content by the plant in terms of % of available moisture content. For example, a management decision is taken that next irrigation must be applied when 85% of available moisture is consumed or 15% of available is present in the soil at the time of irrigation. Such a condition may also be imposed in terms of not allowing the moisture to drop below a given moisture content. For example, next irrigation must be applied before the moisture content reduces to 15% by weight provided it is greater than PWP as shown in Fig 4.1a and, 4.1b.

4.2.4. Soil Moisture Characteristics Curve

The plotted variation of volumetric moisture content with tension (-ve pressure) in the soil gives rise to moisture characteristic curve. As shown in Fig. 4.2 the moisture varies from 0% (dry soil) to 100% (saturation). The volumetric % moisture content at saturation is equivalent to porosity of soil when all the pores are filled with water. Zero moisture tension (or zero pressure) corresponds to water table, where the surface of groundwater is exposed to the atmosphere. Below water table, the water in the soil is at positive pressure, where as above watertable, it is at negative pressure. As the height above watertable increases, the moisture tension (-ve pressure) increases. However, for certain height above water table, the soil remains practically saturated with moisture at negative pressure, which is called capillary fringe. Above this height, the moisture content reduce sharply and meet FC at 1/3 atmosphere tension. Beyond this point the slope of the line becomes very steep showing a small decrease in moisture content with soil moisture tension till it approaches the PWP at 15 atmosphere tension. There is a unique moisture characteristic curve for a given textural class of soil, which is used to assess the moisture characteristics of soil such as FC, PWP, moisture content at saturation, porosity and moisture content at a given soil moisture tension.

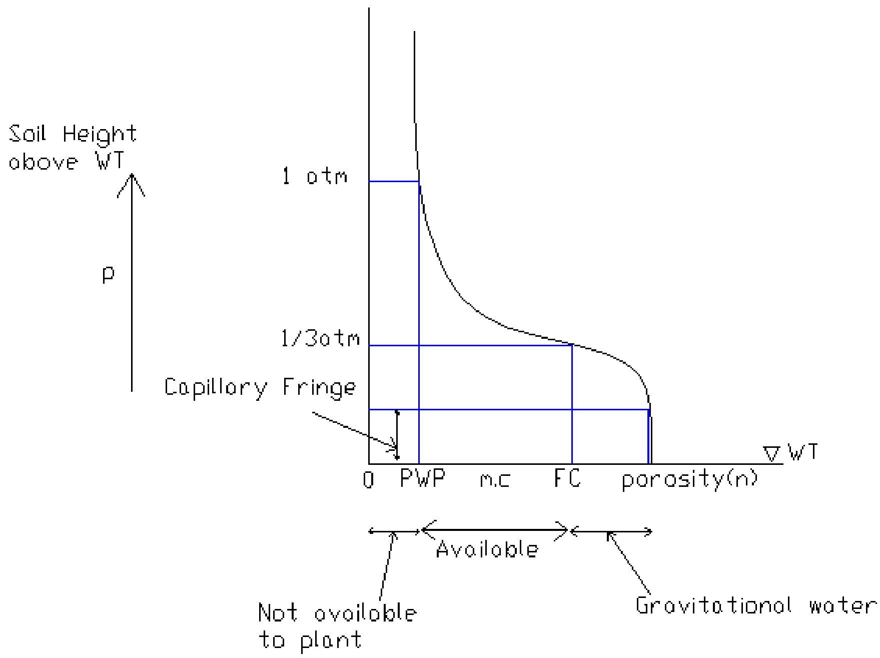


Fig. 4.2 Soil Moisture Characteristics Curve

The soil moisture characteristic curve is generated with the help of pressure membrane apparatus in the laboratory. The purpose of developing the soil moisture characteristic curve is to determine the moisture characteristics of a given soil such as FC, PWP, moisture content at a given tension, porosity and moisture content at saturation etc. Average physical properties of soil pertaining to the moisture characteristics of various soils are given in Table 4.2.

Table 4.2 Physical Properties of Soil

Soil Texture	Bulk Density (g/cm^3)	FC (% by weight)	PWP (% by weight)	Available Moisture (% by weight)
Sandy	1.65	9	4	5
Sandy loam	1.50	14	6	8
Loam	1.40	22	10	12
Clay loam	1.35	27	13	14
Silty clay	1.30	31	15	16
Clay	1.25	35	17	18

4.2.5. Soil Sampling Equipment

A variety of soil sampling equipment have been developed by the professionals to serve a variety of purposes such as measurement of soil moisture, nutrients, organic matter and salt concentration etc. The equipment used to collect the soil samples, both disturbed and undisturbed, include Auger, Slotted Sampling Tube, Screw Sampler, Bulk Density Sampler etc. as shown in Fig. 4.3. Each of these sampling equipment are explained below.

Fig. 4.3 Soil Sampling and Handling Equipment

4.2.6.1 King Tube

It comprises a long tube of desired diameter such as 2 to 3 cm with lower edge sharped and shaped to cut soil sample of diameter smaller than the internal diameter of the tube to facilitate the movement of soil sample through the tube. Soil sample is collected through the upper (other) end of the tube, which is open but supported with a pipe handle. Such tube sampler is provided with a hammer that moves over the pipe to strike against the collar welded to the pipe about 60 cm below the top end. The sample moves by the fall energy of the hammer. The length of the sampler may be as much as 8 feet to collect samples from deeper depth.

4.2.6.2 Slotted Sampler

It comprises a metallic tube of appropriate diameter such as 2 cm with a sharp cutting edge at the lower end and a handle at the top. The sampling tube is slotted for a part of length to facilitate ejection of collected undisturbed soil sample. The cutting edge of the sampler is formed and shaped in such a way that allows taking sample of diameter smaller than the internal diameter of the tube for ease of ejection of soil sample. This sampler can be used to determine dry bulk density of soil by cutting a predetermined length of the soil sampled in the slotted tube followed by drying of the sample to find dried weight and determining volume of the soil sample. Dry bulk density of soil is obtained by dividing the dry weight by the volume of sample.

4.2.6.3 Auger

The auger can be used to collect loose soil samples or undisturbed soil cores, mostly of diameter greater than 2 cm. As auger is rotated, the two cutting bits cut quickly into the soil and force the cut soil up into the interior of cylinder bucket. As the auger is full, it is lifted out of the hole and inverted or beaten with a hammer so that the collected sample slips out of the open end of the bucket. For deeper samples, a number of extensions/ attachments of handle can be made to cut at deeper depth. The bucket of the auger may be closed or open to slip the sample. For sampling in mud or sand, specialized designs of auger are used. Mud auger has an open bucket to prevent clogging and to increase the ease of emptying in sticky wet soils. Cutting bits die-formed from abrasion resistant steel in sizes of 2,3 or 4 inches diameter with 4 feet handle extensions are available in the market. The augers are also used to make bore holes for taking groundwater samples or installing hand pumpes.

4.2.6.4 Screw Sampler

This type of sampling equipment resembles to the wood drilling tool of a carpenter. It has a long handle with a spiral cutting edge ending at a sharp tip to initiate drilling into the soil. The sampler is rotated with appropriate pressure to move into the soil to the desired depth. The soil sample is stored into the spiraled space and is recovered after pulling the sampler gently out of the soil. Care must be taken to avoid sampling though plant roots and limiting the sample to incremental depth of 15 cm. Drilling for deeper depths in one attempt may lead to difficult pulling out. This sampler is generally used for collecting disturbed samples of smaller diameter and cannot be used for undisturbed soil samples.

4.2.6.5 Sample Handling Tools

These include scoop, spatula, knife, rubber hammer and tray as shown in Fig. 4.3, which are used to handle the soil samples collected by the samplers. Rubber hammers help to break the clods and granules without crushing the individual soil particles.

4.2.6. Soil Moisture Measurement

Soil moisture measurement is a pre-requisite to assess the amount of irrigation water to be applied to a field. Therefore, accurate measurement will lead to more efficient irrigation. There are a number of techniques and equipment, which may be field and laboratory oriented, for measuring the soil moisture such as gravimetric, neutron probe, gypsum block, tensiometer, speedy moisture tester and sun drying etc., each having certain limitations and degree of accuracy. Out of these, the most accurate one is considered to be the gravimetric method. Because of limited scope of the book, only a few of these are briefly discussed below.

4.2.7.1 Gravimetric Method

The gravimetric method involves weighing the collected soil sample (wet weight), followed by drying in the oven at 105°C for 24 h and then determining the dried weight of soil. The moisture contents are calculated using the following equation.

$$\text{Moisture Content (\% on weight basis)} = \frac{(W_w - W_d)}{W_d} \times 100 \qquad (4.2)$$

Where:

MC =% Moisture content on weight basis

Ww = Weight of wet soil (as collected in the field)

Wd = Weight of dried soil (air cooled after oven drying)

4.2.7.2 Tentiometer Method

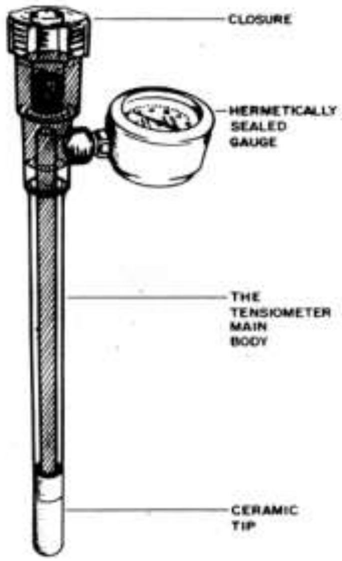
A tentiometer (Fig. 4.4) operates on the basis of soil moisture tension or negative pressure at the given depth where the tensiometer blub is installed. The soil moisture tension or negative pressure measured by the tensiometer ranges from 0 to -100 kPa (i.e. 0 to -1 bar). Tensiometer fundamentally acts in a similar fashion as a plant root, measuring the force that plants have to exert to obtain moisture from the soil. As shown by the soil moisture characteristics curve (Fig. 4.2), the lesser the moisture content, greater would be the tension in the soil. In similar way, as the soil dries the water is lost from the tensiometer via a ceramic cup, the loss of water creates a vacuum in the tensiometer and is reported as a negative pressure reading, i.e. drier the soil, higher is the negative pressure reading and vice versa. The tentiometer dial indicates the tension (in bars) as well as the % moisture on wet basis. The dry soil will read 0 and saturated soil 100% on the gauge. Measurement of soil moisture with tensiometer may be accomplished by the following procedure.

- Saturate the gypsum bulb with water.

- Open the cup or stopper and fill the tensiometer chamber with water. Make sure that water chamber does not contain any air bubble then tighten the cup /stopper so that no air can enter the tensiometer.
- Install the tensiometer in a prepared hole in the soil placing the bulb at a depth where moisture contents are desired. Before installation, the tensiometer must read zero with saturated bulb.
- While installing the tensiometer, make sure that the space between the tensiometer and surrounding soil is filled with soil i.e. the bulb is completely in touch with the soil.

While maintaining the moisture equilibrium between the moisture in the soil and that in the bulb, water will move from the tensiometer chamber to the soil creating a negative pressure or tension in the tensiometer, which would be reflected by the movement of tensiometer dial needle. Record the dial reading at given elapsed time. Increasing dial reading (soil moisture tension) would indicate decreasing soil moisture with time and vice versa. The tensiometers cannot be used to measure soil moisture tensiom greater than1bar as air would enter into the water chamber if tension increases further. Tensiometers operate the best below 0.8 atmosphere tension.

Fig. 4.4 Tensiometer



4.2.7.3 Gypsum Block

Gypsum Block operates on the principal of resistance to the flow of current across a set of electrodes (wire gauze) embedded in the gypsum block. The flow of current is inversely proportional to resistance and directly proportional to the soil moisture. The free ends of electrodes are connected to the Bouyoucos moisture meter as shown in Fig. 4.5. Before installation, the gypsum block is saturated with water and the meter is calibrated against the saturated soil moisture condition by pressing the knob shown as (press to calibrate). At that moment, the needle of the meter must read 100%, which is set by the adjustment knob. After calibration, a hole is drilled in the soil and the gypsum block is installed at desired depth of soil where moisture contents are needed. Refill the hole with the excavated soil and press normally to make sure that the block is firmly connected to the surrounding soil to maintain moisture equilibrium between the moisture content of soil and the gypsum block itself. At this point, press the button to read the soil moisture content or resistance to the current flow in the soil on the dial /gauge of the Bouyoucos Moisture Meter. The meter shows both the scales (Fig. 4.5).

Fig. 4.5 Gypsum Block with Bouyoucos Moisture Meter

4.2.7.4 Speedy Moisture Tester

A Speedy Moisture Tester (Fig. 4.6) comprises a chamber for mixing soil sample with the calcium carbide powder that reacts with soil moisture to produce gas pressure, which is proportional to the moisture present in soil. The dial reads % available moisture in the given soil sample. The procedure of measuring soil moisture with Speedy Moisture Tester can be summarized as given below.

A specified weight of soil sample, equivalent to the counter weight of the balance provided with Speedy Moisture Tester, is taken in horizontally oriented mixing chamber. A spoonful of carbide powder is taken to place in the mixing chamber such that it does not mix with the soil sample before tightly closing the chamber. After the chamber is tightly closed, shake it to thoroughly mix the soil sample with the carbide powder. The moisture contents are read on the speedy dial as % available moisture. The dial may be calibrated against different moisture content values obtained by gravity method to yield a calibration curve that may be used to find the soil moisture of a given soil sample. In most of the cases, the procedure to use the speedy moisture tester is pasted on the speedy box as shown in the Fig. 4.6.

Fig. 4.6 Speedy Moisture Tester with Balance and Powder

4.2.7.5 Neutron Probe Method

A Neutron Soil Moisture Meter (Neutron Probe) uses radioactive material for measuring soil moisture. It contains an electronic gauge, a connecting cable and a source tube containing both the nuclear source and detector as shown in Fig. 4.7. An access tube is installed in the ground and the source tube is lowered into the tube to the desired depth of soil moisture measurement. Access tube material can affect counting rate of neutrons. Highest rate can be achieved with aluminum, and lowest with PVC. However, accurate calibration curves can be developed with PVC, which is less expensive and does not get distorted in the field installation.

The Neutron Probe technique is based on the measurement of fast moving neutrons (generated from an Americium 241/Beryllium source) that are slowed in the soil by an elastic collision with existing hydrogen particles in the soil. Hydrogen (H^+) is present in the soil as a constituent of soil organic matter, clay minerals and soil moisture. The only form of H^+ that will change from measurement to measurement is that existing in soil moisture. Therefore, any change in the counts recorded by the

Neutron Probe is due to a change in the moisture with an increase in counts relating to an increase in moisture content.

Lowering the source tube in the soil causes high speed moisture to colloid with hydrogen atoms in water and soil, which results in loss of energy and creation of low energy or slow neutrons. Some slow neutrons are reflected back to the source tube and counted by the neutron detector and transmitted to the guage that contains a microprocessor, which shows reading of volumetric soil moisture content.

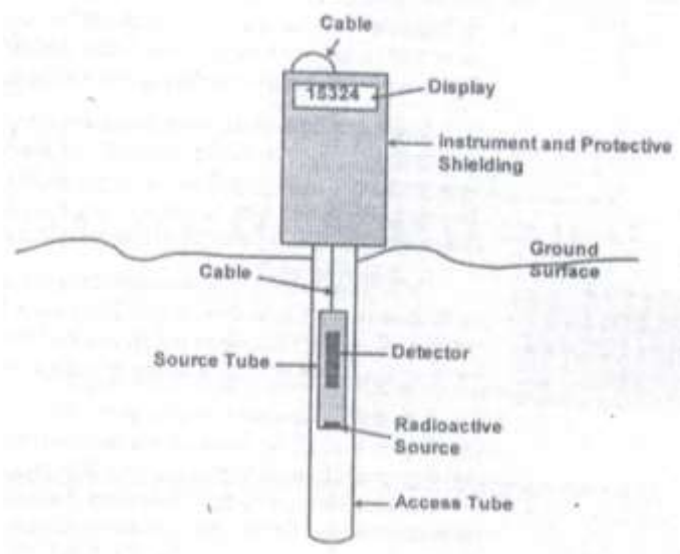


Fig. 4.7 Neutron Probe for Moisture Measurement

Two inch diameter aluminum pipe, class 125 PVC pipe or electrical metal tubing can be used for access tube. Aluminum affects the readings the least while PVC affects the most beause slow neutrons are absorbed by clorine in PVC. In high watertable areas, the bottom of the access tube must be sealed to keep water out.

Installation of Neutron Probe

Select a site and install an access tube in the soil to a depth of about 6 inches deeper than maximum measurement depth. Gap between the access tube and soil should be minimum to keep surface water away from entering down the side of access tube. Cover the top of the access tube. In the field, aluminium tubes are inserted into the soil and stopped to minimise water entry. Readings are taken at depths down the profile (e.g. 20, 30, 40, 50, 60, 70, 80, 100 and 120 cm) with a sixteen second count. The three aluminium tubes are then averaged to counter the effect of spatial variability reducing the value of the measured moisture content data. Measurements may taken two to four times a week and data analyzed for interpretation. Use of the Neutron Probe technique for unsaturated zone monitoring has been utilized to determine contamination and leak detection along specific transport pathways, and to monitor disposal of effluent with the NP technique.

Advantages of Neutron Probe Moisture Measurement

According to Schmugge et al. (1980), the advantages of the neutron probe include:

- Soil moisture can be measurcd regardless of its physical state.
- Average moisture contents can be determined with depth.
- Probes can be interfaced for automatic downloading of stored data.
- Temporal soil moisture changes through seasons can be easily monitored.
- Rapid changes in soil moisture can be detected.
- Readings are related directly to soil moisture.
- Measurements can be made repeatedly and non-destructively at the same site.
- More reliable measurements of soil moisture than can be obtained from generally smaller gravimetric samples.
- Neutron probes are capable of measuring soil moisture even under frozen soil conditions.

Disadvantages of Neutron Probe

- As the device uses radioactive source, it is constrained by the inspection requirements, licensing, storage and operator training.
- Reliable measurements at shallow depths, less than 6 inches, may not be possible because some neutrons escape from the soil surface into the air instead of being detected.

4.3. Infiltration

It is the property of soil, which is characterized by the ease of entrance of water into the soil. It is the process by which water on the ground surface enters the soil. Infiltration is the passage of water into the soil surface and is distinguished from percolation, which is movement of water through the soil profile (Frevert et al. 1981). Infiltration rate into the soil is a measure of the rate at which soil is able to absorb rainfall or irrigation water. It is measured in inches per hour or millimeters per hour. The rate decreases with time as the soil becomes saturated. If the precipitation rate exceeds the infiltration rate, runoff will usually occur unless there is some physical barrier. It is related to the saturated hydraulic conductivity of the near-surface soil. The rate of infiltration can be measured using an infiltrometer. As the soil is a porous material, the water after entering into the soil may move laterally or in vertical direction depending on the textural layers and profile of the soil.

4.3.1. Infiltration Rate

The infiltration rate is the velocity or speed at which water enters into the soil. It is usually measured by the depth of the water entering into the soil per unit of time e.g. mm/min, mm/h or mm/day and inches/day etc. Infiltration rate can be measure by installing a scale or a gauge vertically over the soil surface with standing water and noting the drop of water surface with time. The method is explained further in section 4.3.3.

4.3.1.1. Basic Infiltration Rate

When water is applied to the dry soil, it initially infiltrates rapidly due to high degree of suction in the soil. This is called the initial infiltration rate. As more water moves into the soil, it satisfies the negative pressure as well as replaces the air in the pores, further rate of entrance of water slows down and eventually reaches a steady rate, which is called the basic infiltration rate that is also considered equivalent to the hydraulic conductivity, which is defined as the flow rate per unit hydraulic gradient per unit cross sectional area. The infiltration rate depends on the texture, structure and stratification of soil profile. Table 4.3 gives basic infiltration rate of some of the soil types commonly encountered in the field.

Table 4.3 Basic Infiltration Rates for Various Soil Types

Soil type	Basic infiltration rate (mm/h)
Sand	Greater than 30
Sandy loam	20 - 30
Loam	10 - 20
Clay loam	5 - 10
Clay	1 - 5

4.3.2. Factors Influencing Infiltration Rate

Infiltration is governed by the forces of gravity and capillary. While mega pores possessing positive pressure, are filled by the forces of gravity replacing air, the smaller pores possessing negative pressure, offer greater resistance to gravity. Smaller pores exert capillary forces to the infiltrating moisture through capillary action against the force of gravity. The gravitational force becomes effective after the capillary forces are satisfied. These phenomena can be observed in rain drops falling on a dry soil. The first few drops are absorbed by the micro pores of soil under the capillary forces, followed by filling of mega pores under the force of gravity, which is further followed by surface runoff.

The rate of infiltration is affected by soil characteristics including ease of entry, storage capacity, and transmission rate through the soil. The soil texture, structure, vegetation types and cover, water content of the soil, soil temperature and rainfall intensity all play a role in controlling infiltration rate. Coarse grained sandy soils have higher infiltration rates as compared to the fine grained soils. Vegetation protects the soil from pounding rainfall, which can otherwise close the pores between soil particles. In addition, the root system loosens the soil and makes it more porous for the infiltrating water. That is why forested soils have higher infiltration rates as compared to the bare soils. The top layer of leaf litter that is not decomposed protects the soil from the pounding action of rain, without this the soil can become far less permeable.

The process of infiltration can continue only if there is room available for additional water at the soil surface to enter. The available volume for additional water in the soil depends on the porosity of the soil and the rate at which previously infiltrated water can move downward from the surface through the soil. The maximum rate that water can enter a soil under a given set of condition is the infiltration capacity of soil. Thus, if the oncoming water at the soil surface is less than the infiltration capacity, all of the water will infiltrate. If rainfall intensity at the soil surface occurs at a rate that exceeds the infiltration capacity, ponding begins and is followed by runoff over the ground surface, once depression storage is filled. This runoff is called Horton overland flow.

4.3.3. Measurement of Infiltration Rate

Although, the infiltration rate in the field can be measured by ponding water on a piece of land and using a vertical scale and a watch by observing the drop of water surface with time, yet the most common method to measure the infiltration rate in the field uses single or double ring infiltrometrs (i.e. Single or Double Cylinders). The double ring infiltrometer is shown in Fig. 4.8, which requires the following equipment to run the infiltration test.

- Shovel
- Hammer
- Timber or Steel Plate
- Watch /Clock
- Cylinder
- Bucket
- Piece of Cloth
- At least 100 litres of water
- Double Ring Infiltrometer with inner cylinder of 30 cm diameter and outer cylinder of 60 cm diameter
- Measuring rod or a ruler graduated in mm or inches

The purpose of inner or smaller cylinder is to facilitate measurement of infiltration rate through a meter rod while the outer cylinder contains water to provide buffer area that limits lateral movement of the infiltrating water and forces it to move in vertical direction. In case, only one or inner cylinder is used, a bund could be made to replace the outer cylinder and prevent lateral water flow. Installation of two rings and meter rod along with infiltration measurement procedure are explained below.

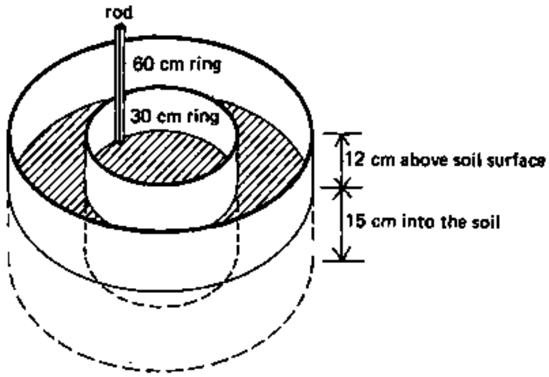


Fig. 4.8 Double Ring Infiltrometer for Measuring Infiltration Rate

- Hammer the 30 cm diameter ring at least 15 cm into the soil. Use the timber or a steel plate on the top of ring to protect the ring from damage during hammering. Keep the side of the ring vertical and drive the measuring rod into the soil so that approximately 12 cm is left above the ground.
- Hammer the 60 cm ring into the soil or construct an earth bund around the 30 cm ring to the same height as that of the ring and place a piece of cloth inside the infiltrometer to protect the soil surface from splashing when pouring in the water.
- The water in the bund or within the two rings is to prevent a lateral spread of water from the infiltrometer. Record the clock time when the test begins and note the water level on the measuring rod.
- After 1-2 minutes, record the drop in water level in the inner ring on the measuring rod and add water to bring the level back to approximately the original level at the start of the test. Record the water level. Maintain the water level outside the ring similar to that inside.
- Continue the test until the drop in water level is the same over the same time interval. Take readings of clock time and depth frequently e.g. every 1-2 min) at the beginning of the test, but extend the intervals between readings as the time goes on e.g., every 5, 10, 20 and 30 min.
- Note that at least two infiltration tests should be carried out at a site to make sure that the correct results are obtained.
- Draw a graph or curve between the time (x scale) and depth (y-scale). Rate of infiltration at any time interval on the curve, with elapsed time (dt) and infiltrated depth (dy) can be determined as dy/dt.

4.3.4. Irrigation Water Movement in Soil

As a cropped field is irrigated, the infiltration process begins and soil becomes saturated after satisfying the micro pores followed by maga pores of soil. After the termination of irrigation, the water continues moving downward. Consequently, the gravitational water is drained out of the root zone and joins the watertable that may take 2 to 3 days depending on the textural type of soil. Sandy soils usually drain earlier as compared to loamy or clayey soils. After draining of gravitational water, the soil is considered at FC, which constitutes mainly the capillary moisture that is available to the plants till PWP is reached. Thus, the available moisture (AM) is the difference between FC and PWP. Although, the plant can easily utilize gravitational water during draining period, yet it is quite shorter than the total period of moisture extraction before next irrigation. Therefore, major component of moisture available to the plants that sustains plant life is between FC and PWP.

4.4. Irrigation Management Alternatives

Irrigation management refers to the assessment of depth of water to apply and the decision- when to apply irrigation to the field. Before determining the amount or depth of water to apply to a given field, the water needed at the root zone of crop is desired. It is, in fact, equivalent to determining the deficiency or deficit of water at the root zone of crop before next irrigation. The deficient moisture must be replenished with irrigation water to bring the moisture content to the FC in order to fully meet the crop water requirements. While planning for irrigation scheduling, the following alternative decisions may be taken by the farmer to accomplish irrigation, which may be called as irrigation management alternatives.

1) Irrigation at PWP: Allow the crop to consume total available moisture from FC to PWP and apply next irrigation when the soil moisture is approaching PWP i.e. apply irrigation just before PWP.
2) Irrigation at instant moisture content: Consistently sample the soil for moisture determination and apply irrigation on the day of sampling, based on the sampled moisture content, i.e. any day before the moisture reaches PWP.
3) Irrigation at Management Allowed Deficit (MAD): Irrigate on the basis of predetermined moisture level such as the MAD i.e. a given moisture level of soil or a given degree of moisture consumption. For example, applying irrigation when 80% of available moisture has been consumed or 20% of available moisture remains in the soil.

Each of these options have been discussed below with example calculations. These determinations may be demonstrated by assuming the following soil moisture characteristics data.

Field Capacity of soil	= 20% by weight
Permanent Wilting Point	= 8% by weight
Rootzone Depth (D)	= 80 cm
Bulk Density (BD)	= 1.6 g/ cm^3
Management Allowed Deficit	= Irrigation at 80% of available moisture has been consumed
Irrigation Application Efficiency	= 60%

4.4.1. Irrigation at PWP

The soil is saturated after the accomplishment of irrigation in the field. Start sampling for the moisture content at the desired depth when the soil allows the researcher to enter the field or its moisture content are at the FC. The soil samples can be taken at prescribed intervals of time such as daily, weekly or any number of days interval. Preferably, take soil samples on daily basis and determine moisture content. The soil moisture content will continue decreasing with time. Apply irrigation on or before the day when soil moisture approaches PWP. However, the decision depends on the availability of water. It is advisable not to delay irrigation beyond PWP as the plants would start wilting, which may cause irreversible death of plants. Using the above given assumed data, the example calculations for determining depth of water needed at the rootzone at PWP are as under.

Available moisture by weight (AM_w) = FC – PWP

= 20 - 8

= 12% by weight

Percent Available moisture by volume (AM_v) = (% AMw) x Bulk Density

= 12×1.6

= 19.2% by volume

Depth of water needed at root zone at PWP = ($\%AM_v$. × D)/ 100

= (19.2 × 80) /100

= 15.36 cm

Divide the depth of water, determined above, by the application efficiency to determine the depth of water to be applied in irrigation.

Depth of water to be applied = 15.36/ 0.6

= 20.56 cm

Considering root zone depth of 40 cm, the depth D to be applied would be :

= 10.28cm

4.4.2. Irrigation at Instant Moisture Content

The farmer may decide to irrigate the field on a day when irrigation water is available under warabandi system. Determine the soil moisture content, find the deficit by subtracting the last day moisture content from the water holding capacity or FC of the field and convert the % deficit moisture content to depth of water required at the root zone.

Generally, it is difficult to keep track of PWP, even if consistent soil sampling is carried out. Moreover, planning irrigation at PWP may put the crop to risk as revival of plant to normal growth after it approaches PWP cannot be assured. Therefore, irrigation must be applied well before the soil moisture reaches the PWP.

Considering the actual moisture content determined through soil sampling before PWP i.e. the moisture content, assessed on the day of irrigation, the moisture deficiency at the root zone may be determined as under:

Instant moisture content before PWP (Assumed) = 11% by weight

Moisture deficiency on the day of last sampling = (F.C. – Actual moisture) or 20-11

= 9% by weight

Depth of water consumed at instant sampling = 9/100 ×B.D × D

= 0.09 × 1.6 × 80

= 11.52 cm

Thus, moisture deficiency at instant moisture content =11.52 cm.

Divide the moisture deficiency, determined above, by the application efficiency to find the depth of water to be applied.

Depth of water to apply in irrigation (before PWP and MAD) = 11.52 / 0.6

= 19.2 cm

Considering Root Zone Depth of 40 cm, the depth D to be applied would be:

= 9.6 cm

4.4.3. Irrigation at Moisture Allowed Deficit

The MAD refers to the decision that irrigation would be applied when the crop had consumed a given % of moisture in the soil, e.g. 80% of the available moisture or it would be applied the day when the existing soil moisture contents are 20%. Such a management decision requires that water must be made available either from canal or from a tubewell on the day of MAD. However, water may or may not be available on the day of MAD. Under such situation, MAD needs to be adjusted to the availability of irrigation water.

Irrigation scheduling demands more planned irrigation events than applying irrigation instantly at sampled moisture content. The MAD permits preplanned time of irrigation by knowing deficiency in terms of pre determined % age of total available moisture.

Once FC, PWP and AM are known, irrigation may be applied for a given management allowed deficit in conjunction with consistence sampling that would enable the irrigator to fix time of irrigation well ahead of irrigation day as explained by the following example.

Assuming MAD is 80% of the available moisture, the moisture deficiency or the depth of water needed at the root zone may be determined as given below:

Available moisture = FC- PWP

= 20 – 8

= 12% by weight

For 80% Available Moisture consumed at MAD = Available x 80/100

= 12 ×80/100

= 9.6% by weight

As the consumed moisture has to be replaced by irrigation, the depth needed at the rootzone can be calculated as:

Depth needed at root zone = (FC - PWP) × 80/100 ×B.D × D (4.3)

= (20-8) × 0.8 × 1.6 × 80

=12.29 cm

Depth of water to apply in irrigation = 12.20 / 0.6

= 20.37 cm

Considering Root Zone Depth of 40 cm, the depth D to be applied would be:

= 10.19cm

Thus, depth needed at the rootzone before irrigation under each of the management alternatives can be summarized as:

At PWP = 15.36 cm

At instant sampling = 11.52 cm

At MAD = 12.29 cm

4.5. Water Requirements of Crop (WR)

The main objective of irrigation is to provide plants with sufficient water to prevent stress that may cause reduction in yield or poor quality of harvest. The other advantages associated with irrigation are given in Chapter 1.

The crop water requirement is influenced by the crop, soil and climatic factors. The crop and soil factors include the type of crop, root zone, depth, growth stage, crop density, soil texture, water holding capacity and current soil moisture status. The climatic factors include the air temperature, solar radiation, wind velocity, humidity and elevation etc. As defined by Doorenbons and Pruitt (1977), water requirement is the depth of water needed to meet the water loss through evapo-transpiration (Et) of a disease free crop growing in large field under non-restricting soil conditions including soil water and fertility and achieving full potential under the given growing environments. According to this definition, using the water balance approach, WR may be determined as:

R = Et – Pe + other beneficial uses (4.4)

Et = Evapotranspiration during the period under consideration

Pe = Effective precipitation

Other beneficial uses include leaching requirements, cooling of crop or protection against frost etc.

- Crop water requirements can be determined using the following methods.
- Water Balance Equation (4.4) as given above.
- Using empirical equations, which utilize climatic data to estimate evapotranspiration for the given crop, soil and growing period.
- Collecting soil samples from a cropped field or using moisture meters to assess moisture content as the growing season continues. The total moisture consumed during the growth of crop would be the crop water requirement.

4.5.1. Evapotranspiration (ET)

It is the combined process (evaporation + transpiration) by which water is lost from earth's surface to the atmosphere in a given crop field. It includes evaporation of water from soil and plant surfaces plus transpiration of water through plant tissues expressed as the latent heat transfer per unit area or its equivalent depth of water per unit area.

4.5.2. Potential Evapotranspiration (ETp)

When evapotranspiration rate of a particular crop is not limited by soil moisture availability and when crop is growing with full effective cover, it is called potential evapotranspiration (ETp) expressed as function of climatic factor only. Jensen (1974) defined that the potential evapotranspiration is the evapotranspiration from alphalpha crop, 30-50 cm in height (full grown), with unrestricted availability of water.

Doorenbos and Pruitt (1977) defined ETp as "the rate of evapotranspiration from an extensive surface of 8-15 cm high green grass cover the uniform height actively growing and completely shading the ground and with no shortage of water". Potential evapotranspiration is usually defined for a reference crop such as (alphalpha or grass) and is regarded as a function of climatic factors only.

4.5.3. Maximum Evapotranspiration (ETm)

The potential evapotranspiration as defined above refers to the reference crop (alphalpha or grass) whereas maximum evapotranspiration refers to the given crop other than the reference crops such as wheat, cotton, sugarcane etc. Etm is given by the equation 4.5.

$$ETm = ETp \times Kco \tag{4.5}$$

Where:

ETm = maximum evapotranspiration for a given crop

ETp = Potential evapotranspiration for a reference crop

Kco =Reference crop coefficient

4.5.4. Actual Evapotranspiration (ETa)

Actual evapotranspiration or crop water requirements of a given crop is influenced by crop growth stage and soil moisture stress. The crop growth stage is taken care of by the crop coefficient (Kco) where as the effect of moisture stress present in the soil during the period of evapotranspiration is reflected by stress factor (Ks) in the given equation. The actual evapotranspiration is therefore, given by the equation:

$$ET_a = ET_m \times K_s \tag{4.6}$$

or ETa = ETp. Kco > Ks

and
$$K_c = K_{co} \times K_s \tag{4.7}$$

Thus
$$ET_a = ET_p \times K_c \tag{4.8}$$

Where Kc is called crop coefficient expressed as ETa / ETp. An empirical equation to estimate Ks has been represented by Kincaid and Heerman (1974).

4.5.5. Consumptive Use

The consumptive use of crop (CU) includes both evapotranspriation and the amount of water used by the plants for building its tissues. The amount used by the plants for their development is, however, less than 1% of total Et and therefore, consumptive use and actual Et are sometimes used interchangeably.

