University of Basrah

College of Engineering

Civil Engineering Department

BSc. Course

IRRIGATION ENGINEERING

1. Introduction

1.1 Irrigation

Irrigation is the process of applying controlled amounts of water to plants at required intervals of time. In many parts of the world, the moisture available in the root zone from rainfall or groundwater is insufficient to meet the plant's water requirement. This lack of available moisture within the soil may be either for the entire crop season as in arid or semi-arid countries (in summer) or for only part of the crop season (in winter for arid or semi-arid countries). Therefore, to achieve optimum growth during the plant growth stages, it is necessary to compensate for the decrease in soil moisture through the irrigation process. The irrigation refers to the process of artificial irrigation to complement the shortage in the moisture content of the root zone for the purpose of agricultural production.

Irrigation systems are often designed to increase efficiency, reduce labor requirements, and increase agricultural yield. The greatest value of water is consumed by irrigation; as large quantities of fresh water are consumed in agriculture; this situation is concentrated in developing countries where irrigation methods are not sufficiently developed. An irrigation system must be designed and installed to efficiently deliver the required amount of water at the appropriate time to the plants. The irrigation system should be operated in such a way that it reduces the amount of surface runoff, the wasted quantities of water by evaporation and deep percolation to ensure optimal consumption of the quantities used for irrigation. This involves adopting strategies such as scheduling irrigation based on crop growth stages, monitoring soil moisture levels, using appropriate application methods (surface, sprinkler, or drip), and maintaining the system in good condition. By doing so, farmers can optimize water use efficiency, reduce operational costs, conserve natural resources, and contribute to the sustainability of agricultural production.

1.2 Importance of Irrigation

The importance of irrigation is due to the conditions in which the amount of rainfall is little or no available to meet the water requirement of the plant. Under these conditions, it is considered an absolute necessity in arid or semi-arid regions. Appropriate irrigation systems can ensure uninterrupted cultivation, especially in areas that suffer from irregular or low rainfall. In addition to the above, there is another importance of the irrigation process as follows:

1. Irrigation cools the soil and plants to provide a suitable environment.
2. Irrigation removes the harmful salts present in the root zone.

3. Irrigation reduces soil erosion.
4. Irrigation improves groundwater storage.
5. Irrigation helps in the process of fertilization and agricultural pest control.

1.3 Irrigation System

An irrigation system can be expressed as a network of channels and hydraulic structures in addition to some service facilities, the purpose of which is to irrigate the land and provide adequate amounts of irrigation water for plants. The irrigation system consists as follows

1. Main intake structure
2. Pumping station
3. Conveyance and distribution system
4. Field application system
5. Drainage system

1.3.1 Main Intake Structure

The intake structure is used at the entrance to the irrigation system for the purpose of directing water from the original water source such as (lake, river, reservoir, etc.) to the irrigation system. The main intake structure consists from following main elements:
1. Diversion structure, this structure is used to control the water level in the stream and to ensure that it is sufficient to provide the intake;
2. Inlet level control is used to control the water supply to the channels;
3. Entrance protection, piles and coarse bars are used to protect the intake from scouring.

Many methods are used to protect the intake structure, gabions can be used around the intake and to deflect the direction of the water, especially when the flow is large and may lead to the erosion of the stream bank (Fig. 1.1), large stones or rocks may be used for this purpose (Fig. 1.2)

1.3.2 Pumping Station

In some cases, the level of the irrigation water source is lower than the level of the irrigated fields. Then a pump must be used to supply water to the irrigation system (Fig. 1.3). A pump is a device that converts mechanical energy into hydraulic energy. It raises the water from a low level to a higher level and delivers it at a high pressure. Pumps can be classified according to principle of operation (displacement pumps, velocity pumps, bouncy pumps, and impulse pumps). The work performed by the pump can be calculated by multiplying the weight of the liquid pumped per unit time by the total head in meters.
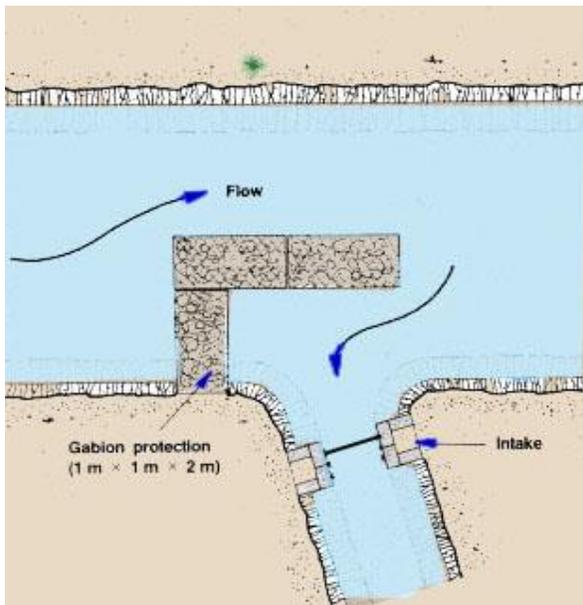
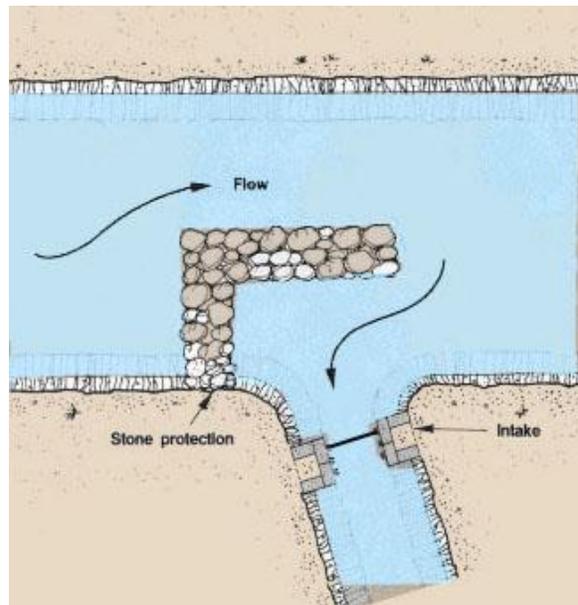**Fig. 1.1** Main intake structure protected by gabions

Fig. 1.2 Main intake structure protected by stone

The input power (P) of a pump is the mechanical power which is measured in Watt (W) or kilowatt (kW). Break horse power (BHP) is used to express the input power. Usually the term input power is defined as the power delivered to the shift of pump, where pump output power is named as hydraulic power (P) which indicates as useful work delivered by pump. The hydraulic power of the pump is determined by the following formula

$$P = 9.81\rho QH \tag{1.1}$$

Where

P : Power transmitted to the liquid by the pump (W)
ρ : The density of liquid (kg/m^3)
Q : The flow rate (m^3/sec)
H : The total pumping head (m)

Efficiency of the pump is the ratio of pump output power to pump input power

$$Pump\ Efficiency = \frac{pump\ output\ power}{pump\ input\ power} \times 100 \tag{1.2}$$

Fig. 1.3 Pumping station used to supply water to the agricultural area

Example 1.1
Calculate the hydraulic power for pump to be used for irrigation at flow rate (0.75 m³/s), assume the required total head is equal to (4m).

Solution
The hydraulic power of the pump
$P = 9.81\rho Q h$
$P = 9.81 \times 1000 \times 0.75 \times 4 = 29.43\ kW$

If the efficiency of the electric motor is equal to E_m, the electric motor capacity is calculated from the following equation

$$P_m = (9.81\rho Q h)/E_m \tag{1.3}$$

Where
P_m : The electric motor capacity (W)

Watts can be converted to horsepower (hp) using the following relation

1 kW = 1.341 hp

By using the information in the example (1.1), assume that the efficiency of the electric motor is (80%), the horsepower of an electric motor can be calculated as follows:

$$P_m = \frac{9.81\rho Qh}{E_m} = \frac{29.43}{0.8} = 36.79 \; kW$$

The horsepower of electric motor $= 36.79 \times 1.341 = 49.34 \; hp$

The required energy of discharging water (water horsepower, WHP) by using ($\rho = \frac{1000kg}{m^3}, g = \frac{9.81m}{s^2}, 1 \; m^3 = 1000\ell$) can be determined as follows:

$$WHP = \left[Q\left(\frac{\ell}{s}\right) \times H(m)\right]/76 \tag{1.4}$$

Example 1.2
In an agricultural field of 90 hectares, the time period between two consecutive irrigations at peak period is 10 days, the total depth of irrigation water is equal to 100 mm. If the required total head for pumping is 20 m, pump efficiency (E_p) is 80%, electric motor efficiency (E_m) is 80%, and total operation period for pumping is equal to 10 hr/day, determine:
1. The hydraulic power of pump
2. The electric motor capacity

Solution

1 hectare $= 10000 \; m^2$, total area $= 90 \times 10000 = 900{,}000 \; m^2$

Total volume of irrigation water = total irrigated area (900,000) × total depth of irrigation water (0.1) $= 90{,}000 \; m^3$

The total period of pumping during 10 days based on maximum allowable operation period per day (10hr/day) $= 10 \times 10 \times 3{,}600 = 360{,}000$ sec

Flow rate capacity of pump $= \frac{Q}{t} = \frac{90000}{360000} = 0.25 \; m^3/sec$

1. The hydraulic power of pump

$P = (9.81\rho Qh)/E_p = (9.81 \times 1000 \times 0.25 \times 20)/E_p = 61.31 \; kW$

1. The electric motor capacity

The electric motor capacity $= \frac{P}{E_m} = \frac{61.31kW}{0.8} = 76.64 \; kW$

The above answer can be calculated using horsepower

The horsepower of electric motor $= 76.64 \times 1.341 = 102.77 \; hp$

1.3.3   Conveyance and Distribution Systems

The distribution network for the irrigation system consists of the main canal, inter-farm, farm, and on-farm distribution canals. The main canal delivers the irrigation water from water source (river, lake, reservoir, well etc.) to inter-farm distributor that is deliver water to individual farms. On-farm is transforming the water to the irrigated sites.

Irrigation canals are arranged so that the following are provided with the lowest construction and operating costs: water supply of the required volume at the right time; canal efficiency with highest ratio (ratio of the flow rate at the tail of the canal to that at the head of the canal); control of main canal (with an attempt to raise the water level at the upstream of the canal compared to the downstream to ensure the flow by gravity. The mathematical formula of the conveyance canal efficiency (E_c) is represented as follow:

$$E_c = \frac{V_f}{V_d} \times 100 \tag{1.5}$$

Where

V_f : The volume of water at the tail of the canal or that reached the farm (m^3)

V_d : The volume of water at the head of the canal or that delivered from the source (m^3)

1.3.4   Distribution Control Systems

Distribution control structures are required for easy and accurate water distribution within the irrigation system and on the farm (Fig. 1.4). The distribution of irrigation water means that the flow in the channel is evenly divided between two or more smaller channels. The division is generally made in proportion to the irrigated area served by each channel, so that every command area receives its fair share of water. Proper distribution control improves system efficiency and reduces disputes among water users.

There are three techniques that can be installed on the stream in the main irrigation channel regarding the distribution of the flow to the secondary channels of the system: proportional distribution; rotational distribution or delivery on demand. Different techniques of control systems for water distribution require different structures.

Fig. 1.4 Distribution control structures

1.3.4.1 Proportional Distribution

The total flow in the channel is divided into the secondary channels. These portions correspond to the portion of the total area that is irrigated by that channel. The following example illustrates this division mechanism.

Example 1.3

What is the discharge that is given to the secondary channels under the following conditions?

- The total discharge of the main channel is equal to (180 ℓ/sec)
- The total discharge is divided into three secondary channels depending on the area irrigated by each secondary channel
- The command areas of the secondary channels are 80 ha, 60 ha, and 40 ha.
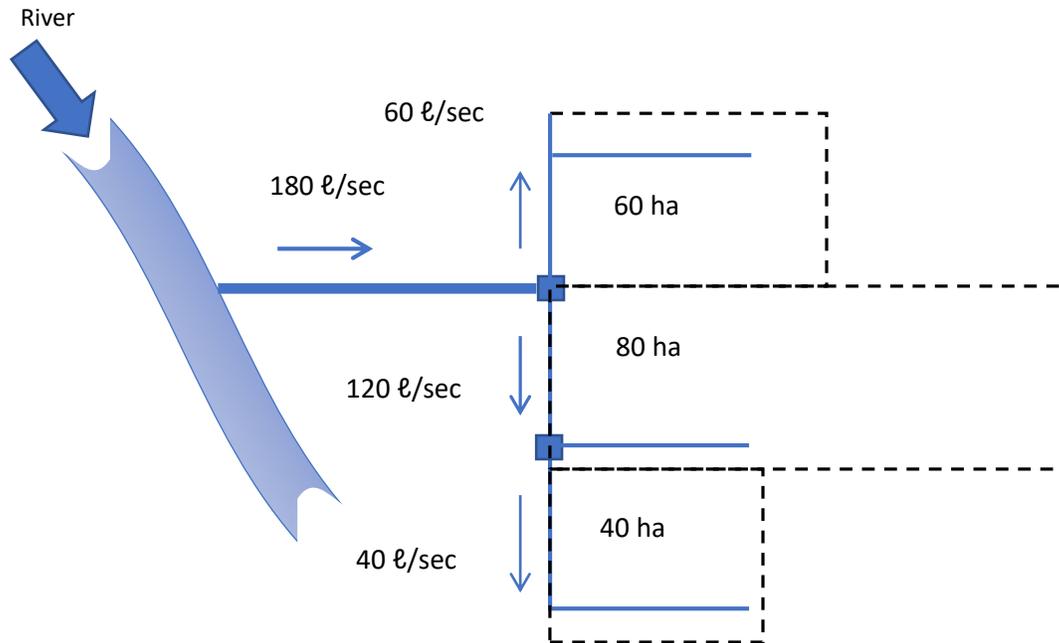
Solution

The total area commanded by the secondary channels is

$80 + 60 + 40 = 180$ ha

The share of the first channel is equal to $\left(\frac{80}{180}\right) \times 180 = 80 \; \ell/sec$

The share of the second channel is equal to $\left(\frac{60}{180}\right) \times 180 = 60 \; \ell/sec$

The share of the third channel is equal to $\left(\frac{40}{180}\right) \times 180 = 40 \; \ell/sec$

River

60 ℓ/sec

180 ℓ/sec

60 ha

80 ha

120 ℓ/sec

40 ha

40 ℓ/sec

1.3.4.2 Rational Distribution

The principle of rotational irrigation water distribution means that the entire flow of the main channel is diverted into each branch channel in succession. For instance, in the case of a main canal and its secondary channels, each secondary channel remains without water for part of the time; however, when supplied, it conveys the entire discharge of the main canal. The same principle applies to the distribution of flow from secondary channels into tertiary channels, and further within the tertiary channels themselves. Under this rotational method, the duration for which a canal carries irrigation water is proportional to the command area it serves, as illustrated in the following example.

Example 1.4

What is the period of time during which the water flows in the secondary canals if the total period of water flows in the main channel is nine days according to the conditions illustrated in Example 1.3?