Consumptive use of water is an important component of soil-water-plant relationship. The knowledge of consumptive use is necessary in planning and development of water resources, irrigation system and farm water management. Consumptive use of water plays an important role in the management decisions whether applied to agriculture, municipal water supply schemes or surface and ground water rights. Under limited water resources, particularly in arid and semi arid areas, accurate assessment of water needs of crops allows efficient use of water resources to feed the over increasing masses. A number of definitions of consumptive of water have been proposed since 1930. The Hydrology Hand Book of ASAE defined consumptive use as “the quantity of water transpired by the plants during their growth or retained in the plant tissues, plus the moisture evaporated from the surface of soil and vegetation, expressed in feet or inches of water lost or used in a specified time”.

4.5.6. Methods of Determining Crop Water Requirements

The methods of determining crop water requirements or consumptive use of water, in general, can be classified under two categories:

1) Methods based on field measurements and observations
2) Methods using climatic data (empirical methods)

4.5.6.1 Based on Direct Measurement or Field Sampling

These methods of estimating crop water requirements are based on actual measurements in the field using various techniques as given below:

a) Soil Moisture Sampling

In this method the soil samples from the root zone depth are taken before and after irrigation. The soil samples are dried to determine the moisture content on weight basis (MC) present in the root zone. The next irrigation is planned when 15 to 20% of available moisture is present in the soil to avoid moisture stress. Since the purpose of irrigation is to raise the moisture contents of soil the field capacity, the crop water requirement at the root zone is given by the equation.

$$R = \frac{(FC - MC)}{100} \times B \times D \tag{4.9}$$

R = Water requirement on the day of soil moisture sampling (cm)

FC = Percent moisture content at field capacity on weight basis

MC = Present moisture content on weight basis as determined on the day of sampling.

B = Bulk density of soil (g/cm^3)

D = Depth of rootzone (cm)

Soil samples may be taken from root zone depth at various intervals (usually daily) during the crop growth. Daily rainfall is also recorded. The results of moisture sampling, rainfall and Et between two consecutive irrigations are plotted as shown in Fig. 4.9, to estimate water requirements. However, this method of measuring crop water requirements and scheduling irrigation is laborious and time consuming and, therefore, cannot be recommended for day to day field operations and farmer's practices.

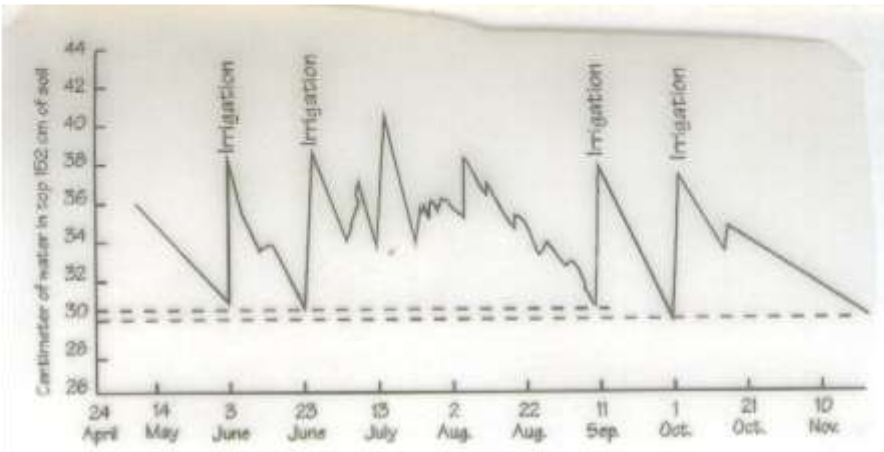


Fig. 4.9 Variation of Soil Moisture with Time Based on Soil Sampling

Using the soil moisture sampling technique, the actual water requirements or consumptive use of major crops (wheat, maize, cotton and sugarcane) determined under optimal management conditions at various sites in Pakistan PARC (1982) are given in Table 4.4. The optimal management conditions refers to the application of irrigations at 60% soil moisture depletion (stress level) under properly fertilized conditions of crop.

b) Lysimeter

A lysimeter is a device in which plant and soil can be isolated from the natural field conditions. It is a cylindrical or rectangular container in which plants can be grown and whole mass can be weighed to determine the moisture lost during a given period of time. Application of water balance equation provides an estimate of Et and water requirements of crop grown.

Table 4.4 Consumptive Use of Major Crops in Pakistan

Location	Wheat	Cotton	Spring Maize	Kharif Maize	Sugarcane
Faisalabad	353	-	-	342	1390
Peshawar	471	-	-	458	-
Bhalwal	401	797	431	-	1195
Mianchannu	475	587	715	319	1360
Bhakkar	520	632	610	385	1482
Tandojam	562	778	-	389	-

4.5.6.2 Water Balance Equation

In a given root zone of a crop, a mass balance relationship can be used to account for the incoming and outgoing moisture from all possible sources whether in the lysimeter or in field conditions. The change in moisture on any given day can be determined by the equation 4.10:

$$W_s = W_i + W_r + W_c - W_e - W_d \tag{4.10}$$

Where

Ws = Change in soil moisture storage over a given period (cm)

Wi = Irrigation water applied (cm)

Wr = Precipitation (cm)

We = Evapotranspiration (cm)

Wi = Deep percolation, runoff or other losses (cm)

Wc = Soil moisture contribution from groundwater

The change in moisture storage on a given day or a period can be added to the moisture content at the end of previous day or period to update the moisture content. Next irrigation is decided when moisture contents reached a predetermined level (e.g. when 15 to 20% of available moisture is present in soil or a predetermined moisture stress level).

4.5.6.3 Based on Climatic Data (Empirical Methods)

For a given crop, one can determine potential evapotranspiration (Etp) using climatic data and empirical equations. There are a number of methods or equations (models) for estimating Etp, the important ones may be listed as:

- Modified Jensen-Haise method
- Blaney-Criddle method
- Modified Perman combination method
- Hargreeve equation
- Pan evaporation method

The climatic data used variably in various equations include temperature, radiation, vapor pressure, wind and day light factors for a given period of time. Due to limited scope of this presentation, only modified Jensen-Haise method, Blaney-Criddle method and Hargreaves methods are explained.

(i) Modified Jensen-Haise Method

The Jensen-Haise method uses temperature and solar radiation adjusted per location and elevation by vapour pressure function. It was primarily developed for Alfa –Alfa crop having 30-50 cm of top growth. The equation must be calibrated for the given region before it can be applied to predict the crop water requirement (Jensen and Haise 1963). The equation for estimating potential evapotranspiration is:

$$\mathrm{Et_P} = \mathrm{C_t} \times (\mathrm{T} - \mathrm{T_X})\ \mathrm{R_s} \qquad (4.11)$$

Where

Et_p = Average daily potential evapotranspiration(cm/day)

T = Mean daily temperature (°C)

R_s= Total solar radiation, in langelys (multiplied by 0.0171 to get cm/day).

T_x = Intercept of the temperature axis on a solar radiation-temperature graph. The coefficients C_t and T_x can be estimated as under:

C_t = Temperature Coefficient

$$\mathrm{C_t} = \frac{1}{\mathrm{C_1 + C_2 C_H}} \qquad (4.12)$$

$$\mathrm{C_1} = 38 - 2^{\circ}\mathrm{C} \times {}^{\mathrm{Elevation\ (m)}}/_{305} \qquad (4.13)$$

$$\mathrm{C_H} = \frac{50\mathrm{mb}}{(\mathrm{e_2^o - e_1^o})} \qquad (4.14)$$

$$\mathrm{C_2} = 7.6°\mathrm{C} \qquad (4.15)$$

$$\mathrm{T_X} = {}^{-2.5 - 0.14 - (\mathrm{e_2^o - e_1^o})}/_{\mathrm{mb}} - (\tfrac{\mathrm{E(m)}}{550}) \qquad (4.16)$$

e2°, e1° = saturation vapor pressures at the mean maximum and mean minimum temperature respectively for the warmest month of the year in mb (milibar).

In order to relate Etp (as calculated above) to actual evaptranspiration values for other crops, a crop growth stage coefficient is defined as:

$$Kco = Eta / Et_p \quad (4.17)$$

Where:

E (m) = Elevation above mean sea level, m.

In English units (FPS system), the equations 62, 64 and 65 (Hill 1983) can be written as:

$$C_1 = 68-(3.6)\ (\text{Elevation, ft/ }1000) \quad (4.18)$$

$$C_2 = 13 \quad (4.19)$$

$$T_x = 27.5 - 0.25\ (e_2 - e_1) - \text{Elevation }/1000) \quad (4.20)$$

(ii) Blaney-Criddle Method

Blaney and Criddle (1950) presented the following equation for estimating potential evapotranspiration.

$$E_{tp} = \frac{Kt.\ t.\ p}{100} \quad (4.21)$$

Where:

E_{tp} = Monthly potential evapotranspiration (inches)

Kt = Climatic Coefficient which equals 0.0173 t – 0.314

t = Mean monthly temperature (°F)

p = Mean monthly% of annual day time (h)

Actual evapotranspiration (E_{ta}) from potential evapotranspiration (E_{tp}) can be estimated using crop coefficient (Kc) as:

$$Et_a = Et_p \times Kc \quad (4.22)$$

(iii) Hargreaves Method

This method is defined by the equation:

$$Etp = 0.0075\ T.\ Rs \quad (4.23)$$

Where:

Etp = Reference Et for well watered grass, Langleys per day

T = Mean Temperature (°F)

Rs = Incident global Solar Radiation, Langleys per day

To obtain Etp in inches per day , the above given equation must be multiplied by a conversion factor of 0.3937/λ , where λ is the latent heat of water in calories per cm^3 and is given by the equation:

$$\lambda = 595 - 0.51\ T\ (T \text{ in } ^\circ C) \tag{4.24}$$

4.6. Implementing Irrigation Scheduling

The process of implementing irrigation scheduling involves the following steps:

1) For each time (days or weeks or months etc.) since the last updated moisture content or known soil moisture, calculate the evapotranspiration for the selected period.
2) For example, the last irrigation was applied on January 1 and soil sampling was initiated on January 4 when the soil was at FC. The soil moisture sampling was continued for 16 days with intervals of 2 days (i.e. till Janurary 20). Plot the moisture content against the period. The last day moisture content was 12% against the FC of 30%, which gives a deficit of 18% for the selected period of 16 days. This deficit can be used to estimate the amount of water to apply using bulk density, depth of root zone and the application efficiency (Equation 35). Thus, irrigation can be executed on Janurary 21.
3) In case, the data on sampled moisture are not available but crop needs irrigation application on Janurary 21, calculate the ET value for 20 days (Janurary 1-20), using climatic data and the selected empirical equation. The calculated ET for the given period is equivalent to the water deficit that may be further used to assess the depth of water to be applied.
4) Subtract the calculated Et from the last known soil moisture storage in the root zone (FC) to find the moisture deficit and update the current soil moisture status considering the rainfall (if any). The rainfall should be added to soil moisture storage, which would reduce the deficit.
5) If the current soil moisture is less than or equal to a pre-selected or allowable value of moisture status (MAD), initiate an irrigation.
6) Calculate the date of the next irrigation using the previous data on rate of loss of moisture per day and moisture content required under MAD.
7) When the date of next irrigation is decided, the amount of water to be applied will equal the allowable depletion divided by the application efficiency. The date of next irrigation should match with the schedule of water availability (warabandi). This is achieved by adjusting the amount of application by the number of days of adjustment. Access to water supply from a tubewell may solve the water availability problem.
8) After deciding over ‘how much to apply’ and ‘when to apply’ the actual irrigation to the field is accomplished considering the flow rate, area to be irrigated, duration of irrigation and depth to be applied using Equation 49.

4.6.1. Depth of Water to be Applied (Based on Depth Needed at Root Zone)

The purpose of irrigation is to replenish the soil moisture deficiency in the root zone. However, under field conditions, it is practically difficult to replenish exactly the consumed amount without losing certain amount of water that induces inefficiency in the system. Thus, application efficiency must be considered, in addition to the depth of water needed at the root zone, while deterring the depth of water to be applied during each irrigation. Consequently, the depth of water to be applied is given by dividing the depth needed at the root zone (i.e deficiency in the root zone) by the application efficiency (Ea). The average application efficiency in Pakistan ranges between 50 to 67% under basin or border irrigation methods and 80 to 85% for sprinkler irrigation and above 85% for drip irrigation.

4.6.2. Depth of Water to be Applied (Based on Crop water Requirement)

The depth of water to be delivered during application (gross water application) is a function of crop-water required at the root zone (net water requirement) and application efficiency as discussed above. However, the leaching requirement (LR) to leach down the excess salt is applied in addition to the crop water requirements at the root zone. After determining the crop water requirement and leaching requirement, the depth of water to be applied can be determined as:

$$\text{Depth of water applied} = \frac{\text{Crop water requirment}}{\text{Application efficiency}} + \text{LR} \tag{4.25}$$

4.6.3. Relationship Between Time Area and Depth of Irrigation

Accomplishment of irrigation in the field requires information regarding discharge (Q), area of the field (A) and time of irrigation (T) in addition to the depth to be applied (d), which are related by the following equation:

$$\text{QT} = \text{Ad} \tag{4.26}$$

(i)* FPS *Units

Equation 74 with FPS units can be defined as:

Q = Discharge or inflow to the field (cusecs)

T = Time of irrigation application (h)

A = Area of field (acres)

d = Depth of irrigation (inches)

The equation (74) is not exactly balanced with FPS units as shown below. To irrigate one acre field for a depth of one inch, served with one cusec inflow applied for 1 hour and expressing both sides of Eq. 74 in terms of FPS units i.e. cubic feet of water applied to the field (LHS) and that stored on the field (RHS), it yields:

LHS:

Water Applied = (1 cusec) (1 h)

= (1 ft^3/sec) (3600 sec)

= 3600 ft^3

RHS:

Water Stored = (1 acre) (1 inch)

= (43560 ft^2) (1/12 ft)

= 3630 ft^3

It shows that the both sides are only approximately equal i.e. LHS ≈ RHS

There is a difference of 30 ft^3 between the two sides of the equation, which is practically a small amount of water and therefore, considered an approximate equality. True balance of the equation can be found using metric units as explained below.

(ii) Metric Units

The Eq. 74 becomes more balanced when expressed in metric units i.e. Q in m^3/sec, T in seconds, area in m^2 and depth in meters to yield:

Total inflow of irrigation water = Total stored over the field

or (m^3/sec discharge)(seconds) = m^2 area× m depth

m^3 = m^3

i.e. If a flow rate of 1 m^3/sec is diverted to 1 m^2 area for 1 sec, it will result in 1 m depth of water on the field. Therefore, Eq. 74 is valid for metric units and is correctly balanced i.e. Eq. 74 can be written as:

Q (in m^3/sec) x T (in sec) = A (in m^2) × D (in m)

(iii) Metric Units with Hectare

If the field area is expressed in hectares, Eq. 74 can be defined as given below:

Defining discharge (Q) as m^3 /sec, area (A) as hectare (10000 m^2) and period as hour (3600 sec.) and the depth (D) in meters, the Eq. 74 can be written as:

1 m^3/s × 1 hour as 3600 sec = 1 hectare as10000 m^2 × 1m

As such the equation is not balanced as LHS = 3600 m^3

and RHS = 10000 m^3

The unit conversion factor can be determined as:

1Q x 1T = (10000 /3600) x 1D

Or Q.T = (10000/3600) A.D

Or Conversion factor = 10000 / 3600

$= 2.77$

In order to balance the above equation for area in hectares, D in meters and time in hours, RHS must be multiplied with (10000/3600) or 2.77. Thus, equation 4.26 will become:

$$Q\,T = 2.77AD \qquad (4.27)$$

Where:

$Q = m^3/sec$

$T = h$

$A = ha$

$D = m$

Thus, in metric system, the application of water at the rate of $1m^3/sec$ for 2.77 hours can irrigate one hectare of land to a depth of one meter. For a given discharge, depth and area, the time of irrigation can also be determined using the Eq. 74 or Eq. 75 utilizing FPS or metric units, respectively. The following example would demonstrate the procedure of determining the time of irrigation using different units.

Example

To demonstrate the use of different unit systems in Eq. 4.26.

"A farmer plans to apply 2.4 inches of water to a field of 2.5 acres with flow rate of 2 cusecs. Determine the time of irrigation application using (i) FPS units (ii) Metric Units (iii) Metric Units with area in hectare."

Given Data

(i) FPS Units	(ii) Metric Units	(iii) Metric Units with Hectares
$Q = 2\ ft^3\ /sec$	$= 0.0567\ m^3/sec$	$= 0.0567\ m^3/sec$
A = 2.5 acres	= 10121/46	= 1.012 hectares
D = 2.4 inches	= 0.061m	= 0.061m
T = h	= sec	= h

Required:

T = Required Time

Solution

FPS Units

Using Balance Eq. 74 we have:

$$QT = AD \rightarrow \rightarrow T = AD / Q$$

Replacing by the values in FPS system given above, we get:

$$2 \times T = 2.5 \times 2.4$$

$T = (2.5 \times 2.4)/2$

$T = 3$ h

Metric Units

$T = AD / Q$

Replacing with the values in Metric System

$T = 10121.46 \times 0.061 / 0.0567$

$T = 10889.05$ sec

$T = 3.024$ h

Metric Units with Hectares

$T = 2.77AD / Q$

$T = 2.77 \times 1.012 \times 0.061 / 0.0567$

$T = 3.017$ h

Thus, it is concluded that equation 74 and 75 yield the same results provided appropriate units are used. Small variation comes from the conversionfactors of units.

4.6.4. When to Apply Next Irrigation?

When to irrigate pertains to predicting the time of next irrigation. This can be accomplished both using;

1) Field determination of soil moisture and
2) Based on empirical equation and/ or soil moisture balance equation

In fact, both procedures use moisture balance equation and updating the soil moisture in the root zone on a particular day or period.

The root zone moisture budget is projected into future until the soil moisture level at which an irrigation should occur (e.g. a given moisture stress level or when 15 to 20% available moisture is left) is achieved. Assuming a linear rate of evapotranspiration, the date of next irrigation can also be decided by dividing the total consumable moisture by the average rate of evapotranspiration.

$$\text{Number of days to next } irrigation = \frac{\text{consumable or available moisture}}{\text{daily actual evapotranspiration}} \qquad (4.28)$$

$$= \frac{\text{Present moisture - minimum allowable moisture}}{\text{daily } Et_a}$$

4.7. Irrigation Scheduling Example

The development of irrigation schedule for a given cropping pattern, growing season and irrigation water supply system can be elaborating with the following field example.

4.7.1. Given Data

A 4 acre farm is located 2000 feet downstream of canal outlet. The farm is cropped with wheat and fodder as shown below in Fig. 4.10. The canal outlet and a tubewell jointly deliver 4.45 cusecs (127 lps) of water at the head of watercourse that has conveyance efficiency of 83% (or a loss of 0.38 cusecs/1000 feet). The soil of the farm has field capacity of 25% and PWP 5% by weight. The bulk density (BD) is 1.20 gm/cm^3. The root zone depth (D) of wheat is considered 60 cm and application efficiency of 80% can be achieved. Assume that next irrigation should be applied when soil has 15% of available moisture or 85% of available moisture content by weight have been consumed (MAD). The daily consumptive use at the farm is approximately 5.1mm per day or 0.57% by weight per day.

1- Wheat	3- Fodder
2 - Wheat	4 - Fodder

Fig. 4.10 Cropping Pattern of a 4-Acre Farm

The data on moisture sampling and rainfall are available as given in Table 4.4.

Table 4.4 Soil Moisture Sampling Data for Irrigation

Field No.	Date	Event	Moisture content (% by weight)	Moisture Loss per day (%)
1 and 2	Nov. 10	Pre-sowing Irrigation	At Saturation i.e.30%	
	Nov. 11	Sampling	After Irrigation (28%)	
	Nov. 13	Sowing	At FC (25% by weight)	
	Nov. 22	Sampling	18	
	Nov. 24	Before Rainfall	16	
	Nov. 24	After Rainfall	18	
	Nov. 29	Sampling	14	0.57% by weight
	Dec. 6	Sampling	10	
3 and 4	Oct. 1	Pre-sowing irrigation	At saturation (i.e. 30% by weight)	
	Oct. 4	Sampling	At FC 25	
	Oct. 10	Sampling	18	0.71% by weight
	Oct. 16	Sampling	13	
	Oct. 23	Sampling	8	

The irrigation turn of the farmer is on weekly basis for 1.13 hours or 68 minutes (at the rate of 17 minutes per acre for 4 acres of land) as scheduled below.

- Nov. 11
- Nov. 18
- Nov. 25
- Dec. 2
- Dec. 9

Develop an irrigation schedule for the above given wheat fields in terms of irrigation amounts and times of application to wheat fields.

4.7.2. Solution

The example has been set to demonstrate the irrigation scheduling procedure for given wheat crop. Scheduling for fodder is left to the exercise of students.

Discharge at the head of watercourse	= 4.45 cusecs
Conveyance efficiency	= 83%
Discharge available at the farm	= $4.45 \times 0.83 = 3.69$ cusecs
Field capacity of soil	= 25% by weight
Permanent wilting point	= 5% by weight
Available moisture	= 20% by weight

Considering MAD that irrigation should be applied when 85% moisture has been consumed or 15% of the available moisture is present in the soil i.e. the minimum soil moisture content at which the next irrigation is needed to be applied, can be determined as:

$$\text{PWP} + (15\% \text{ of available}) = 5 + (20 \times 0.15) = 8\% \text{ by weight}$$

Thus, minimum level of soil moisture for irrigation application is 8% by weight.

Referring to the Table 4.4, the last moisture sampling was done on Dec. 6, which shows moisture content of 10% that was above the minimum moisture level (PWP =5% and MAD =8%) permissible for the irrigation. Therefore, irrigation may be applied on Dec. 7. However, water is not available on Dec. 7 as the irrigation turn falls on either Dec. 2 or Dec. 9. If the farmer waits till Dec. 9, the soil will lose at the rate of 0.57% per day for 3 days and moisture contents would approach 8.3% by weight i.e. the moisture will get closer to the predetermined minimum level of 8% by weight. However, it is still within the permissible range of 15% of available. Thus, he may choose to apply irrigation either on Dec. 2 or Dec. 9. Any further delay may cause the moisture content to fall below minimum permissible limit of 8% and therefore, delay beyond Dec.9 cannot be allowed.

4.7.3. Amount of Water to be Applied

Option 1: In case, the farmer chooses to apply on Dec. 2, moisture content on Dec. 1 (MC) is approximately 13%, while the moisture content at FC is 25%.

Since the purpose of irrigation is to replenish moisture and to raise the moisture content to FC, the amount needed (deficiency) in the root zone is.

F C – MC = 25 – 13 = 12% by weight

Depth of water needed at root zone = (12% by weight × BD) × D

= 0. 12 × 1.2 × 60

= 8.62 cm = 3.38 inches

Considering the application efficiency of 80%, the amount to be applied can be calculated as:

Depth to be applied $= \dfrac{3.38}{0.8}$ = 4.22 inches

Area that may be irrigated on Dec. 2

Flow rate (Q) = 3.7 cusecs

Flow rate = 1.13 hours

Depth to be applied (d) = 4.22 inches

Using Water Balance Eq. 49, Area that may be irrigated (A) = Q/td

Or A $= \dfrac{3.7 \times 1.13}{4.22}$

Thus, A = 0.991 acre

Option 2: Water to be applied if irrigation is scheduled on Dec. 9

Moisture content on Dec. 9 (MC) = 8.3%

Field capacity = 25%

Available Moisture = 16.7% by weight

Depth of water needed at the root zone = 0.17 × 1.2 × 60

= 12.24 cm = 4.6 inches

Amount to be applied on Dec. 9 $= \dfrac{4.6}{0.8}$ = 5.7 inch

Area that may be irrigated on Dec. 9 with the above given flow rate, depth and irrigation turn time (Eq. 51).

$$A = \frac{3.7 \times 1.13}{4.22} = 0.72 \text{ acre}$$

In order to complete irrigation of one acre of land on Dec. 9, either the irrigation depth needs to be reduced to 4.2 inches, or time of application needs to be increased to 1.6 hours or discharge has to be increased to 5.1 cusecs.

Thus, with the available supplies and irrigation turn, the farmer can irrigate only one or less than one acre at a time. As the farmer has two acres of wheat to irrigate, he should choose to irrigate one acre on Dec. 2 and other on Dec. 9. However, on Dec. 9, he may have to reduce the depth of irrigation or to increase the time or discharge as mentioned above to complete irrigation of wheat field.

4.8. Consumptive use

The results of soil moisture sampling, given in Table 4.4 have been plotted on Fig. 4.11. These data can be used to determine seasonal consumptive use or water requirements of crop as well as the average daily Eta for a given period of time. The consumptive use of water (Eta) for different periods can be calculated as:

Period between Nov. 11 and 22

Depth of water available or consumed between Nov. 11 and 22

$$= \frac{28-18}{100} \times (1.2)\ (60)$$

$$= 7.2 \text{ cm}$$

Average daily Eta or consumptive use $= \frac{7.2}{12} = 0.6$ cm/day

Period between Nov. 24 and 29

Depth of water consumed between

$$= 1.2 \times 60\ (18-14)\ /100$$

$$= 2.88 \text{ cm}$$

Consumptive use between Nov. 24 and Nov.29 = 2.88 / 5

$$= 0.576 \text{ cm /day}$$

Depth of water consumed between Nov 29 and Dec. 6

$$= 1.2 \times 60\ (14-10)\ /100 = 2.88\text{cm}$$

Consumptive use of water between Nov 29 and Dec. 6

$$= 2.88 / 7$$

$$= 0.411 \text{ cm / day}$$

4.9. Frequency of Irrigation

The plant can only consume the amount of water stored in the root zone. Therefore, the frequency of irrigation, which is the number of days between two consecutive irrigations, can be determined on the basis of the amount stored in the root zone during 1st irrigation that would be subsequently utilized by the plants before next irrigation, divided by the daily Eta. The consumed or utilized amount of water between two dates, can be computed on the basis of FC, PWP, MAD or sampled moisture content (MC) as explained earlier. The daily consumptive use for the given crop, soil and climatic data, can be obtained using empirical equations. The following example would help determining the frequency of irrigation based on the assumed data:

Date of previous irrigation = Nov. 11

Depth of water stored in the root zone = 7.2 cm

Eta (computed from sampling data or empirical equation) = 0.4cm/day

Frequency of irrigation = 7.2/0.4

= 18 days

Hence, next irrigation would fall on Nov. 29 i.e. Nov. 11 + 18 days

Thus, frequency of irrigation for the given situation is 18 days. In case, the irrigator is following a given management criteria such as to apply next irrigation based on PWP, MAD or a given moisture content, the available moisture for consumption would be:

Available moisture = FC- Management decision moisture

Frequency of Irrigation = Available moisture / Daily Consumptive Use

The amount of water actually applied, obtained from Qt = Ad considerations, should not be used to determine the frequency as this amount also includes the losses, which cannot be utilized by theplants.

4.10. Efficiencies and Coefficients

Efficiency is generally defined by the ratio of output to input. However, it may also be expressed in % age by multiplying the ratio with 100. After determining the net irrigation water requirements or water requirements at the root zone, an estimate of gross irrigation water requirements or the amount of water to be applied is needed. No irrigation system is capable of applying exactly needed amount of water as some water is expected to be lost through deeper percolation, over spilling, mole holes, cracks in the banks and evaporation during application. Efficiency is one of the major indices used in the performance evaluation of a given irrigation system. Higher efficiency indicates a higher performance or lower degree of water losses in the system. Therefore, steps must be taken to achieve the highest efficiency or to improve the system for higher efficiency of the irrigation system.

Efficiencies are involved during conveyance, application, distribution over the field, storage in the reservoirs, storage in the root zone and plant use etc. The overall farm irrigation efficiency, which may also be termed as system efficiency, results from all components of the system that affect the irrigation efficiency. Various Efficiency terms are defined below.

4.10.1. Application Efficiency (Ea)

The application efficiency (Ea) denotes the effectiveness of the applied water to store in the root zone. The stored amount is considered output and the applied amount is considered input. Therefore, application efficiency in surface irrigation is given by the Equation 46:

$$\mathrm{Ea} = \frac{\text{Volume of water stored in root zone}}{\text{Volume of water applied or delived to field}} \times 100 \qquad (4.29)$$

Thus, application efficiency can also be defined as the average depth of irrigation water infiltrated and stored in the root zone to the average depth of water applied or delivered to the field.

In case of sprinkler irrigation, the Application Efficiency of Low Quarter (AELQ) as defined by Jensen (1980), is the ratio of the average low quarter (LQ) depth of irrigation water infiltrated and stored in the root zone (calculated from soil moisture sampling) to the average depth of irrigation water applied (calculated from nozzle discharge) expressed either as ratio or as %. The average low quarter depth infiltrated is the average of the lowest one fourth of the measured values where each value represents an equal unit of area and cannot exceed the soil moisture deficiency. When the low quarter value (LQ) is less than Soil Moisture Deficiency (SMD) or the desired Management Allowed Deficit (MAD), the numerical value of low quarter average depth indicates the adequacy of irrigation. Thus, AELQ is defined as:

AELQ = Av. Low Quarter stored / Average depth applied (4.30)

4.10.2. Conveyance Efficiency (Ec)

The Conveyance Efficiency (Ec) shows the effectiveness or efficiency by which a given watercourse or irrigation channel can deliver or convey water and is given by the Equation 79.

$$\mathrm{Ec} = \frac{\text{Volume of water applied or delievered to the feld}}{\text{Volume of water diverted from source}} \times 100 \qquad (4.31)$$

$$\mathrm{Ec} = \frac{\text{Discharge at the downstream section Wc}}{\text{Discharge at the upstream section of Wc}} \times 100 \qquad (4.32)$$

4.10.3. Irrigation efficiency (Ei)

It denotes the effectiveness of the irrigation system (e.g. from canal outlet to field) to convey irrigation water from the source to the point of use. It is the multiple of Ec and application efficiency (Ea) as given below:

$$\mathrm{Ei} = \mathrm{Ea} \times \mathrm{Ec} \qquad (4.33)$$

4.10.4. Storage Efficiency (Es)

It is the ratio of the volume of water available from the reservoir for irrigation, to the volume of water delivered to the storage reservoir (Jensen 1980).

$$Es = (Va/ Vd) \times 100 \qquad (4.34)$$

Where:

Va = Water available from reservoir

Vd = Water delivered to the reservoir

4.10.5. Rootzone Storage Efficiency (Ers)

It is the ratio of volume of water stored in the root zone (Vrsa) to the amount needed to be stored in the rootzone or capacity of root zone (Vrs). In case, the actual storage is less than the capacity of rootzone, Ers <100%. If the amount stored is equal to the capacity of root zone or the stored amount is equal or greater than the storage capacity, Ers = 100%.

$$Ers = Vrsa/Vrs \qquad (4.35)$$

4.10.6. Water Requirement Efficiency (Ewr)

Water requirement efficiency indicates how efficiently, the water requirement of a crop has been met. It is the ratio of the water requirement actually met (WRa) to the water requirement of crop that has to be met for maximum production (WRc).

$$Ewr = WRa / WRc \qquad (4.36)$$

4.10.7. Christiansen's Uniformity Coefficient (UC)

It is the ratio of the average depth of water infiltrated or caught minus the absolute average deviation from the average depth, to the average depth infiltrated or caught. (Charistiansen 1942). This parameter can be used for evaluating the distribution pattern of water delivered by the sprinklers. It is also applicable to sprinkler irrigation pattern overlapped during an irrigation.

$$UC = (DA - XA) / DA \qquad (4.37)$$

Where:

UC = Christician Uniformity Coefficient

DA = Av. Depth of water Infiltrated or caught

XA = Absolute Average Deviation from Av. Depth

UC may also be defined as

$$UC = 1 - (\text{Average Deviation} / \text{Average depth infiltrated}) \qquad (4.38)$$

4.10.8. Distribution Uniformity (Du)

Under sprinkler irrigation system, the Distribution Uniformity (Du) is the ratio of the average LQ depth of irrigation water infiltrated or caught to the average depth of irrigation water infiltrated or caught, expressed as %. The water delivered by the sprinklers may infiltrate into the soil or may be caught in cans uniformly placed under sprinklers. The distribution uniformity mainly applies to the sprinkler irrigation. Eq. 4.39 shows the mathematical definition of Distribution Uniformity.

$$Du = (D\ LQA / DA) \times 100 \qquad (4.39)$$

Where:

Du =% Distribution Uniformity

D LQA = Average Low Quarter Depth of infiltrated water

DA = Average depth of infiltrated water

4.10.9. Uniformity Coefficient (Cu)

The uniformity coefficient measures the uniformity with which irrigation water is distributed over the surface of the field. Christiansen (1942) developed a numerical equation to express the uniformity achieved with sprinkler irrigation.

$$Cu = 100\ (1.0 - \sum x / mn) \qquad (4.40)$$

Where:

Cu = Uniformity Coefficient in %

$\sum x$ = Summation of absolute deviation from mean application

m = Mean depth of applied water

n = Number of observations of applied depth

4.11. Solved Examples Related to Soil-Water Plant Relationship

Example 4.1

During field evaluation, the soil moisture content at the time of soil sampling before irrigation were 10% by weight and the field had the following characteristics;

Given Data:

Application Efficiency (Ea)	= 65%
Field Capacity (FC)	= 20% by weight
Permanent Wilting Point (PWP)	= 7% by weight
Bulk Density (BD)	= 1.8 g/cm^3

Root Zone Depth (D)	= 75 cm
MAD	= 80% of Available Moisture
Soil Moisture at Sampling	= 10% by weight

Required:

i. Total Depth of Available Water in the Root Zone (AM)
ii. Moisture Deficiency at the root zone at the time of sampling
iii. Depth of water to be applied at PWP
iv. Depth of water to be applied at sampled moisture content
v. Depth to be Applied at MAD
vi. Number of days the farmer can delay the irrigation before PWP, if daily consumptive use is 4 mm/day.

Solution

Total Available Moisture at Root Zone (AM) = FC – PWP

= 20 – 7 = 13% by weight

Moisture Consumed at Sampling = FC – Sampled Moisture Content

= 20 – 10 = 10% by weight

Moisture Consumed at PWP (20-7) = 13% by weight

Moisture Consumed at MAD = Available × 80/100

= 13 × 80/100 = 10.4% by weight

Moisture present in soil at MAD = 20 – 10.4 = 9.6% by weight

(i) Total Depth of Available Water at Rootzone = AM × BD × Depth

= 13/100 × 1.8 × 75

= 17.55 cm

(ii) Moisture Deficiency (Deficit) at Sampling = 10/100 × 1.8 × 75

= 13.5 cm

(iii) Depth to apply at PWP = Deficiency at PWP / Ea

= 17.55/ 0.65 = 27 cm

(iv) Depth to Apply at Sampling = 13.5 / 0.65 = 20.8 cm

(v) Depth to Apply at MAD = Deficiency / Ea

= (10.4/100 × 1.8 × 75) /0.65

= 21.6 cm

(vi) Number of Days to delay irrigation from Sampled Day to PWP

Moisture to be consumed between sampling and PWP = 10 – 7

= 3% by weight

Depth to be consumed between sampling and PWP = 3/100 × 1.8 × 75

= 4.05 cm

Or Difference of Moisture deficiency = 17.55 -13.5 = 4.05 cm

Consumption Per day = 0.4 cm / day

Number of days to consume before PWP = 4.05 /0.4

= 10 days

Example 4.2

A one- hectare field of wheat crop has to be irrigated with a stream of 1.5 cusecs (42.45 lps). If the deficiency of moisture at the root zone is 6 cm at the time of irrigation and application efficiency is 70%, determine the time of irrigation in minutes.

Given Data:

FPS Units

Area (A) = 1 ha or 2.47 acre

Discharge (Q) = 1.5 cusecs

Moisture Deficiency at Rootzone (d) = 6 cm or 2.36 inch

Application Efficiency (Ea) = 70%

Metric Units

Area = 1 ha or 10,000 m^2

Discharge (Q) = 0.0425 m^3/s Moisture Deficiency at Rootzone (d) = 6 cm

Application Efficiency (Ea) = 70%

Required:

Time Required to Irrigate the Given Field (T) = ?

Solution

a. Using FPS Units

Deficiency at Rootzone (d) = 2.36 in

Depth of water to be Applied (D) = d / Ea

= 2.36 / 0.70 = 3.375 in

Using the Balance Equation i.e. Q x T = A × D

Time to Irrigate T = A × D / Q

Substituting the values (given above) in FPS units

Time to Irrigate	T	= 2.47 × 3.375 / 1.5
	T	= 5.6 h

b. Using Metric Units

Deficiency at Rootzone	(d)	= 6 cm
Depth of Water to be Applied (D)		= d / Ea or 6/ 0.70
		= 8.57 cm or 0.0857 m
Using Balance Equation	Q x T	= A × D
Time to Irrigate	T	= A × D / Q

Substituting the above given data in metric units,

Time to Irrigate (T)	= 10000 × 0.08573 / 0.0425
	= 20172 sec or 5.6 h

Thus, using FPS or Metric system of units, result remains the same i.e. T= 5.6 h under both systems of units.

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Chapter 5

Irrigation Pumping System

5.1. Methods of Well Drilling

Historically many well drilling methods have developed because of varying geologic conditions ranging from hard rock as granite and dolomite to completely unconsolidated sediments such as alluvial sand and gravel. Particular drilling methods have become dominant in certain areas because they are most effective in penetrating the local aquifer and thus offer economic advantages. Well drilling methods are numerous, however, only the important ones are descried here:

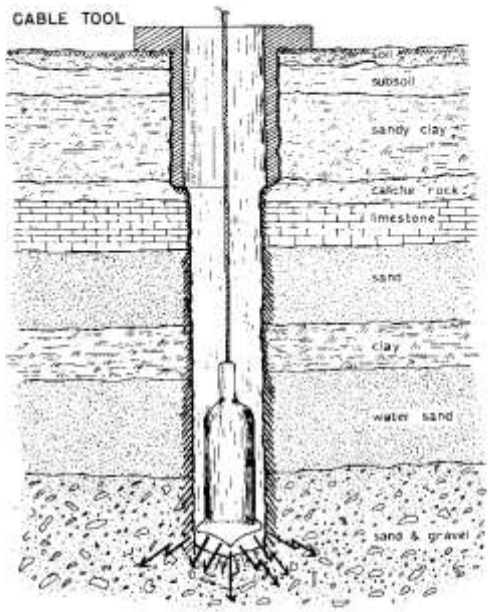
1. Cable tool method
2. Rotary methods
    i. Direct rotary
    ii. Reverse rotary

5.1.1. Cable Tool Method

As the name shows, this method uses a cable and tool to drill a well. It is also termed as percussion method. It is the earliest drilling method developed by Chinese and had been continuously in use for about 4000 years. Cable tool method can be practiced manually or by using cable tool drilling machines. In both cases, the drilling process is identical. Cable tool drilling machines, also called percussion or Cable Tool Rigs (Fig. 5.1a), operate by repeatedly lifting and dropping a heavy string of drilling tools into the bore hole as shown in. The drill bit breaks or crushes consolidated rock into small fragments, where as the bit primarily loosen the material when drilling in unconsolidated formations (Fig. 5.1b). In both instances, the reciprocating action of the tools mixes the crushed or loosened particles with water to form a slurry or sludge at the bottom of the bore hole. If little or no water is present in the penetrated formation, water is added to form slurry. Slurry accumulation increases as drilling proceeds and eventually it reduces the impact of the tools. When the penetration rate becomes unacceptable, slurry is removed at intervals from the borehole by a bailer. Bailers used to remove the mud or rock slurry consists of a pipe with a check valve at the bottom. A bail handle at the top of this tool attaches to a cable of the drilling machine.

Fig. 5.1a Cable Tool Rig

Fig. 5.1b Cable Tool Bit Action



5.1.1.1 Components of Cable Tool Rig

The components of a full string of cable tool drilling equipment include:

- Drill bit
- Drill stem
- Drilling jars
- Swivel socket
- Cable
- Engine and power transmission

5.1.1.2 Advantages

The cable tool method offers the following advantages:

- Rigs are relatively inexpensive
- Rigs are simple in design and require little sophisticated maintenance
- Machines have low energy requirements
- Bore hole is stabilized with casing pipes during the entire drilling operation, recovery of reliable, samples is possible from every depth
- Generally, only one person is needed to operate the drilling rig
- Because of smaller size, machines can be operated in more rugged, inaccessible terrain or in other areas where space is limited
- Wells can be drilled where little water is available
- Little chance of contamination of soil and water samples from different depth

5.1.1.3 Disadvantages

- Some disadvantages of the cable tool method include the following:
- Penetration rates of tools are relatively slow
- Casing costs are usually higher because heavier wall or larger diameter casing may be required
- It may be difficult to pull back long strips of casing in some geologic conditions
- Drilling depth are limited
- Not recommended for hard rocks

5.1.1.4 Drilling Consolidated Formations

Mostly, the bore holes drilled in consolidated formation require no casing during drilling operation. The cable tool kit bit used in consolidated rocks is essentially a crusher. Its performance depends on the energy delivered to the bottom of the hole. The factors affecting drilling efficiency include:

- Resistance of the rock

- Weight of drill tool
- Length of stroke
- Number of strokes per unit time
- Design and sharpness of bit
- Density and depth of accumulated slurry and above all the skill of driller

5.1.1.5 Drilling Unconsolidated Formations

During drilling operation in unconsolidated formations, the pipe or casing must follow the drill bit to prevent caving in and to keep the bore hole open. Usually, the casing has to be driven by additional weight or by an operation similar to pipe driving. The drilling action of the bit is primarily a loosening and mixing process rather than crushing as in cable tool bit.

A drive block or drive clamps are used on drilling tools for driving the casing. The block is lifted and dropped to impart energy to move casing. Material in the casing is mixed with water by the drill bit to form slurry. Most of the slurry is bailed out before draining the casing. In some cases, the hole is drilled 1 to 2 m below the casing to push the casing to undisturbed material and then drilling is continued.

5.1.2. Rotary Drilling

Rotary drilling process involves boring a hole by using a rotating bit on which a downward force is applied. The bit is supported and rotated by a hollow stem composed of high quality steel through which a drilling fluid is circulated. The hydraulic rotary methods are classified into "direct rotary" and 'reverse rotary" depending upon the mechanism of using drilling fluid for removal of drill cuttings.

5.1.2.1 Direct Rotary

The direct rotary drilling process is explained below. The bore hole is drilled by a rotating bit attached to the lower end of drill pipe. The rock cuttings are removed by continuous circulation of a drilling fluid. In this case, the fluid used is a mixture of water and bentonite clay, which plasters the hole with mud and also brings out the drill cuttings. This method is suitable for drilling only relatively small diameter, shallow wells in unconsolidated formations. The direct rotary machines are limited to bore holes with maximum diameter of 559-610 mm (Fig.5.2).

In direction rotary operation, the drilling fluid is pumped own through the drill pipe, which jets out through the bit. After mixing with the cuttings, the fluid flows upward in the annual space between the hole and the drill pipe and finally joins the circulation tank on setting pit where most of the cuttings drop out and clean fluid is re-circulated to the system.

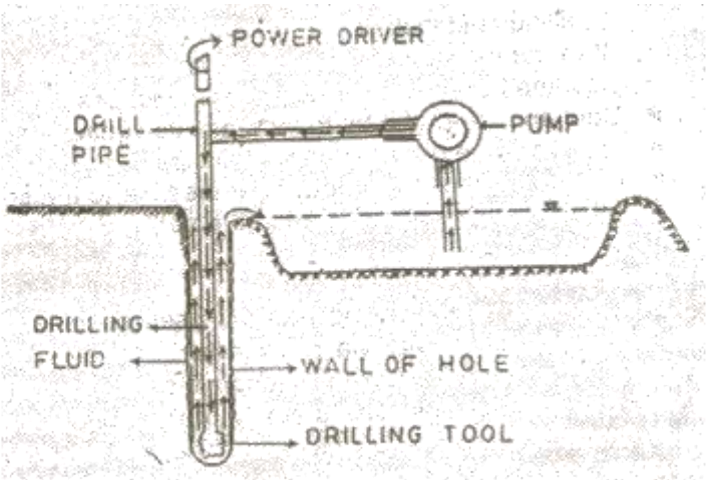


Fig. 5.2 Direct Rotary

Advantages

Requires little or on casing in most rocks

- Penetration rates are relatively high in all types of formations
- Drill small-diameter holes to greater depth
- Minimum casing is required during drilling operation
- Well screens can be installed easily as part of casing

Disadvantages

- Direct rotary drilling rigs are costly as compared to percussion rigs
- Drilling rigs require high level of maintenance
- Collection of accurate samples requires special procedures
- Use of drilling may cause plugging of certain formation
- Mobility of rigs may be limited by slope and topographic conditions of the area
- Drilling fluid management require skilled manpower

5.1.2.2 Reverse Rotary Drilling

The reverse rotary drilling method uses plain water, which fills the hole through gravity via channel connecting it with the water pond. This fluid is sucked through the bit via drilling stem along with the drill cuttings. The stability of the hole is maintained by the hydrostatic pressure of the water filled in the drilling hole. The reverse rotary drills are used for hole diameter beyond 24 inches. The design of reverse rotary rig is essentially the same as that of direct rotary except the equipment is larger in size (Fig. 5.3).

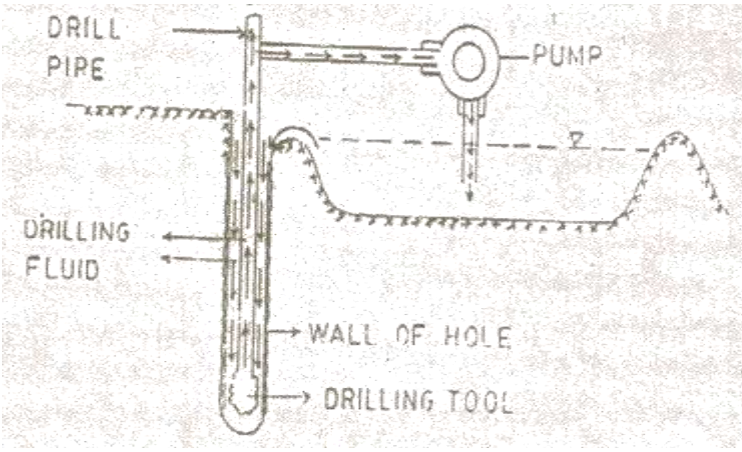


Fig. 5.3 Reverse Rotary Drilling

Advantages

- Utilizes fast and continuous drilling action
- Requires little or no casing
- Utilizes clear water as drilling fluid, thus minimizing aquifer plugging
- Readily drills large diameter holes
- Wells screens can be set easily as part of the casing installation
- Most geologic formations can be drilled except igneous and metamorphic rocks.

Disadvantages

- Large water supply is generally needed
- Reverse rotary rigs and their components are usually larger and thus more expensive
- Large mud pits are required
- Some drill sites are inaccessible because of the rig size
- More personal are generally required

5.1.3. Functions of Drilling Fluid

Drilling fluids include air, clean water and mixtures of water and additives such as bentonite. However, most of the rotary drilling operations use water based drilling fluids. The essential functions of drilling fluids. The essential functions of drilling fluid include the following:

- Stabilize the walls to prevent caving
- Seal the bore-hole wall to reduce fluid loss
- Cool and clean the bit to increase its cutting efficiency

- Lubricate the bit, pump and drill pipe
- Lift the cuttings from bottom of the hole to the settling pit
- Allows cutting to drop out in the setting pit

5.2. Development of Wells

Development of water wells aims at removing the finer material from the aquifer, thereby, cleaning out openings or enlarging passages in the formation so that water can enter the well more freely. It is an essential operation in the completion and proper operation of a water well and has the following benefits:

- It stabilizes the sand formation around a screened well so that the well may yield sand free water
- It increases the porosity and permeability of the natural formation in the vicinity of the well and thereby increases the well yield
- Corrects any damage, which occurs as a side effect from the drilling

All new wells should be developed before they are put to operation to achieve a free water movement at the highest specific capacity. The older wells should also be periodically developed to improve the yield. Aquifer development is done when it will not yield enough water even after the well developed.

5.2.1. Methods of Well Development

Various methods of well developments are summarized below:

5.2.1.1 Mechanical Surging

An effective means of developing the well by surging is to operate a plunger up and down in the casing like a piston in a cylinder. The tool normally used is called a surging plunger, or surge block. Surge plungers some times produce unsatisfactory results where the aquifer contains many clay streaks as it may cause the clay to plaster over the screen surface. A surge plunger should be operated only if sufficiently free flow of water has been established so that the tool runs smoothly and freely. This method is suitable in developing tight formations.

5.2.1.2 Surging with Air

Compressed air can be used effectively as a development tool to do the job. In this method, air is injected into the well to lift the water to the surface. Then air supply is stopped allowing the water column to fall. The equipment needed for this method includes; air compressor, pumping pipe, air line in the well, pressure gauge and relief value to safeguard against accidental over loading and quick opening valve in the outlet of the tank for controlling air flow.

5.2.1.3 Over Pumping

The simplest method of removing fines from the water bearing formation is by ‘over-pumping’. Over-pumping means pumping the well at a higher rate than it will be pumped when it is put to service. It may be a simple matter to over-pump in small well or poor aquifers, but where a large quantity of water must be pumped, it may be difficult to obtain equipment of ample capacity at reasonable cost. Over-pumping may however, leave some of the sand grains bridged in the formation which may reduce the porosity.

5.2.1.4 Back Washing

This method consists of alternately lifting water to the surface by pumping, and then letting the water run back into the well through the pump column pipe. The type of pump, besides the air lift that can be used practically for this purpose is a deep well turbine pump without a foot valve. The pump is started, but as soon as water is lifted to the ground surface, the pump is shut off, the water in the column pipe then falls into the well. This process is repeated which produces alternate inflow and out conditions through the screen and aquifer openings which removes the fine particle and develop the well.

5.2.1.5 High Velocity Jetting

Development by jetting may be done with either water or air. The procedure consist of operating a horizontal water jet inside the well in such a way that the high velocity streams of water shoot out through the screen openings. By slowly rotating the jetting tool and gradually raising or lowering it, the entire surface of the screen gets the vigorous action of the jet. Jetting with water at high velocity is considered the most effective method of well development. This method is particularly successfully in developing highly stratified unconsolidated formation. Jetting with air can be successfully practiced where water is not readily available. It gives satisfactory results in both consolidated and unconsolidated formation.

5.3. Aquifer Development

Aquifer development, which is also called as aquifer stimulation, is second level development to increase the well yield. Development of limestone or dolomite aquifers can be effectively done with acid. It dissolves carbonate minerals and opens up the fractures and crevices in the formation around the wells. Acid is forced into cracks and fissures much farther from the well bore. Well development techniques are recommended to be used before any aquifer development is initiated.

5.4. Well Strainers or Screens

A well strainer or a screen is a filtering device that serves as the intake portion of wells constructed in unconsolidated or semi-consolidated aquifers. A screen permits water to enter to the well from the saturated aquifer, prevents sediments from entering the well, and serves structurally to support the unconsolidated aquifer material. Well screens play vital role in improving the hydraulic efficiency of a well. The criteria and functions of a screen include:

- It can be developed and manufactured easily with available materials such as steel, iron, brass, plastic, cement and coir etc.

- Minimum tendency of incrustation.
- Minimum head loss through the openings of screen.
- It should offer large %age of open area to the flow.
- The slots or openings should be non clogging, non corroding and non eroding.
- Sufficient strength against aquifer pressure.

5.4.1. Types of Well Strainers

The continuous slot screen is widely used throughout the world for water, soil and gas wells. It is made by winding cold rolled wire, approximately triangular in cross-section around a circular array of longitudinal rods. The wire is attached to the rods by welding. Therefore, each slot opening between adjacent wires is V-shaped, resulting from the special shape of the wire used to form the screen surface. The V-shaped openings, designed to be non-clogging, are narrowest at the outer face and widen inwardly. Any sand grain that will pass through the narrow outer part of the V-shaped opening enters the screen without wedging in the slot. These screens are fabricated in slot sizes ranging from 0.15-6.4 mm width (Driscoll 1987).

5.4.1.1 Slotted Metal Pipe Strainer

Slot openings in the pipe may be cut with a saw, cutting torch or punched with a chisel or die. Pipe with slots, produced by one of several means, is used in water well as a screening device (Fig. 5.4). Important limitations of slotted metal pipe are:

i) Openings are not closely spaced
ii) Percentage of open area is low
iii) Size of slot openings varies significantly

The %age of open area is reduced if adequate casing strength is to be maintained. Slotted steel pipe is not corrosion resistant. In general, its use will increase maintenance cost and may also reduce the life of well.

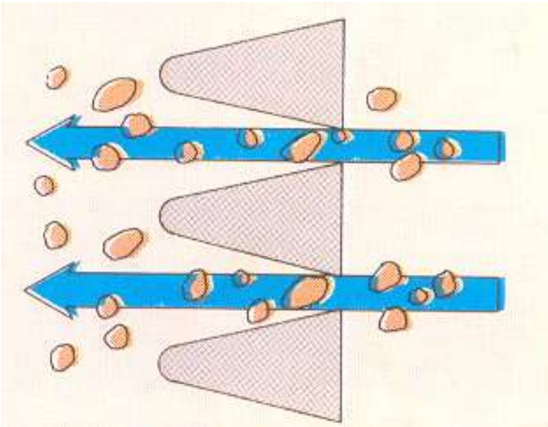
Fig. 5.4 Slotted Metal Strainer (Gravel Packed)

Fig. 5.5 V-Shaped Wire Screen.
Source: Driscoll (1987)

5.4.1.2 V-Shaped Wire Strainer

Such strainers constitute a steel pipe having wide holes around which V-shaped wire is wrapped as given in Fig. 5.5. The wire is placed in such a way that any particle entering into smaller end of V-shape should easily pass through the holes of pipe in order to avoid blocking of strainer as shown in Fig. 5.6.

Fig. 5.6 Particles Passing Through V-Shaped Wire



This type of screen openings allow free passage of particles entering the opening.

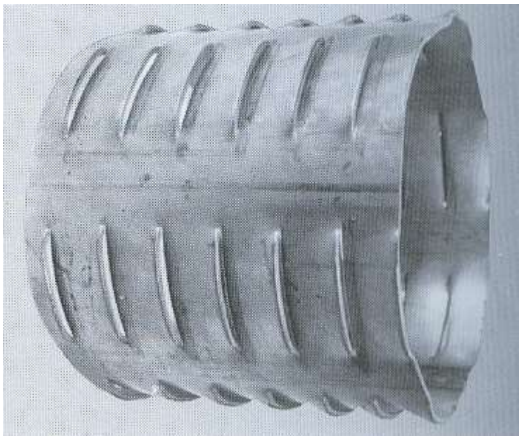
In case the direction of v- shape is reversed, blocking of screen will take place.

Continuous slot screen is another example of winding cold-rolled wire around a circular array of longitudinal bars. The wire is attached to the rods by welding and therefore producing one piece unit having high strength and minimum weight (Driscoll 1987).

5.4.1.3 Punched Slot Strainer

Punched slot screens are produced from a variety of materials including steel and iron sheets by crude method of punching. These are low cost but perform with less efficiency.

Fig. 5.7 Punched Slot Strainer



5.4.1.4 Arc Cut Slotted Strainer or Slotted Metal Pipe

Slot openings in this kind of screen may be produced with a saw or oxy acetylene cutting torch or punched with a chisel and die casing perforator. However, in such screens, percentage of open area is low and the size of slot openings varies significantly. Moreover, at some locations, the openings are so small that it may not effectively control fine or medium sand particles.

Fig. 5.8 Arc Slotted Strainer

5.4.1.5 Steel Strainer

Slotted steel pipe is not corrosion resistant and most methods of perforation tend to encourage corrosion attack. It may increase maintenance cost and reduce the life of well because of corrosion problems.

5.4.1.6 Slotted Plastic Pipe Strainer

Slotted plastic pipe such as PVC is also used to screen wells in some areas particularly in clay rich sediments. Slotted plastic screens are not affected by corrosive water, easy to install and relatively inexpensive. In cold climates, some plastic materials must be handled with care to avoid breakage. Plastic materials are from one-sixteen to one-tenth as strong as stainless steel well strainers. Slotted plastic screens may have less than half of open area of continuous slot plastic screens. The desirable %age of open area in a well screen should be at least equal to the porosity of the water bearing formation sand or filter pack. Greater the open area better is its performance.

5.4.1.7 Slotted Concrete Strainer

Concrete strainers slotted for a given size of opening provided by punching during manufacturing process, are successfully being used in tubewell installations along Lower Bari Doab canal in Sahiwal and Mian Channu. The strainers can be made from locally available materials and technology. They are cheaper than plastic and copper materials. However, they are liable to break during installation if special care is not taken. These strainers cannot be reused. Efficiency of concrete strainers is less than those of steel and plastic materials.

5.4.1.8 Coir Strainer

Coir or coconut fibre is a natural fibre extracted from the husk of coconut and used in products such as floor mats, doormats, brushes, mattresses and tubewell filters. Coir is the fibrous material found between the hard, internal shell and the outer coat of a coconut. Brown coir (from ripe coconut) are used for padding, sacking, horticulture packing and irrigation water tubewells. White coir, harvested from unripe coconuts, is used for making finer brushes, string, rope and fishing nets. The strainer is constructed with steel bars closely spaced in cylindrical fashion with coir string tightly wrapped around the bars. This strainer has been in common use before plastic strainers.

5.5. Pumps

It is a mechanical device for lifting water from a lower level to a higher level which means that a pump primarily imparts energy to the fluid. In its water sense as the pump is used to perform various functions such as lifting, circulating, forcing, exhausting etc., a more appropriate definition of pump would be that it is a mechanical device which when interposed in a pipe line transfers energy from some external source such as a motor or engine to the liquid flowing in the pipe. The pump converts the mechanical energy into hydraulic energy (potential, pressure or kinetics).

5.5.1. Types of Pumps

1) Positive Displacement
2) Reciprocating pumps
3) Gear or Lobe Operated Pumps
4) Centrifugal pumps
5) Multistage Pumps
6) Deep well turbine pumps
7) Jet pumps
8) Submersible pumps

5.5.1.1. Positive Displacement

In a positive displacement pump, the liquid is physically moved from inlet to the discharge point. The positive displacement pump, unlike centrifugal pump or roto dynamic pump, can produce the same flow rate at a given speed, no matter what the discharge pressure is. Thus, positive displacement pumps are constant flow pumps. A positive displacement pump must not operate against a closed valve on delivery line because it has no Shutoff Head like centrifugal pumps. Thus, a safety valve must be provided on delivery line to avoid any damage to the delivery pipe or pump.

A reciprocating pump also is a positive displacement pump, which includes piston type, plunger type and diaphragm type pumps. The liquid is first sucked into a cylinder through a suction value and then displaced or pushed out of cylinder through delivery valve by the thrust of a piston or a plunger. Thus, piston pump consists of a cylinder, piston and a set of valves. The piston is moved back and forth in the cylinder by an external source of power such as motor or engine by converting rotary motion of motor to linear motion of piston through connecting rod. The movement of the plunger produces alternately a negative and positive pressure in the cylinder, due to which the liquid is first sucked into the cylinder and then pushed out of cylinder and is raised to higher level.

Reciprocating pumps can be either single or double acting. The performance of these pumps is limited by the suction conditions and depth to watertable. The suction lift (from watertable to the eye of impeller) is limited to 1 atmosphere, which is approximately equal to a lift of 34 feet. Generally, delivery head is independent of suction head. The delivery head i.e. the height to which fluid can be lifted, depends on the force applied by the prime mover. Priming is a pre-requisite for performance of reciprocating pump unless installed on a pressurized water line (Fig. 5.9).

Reciprocating or piston pumps are capable of developing high pressures but have relatively small capacities. They are not ordinarily suitable for drainage and irrigation, especially if the pumped water is sediment loaded. However, small piston pumps may be used to inject chemicals into pressurized irrigation systems. These pumps are also extensively used for domestic water supplies. Other types of reciprocating pumps include rotary-lobe, gear and roller but none of those are commonly used for medium- to large-scale irrigation or drainage applications.

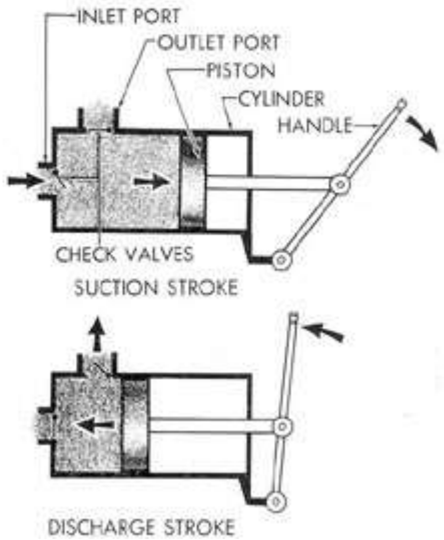


Fig. 5.9 Single Acting Reciprocating Pump

5.5.1.1a Rotary Gear and Lobe Pumps

Rotary type positive displacement pumps, which include gear, screw, shuttle block, flexible vane, sliding vane or flexible impeller types. Rotary pumps move fluid using a rotating mechanism that creates a vacuum, which sucks the fluid in followed by its delivery. Rotary pumps are more efficient because they naturally remove air from the flow lines. Gear type and lobe type pumps are essentially positive displacement pumps. At the inlet, a given volume of water is moved through the space between the gear teeth or lobes, which is displaced by the idler gear teeth to exit to the delivery pipe as shown in Fig. 5.10a. Rotary Lobe pump is shown in Fig. 5.10b, which essentially operates with the same principle as the gear pumps.

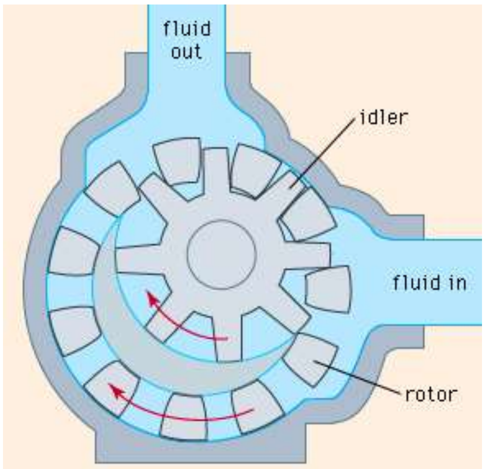


Fig. 5.10a Rotary Gear Pump

Fig. 5.10b Rotary Lobe Pump

5.5.1.1b Diaphragm Pumps

A diaphragm pump or a membrane pump is a positive displacement pump that uses reciprocating action of rubber, thermoplastic or Teflon diaphragm with some type of check valve, butterfly valve or shutoff valve to pump a fluid (Fig. 5.11) There are 3 types of diaphragm valves that include:

Pumps where the diaphragm is sealed with one side is towards the fluid to be pumped and the other towards hydraulic fluid that causes the diaphragm to move back and forth allowing and stopping the flow of fluid. Where the prime mover of diaphragm uses electromechanical mechanism such as a crank or a motor drive, it flexes the diaphragm through simple mechanical action. The other side of diaphragm is open to the air. Where unsealed diaphragms are employed causing the volume of chamber to change and allow the fluid to be pumped on both sides.

In general, the working principle of diaphragm pump (Fig. 5.11) comprises increasing the volume of chamber and thereby decreasing the pressure inside, the fluid is drawn into the chamber. This is followed by moving the diaphragm on other side that decreases the volume and increases the pressure, forcing the fluid to delivery line.

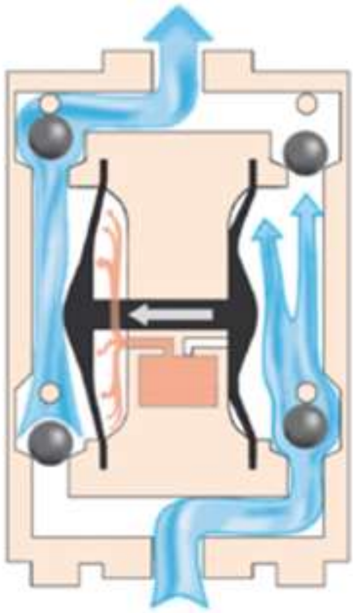


Fig. 5.11 Diaphragm Pump

5.5.1.3 Centrifugal Pumps

The centrifugal pumps utilize the centrifugal force in pumping process. Whenever a body is rotated with a fixed center, there is a force called the centrifugal force, which tends to impel the body outward from the center. In its simplest form (Fig. 5.12), it comprises an impeller, rotating in a casing, which in turn is connected to the suction pipe at one side and a delivery pipe at the other. Before the pump starts functioning, it is necessary that casing, the suction pipe and a portion of the delivery pipe must be filled with water. This is called priming of centrifugal pump.

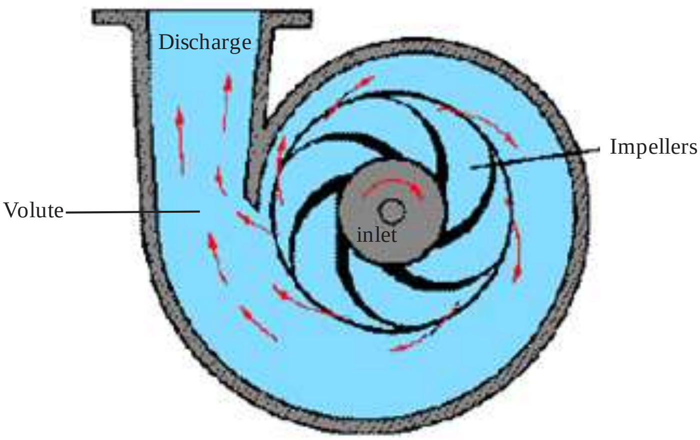


Fig. 5.12 Centrifugal Pump and its Operating Principle

When the impeller starts rotating, it imparts energy to the body of fluid in the vanes and forces it outward with centrifugal force. In turn, the moving water, it creates reduction of pressure at the eye of the impeller which causes the water in the suction pipe to rush into the eye of the impeller. Rapid rotation of the impeller sets up a centrifugal force, which forces the water outwards against the casing through the delivery pipe. Water leaves the casing with a high pressure which is utilized in overcoming the delivery head of the system. The casing is so designed that the velocity head is converted into pressure head before the water leaves the casing. This considerably increases the efficiency of the pump. Suction lift of centrifugal pump is limited by atmospheric pressure, temperature of liquid, relative surface of water body and friction in the suction line.

The kinetic energy of the fluid leaving the vanes is converted to pressure energy at the delivery point by the following methods (Ramamrutham 2006).

1) Volute chamber: It is a spiral casing surrounding the impleller. The water leaving the vanes, is directed to move into the volute chamber circumferentially, where the area of volute increases gradually that allows the velocity to decrease and pressure to increase gradually. Thus, as the water reaches the delivery pipe, a considerable part of kinetic energy has converted to pressure energy leading to the pressure head developed by the pump.
2) Vortex chamber: The impeller is surrounded by a circular chamber and a spiral chamber, which increases the pumping efficiency.
3) Guide Blades: These guide blades are provided in addition to the moving vanes, which provide a gradually increasing area of flow leading to increase of kinetic energy and increase of pressure energy. These guides are fixed over the circular ring around the moving vanes. Such a ring with guide blades is called diffuser.

The centrifugal pumps are used mostly in pumping installation with high head and high discharge requirement. The examples include the following:

1) Irrigation of agricultural fields
2) Drainage of agricultural land
3) Municipal water works, i.e. water supply to the towns
4) Household domestic water supplies

Where a high head and low discharge is a requirement such as muti storey buildings, a reciprocating pump may be preferred. A comparison of characteristics of centrifugal pumps with those of reciprocating pumps is given in Table 5.1.

5.5.1.4 Multistage Pumps

As the name implies, a multistage pump comprises more than one stage/ impeller/pump or bowl assembly. These stages are connected in series on a common shaft. For example, a 4-stage pump will consist of 4 impellers or bowl assemblies producing head against given discharge. The position of these stages may be along a vertical shaft such as a Deep Well Turbine (Fig. 5.13) or along a horizontal shaft

(Fig. 5.14). Under each situation, the total head generated by the multistage pump is the summation of heads produced by each stage.

Table. 5.1 Comparison Between Centrifugal and Reciprocating Pumps

Centrifugal pump	Reciprocating pump
It is high speed, high head pump	It is low speed pump
This is meant for large discharge	It is meant for smaller discharge and large heads
Water moves by centrifugal force	Water is moved physically by plunger through displacement action.
Rotary motion	Reciprocating motion
Delivery and suction heads are inter-dependent	Delivery head is independent of suction.
Head is added up in series connection	Head cannot be added up
Suitable for Irrigation Purposes	Suitable for household Purposes

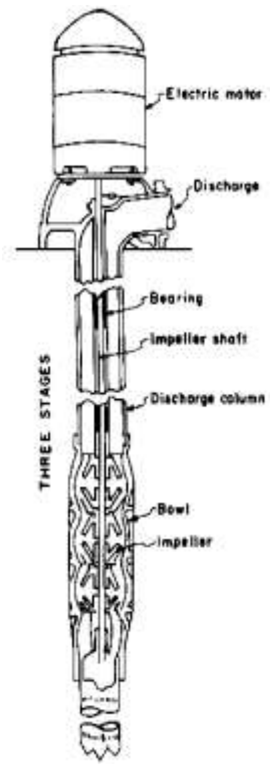


Fig. 5.13 Deep Well Turbine Pump (Vertical Axis Multistage Pump)

Source: Driscoll (1987)

As the water flows along the same axis, whether vertical or horizontal, the total discharge remains the same as the flow rate through first stage. These stages may or may not be located under water. In case of Deep Well Turbine Pump, all the stages are submerged under water. In each case, the lowest or the first stage is connected to the suction line and follows the principles of suction the same way as applied to single centrifugal pump. Other stages are subjected to positive inlet pressure as water moves from Ist stage to the others and therefore, principle of suction does not apply.