Solution

The total area commanded by the secondary channels is

$80 + 60 + 40 = 180$ ha

Each secondary channel will receive the whole flow during a period of 9 days that is proportional to its command area

The period for the first channel is equal to $\left(\frac{80}{180}\right) \times 9 = 4\ days$

The period for the second channel is equal to $\left(\frac{60}{180}\right) \times 9 = 3\ days$

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The period for the first channel is equal to $\left(\frac{40}{180}\right) \times 9 = 2 \; days$

Note:
In this example, the portions of the period in which a secondary channel carries the entire flow have been computed. The first secondary channel carries the flow for 4 days every 9 days, but this does not mean that the irrigation period in the fields is 9 days. This period depends on soil type, yield, growth stage, and evapotranspiration.
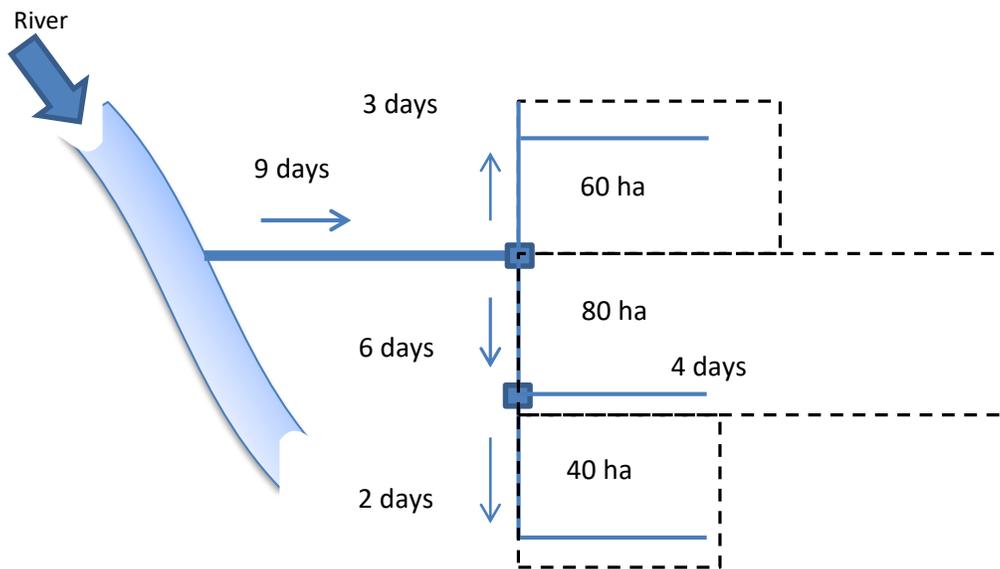
1.3.4.3 Delivery on Demand

In addition to the delivery of water on the basis of regions, as in the case of proportional or rotational supply, the pumping of water into irrigation channels can be depending on the demands of the farmers. In such a delivery system, water is only directed to those channels in the event that farmers demand water.

1.3.5   Field Application Systems

The field application system in the irrigation system helps to transport water within the fields. There are many ways to apply water in the field. The simplest one consists of bringing water from a source of supply, such as a well, to each plant for example using a bucket. There are three basic methods of field irrigation:

   a. Surface irrigation
   b. Sprinkler irrigation
   c. Drip irrigation

1.3.5.1 Surface Irrigation

Surface irrigation is mainly used to irrigate field crops. In these methods, gravity is used to supply water from the irrigation source to the crop with the help of channels, pipes, etc. Basically the efficiency of this irrigation method depends on the type of soil, land grading, type of crop, etc. Among all irrigation methods, water loss due to deep percolation and surface runoff is highest in this method. Therefore, it is said to be the least effective irrigation method compared to other methods. There are three types of the surface irrigation methods:

1. Border Irrigation

This method is considered one of the oldest methods used in surface irrigation. The field is divided into blocks, and boundaries of border are created with the help of earth. In this method, water flows between embankments that divide the sloping field into rectangular strips with free drainage at their end. The primary purpose of dikes is to contain water as it flows through the field. This method is more suitable for fields with slopes compared to the basin method.

2. Furrow Irrigation

The field is irrigated in this way using furrows. It can be operated with less technical knowledge compared with other techniques such as sprinkler or drip methods need more technical knowledge. The Furrow method also requires less capital investment. More surface runoff occurs with this method, so it is difficult to regulate the water flow.

3. Basin Irrigation

In this surface irrigation method, water is applied to crops within prepared basins that are interconnected by small channels. After one basin is irrigated, the water flows sequentially into the next.

1.3.5.2 Sprinkler irrigation

This method of irrigation is similar to natural rain. Pumps are used to transfer water through the pipes of the sprinkler irrigation network, as this water comes out through small holes in the form of small droplets. Small droplets sprayed into the air through sprinklers so that it breaks up into tiny water droplets that fall on the ground. The pump supply system, sprinklers and operating conditions must be designed to enable uniform application of water.

1.3.5.3 Drip irrigation

This irrigation system mainly supplies the crop with water according to the consumptive use. Drip irrigation is a system with high water efficiency among all irrigation methods. This method is considered the most widely used and suitable method in arid regions of the

world, arid regions suffer from limited water resources. It is necessary to supply the water with the pressure water lift pump. After lifting, the water passes through the water cyclone filter, sand filter; fertilizer mixing tank and screen filter at last through the drip. All these filters are necessary for a smooth flow of water through drip; otherwise, pipelines may tend to become clogged.

1.4 Root Zone

The root zone is defined as the volume of soil occupied by plant roots, as the plant can extract water through this zone. The depth of this zone (d_R) varies according to soil type, type (Table 1.1) and age of the plant. An increase in the amount of water in this zone as well as a decrease leads to a deterioration in crop growth and reduces crop yields.

Table 1.1 Typical rooting depths for selected crops (FAO 1977)

Crop	Rooting depth (m)	Crop	Rooting depth (m)
Beans	0.5-0.7	Maize	1.0-1.7
Beets	0.6-1.0	Onion	0.3-0.5
Cabbage	0.4-0.5	Potatoes	0.4-0.6
Citrus	1.2-1.5	Sorghum	1.0-2.0
Cotton	1.0-1.7	Soybeans	0.6-1.3
Grapes	1.0-2.0	Sugarbeet	0.7-1.2
Grass	0.5-1.5	Tobacco	0.5-1.0
Groundnuts	0.5-1.0	Vegetables	0.3-0.6
Lucerne	1.0-2.0	Wheat	1.0-1.5

1.5 Leaching Requirements (LR)

More salt is added with the applied irrigation water during the irrigation process. The added salt must be leached from the root zone before the concentration affects crop yield. Leaching is done by using enough water so that a portion seeps through and under the entire root zone and carries with it a portion of the accumulated salts. Leaching requirement is expressed mathematically by using the following equation

$$LR\% = \frac{d_d}{d_i} \times 100 \tag{1.6}$$

Where

d_d : The depth of drainage water (mm)
d_i : The depth of irrigation water (mm)

By using mass balance of salt

$$V_1 C_1 = V_2 C_2 = \cdots \tag{1.7}$$

Where

$V_1 V_2$: The volumes of water containing salt for state 1 & 2 respectively

$C_1 C_2$: The concentrations of the salt in the water

The above equation can be written as

$$\frac{V_1}{V_2} = \frac{C_2}{C_1}$$

With a fixed surface area, the above equation can be written as

$$\frac{D_1}{D_2} = \frac{C_2}{C_1}$$

Where

$D_1 D_2$: The depths of water containing salt for state 1 & 2 respectively

The above equation can be expressed in terms of electrical conductivity

$$\frac{D_1}{D_2} = \frac{EC_2}{EC_1}$$

The total (gross) depth of irrigation is consisted from net irrigation depth (d_m) plus leaching or drainage water depth (d_d)

$$d_i = d_m + d_d \tag{1.8}$$

Where

d_m : The depth of water lost to consumptive use by crops.

Equation (1.6) can be written as

$$LR\% = \frac{d_i - d_m}{d_i} \times 100 = \left(1 - \frac{d_m}{d_i}\right) \times 100 \tag{1.9}$$

The efficiency of the field water application (E_a) is expressed by the following equation

$$E_a = \frac{d_m}{d_i} \tag{1.10}$$

Equation (1.9) becomes

$$LR\% = (1 - E_a) \times 100$$

Or, can be expressed as (Ayres and Westcot 1985)

$$d_i = \frac{d_m}{E_a} = \frac{d_m}{1-LR} \tag{1.11}$$

The LR can be determined according to the salinity of applied water using the following equation (Rhoades 1974):

$$LR = \frac{EC_i}{5EC_t - EC_i} \tag{1.12}$$

Where
  EC_i : The salinity of applied irrigation water,
  EC_t : The average soil salinity tolerated by the crop.

  EC_t : The average soil salinity tolerated by the crop.

Example 1.5

The date palm is planted in a loam soil; the water used for irrigation from a river has an electrical conductivity equals 1.2 dS/m (deciSiemens per meter). The depth of water lost to consumptive use by crops equals 800 mm / season. How much additional water should be used for leaching?

Solution
$$LR = \frac{EC_i}{5EC_t - EC_i}$$

$EC_i = 1.2$ dS/m
$EC_t = 4.0$ dS/m at a 100 percent yield potential (Table 1.2)
$EC_t = 6.8$ dS/m at a 90 percent yield potential (Table 1.2)

$$LR = \frac{1.2}{5(4)-1.2} = 0.064 \text{ (for a 100 percent yield potential)}$$
$$LR = \frac{1.2}{5(6.8)-1.2} = 0.037 \text{ (for a 90 percent yield potential)}$$

The actual amount of water to be applied to supply both water lost to consumptive use and leaching

$$d_i = \frac{d_m}{1-LR} = \frac{800}{(1-0.064)} = 855 \ mm/season \text{ (for a 100 percent yield potential)}$$

$$d_i = \frac{d_m}{1-LR} = \frac{800}{(1-0.037)} = 831 \; mm/season \text{ (for a 90 percent yield potential)}$$

Table 1.2 shows crop tolerance and yield potential of selected crops as influenced by irrigation water salinity EC_i (dS/m) or soil salinity EC_t (dS/m)

Table 1.2 Percent yield potential of selected crops as influenced by EC_i or EC_t (Maas and Grattan 1999, Grattan 2002)

Crops	100%		90%		75%		50%		0%	
	EC_t	EC_i	EC_t	EC_i	EC_t	EC_i	EC_t	EC_i	EC_t	EC_i
Barley	8.0	5.3	10	6.7	13	8.7	18	12	28	19
Cotton	7.7	5.1	9.6	6.4	13	8.4	17	12	27	18
Sugarbeet	7.0	4.7	8.7	5.8	11	7.5	15	10	24	16
Sorghum	6.8	4.5	7.4	5.0	8.4	5.6	9.9	6.7	13	8.7
Wheat	6.0	4.0	7.4	4.9	9.5	6.3	13	8.7	20	13
Date palm	4.0	2.7	6.8	4.5	11	7.3	18	12	32	21
Grapefruit	1.8	1.2	2.4	1.6	3.4	2.2	4.9	3.3	8.0	5.4
Orange	1.7	1.1	2.3	1.6	3.3	2.2	4.8	3.2	8.0	5.3
Peach	1.7	1.1	2.2	1.5	2.9	1.9	4.1	2.7	6.5	4.3
Apricot	1.6	1.1	2.0	1.3	2.6	1.8	3.7	2.5	5.8	3.8
Maize	1.7	1.1	2.5	1.7	3.8	2.5	5.9	3.9	10	6.7

1.6   Soils

Soil is a complex mixture of minerals, water, air, organic matter, and many organisms. It is formed the earth's surface crust; soils are able to support plant life. Soil provides a structural base for plants and allows roots to spread through the soil structure to gain stability, supplies plants with nutrients and aids in aeration. Both soil and water are essential for plant growth. Soil pores within the root zone retain moisture which adheres to soil particles that are partially or completely filled with beneficial nutrients dissolved in water, essential for plant growth. Most plant roots also require oxygen to respiration; therefore, the fully saturation of the soil pores restricts the growth of the roots of these plants.

The basis of the irrigation process is the timely supply of water to the root zone of the plant to obtain an ideal crop, studying the relationship between soil pores, its ability to retain water and the rate of water absorption by the plant is fundamentally important. It is considered one of the essentials for an irrigation engineer who is considering developing a command area through a scientifically designed irrigation system.

For a given volume of soil (V) (Fig. 1.5), the parameters in the figure are explained as follows:
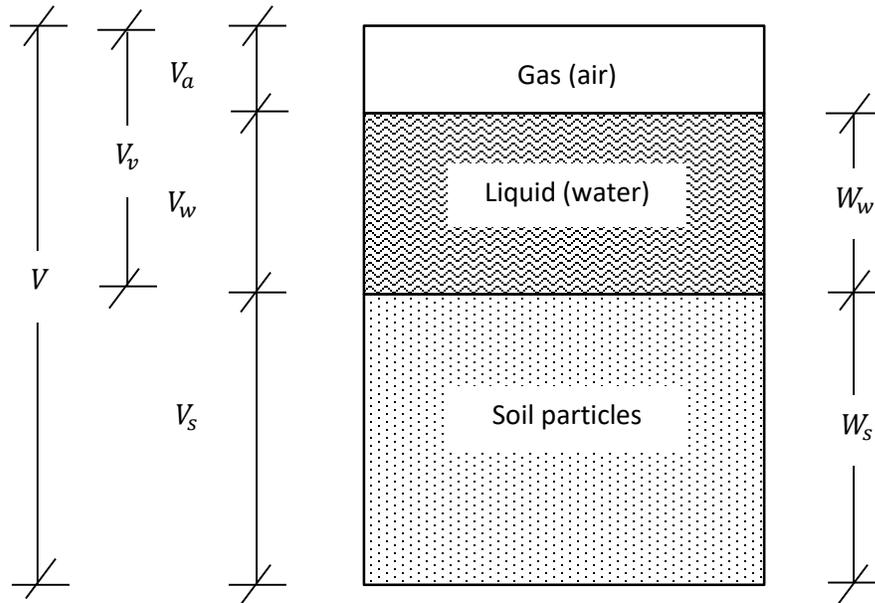

Fig. 1.5 Phase diagram

V_s : Volume of solid
V_w : Volume of liquid (water)
V_a : Volume of gas (air)
V_v : Volume of voids, $V_v = V_w + V_a$
If a fully saturated soil sample is present ($V_a = 0, \quad V_v = V_w$)
In the case of completely dry soil ($V_w = 0, \quad V_v = V_a$)
The air weight is zero compared to water weight and soil grains.
The void ratio (e) can be defined as:

$$e = \frac{V_v}{V_s}$$ (1.13)

The porosity (n)

$$n = \frac{V_v}{V}$$ (1.14)

While the volumetric moisture content (w) and the saturation (S) are determined by the following equations:

$$w = \frac{V_w}{V}, \ S = \frac{V_w}{V_v}$$ (1.15)

It is observed through the above equations that the porosity (n) is always less than 1.0. But the value of the void ratio (e) may be less, equal or greater than 1.0
If the water weight in a wet soil sample is (W_w) and the dry weight of the sample is (W_s), then the dry weight moisture fraction (W) will be expressed as:

$$W = \frac{W_w}{W_s} \qquad (1.16)$$

Soil porosity represents the distance between soil particles, as these spaces contain different amounts of water and air. Soil porosity is affected by the soil texture and structure. For example, fine soils have smaller pores but are more numerous pores than coarse soils. Coarse soil contains larger particles than fine soil, but it has less porosity. Water is held tighter in small pores than large pores, so fine soils can hold more water than coarse soils. Bulk density (γ_b), also called bulk specific weight or the bulk unit weight, it is defined as the total weight of the soil (including water) per unit bulk volume.

$$\gamma_b = \frac{W_T}{V} \qquad (1.17)$$

$$W_T = W_s + W_w$$

The unit weight of the solid particles is the ratio of dry weight of the soil particles to the volume of soil particles

$\frac{W_s}{V_s}$, thus; the bulk specific gravity of the solid soil particles (G_b) is given by:

$$G_b = \frac{W_s}{\gamma_w V} \qquad (1.18)$$

The relative density of the soil grain (G_s) is presented by:

$$G_s = \frac{W_s}{\gamma_w V_s} \qquad (1.19)$$

Where, γ_w is the unit weight of water

By dividing Eq. (1.18) by Eq. (1.19), yield:

$$\frac{G_b}{G_s} = \frac{V_s}{V}$$

$$1 - n = 1 - \frac{V_v}{V} = \frac{V - V_v}{V} = \frac{V_s}{V} = \frac{G_b}{G_s}$$

$$\therefore G_b = (1 - n)G_s \qquad (1.20)$$

From the volumetric moisture content (w) equation:

$$w = \frac{V_w}{V} = \frac{\frac{W_w}{\gamma_w}}{\frac{W_s}{G_b \gamma_w}} = G_b \frac{W_w}{W_s}$$

By using the dry weight moisture fraction (W), yield:

$$w = G_b W \qquad (1.21)$$

Substituting equation (1.20) into equation (1.21), yield:

$$w = (1 - n)G_s W \qquad (1.22)$$

For a field, the depth of its root zone is equal to d_R and its area is equal to A

Bulk volume = $A d_R$

$$W_s = V_s G_s \gamma_w = V(1 - n)G_s \gamma_w = A d_R (1 - n)G_s \gamma_w$$

The dry weight moisture fraction (W)

$$W = \frac{V_w \gamma_w}{A d_R (1-n)G_s \gamma_w} \qquad (1.23)$$

The volume of water in the root zone is obtained by:

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$$V_w = WAd_R(1 - n)G_s \tag{1.24}$$

Assume that the water is spread evenly over the area (A), the depth of water (d_w) is calculated as follows:

$$d_w = \frac{V_w}{A}$$

$$d_w = Wd_R(1 - n)G_s \tag{1.25}$$

Substituting equation (1.22) into equation (1.25), yield:

$$d_w = wd_R \tag{1.26}$$

Field capacity is determined in terms of moisture fraction

$$W_{fc} = \frac{W_w}{W_s}$$

The volumetric moisture content at the field capacity (w_{fc}) is equal to $(G_b W_{fc})$. The volumetric moisture content at the permanent wilting point (w_{pwp}) is measured by $(G_b W_{pwp})$, W_{pwp} is the dry weight moisture fraction of the permanent wilting point.