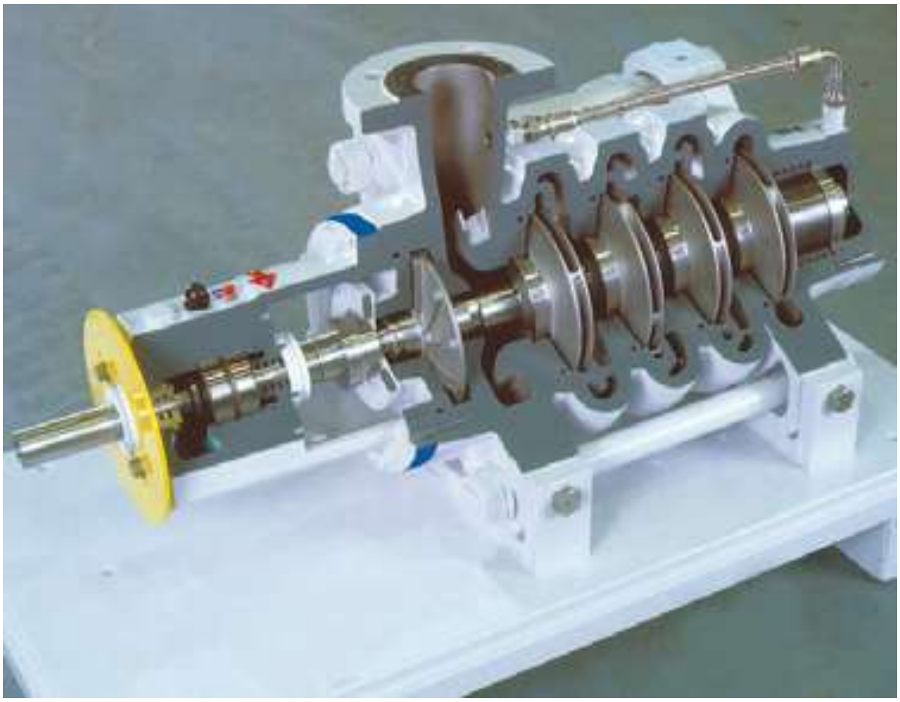


Fig. 5.14 Horizontal Axis Multistage (4-Stages) Pump

5.5.1.5 Deep-well Turbine Pumps

A deep well turbine pump is multistage pump and is also called vertical turbine pump as shown in Fig. 50. When the impellers is suspended vertically in drive shaft within a long discharge pipe, the pump is termed as a deep well turbine unit. The bowl of the pump houses the impeller and guide vanes. When several bowls are connected in series to obtain the desired total head, the pump is referred to as a multiple stage pump. Nearly all of these bowl assemblies are located beneath the water surface.

Deep-well turbine pumps are used for irrigation when the water surface is below the practical lift of centrifugal pumps. The pump is driven by an electric motor or diesel located at the ground surface. These pumps are used for high capacity, large diameter wells. Being submerged under water, the deep-well pumps have the advantage of requiring no priming. However, they have the disadvantage of the operating parts being inaccessible and consequently difficult to inspect. Individual sections of the pump columns may be as large as 3-6 meters supported by bearings. Because of greater length, the shaft and bearings may also cause operating problems.

5.5.1.6 Submersible Pumps

A deep-well turbine pump close-coupled to a small diameter submersible electrical motor is termed as submersible pump (Fig. 5.15).

Efficiency of a pump is increased by direct coupling and effective cooling, resulting from complete immersion under water, which permits a reduction in the amount of iron and cropper in the core. Submersible pumps have been used in wells over 4000 meters deep. Submersible motors as large as 250 hp are in use in 20 centimeter diameter casings.

The principal advantage of submersible pumps is that they can be used in very deep wells where long shafts would not be practicable. They are popular where the entire pumping plant needs to be below ground such as in park and golf courses.

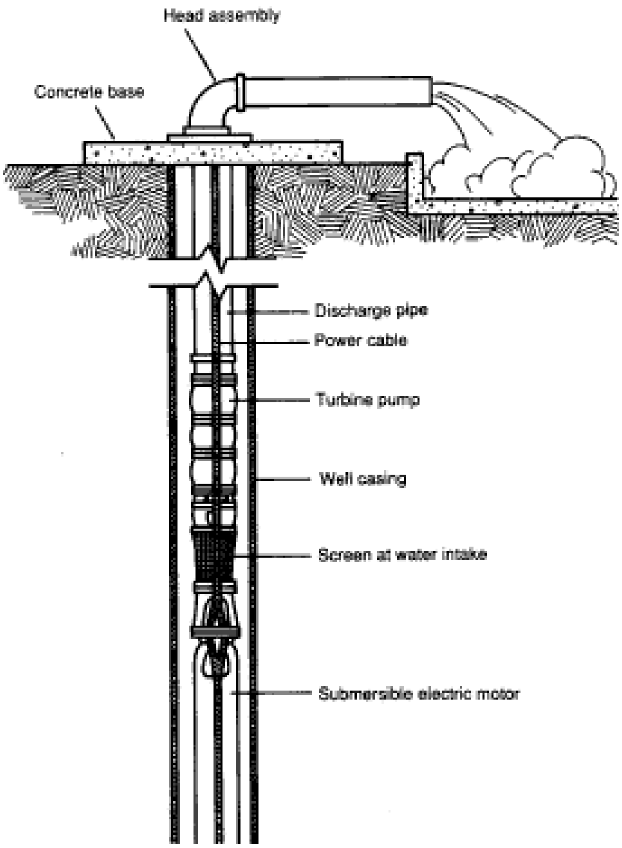


Fig. 5.15 A 3-Stage Submersible Pump

5.5.1.7 Jet Pumps

The jet pump which is locally called as 'Lal Pump' because of the colour of the first introduced model, is primarily, a centrifugal pump combined with a nozzle venturi arrangement. A part of the delivered discharge is re-circulated through the venturi nozzle, which reduces the pressure and thus increases the suction head in the suction line of the pump due to increased velocity as shown in Fig. 5.16.

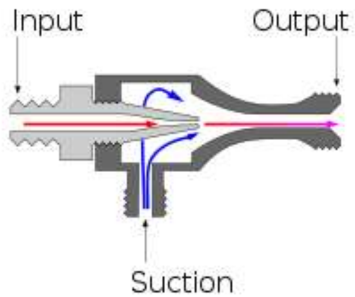


Fig.5.16 Venturi Action in Jet Pump to Enhance Discharge

The increased suction head allows the pump to suck more water from depth deeper than that by the centrifugal pump. Re-circulated water joins the total volume of water flowing beyond the venture. A complete set of jet pump installation showing the components and operating principle is shown in Fig. 5.17. The components include Motor, Impeller, Throat, Nozzle, Volute or Housing, Discharge line, Suction Line, Casing, Foot Valve and Screen.

Where as the jet pump allows greater suction lift, is inefficient when compared with ordinary centrifugal pumps, which makes them unfit for irrigation water pumping. However, this is not objectionable in domestic installations for other benefits. Consequently, the jet pumps are frequently used in house hold installations where low discharge high head and greater suction lift is required. The jet action can also be created by connecting the suction pipe of the pump to a source of pressurized fluid as shown in Fig. 5.16, which may be forced through tapered nozzles, called venture or eductor jet, mounted axially on the inside of the suction line, pointed in the direction of the pump chamber. The passage of the pressurized fluid through the chamber creates additional suction head in the suction line of the pump fetching more water from the water source at greater depth.

5.5.1.8 Propeller Pumps

Propeller pumps are used under low lift and high flow operating conditions. There are two types of propeller pumps, axial flow and mixed flow. The difference between the two is the type of impeller. The axial flow pumps use open impellers and are essentially very low head pumps. A single-stage propeller pump typically will lift water no more than 6m. For greater TDH requirement, more stages need to be added. The mixed-flow pumps use either semi-open or closed impellers similar to the turbine pumps.

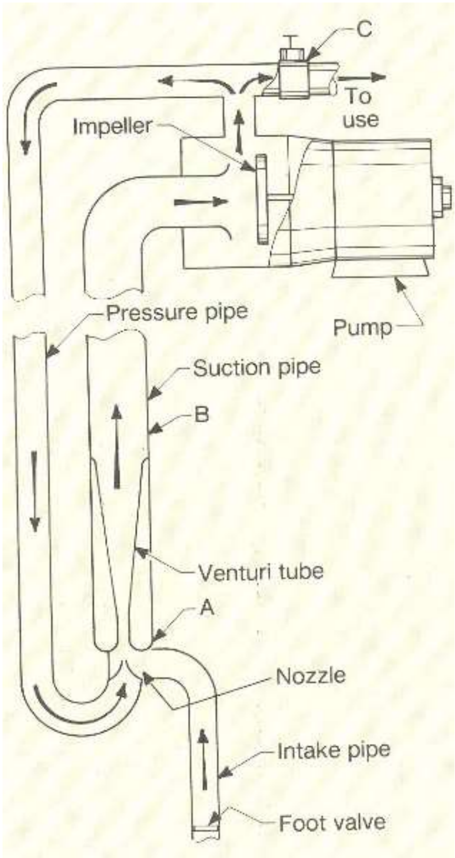


Fig. 5.17 Jet Pump Components and Operating Principle

Source: Driscoll (1987)

In permanent installations, propeller pumps are mounted vertically. Portable propeller pumps are commonly mounted at low angles (in almost horizontal positions) to allow them to pump into pipelines easily. Portable propeller pumps may be powered by the power-take-off (PTO) on tractors. On many farms, propeller pumps are used to pump out waste increase directly with the TDH. These are not suitable under conditions where it is necessary to throttle the discharge to reduce the flow rate. It is important to accurately storage lagoons. Power requirements of the propeller pump determine the maximum TDH against which this type of pump will operate.

Propeller pumps are not suitable for suction lift. The impeller must be submerged to a depth as recommended by the manufacturer. Generally, greater the diameter of pump, deeper is the impeller submergence. Proper submergence depths will ensure that the flow rate is not reduced due to vortices. Failure to observe required submergence depth may cause severe mechanical vibrations and rapid deterioration of the impellers.

5.5.2 Arrangement of Impellers

There are two ways the impellers are arranged in multiple pumping systems.

(i) Impellers in Series

In this case impellers are mounted on a common shaft. The discharge from the first impeller is guided into the inlet of the second impeller and so on and finally the discharge from the last impeller is directed to the delivery pipe.

The total head (Ht) developed is given by summation of heads developed by each impeller (H_1, H_2, H_3,).

$$\mathrm{Ht} = \mathrm{H}_1 + \mathrm{H}_2 + \mathrm{H}_3 + \cdots \mathrm{H}_n \tag{5.1}$$

The discharge 'Q' remains constant for any number of impellers as the same volume of water passes through each impeller.

Thus

$$\mathrm{Q}_1 = \mathrm{Q}_2 = \mathrm{Q}_3 \ldots \ldots = \mathrm{Q}_n \tag{5.2}$$

(ii) Impellers in Parallel

Impellers in this case are mounted on the separate shafts. Each impeller acts as a separate pump unit. The discharge from the various delivery pipes are collected in a common pipe, which finally flows through one delivery pipe. Therefore, total flow (Q_t) is the sum of deliveries from individual pumps Q1, Q2, Q3 and so on.

Thus, total discharge

$$\mathrm{Q}_t = \mathrm{Q}_1 + \mathrm{Q}_2 + \mathrm{Q}_3 \ldots \ldots + \mathrm{Q}_n \tag{5.3}$$

As each impeller discharges through same delivery pipe, same head (H) is maintained at each impeller even if their characteristics are different.

$$\mathrm{H} = \mathrm{H}_1 = \mathrm{H}_2 = \mathrm{H}_3 \ldots \ldots \ldots = \mathrm{H}_n \tag{5.4}$$

5.5.3 Power Requirement for Pumping

Power requirements at various sections of pumping plant are shown in Fig. 5.18.

5.5.3.1 Water Horse Power (WHP)

It is defined as the horse power required to pump water against a given head without consideration of efficiency or friction losses. It is the net output of the pump at the destination of water delivery.

$$\mathrm{WHP} = \frac{\mathrm{QH}}{76} \tag{5.5}$$

Where

WHP = water horse power

Q = discharge (L/sec)

H = Head (m)

If fps system of units

$$\text{WHP} = \frac{\text{r.QH}}{550} \tag{5.6}$$

Where r = unit weight of water (62.4 lbs/ft3)

Q = flow rate, ft^3/sec

H = head, feet

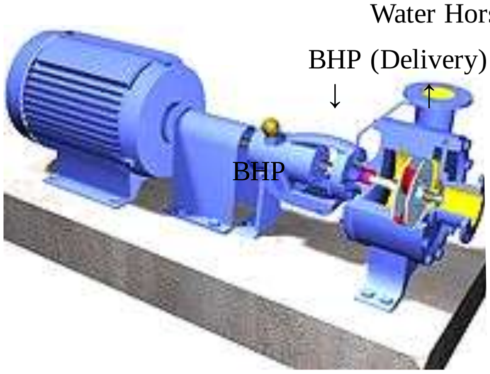


Fig. 5.18 Electric Motor Driven Centrifugal Pumping Unit

5.5.3.2 Efficiency

Efficiency of a system is defined as output divided by input. Input power to a pump is called 'brake horse power' and the output power is called 'water horse power'. It may be expressed in ratio of in %age when ratio is multiplied with 100. Maximum pump efficiency is achieved at about 50% of pump's maximum capacity or 0.7 x shut off head. A pump efficiency of 60-65% is commonly achieved.

(i) Pump Efficiency (Ep)

$$E_p(\%) = \frac{\text{WHP}}{\text{BHP}} \times 100 \tag{5.7}$$

Where BHP = input power of a pump or break horse power of motor

(ii) Motor Efficiency (Em)

$$E_m = \frac{\text{BHP}}{\text{IHP}} \text{ or } E_m(\%) = \frac{\text{BHP}}{\text{IHP}} \times 100 \tag{5.8}$$

Where:

BHP = break horse power or output power of motor/engine

IHP = input horse power or input power of electricity/engine

(iii) Pumping Plant Efficiency (Epp)

It indicates the effectiveness of complete pumping plant including the power source (motor or engine) and pumping unit, thereby it is also called as wire to water efficiency. It is given by the product of Motor Efficiency (E_m) and Pump Efficiency (E_p).

$$E_{pp} = E_m \times E_p \tag{5.9}$$

$$E_{pp}\,(\%) = Em \times Ep \times 100$$

Example 5.1

An irrigator desires to lift a stream of 38 L/sec to a vertical height of 12 m. If the loss of head in the casing and pump results in a 62% pump efficiency, while 91% is motor efficiency, calculate the input horse power which will be required by the motor. How many kilo watts will it use while pumping?

Solution

Q	=	38 L/sec
Ep	=	62%
Em	=	91%
H	=	12 m
IHP	=	?

We know that

WHP = QH/76

Hence

$$\text{BHP} = \frac{WHP}{Ep} \times 100$$

$$\text{BHP} = \frac{QH}{76 \times Ep} \times 100$$

Substituting the values, we get…

$$\text{BHP} = \frac{38 \times 12}{76 \times 12} \times 100$$

BHP = 9.67 hp

We know that

$$\text{IHP} = \frac{BHP}{Ep} \times 100$$

Therefore,

$$IHP = \frac{9.67}{91} \times 100$$

IHP = 10.64 hp

We know that

1 hp = 0.746 kW

Therefore

IHP = 10.635 × 0.746

= 7.933 kW

Example 5.2

Compute the horsepower required to pump a stream of 56.64 L/sec against a head of 13 m and pumping plant efficiency of 60%. What would be the charges per month (30 days) if the motor runs continuously 8 hours per day and cost for 1KWH electricity is 0.5 rupees?

Solution

Q	=	56.64 L/sec
H	=	13 m
Epp	=	60%
IHP	=	?

We know what

Epp = Ep × Em

Or Epp $= \frac{WHP}{BHP} \times \frac{BHP}{IHP}$

Or EPP $= \frac{\text{WHP}}{\text{IHP}}$

Or IHP $= \frac{\text{WHP}}{\text{Epp}}$

We know that

WHP = QH/76

Therefore,

$$IHP = \frac{QH}{Epp \times 76} x100$$

Substituting the volume, we get;

$$\text{IHP} = \frac{56.64 \times 13}{60 \times 76} \times 100$$

IHP = 16.15 p

or

IHP = 1614.74 × 0.746 kw as IHP = 0.746 kw

IHP = 12.05 kw

IHP for one month = 12.05 × 30 x 8

= 2892.00 kwh

Charges for 1 kwh = 0.5 Rs

Charges per month = 0.5 × 2892.00

= 1446.00 Rs

Example 5.3

An Irrigator desires to lift water with a motor driven centrifugal pump from groundwater where suction lift is 8m, discharge head is 10m, Head loss through the system is 1.5m and delivery head is 5.5m. The pump efficiency is 65% and motor efficiency is 85%. If the pump has to deliver water at the rate of 28 lps with daily operational hours of 12 in the month of February, costing @ Rs. 2.5 per kwh, determine the following parameters:

1) WHP
2) BHP
3) IHP
4) Monthly Electricity Bill

Solution

Total Head (H) = 8+10 + 1.5 + 5.5 = 25m

Discharge (Q) = 28 lps

WHP = Q × H /76

= 28 × 25 / 76 = 9.2 HP

= 9.2 × 0.746 = 6.87 kw

BHP = 6.87/ 0.65 = 10.57 kw

IHP = 10.57/ 0.85 =12.43 kw

IHP = 12.43 × 12 = 149.23 kwh

Cost of Electricity for the Month of February = 149.22 × 2.50 × 28

= 10445.40 Rs

5.5.4 Pump characteristics

The characteristics of pump capacity, head, power and efficiency are called the pump characteristics. These interrelations are best shown graphically and the resulting graphs are called the characteristics curves of the pump. The head, power and efficiency are usually plotted against capacity at a constant speed. Typical characteristic Head – Discharge curve, for a pump is shown in Fig. 5.19.

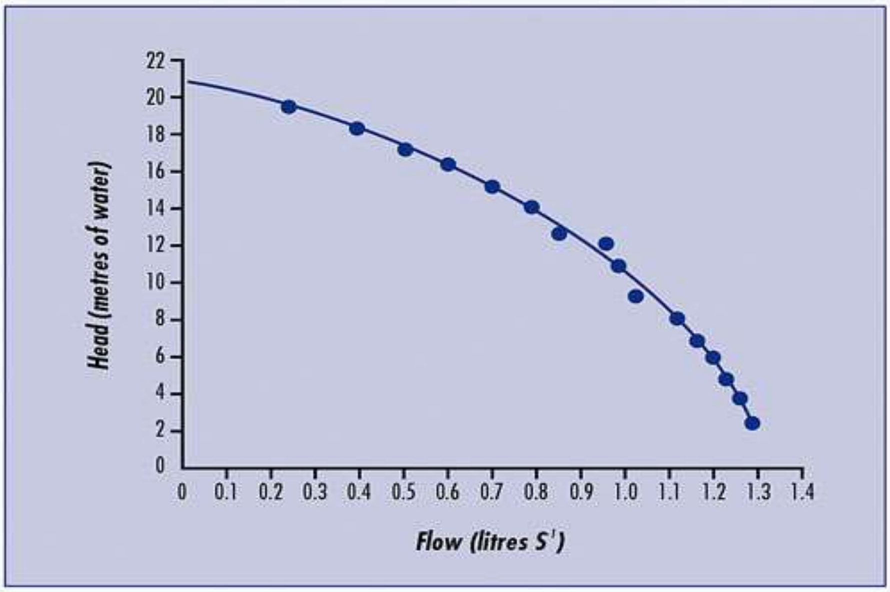


Fig. 5.19 Pump Head – Discharge Characteristic Curve

5.5.4.1 TDH – Q Curve

This curve can be used to evaluate how the discharge (Q) will vary due to fluctuations in the total dynamic head (TDH) of the system. As the head increases the discharge decreases and vice versa. If a pump is operated against a closed valve (Q = 0), the head generated is referred to as the shut-off head.

5.5.4.2 E-Q Curve

There is generally one peak efficiency which is related to a specific discharge. If the pump is operated at this discharge then, for a given amount of energy input to the pump, the output work will be maximized. A pump is selected to operate at a discharge greater than the maximum efficiency point. This is because the operational losses tend to reduce head and discharge and therefore, during operation as the pump operate at lower discharge, it will try to operate at highest efficiency.

5.5.4.3 BP-Q Curve

The input power is referred to as the brake power required to drive the pump. The shape of BP-Q curve can take several different forms depending on TDH-Q and E-Q curves. The most common form for irrigation pumps is similar to the curve of Fig. 23. In other instances, the BP-Q curve may have the highest power demand at the lowest discharge rate, and the required input power will continue to decline as Q increases. It should be noted that even at zero discharge when the pump is operating against the shut-off head, an input of energy is needed.

5.5.4.4 NPSHR-Q curve

Liquid enters into the eye of the first stage impeller (centrifugal or multistage pump) due to the pressure provide by atmospheric pressure minus losses if any. This is called Net Positive Suction Head Available (NPSHA) under the given pumping situation. Thus, NPSHA is a function of the system where the pump has to operate. However, each pump has different requirement of suction head for satisfactory operation which is called Net Positive Suction Head Required (NPSHR) that is a function of pump design. For satisfactory operation of a pum, NPSHA must be greater than NPSHR.

$$\mathrm{NPSHA} = \mathrm{H_a} + \mathrm{H_e} - \mathrm{H_f} - \mathrm{H_v} \tag{5.10}$$

Where:

H_a= absolute atmospheric pressure on the surface of water

H_e = Elevation of liquid above (+) or below the eye of impeller

H_f = Head loss due to friction in suction line

H_v = Absolute vapor of liquid at pumping temperature

The amount of energy required to move the water into the eye of the impeller is referred to NPSHR, and is a function of the pump design. Pump manufacturer specify the NPSHR for their pump design. Under a given situation the engineer computes the NPSHA based on the local altitude, operating temperature, location of eye of impeller with respect to water and losses.

5.5.5 Pump Selection Criteria

The selection of an irrigation water pump must be based on the relationship between the TDH and the flow rate at a given efficiency as shown by the pump characteristic curve curves. Table 5.2 can be used as a guide to narrow down the selection of a pump type for a broad range of flow rates and total dynamic heads as applied to a specific situation. The TDH values given in the Table 5.2 do not include the suction lift. In case the water has to be lifted with a suction lift, a centrifugal pump is recommended. Table 5.3 summarizes the pump characteristics that may help to select an appropriate pump.

Table 5.2 Types of pumps for a given range of flow rates and TDH

Discharge (lps)	**TDH (m)**		
	15 or less	15 to 150	150 or more
0 to 22.8	Propeller Centrifugal	Centrifugal Vert.Turbine Submersible	Centrifugal Vertical Turbine Submersible
22.8 to 380	Propeller	Centrifugal Vertical Turbine Submersible	Centrifugal Vertical Turbine Submersible
380 or more Propeller	Propeller	Centrifugal Vert. Turbine Propeller Submersible	Centrifugal Vertical Turbine

Table 5.3 Summery of pump characteristics

Pump Type and Advantages	Disadvantages
Centrifugal	
1. High efficiency over a range of operating conditions 2. Easy to install 3. Simple, economical and adaptable to many situations 4. Electric, internal combustion engine or tractor power can be used 5. Does not overload with increased TDH 6. Vertical centrifugal may be submerged and not need priming	1. Suction lift is limited. It needs to be within 20 vertical feet of the water surface 2. Priming required 3. Loss of prime can damage pump. 4. If the TDH is much lower than design value, the motor may overload
Vertical	
1. Adapted for use in Turbine wells 2. Provides high TDH and flow rates withhigh efficiency 3. Electric or internal combustion power can be used 4. Priming not needed 5. Can be used where water surface fluctuates	1. Difficult to install, inspect, and repair 2. Higher initial cost than a centrifugal pump. 3. To maintain high efficiency, the impellers must be adjusted periodically 4. Repair and maintenance is more expensive than centrifugals

Pump Type and Advantages	Disadvantages
Submersible 1. Can be used in deep wells 2. Priming not needed 3. Can be used in crooked wells 4. Easy to install 5. Smaller diameters are less expensive than comparable sized vertical turbines	1. More expensive in larger sizes than deep well vertical turbines 2. Only electric power can be used 3. More susceptible to lightning 4. Water movement past motor is required
Propeller 1. Simple construction 2. Can pump some sand 3. Priming not needed 4. Efficient at pumping very large flow rates at low TDH 5. Electric, internal combustion engine and tractor power can be used 6. Suitable for portable operation	1. Not suitable for suction lift 2. Cannot be valved back to reduce flow rate 3. Intake submergence depth is very critical 4. Limited to low (less than 75 feet) TDH

5.5.6 Factors Affecting Pump Performance

Performance of a pump may be influenced by a number of factors including Specific Gravity, Viscosity and Temperature of fluid being pumped. In addition, the performance may also be affected by atmospheric pressure, net positive suction head and cavitation (Sahu 2000). These factors are discussed below.

5.5.6.1 Specific Gravity

The permissible suction lift will vary with the specific gravity of the liquid. The pressure developed by the pump, efficiency and the power requirement will be proportional to the specific gravity.

5.5.6.2 Viscosity

Viscosity of liquid tends to reduce the capacity and head developed by the pump. Moreover, it increases the power requirement and lowers efficiency. Therefore, the viscosity of liquid used in pumping must be considered in evaluating the pump performance.

5.5.6.3 Temperature

As the specific gravity, viscosity and vapor pressure are affected by the temperature, the performance of pump will be affected by the temperature and the suction capacity would decrease as the temperature rises.

5.5.6.4 Atmospheric Pressure

The atmospheric pressure on the surface of the liquid decreases with the height of pumping installation. At sea level, the atmospheric pressure is equal to 1 atmosphere or 1.0332 kg/cm^2. The drop in atmospheric pressure with the increase in the altitude above sea level is at the rate of 1m/1000 m. The permissible suction lift of a pump will reduce with increase in the altitude of pump installation.

5.5.6.5 Net Positive Suction Head (NPSH)

NPSH is the total suction head determined at the inlet nozzle of the pump, which is equal to the atmospheric pressure minus the vapor pressure at pumping temperature. When the pump operates, the fluid enters into the eye of the pump due to the available NPSH, termed as NPSHA. Each pump design requires certain NPSH called NPSHR. The pump will operate successfully as long as NPSHA is greater than NPSHR. Thus, pump performance depends on the NPSHA relative to the NPSHR of a pump. When the vapor pressure of the fluid reaches the atmospheric pressure of the pump installation, the liquid begins to vaporize and cavitation will take place that will reduce the performance of the pump (Colt Industries, 1979).

5.6 Tubewells

A well is a steep sided excavation or drilled hole in the formation deriving water from the zone of saturation through filter and delivering through delivery pipe.

5.6.1 Components of Tubewell

Various components of tubewell including strainer, blind pipe, pump, power unit, delivery pipe etc. are shown in Fig. 5.20. Functions of each component are described below.

i) **Power Unit:** A power unit provides mechanical energy to drive the pump through drive shaft. Power unit consists of electrical.

ii) **Pump:** A pump is a device which converts mechanical energy into hydraulic energy.

iii) **Suction Pipe:** This is a pipe, which runs from pump to the water source. It consists of two parts
   a) Blind pipe
   b) Strainer

iv) **Check Valve or Foot Valve:** The function of this valve is to allow the flow in one direction only. This is installed at the foot of the pipe to allow entry of water in suction line and prohibit the flow of water back from the pump to aquifer or water body, when pump is stopped.

Delivery Pipe: This pipe delivers water from the pump to the destination, which may be an overhead tank or a channel. Deliver pipe is usually placed above ground to allow free fall of discharging.

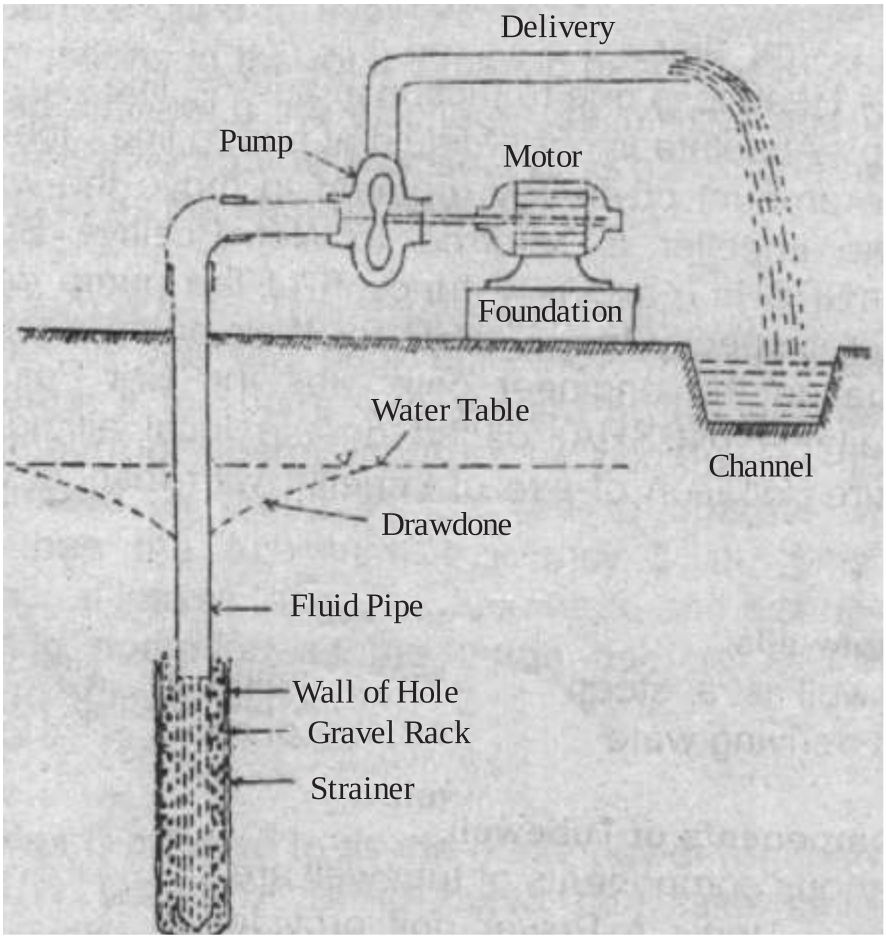


Fig. 5.20 Components of Tube Well

5.6.2 Maintenance of Tubewells

Tubewell maintenance includes providing the proper lubrication to the pump and drive units. Manufacturer's recommendations should be followed. Other time for consideration include the periodic adjustment of impeller clearance in turbine pumps, the periodic maintenance and adjustment of engines to obtain good efficiency, and the need to check the tubewell operation in relation to the changing operating conditions.

In the colder regions where freezing occurs, it is necessary to drain the pipe line and pumps or provide protection against temperature if the pumps must operate during freezing temperatures. A good maintenance program should be implemented. Although there is a cost associated with such a program, yet it will maintain high pump efficiency, help to reduce power costs, improve dependability of equipment, and provide extended tube well life.

5.7 Pump Industries in Pakistan

Because of varied characteristics and uses of pumps in different fields such as irrigation, drainage, agriculture, domestic, industries and cooling systems; a wide variety of pumps are manufactured in Pakistan. Prominent manufacturing companies with their locations are given in Table 5.4.

Table 5.4 Pump Industries and their Location in Pakistan

S. No.	Name of Company	Location
1	Butt Engineering Corporation	Karachi
2	Anwar Pumps	Karachi
3	Kadmarks Technlogy	Karachi
4	PES Engineering	Karachi
5	Allied Group	Karachi
6	Castle Pump	FATA
7	Zahoor Pumps Works	Azad Jamu and Kashmir
8	KSB Pumps	Hasan Abdal and Lahore
9	Mapco Pumps	Lahore
10	MECO Pumps	Lahore
11	PECO Pumps	Lahore
12	OK Tubewells	Lahore
13	Al-Haseeb Corporation	Faisalabad
14	Shehzad Pumps	Faisalabad
15	Golden Pumps	Guranwala
16	Bilal Pumps Industries	Guranwala
17	H.A. Industries	Guranwala
18	MAK Pumps	Peshawar

5.8 Application of Pumps

Fields of application of pumps and their specific uses are summarized in Table 5.5.

Table 5.5 Fields of Application and Specific Uses of Pumps

S.No.	Field of Application	Specific Use
1	Irrigation water application	Drip irrigation, sprinkler irrigation and centre pivot systems
2	Groundwater	Groundwater pumping, skimming wells, scavenger wells and canal water lifting
3	Drainage	Vertical drainage tubewells, surface drainage pumps
4	Agricultural water treatment	Chemigation and fertigation
5	Commercial and domestic building services	Domestic water supply, cooling and heating, waste water disposal, fire protection, water treatment, drinking and animal feeding and milking
6	Food industries and Bio fuels	Cooling and refrigeration, milk processing, beverages and boilers

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Chapter 6

Soil and Water Conservation

6.1. Concept of Soil and Water Conservation

Soil and water conservation is the application of engineering principles to the solution of soil and water management problems resulting in protection against wastage of soil and water. Soil and water management problems generally emerge due to loss of soil and water. Soil erosion is the removal of soil particles by water or wind and their movement across the ground surface. This process may lead to physical damage to the watersheds as well as cultivable lands. Excessive surface runoff, causing loss of water, further accelerates the erosion process. Removal of fine particles and organic matter means reduction in fertility of soil and consequent reduction in yield. The conservation of these vital resources essentially implies utilization without waste so as to make possible a high level of production that can be continued indefinitely.

Study of soil erosion is an essential component of soil conservation. Soil erosion sometimes results in smoothing or leveling process with soil and rock particles being carried, rolled or washed down the gravity and settling at downstream location when we are dealing with geological or natural erosion. The example includes erosion of rocks resulting in soil formation and sediments brought by rivers from upland (Hudson, 1981). Soil erosion is also defined as the degradation of soil which reduces its ability to grow crops or support foundations.

Soil erosion can generally be defined as any degradation of soil, which reduces the ability to grow crops. Such degradation may occur by physical movement of soil mass. Various forms of soil erosion include water erosion, wind erosion, pedestal erosion, pinnacle erosion, piping erosion, slumping, fertility erosion, puddle erosion and vertical erosion (Hudson, 1981). When easily erodible soil, protected by stones or tree roots, is exposed to splash erosion pedestals, capped by the protecting material, develop leaving the surrounding soil eroded. Pinnacles or vertical lumps of soil develop in the bottom or sides of gully erosion, which result from highly erodible soils around tough soil material.

Piping erosion refers to formation of continuous pipes or channels underground or below the soil surface mainly because of loose subsoil. Slumping is usually a process of geological erosion, which may be prominent in high rainfall areas with deep soils. Examples of slumping include river bank collapse, progressive gully erosion and coastal erosion. Fertility erosion refers to the loss of plant nutrients by erosion. Puddle erosion takes place by physical breakdown of soil structure by rain and washing of fine particles into depressions resulting in structure less soils with poor productivity. Vertical erosion is washing of fine clay particles to lower horizons of soil profile because of presence of porous sand or gravel at lower layers.

6.2. Importance of Soil and Water Conservation

Depletion of watersheds and erosion of soil are indeed very serious problems in Pakistan. The sustainability and efficient operation of storage dams and Indus Basin Irrigation system are largely dependent on proper management of northern watersheds. Because of excessive cutting of trees and vegetation overgrazing and improper cultivation on steep slopes, accelerated soil erosion is excessively draining silt load into the Indus and its tributaries. Soil at the rate of 2000 to 4000 tons per square kilometer is reported to be eroded annually from our watersheds. In case of Mangla reservoir, the incoming sediment load has been estimated to about 5178 ha m annually.

In addition to threatening the life of irrigation facilities, the country's land resource is greatly affected by water erosion. It is estimated that 36% of the total land is affected by water erosion. Out of this, about 20.5% lies in northern area and potohar area. Therefore, understanding of soil and water conservation concepts and measures and measures is very vital to agricultural and engineering graduates.

6.3. Agents of Erosion

The main agents which loosen soil particles include water and wind. Accordingly, the resulting erosion will be referred as water erosion or wind erosion accordingly.

6.3.1. Water Erosion

The process of water erosion constitutes two distinct but not separable, phases – splash erosion and wash erosion. Splash erosion is the detachment of soil particles from their moorings by the impact of rain drops, while wash erosion is the removal of soil from the land by running water including runoff from melted snow or ice. Water erosion is more pronounced in Rawalpindi, Jehlum, Peshawar, Chakwal, D.I. Khan, D.G. Khan and Kohat, particularly the areas hit by hill torrents.

6.3.1.1. Factors Affecting Erosion by Water

a) Climate
b) Soil
c) Vegetation
d) Topography
e) Flow Velocity Flow Rate of Water

a) Climate

(i) Precipitation
(ii) Temperature

(iii) Humidity

(iv) Solar radiation

i. **Precipitation:** Refers to the rainfall erosivity which accounts for the energy of falling drops of water. The intensity, drop size, distribution and duration of rainfall are effective indices of erosivity.

ii. **Temperature and wind:** Both the factors affect evaporation and transpiration. Wind also changes raindrop velocities and the angle of impact. Under higher temperature, lack of moisture and concentration of salts on the surface layer of soil tends to reduce the granulation of soil particles. Therefore, individual soil particles can easily be eroded by rainfall run off.

iii. **Humidity and Solar Radiation:** These factors are somewhat less directly involved in that they are associated with temperature. Solar Radiation raises the temperature of the earth and atmosphere and thereby increases the rate of evapotranspiration.

b) Soil

Physical properties of soil affect the infiltration capacity due to which soil can be dispersed and transported. The soil characteristics contributing to water erosion include:

i) Texture – silty and clayey soils are more susceptible to water erosion. In creasing soil contents tend to reduce erosion by increasing the infiltration rate and reducing runoff rate.

ii) Structure – structureless soils are easily eroded by water as compared to granular structured soils.

iii) Organic Matter – organic matter not only increases the infiltration rate but also binds the soil particles to form granules. It increases compactness of soil particles against water erosion. Soils having low organic matter are more vulnerable to water erosion.

iv) Moisture Contents – greater moisture content in the soil tend to increase runoff. However wet conditions improve biological activities and encourage vegetative cover which protects the soil from erosion.

v) Density or Compactness – compact soil having higher density provide more resistance to water erosion as compare to loose soil.

vi) Chemical and biological characteristics – Higher sodium contents tend to deflocculate the soil and thereby increase the soil erosion. A well developed root system improves the infiltration characteristics of soil that reduces the rate of runoff, which helps decreasing the rate of water erosion.

c) Vegetation

Effects of vegetation in reducing erosion are:

- Interception of rainfall by absorbing the energy of rain drops and reducing runoff.

- Retardation of erosion by decreased surface velocity.
- Physical resistance to soil improvement.
- Improvement of aggregation and porosity of the soil by roots and plant residues.
- Increased biological activities in the soil.
- Transpiration, which decreases soil moisture, resulting in increased storage capacity of soil.
- These vegetative influences vary with the season, crop, degree of maturity, soil and climate etc.

d) Topography

Topographic factors influencing erosion include:

- Degree of slope
- Length of slope
- Size and slope of watershed. Steeper slope tends to increase water erosion because of increased energy of moving water particles. The degree of erosion decreases with increasing length of slope, longer gathering time, and lower maximum rate of runoff.

e) Flow Velocity and Flow Rate

- Flow velocity of water depends on the degree of slope and roughness of soil topography and concentration of water. The erosion also depends on the flow rate. A higher flow rate would develop greater erosive energy to erode the soil under given conditions of soil, topography, vegetation and climate.

6.3.1.2. Types of Water Erosion

(i) Rain-Drop or Splash Erosion

It is the soil splash resulting from the impact of water drops directly on soil particles. Soil particles get displaced from their moorings and may become part of runoff. In fact, it is the point of initiation of water erosion. The kinetic energy of falling rain drops disintegrate the soil crumb into finer component particles, and then splashes away to down hill. The moving particles also cause scaling of pore spaces, which in turn tend to increase runoff, thus accelerating erosion process.

(ii) Sheet Erosion

It is uniform removal of soil in thin layers from sloping land, resulting from over-land flow occurring in thin sheet of water. This stage of erosion may cause loss of plant nutrients, which may be 3 to31 times the amount of nutrients extracted by harvested crops. Sheet erosion destroys the crumb structure of soil, thereby reducing the soil to an amorphous powder. Of all the damages caused by water erosion, sheet erosion is the most demonstrating as it removes the top soil, which is the productive portion of land.

(iii) Rill Erosion

It is the removal of soil by water from well defined channels. When there is a concentration of over-land flow. It is advanced stage of sheet erosion. Rills can be overcome by farm operations.

(iv) Gully Erosion

As shown in Fig.6.1, Gully erosion produces channels larger than rills. These channels carry water during and immediately after rain. Gullies cannot be removed by tillage as in the case of rill erosion. It is an advanced stage of rill erosion (Frevert et al., 1981). Gully erosion is well developed in unstable and unprotected steeper lands. Gully erosion is quite pronounced in Balochistan and Pothohar area. Gullies not only take the land out of cultivation and reduce area under crops, but also create problems in trafficability, efficient management and use of land. Major factors influencing the gully development include type and condition of soil, presence of compacted layer on or near the surface, type and intensity of storm, absence of protective vegetative cover etc.

Fig.6.1 An Example of Gully Erosion

Gullies may be classified as small, medium and large with depth ranging <1m, 1-5m and > 5m, respectively. Cross sectional shape may develop as V- or U- shaped depending on soil and climatic conditions, age of gulley or type of erosion (Frevert et al. 1981).

6.3.1.3. Control Practices

1) **Contouring:** In this case, field operations like plowing, planting, cultivation and harvesting are done on the contours. It produces surface runoff by ponding water in small depressions and decreases the development of rills in which the high water velocity results in destructive erosion.
2) **Tide Ridging:** It consists of converting the field surface with closely spaced ridges in two directions at right angle so that the land surface is formed into a series of rectangular depressions to retain surface runoff (Hudson, 1981).
3) **Strip Cropping:** It is the practice of growing alternate strips of different crops in the same field. For controlling water erosion, strips are always on contours, but in dry regions strips are placed x-wise to the prevailing wind direction for wind erosion control.
4) **Tillage Practices:** Tillage is the mechanical manipulation of the soil to provide soil conditions suitable to the growth of crops, control of weeds, and for the maintenance of infiltration capacity and aeration. Tillage is a primary tool in applying effective conservation to the land. The tillage practices consist of plowing, disking, harrowing and cultivation. The effect of tillage upon erosion is a function of the following factors:
   a. Aggregation
   b. Surface sealing
   c. Infiltration
   d. Resistance to wind movement
5) **Storm Water Diversion Drain:** It is a channel or a drain, which intercepts storm water or flood water and leads water safely to the down slope to the arable land. It acts as first line of defense against erosion of arable land receiving flood water from upland. It may also be called diversion ditch or diversion terrace. The examples can be seen along motorway running from Islamabad to Lahore in Pakistan.
6) **Terracing:** Terracing is a practice to reduce runoff, soil erosion, and sediment delivery from upland areas by constructing broad channels across the slope of rolling land. Terracing is accomplished by constructing broad channels across the slope of rolling land. Terracing of cultivated land is combined with contouring for effective conservation. The major function of a terrace is to decrease the length of hill side slope, thereby retaining runoff in soil and reducing chance of sheet, rill and gully erosion. There are many types of terraces. However, the bench and broad base terraces are the common ones. Moreover, the terraces may be graded or level.

For irrigation purposes, the terraces may be further divided into Channel terrace, Bench Terrace and Irrigation Terrace (Hudson, 1981). The Channel Terrace is excavated channel across the steep slope to intercept the runoff flowing down and overcome water erosion. A grassed waterway along the steep slope may be built to receive water from a number of channel terraces. A Bench terrace converts a steep slope into a series of leveled steps along the slope, which may also be called as Bench

Terraces or Level Benches. A flat Bench terrace having raised lips at the outer edge to retain irrigation water for production of rice crop, tea or fruit trees and high value crops, is called irrigation channel (Huson, 1981).

During the 1940s and 1950s Broadbase Terraces with vegetated outlets were commonly installed. Since the 1970s, parallel terraces with steep, grassed back slopes have become popular. Since terracing requires additional investment and causes some inconvenience in farming, it should be considered only where other cropping and soil management practices will not provide adequate erosion control or water management.

6.3.1.4. Functions of Terraces

- Terraces decrease the length of the hillside slope, reducing rill erosion.
- Prevent formation of gullies by diverting overland flow.
- Allow sediment to settle from runoff water, and improve the quality of runoff water leaving the field.
- In drier areas, terraces serve to retain runoff and increase the amount of water available for crop production.
- Retention of water reduces the risk of wind erosion.
- Terracing can aid in surface irrigation on steeper land, particularly in paddy rice production.
- Diversions can protect downhill lands, buildings, and special areas from unwanted overland flow and water erosion.
- In sensitive forest watersheds, similar structures may be constructed after a severe wildfire to reduce upland erosion and downstream sedimentation.

6.3.1.5. Terrace Classification

Terraces may be classified according to the alignment, cross section, grade or outlet as summarized below:

a) **Nonparallel Terraces:** These follow the contour of the land regardless of alignment. Some minor adjustments are frequently made to eliminate sharp turns and short rows by installing additional outlets, using variable grade, and installing vegetated turning strips. Nonparallel terraces are best suited to applications other than row crop farming, such as small-grain agriculture or pastures.

b) **Parallel Terraces:** These terraces are constructed parallel to each other across the slope. They are preferred for row-crop farming operations. They generally require greater cut and fill volumes during construction than nonparallel systems.

c) **Bench Terraces:** These are built on steep slopes ranging from 20 to 30%. They are more efficient at distributing water under both irrigated and dryland production. Bench terraces on very steep slopes may be too narrow or too inaccessible for mechanized farming systems. These terraces are generally inaccessible by machinery and therefore constructed with manual labor.

d) **Broadbase Terraces:** These include the three-segment section, the conservation bench, and the grassed backslope terrace. The three-segment section terrace is more common in mechanized farming systems on moderate slopes i.e. 6-8%. All slopes on the three-segment section broadbase are sufficiently flat for the operation of farm machinery. Lengths of each side slope are designed to match the width of equipment that operates on those slopes. The conservation bench variation incorporates a wide, flat channel uphill of the embankment to provide a maximum area for infiltration of runoff. The grassed backslope terrace is constructed with a 2:1 backslope that is usually seeded to permanent grass because it is too steep to farm. Fill soil may be obtained from the lower side of the terrace, which tends to reduce the land slope between terraces, improving farmability. When field slopes are uniform, a constant terrace cross section is recommended.

e) **Graded or Channel-type Terraces:** These terraces control erosion by reducing the hillside slope length of overland flow, and then by conducting the intercepted runoff to a safe outlet at a nonerosive velocity. The reduced flow velocities in the channel minimize channel erosion and promote deposition of eroded sediment. Because of the importance of constructing and maintaining a satisfactory channel, graded terraces should not be built on soils that are too stony or steep, or that have topsoil too shallow to permit adequate construction. Water conservation may be a primary or secondary benefit of a graded terrace system.

f) **Level Terraces:** Level terraces are constructed to conserve water and control erosion. In low to moderate rainfall areas, the rain water is trapped and held in the terraces for infiltration into the soil profile. They may be suitable for this same purpose on permeable soils in high-rainfall areas. They are designed with embankment on both sides to store the design runoff without overtopping or piping through the embankment. The channel is level and is closed at both ends to allow the moisture to infiltrate into the soil for crop use. Overtopping may cause excessive erosion and failure of the terrace.

g) **Blocked Terrace:** These terraces are blocked at both downstream anmd upstream to allow all water to infiltrate in the root zone of terrace channel.

h) **Permanently Vegetated Terrace:** This is also called as Grassed Waterway or a Vegetated Water Way where water is allowed to seep into the soil and excess water is allowed to safely drain through vegetated or removed through subsurface pipe drains. Combinations of outlets may be employed to meet specific conditions.

6.3.1.6. Universal Soil Loss Equation

Soil erosion by water is influenced by a number of factors such as soil, water, rainfall, soil topographic conditions, crop management and conservation practices. The Universal Soil Loss Equation attempts to isolate each variable and represent in quantitative values so that when numbers are multiplied, the resultant is quantity of soil loss (Hudson, 1981). The Universal Soil Loss Equation is given as;

$$A = R \times K \times L \times S \times C \times P \tag{6.1}$$

Where:

A= Soil Loss in tons (1 ton = 2000 lbs) per acre

R= Rainfall Erosivity Index – a number, which indicates the effect of rainfall based on EI30 index.

K= Soil Erodibility Factor – a number, which indicates the liability of soil type to erosion.

L= Topographic Length factor – a ratio, which compares the soil loss with that from a field of specified length of 22.6 meters.

S= Topographic Slope Factor – a ratio, which compares the soil loss with that from a field of specified slope of 9%.

C= Crop Management Factor – a ratio, which compares the soil loss with that from a cultivated bare fallow field.

P= Conservation Practice Factor – A ratio, which compares the soil loss with that from a field with no conservation practice i.e. plowed up and down the steepest slope.

Soil loss is measured as annual loss of soil, which occurs under the conditions defined by the other factors of equation. Erosivity ® is defined as the potential ability of rain to cause erosion, which is related to the physical characteristics of rainfall including drop size, falling velocity and intensity etc. The EI30 index is the product of kinetic energy of the storm and the 30-minutes intensity. Practically, it is the greatest average intensity of rainfall experienced in any 30 minutes period during a storm, computed from recording rain gauge chart (intensity vs. Time duration).

Erodibility (K) is the vulnerability or susceptibility of soil to erosion. It is affected by the physical properties of soil such as texture, structure and organic matter. Soil erodibility tends to decrease with greater sand contents and increase with greater silt content of soil. The values of erodibility of 10 tropical soils, tested under laboratory conditions, ranged from 0.043 to 0.280 (Choudhry, 1973).

Effective Length of Slope (L) is the distance between the two channel terraces. However, the influence of L and S factors may be joined to single factor LS to represent the effect of both on the soil loss. Soil loss increases as the degree of slope as well as the length of slope increase. The graphic representation of LS is given by Hudson (1981).

The Agricultural Handbook 282 lists 128 possible cropping practices to evaluate Cropping factor in soil loss equation. Table 6.1 summarizes some of the annual values of crop practice factors as given by Hudson (1981).

Table 6.1 Annual Values of the Crop Practice Factor C

S. No.	Crop Period	Crop Practice Factor - C
1	Fallow	180
2	Seeding	168
3	Establishment	114
4	Growing and Maturing	564
5	Residue and Stubble	414

The standard conditions with respect to the conservation practices considered in the Soil Loss Equation are that bare soil is cultivated up and down the steepest slope. Based on the field observations, Hudson (1981) has listed the P values for conservation practice such as contour cultivation without other mechanical protection works as given in Table 6.2.

Table 6.2 Values of Conservation Practice P

S.No.	Topographic Slope (%)	Conservation Factor P
1	1-2	0.40
2	2-7	0.50
3	7-12	0.60
4	12-18	0.80
5	18-24	0.90

Example 6.1

Using Soil Loss Equation (59), determine the annual soil loss (tons per acre) for the field data (Hudson, 1981) given below:

Rainfall Index (R)	= 300
Soil Erodibility (K)	= 0.33
LS Factor	= 1.0 (For 4% slope and 600 ft length of slope)
Conservation Practices	= 0.5
Cropping Practices	= 0.2

Solution

According to the Soil Loss Equation 6.1

$$A = R \times K \times LS \times C \times P$$

$$A = 300 \times 0.33 \times 1.0 \times 0.2 \times 0.5$$

$$A = 10 \text{ tons per acre per year}$$

6.3.2. Wind Erosion

It is the removal of soil particles from the soil surface by wind energy. Although humid areas may also be affected by wind erosion, the most affected areas include, silty and sandy soils and deserts. The Cholistan and Thal deserts are some of the examples where wind erosion is pronounced. Formation of sand dunes in Bhakhar and Khoshab is also the result of wind erosion. In general, wind erosion in Pakistan is quite pronounced in districts of Mianwali, Sargodha, Muzaffar Garh, Hyderabad, Khairpur, Bahawalpur, Kalat and Karachi.

6.3.2.1. Mechanism of Wind Erosion

It involves two processes

i) Detachment
ii) Transportation

The abrasive action of wind results in detachment of tiny soil particles from the granules or clods of which they are a part. When the wind is laden with soil particles is abrasive action is greatly increased. The impact of these rapidly moving grains, dislodges other particles from soil clods and aggregates. Later, the soil particles are carried away by the following three methods:

Suspension: Soil particles remain suspended while being transported normally at a height more than one meter. Suspension is the movement of very fine soil particles, mainly less tha 0.1 mm diameter.

Saltation: It is movement of soil particles by bounces along the surface of ground.

Surface Creep: It is the movement of soil particles very close to the surface of soil.

6.3.2.2. Factors Affecting Wind Erosion

- Moisture contents
- Wind velocity
- Soil surface conditions
- Soil characteristics
- Nature and orientation of vegetation
- Mechanical stability of dry soil clods or aggregates
- The presence of stable soil crust
- Bulk density and size of erodible soil fractions

6.3.2.3. Control of Wind Erosion

Methods of reducing surface wind velocities for effective control includes:

- ➢ Vegetative measures
- ➢ Tillage practices
- ➢ Mechanical Method

Vegetative measures are generally most effective and economic means of controlling wind erosion. It consists of inter-tilled crops, close growing crops and woody plants such as shrubs and trees. The tillage practices include contour strip cropping, primary and secondary tillage practices. Mechanical barriers such as wind breaks comprise brushy fences, board walls and surface protection measures such as brush matting, rock and gravel. Wind erosion can be effectively controlled by the following measures:

i) If soil is moist, there is very little danger of erosion.
ii) A vegetative cover also discourages soil blowing.
iii) By roughing the soil surface, the wind velocity can be decreased.

Strip cropping and alternate strips of crops and fallow land should be perpendicular to the wind.

6.4. Conservation of Soil and Water Resources

Conservation of soil and water resources at every level from the origin to the point of use are needed to ensure food security. Technologies to conserve both the resources may be accomplished through the following means as summarized below Ahmad and Chaudhary (1988).

1) Watershed management.
2) Improving and lining of water conveyance channels including canals, distributaries, minors and watercourses.
3) Improved farm layout.
4) Replacing flooding irrigation to furrow-bed and furrow-ridge irrigation practices.
5) Use of sprinkler and drip irrigation techniques for selected crops to improving irrigation efficiency.
6) Reuse of waste water.
7) Developing farm water storage reservoirs.
8) Construction of mini dams.
9) Conserving flood water through mini as well as major storage dams at appropriate locations.
10) Soil and water conservation in hilly areas through contouring, strip cropping, terracing, tillage practices and constructing conservation structures.
11) Rainwater harvesting.
12) Reducing evaporation from open surfaces.
13) Use of salt tolerant plants in saline groundwater areas.

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Chapter 7

Drainage of Agricultural Land

Drainage of agricultural land is the natural or artificial removal of excess water from the soil to create favorable environments for plant growth. It provides means of controlling water logging in the soil. Presence of excess water in the soil and consequent saturation of root zone causes obstructed aeration of plant roots and therefore, adversely affect the production of the crops. Waterlogged conditions may result from a number of factors including impaired downward or lateral movement of water, impermeable layers in soil profile and excessive seepage from a number of sources as summarized below:

7.1. Sources of Excess Water

1) Seepage losses from rives, reservoirs, canals and water course system.
2) Deep percolation losses (irrigation return flow) from irrigated lands.
3) Flooding of lower valley lands are subjected to excess water because of irrigation of the high land.
4) Perched watertable due to underlying impermeable layer.

7.2. Types of Drainage

Depending on the location of excess water, land drainage may be categorized as surface and groundwater drainage.

7.2.1. Surface Drainage

If the water stands on the surface of the land, the problem is the surface drainage. It may be the result of impaired infiltration into the soil or due to impermeable soil layer.

7.2.2. Groundwater Drainage

It results from the rise of watertable close to the surface of land saturating the root zone of crops. Groundwater when rises above the surface of land, groundwater drainage is also accompanied by surface drainage.

Frequently the presence of high water table is not evident from an inspection of the soil surface. In many instances, the soil surface may appear to be dry, although waterlogged conditions at depths of two or more feet beneath the surface may cause serious damage to the crops.

7.3. Purpose of Drainage

The major purposes of drainage can be stated as follows:

1) To provide a root environment that is suitable for maximum growth of plants. It includes maintaining a desirable moisture-air balance in the root zone.
2) To increase crop production and to sustain yields over a longer period of time.
3) To maintain soil fertility.
4) To maintain healthy environmental conditions.
5) Adequate drainage permits greater control of the soil water and hence greater ease in the conduct of the farming operations.
6) To reduce the damage resulting from 'scalding' of the crops which is discoloration of plant tissues due to improper conditions of growth.
7) To improve the soil structure destroyed by water logging.
8) To raise the temperature of soil and to improve the biological activities.

7.4. Influence of Waterlogging and Drainage on Physical Soil Conditions

Physical soil conditions influenced by drainage are structures, aeration, organic matter and temperature, as discussed below:

7.4.1. Soil Structure

Good soil structure refers to the arrangement of soil particles forming granules. Waterlogged conditions destroy structure where as drainage improves soil structure. Structure provides favorable conditions for simultaneous aeration and storage of soil moisture. It reduces mechanical impedance to root growth and stabilize traction for farm implements. In soils with groundwater depth less than 60 cm below the soil surface, a deterioration of structure leading to more compact and sticky top soil than in soil with deeper groundwater is faced (Smedma and Rycoft, 1983).

7.4.2. Soil Aeration

The volume of air is in the soil varies inversely with the water content of the soil and is very low in waterlogged or flooded soil. When a soil is permanently flooded, oxygen disappears within a few days. In a well-drained soil, air not only penetrates into deeper soil layers, but the volume of air in the surface layers is much greater.

7.4.3. Soil Organic Matter

Organic matter is very important for improving soil structure as well as for supply of nutrients. Loss of organic matter will have adverse influence on soil structure, which will impede internal drainage. The soil would become compacted leading to restricted root penetration. Organic matter must then be supplied either in the form of stable manure, compost or green manure. In organic soil, drainage may lead to subsidence of the land surface. The subsidence is caused by shrinkage due to irreversible drying, oxidation and compaction.

7.4.4. Soil Temperature

The outflow of water and inflow or air drainage will result in a lowering of specific heat of the soil. This means that the soil will warm up sooner, but at the same time it will lose its heat sooner. Waterlogged soil requires five times more heat to raise its temperature than the dry soil does. Consequently, waterlogged soil, with approximately 50% moisture, requires about 2.5 times more heat to warm up than a dry soil does. In addition, the cooling effect of the great evaporation from a soil delays a temperature rise.

7.4.5. Root Development

The plant roots require aerated zone for their development. It has been observed that under high watertable conditions, the roots do not enter watertable. Instead, they tend to spread laterally in the zone above watertable. This tends to reduce the nutrient availability and results in stunted growth. Thus, drainage of such soils improves root development and plant growth.

7.5. Measures to Control High Watertable

The following measures would help the controlling high watertable.

- Lining of canals to reduce seepage loses should be encouraged.
- More efficient application of irrigation water (irrigation scheduling) can reduce the deep percolation losses.
- Proper maintenance of the natural drainage system helps in reducing the deeper percolation of water from rain and snow melting.
- Artificial removal of water from the root zone soil by:
    a. Opens drains
    b. Tile or pipe drains
    c. Pumping groundwater

7.6. Field Drainage

A field drainage system is a system that receives excess water directly from the farm or field and conveys it to the main drainage system, which evacuates the water from the area. The main drainage system must provide a free and reliable outlet for the field drains. Components of a field drainage system include, field laterals, collectors and main drains.

Field laterals also referred as field drain, farm drains or suction drains. Laterals serve primarily to check the fluctuations of the groundwater table, but may also collect surface runoff. The water in the laterals flow into collectors, which may convey it to the main drainage system through that, is further carried to the outlet of the area. The rivers, lakes, evaporation ponds and barran areas are acting as drainage outlet. However, environmental problems may emerge from diverting drainage effluent to these good quality surface water bodies.

7.7. Classification of Drainage Systems

7.7.1. Vertical or Tubewell Drainage

Vertical drainage means extraction of excess water from a waterlogged land in vertical direction, which may be accomplished through tubewells as shown in Fig.7.1. These tubewells are specifically installed for lowering the watertable to some acceptance level. Drainage with tubewells is generally practiced where drainage of groundwater at deeper depths is desired.

The success of tubewell drainage depends on many factors including hydrological conditions of the area and aquifer characteristics (Boehmer and Boonstra, 1994). Groundwater drainage in the Indus Basin of Pakistan, where turbines of >100 lps capacity have been installed at a depth of 70 m along the canals under Salinity Control and Reclamation projects, is a successful example of tubewell drainage.

Thus, control of watertable and salinity can be achieved effectively using dug wells and tubewells. This is the quickest method of removing the excess water from deeper depths of soil as has been experienced in many parts of Pakistan under the Salinity Control and Reclamation Projects (SCARP), which were initiated by WAPDA in 1960. By 1997 more than 55 SCARP’s had been implemented which comprised about 2500 tubewells. In SCARP areas, the tubewells have effectively lowered the watertable. However, high salinity of pumped water has created increasing soil salinity challenges to the SCARP areas.

The pumped water can be disposed off to some outlet or used for irrigation purposes singly or in combination with canal water depending on the quality of pumped water. The pumping activities not only lower water but also provide additional source of irrigation water. Approximately more than 700,000 tubewells including SCARP tubewells operating in the Indus Basin are pumping more than 48 maf of water annually in both public and private sectors.

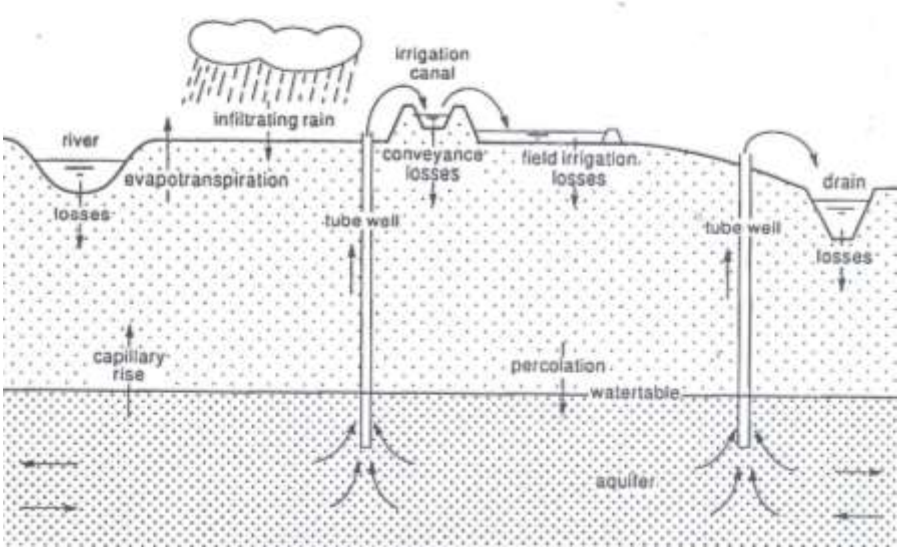

Fig. 7.1 Vertical Drainage by Tubewell

7.7.1.1 Advantages of Tubewell Drainage

- An undulating topography of land with depressions may not have a natural outlet, and therefore, cannot be drained naturally unless it is supported with a network of interconnected open or pipe drains. Although the water collected in depressions can also be pumped to safely drain the area, yet tubewell drainage can be an alternate solution in such difficult situations.
- Vertical drainage through dug wells or tubewells facilitates lowering of watertable to deeper depth, which cannot be achieved through pipe drainage or surface drainage systems.
- In case, the quality of water at deeper depth is better, it can be utilized for irrigation only through vertical drainage supported with pumping system.
- Tubewell drainage is preferable option where open drainage is interfering with the agricultural operations.
- Maintenance cost of tubewell drainage is cheaper than open as well as tile drainage.
- Skimming well or scavenger well technologies can be used to lower the watertable as well as to utilize good quality groundwater for irrigation, which is not possible with open or pipe drainage systems.

7.7.1.2 Disadvantages of Tubewell Drainage

- Vertical drainage achieved by installing tubewells or turbines, is more complex and costlier than open or pipe drainage systems.

- The energy required to operate the pumping system may not be available at the drainage site. In case it is available, it would add to the operational cost of the drainage system.
- The success of the well drainage is subject to the favorable characteristics of the aquifer such as hydraulic conductivity and depth of aquifer.
- Tubewell drainage may not be technically and economically feasible where artesian pressure in the aquifer is too high (Boehmer and Boonstra, 1994).
- The groundwater salinity generally increases with the depth of aquifer as in case of the Indus Basin aquifer. Therefore, if the sanity of pumped water is considerably higher, it would be difficult rather undesirable to use it for irrigation.
- Legal restrictions may sometimes not allow to install a tubewell in waterlogged areas, and therefore, tubewell drainage cannot be used in these areas to drain the drainahe surplus (Boehmer and Boonstra, 1994).
- Installation of tubewells along the irrigation canals as in case of SCARP projects in Pakistan, it may interfere with the channel flow and with the desirable recharge of good quality water for the adjoining skimming wells.

7.7.2. Horizontal Drainage

The horizontal drainage refers to the removal of excess water from waterlogged area in horizontal direction such, which may be accomplished by Open Ditch Drains or Surface Drains, Tile Drains, Pipe Drains and Mole Drains, which are also called Subsurface Drains.

7.7.2.1. Open Ditch Drainage (Surface Drainage)

Surface drainage is the oldest system installed in the Indus Basin. Its primary purpose is to dispose off the excess surface runoff resulting from rain storms. However, it may also partially drain groundwater if depth is sufficient to intersect watertable. So, far more than 10750 km of surface drains have been constructed in the Indus Basin. The examples include Samundari drain, Madhuwana, Kharianwala and Paharang drain etc. of Fourth Drainage Project Faisalabad (Fig.7.2 and Fig.7.3).

Advantages of Surface Drainage

1) Surface drains can serve to receive both groundwater and surface runoff. In Pakistan, they are specifically suitable for removing excess runoff from monsoon rains as shown in Fig. 5.24.
2) Surface drains serve as outlet and for disposal of drainage effluent from pipe drainage systems (Fig.7.4).
3) The gradient required for water transport in ditches is much less than in pipe drains, being approximately 0.01% in ditches and 0.1% in pipe drains.
4) They enable easy construction, inspection and maintenance (Fig. 7.5).
5) Initial cost is lower than pipe drainage system.

Fig. 7.2 Surface Drain at Khan Garh, Distt. Muzaffer Gardh, Pakistan

Fig. 7.3 A Well Maintained Surface Drain at FDP Faisalabad, Pakistan

Fig.7.4 Pipe Drainage Supported with Surface Drain as Outlet at FDP, Faialabad

Fig. 7.5 Cleaning and Maintenance of Surface Drain at FDP, Faisalabad

Source: FDP Faisalabad

Disadvantages

- Surface drains cause greater loss of land as compared to vertical or pipe drainage systems.
- Surface drains usually experience excessive weed growth and bank erosion both causing frequent maintenance as shown in Fig.7.6 a, b and c.
- Maintenance cost is higher than the pipe drainage system.
- The land is split up into separate parcels which may considerably hamper efficient farming particularly if ditch spacing is narrow.
- Bank erosion and washing down of excavated material on the banks back into the drain is a continuous source of sediments thereby causing heavy cleaning and maintenance.
- Open drains are exposed to human interference and attract undesirable foreign materials which increase maintenance cost.
- The flow velocities are not sufficient to carry the sediments and other materials which tend to settle in the drains.
- Seepage from unlined surface drains may cause groundwater pollution.
- Poorly maintained surface drains usually cause environmental and health hazards.

Fig.7.6a Bank erosion and vegetative growth in open drain at FDP Faisalabad

Fig.7.6b Bank Erosion in Surface Drain at FDP Faisalabad, Pakistan

Fig.7.6c Unmaintained Open Drain at FDP, Faisalabad, Pakistan

Location: The open drains should be located in the lowest parts of the drainage area to facilitate drainage with a minimum excavation. Further such ditches will serve as an efficient outlet for surface water, which tends to accumulate in depressions.

Maintenance: Unless properly maintained, ditches will rapidly lose their effectiveness owing to vegetative growth and accumulation of sediments. Maintenance practices include;

- Control over vegetation.
- Removal of sediments and deposited materials from the drain and bringing it to the design specifications.

Fig. 7.7 Bank Erosion at Kot Adhu Surface Drain, Khan Garh, Distt. Muzaffer Garh, Pakistan

7.7.2.2. Mole Drains

Mole drains are unlined underground channel, formed by a mole plough, without digging trenches. Its installation cost is lower than that of pipe drainage. Mole drainage is particularly appropriate in dense clay soils, which have a certain general slope. Its primary aim is not to control the groundwater table, but to remove excess water from the field surface or from the top soil where it may constitute a ‘perched watertable’ Fig.7.8.

Mole channels are susceptible to deterioration. Their rate of deterioration, and consequently their effective lifetime is governed by a number of factors, the most important ones are:

- Soil properties
- Topography
- Moisture conditions during construction

- Implements and method of construction
- Flow velocities in channels
- Maintenance

Soil Conditions: Soil should have a certain 'plasticity' to allow the mole channels to be shaped. It should also be stable enough to ensure longer effective life.

Topography: Mole drains should have a continuous slope in the direction of the outlet. Since most of the machines can only draw the channel parallel to the land surfaces, the land should also have a certain general slope. Flat land and land with irregular topography are less suitable.

Maintenance: The mole channels gradually lose their effectiveness. Their working life time, is fair to good conditions, being of the order of 5 to 12 years. The only effective method of repairing is to reconstruct the channels. High velocities tend to cause scoring and erosion. Therefore design of mole drains should consider threshold velocities.

To determine the necessity of re-modeling, drainage conditions and drain outflows should be observed during wet periods.

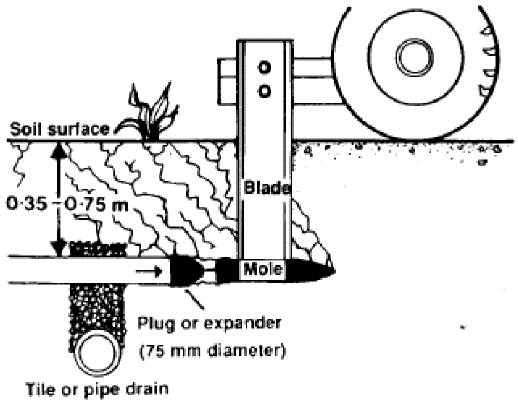

Fig. 7.8 An Example of Mole Drainage

7.7.2.3. Tile or Pipe Drainage

The pipe drainage is the latest system for controlling the high watertable. So far, more than 10 such projects have been implemented in Pakistan which include East Khairpur, Mardan, SCARP, Foruth drainage and Khoshab etc. Pipe drainage system can effectively control watertable at specified depth. The drainage effluent is disposed through sumps and open drain to the desired outlet. The older version of pipe drains was tile drains, which were usually made of fired clay and concrete pipes. The drainage effluent was allowed to enter the drains either through slotted areas or through permeable junctions (Fig. 7.9).

The following guidelines must be considered while designing a pipe drainage system.

- Spacing and depth of laterals.
- Diameters and gradients of laterals and collector pipes.
- Layout of laterals and collectors must be adapted to the topographical feature of the area and to the outlet conditions.
- Design criteria (discharge and depth to watertable) must be determined carefully.
- Soil characteristics such as texture permeability, stability against erosion must be evaluated accurately.