The total available moisture d_m (in terms of depth) for a plant is determined by

$$d_m = (w_{fc} - w_{pwp})d_R \tag{1.27}$$

The volumetric moisture content at the available moisture (w_{am}) is equal to $(w_{fc} - w_{pwp})$, then

$$d_m = w_{am}d_R$$

Where

$\quad\quad d_m$: The depth of water in the soil available for plant growth.

Example 1.6

If the depth of the root zone of an orange tree is equal to 1.5 meters. Soil contains 60% solid particles and 40% voids. The soil was well drained after several days of wetting, voids volume contains water (50%) and 50% retains air. At the permanent wilting point, the volume of void is 25% of water. What is the depth of water in the soil available for plant growth?

Solution

Field capacity = voids volume contains water (50%) after several days of wetting × the percentage of voids in the soil

$FC = 0.5 \times 0.4 = 0.2 = 20\%$

Permanent wilting point $= 0.4 \times 0.25 = 0.1 = 10\%$

$w_{am} = w_{fc} - w_{pwp} = 0.2 - 0.1 = 0.1$

$d_m = w_{am}d_R = 0.1 \times 1500 = 150 \; mm$

Depletion is the depth of water that is consumed by a plant (after the soil has reached field capacity by gravity drainage) by evapotranspiration. At the field capacity, percentage depletion is 0% and 100% at permanent wilting point. The depletion and the percentage of depletion are determined as follows

Depletion $= (w_{fc} - w)d_R$

Percentage of depletion$(d_p)\% = \dfrac{w_{fc}-w}{w_{fc}-w_{pwp}} \times 100$ $\qquad\qquad\qquad$ (1.28)

1.7 Irrigation Interval (II)

The irrigation interval refers to the time period between two successive irrigations. Irrigation is mostly done after depletion (40%-60%) of the water depth in the soil available for plant growth. The interval of irrigation is chosen so that the crop does not suffer from water stress. Traditionally, 'plant water deficit' or 'plant water stress' has been defined as being when plant water status is reduced sufficiently to affect normal plant functioning. When plants suffer from water deficit (WD), a plethora of negative physiological alterations such as cell turgor loss, reduction of CO_2 net assimilation rate, oxidative stress damage, and nutritional imbalances, among others, can lead to a decrease in the yield production and loss of commercial quality (Nieves-Cordones 2019).

$$Irrigation\ Interval\ (II) = \frac{(w_{fc}-w)d_R}{rate\ of\ consumptive\ use} = \frac{d_p \times d_m}{d_c} \qquad\qquad (1.29)$$

Consumptive use is defined as the amount of water needed to meet the water requirement through evapotranspiration.

Example 1.7
The area of a field is (1acre), the type of its soil (loam soil), it is cultivated with a crop that has daily water consumption (20 mm/day). This plant needs a root zone (1m). Volumetric water content at the field capacity and permanent wilting point are 30.8% and 14%, respectively. Determine the following: -

1. Depth of water lost to consumptive use.
2. Depth of water lost to consumptive use by crops with leaching requirement (50%).
3. Pumping period for pump that has discharge equals 400 m³/hr.
4. II
5. II with percentage of depletion (50%).

Solution
1. $d_m = (w_{fc} - w_{pwp})d_R = (30.8\% - 14\%) \times 1000mm = 168\ mm$

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2. $di = \dfrac{dm}{1-LR} = \dfrac{168}{1-0.5} = 336\ mm$

3. $Time\ (T) \times Discharge\ of\ pump\ (Q) =$
   $Total\ gross\ depth\ of\ irrigation\ (d_i) \times Area\ of\ the\ field\ (A),\ \ 1\ acre =$
   $4046m^2$

   $T \times 400 = 0.336 \times 4046 \rightarrow T = 3.4\ hours$

4. II with percentage of depletion (100%), $depth\ of\ depletion = \ d_m$

   $$II = \frac{d_m}{d_c} = \frac{168}{20} = 8.4\ days \qquad \therefore use\ II = 8\ days,$$

   $$percentage\ of\ depletion\% = \frac{8}{8.4} = 95\%$$

   Or, total consumption during eight days = $8 \times 20 = 160\ mm$

   $$Percentage\ of\ depltion\% = \frac{depltion\ depth}{d_m} = \frac{160}{168} \times 100 = 95\%$$

5. $depth\ of\ depletion = w_{am} \times d_R \times d_p = 0.168 \times 1000 \times 0.5 = 84\ mm$

   $$II = \frac{d_p \times\ d_m}{d_c} = \frac{84}{20} = 4.2\ days = 4.0\ days\ (say)$$

1.8   Efficiency, Adequacy, and Uniformity of Irrigation

Field Application Efficiency (E_a) is fundamentally dependent on the irrigation method and the level of farmer discipline. Some guideline values for average field application efficiency (E_a) are shown in Table 1.3. The lack of discipline of the farmers may lead to lower values in this table. It is influenced, for example, by the supply rate, soil infiltration rate, storage capacity of the root zone, land grading, etc. For border and furrow surface irrigation, water is mostly lost through deep percolation at the head end and through runoff at the tail end, while for basin irrigation; water is mostly lost through evaporation and deep percolation, since the basin is closed. It can be defined as the ratio of the average depth of irrigation water that is used beneficially to the average depth of applied irrigation water.

$$E_a = \frac{average\ depth\ of\ irrigation\ water\ used\ beneficially}{average\ depth\ of\ applied\ irrigation\ water} \qquad\qquad (1.30)$$

Table 1.3 Indicative values of the E_a

Irrigation method	E_a
Surface irrigation (border, furrow, basin)	60%
Sprinkler irrigation	75%
Drip irrigation	90%

The loss of water through drainage beyond the root zone is reflected in the deep percolation ratio, DPR, defined as:

$$DPR = \frac{deep\ percolation\ volume}{applied\ water\ volume} \qquad (1.31)$$

Losses from the irrigation system via runoff from the end of the field are indicated in the tail water ratio, TWR:

$$TWR = \frac{runoff\ volume}{applied\ water\ volume} \qquad (1.32)$$

Irrigation adequacy can be defined as the percentage of a field that receives the desired amount of water. In arid and semi-arid regions with low rainfall, the irrigation system is used to fill the soil profile to field capacity or to a lesser planned level. In sub-humid and humid areas, this may be less than field capacity to provide storage for rain that may occur between irrigation. The choice of the amount of water to be applied may also be a function of the crop's sensitivity to water stress, its market value, and the water supply. Irrigation adequacy can easily be assessed by plotting the depth of distribution of infiltrated irrigation water along the field as shown in Fig. 1.6. The point at which the curve intersects the line for the required depth of application indicates the percentage of the field being adequately irrigated. Deep water percolation carries chemical materials below the root zone to the groundwater storage, where it is contributing to groundwater pollution. Both insufficient and excessive irrigation can reduce crop yield and quality reduction.

Application uniformity concerns the distribution of water over the actual field. Flow rate in downstream decreases as infiltration occurs in surface irrigation systems. The infiltration opportunity time and water volume for a given surface area decrease somewhat from the inflow to the outflow end of the field, as the water movement along the furrow or border is directly related to the available stream size. Actual uniformity varies depending on whether the system is stationary, periodically moving, or continuously moving and the associated sprinkler nozzle discharge pattern in sprinkler irrigation systems. Systems are typically uniformity in the absence of wind and extremely high temperatures, properly designed, maintained and managed. A number of technical sources suggest the Christiansen's coefficient (CU) as a measure of uniformity, CU is calculated as:

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$$CU = 100 \left(1 - \Sigma_{i=1}^{n} \frac{|\bar{x} - x_i|}{n\bar{x}}\right)$$
(1.33)

Where

x_i : The depth of observation i,

$\bar{x}$: The mean depth for all observations,
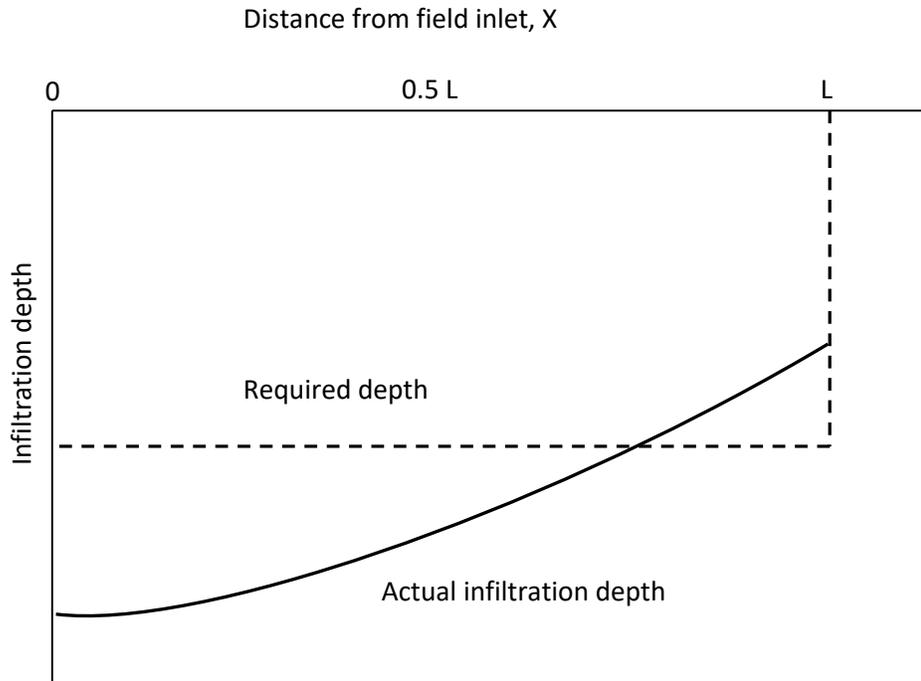
n : The number of observations.

Fig. 1.6 Distribution of applied water along a surface irrigated field

1.9   Water Infiltration into Soil

Infiltration rate $I(t), [L/T]$ can be defined as the rate at which water infiltrates into the soil through the soil surface (Fig. 1.7, dashed line), the maximum volume of water which can enter the soil per unit area is known as the cumulative infiltration, $D(t), [L]$ (Fig. 1.7, solid line). Over time, as the water redistributes through the soil profile, it displaces the air and fills the pores resulting in more resistance to flow, thus reducing the hydraulic gradient and the ability to infiltrate. Indeed, the infiltration capacity at the beginning of the process (when the soil is not saturated) represents the maximum values, while, as the soil saturates, it decreases towards a constant approximate value. This constant rate of infiltration is also termed the basic infiltration rate and is approximately equal to the permeability of the

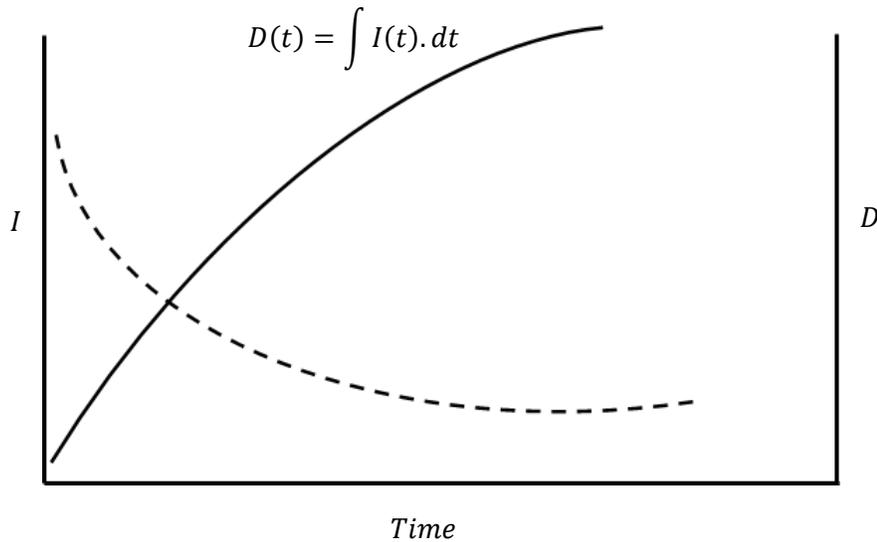saturated soil. It should be noted that the cumulative infiltration, $D(t)$ is the time- integral of $I(t)$.

$$D(t) = \int I(t).dt$$

I

D

Time

Fig. 1.7 Infiltration rate and cumulative infiltration curves

One popular infiltration equation is the Kostiakov equation.

$$D = c.t^m \tag{1.34}$$

Where

D : The depth of infiltration (mm),

t : The time (min),

$c, and\ m$ are the constants.

Kostiakov equation is an empirical equation which assumes that the infiltration rate declines over time according to a power function.

$$I = \frac{dD}{dt} = c.m.t^{m-1} = k.t^n \tag{1.35}$$

Where

$k, and\ n$ are the constants.

The Kostiakov equation can be evaluated by graphically using log-log paper, or analytically with regression techniques. In the conventional graphical method, the parameters in the Kostiakov model are estimated by fitting a straight line to an arithmetical plot of log (D) against log (t) (Fig. 1.8)
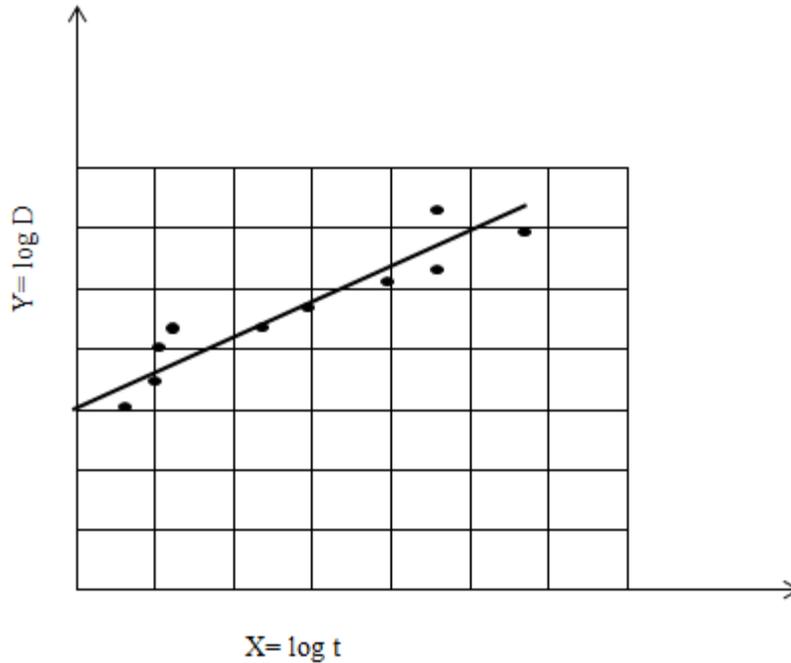
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Fig. 1.8 Arithmetical plot of log (D) against log (t)

The intercept of the best fit straight line on the ordinate axis will then represent the value of (a) and its slope will be equal to (b).