Fig. 7.9 Laying of Pipe Drainage System

Spacing and Depth of Laterals: Deeper the drains allow greater spacing. However, there are several restrictions to the depth of drains installation which include:

- The water level that needs to be maintained in the collector ditch.
- The occurrence of less permeable layer.
- The depth that can be reached by available drainage machinery.

Diameter and Gradient: The following factors are considered regarding the pipe diameter and gradient.

- Area to be drained by the pipe line.
- Design discharge.

- Length of lateral or collector to be used.
- Material and hydraulic characteristics of pipes.
- Flow in a drain pipeline increases in the direction of flow as the drain takes up water over its entire length.
- A safety factor to allow for some decrease in capacity due to certain degree of sedimentation.

Problems of Pipe Drainage System

A number of problems associated with pipe drainage systems include:

- Sink holes may develop in back filled drains when subjected to rainfall or irrigation application in the fieldas shown in Fig.7.10.
- Blocking and clogging of pipe drain openings with unstable soil particles thereby reducing the permeability and drainage efficiency.
- Deposition of sediments in the pipes, particularly at sink holes as shown in Fig.7.11 and 7.12. The sink holes develop in back fill materials and cause failure of pipe drains.
- Bulging and deshaping of pipe sections by overlying earth loads. Such tendency is reduced by using the corrugated pipes.

Fig.7.10 Sink Hole in Back Filled Material at FDP, Faisalabad

Source: USBR (1989)

Fig. 7.11 Sediments Deposition in PVC Pipe Through sinkhole (FDP, Faisalabad)

Source: USBR (1989)

Fig. 7.12 Sediment Settling in the pipe drains at FDP, Faisalabad

Source: USBR (1989)

Types of Drain Pipes

Originally, the underground drains consisted of no more than a trench filled with stores or bush wood and backfield with soil. Since the beginning of 20^{th} century, regular pipes have been developed which have been improved over time as listed below.

i) **Clay Pipes:** Generally termed as clay tiles. They were made from clay materials in standard size of 5 to 20 cm diameter and 30 cm long sections and backed to impart strength. The clay tiles were quite resistant to deterioration under soil conditions. Tile sections were made with or without collars.

ii) **Concrete Pipes:** Concrete pipes (blind and perforated) were made in 30 cm long sections with diameter varying from 15-20 cm. Water could enter through joints or perforations. They were stronger than clay tiles. However, concrete pipes may deteriorate under acidic or salty soil conditions.

iii) **Plastic Pipes:** Plastic pipes have been introduced since 1960. They are made from polyethylene (PE) or poly vinyl chloride (PVC) materials. It is quite durable under all types of soil conditions and can take overlying loads. However, they may deteriorate and become brittle when exposed to sun radiation and extreme cold or high temperatures. These pipes may be used as blind or perforated.

iv) **Corrugated Plastic Pipes:** Corrugated plastic pipes are provided with grooves and bulging on the surface. Perforations of standard size are made in the grooves which improves the chance of water entry into the pipe. Corrugated surface provides extra strength to the pipe against loads but tends to increase the diameter of pipe by 20% for the same flow in smooth pipes. An important advantage with corrugated pipe is the ease of laying with machinery such as trencher. Standard size of 10 to 37 cm diameter are available in the market. The Fourth Drainage Project near Faisalabad is one of the examples where corrugated and perforated plastic pipes have been used to drain about 75000 acres of land to a minimum depth of 4 feet. Laying of Corrogated pipe is shown in Fig. 7.11.

7.8. Pipe Envelope Materials

It is the material placed around the clay, plastic or corrugated pipes to enhance the drainage efficiency of the pipe drainage system. Major functions of envelope material include improving hydraulic and filter conditions around the drain pipe. Envelope includes the following permeable materials:

- Organic materials such as coconut fibre.
- Granular Envelope Materials: These include natural gravel, river run sand and crushed rock materials.
- Synthetic Envelope Materials, which include:

1. Voluminous or felt like materials having a thickness of 3 to 10 mm mostly used in Europe.
2. Thin fibers or sheet like materials having thickness of 1mm and mostly used in UAS and Canada.

As the concrete and clay tiles have been replaced by corrugated and PVC plastic pipes, geo-synthetic / synthetic fabric envelopes are being used worldwide because the fabrics can be easily wrapped around perforated plastic pipes during manufacturing. The synthetic materials may be chemical based such as polyester, polypropylene, polystyrene nylon and fiber glass etc. These may be woven, knitted or needle punched.

The granular envelopes are characterized by their particle size distribution while synthetic envelopes are characterized by their pore size distribution. IWASRI (1998) tested the following materials as pipe envelopes:

- Polyfelt (Austria)
- Texel and Big-O-Sock (Canada)
- Olympia (Pakistan)
- Crushed Rock and Sand
- River Run Sand

These studies concluded that:

1. Geo-synthetic envelopes controlled watertable and retained sediments successfully without clogging of drains. However, with the passage of time, the soil particles accumulating on the surface may reduce the permeability.
2. All types of gravel envelopes also performed good with and without sand mixing.

7.8.1. Functions of Envelope Materials:

The envelope materials are used around pipe drains to perform the following functions (IWASRI, 1998):

- Filtring function to protect drain pipes against soil particles invasion. This would permit only specified sizes of soil particles to filter though the drain pipes and detaining the larger sizes outside the drain.
- Hydraulic function to facilitate water entry into the drain pipe by creating a more permeable zone around the pipe.
- To provide bedding for the drain pipe.
- To stabilize the trench and drain line.
- To maintain the pipe elevation at predetermined elevation.
- To protect the plastic pipe drain from crushing against trench collapsing.

In cohesive soils, the need for envelope material depends on the soil structural stability, which may be related to soil texture, chemical composition, ability to withstand lateral pressure, bulk density and degree of cohesiveness. Consequently, for drain pipes installed in sandy or non-cohesive soils, envelope materials are indispensable (Dierickx, 1991; Wllardson, 1974).

7.9. Drainage in Pakistan

A dependable drainage system is complementary to irrigation system for successful agriculture. However, in Pakistan there was no effective drainage at the time of developing irrigation system. As a result of extensive developments of irrigation schemes such as unlined storage reservoirs, link canals, irrigation channels, a network of watercourses and year round water application of water to irrigated crops, uncontrolled seepage of water occurred to the groundwater that caused consistent rise in water table in the Indus Basin. As a result of this, the country had to face severe waterlogging and salinity problems during late 1950’s causing a lot of productive land to lose its potential productive capability. Consequently, drainage of agricultural lands became essential. Thus, efforts were initiated to control waterlogging and salinity by creating the Water and Power Development Authority (WAPDA) in 1958 and implementing the Salinity Control and Reclamation Projects with the technical input from USA. Since then, a large number of surface and subsurface drainage projects were constructed using a variety technologies such as tubewells, open drains and tile drains individually or in combination as given in Table 7.1.

Table: 7.1 Drainage Projects Constructed in Pakistan

Province	No. of Drainage Projects	Area Served (ma)	Number of Tubewells Installed	Surface Drains (km)	Tile Drains (km)
Punjab	30	10.06	10050	3243	2703
Sindh	18	5306	4555	7178	976
KPK	8	0.603	491	773	5781
Balochistan	2	0.177	--	322	-
Total	58	16.45	15096	11525	9520

Source: Alam (2000)

The drainage projects were mostly constructed by WAPDA and subsequently handed over to the Provincial Irrigation Departments (PID’s) for operation and maintenance. Since the emergence of water logging and salinity problems in the Indus Basin, major thrust of Government’s investments has been on controlling excess water and salts by installing tubewells to lower down the watertable and surface drainage and to dispose off the effluent.

Since 1980's, subsurface pipe drainage systems in conjunction with remodeled surface drainage systems were installed in Khairpur, Mardan, Faisalabad and Khoshab. Recently, the farmers' participation has been encouraged by phasing out SCARP tubewells through SCARP Transition Programs and replacement of deep SCARP tubewells with shallow Community tubewells. Major drainage projects executed since 1960 included:

1) Salinity Control and Reclamation Projects (SCARPs)
2) Pipe Drainage Systems
3) National Drainage Program (NDP)
4) Left Bank Outfall Drain (LBOD)
5) Right Bank Outfall Drain (RBOD)

7.10. Disposal of Drainage Effluent

There are a limited number of options available to dispose off the agricultural drainage water into the existing natural hydrological system. According to Boehmer et al (1994) the common options include:

- To return the surplus water to the land as part of the irrigation water supply either independently or after mixing with canal water.
- To dispose to canals and rivers and finally to the ocean.
- Deliver to lakes or to salt sinks where the water would evaporate leaving the salts in the tank.

Use of saline drainage surplus for irrigation or delivering to the irrigation canals or rivers may become problematic as it may turn a good fertile soil to a saline soil or even make it non productive. The salts infiltrating into the soils may deteriorate the groundwater quality. The drainage effluent of SCARP tubewells in Pakistan have high salt contents, has been either returned to the canals running beside the tubewells thereby making a part of the existing irrigation system, or is being mix with canal water and applied directly to the irrigated fields. This practice of disposing drainage effluent becomes problematic not only for irrigated land but also for the downstream users for domestic, agricultural, industrial and livestock purposes.

In another case, the effluent of Left Outfall Drain and Right Outfall Drain, is being disposed to the Arabian Sea. However, the options available to any single situation may be limited because of water quality concerns. Drainage water quality may vary within a catchment, pumping depth and type of drainage water, particularly, where agricultural drains are used for the disposal of domestic and industrial waste and other non-agricultural sources, which may be highly polluted because of chemical and solid waste materials. Various alternatives of safe disposal of drainage surplus may include evaporation ponds, solar evaporators, salt harvesting sites, water treatment plants or disposal to the sea. However, the quality of agricultural drainage water would decide its potential for reuse.

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Chapter 8

Surveying and Land Leveling

8.1. Surveying

Surveying is an engineering science and art by which lines, distances, angles and elevations are established and measured on the earth surface. From these measurements, locations, directions, areas and volumes are determined. Surveying information, recorded in the field, can be represented graphically by diagrams, maps, profiles and cross-sections. In practice surveying is the art of determining the relative height or elevation of points or objects on the earth surface. For this purpose, engineer's level and staff rod are used to accomplish the job. Surveying data are used to determine the vertical and spacial position of natural and man-made features on the surface of earth.

8.1.1. Classification of Surveys

Classification of surveys can be made on the basis of the nature of field surveys, objectives, methods and instruments used for surveying. The detailed classification is out of scope of this book. However, for the purpose of irrigation and drainage practices, the is broadly classified as topographical and engineering surveys, which may be further classified as Profile Survey, differential surveying, Bench Mark Survey, Check Surveying, Reciprocal Surveying, Reconnaissance Survey, Preliminary Survey, Final Survey and Location Survey as explained below.

8.1.1.1 Topographical Survey

This survey is for determining the natural features of land such as hills, valleys, rivers, lakes, buildings, towns, villages, fields and vegetation etc. It also takes into account the degree and length of slope, shape and erosion hazards of the area.

8.1.1.2 Engineering Surveys

This is for determining quantities and collecting data for the design of engineering works such as roads, railways, reservoirs and works in connection with supply and sewerage etc. Engineering surveys include Differential Survey, Bench Mark Survey, Profile Survey, Reconnaissance, Preliminary and Location surveys as defined below.

8.1.1.3 Profile Survey

It is the type of survey in which elevation of the surfaces at series of points at measured intervals along a line such as watercourse, drainage ditch, terrace, waterway, road or any other purpose, are measured to consider change in elevation along a path. Profile survey of a watercourse may be conducted and recorded along the bed or water surface, and accordingly, may be termed as bed profile and water surface profile of a watercourse.

8.1.1.4 Differential Surveying

It is the process of finding the differences in elevation between two or several points. It may require a number of settings of Engineer’s level / Surveying instrument along general line between two points. Each setup requires a rod reading on a point of known or unknown elevation until closure of survey. The elevation of a given point is determined with reference to a Bench Mark (BM) or any other selected datum, which may be of temporary or permanent nature such as top of a culvert or mean sea level.

8.1.1.5 Bench Mark Survey

This survey is carried out to provide a widely spaced series of points of known elevation that may act as reference points during the detailed survey. These may include bench marks (temporary or permanent), head gates, culverts or bridges etc. This provides a skeleton for topographic survey that may be conducted later. The bench mark survey when completed to the required standard of accuracy, becomes starting points for any subsequent profile survey of watercourse or topographic survey of the command area. Use of portable bench marks or turning points may avoid any errors of rod reading that may occur by wrong placement or rotating of staff rod on the marked ground surface.

8.1.1.6 Check Surveying

It is the operation of running levels for the purpose of checking a series of elevations, which have been previously fixed. This kind of survey helps to identify any error that may have occurred during the survey.

8.1.1.7 Reciprocal Surveying

It is the methods of surveying in which the difference in elevation between two points is accurately determined by two sets of observation when it is not possible to set up the level midway between the two points.

8.1.1.8 Reconnaissance Survey

This type of survey is carried out for determining the rough cost of the scheme. Information collected through reconnaissance type field investigation is broad and general. It provides rough outline of possible solutions and delineates project area and its subdivisions.

8.1.1.9 Preliminary Survey

The purpose of this survey is to collect more precise data, to choose the best location for the work and to estimate the exact quantities and costs. These surveys are carried out to demonstrate convincingly that the project plans are technically sound and economically feasible.

8.1.1.10 Final Survey

Final survey of the area is carried out to determine the exact specifications and detailed data collection for developing design specifications as well as final cost assessment of the project.

8.1.1.11 Location Survey

It is meant for setting out the planned work on the ground. It includes information collected through detailed type of field investigations. It provides elaborated plans comprising design, drawings and specifications to serve as working document for implementation.

8.1.2. Surveying Equipment

The principle surveying instrument and their accessories are as follows (Ministry of Food, Agriculture and Cooperatives, 1980).

(i) Range Pole

Range pole is one-piece rod about 2 min length, painted red and white. It is used generally to establish a 'line of sight'. One edge of ranging pole is wedge shaped to facilitate installation on ground surface.

(ii) Level Rod or Staff Rod

The leveling rod is used for the purpose of measuring vertical distances. It is graduated in meters and centimeters with alternate black and white subdivisions each indicating 1 cm. Thus, its accuracy is 0.01 as shown in Fig.8.1. It is used to take elevation shots when working with engineer's level for surveying or land leveling. The leveling rod may also be graduated in feet, 1/10 ft and 1/100 ft divisions.

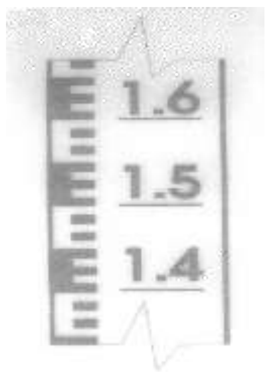

Fig. 8.1 A Section of Metric Leveling Rod

(iii) Engineer's Level (Dumpy Level)

This instrument is used primarily for measuring angles, horizontal and vertical distances, prolonging or setting points in lines and for leveling operations. The Engineer's Level along with its labeled components is shown in Fig.8.2.

Fig. 8.2 Engineering Level with Components

(iv) Tripod/Stand

It is a 3-leg stand, which is adjustable to change the height of installation as well as height of line of sight. The level is installed on the top of tripod with the help of screws concentric leveling bubble provided for the purpose.

(v) Field Book

Engineering field book is used for the recording of survey notes, layout and construction data. These are valuable documents because of the time and expense involved in obtaining such data. The stations and distances on the field book are recorded as 00+30, 1+00 and 1+30 etc. The first digits left of + are whole numbers as multiple of 100. The distance less than 100 is shown on the right of + sign. Thus, 1+30 means station at 130 m distance.

8.1.3. Use and Adjustment of Engineer's Level

Engineering surveys with an acceptable degree of accuracy cannot be made unless the instrument is maintained in proper adjustment. The adjustments generally encountered in the field include adjustment for line of sight, bubble and cross hair.

8.1.4. Adjustment for line of Sight

To make the line of sight parallel to the axis of the level, perform the direct or two peg test. Set two pegs or stakes (A & B) about 120 m apart. Set up and level the instrument midway between the two stakes at point C. Take rod reading on each stake A and B. The difference in reading will be the difference of elevation between them. Next move the instrument and set up so that when the rod is at A, the eye piece will not be over three meters on opposite side of rod at B. This will be point D. Read and record reading at A. Then to the eading at A, add or subtract (depending upon whether B is lower or higher than A) the true difference of elevation to find estimated reading at B. Sight on the rod at B. If the estimated reading of B is same as obtained by sighting, the instrument is in perfect adjustment. If it is out of adjustment, move the horizontal cross-hair until the line of sight intercepts the true reading on rod at B.

A	C	B	D (Eye Piece)
*	*	*	*

Fig. Instrument / Staff Rod Locations for Adjusting Line of Sight

8.1.5. Adjustment of Bubble (Circular Vile)

To check the bubble, set up tripod and instrument, level the instrument. Rotate the instrument 180 degrees, bubble should remain in circle. If any part of the bubble is out of circle, bring bubble to center with leveling screw. Repeat the above operation until bubble remains in center in any position of the instrument to the horizontal plane as shown in Fig.8.3. In a 3-screw arrangement, the bubble may be efficiently brought in the center by rotating two screws across the level (both inward or outward) while rotating the third one in any direction to move the bubble towards center of circle.

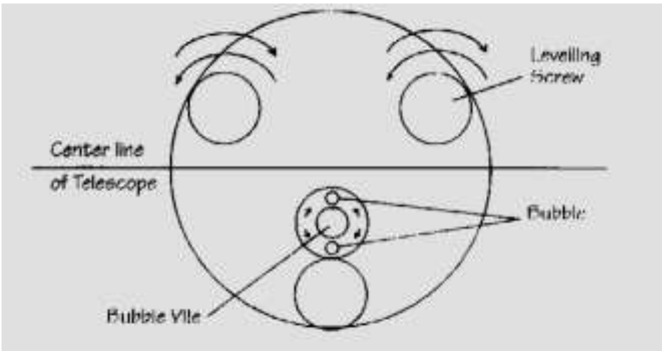

Fig.8.3 Adjustment of Bubble

8.1.6. Measurement of Distances

The measurement of distance is probably one of the most frequently used procedures in surveying. There are two main methods of determining distance:

(i) Direct method in which the distance is measured by chain, tape or other instruments.

(ii) Computative method in which the distance is measured by calculation such as in tachometry, telemetry, triangulation or trigonometry.

8.1.6.1 Direct Methods

Distances are actually measured on the ground by means of manual paces, chains, tapes or other instruments as summarized below:

Pacing: When approximate results are required, distance may be measured by pacing. Distance measurements for application in agriculture, engineering, geology, forestry and military field sketching, are frequently made by pacing. The method consists of walking over a line and counting the number of paces. Knowing the average length of one pace (approx. 2 ½ ft.), the required distance may be obtained by multiplying the number of paces with the average length of individual's pace. The length of pace varies with the individuals. Instruments used for Pacing include Passometer, Pedometer, Odometer and Speedometer discussed below.

Passometer

The problem of counting paces may be overcome by using an instrument called a passometer. It is a pocket instrument like a watch that automatically records the number of paces taken in pacing a given distance.

Pedometer

It is an instrument used for the same purpose as passometer but it registers the distance traversed by the person carrying it.

Odometer

Distance traversed by the vehicles is measured by means of a simple device called odometer. It can be attached to the wheel of any vehicle such as carriage, cart, bicycle etc. and registers the no. of revolutions of the wheel. The distance may be estimated by using Equation 28.

$$\text{Distance traversed} = 2\pi r \times n \tag{28}$$

Where

r = radius of the wheel

n = No. of revolutions

Speedometer

The speedometer of an automobile may be used to measure distance traversed by the vehicle. It gives more accurate results than pacing and odometer provided the route is smooth as along a high way. The meters installed in the vehicle, which indicate the total distance traveled and the speed of vehicle, are called speedometers.

Chaining

Chaining is the most accurate and common method of measuring distances. For work of ordinary precision, a chain is used, but where great accuracy is required, a steel tape is used. Main types of chains are discussed below:

a) Gunter's Chain

It is 66 feet long and divided into 100 links. It is mainly used for land surveying. It is very convenient for measuring the distance in miles and for measuring land when the unit of area is an acre.

80 Gunter's chains = 1 mile

10 square gunter's chains = 1 acre

b) Engineer's Chain

It is 100 feet long and divided into 100 links, each one foot in length. It may be used when chaining along steep slopes.

c) Meter Chain

Chains of 10, 20 and 25 meters lengths are commonly used. Twenty m chain is divided into 100 links, each being 2 decimeter in length and the brass tags are attached at every 2 meter from each end.

(f) Tapes

Tapes are made of various materials and are therefore, divided into four classes:

1) Cloth or linen tape
2) Metallic tape
3) Steel tape
4) Invar tape

Cloth or Linen Tape

Linen tapes are available in 33, 50, 66 and 100 feet length. These consist of varnished strips of woven linen ½ to ¾ inch wide and winds in a well sewn leather case with either folding or flash handle.

Metallic Tape

It is the cloth tape having very fine brass or copper wires woven into it, to prevent stretching and twisting. It is better than the linen tape, but it is more suitable for accurate and important measurements.

Steel Tape

For accurate measurements, the steel tape should be invariably used. It is made of steel ribbon varying in width from ½ to 3/8 inch. It is available in lengths of 50 to 100 ft, but most common lengths are 50, 100 and 300 ft.

8.1.7. Measurement of Areas

Measurement of distances are frequently utilized to compute the areas of farms or fields using mathematical equations in accordance with the shape of the area. The area of different shapes or fields can be computed by following formulae:

For Rectangle, $A = L \times W$ (8.1)

For Triangle, $A = ½ (b \text{ x } h)$ (8.2)

For Trapezoid, $A = \frac{W_1+W_2}{2} \times L$ (8.3)

For Circle, $A = л\ r2$ (8.4)

Where:

A= area

L= length

W = width

W1, W2 = lengths of top and bottom of trapezoid

b = width of the base of triangle

h = height of triangle

r = radius of Circle

Planimeter method may be used to compute areas of different shapes, which consists of determining the area of a given figure by use of planimeter. The planiemter is most useful in determining the areas of figures plotted to scale, especially when the boundaries are irregular or curved such as river boundaries and watercourse commands etc.

8.2. Leveling

8.2.1. Land Leveling and Land Planning

This is reshaping the surface of the land to be irrigated in which cutting and filling an area or field is planned and done to achieve a desired grade to improve management and control of irrigation water or precipitation. Land planning is a complimentary operation to land leveling. It is an operation to smoothen out minor irregularities (ups and downs) in the soil surface caused by tillage operations.

8.2.2. Benefits of Land Leveling

The benefits of precision land leveling include:

1) Saving of water by reducing time to irrigate.
2) Reduction of plowing, planting and pumping energy requirements.
3) Uniform application of fertilizer with irrigation water.
4) Better and more uniform plant population.
5) Production of high yields .
6) More efficient cropping and mechanized farming operations.
7) Improves surface drainage during rainy season.
8) Saves labor.
9) Improves irrigation application efficiency.
10) Reduces waterlogging and salinity problems.
11) Economically feasible farming operations.

8.2.3. Disadvantages of land leveling

No doubt the advantage of land leveling have established its need in improving agricultural production, major disadvantage and problems associated with land leveling may includes.

- Deep cuts can expose infertile subsoil, which may decrease crop yield till fertility is restored.
- The cost of land leveling and its sustained maintenance may not justify potential returns. To maintain benefits on sustained basis, leveling of agricultural land needs to be done after every 2 to 5 years.
- Topographic breaks in slope may not justify leveling of longer borders economically because of heavy earth movement.
- Land leveling operations may disturb cropping schemes of farms temporarily.

8.2.4. Equipment for land leveling

The basic equipment for precision land leveling include:

8.2.4.1. Surveying Equipment

The equipment, required for surveying include dumpy level with tripod, staff rod, range poles, stakes, measuring tape, flags and field book.

8.2.4.2. Earth moving equipment

These include those equipment, which help to loosen and to shift the soil from high spots to low spots of the area. These include Subsoiler, Chisel plow, Ditcher, Ridger, scraper and Planer etc.

8.2.5. Terminology used in land leveling

Elevation (E)

A measurement of vertical distance to a point above or below a fixed datum, which may be assumed or may be related arbitrarily to the mean sea level (MSL). It is computed by subtracting the foresight from the height of instrument as shown in equation (8.1).

$$E = HI - SF \qquad (8.5)$$

Where

E = Elevation of the point

HI = Height or Elevation of instrument above Datum

FS = Foresight

Height of instrument (HI)

It is the elevation of the line of sight above given datum when the instrument is leveled. It is also regarded as the height of the horizontal line passing through the center of cross hair of the object and eye pieces (telescope) above the datum. It is determined by adding the backsight to the known or assumed elevation of a point such as Bench Mark (BM).

$$\mathrm{HI} = \mathrm{E} + \mathrm{BS} \tag{8.6}$$

Where:

BS = Back Sight

E = Known or assumed elevation of turning point or bench mark

Foresight (FS)

Rod reading taken at the point of unknown elevation and it is always subtracted from the height of instrument to determine elevation of the point where rod reading has been taken. The foresight may be regarded as a minus sight in the process of determining the elevation of a given point.

Backsight (BS)

Rod reading taken at the point of known elevation and is always added to the elevation of the point to get the height of instrument. It is usually a rod reading taken at bench mark or turning point of known elevation i.e. it is the vertical distance between the bench mark elevation and the line of sight of the instrument. the back sight may be regarded as plus sight in the process of determining the elevation of a point in differential leveling.

Datum Surface (DS)

It is any known or arbitrarily assumed level surface or line from which vertical distances are measured. Thus, the datum is initial point for all elevations determined in a survey i.e. E= 0.0. It is used as reference point in obtaining the elevations of subsequent stations. It provides opportunity to compare the elevation of various points with respect to a common reference. Mean sea level is generally considered as a natural datum surface.

Grid point (GP)

A point on the grid or field area marked by stakes or by other means. Generally, grid points are located on 20 x 20 meter intervals or otherwise considered such as 15x15 or 10x10 m depending on the size of area and accuracy desired. Smaller the size of grid area, greater is the accuracy. Grid area can be regarded as the size of the standard or smallest unit considered while sub dividing the given area for the purpose of surveying.

Bench Mark (BM)

A bench mark may be permanent or temporary. It is natural or artificial object bearing a marked point whose elevation is assumed or known. It is used as a common reference for comparing elevation of various points and as a reference for initiating subsequent surveys of the area.

Temporary Bench Mark (TBM)

Point conveniently established on the ground to act as a bench mark. It is used to determine the elevation of subsequent stations when bench mark is not accessible

Turning point (TP)

A temporary bench mark or point where an elevation is established in order to change the location of Engineer's level and to continue survey. Sometimes, portable turning points are provided for field surveys at a given turning point. A foresight at turning point is taken before shifting the instrument to new position and a back sight is recorded at the same turning point after shifting the instrument. Thus, at each turning point two readings are taken, one foresight and one back sight.

Intermediate Sight (IS)

Staff reading taken on a point of unknown elevation from the same set up of the level. All sights taken between the back-sight and the foresight are intermediate sights. While determining the elevation of a given point. It is subtracted from the HI.

Line of Collimation (LC)

It is the line joining the intersection of the cross-hairs to the optical center of the object glass and its continuation.

Station (S)

A point or position on a measured line which represents a measured distance.

Full Station

A full station is 0 + 00, 1+00, 2+00 etc. Stakes set at any other point between the full stations are called plus station and are designated as 1+25, 1+50 etc. Thus, the distance along the line indicates full station and plus station. For example, a distance of 150 m is written as 1+50m i.e. The digit to the left of the + sign shows the distance in multiple of 100 and that to the right indicates less then 100m.

Closing of Surveys

To verify the accuracy of survey, a return check must always be made. This is achieved by continuing the line of levels from the last station back over a slightly different but shortest route to the initial starting point i.e. the first bench mark. In closing of survey, the last reading taken at the Bench Mark is fore sight, which is subtracted from the HI of last setting of instrument to determine the closing or new elevation of BM. Comparison of initial (known) elevation of BM to its closing elevation, determines the error of closure.

Error of closure

If there is no error in a closed survey, then the elevations determined for the first bench mark by return check would be the same the original elevation of that bench mark. The error of closure is the amount by which the original BM elevation differs with the elevation of BM determined at the return check. Allowable error of closure for a survey is a function of the accuracy of the instrument and the length of survey or the number of times the instrument is set up. The allowable error of closure equals 0.01 m per two instrument settings. Error of closure may be obtained by subtracting the sum of foresights from the sum of back sights. However, for profile survey the foresights and back sights used for computing the error of closure include only those taken at TP or BM. The intermediate sights are not used in such computations.

8.2.6. Examples for determination of elevation

The following examples demonstrate the procedure of determining the elevations of given points during an engineering survey.

Example 8.1

The assumed elevation of a bench mark (BM) was 10 m. The rod reading at the bench mark was 1.54 m. The foresight reading of staff rod at points A, B and C were 1.75, 1.50 and 1.20 m, respectively. Determine the elevations of point A, B and C (Fig.8.4).

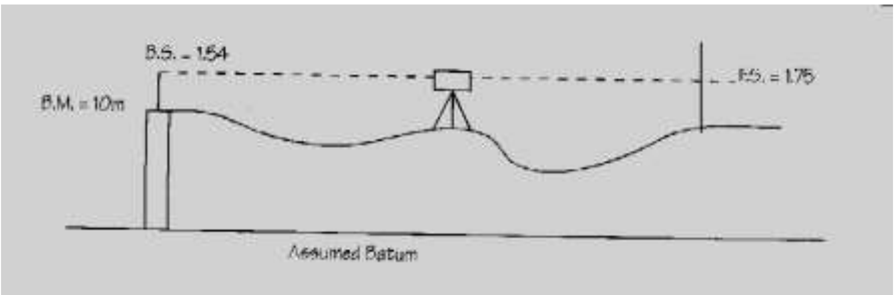

Fig. 8.4 Surveying by Engineer's Level With 10 m Elevation of BM from Assumed Datum

With the given data, the calculation procedure and the elevation of given points thus determined, are given in Table 8.1

Table. 8.1 Determination of elevation from Survey data.

Station	BS (m)	HI (m)	IS & FS (m)	Elevation (m)	Remarks
BM	1.54	11.54		10.00	Elevation above assumed datum
A			1.75	9.79*	EA = HI - FSA 9.79 = 11.54 -1.75
B			1.50	10.04*	EB = HI - FSB 10.04=11.54 -1.50
C			1.20	10.34*	EC = HI - FSC 10.34=11.54 -1.20

Example 8.2

The elevation of bench mark is assumed to be 100 m from datum. The rod reading taken at the bench mark is 2.40 m. The foresight reading at A and B stations are 1.3 and 0.45 m, respectively. The backsight reading at B (a turning point) is 2.20. Find the elevation of point A, B, C and D when foresight readings at C and D are 0.55 m and 1.02 m, respectively (Fig.8.5). Also find the error if foresight at BM while closing the survey is 4.10 m.

Solution

The data given in the example 12 and the elevations of the points along with procedure of determination, are given in Fig. 8.5 and Table 8.2.

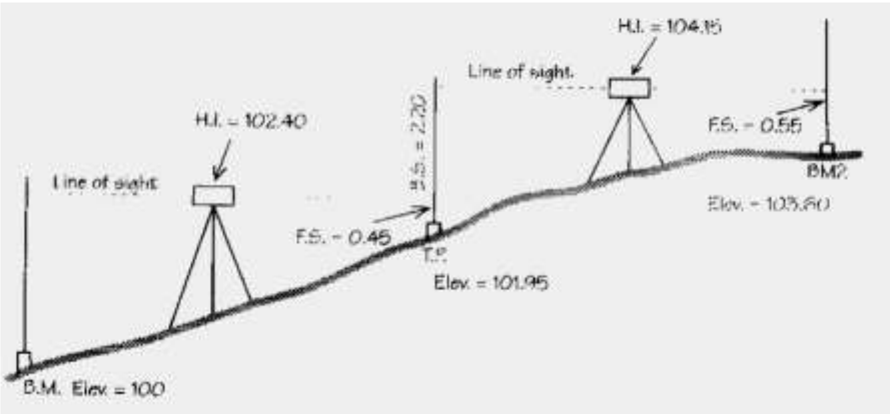

Fig. 8.5 Differential leveling by Engineer's Level

Table 8.2 Determination of Elevation from Survey Data

Station	BS	HI	IS & FS	Elevation	Remarks
BM	2.40	102.40		100.00	Assumed elevation
A			1.30	101.10	EA = HI1 - FSA 101.1 = 102.4 - 1.30
B			0.45	101.95	EB = HI1 - FS B 101.95 = 102.40 -0.45
B (TP)	2.20	104.15			HI2 = EB + BSB 104.05 = 101.95+2.20
C			0.55	103.60	EC = HI2 - FS C 103.6 = 104.15 – 0.55
D			1.02	103.13	ED = HI2 - FS D 103.13 = 104.15 – 1.02
BM (closing)			4.10	100.05	E BM = HI2 - FSBM 100.05 = 104.15 – 4.10

Closing Error = Return E_{BM} – Original E_{BM} (8.7)

Or 100.05 – 100.00 = 0.05m

The error of closure can also be determined as:

$\sum$ BS = 4.6m

$\sum$ FS = 4.55m

Error of closure = $\sum$ BS - $\sum$ FS (8.8)

= 4.60- 4.55 =.05m

8.3. Precision Land Leveling

Precision land leveling consists of grading or smoothing the land surface with little or no slope to precisely determine elevation plan with an accuracy of ±2 cm. Without uniform grade, low areas may reserve receive excess water and cause submergence of crops, excessive leaching of nutrients, delayed workability and wastage of water. High spots on the other hand may not receive sufficient water, accumulate salts, obstruct surface flow of water and reduce crop yield.

A precisely leveled field will improve soil water and crop conditions favorable to increased production. A more uniform crop stand, uniform distribution of water and of fertilizer, saving of irrigation water, labour and other farm operations and above all the improved crop yields are some of the major benefits associated with land leveling. Land graded to appropriate slope overcomes erosion hazards and enjoys good drainage to facilitate farm operations.

Land grading/leveling should never be done without first knowing the soil profile conditions and the maximum cut that can be permissible. Although land leveling is usually done one a field – by field basis, it is extremely important that the entire farm be studied before any grading work is attempted. Even the most economical design for an individual field may be undesirable if the farm system prohibits it. The location and elevation of water delivery, topography of each field and the regular farming operations should be considered by the designer in planning the overall farm development.

A topographic map should be used to plan land grading which may be developed using grid staking procedure by any of the conventional surveying methods. The grid pattern for topographic planning is of greater spacing than used for land leveling work (e.g. 60 to 90m instead of 20 m spacing in grading of individual field).