1. $Y = a + b.X$
2. $logD = a + b.logt$
3. $logD = log10^a + logt^b$
4. $logD = \log(10^a.t^b) \rightarrow D = 10^a.t^b$

Linear regression attempts to model the relationship between two variables by fitting a linear equation to observed data. The most common method for fitting a regression line is the method of least-squares. This method calculates the best-fitting line for the observed data by minimizing the sum of the squares of the vertical deviations from each data point to the line (if a point lies on the fitted line exactly, then its vertical deviation is 0). Because the deviations are first squared, then summed, there are no cancellations between positive and negative values.

$$b = \frac{n \sum X.Y - [\sum X. \sum Y]}{n \sum X^2 - [\sum X]^2}$$

$a = \frac{1}{n}[\sum Y - b.\sum X]$

$Y = \log D, \quad X = logt, n \text{ is the number of observations}$

The main limitation of the Kostiakov equation is the zero final intake rate after a period of time. In most cases, the infiltration rate approaches instead to a finite constant value, which in some cases may occur after short periods of time. The Kostiakov-Lewis variable, also known as the "modified Kostiakov" equation, corrects this by adding a steady intake term to the original equation:

$$I = h.t^n + P \tag{1.36}$$

Where

$h, and\ n$: The empirical coefficients,

P : The approximate, but does not necessarily equate to the final infiltration rate of the soil.

When integrating form, the cumulative infiltration is expressed as:

$$D = f.t^m + P.t \tag{1.37}$$

Where

$f, and\ m$: The empirical coefficients.

The Natural Resource Conservation Service (NRCS) classifies soils into families according to intake. The NRCS equation for infiltration is a modified form of Kostyakov's equation.

$$D = a.t^b + c \tag{1.38}$$

Where

D : The cumulative depth of infiltration (cm),

t : The time (min),

$a, b\ and\ c$: The constants.

The term (c) represents primary infiltration into cracks and wormholes. This type of intake doesn't take time so it doesn't change over time. It is assumed by NRCS that c is the same for all intake families. However, c can be much larger for some soils. For example, the largest amount of infiltration into swelling clay soil occurs in cracks. Infiltration effectively stops once the cracks are closed within a few minutes. Intake family coefficients are shown in Table 1.4.

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Table 1.4 NRCS intake family coefficients (c is 0.6985 for all intake families) for time, t, in minutes (Cuenca 1989)

Intake family	a (cm)	b	Intake family	a (cm)	b
0.05	0.0533	0.618	0.6	0.1321	0.757
0.1	0.0620	0.661	0.7	0.1443	0.766
0.15	0.0701	0.683	0.8	0.1560	0.773
0.2	0.0771	0.699	0.9	0.1674	0.779
0.25	0.0853	0.711	1.0	0.1786	0.785
0.3	0.0925	0.720	1.5	0.2283	0.799
0.35	0.0996	0.729	2	0.2753	0.808
0.4	0.1064	0.736	3	0.3650	0.816
0.45	0.1130	0.742	4	0.4445	0.823
0.5	0.1196	0.748			

Infiltration is determined using a double-ring infiltrometer, which consists of two concentric metal cylinders inserted vertically into the soil surface. The inner cylinder, typically about 0.3 m in diameter, is used to measure the actual rate of water infiltration, while the outer cylinder, usually about 0.6 m in diameter, serves to minimize lateral water movement away from the inner ring. Both cylinders are commonly about 0.4 m in height and are driven a few centimeters into the soil to ensure proper sealing. During the test, water is poured simultaneously into both rings, and the rate at which the water level falls inside the inner cylinder is carefully observed and recorded, as illustrated in Fig. 1.9.
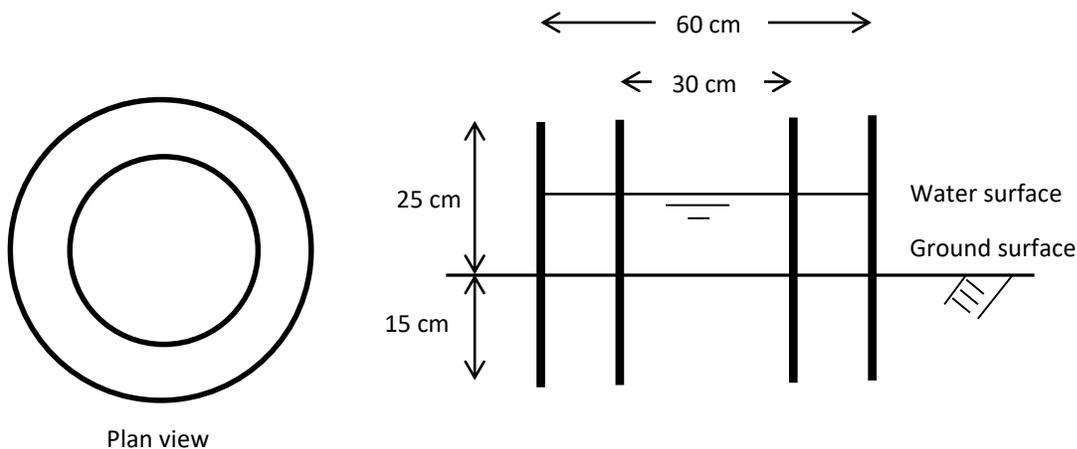

Fig. 1.9 Infiltration test equipment.

The following procedures are used to determine infiltration:

1. Push the 30 cm diameter ring about 15 cm into the soil with an appropriate hammer. Use wood to protect the ring from damage while hammering. Keep the side of the ring vertical and push the measuring rod into the soil,

2. Hammer the ring 60 cm into the soil at the same height,
3. Start the testing process by pouring water into the ring until the depth is about 70-100 mm. At the same time, add water to the space between the two rings. Do this quickly. Water between the two rings is used to prevent lateral diffusion of water during infiltration measurement,
4. Record the test start time and note the water level on the measurement rod,
5. After 1-2 minutes, record the water level drop in the inner ring on the measuring rod and add water to bring the level back to roughly the original level at the start of the test. Keep the water level outside the ring the same as the inside,
6. Continue testing until the water level drops the same during the same period of time. Take readings frequently (for example every 1-2 minutes) at the start of the test, but extend the interval between readings over time (for example every 20-30 minutes).

Example 1.8

Determine m and c in the Kostiakov equation (Eq. 1.34) for the ponded infiltrometer data as shown in Table 1.5.

Table 1.5 Infiltrometer data

Time (min)	Depth of water (cm)	Infiltrated depth (cm)
0	15	0
1	14.6	0.4
2	14.2	0.8
5	13.8	1.2
10	13.0	2
20	11.5	3.5
40	9.1	5.9
60	7.0	8

Solution

Take the log of time and infiltrated depth (Table 1.6).

Table 1.6 Logarithm of infiltrometer data

Log (t)	Log (D)	Log (Time)	Log (D)
		1	0.301
0	-0.4	1.301	0.544
0.301	-0.1	1.6021	0.771
0.699	0.079	1.7782	0.903

The line formed by the logarithm data has slope 0.7122 (Fig 1.10).

$$D = 10^a . t^b = 10^{-0.3793} . t^{0.7122} = 0.42. t^{0.7122}$$

This equation represents the cumulative infiltration curve for the soil under the test conditions and can be used to estimate infiltration at any time within the measured range.
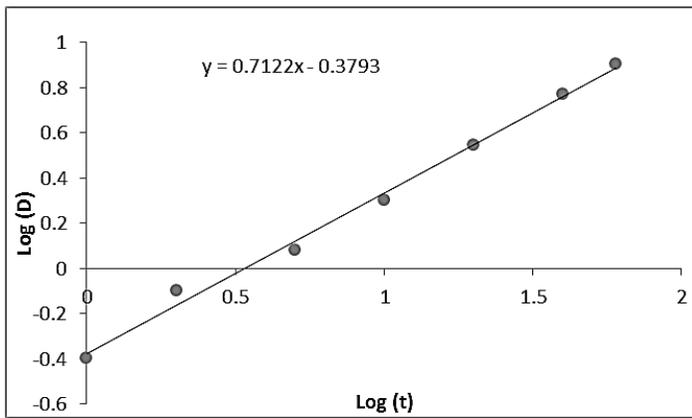

Fig 1.10 Regression of logarithm data.

2. Land Grading Design

2.1 Basic Principles and Goals of Grading

Land grading is reshaping the land surface into suitable surface for irrigation and subsequent drainage. Land grading allows for efficient and high uniformity for the application of irrigation water without excessive erosion and at the same time provides adequate surface drainage. Availability of suitable land area is one of the basic requirements for a successful farming practice.

The nature of the soil and the topography of the land area are important factors in this context. The natural terrain is undulating land consisting of humps and depressions. The nature and severity of this terrain varies from region to region. If farms are set up on undulating land, the plants and crops that grow there have to deal with the irregular distribution of soil moisture during growth. Non-uniform distribution of soil moisture and fertilizer leads to low yields and poor quality. This will be the case regardless of whether the water is obtained directly from the rain or through irrigation. If the surface irrigation method is adopted, the land grading should be carried out and gradient may be required.

2.1 Contour Lines

A contour is a line drawn on a plan which connects all points of equal level (Fig. 2.1) in relation to a defined datum or base level. It denotes a line of junction between a horizontal plane of a specific level and the ground surface; note that these horizontal planes extend above, below, and beyond the land surface. The difference in level between two consecutive contours is the Contour Interval. In most drawings, contours will be at whole meter intervals. Smaller intervals may be at half or quarter meter. Index contours are bold or thicker lines that appear at every fifth contour line. Estimating land levels and shape from a map is a useful skill to learn, especially in mountainous landscapes (Fig. 2.2). Hills and mountains are represented on the map using contour lines. By studying the contour lines, you can learn a lot about the surrounding terrain including the slopes of hills, valleys, and steep slopes. If the numbers associated with specific contour lines are increasing, the elevation of the terrain is also increasing. If the numbers associated with the contour lines are decreasing, there is a decrease in elevation. As a contour approaches a stream, canyon, or drainage area, the contour lines turn upstream.
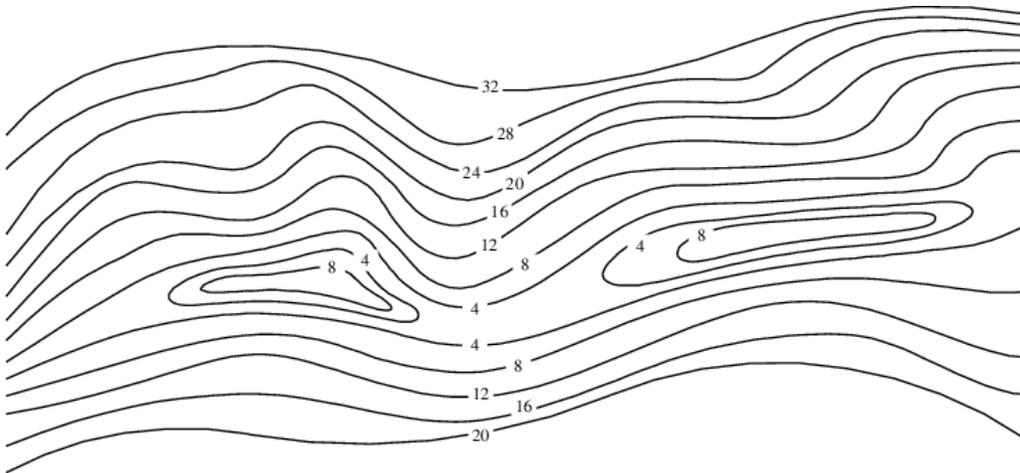
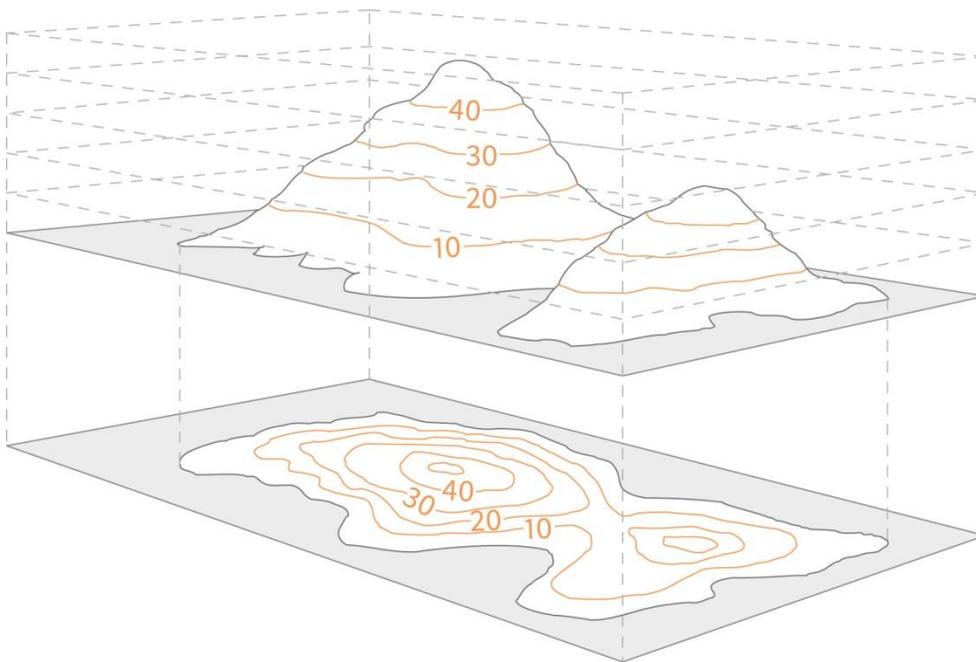**Fig. 2.1** Contour lines representing a topographic surface

Fig. 2.2 Contour lines show a pair of small hills

2.2 Slope

The slope range of the land designated for cultivation is greatly influenced by the type of soil and the method of surface irrigation. Three types of soil namely, heavy (clayey) soils, medium (loamy) soils, and light (sandy) soils can have surface slopes lying in the ranges of 0.05 to 0.25%, 0.20 to 0.40% and 0.25 to 0.65%, respectively. The advance slopes used in border irrigation can vary between 0.1% and 6%, although in some cases a zero slope is considered possible. Slopes in furrow irrigation usually vary from 0.05% to 2%, whereas level basins are constructed without any slope in the advance direction of flow or in the cross-field direction.

The gradient between any two points is determined by the ratio of the difference in level (H) between the two points and the horizontal or plan length (L) between the same two points.

$$G = \frac{H}{L} \tag{2.1}$$

Where

G : The gradient,
H : The difference in level,
L : The length, horizontal (plane).