8.3.1. Land Grading Procedure

- Plan a grid system using an appropriate grid interval
- Stake the planning grid system
- Prepare grid sheet using graph paper

- Conduct survey for precision land leveling using engineer's levelor laser equipment
- Record survey data
- Determine the average rod reading/elevation at each grid
- Make adjustment for shrinkage, borrow of extra fill
- Determine the volumes of cut and fill and prepare the guide map
- Determine the volume of earth work
- Physically move the earth using scraper or laser leveling equipment
- Check the elevation of finished grid points

8.3.2. Planning of Grid System

- A grid system of 20 m interval will be suitable for leveling of most agricultural fields. Under special circumstances such as gardens, a different interval such as 5x5 or 10x10 m may be used
- Locate sharp breaks in natural land slope on the map. Use land owner map or self prepared map to note the irrigation facilities, permanent structures, roads, natural boundaries and electric poles etc
- Clear the vegetation in the field to be leveled to avoid interference and obstruction to survey or field operations
- Area where grids are to be established, should be smoothed

8.3.3. Staking Grid System

The following procedure should be followed for staking a grid system on the field to be leveled (Federal Water Management Cell, 1996).

i) Establish corner of the field where at least two straight edges intersect at approximately right angles and select one side of the field as base line. As shown in Fig.8.6, the eastern and southern sides of the field are straight lines, which intersect at south eastern corner. The grid interval is 10 m x 10 m.

ii) Measure one-half the grid interval (5 m) from the base line and set two ranges poles at widely separated locations, 1 and 2 as shown in Fig. 8.6. On the projection of the line passing through the range poles and one-half grid interval (5 m) from perpendicular edge of the field, set a grid stake which will be numbered as B-1. Set the level over this grid point, and with the horizontal circle set at 0°, orient the instrument by sighting on the most distant range pole. From this grid point B1, measure and sight to set straight line, by the use of level, the remaining grid points (B2, B3, B4 and B5.

iii) Turn the Engineer's level to 90° angle from this point. Measure and locate grid points at grid interval of 10m, by the use of level, the perpendicular grid points (C-1, D-1, E-1 and F-1).

iv) Move the level and set it over grid point C-1. Orient the instrument by aligning on the grid stakes in both directions on the perpendicular line. Turn the instrument to 90° angle from the perpendicular line and sight, measure and locate the grid points (C-2, C-3, C-4 and C-5) parallel to the firstly established points.

v) Move the level to point C-2 and orient by aligning on grid points set in step iv above. Turn 90° and sight and measure the grid points (D-2, E-2 and F2)

vi) Set the remainder of the grid points without the use of level by aligning them by eye and placing the stakes or pole at the intersection of lines and diagonals. Following this procedure, all the grid points can belocated where the staff rod will be placed to take the rod readings during topographic survey.

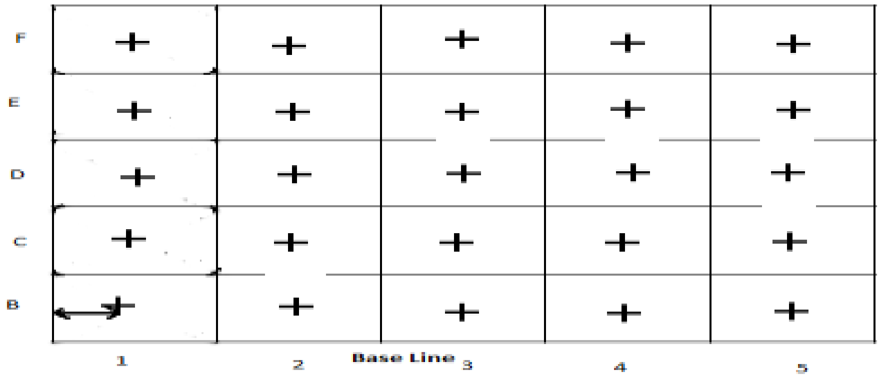

Fig. 8.6 Establishing of Grid Points during Topographic Survey

8.3.4. Preparation of Grid Sheet

- Prepare the grid sheet using graph paper. It should show the location of all grid points and stakes if used. Locate the field boundaries.
- Locate irrigation channels, drainage channels, pump sites, wells, pipelines, farmstead and any other topographic or physical feature of importance.
- Record directional orientation of the map, north should generally by the top of the sheet.

8.3.5. Survey Data for Precision Land Leveling

1) Establish elevation datum. Use known elevation if available, if not, use assumed elevation. Properly identify a Bench Mark or Temporary Bench Mark (BM or TBM) with known or assumed elevation.
2) If field size and topography are such that at all the grid points, rod readings can be taken from a single instrument setup, set the instrument near the center of the field, such as near grid point D-3 in Fig. 8.6.

3) If two or more instrument setups are required, divide the field into as many parts as necessary and locate the instrument near the center of each part.
4) For each setup, establish the height of instrument (HI) to the nearest 0.005m, by back sighting on the benchmark. Add the backsight reading to the benchmark elevation to obtain HI (i.e. H = BS + BM).
5) Subtract all intermediate or foresights from height of instrument for the same setting of instrument to determine the elevation of subsequent grid points i.e. E = HI – FS.
6) Check the the accuracy of the survey by taking a foresight on the bench mark where the survey was initiated. Find the new elevation of BM at closing (i.e. E = HI – BS). Determine the error of closure by subtracting the new elevation of BM from the original elevation of BM (known or assumed) where the survey was initiated.

   Error of Closing = Original elevation at BM – new elevation at BM.

   In case, the error of closing is greater than 0.01m, check or repeat the survey to bring it at required accuracy.

8.3.6. Recording of Survey Data

The survey data should be recorded on the field book showing bench mark, stations, back sights, foresights, intermediate sights, turning points and any other feature of the topography observed during survey. The data comprises elevations and rod readings taken with the engineer's level (Ministry of Food, Agriculture and Cooperatives,1996).

Determining and average rod reading or average elevation

- Determine the average rod reading or the average elevation of the segment to be leveled in a single plane. If there are more than one segments, handle each separately. Follow the following procedure to determine the average rod reading.
- Add columns and rows of rod readings. The summation in both directions should be equal.
- Determine proportionate size of odd grid areas. Each grid point represents a specific area (400 sq. m in this case) known as the standard grid area. Some of the areas may be either larger or smaller than the standard area called odd areas. The average rod reading is determined by dividing the sum of adjusted rod readings by the sum of adjusted grid areas. The average elevation is obtained in the same manner.

8.3.7. Adjustment of Rod Reading or Average Elevation for Shrinkage

Shrinkage of soil varies according to the soil texture, construction equipment used, condition of the field (i.e. plowed or unplowed) and the depth of cut and fill. It is expressed as the Cut:Fill(C:F) ratio. For example, if one cubic meter of excavation makes 0.8 cubic meter of fill, the C:F ratio would be 1: 0.8 or 1.25. Shrinkage means the entire field is lowered by a certain amount by the compaction due to the earth moving equipment during leveling. Adjust the average rod reading or average elevation to allow for soil shrinkage as summarized below:

- Add the amount of shrinkage adjustment to the average rod reading or subtract the amount from the average elevation.
- The shrinkage adjustment should be at least 0.01 m for compact soil but less than 0.03 m for loose soil.
- As a first trial a shrinkage value of 0.01 m should be used.
- After making adjustment for shrinkage, check for C:F ratio and bring it to acceptable range (i.e. 1.10-2.00).

8.3.8. Adjustment for Borrow

For a grid area of 20 m x 20 m, the value of adjustment for borrow in the soil from the field (H_b) to construct farm roads, ditches or dikes is calculated as under.

$$H_b = \frac{(0.0025 \times \text{cubic meter of borrow})}{\text{Adjusted grid area}} \qquad (8.9)$$

The amount of borrow adjustment is added to the average rod reading or subtracted from the average elevation. The factor "0.0025" comes from the ratio of 1: 400 Standard Grid Area i.e. 0.0025 = 1/20×20. In case of different grid area, this factor needs to be adjusted accordingly.

8.3.9. Adjustment for Extra fill

If there is excess soil material such as sand dune, high mount or spoil from a watercourse, it can be used as fill material in a leveling job. The adjustment for extra fill (H_e) for standard grid area of 20 × 20, can be calculated using the equation 8.10.

$$H_e = \frac{(0.0025 \times \text{cubic meter of fill}}{\text{Adjusted grid area}} \qquad (8.10)$$

The amount of extra fill adjustment is subtracted from the average rod reading or added to the average elevation. In case of different grid area, the coefficient needs to be calculated as 1/grid area.

8.4. Laser Land Leveling

8.4.1. Introduction to Laser Leveling

A laser is a device that emits light through a process of optical amplification based on the stimulated emission of electromagnetic radiation. Generally, the laser beam exhibits a high degree of spatial coherence that allows a laser beam to stay narrow over lager distance such as line of collimation in engineering level. It enables laser beam to use in laser leveling and laser pointers.

In surveying, the laser level is a tool that consists of a beam projector fixed on a tripod and projects a red or green beam along the horizontal axis. A rotary level such as the one used in laser surveying, is a more advanced laser level, which spins the beam of light fast enough to give the effect of a complete 360 degree horizontal plane.

Leveling of land using laser technology is referred as Laser Land Leveling. This system increases the accuracy of leveling and reduces the labor required as compared to the leveling with engineer's level and scraper. Laser system is equipped with Scraper, Receiver, Hydraulic cylinder, Pressure hoses, Solenoid, Control box, Transmitter, Tripod, Level eye detector and Tractor with three point hitch and hydraulic lift. It is the most sophisticated equipment available for land leveling. The laser technology is mainly based on a laser plane which is created by emitting a horizontal rotating beam of laser from a transmitter rotating in 360^{o} plane. The laser beam is received and intercepted by laser receiver which is mounted on the scraper. The receiver further sends laser signals to the tractor hydraulic system to operate the rear mounted scrapper through a solenoid system maintaining a pre-determined level of scraper edge and therefore levels the surface of land by executing on cuts and fills guided by an automatic control system to operate the scraper at a predetermined or desired average elevation/ rod reading. Fig. 8.7 demonstrates the principle of leveling by laser system.

8.4.2. Difference Between Traditional and Laser Leveling

Primarily, the land leveling process is the same whether an engineer's level or a laser equipment is used. The ultimate aim in both the technologies is to eliminate or minimize the high and low points (cuts and fills) on the land surface creating a plain topography at a pre-determined slope. However, there are a few differences between laser leveling (with laser system) and the traditional leveling (with engineer's level) technologies as summarized below:

1) Laser leveling (Fig.8.7) is more advanced, sophisticated and accurate technology as compared to the traditional leveling.
2) Laser surveying reduces labor requirements as only one person holding telescopic grade rod (staff rod) can complete the survey. Contrary to this, at least two persons are required (one handles engineer's level while the other holds staff rod) to carry out survey with engineer's level.
3) Laser leveling is much easier than traditional leveling. Laser provides an automatic control on scrapper during leveling process through a control box and therefore manual interference of the operator is not needed unless the scrapper is overloaded or scraper operates too high. In case of traditional leveling, the operator manually controls the scrapper while two persons check the achieved surface elevation using engineer's level and staff rod.
4) In case of traditional leveling, an assumed datum or mean sea level is used to compute elevations of grid points with reference to a fixed bench mark (BM). In case of laser leveling the laser plane (generated by the laser beam) is used as reference/datum for all rod readings at grid points. No assumed bench mark reading is needed in laser technology if survey and leveling is completed in one setting. A bench mark reading is definitely required if

transmitter is to be shifted or survey has to continue next day with different transmitter setting.

5) In traditional leveling, either rod readings or computed elevation may be used to calculate cuts and fills as well as average elevation, while in laser leveling only rod reading with reference to laser plane may be used to calculate the cut and fills as well as the average rod reading.
6) The traditional method requires establishing a reference point (Bench mark), setting of survey stakes on a measured grid pattern, taking rod reading at each grid point, calculating correct elevation, setting the stakes at correct elevations and marking for cuts and fills. The laser system on the other hand requires no setting of stakes. Instead, the rod readings are directly taken with level eye detector at the established grid points.
7) In traditional leveling, tractor operator has consistently to make reference to to stakes, cut and fill map; and therefore the leveling operation requires continuous checking of elevations; while in laser systems, the tractor operator utilizes an automatic control system and the operation continues with reference of laser plane through signal displays with no reference to stakes. The level plane is automatically achieved without continuous checking.
8) Laser system can command much larger area with greater confidence in one setting of the instrument as compared to the commanded area under traditional leveling.
9) Initial cost of laser equipment is much higher while the operational cost is much lower than that of traditional leveling. Preliminary comparison shows that cost of laser leveling is about 60-70% of traditional method (MINFALL and NESPAK 1996).
10) Laser leveling requires fine soil preparation and relatively a smooth plane field than traditional leveling before leveling equipment is put to operation.

8.4.3. Components of Laser Leveling System

A complete laser system operated with scraper and tractor is shown in Fig. 8.7 and 8.8. It comprises the following components.

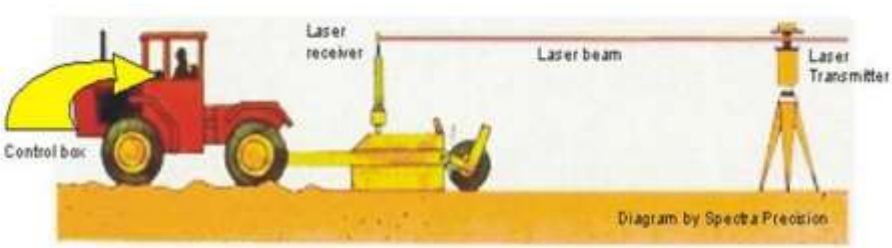

Fig. 8.7 Demonstration of Laser Leveling Principle

Fig. 8.8 Laser Land Leveling Equipment in Operation

- Laser Transmitter
- Tripod and Elevating Base
- Telescopic Grade Rod
- Level Eye Detector
- Receiver
- Mast
- Laser Scraper
- Three Light Display
- Control Box
- Battery and Electrical Connections
- Hydraulic System
    - i) Hydraulic Pump
    - ii) Hoses
    - iii) Hydraulic Oil Reservoir
    - iv) Solenoid Control Valves
    - v) Hydraulic Cylinder
    - vi) Hydraulic Oil Filter

8.4.3.1 Laser Transmitter

Laser transmitter (Fig. 8.9) is electronically self leveling instrument with rotating laser beam that establishes a leveled reference plane of laser beam above the working plane (field area). The laser beam is produced by an internal or external battery powered diode. It contains built-in 12 volt battery that may last for 25 to 30 hours. Low voltage indicator lamp flashes when battery voltage falls below operating voltage. The beam rotates at a speed of 600 rpm and automatically shuts off if level is disturbed. It can effectively operate within 300 m radius, but operates the best for 200 m radius.

For a single or dual grade transmitters (i.e in single directional or two directional leveling), the reference plane of laser beam can be tilted for 0-10 degrees in single or dual axis, respectively with an accuracy of 5 degrees.

Fig. 8.9a Transmitter of the Laser System

8.4.3.2 Tripod

The transmitter is fitted over the tripod legs through an elevating base. The desired height of instrument is achieved with leg extension and fine height adjustment with elevating base, which has a locking and quick release mechanism for laser transmitter.

Fig. 8.9b Tripod for Mounting Transmitter

8.4.3.3 Telescopic Grade Rod (Level Eye Detector)

The telescopic grade rod is used in conjunction with the level eye detector to conduct topographic survey of the field to be leveled (Fig. 8.10). The survey may be accomplished by one man carrying the telescopic grade rod. It is calibrated similar to the staff rod or provided with a tape which automatically indicates the height of laser plane through level eye detector. The detector receives the laser beam and beeps continuously when inline with the laser beam. In case the beam is above or below the detector, intermittent beep whistles. A leveling bubble enables the alignment of rod vertically for accurate topographic reading.

8.4.3.4 Mast

It mounts the receiver over the scraper as shown in Fig. It may be rigid, electrically or manually operated to set the appropriate height of receiver. It may be one piece solid rod or may consist of two pieces, one sliding into the other to facilitate height adjustment.

Fig. 8.10 Telescopic Grade Rod or Laser Staff Rod

8.4.3.5 Receiver

The function of a receiver (Fig.8.11) is to receive the laser beam emitted by the emitter and to transmit signals to the hydraulic system and control box. It has a vertical sensing range of 0 to 18.6 cm. it is 360 degree detector mounted on the mast of the scraper. Thus, vertical movement of the scraper is the same as that of receiver with respect to the laser plane.

Fig. 8.11 Laser Receiver

8.4.3.6 Light display

The light display (Fig. 8.12) resembles with the traffic signals. It receives signals from the receiver and indicates the relative position of the cutting edge of the scraper to the desired grade/ level of field through green and yellow lights. It guides the operator through visual light display regarding the position of cutting edge of the scraper relative to the desired elevation of the field as summarized below.

Table 8.3 The Description of the Light Display and their Interpretations.

Description	Light Display Indicator	Range
Higher position of cutting edge	Solid Amber	3 – 7.5 cm
Near Center high	Flashing amber	< 3 cm high
Correct position (Desired Level)	Flashing Green	Within accuracy
Near Center Low	Flashing amber	< 3 cm below
Lower position of Cutting Edge	Solid	3 -7.5 below

Fig.8.12 Light Display

8.4.3.7 Hydraulic Pump and Hoses

The hydraulic pump system of laser operated scraper works independent of tractor hydraulic system. It is operated by PTO of tractor @ 540 rpm. The hydraulic pump transmits power to the hydraulic cylinder of the Scraper through high pressure hoses for lowering and raising of the scraper, cutting edge.

8.4.3.8 Hydraulic Oil Reservoir and Solenoid

The reservoir provides storage for the hydraulic oil. It also dissipates heat produced when system operates. The solenoid receives signals from receiver through control box and controls the raising and lowering of the cutting edge of blade in accordance with the laser plane.

8. 4.3.9 Laser Scraper

Laser scraper (Fig.8.13) performs the function of cutting the high spots of soil and collects it to unload at lower elevation points as guided by the laser system. It is hinged through the link bars to the tractor hook and mounted on the rear wheels. The elevation of the cutting edge of the scraper is controlled by the hydraulic cylinder as operated by the Hydraulic system and solenoid under laser signals received through the receiver mounted on the scraper mast. Position of the cutting edge of the scraper is displayed on the control box as well as on the light display, which guide the operator whether the scraper is operating above or below the average elevation.

Fig. 8.13 Scraper Mast and Hydraulic System in Laser Land Leveling

8.4.3.10 Control Box

The control box (Fig. 8.14) is fixed on the right front position of the operator. It is provided with a number of functions and indicators, which guide the operator about the performance of the laser leveling operation. It also permits the operator to manually raise or lower the scraper as indicated by display mechanism. The control box shows the following keys/functions:

- Power supply indicator
- On-off switch controls power to the laser system from battery.

- Light display or Grade Indication Lights to indicate the relative position of the cutting edge of scraper to the desired grade.
- Auto – Manual switch: It permits the operator to put the laser system to automatic or manual mode of operation. In automatic position, signals are sent to hydraulic control valve for automatic control of the blade with respect to predetermined grade/ level of field surface. The manual mode is used when the scraper cuts too low or the scraper gets heavily loaded with earth material. When shifted to manual position, it over rides the automatic control system.
- Raise- lower Switch: This switch is used by the operator to raise the scraper when it cuts too deep or the scraper gets heavily loaded with soil material. This key is used to manually lift the scraper for releasing the extra soil load. Such a function is performed when the laser is set to manual mode.
- The control box is provided with 4 to 5 sockets for electrical connections to the battery, 3-light display, solenoid valve and receiver.

Fig. 8.14 Control Box of a Laser System

8.4.4. Laser Leveling Procedure

The following procedure is adopted for carrying out laser leveling (USDA 1076).

1) Select the size of area to be leveled with laser system. The laser beam can be used with radius of 300 m. However, for best results the effective radius should be limited to 200 m.
2) Set the layout of the field to be leveled in a grid pattern with pre selected grid size such as 10 × 10 m, 15 × 15 m or 20 × 20 m as the field size may permit. Accordingly, one grid area would be 100, 250 or 40m2. Smaller the grid size, more accurate is the leveling. This step is similar to that explained under traditional leveling.

3) The transmitter can be set at the center or at periphery of the area to be leveled as permitted by the effective radius of laser plane and convenience of the operator. The following steps may be taken to install the transmitter for successful leveling operation.
4) Set the transmitter on a firmly placed tripod and adjust its height nearly to the position of the receiver.
5) Level the transmitter by brining the leveling bubble in the centre to start laser beam. Remember that laser beam would not operate unless the instrument is properly leveled.
6) Make sure that the transmitter is charged if internal battery is provided; otherwise hook up-to a 12 volt battery.
7) Check the level eye detector batteries and all connections of the laser systems with hydraulic system of the tractor and control box.
8) Survey the area to be leveled using level eye detector and record rod readings at established gird prints with reference to an established bench mark (B.M.). Since staking is not required with laser system and only flags can be placed at grid points, survey can be completely by one man holding level eye detector.
9) Record rod readings on grid sheet (Fig. 8.15) or field book and prepare the topographic map.
10) Determine the average rod reading (Table 8.4), Depth of Cuts and Fills (Table 8.5), volume of cut (Table 8.6), volume of fills (Table 8.7) and cut-fill ratio as given in the following example. This applies the same way to the leveling procedure with engineer’s level.

8.4.5. Example 8.3

Calculation of C:F Ratio

Given the grid sheet measuring 56mx55m as in Fig. 8.15where rod readings (m) have been recorded using level eye detector placed at center of each grid, which measure as under:

1.84	1.86	1.87	1.88	1.86	1.84
1.82	1.84	1.86	1.87	1.85	1.84
1.80	1.82	1.84	1.86	1.84	1.82
1.79	1.80	1.82	1.84	1.82	1.81
1.77	1.78	1.80	1.84	1.80	1.79

Fig. 8.15 Grid Sheet showing the Rod Readings

Legend

Standard grid size = 10 × 10 cm

Standard grid area = 100 m^2

Total Number of Grid areas = 30

Number of Standard Grids Measuring 10 × 10 m (Bold/White) = 20

Number of Grids Measuring 15 m × 10 m (Italic & underlined) = 5

Number of Grids Measuring 6 m × 10 m (Italic Values/Yellow) = 4

Number of Grids Measuring 6 m × 15 m (Normal /Red) = 1

Table 8.4 Determination of Average Rod Reading

Proportion of standard grid areas	No. of specific size areas	Sum of rod readings	Adjusted rod readings	No. of adjusted grid areas
1	20	36.78	36.7×1 = 36.78	20×1 =20
1.5	5	8.92	8.92×1.5=13.38	5×1.5=7.5
0.6	4	7.31	7.31×0.6= 4.39	4×0.6=2.40
0.9	1	1.79	1.79×0.9= 1.61	1×0.9=0.9

Total Sum of Adjusted Rod Reading =56.16

Total number of adjusted grid areas = 30.80

Average rod reading (56.16 / 30.80) = 1.823 m

Adjustment for shrinkage or compaction = 0.004 m

Adjusted average rod reading (1.823 m + 0.004 m) = 1.827

Table 8.5 Work Sheet to Calculate Depth (m) of Cuts and Fills

1.827-1.84 = -0.013*	1.827-1.86 = -0.033	1.827-1.87 = -0.043	1.827-1.88 = -0.053	1.827-1.86 = -0.033	1.827-1.84 = -0.013
1.827-1.82 = 0.007**	1.827-1.84 = -0.013	1.827-1.86 = -0.033	1.827-1.87 = -0.043	1.827-1.85 = -0.023	1.827-1.84 = -0.013
1.827-1.80 = 0.027	1.827-1.82 = 0.007	1.827-1.84 = -0.13	1.827-1.86 = -0.033	1.827-1.84 = -0.013	1.827-1.82 = 0.007
1.827-1.79 = 0.037	1.827-1.80 = 0.027	1.827-1.82 = 0.007	1.827-1.84 = -0.013	1.827-1.82 = 0.007	1.827-1.81 = 0.017
1.827-1.77 = 0.057	1.827-1.78 = 0.047	1.827-1.80 = 0.027	1.827-1.84 = - 0.013	1.827-1.80 = 0.027	1.827-1.79 = 0.037

Note:

* Where the actual rod reading is greater than the average, shows a fill (Minus Values in unshaded grids)

** Where the actual rod reading is less than the average, shows a cut (Plus values in shaded grids)

Total number of grids with cuts = 14 Cuts = Italic & Underlined

Total number of grids with fills = 16 Fills = Bold

Table 8.6 Determination of Volume of Cuts

Proportion of standard grid area	No. of specific size areas	Sum of cuts (m)	Adjusted sum of cuts (m)	Specific size of grid (m^2)
1	7	0.119	0.119 × 1= 0.119	Standard Girds 10x10 = 100 m^2
1.5	4	0.158	0.158 × 1.5=0.237	1.5 times the standard grid 15 × 10 = 150 m^2
0.6	2	0.024	0.024 × 0.6=0.0144	0.6 times the standard grids 6 m ×10 m = 60 m^2
0.9	1	0.037	0.037 × 0.9=0.033	0.9 times the standard grid 9 m × 10 m =90 m^2

Total Adjusted Sum of Cuts = 0.4034m

Total Volume of Cuts = 0.4034 m^3 × 100 = 40.34 m^3

Table 8.7 Determination of Volume of Fills

Proportion of standard grid area	No. of specific size areas	Sum of Fills	Adjusted sum of Fills	Specific size of Grid
1	13	0.359	0.359 × 1=0.359	Standard Grids =10 m ×10 m
1.5	1	0.013	0.013 × 1.5=0.0195	Grid Area =15x10
0.6	2	0.026	0.026 × 0.6=0.0156	6 × 10
0.9	0		0	

Total Adjusted Sum of Fills = 0.3941m

Volume of fills = 0.3941 m × 100 m^2 = 39.41 m^3

Ratio between volume of cuts and volume of fills i.e.

Cut: Fill Ratio or C: F Ratio = Volume of cut / Volume of fill

C:F Ratio = 40.34 / 39.41

or 0.4837 / 0.3941

= 1.023

In principle, if the C:F ratio, after considering the adjustments for shrinkage, borrow and extra fill (if any), falls within the range of 1.0 to 2.0, the computed cuts and fills would satisfy the precision leveling is satisfied. As the C:F Ratio calculated above, falls within range, the estimated average rod reading is acceptable. In case the C:F Ratio falls out of the above mentioned range, a new average rod reading is desired by increasing or decreasing the shrinkage factor (0.001 to 0.03) in accordance with the soil type. The map (Fig. 8.15) showing grids with finalized Cuts and Fills should be handed over to the laser operator for leveling operation.

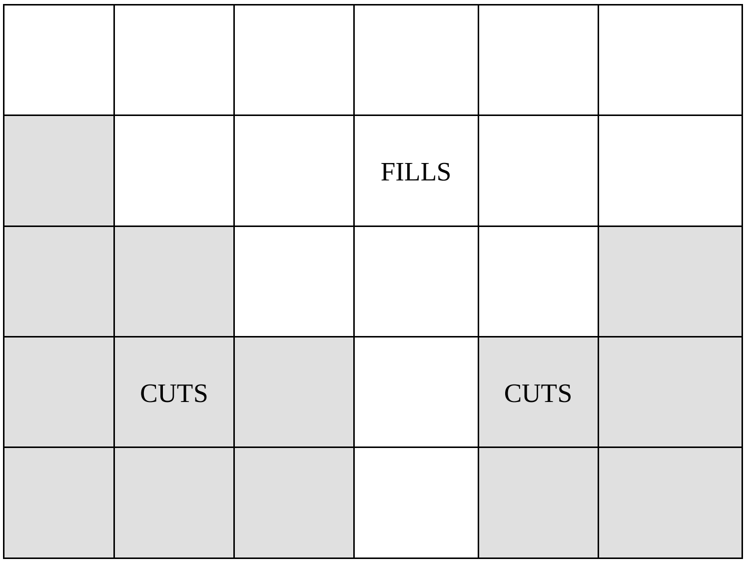

Fig. 8.15 Map Showing Areas with Cuts (Shaded) and Fills (Unshaded) to Guide the Operator

Fig. 8.16 shows a completed laser leveled field where the soil has been moved from the cut areas to the fill areas, achieving average elevation at all the points in the field. This field has been leveled at zero grade in all d irections.

Fig. 8.16 A laser Leveled Field

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Chapter-9

Watercourse Design and Construction

9.1. Watercourse

A watercourse is an open channel in which water is exposed to the atmospheric pressure and therefore flows under the force of gravity. It is the last component of tertiary irrigation system for official water distribution to the commanded area. A watercourse is an open channel, which receives water from the source such as a distributary or a minor distributary through canal outlet and delivers to the farms and fields in accordance with warabandi. The watercourse that originates from the canal outlet and delivers water to the farmer branches through turnouts or “Nacca”, in a watercourse command, is usually termed as Main Watercourse or “Sakari Khal”. Under warabandi system of water distribution, each turnout receives the same flow rate as in the main watercourse, in accordance with the water distribution schedule. Generally, the main watercourse is constructed and renovatedby the water users of the command but managed by the Provincial Irrigation Department. Most of the watercourses have been reconstructed by the Provincial Department of Agriculture under the On-Farm Water Management Project.

A watercourse command refers to the agricultural area commanded or irrigated through the given watercourse, which may carry canal water, pumped water or both. A typical watercourse command would on the average comprise 400 acres of land and 50 shareholders. Under the On-Farm Irrigation System, Irrigation channels at the watercourse command may comprise main watercourse (“Sarkari Khal”) and Farmer Branches or Field Channels. The main watercourse is managed by the Provincial Irrigation Department (PID) and maintained by the farmers, while farmer branches are completely controlled by the owning farmers.

The channel that receives water from the main watercourse and leads it to farmers’ fields is termed as farmer branch. The design specifications of both types of channels are the same because whole of the flow has to be diverted from main watercourse to the farmer’s branch under the existing on-farm water distribution (warabandi) system.

A closed conduit (pipe), on the other hand, contains water flowing under pressure i.e. gauge pressure (above atmospheric pressure). Usually it happens when the pipe flows full. In case, the pipe is not full of water, the empty space allows the water to be exposed to atmospheric pressure, and therefore, flows under the force of gravity just like an open channel. Flow through circular pipes, embedded or above ground may be under atmospheric pressure or under gauge pressure. Care must be done to use appropriate design equations for the given pipe flow system. Under the existing On-Farm irrigation system, both the main watercourse and farmer branches usually constitute semi circular unlined cross section. The lined channels are constructed in rectangular brick lined or precast concrete lined semi circular cross sections.

9.1.1. Watercourse Selection for Improvement

Selection of a watercourse for improvement / renovation or construction requires meeting the following criteria.

- The share holders of the watercourse command must be willing to participate in the improvement program, provide necessary labor force and financial input as and when required.
- Priority should be given to the watercourses, which have potentially higher water losses, located in the salinity affected areas and are badly shaped to maintain.
- Watercourses having higher %age of small land owners should be preferred.
- Farmers are willing to reconstruct in accordance with the right-of-way approved by the Government without getting into problems, conflicts or litigation among the share holders.
- Farmers are willing to cooperate with the Government, implementing agencies as well as with the Water User Association in the best interest of conservation of water resources, better productivity and uplift of rural community.

9.1.2. Objectives of Watercourse Improvement

The purpose of watercourse improvement or lining is to convey irrigation water from the canal outlet to the fields and achieve the following benefits.

- To minimize conveyance water losses through the watercourses leading to maximum conveyance and application efficiencies.
- Prevent dead storage, over spilling, loss of adjoining land in water diversion (opening and closing of “nacca”), excessive silting and erosion during conveyance, elimination of mole holes, cracks or undesirable vegetation and creation of uniform cross section and slope etc., thereby improving the uniformity and equity of water distribution among the land owners.
- Maintain full supply level without over spilling and leaking opportunities through banks and joints.
- Provide sufficient working head for irrigation of the commanded area.
- To avoid submergence of canal outlet, which may obstruct the inflow to the watercourse.

9.2. Purpose of Watercourse Design

The purpose of designing a watercourse is to establish geometric specifications (width, depth as well as longitudinal and side slopes) for given discharge, roughness coefficient and topographic slope of the commanded area. In order to facilitate irrigation of each field of the command, the watercourse design establishes the Full Supply Level (FSL) in the watercourse to provide a sufficient working head or effective head for irrigating the highest commanded fields without submerging the canal outlet commonly known as "mogha".

When a system of watercourses has to be designed for a given farm or a command, it is generally advisable to design the branch watercourses or field channels or farmer branches first, followed by the main watercourse ("Sarkari Khall"). The topographic map of the watercourse command, profile map of the existing watercourses, crest of mogha, the right-of-way, authorized locations of farm outlets and the design discharge of the outlet are the basic data required for design of a watercourse. The procedure of designing a watercourse is summarized below:

9.3. Design Steps

The following steps are adopted for designing a watercourse.

1) Obtain a map of the commanded area from the Provincial Irrigation Department comprising location of "mogha", watercourse layout, field boundaries, authorized location of farm outlets, farm buildings and any other features of the command.
2) Conduct a topographic survey of the watercourse command starting at a Banch Mark (BM) such as top of canal outlet ("Mogha") or any other permanent structure. Prepare a Topographic Map of the commanded area and indicate the elevation of each unit area.
3) Obtain the full supply level of the distributary, crest elevation and throat width of outlet, elevation of the lower tip of roof block.
4) Measure the actual discharge of the outlet preferably during summer season to enable the designed watercourse to accommodate highest flow rate.
5) Conduct a watercourse profile survey recording the existing bed elevations, actual water surface elevations and existing full supply level along the length of the watercourse at equidistant specified locations. Also determine the bed and top elevations, dimensions and locations of all existing structures in the watercourse such as drop structures and turnouts etc.
6) Record the bed elevation of farmer branches below each turnout (farm outlet).
7) Find elevation of the highest field of the command from topographic map to facilitate irrigation of all the commanded area. Scraping of excessively high elevation fields may also be recommended to avoid the need of excessively raised channels and reduce the cost.

8) Determine the required FSL of the water surface in the watercourse to permit irrigation of the highest commanded field by each turnout. This can be achieved by adding 0.15 m (6 inches) to the highest field elevation for providing the requisite working head i.e. to facilitate movement of water from the watercourse to the field. Also add a minimum of 0.01 m to 0.04 m for each 100 m distance starting from the turnout depending on the availability of head to provide sufficient bed slope (0.0001m/m) for gravity flow.
9) Prepare a profile map of the main and branch watercourses on a graph paper, by plotting the elevations of water surface from watercourse bed. Indicate the elevations of existing bed, existing FSL, the location and elevation of all exiting water control structures, the FSL of distributary and crest of "mogha" and tip of roof block on the graph paper.
10) Design calculations should start at the elevation of highest field to be irrigated and work back to allow all the losses to include working head of at least 0.15 m, channel slope of 0.0003 to 0.00045, desired drops in the bed of channel, check structures and culverts etc.
11) In case, the resultant design elevation of the water surface in the distributary is too high, review the design by making adjustments in channel bed slope, water surface profile or drops etc. to match with the elevation of highest field and permissible water surface elevation at canal outlet for unobstructed flow and minimum head loss.

9.3.1. Topographic Survey

A topographic survey comprises determining the elevations of the fields of a watercourse command, particularly along both sides of the watercourse. These elevation data are used to prepare a topographic map, which gives the highest elevation where water must reach to irrigate all the fields commanded by the watercourse. These data alongwith the profile map data are utilized to design the watercourse. Following procedure is followed to coduct topographic map:

- Obtain an official copy of the watercourse command area map from the booking clerk of the irrigation department and use it as a reference to correctly locate the features of the command in developing the topgraphic and profiles maps of the command.
- Draw the map of the watercourse command showing the line diagram of the watercourse and its branches, acre fields, turning points, junctions, culverts, drops, buildinsgs and other features of the command.
- Using the nearby available Bench Mark and assumed elevation above Datum, record the topographic elevation of each field in the Field Book by placing the staff rod in the centre of each field. Record the field elevations on the map also.
- Record the elevation of Crest of the canal outlet as well as the Full Supply level in the watercourse as well as in the canal.