Example 2.1
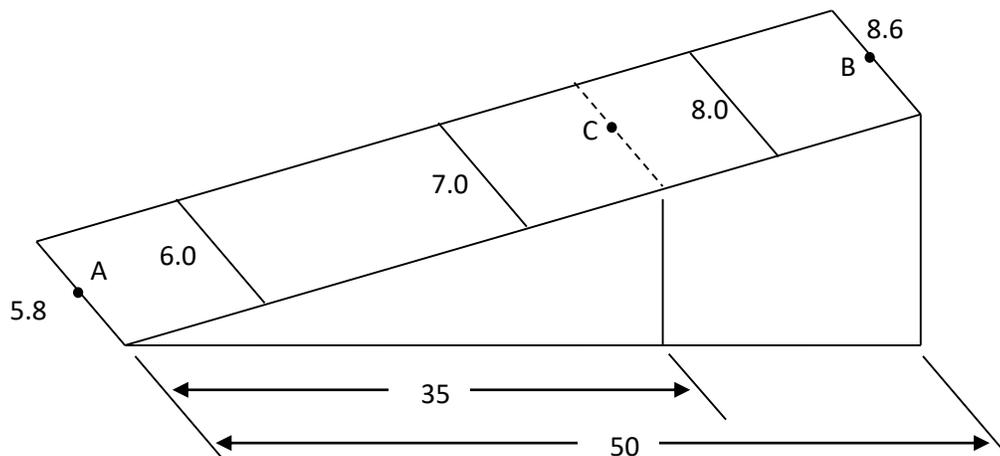Find the slope gradient and level at point C (Fig. 2.3)

Fig. 2.3 Slope gradient example

Solution
To find the gradient
By using Eq. (2.1)

$$G = \frac{H}{L} = \frac{8.6 - 5.8}{50}$$
$$= 0.056 = 5.6\%$$

For obtaining the length of horizontal plane for 1 m (difference in level)

$$\frac{5.6}{100} = \frac{1}{L} \rightarrow L = 17.86$$
$$= 1:17.86$$

To find a horizontal length (e.g. to locate level 7 in relation to point A)

$$L = \frac{H}{G} = \frac{7.0 - 5.8}{0.056}$$
$$= 21.43$$

To find a level (e.g. level of C)

$$H = difference\ in\ level\ (A - C)$$
$$= G \times L = 0.056 \times 35$$
$$= 1.96$$
$$\therefore level\ C = 5.8 + 1.96$$
$$= 7.76$$

2.3 Methods of Expressing Slopes

Slope is expressed in terms of a percentage, a proportional ratio, or a degree of slope. Percentage of slope is expressed as the number of meters rise in 100 m of horizontal distance, typically referred to as rise/run. If the slope rises 1 m in 100 m, it is considered a 1 percent slope. The percentage of slope can be calculated by the following formula:

$$G = \frac{H}{L} \times 100 \tag{2.2}$$

Where

> H : The vertical rise (m)
> L : The horizontal distance (m)
> G : The gradient (%)

Slope can also be expressed as a ratio of the horizontal distance to the vertical rise, such as two to one (2:1).

Spot elevations are used to establish limits of slope, to locate contour lines, and to provide detail for establishing control points that cannot be obtained via contour lines.

The elevation of any point on an accurately drawn contour plan may be determined by interpolation.

Point A lies about 6/10 the distance from contour 39 to contour 40; thus, A has an approximate elevation of 39.6 (Fig. 2.4). Interpolation assumes, of course, that slopes are uniform, which in many cases is not true in reality. Therefore, interpolated figures are approximations and should not be relied on as much as surveyed spot elevations for crucial measurements.
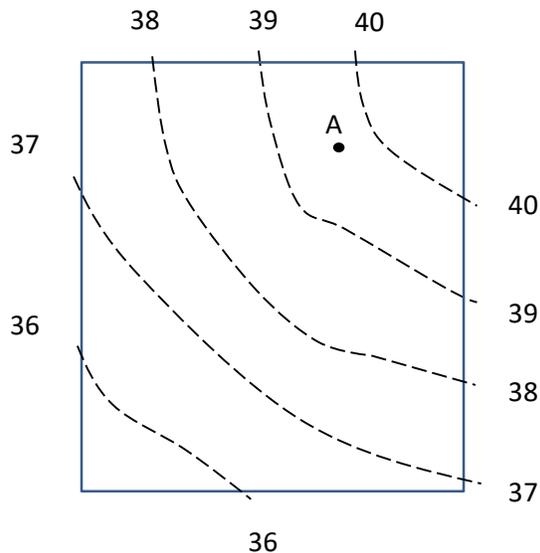

Fig. 2.4 Spot elevation by interpolation

2.4 Slope for Graded Irrigation Methods

All lands to be graded for irrigation should be suitable for use as irrigated land and for the proposed methods of water application. Water supplies and the delivery system should be sufficient to make irrigation practical for the crops to be grown and the irrigation water application methods to be used.

In irrigating by the contour-levee method water is applied to nearly level strips or areas of predetermined size at a rate enough in excess of the intake rate of the soil to permit rapid coverage. Water is retained by small dikes or levees that surround the strips and are constructed longitudinally on the contour. For this method, the best results are obtained where the slopes do not exceed 0.5 percent. However, larger slopes can be reduced by leveling the ground.

The maximum slope in the direction of irrigation shall be 0.5'/100' and the minimum slope shall be 0.02'/100' (a minimum slope of 0.05'/100' is recommended). The minimum slopes shall be applied only where adequate drainage of the land unit can be achieved for the specific crops, both primary and rotational, being cultivated.

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2.5 Creating a Profile

It is useful to plot profiles in both directions, and it is very common to plot at least some key profiles in the opposite direction from the regular plot. These features are often drawn directly on the same sheet but at right angles to the principal plot.

Another form of plotting is "two way plots" (Fig. 2.5). This method is providing a three-dimensional image of the earth's surface, which is sometimes useful for those who are rarely called upon to design of the land-leveling. The datum elevation is less than the lowest field elevation, and each point is plotted above both the datum and to the right of the reference point. The "two way plots" cannot be conveniently used in fields with large differences in elevation.
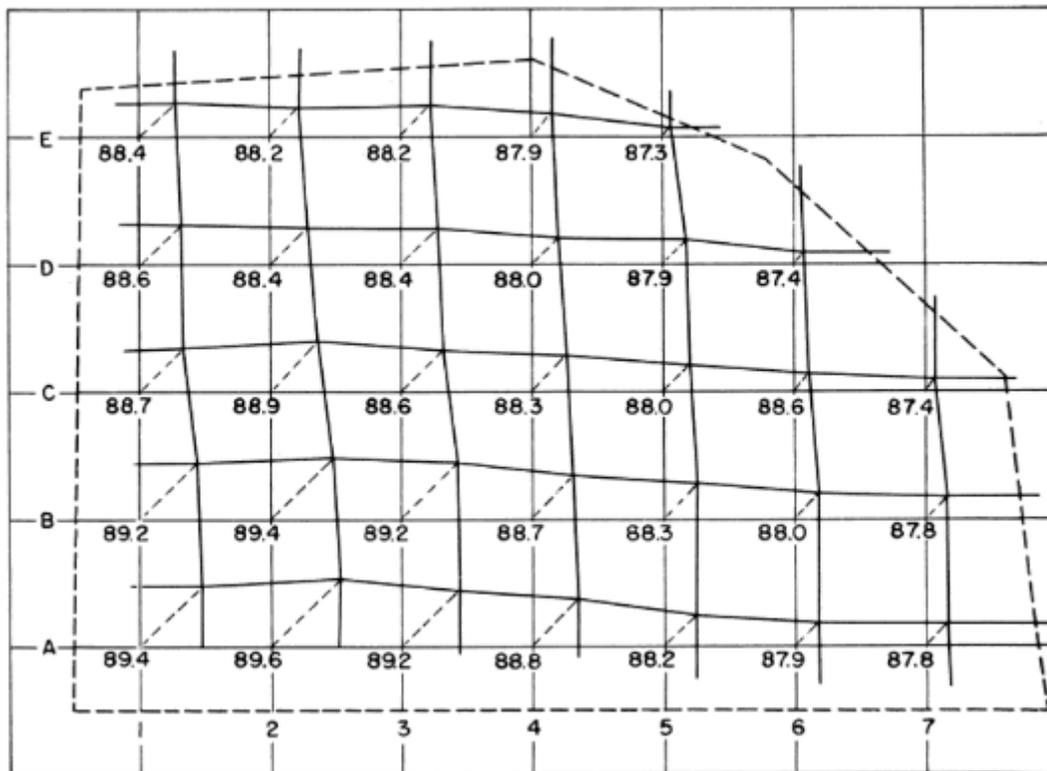
Fig. 2.5 Two way plots of profiles

2.6 Design Criteria

Irrigation design and management affects the performance, productivity and quality of crops. Irrigation design determines the efficiency and effectiveness of water use, and thus affects the profitability of the business. There are two aspects of irrigation that affect the performance and efficiency of an irrigation system, namely design and operation. Soil structure influences the maximum rate of water infiltration into the soil, and this should

determine the maximum rate of irrigation application. If water is used at a rate faster than filtration, then runoff will occur. An irrigation system also needs to be able to deliver the right amount of water in the period required. The topography of the area will affect the hydraulic design of the irrigation system.

2.6.1   Furrow Grades

Land graded for irrigation with subsequent drainage will have a slope in the direction of the row between 0.1 percent and 0.5 percent in deep alluvial soils. Fields graded to minimum slope will require more maintenance of grade than steep slopes. If the ground surface slope is steeper than 0.5%, the furrows can be set at an angle to the main slope or even along the contour to keep the furrow slopes within recommended limits. Furrows can be set in this way when the main land slope does not exceed 3%. Beyond this there is a major risk of soil erosion following a breach in the furrow system.

2.6.2   Cross Slope

If the land is not properly graded and there is a cross slope, the irrigation water will not spread evenly over the field. It will flow down the slope and always look for the lower side of the boundary (Fig. 2.6). This can be corrected by resetting the boundary to remove the cross slope or by constructing guide bunds in the boundary to prevent the cross flow of water. Recommended cross-slope is illustrated in Table 2.1.
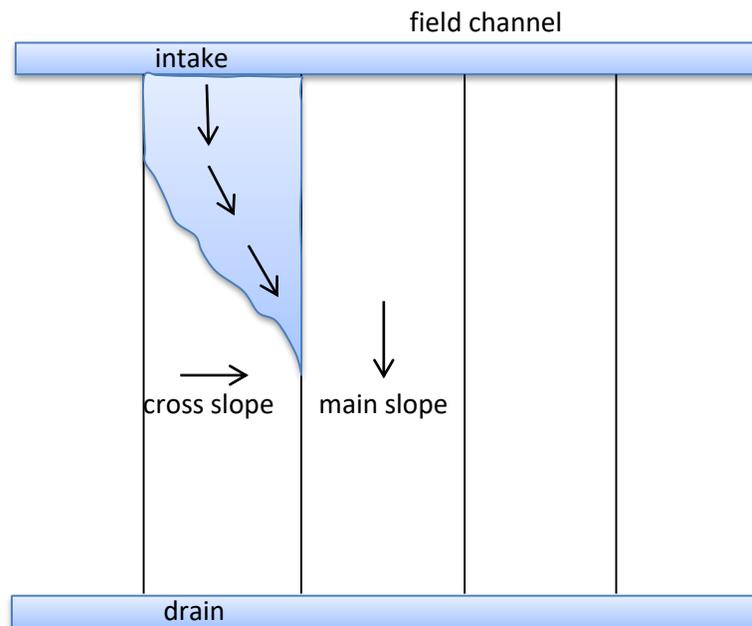
Fig. 2.6 Effect of a cross-slope on the water movement in a border

Table 2.1 Maximum recommended cross slope

Furrow grade %	Cross slope %
0.1	0.3
0.2	0.3
0.3	0.3
0.4	0.4
0.5	0.5

2.6.3   Maximum Length of Runs for Irrigation

The maximum length of irrigation runs is greatly influenced by available furrow flow rates, furrow cross-sectional area, erosion risk on the furrow, and water infiltration properties of the soil. Soil texture, crop residues and slope have an effect on soil erosion risks. A common and frequent guide to maximum flow rates is:

$$Q = \frac{10}{s} \tag{2.3}$$

Where

Q : The maximum flow rate (gallons/min),

s : The furrow slope (%).

An upper limit of 50 gpm per furrow is usually set due to furrow cross-section limitation. The recommended maximum length of runs for various row slopes and soil textures is shown in Table 2.2. A range of run lengths is provided for some conditions.

Table 2.2 Recommended maximum length of run in feet for 2- to 3-inch application

Row grade (%)	Maximum furrow stream (gpm)	Length of run (feet)			
		Soil texture			
		H	F	M	S
0.1	50	1320	1320	1320	800 to 1320
0.25	40	1320	800 to 1320	880	660 to 880
0.75	13	660 to 880	660	660	-----
1.0	10	660	660	-----	-----

H: Fine textured (sandy clays, silty clays and clays). Typical soils in this group are Sharkey clay, Osage clay. Carlow silty clay, and Wabash silty clay.
F: Moderately fine textured (sandy clay loams, clay loams and silty clay loams). Typical soils in this group are Onawa silty clay loam, Zook silty clay loam and Colo Silty clay loam.

M: Medium textured (very fine sandy loams, loams and silt loams). Typical soils in this group are Putman silt loam, Parsons silt loam, Robinsonville fine sandy loam, Dundee loam, Westerville silt loam, Nodaway silt loam and Blackoak silt loam. S: Moderately coarse textured (Sandy loams and fine sandy loams), typical soils in this group are Bruno sandy loam and Bosket fine sandy loam.

2.6.4   Maximum Field Elevation

Maximum elevation of the field should be far enough below the water source to permit delivery of required irrigating streams onto the surface of the field. Farm irrigation systems should be including facilities for removing or controlling excess storm water runoff. Leveling designs must provide field elevations and grades that will permit proper functioning of planned drainage facilities.

2.6.5   Construction Specification

The land to be graded must be cleaned of brush and excess crop residue, trash or plant matter. Grading should not be attempted when soil moisture exceeds that which would allow normal tillage. Work will be completed by land plane so that the field is free of depressions that may cause water accumulation. The land plane should be operated over the field three times: once at an angle of 45 degrees to the direction of the rows; once at right angles in the direction of the rows; and finally in the direction of the rows. The field checking to determine compliance with design grades shall have a maximum tolerance of plus or minus 0.1 feet at any network point, with no reversal grade.

2.7   Subdivide the Field into Subareas

The topography of many fields cannot be economically uniform. The field can be divided into parts, each part of which is one level. Here, divide the field into parts, each of which can be turned into a flat surface. When making these subdivisions, the quality of the leveling task may be insufficient if the subdivisions do not match the proposed drains or water supply sites. The study of the irrigation plan, the topographic map and the lower field profiles will assist the engineer in determining the boundaries of the subdivision correctly. Fig. 2.7 is an example of a field that has been divided into three sub-regions of leveling. Both the profiles and the topographical map were used as a guide in the subdivision process. Then each sub-area is considered as a separate field except that the common boundaries of the sub-areas are integrated into the design of the adjacent sub-area.
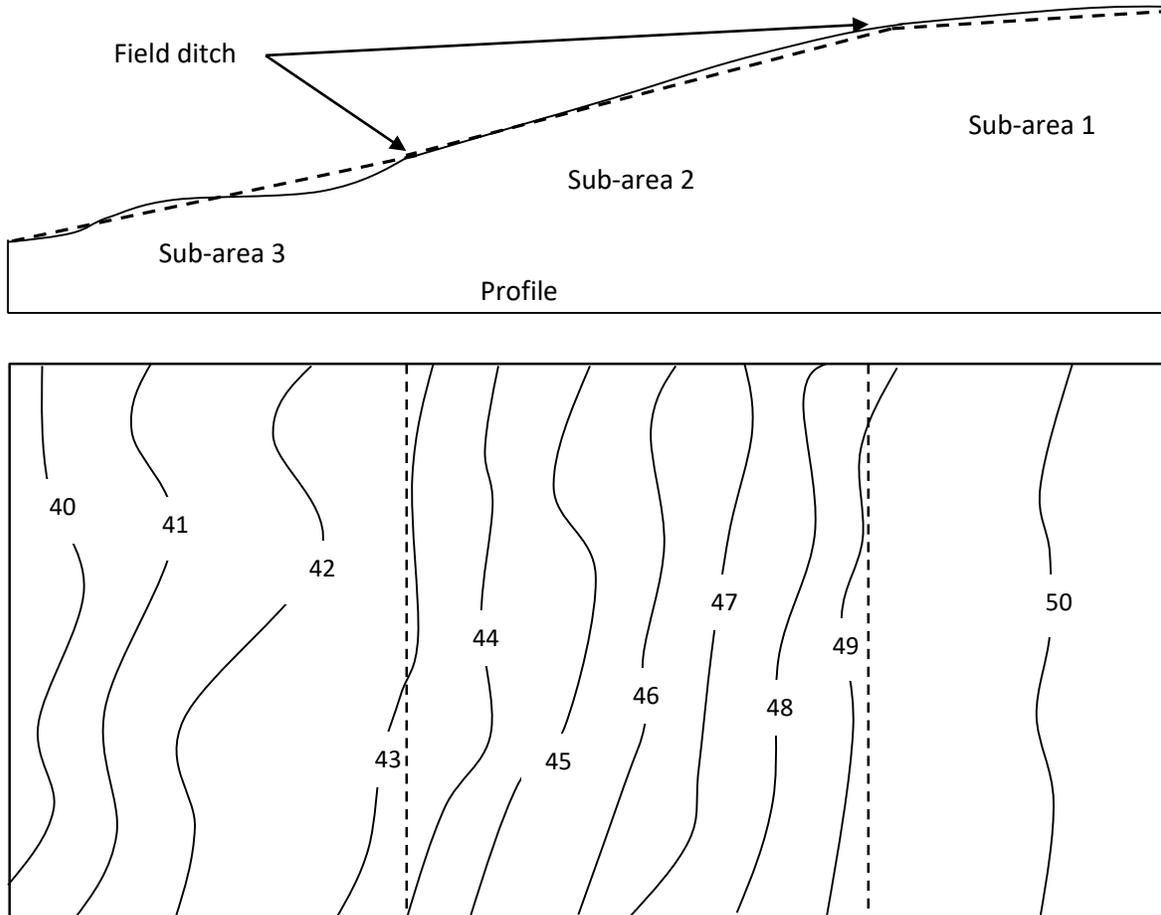
Fig. 2.7 Field subdivision for leveling

2.8 Locate the Centroid

The field has different shapes; it can be rectangular, triangular or irregular in shape. The centroid of a rectangular field is located at the point of intersection of its diagonals, while the intersection of lines drawn from its corners to the midpoints of opposite sides of the triangle is the centroid of the triangle field. In the case of an irregular shape, the area is divided into rectangles and right-angled triangles to determine the centroid. The centroid is located by calculating the moments of two reference lines at a right angle to each other. The distance to the centroid of the field from any reference line is equal to the sum of the products obtained by multiplying the area of each part by the distance from the reference line to the centroid, divided by the total area of the field. By calculating the distance to the centroid of two reference lines perpendicular to each other, the exact point of the centroid can be obtained.