9.3.2. Profile Survey

Profile survey is the process of determining elevations of the existing bed, banks, Full Supply Level (FSL) along the length of the watercourse at specified intervals with reference to the Bench Mark established for topographic survey given above. The stakes are usually set intervals of 20 or 30 or 40 meters depending on the required accuracy of survey. In addition, the stakes should also be set at points where the watercourse changes direction or point of diversion or control structure installation. Station ponts are designated on the field book as 0+00, 1+00, 2+00 ….. etc., at beginning and successive stations where rod readings are taken.The digit to the left of plus sign indicates the distance in multiples of 100 meters and digits to right show the distance less than 100 m. The procedure of conducting profile survey can summarized as under:

- Set the leveling instrument at appropriate location beside the watercourse from where maximum number of observations can be taken.
- Set the stakes at selected intervals (e.g. 20, 30 or 40 m) along the watercourse and take rod readings at the bed and FSL of watercourse at all slected points. Also take rod reading at FSL of supplying canal as well as the crest of oulet after taking first reading at BM.
- Also record the rod readings at the centre of each field on both sides along the watercourse. Complete the profile survey from beginning to the end of watercourse.

9.3.3. Watercourse Design Example 9.1

The following example of design work sheet for a main watercourse with given 2 turn outs, canal outlet and longitudinal slope and other field data, would demonstrate the calculation procedure as given in Table 9.1. The watercourse design has been initiated at the highest field elevation. This is followed by addition of desired working head (0.15m), increase in elevation for the given bed slopes over given lengths of branch and main watercourses i.e. S=0.00045 and 0.00042, respectively, head loss through two check structures and the canal outlet, which concludes the desired head of water surface in the distributary or canal. The FSL found above the canal outlet should match with the elevation of water surface in the canal. In case, some drops are suggested in the bed of watercourse, the desired elevation drop should be added or subtracted to reach the desired water stage in the canal. The design of irrigation channel or watercourse is carried out using design data of bed width, side slope, bed slope, manning's n etc. as given below.

Table 9.1 Watercourse Survey Data and Design Worksheet with assumed datum

Station	Description	Elevation (m)	Head Loss (m)	FSL (m)
1	Elevation of BM	8		
2	Elevation of highest field	8.85		
3	Recommended working head		0.15	8.85+0.15=9.00 (FSL at field inlet)
4	Branch 134 m upstream with S=0.00045		0.06	9.00+0.06=9.06 (FSL at branch inlet)
5	Loss through Check structure-1		0.03	9.06 +0.03= 9.09 (FSL at upstream of structure)
6	Loss through Main channel, 336 m upstream of check structure for bed slope =0.00045		0.14	9.09+0.14= 9.23 (FSL at upstream of main channel below structure-2)
7	Head Loss through check structure -2		0.03	9.23+0.03=9.26 (FSL at upstream of structure 2)
8	Head loss through 167 m to canal outlet (mogha) @ S=0.00042		0.07	9.26+0.07=9.33 (FSL below mogha)
9	Head loss through mogha		0.05	9.33+0.05=9.38 (FSL above mogha)
10	FSL at canal/ upstream of Mogha			9.38m

9.3.4. Design Considerations

The flow of water in the open channels is controlled primarily by two opposing forces i.e. gravity and friction. Gravity forces the water to flow downstream, where as friction tends to resist the flow which depends on the degree of roughness, i.e. the condition of vegetation, type of material of the watercourse, and the velocity of flowing water. When the flow is steady and uniform, the gravitational and resisting forces are in equilibrium, their relationship is defined by the Manning's equation as given below:

$$V = \left(\frac{1}{n}\right) R^{\frac{2}{3}} S^{\frac{1}{2}} \quad \text{(metric units)} \qquad (9.1)$$

Where:

V= Velocity of flow of water, m/s

R= Hydraulic radius, m, defined by A/P

S= Slope of hydraulic grade line ($Z+V^2/2g+P/\gamma$)

A= Cross-sectional area of channel, m^2

P= Wetted Perimeter, m

n= Manning's, Roughness Coefficient

Equation 9.1, when used with FPS units, may be written as:

$$V = \left(\frac{1.49}{n}\right) R^{\frac{2}{3}} S^{\frac{1}{2}} \quad \text{`} \qquad (9.2)$$

Where:

V= Velocity of flow of water, ft/s

R= Hydraulic radius, ft, defined by A/P

A= Cross-sectional area of channel, ft^2

P= Wetted Perimeter, ft

The other variables carry the same meanings and units (if any) as given above.

Manning's equation is the most commonly used as design equation for steady, uniform flow in open channels in conjunction with continuity equation as given below:

$$Q = AxV \qquad (9.3)$$

Where:

Q = Flow rate in the channels, m^3/s and

A and V carry the same meaning as given above

Replacing V in eq. 9.3 by the Eq. 9.1, the flow rate in channel (Q) is given by the Eq. 9.4(Metric System) as under:

$$Q = A \times \left(\frac{1}{n}\right) R^{\frac{2}{3}} S^{\frac{1}{2}} \qquad (9.4)$$

When dimensions of Feet and Cusecs are used (FPS System), the Flow Equation is defined as:

$$Q = A \times \left(\frac{1.49}{n}\right) R^{\frac{2}{3}} S^{\frac{1}{2}} \qquad (9.5)$$

Where, A is in ft^2, R in ft and, n and S carry the same meanings as given above.

9.3.5. Conversion of Metric System Equation to FPS System Equation

The constant of 1.486 comes from conversion of metric system to FPS system as shown below. Manning's Equation in metric system is given by Eq. 29. Thus,

$$V(m/s) = 1/n\ (m)2/3\ S\ ½ \quad (9.6)$$

When converted to FPS system, the equation becomes:

$$V\ (m/s) \times 3.28 = \text{Conversion Coefficient} /n\ (m \times 3.28)^{2/3}\ S^{1/2}$$

i.e. $V(m/s) \times 3.28 = \text{Conversion Coefficient} /n \times 2.202\ (R \text{ in ft})^{2/3}\ S^{1/2}$

For balancing the equation, the coversion Coefficient = 3.28 / 2.202 = 1.486

Thus, $V(ft/s) = 1.486/n\ (R \text{ in ft})^{2/3}\ S^{1/2}$ (9.7)

While designing an open channel, the problem usually is to determine the channel specifications (width and depth) for given flow rate, slope, side slope, permissible velocity and roughness coefficient. The channel dimensions may be defined for different cross sections as summarized below:

9.3.6. Rectangular Cross Section

In rectangular cross sectioned channel (Fig. 9.1), the geometric parameters are related by the equations:

$$A= b\ d \quad (9.8)$$

$$P = b +2d \quad (9.9)$$

$$R = A/P = bd/ (b+2d) \quad (9.10)$$

$$Q = 1/n\ R^{2/3}\ S^{1/2}\ A \quad (9.11)$$

$$Q = 1/n\{ (bd) /(b+2d)\}^{2/3}\ S^{1/2} \quad (9.12)$$

For optimal cross section b=2d

Therefore,

$$Q = 1/n\ \{2d^2 / 4d\}S^{1/2} \quad (9.13)$$

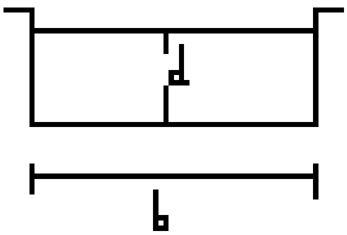


Fig.9.1 Rectangular Cross-Section of Channel Showing Geometric Parameters.

9.3.7. Trapezoidal Cross Section

In trapezoidal cross-sectional channel (Fig.9.2), the geometric parameters can be related by the equation 9.14.

$$A = bd + zd^2 \quad (9.14)$$

$$P = b + 2d\,(z^2 + 1)^{1/2} \qquad (9.15)$$

$$R = (bd + zd^2) / (b + 2d\,(z^2 + 1)^{1/2})$$

$$\text{or} \quad AR^{2/3} = \frac{(bd + zd^2)^{5/3}}{(b + 2d\,(z^2 + 1)^{1/2})^{2/3}} \qquad (9.16)$$

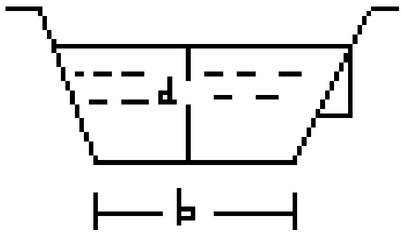
Where,

b = bottom width, m

d = depth of flow, m

z = side slop (i.e. horizontal/vertical)

Fig. 9.2 Trapezoidal Open Channel Cross Section Showing the Geometric Parameters



Solution to the above given Manning's equations for the channel specifications (i.e. b, d, and z) is possible only by trial and error. The solution will therefore follow by assuming one value of b and z and finding value of d, till the two sides of the equation are balanced. Trapezoidal section is the most commonly used shape for both earthen and lined channels. Rectangular shapes cannot be maintained in earthen watercourses, while the lined channels are mostly constructed in rectangular shapes.

9.3.8. Semi-Circular Cross Section

In Semi Circular cross-section (Fig.3); the geometric parameters are related by the equations:

Area (A)	=	$(\pi r^2)/2$	(9.17)
Wetted Perimeter (P)	=	πr	(9.18)
Hydraulic Radius (R)	=	A/P	(9.19)
$R = r/2$			(9.20)

Where, r = Radius of the wetted channel

9.4. Design Criteria

The criteria to design a watercourse as defined by the MINFAL (1980) include:

1) Achieve maximum conveyance efficiency and minimize water losses.

2) Prevent erosion and silting.
3) Provide high enough FSL to ensure proper working head to the command area and low enough to prevent submergence of canal outlet (“mogha”).

9.5. Optimum Cross-section

For a given Q, there may be many combinations of width and depth of the channel. The economic designing of an open channel of course, requires minimum size specifications for given flow rate and slope, which introduces the concept of optimum cross-section. The channel cross-sections in the field may be rectangular, trapezoidal or semi circular. The optimum cross-sections in each case may be achieved if following conditions are met:

1) Maximum flow (Q) is achieved for a minimum cross sectional area.
2) The wetted perimeter is minimum for a given cross section and flow rate.

The semi circular cross-section is the optimum (most efficient) cross-section among the above given shapes, which utilizes minimum wetted perimeter of channel allowing maximum discharge for the given design specifications. The concept of optimum cross-section (minimum wetted perimeter) can be better understood by the Fig.9.3:

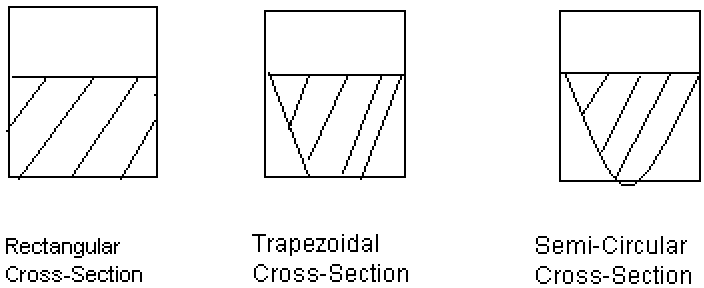


Fig.9.3 Types of Channel Cross Sections

The following equations will give the hydraulically optimum cross section.

1) For Trapezoidal Cross Section $b = d(2+ (z^2+1) - 2z)$
2) For Rectangular Cross Section $b = 2d$

Where, the variables carry the same meanings as given earlier.

9.6. Roughness Coefficient

When the fluid is in motion, there is friction between the fluid and the surface of the channel depending on the roughness and type of the channel material. The commonly adopted values of roughness coefficient (n) for different materials of watercourse construction are given in Table 9.2 and those measures in Pakistan are given in Table-9.3. The values of Roughness Coefficient as adopted by the Ministry of Food Agriculture and Cooperatives MFA&C, Government of Pakistan 1996, are given in Table 0.4.

Table 9.2 Value of ‘n’ for Earthen Vegetated and Lined Channels

Type of Channel	Avg. Roughness Coefficient(n)
a) No Vegetation	
Earthen Channel	0.018
Clean and Fresh	0.022
Clean and Old	0.025
Gravel Uniform Section	0.025
b) Vegetated	
Short Grass	0.027
Grass with some Weeds	0.030
Dense Weeds	0.035
Stony Bottom and Weedy	0.035
c) Lined Channel	
Neat, Smoot Surface	0.011
Cement Mortar	0.013
Concrete	0.015
Glazed Brick	0.013
Cement Mortar	0.015
Masonry Cemented	0.025

Table 9.3 Values of ‘n’ as Measured in Pakistan

Watercourses/Channel Condition	“n” value
Lined with brick masonry	0.018
Earthen newly built, Uniform clean	0.017-0.03
Earthen winding with no vegetation	0.03-0.035
Earthen uniform with short grass	0.026
Earthen winding with grass and some weeds	0.035-0.055
Earthen with dense weeds	0.05-0.2

Table 9.4 Recommended Values of Roughness Coefficient "n" For Small Channels as Adopted by MFA&C ((1980)

S.No.	Description	"n" Value
	Unlined Earthen Channels	
1	Newly Built Straight and Uniform	0.025
2	Aged and Vegetated	
	(i) Short Grass	0.035
	(ii) Long Grass	0.065
	Lined Channels	
1	Concrete	0.014
2	Brick Plastered	0.013
3	Brick Un-plastered	0.018

Source: USDA and FAO (1977)

9.7. Permissible Velocity

In earthen channels, excessively higher velocities tend to erode the materials while lower velocities tend to deposit the eroded materials causing degradation of channels. Therefore, the earthen channels should be designed to develop velocities that are neither erosive nor sediment depositing. In order to minimize sediment deposition, in the channel, a minimum velocity of 0.2 m/s should be maintained. Maximum permissible flow velocities and side slopes for different soil materials are summarized in Table 9.5.

Table 9.5 Maximum Permissible Flow Velocities and Side Slopes (Booher, 1974)

Channel Surface	Maximum Velocity (V) M/Sec	Maximum Side Slop (z)
Sand	0.3-0.7	3
Sandy	0.5	2-25
Sandy Loam	0.75	2
Silty Loam	0.75- 0.9	1.75-2.0
Loam	0.6-0.9	1.5-2.0
Silty Clay Loam	1.0	1.0-1.5
Clays	0.9-1.5	1-2
Gravel	0.9-1.5	1-1.5
Rock	1.2-1.8	0.25-1

9.8. Free Board

It is the height of channel bank (berm) above the maximum Full Supply Level (FSL). As it is above the designed depth of water, it does not become the part of design equations such as manning and continuity. It is provided to accommodate excess flow rate or flood water under most severe operating conditions. Free Board should not be less than 1/3 of maximum design flow depth and in no case less than 15 cm for earthen channels, 10 cm for rectangular lined channels and 7.5 cm for trapezoidal lined channel sections. (MFA&C, 1996). The berm width measured at the elevation of free board should not be less than 30 cm.

9.9. Full Supply Level

The Full Supply Level is the highest elevation of flowing water, which is achieved by adding working head to the design Depth of the channel above elevation of the field where water would be delivered. A minimum working head of 10 to 15 cm should be provided at each turnout.

9.10. Side Slope

Unlined irrigation channels must be designed with stable slopes provided in the sides of trapezoidal channels. The side slope (Z) is defined by Horizontal /Vertical and would mainly depend on the texture of soil that determines the stability of sides. Recommended side slopes for different soils are given in Table 9.6 (MFA&C, 1996).

Table 9.6 Permissible Side Slope for Earthen Watercourses

Soil Texture	Excavated Section	Fill Section
Loam, Silt loam, Silty Clays	1:1	1.5:1
Sandy Loam	1:1	2.:1
Loamy Sand and Sand	1.5:1	3:1

Example 9.2

Design a lined rectangular optimal cross section irrigation channel with the following data.

Discharge (Q) = 60.0lps or 0.06m^3/s

Bed Slope (S) = 0.035%

Manning's n = 0.016

Desired Solution: d = ? and b = ?

Solution

Hydraulics of Flow in the channel is defined by:

Manning's equation as

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

And continuity Equation as

$Q = A.V$ or $V = Q/A$

Considering the notation of channel specifications given in Fig. 67

Area = bxd

Wetted Perimeter (P) = b+2d and

Hydraulic Radius (R) = bd/b+2d and

Replacing R and V in the Manning's Equation, we get:

$Q/A = 1/n\ (bd/b+2d)^{2/3}\ S^{1/2}$

Optimal Cross Section Criteria of Rectangular Channel b = 2d

Replacing value of b in the equation, we get

$Q/2d^2 = 1/n\ (2d.d/2d+2d)^{2/3}\ S^{1/2}$

$= 1/n$ (d/2) 2/3 S1/2

Putting the given values of parameters, we get:

$0.06/2d^2 = 1/0.016\ (d/2)^{2/3}\ (0.00035)^{1/2}$

$(0.06x\ 0.016) = 2d^2\ (d/2)^{2/3}\ (0.0187)$

$(0.06x\ 0.016)/\ (0.0187)\ x\ 2= d^2(d/2)^{\ 2/3}$

Consequently, we get $d^{8/3} = 0.04$ which gives $d= (0.04)^{3/8}$

Or d =0.299

As b = 2 × d therefore b = 2 × 2.99

Thus design specifications are d = 0.3 and b = 0.6

Adding Free board of 0.015 m, it gives

D = 0.315 m and d=0.615 m

9.11. Conveyance Losses in Irrigation Channel

As the soil is a porous media, there is always a loss of water due to seepage during conveyance of water through earthen channels depending on the textural class and compaction of soil. The water losses may also occur through watercourse banks, plant roots, cracks and crevices, mole holes, over spilling and bank failure, dead storage and evaporation etc. The water loss can be represented in the form of;

(i) Rate of decrease in flow rate or water loss per unit length of channel as given in Eq.

$$Q_L = (Q1 - Q2) / L \quad (9.21)$$

Where :

QL = Loss rate, lps per 100 m length

Q1 = Flow rate at upstream section of watercourse

Q2 = Flow rate at down stream section of watercourse

L = Length of watercourse section between Q1 and Q2.

(ii) in terms of% loss in flow per unit upstream discharge of channel as given by the Eq.

$$Q_{ud} = ((Q1 - Q2) / (Q1)) \times 100 \quad (9.22)$$

Where:

Qud = Loss rate in% per unit upstream flow

(iii) Water Loss Per Unit Wetted Area of channel (QwA), which is given by the equation:

$$Q_{wA} = ((Q1 - Q2) /(PL)) \times 100 \quad (9.23)$$

Where:

P = Wetted Perimeter of channel

(iii) in terms of volume of water, which comprises measuring the change in volume of water at given elapsed times and is usually expressed in terms of volume per unit time e.g. m^3 / day

9.12. Measurement of Conveyance Loss

The methods of measurement of loss of water include:

1) Ponding Water Losses
2) Steady State Losses
3) Inflow - Outflow Method

The ponding method constitutes blocking of a given watercourse section at upstream and downstream with impermeable earth fill material or compacted soil, which is filled with water. The level of water is marked and let the water seep under normal conditions. Such seepage losses do not include transient losses that may occur due to movement of water.

The steady state water losses are measured when the flow in a channel is in steady state i.e. inflow is equal to the out flow across a section at given cross section. The method comprises installing flow measuring devices such as cut-throat flumes at upstream and down stream sections of the selected length of the channel. The consecutive flow rates observed at both the flumes when the flow has attained steady state, are recorded and loss of water is calculated in accordance with the Eq. The water loss may be expressed as % of inflow or flow rate per unit wetted perimeter as calculated using equation 9.22 and 9.23, respectively.

The inflow-outflow method involves measuring the incoming and outgoing volume of water for a given section (length of watercourse and time duration, provided no water is being diverted between the two measuring points. Also make sure that there is no breach, crack or mole hole within the measuring section. The decreasing outflow as compared to the inflow rate gives the loss of water over the test length of the channel.

9.13. Watercourse Construction

After preparing survey and design sheets, steps to follow during construction of a watercourse may be ssummarized as under (Ministry of Food, Agriculture and Cooperatives, 1996):

- Establish stakes on the centre line of right-of way for watercourse agreed among the famers and associated structures using profile map of the watercourse. The stakes may be placed at 15 m interval.
- Clear the right-of-way of all kinds of materials including trees, brushes, grasses, sand dunes and any structure that may hinder the staking or construction activities.
- After clearance of the right-of- way according to the design specifications and layout plan, refresh the centerline stakes.
- Prepare the watercourse pad eliminating all the high and low elevation points on the way of watercourse. Soil materials used for fill should be free of stones roots, branches and organic materials and should be borrowed from outside the right-of-way.
- Compact the pad at optimum moisture contents using standard procedure and equipment (Hand temper, vibrator or sheep foot roller) achieving the desired bulk density.
- Set the stakes elevations in accordance with the design elevations of the watercourse and structure. Also set the stakes at one half the top width of the watercourse on both sides of the centre line.
- Excavate the compacted pad according to the design specifications (Bottom width, depth, side slope, top width, bed elevation and outlet structure locations. Use of a ditcher with a tractor can help achieving the compaction efficiently.
- Set the stakes at desired elevations in the centre of the watercurse and construct bed and side walls with bricks and plaster in accordance with the specifications.

9.14. Canal Outlet (Mogha)

As the canal outlets and their flow characteristics are important components of watercourse design, their understanding is important for successful operation of watercourse. A canal outlet is a control structure installed at the distributary or minor to convey water to the main watercourse for further distribution to the water users. It can also be used to measure the water delivered to the main watercourse. There are three main types of outlets (Ministry of Food, Agriculture and Cooperatives, 1980):

1) Non- Modular
2) Semi-Modular
3) Modular

The discharge through a non-modular outlet is dependent on the difference between the hydraulic head in the canal and that in the watercourse. Therefore, the outlet discharge may be increased by lowering the water level in the watercourse. A submerged outlet will always operate as non-modular.

A semi –modular outlet is the one in which the discharge depends upon the water level (hydraulic head) in the canal only and is independent of the water level in the watercourse. Once the outlet is free flowing (not submerged), it operates as semi-modular. Adjustable Proportionate Module (APM) outlets fall in this category. Most of the outlets used in Pakistan are of semi-modular type. It is a long throated flume with a roof block, which can be adjusted vertically to change the cross sectional area of flow through the outlet. The roof block is usually made of cast iron with base plate fixed in the crest. A pipe or barrel outlet (fixed above the FSL of watercourse) will operate as a semi modular outlet.

In modular outlet, the discharge is independent of the water level in the canal as well as that in the watercourse. It may be fixed for a given design value. This is achieved by creating a free vortex and eliminating extra head over and above that required for design discharge. These types of outlets are not used in Pakistan.

The Open Flume outlet operates like a cutthroat flume where constriction provided to create super critical velocity at the throat section that lies on the crest or bed of mogha. A control section is the one where the flow is critical. This is followed by expanding section to create subcritical flow downstream

9.14.1. Requirements of Canal Outlet

- An outlet should be strong enough to resist external influences of channel flow and human interference
- Preferably, it should not have removeable parts to minimize periodic maintenance.
- It may draw its fair share of flow and silt to be equitably distributes among share holders.
- It should work efficiently under all working head conditions and flow variations.

- It should be cost effective and does not require manual power to operate.

9.14.2. Terminology

- Flexibiliy: It is defined as the ratio of the rate of change of discharge of the outlet to the rate of change of discharge of the distributary.
- Proportionality: The outlet is said to be proportional when the rate of change of outlet discharge equals the rate of change of channel discharge. In other words, the outlet is considered proportional when flexibility equal unity.
- Setting of Outlet: It is the ratio of the depth of crest level of the outlet below the FSL of the distributary to the Full Supply Depth of the distributary.
- Sensitivity: It is defined as the ratio of the rate of change of discharge of the outlet to the rate of change of water level of the distributary.

References

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Ministry of Food, Agriculture and Cooperation's (MFA & C). 1980. Watercourse Design and Improvement, On Farm Water Management Field Manual, Federal Water Management Cell, Govt. of Pakistan, Islamabad, Pakistan.

Ministry of Food, Agriculture and Livestockn's (MFA & C). 1996. Watercourse Design and Improvement, On Farm Water Management Field Manual Vol. IV. Federal Water Management Cell, Govt. of Pakistan, Islamabad.

Glosary

Abiana: It means rate of charging irrigation water delivered to the farmers on the basis of cropped area or land holding as provided under the Government policy.

Acre-foot: A volume of water required to cover an area of one acre to a depth of 1 foot.

Application Efficiency: Efficiency of the system achieved during the application of irrigation water to the field. The output in this case is the depth of water stored in the root zone and input is the depth applied to a given area of the field.

Aquifer Development: It is a second level development to increase the well yield. Development of lime stone or dolomite aquifers can effectively be done with acid treatment that can dissolve the carbonate materials from the aquifer.

Back sight: Rod reading taken at the point of known elevation during field survey.

Barrage: Barrage is an artificial barrier across a river to prevent flooding, aid irrigation network extension, support navigation system and to generate electricity. It is a low head diversion dam to raise the level of water and divert water through canal system that originates at the barrage. It is a higher level diversion system than a headwork.

Basin Irrigation: Irrigation of a field, which is level in both longitudinal and transverse directions and bounded by low bunds.

Bench Mark: It is a natural or artificial object bearing a marked point whose elevation is assumed or known. It is used as a common reference point for initiating subsequent survey.

Border Irrigation: Irrigation of field comprising narrow and longer strips of land bounded by parallel ridges to guide a sheet of flowing water.

Canal Water Allowance: Number of cusecs of flow designed for the canal to irrigate 1000 acres of commanded land.

Cippoletti Weir: A weir, which is fully contracted sharp crested weir with a trapezoidal control section. The crest is horizontal and side slopes outward with an inclination of 1:4 side slope.

Contracted Weir: A weir, which does not extend to the full width of approach channel i.e. where the crest length is less than the width of the weir.

Conveyance Efficiency: The efficiency with which a given component of irrigation system or a channel conveys water from one location to another. The output in this case is the downstream flow rate and the input is the upstream flow rate.

Cutthroat Flume: A flow measuring flume which has no throat but only conversing and diverging sections. It was designed by G.V. Skogerboe professor of Colorado state University. It maintains depth-discharge relationship at throat section.

Dam: It is a control structure built across a river to store surplus/ flood water, regulate irrigation water supplies, generate electricity and originate irrigation canals by raising water level.

Datum: It is any known or arbitrarily assumed level surface or line from which vertical distances are measured such as mean sea level.

Development of Wells: Process of removing the fine materials from the aquifer, thereby cleaning the openings and enlarging the passages in the formation so that water can flow towards the well more freely.

Differential Surveying: It is type of surveying in which the difference in elevation between two or several points, are found.

Direct Rotary Drilling: It is a drilling process where rock cuttings are removed by continuous circulation of drilling fluid, which is a mixture of water and bentonite clay. The fluid is removed through a pump from storage tank to drill bit through drilling pipe and is returned to the tank with drilled cuttings.

Drip or Trickle Irrigation: Irrigation method, which applies water directly to the root zone of the crops drop by drop under low pressure. It is high efficiency Irrigation system that saves water and fertilizer.

Efficiency: Ratio or percentage of output to input of a given system.

EI30 Index: It is an index, which is defined by the product of Kinetic Energy of storm and the 30 minutes intensity.

Elevation: A measurement of vertical distance to a point above or below a fixed datum, which may be assumed or related to mean sea level.

Field Capacity: Soil moisture contents at which all the gravitational water has drained out of root zone of crop.

Fixed (Pacca) Warabandi: Refers to a fixed schedule of irrigation water distribution developed by the Irrigation Department, which is a binding on the shareholders to follow.

Flexible (Kacha) Warabandi: Refers to the irrigation water distribution schedule developed by the Water Users of the command themselves.

Floodwater Harvesting: Accumulation, storage and use of runoff or flood water before it may be wasted elsewhere. Floodwater may be harvested from watershed, farm areas or urban areas.

Foresight: Rod reading taken at the point of unknown elevation during the survey.

Free Board: It is the height of channel bank (berm) above the maximum water level in the channel (Full Supply Level).

Frequency of Irrigation: Number of days between two consecutive Irrigations for a given crop in a growing season. It is determined by dividing the depth of stored amount by daily ETA.

Full Supply Level (FSL): It is the highest elevation of flowing water, which is achieved by adding working head to design depth of the channel above the elevation of field.

Furrow Irrigation: Irrigation of a field where surface is shaped in small channels called furrows, which carry water down the slope and crop on ridges.

Grid Point: A point on the field area marked by stakes to use as a point for placing staff rod and determining the elevation. A network of grid points are located before starting topographic survey.

Headworks: A headwork is a water diversion structure provided over a main or branch canal to raise the level of water in the canal to divert water to branch canal or distributary originating from the headwork.

Height of Instrument: It is the elevation of line of sight above given datum when instrument is leveled.

Hill Torrent: A non perennial stream of water developed in the hilly areas as a result of heavy rains and descending downstream with a high peak flow rate and impregnated with high silt charge. It is an instantaneous flow through a channel, which is not a part of regular irrigation network.

Horizontal Drainage: It refers to the removal of excess water from waterlogged soil in horizontal direction, which may be accomplished by open ditch drain, surface drain, tile drain or pipe drain.

Inequity of Distribution : It means that the distribution of available water supplies at a watercourse command is not in accordance with the rights of the shareholders.

Intensity of Cropping: Percentage area of a farm put under crop on seasonal or annual basis. For example, if 86 hectares out of 100 hectares in a command is cultivated and cropped, and remaining 14 hectares are not cropped for any reason during summer season, the kharif season cropping intensity is 86%. Similarly, if winter cropping intensity is 75%, the annual cropping intensity would be 161%.

Irrigation : Artificial application of water to the irrigated field through a manually, electrically or mechanically managed system for the purpose of supplying moisture essential for plant growth.

Irrigation Scheduling: Process of predicting the plant future needs for water and developing plans for growing seasons of a crop and application of water to the crop with right amount of water at right time.

Land Drainage: Drainage of agricultural land is the natural or artificial removal of excess water from the soil to create favorable environment for plant growth.

Link Canal: Canal constructed to create a link between 2 rivers for diverting some of the flow from one river to another. Under Indus Water Treaty 1960, the waters of western rivers have been diverted to eastern rivers through the link canals.

Modular Flow: A flow over a weir or through the flume becomes modular when it is independent of variation in tail water level.

Mole Drain: Mole drains are unlined underground drains or channels formed by a mole plow without digging trenches.

Multistage Pumps: A multistage pump comprises more than one stage, which are connected to the central axix. Each stage indicates an impellor. The total head produced by a multistage pump is the summation of heads produced by each stage.

Non Perennial Canal: Canal that delivers water only during Kharif or Summer season and remains dry during Rabi or Winter season.

Optimum Cross Section: Channel cross section that offers maximum flow for a minimum cross sectional area and minimum wetted perimeter for a given cross sectional area that allows maximum discharge.

Orifice: It is an opening in the side of a water container, the top of which is well below the upstream water level and through which the flow takes place. Orifice may operate under submerged or free flow condition depending on the downstream level of outflowing water.

Parshall Flume: It is a flow measuring device designed by the scientist R. Parshall. It maintains Depth-Discharge Relationship at throat section and comprises a converging section, a log throat and a diverging section.

Perennial Canal: Canal that delivers water throughout the year i.e both the summer and winter seasons.

Pipe Envelope Materials: It is the material placed around the clay tile, plastic tile or corrugated pipes to enhance drainage efficiency of pipe drainage system.

Precision Land Leveling (PLL): Land leveling process that consists of grading or smoothing of land surface with little or no slope to precisely determine the elevation of land surface with an accuracy of ± 2cm.

Profile Survey: It is the type of survey in which elevation of surfaces is determined at series of points with measured intervals along a line such as a watercourse, drain, terrace and road etc.

Pump Characteristics: The Characteristics of a pump, how it performs in relation to capacity, head, power and efficiency.

Pump: It is a mechanically or electrically driven device that imparts energy to the fluid for lifting water from a lower level to higher level or producing pressure energy to the fluid at the same level.

Pumping Plant Efficiency: It indicates the performance efficiency of complete pumping plant including motor or engine and pump. It is given by the product of motor and pump efficiencies.

Rabi: Kharif Ratio: The ratio between Rabi and Kharif seasons with respect to river flows. In Pakistan, it is almost 84: 16% during kharif and Rabi seasons, respectively.

Rainwater Harvesting: It refers to the conservation of rainwater. It is accumulation of rainwater from surfaces where the rain falls and stored at a given storage facility for subsequent utilization for useful purposes such as irrigation, domestic uses and groundwater recharge etc.

Reverse Rotary Drilling: This method uses plain water. The fluid is sucked by a pump through the bit via drilling stem alongwith drill cutting and delivered to the tank. In the absence of clay, the stability of hole is maintained by the hydrostatic pressure of water filled in the drilling hole.

Rodkohi Irrigation: It is a system of irrigation, which is accomplished with the water of Hill Torrent.

Roughness Coefficient: When fluid is in motion, there is friction between the fluid and surface of the channel depending on the roughness and type of channel material.

Run-of-the River Power Project: A hydroelectric power generation project installed on the flowing river where little or no water storage is permitted. It is a power generation plant without storage and is subjected to seasonal river flows.

Second-Foot-Day: Volume of water flowing at a rate of one ft3 per second and collected for 24 hours.

Soil and Water Conservation: It is the application of engineering principles to the solution of soil and water management problems resulting in protection against loss of soil and water.

Sprinkler Irrigation: It is a matter of applying Irrigation water under pressure to the crops in the form of rain fall.

Suppressed Weir: A weir, which extends across the full width of a rectangular approach channel.

Surface or Gravity Irrigation: Application of irrigation water by gravity flow to the surface of a field. It includes basin, Border and furrow irrigation systems.

Surveying: It is an engineering science and art by which lines, distances, angles and elevations are established and measured on the earth surface. For this purpose, Engineer’s level, staff rod, tapes and field books are used to accomplish the job.

Terracing: Engineering and agricultural practices to reduce runoff, soil erosion and sediment delivery. Terraces are developed by cutting through the slope that permits to conserve soil and water in hilly areas. Common types include irrigation terraces, bench terraces and channel terraces etc.

Transboudary Water Issues: Issues related to transboundary rivers, which cross boundary between two countries, provinces or regions. Issues related to the Indus river system between India and Pakistan is one of the examples.

Turning Point: A temporary bench mark or point where an elevation is established in order to change the location of subsequent points.

Universal Soil Loss Equation: The equation that predicts soil loss by water erosion based on the influencing factors including soil, water energy, rainfall, topographic conditions, crop management and conservation measures.

Vertical Drainage: Extraction of excess water from a waterlogged land in vertical direction, which can be accomplished by tubewell.

Warabandi: Fixing the turn of irrigation water for the share holders of the watercourse command. It may also be called a rotational method of equitable distribution of water supply in a command by turns fixed according to a predetermined schedule.

Water Erosion: It is the removal of soil particles from land by the impact of running or falling water including runoff and rainfall.

Water Horse Power: The power required to pump water against a given head without considering the friction losses.

Watercourse: An open channel (lined or unlined) in which water flows under the force of gravity and it is the last component of tertiary irrigation system for official water distribution (Warabandi) schedule.

Watershed: Area draining into a river or a river system through a network of small channels or rills resulting from rainfall or snow melt in the area.

Weir: A weir is an obstruction in the channel that causes the water level to rise behind the weir and then to flow over it.

Well Strainer or Screen: A well strainer or screen is a filtering device that serves as intake portion of a well. It permits water and prevents sediments to enter the well.

Wind Erosion: It is removal of soil particles from the soil surfaces by wind energy.

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