Example 2.2
Compute the centroid (Fig. 2.8), assuming a line of reference 25 m south of line A, and another line of reference was assumed to be 25 m west of line 1.
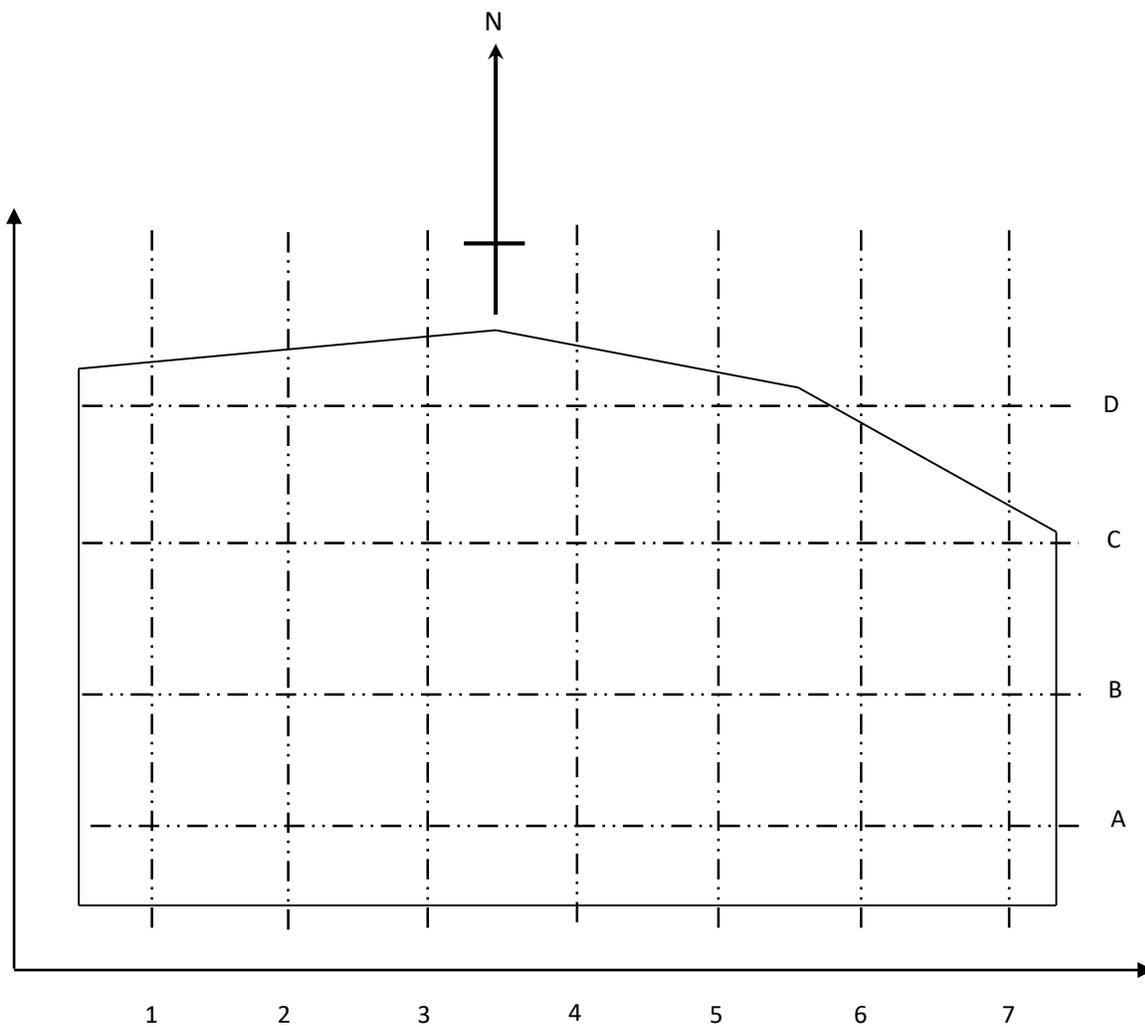
Fig. 2.8 Location of the centroid of a field

Solution

Line	Distance	No. of stakes	Product
A	1 station	7	7
B	2	7	14
C	3	7	21
D	4	5	20

Total		26	62

The distance of the centroid from the reference line is then found by dividing the sum of the products by the total number of stakes or

$$\frac{62}{26} = 2.38 \; stations \; or \; 2.38 \times 25 \; m = 59.61 \; m$$

The centroid of the field is located at 59.61 m from the reference line, or 9.61 m north of line B.

For the second reference line

Line	Distance	No. of stakes	Product
1	1 station	4	4
2	2	4	8
3	3	4	12
4	4	4	16
5	5	4	20
6	6	3	18
7	7	3	21
Total		26	99

The distance of the centroid from the reference line is then found by dividing the sum of the products by the total number of stakes or

$$\frac{99}{26} = 3.81 \; stations \; or \; 3.81 \times 25 \; m = 95.19 \; m$$

The centroid of the field is located at 95.19 m from the reference line.

2.9 The Elevation of the Plane at the Centroid

Any plane passing through the centroid at the average elevation will produce equal volumes of embankment and excavation. The average elevation of the field is obtained by adding the elevations at all corners of the grid or at all centers of grid in the field and dividing the sum of the points number. Fig. 2.5, the total elevations of 32 points on the corners of the grid is 2,827.7.
The average elevation is

$$\frac{2827.7}{32} = 88.4$$

Any plane passing through the centroid at this elevation will produce equal volumes of cut and fill.

Example 2.3

Compute the elevation of centroid of a rectangular field. The elevations at grid points as obtained from a topographic survey are stated below.

Elevation of stations at lines					
Stations	1	2	3	4	5
A	9.56	9.34	9.02	8.84	8.76
B	8.37	8.24	8.98	8.68	8.57
C	9.22	9.04	8.94	8.56	8.48
D	8.92	8.84	8.76	8.31	8.02

Solution

Sum of elevations of the 20 stations = 175.45 m.

Total number of stations = 20

$$\text{Elevation of centroid} = \frac{\text{sum of the elevations of the grid points}}{\text{number of the grid points}}$$

$$= \frac{175.45}{20} = 8.77\text{m}$$

2.10 The Elevation for each Grid Point

In order to complete the procedures for calculating the design elevation for each grid points, we assume that the field shown in the Fig. 2.9 with grid area (25m×25m) and centroid at point A, this field has the average elevation is equal to 20.8 m, assume the design plane slopes in the direction of the x and y axes are 0.2 m/25 m, and 0.1 m/25m, respectively. Assume that both axes (X and Y) are away from the field boundary by half the length of the grid side

$S_X = \frac{0.2m}{25m}$, this means that the slope of the design plane rises by 0.2 m for every distance of 25.0 meters (grid space) in the direction of X

$S_Y = \frac{0.1m}{25m}$, this means that the slope of the design plane rises by 0.1 m for every distance of 25.0 meters (grid space) in the direction of Y
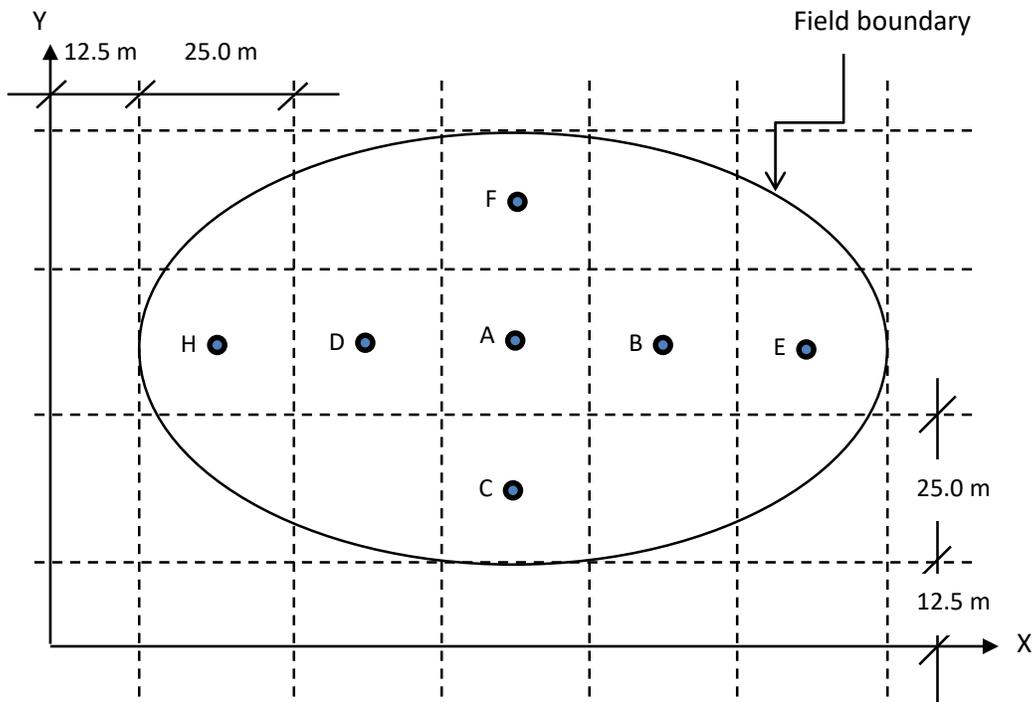
Fig. 2.9 The elevation of grid points (the centroid of the field at center of grid)

As mentioned earlier (any plane passing through the centroid at the average elevation will produce equal volumes of embankment and excavation), so, the average elevation at point (A) is 20.8m.

The elevations of other points can be calculated as follows:

Point (B), $20.8 + 0.2 = 21.0$ m
Point (D), $20.8 - 0.2 = 20.6$ m
Point (F), $20.8 + 0.1 = 20.9$ m
Point (C), $20.8 - 0.1 = 20.7$ m
Point (E), $20.8 + 0.2 \times 2 = 21.2$ m (the located of this point at two cells to the right of point A), so the slope of the design plane is multiplied by two
Point (H), $20.8 - 0.2 \times 2 = 20.4$ m (the located of this point at two cells to the left of point A), so the slope of the design plane is multiplied by two.

In the case of the centroid located on the line between two cells (Fig. 2.10), the elevations of points (D and B) can be calculated as follows:

Point (D), 20.8 – (0.2/2) = 20.7 m (point D is located at half grid space to the left of point A)
Point (B), 20.8 + (0.2/2) = 20.9 m

The elevations of points (D and B) can be used to calculate the levels of the others points in the direction of the X and Y axes.

Point (F) = 21.0 m
Point (C) = 20.8 m
Point (E) = 21.1 m
Point (H) = 20.5 m
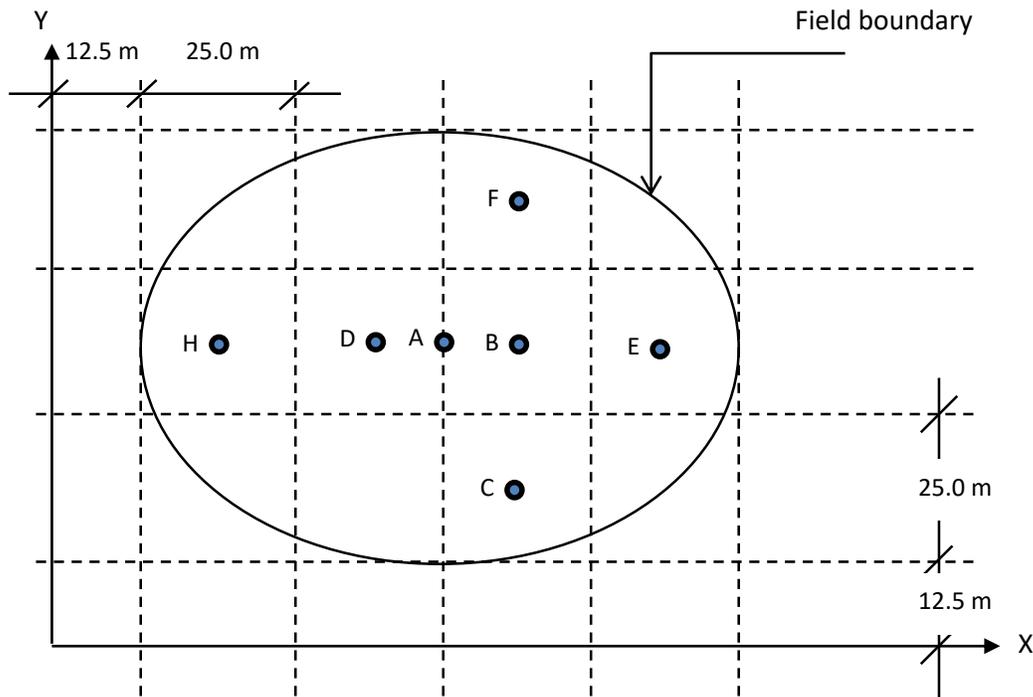
Fig. 2.10 The elevation of grid points (the centroid located on the line separating two cells)

2.11   The Cuts and Fills

The desired cut or fill can be calculated from comparing original and suggested elevations. The calculations can be checked by determining the variation per stake. If the elevation of the natural ground in the center of the cell is greater than the design elevation, the difference between them represents the average depth of cut. If the elevation of the natural ground in the center of the cell is less than the design elevation, the difference between them

represents the average depth of fill. The depths of cut and fill are denoted by the letter C and F, respectively. There are several methods for computing cut and fill for earthworks projects:

Cross-section method: The cross-section method involves drawing cross-sections of the existing and c elevations at regular intervals across the project site. The cutting area and filling area are determined for each cross section. The volume between each pair of sections is estimated by multiplying the average cutting or filling area of the two sections by the distance between them. Once these volumes are obtained for each pair of sections, the total cutting and filling volumes are obtained by adding them all together.

The volume between two sections is calculated as the average area of the two sections multiplied by the distance between them (Fig. 2.11). By summing up the volumes among all the sections, the total fill and cut volumes are determined.
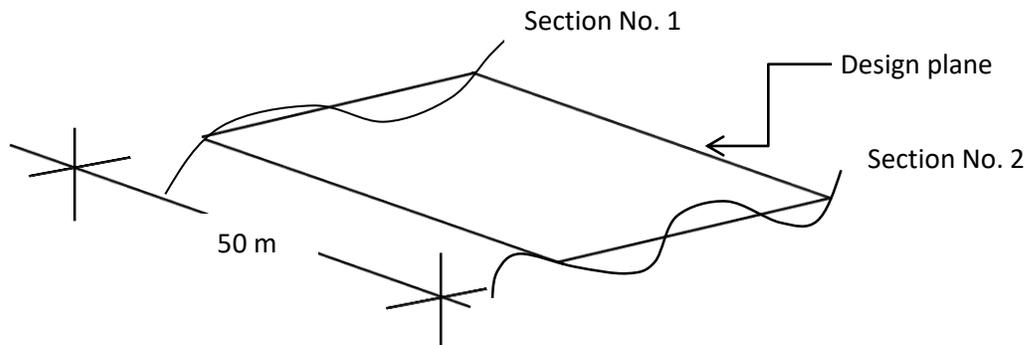
Fig. 2.11 Cross-section method

The cut area and fill area for the sections shown in Fig. 2.11 are stated below

Section No.	Cut area (m^2)	Fill area (m^2)
1	1235	1564
2	1425	1732

$$Average\ cut\ area = \frac{1235 + 1425}{2} = 1330\ m^2$$

$$Average\ fill\ area = \frac{1564 + 1732}{2} = 1648\ m^2$$

$$Distance\ between\ sections = 50\ m$$

$$Cut\ volume = 50\ \times 1330 = 66500\ m^3$$

$$Fill\ volume = 50\ \times 1648 = 82400\ m^3$$

This calculation is repeated for all of the sections, and the values are added together to get the total cut and fill volumes.

Grid method: The grid method involves drawing a uniform grid onto a plan of the earthworks project (Fig. 2.12). The volume for each cell is obtained by multiplying the depth by the cell area. By adding the volumes for each cell together, the total cut and fill volumes can be estimated. One advantage of the grid method is that the entire basis of the estimate can be summarized on the site's graphics, which provide a very clear summary of the calculations that others can verify. A disadvantage is not to create a graphical summary of the estimate. Also, like the cross-section method, the grid method is time-consuming and tedious to implement.

Fig. 2.12 Grid method
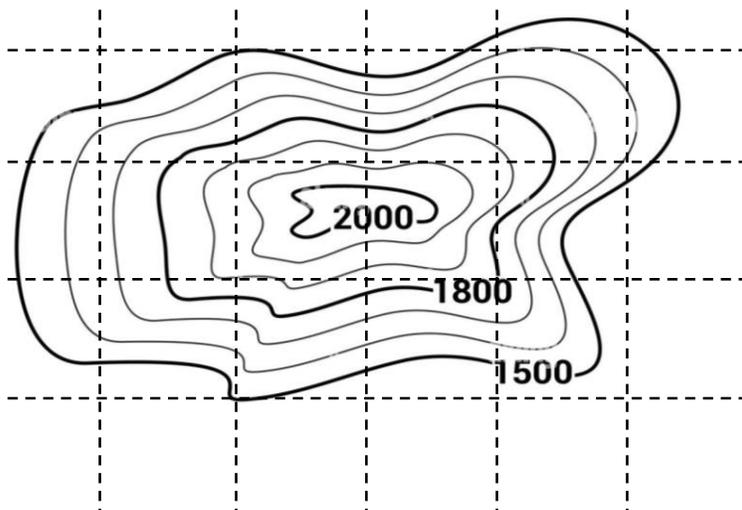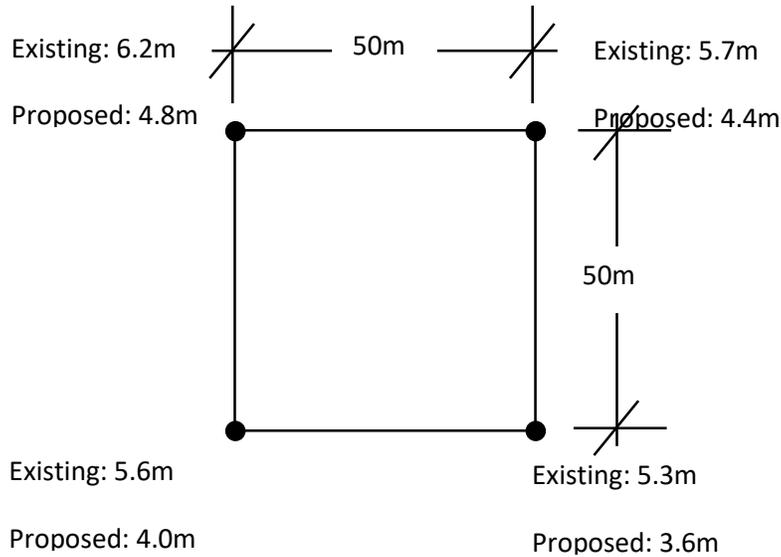
The average cut or fill depths are determined for each cell in the grid. From these depths the volumes of each grid cell can be calculated, and by adding the cell volumes together the total cut and fill volumes are obtained.

In the following example, the volume for one of the grid cell is calculated, the existing and proposed elevations at corner of the grid is illustrated below:

Existing: 6.2m   50m   Existing: 5.7m

Proposed: 4.8m   Proposed: 4.4m

50m

Existing: 5.6m   Existing: 5.3m

Proposed: 4.0m   Proposed: 3.6m

$$\text{Average existing} = \frac{6.2 + 5.7 + 5.6 + 5.3}{4} = 5.7m$$

$$\text{Average proposed} = \frac{4.8 + 4.4 + 4.0 + 3.6}{4} = 4.2m$$

Average cut depth $= 5.7 - 4.2 = 1.5\text{m}$
Cell area $= 50 \times 50 = 2500\text{m}^2$
Cell volume $= 2500 \times 1.5 = 3750\text{m}^3$

2.12   Land Grading Design Methods

Land grading improves irrigation efficiency. This also reduces labor requirements for water application. The land that has been leveled can be irrigated properly and the excess water can be drained. Major topographical changes in land leveling may reduce crop production in cut areas or additional soil may need to be added in cut areas to improve soil fertility.

2.12.1 Plane Method (Best Fit Method)

Let the existing elevation of ground surface be z (x, y) and the appropriate formation surface is represented by h (x, y). All points of h (x, y) are determined by using the optimum slopes (S_x, S_y), and the elevation of the formation surface at the origin h (0, 0). The equation of the formation surface can be expressed as follows:

$$h(x, y) = h(0,0) + S_x \times x + S_y \times y$$

Then, the cut or fill to be executed at any point would be calculated by:

$$\Delta h(x, y) = z(x, y) - h(x, y)$$

This amount would be a cut if it is positive, and it would be a fill if it is negative.

The values of S_x, S_y and $h(0,0)$ can be obtained based on the **least squares error criterion**. The following four-step procedure can be followed for this purpose:

1. Determine the average elevation of the field;
2. Determine the centroid of the field;
3. Compute the best-fit slopes S_x and S_y using the equations related to the least square method; and
4. Calculate the parameter $h(0,0)$, which is the elevation of the formation surface at the origin, on the condition that the average field elevation should be the elevation of the formation surface at the centroid, for the cut volume to balance the fill volume.

Let us use the subscript **i** to identify the series of stakes aligned in y-direction direction and **j** for the series of stakes aligned in x-direction. Let the origin be located at one half the grid spacing in each direction from the first stake position. Then the distances of the centroid from the origin are subscripted by X_c and Y_c. The optimum slopes are computed using the following equations.

$$S_x = \frac{\sum_{i=1}^{m}\left(\left(\sum_{j=1}^{n} H_j\right) \times i\right) - X_c\left(\sum_{j=1}^{n}\sum_{i=1}^{m} H_{i,j}\right)}{\sum_{i=1}^{m}(N_i \times i^2) - \left(\sum_{i=1}^{m}\sum_{j=1}^{n} N_{i,j}\right) \times X_c^2} \qquad (2.4)$$

$$S_y = \frac{\sum_{j=1}^{n}\left(\left(\sum_{i=1}^{m} H_i\right) \times j\right) - Y_c\left(\sum_{j=1}^{n}\sum_{i=1}^{m} H_{i,j}\right)}{\sum_{j=1}^{n}(N_j \times j^2) - \left(\sum_{i=1}^{m}\sum_{j=1}^{n} N_{i,j}\right) \times Y_c^2} \qquad (2.5)$$

Where

$\sum_{j=1}^{n}\sum_{i=1}^{m} H_{i,j}$ is the sum of the elevations of the grid points

$\sum_{i=1}^{m}\sum_{j=1}^{n} N_{i,j}$ is the number of grid points

$\sum_{j=1}^{n} H_j$ is the sum of the elevations of the ith column

$\sum_{i=1}^{m} H_i$ is the sum of the elevations of the jth row

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N_i is the number of grid points of the ith column

N_j is the number of grid points of the jth row

The following example illustrates the method to estimate average elevation and elevation at different grid points for a desired slope.

Example 2.4

A square field 250 m × 250 m is leveled to make it suitable for irrigation. The natural ground levels are shown in the following figure.
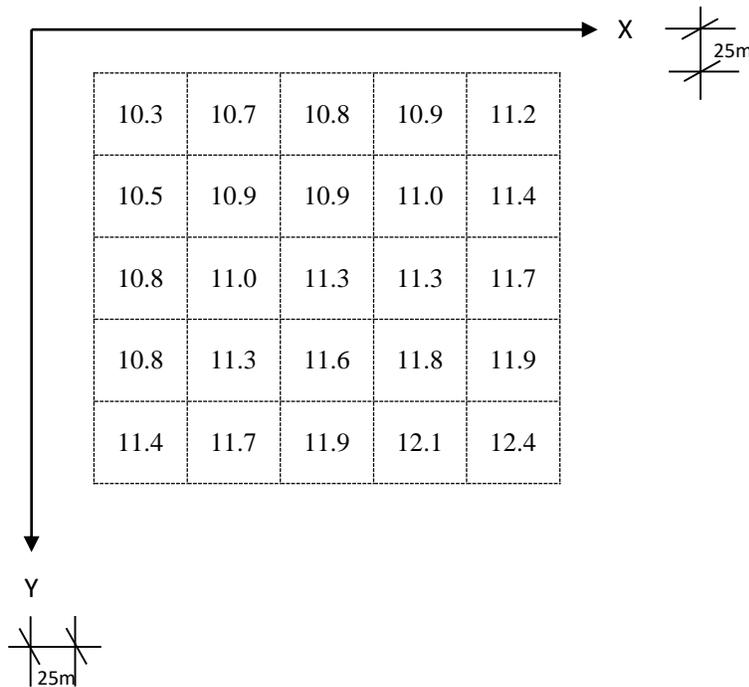
Solution

The field is assumed to have grid area (50 m × 50 m), assuming the reference lines are 25 m away from the field corner (half the grid spacing) and centroid is computed as follows:

Distance from the reference line (X)	No. of stakes	Product
1 station	5	5
2	5	10
3	5	15
4	5	20
5	5	25

Total	25	75

The distance of the centroid from the reference line is then found by dividing the sum of the products by the total number of stakes or

$$\frac{75}{25} = 3.0 \ stations \ or \ 3.0 \times 50 \ m = 150.0 \ m$$

In the same way, the distance of the centroid from the reference line (Y) is

$$\frac{75}{25} = 3.0 \ stations \ or \ 3.0 \times 50 \ m = 150.0 \ m$$

Therefore, the centroid is located at the point A (see figure below)
The optimum slopes are computed using the equations (2.4 and 2.5).

10.3	10.7	10.8	10.9	11.2
10.5	10.9	10.9	11.0	11.4
10.8	11.0	A	11.3	11.7
10.8	11.3	11.6	11.8	11.9
11.4	11.7	11.9	12.1	12.4

j	N_j	$N_j * j^2$	$(\sum H_i)j$	$(\sum H_i)j \times j$
1	5	5	53.9	53.9
2	5	20	54.7	109.4
3	5	45	56.1	168.3
4	5	80	57.4	229.6
5	5	125	59.5	297.5
$\sum$	25	275	281.6	858.7

i	N_i	$N_i \times i^2$	$(\sum H_i)i$	$(\sum H_i)i \times i$
1	5	5	53.8	53.8
2	5	20	55.6	111.2
3	5	45	56.5	169.5
4	5	80	57.1	228.4
5	5	125	58.6	293.0
$\sum$	25	275	281.6	855.9

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$$S_x = \frac{855.9 - 3 \times 281.6}{275 - 25 \times 3^2} = 0.22 \text{ m/ for grad spacing (50m)}$$

$$S_y = \frac{858.7 - 3 \times 281.6}{275 - 25 \times 3^2} = 0.27 \text{ m/ for grad spacing (50m)}$$

That is, the optimum slope is in the X direction is 0.22 m for grad spacing (50 m), and the optimum slope is in the Y direction is 0.27 m for grad spacing (50 m)

The average elevation of the field is obtained by adding the elevations at all centers of grid in the field and dividing the sum of the points number, the total elevations of 25 points on the grid centers is 281.6.
The average elevation is

$$\frac{281.6}{25} = 11.26$$

So, the average elevation at point (A) is 11.26 m. The elevation for each grid point is determined, as shown in the figure below.

10.28	10.50	10.72	10.94	11.16
10.55	10.77	10.99	11.21	11.43
10.82	11.04	**11.26**	11.48	11.70
11.09	11.31	11.53	11.75	11.97
11.36	11.58	11.80	12.02	12.24

Example 2.5
For the same example above (2.4), calculate the depth of fill and cut for each cell.

Solution
The required cut or fill is determined by comparing the original ground elevations with the proposed design elevations. Each cell is divided into four parts: the upper-left portion represents the natural ground elevation, the lower-left portion shows the proposed elevation, and the lower-right portion indicates the corresponding depth of cut or fill.

10.3		10.7		10.8		10.9		11.2	
10.28	C 0.02	10.50	C 0.2	10.72	C 0.08	10.94	F 0.04	11.16	C 0.04
10.5		10.9		10.9		11.0		11.4	
10.55	F 0.05	10.77	C 0.13	10.99	F 0.09	11.21	F 0.21	11.43	F 0.03
10.8		11.0		11.3		11.3		11.7	
10.82	F 0.02	11.04	F 0.04	11.26	C 0.04	11.48	F 0.18	11.70	0
10.8		11.3		11.6		11.8		11.9	
11.09	F 0.29	11.31	F 0.01	11.53	C 0.07	11.75	C 0.05	11.97	F 0.07
11.4		11.7		11.9		12.1		12.4	
11.36	C 0.04	11.58	C 0.12	11.80	C 0.1	12.02	C 0.08	12.24	C 0.16

When it is necessary to calculate the total depth of both fill and cut, the process can be summarized in the following table, with calculations presented row by row.

C_j (m)	0.34	0.13	0.04	0.12	0.5
F_j (m)	0.04	0.38	0.24	0.37	0

The total cut is equaled to 1.13m, while the fill is equaled to 1.03m.

2.12.2 Plane Method (Average Slope Method)

It is one of the simplest methods, but the least accurate. In this method, the slope in the direction of the X axis is obtained by calculating the average of slopes for all rows, taking into account the number of cells in each row:

$$S_x = \left(\sum_{j=1}^{n} N_j \times S_{x_j}\right) / \left(\sum_{i=1}^{m} \sum_{j=1}^{n} N_{i,j}\right) \tag{2.6}$$

If the number of cells on all rows is the same, then:

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$$S_x = \left(\sum_{j=1}^{n} S_{x_j} \right) / n \qquad (2.7)$$

Where

S_{x_j} : The slope of the jth row, it can be calculated as follows:

$$S_{x_j} = \frac{H_{mj} - H_{1j}}{(m-1)L_1} \times 100\% \qquad (2.8)$$

Where

H_{m_j}, H_{1_j} : The elevations of last and first points of the jth row,

L_1 : The grid spacing in the direction of X axis.

By the same way, the slope in the direction of the Y axis is obtained by calculating the average of slopes for all columns, taking into account the number of cells in each column:

$$S_y = \left(\sum_{i=1}^{m} N_i \times S_{y_i} \right) / \left(\sum_{i=1}^{m} \sum_{j=1}^{n} N_{i,j} \right) \qquad (2.9)$$

If the number of cells on all rows is the same, then:

$$S_y = \left(\sum_{i=1}^{m} S_{y_i} \right) / m \qquad (2.10)$$

Where

S_{y_i} : The slope of the ith column, it can be calculated as follows:

$$S_{y_i} = \frac{H_{i_n} - H_{i_1}}{(n-1)L_2} \times 100\% \qquad (2.11)$$

Where

H_{i_n}, H_{i_1} : The elevations of last and first points of the ith column,

L_2 : The grid spacing in the direction of Y axis.

Example 3.7

Calculate the slopes of the desired plane using the average slope method.

→ X

14.0	14.8	16.4	18.8
14.0	15.8	14.8	14.0
13.2	13.6	14.8	16.4
12.4	13.2	14.0	13.2

Y

Solution

j	$S_{x_j} = \dfrac{H_{m_j} - H_{1_j}}{(m-1)L_1} \times 100\%$	i	$S_{y_i} = \dfrac{H_{i_n} - H_{i_1}}{(n-1)L_2} \times 100\%$
1	$S_{x1}\% = \dfrac{18.8 - 14.0}{(4-1) \times 100} \times 100\% = 1.60\%$	1	$S_{y1}\% = \dfrac{12.4 - 14.0}{(4-1) \times 100} \times 100\% = -0.53\%$
2	0.0%	2	−0.53%
3	1.07%	3	−0.80%
4	0.27%	4	−1.87%

$$S_x = \sum_{j=1}^{n} \frac{S_{xj}}{n} = \frac{2.94}{4} = 0.73\% \qquad S_y = \sum_{i=1}^{m} \frac{S_{yi}}{m} = \frac{-3.73}{4} = -0.93\%$$

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3. Surface Irrigation

3.1 The Four Phases of Surface Irrigation

Surface irrigation is the oldest and most common method of using water to irrigate crops. During the surface irrigation process, water is moved over the soil to wet it completely or partially. Water flows over the soil surface and gradually infiltrates to the required depth. Surface irrigation methods are best suited for soils with low to moderate infiltration capacities and slopes less than 2-3%.

When water is used to irrigate the field by any of the three surface irrigation methods (border, furrow or basin), water will infiltrate through the soil to the desired depth to return the soil to its field capacity. With the border and basin irrigation method, the entire soil surface is wetted and the movement of water through the soil is mostly vertical. With the furrow irrigation method, part of the soil surface is wetted and the movement of water through the soil is vertical and lateral. There are four phases of surface irrigation as shown in Fig 3.1.
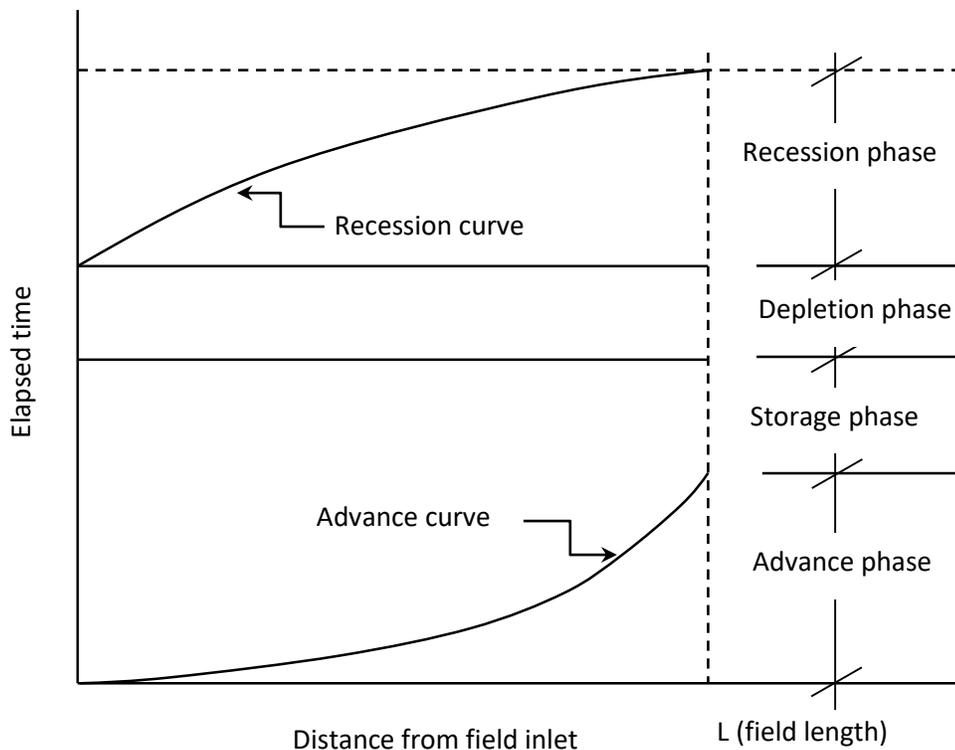

Fig. 3.1 Surface irrigation phases

3.1.1   The Advance Phase

The advanced phase starts when the field is irrigated at the upstream and ends when it reaches the end of the field. The magnitude of the stream size applied at the head of the border, furrow and basin should be greater than the rate of soil infiltration. This means that part of the water advances above the soil surface to the end of the field and part of the water infiltrates into the soil. Advance phase is the amount of time between the start of irrigation and the advance of water to the end of the field. The advance curve (Fig. 4.1) is the line that shows the relationship between elapsed time (y axis) and the advanced distance (x axis).

3.1.2   The Storage Phase

When the irrigation water reaches the downstream of the field and the water supply at the upstream of the field is continued, the water covers the entire field. Some water continues to infiltrate into the soil, and some of the excess water is collected as runoff. The time that elapses between the reaching of the water at the downstream of the field and the stopping of the flow at the upper end is called the storage phase. This stage ends when the inflow at the head of the field is stopped.

3.1.3   The Depletion Phase

When the inflow is stopped at the head end, the water may exist on the soil surface for a period of time. Some of the water still infiltrates into the soil, where the excess is collected as runoff. At a certain moment, the water will begin to recede from the end of the head. The time between the cessation of the flow at the head end and the appearance of the first bare soil that was under water is called the depletion phase.

3.1.4   The Recession Phase

The water begins to recede from the end of the head, this receding continues until the end of the tail. The time at which the water begins to disappear at the end of the head until it finally recedes from the entire field is called the recession phase.

3.2   Infiltration Opportunity Time and Application Depth

The time-difference between the recession and advance curve is called the contact time or the infiltration opportunity time. During this time any particular point in the field is in contact with water. Depending on increasing or decreasing the infiltration opportunity time within certain limits, the applied water depth can be regulated.

It doesn't matter where the water infiltrates rapidly when it first arrives, and then slows down until it reaches a steady state. This steady state is referred to as the basic infiltration

rate, which is close to the value of saturated hydraulic conductivity. When the basic infiltration rate is reached, the cumulative infiltration curve becomes a straight line and the basic infiltration rate curve becomes a horizontal line.

Soil infiltration rates are influenced by soil texture. Heavy soils have low infiltration rates due to their small pore sizes, while light soils have high infiltration rates due to their large pore sizes. Some typical infiltration rates for different soil types are illustrated in Table 3.1.

Table 3.1 Typical infiltration rates for different soils

Soil type	Infiltration rate (mm/hr)
Sand	>30
Sandy loam	20-30
Silty loam	10-20
Clay loam	5-10
Clay	<5

Typically, the primary purpose of surface irrigation system design is to determine appropriate inflow rates and cut-off times so that the optimum or potential performance required for a given field condition is achieved. The length of the field is often determined by farmers because it greatly affects the efficiency of equipment operations (Walker and Skogerboe 1987).

The required depth, Z_{req}, considered as the design depth should equal the minimum infiltrated depth, Z_{min}, and usually occurs at the lower end of the field.

The minimum infiltration opportunity time at the end of the field is determined as:

$$T_{min} = T_{str} + T_{dep} + T_{rec} \qquad (3.1)$$

Where

$\quad\quad T_{str}$: The storage time,
$\quad\quad T_{dep}$: The depletion time,
$\quad\quad T_{rec}$: The recession time.

The storage time (T_{str}) is calculated as follow:

$$T_{str} = T_{cof} - T_{adv} \qquad (3.2)$$

Where

$\quad\quad T_{cof}$: The cut-off time,
$\quad\quad T_{adv}$: The advance time.

Substitute Eq. (3.2) into Eq. (3.1), yield:

$$T_{min} = (T_{cof} - T_{adv}) + T_{dep} + T_{rec} \tag{3.3}$$

Rearranging and solving for T_{cof}

$$T_{cof} = T_{min} + T_{adv} - T_{dep} - T_{rec} \tag{3.4}$$

The four terms in the right-hand side of the above equation must be known in order to find the appropriate cut-off time for a given condition of field boundary, geometry and soil properties.

Since Z_{req} is considered as Z_{min}, the minimum infiltration time, T_{min}, is equal to the required infiltration time, T_{req}. That is, to compute T_{min} using an infiltration function relating the depth of infiltration Z_{inf} to the time available for infiltration T_{inf}. By using Kostiakov equation:

$$Z = c.T^m$$

Replacing Z by Z_{min} , the minimum infiltration time is calculated as:

$$T_{min} = \left(\frac{Z_{min}}{c}\right)^{1/m} = \left(\frac{Z_{req}}{c}\right)^{1/m} \tag{3.5}$$

3.3 Water Balance Concept

The Equations describing the hydraulics of surface irrigation are the continuity and momentum equation. These equations are known as the St.Venant equation. In general, the continuity equation expressing the conservation of mass can be written as:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} + I = 0 \tag{3.6}$$

The momentum equation expressing the dynamic equilibrium of the flow process is:

$$\frac{1}{g}\frac{\partial V}{\partial t} + \frac{V}{g}\frac{\partial V}{\partial x} + \frac{\partial y}{\partial x} = S_0 - S_f + \frac{VI}{2gA} \tag{3.7}$$

Where
- y : The depth of flow (m),
- t : The time from beginning of irrigation (sec),
- V : The velocity of flow as $f(x, t)$ (m/sec),
- x : The distance along the furrow length (m),
- I : The infiltration rate as $f(x, t)$ (m/sec),
- g : The acceleration due to gravity (m/sec^2),
- S_0 : The longitudinal slope (m/m),
- S_f : The slope of energy grade line (m/m),
- A : The cross sectional area as $f(x, t)$ (m^2),
- Q : The discharge (m^3/sec).

These equations are first-order nonlinear partial differential Eq. without a known closed-form solution. Appropriate conversion or approximations of these equations are required. So, several mathematical simulation models (Full hydrodynamic, zero-inertia, kinematic-wave and volume-balance) have been developed, however, among them volume balance models are more commonly used for design. The volume balance models consider only the continuity Eq. (3.6) and ignore the momentum Eq. (3.7).

3.4 Volume Balance Concept

Volume balance concept is the simplest and least complex model. The volume delivered to the field should equal the volumes of the surface and subsurface volumes during the advanced phase, the volume balance equation is:

$$\int_0^t Q(t)dt = \int_0^x A(s,t)ds + \int_0^x Z(s,t)ds \tag{3.8}$$

Where
- Q : The inflow rate,
- A : The cross-sectional area of the surface flow at distance s and time t,
- Z : The cross-sectional area of the subsurface flow at distance s and time t.

Given unit width of border and constant flow rate, constant average stream depth, and assuming Z is independent of water depth and a function of infiltration opportunity time only, t-t$_s$, where t$_s$ is the advance time at distance s, the Lewis and Milne (1938) equation for advance can be shown as follows:

$$Q_0 t = \bar{y}x + \int_0^t z(t - t_s) \frac{dx}{dt}\Big|_{t=t_s} dt_s \tag{3.9}$$

Where

Q_0 : The constant inflow rate per unit width,

$\bar{y}$: The distance-average depth of the irrigation stream,

z : The cumulative infiltration in volume per unit area of border.

The simplest solution to equation (3.9) is as follows:

$$Q_0 t = \sigma_y Y_0 x + \sigma_z Z_0 x \tag{3.10}$$

Where

Y_0 : The depth of flow at the inlet (Fig. 3.2),

Z_0 : The depth of infiltration at the inlet,

t : The elapsed time since the irrigation started,

x : The advance distance,

σ_y : The surface storage shape factor, which is defined as a constant of 0.70 to 0.80,

σ_z : The subsurface shape factor, it is a function of the exponent term in the Kostiakov infiltration equation assuming a uniform advance rate (Katopodes and Strelkoff 1977a)
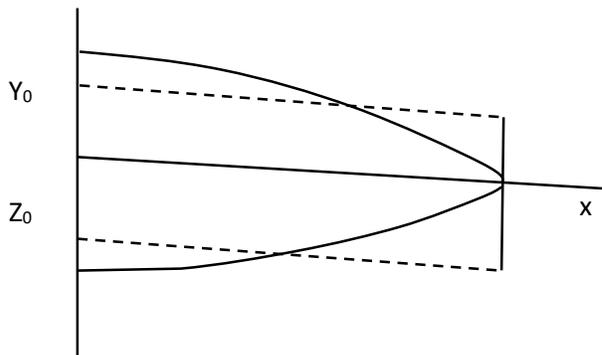
$$\sigma_z = \frac{1}{1+m} \tag{3.11}$$

Fig. 3.2 Depths of flow and infiltration at the inlet.

Elliott and Walker (1982) gave the following solution to the volume balance considering Modified Kostiakov Eq.:

$$Q_0 t = \sigma_y A_0 x + \sigma_z x f t^m + \frac{txP}{1+b} \tag{3.12}$$

Where

A_0 : The cross-sectional area of flow at the inlet,

P : The basic infiltration rate,

$f, and\ m$: The empirical coefficients,

b : The empirical coefficient of advance curve, assuming that the relationship between the distance along the furrow length (x) and the time (t) follows a power function:

$$x = at^b \qquad (3.13)$$

Where

x : The advance distance,

a : The empirical coefficient of advance curve.

σ_z is the subsurface shape factor can be defined as:

$$\sigma_z = \frac{m+b(1-m)+1}{(1+m)(1+b)} \qquad (3.14)$$

$$A_0 = \sigma_1 y^{\sigma_2} \qquad (3.15)$$

$$WP = \gamma_1 y^{\gamma_2} \qquad (3.16)$$

Or as a simple substitute,

$$A_0 R^{0.67} = \alpha_1 A_0^{\alpha_2} \qquad (3.17)$$

Where

y : The depth of flow,

R : The hydraulic radius,

WP : The wetted perimeter,

$\sigma_1, \sigma_2, \gamma_1, \gamma_2, \alpha_1, \alpha_2$ are empirical shape coefficients, it should be noted that for border irrigation systems, $\sigma_1, \sigma_2, \gamma_1\ and\ \alpha_1$ are equal to 1.0 and γ_2 is equal to zero, and α_2 is 3.333.

The inlet flow area can be determined as:

$$A_0 = \left(\frac{Q_0 n}{60\alpha_1\sqrt{S_0}}\right)^{\frac{1}{\alpha_2}} \qquad (3.18)$$

Where

> n is the Manning roughness coefficient, range from about 0.02 for previously irrigated and smooth soil, to about 0.04 for freshly tilled soil, and approximately equal to 0.15 for dense growth obstructs the water movement.

In a level slope condition, such as a basin, it is assumed that the friction slope is equal to the inlet depth, Y_0 in m, divided by the distance covered by water, x in m. This leads to the following expression for A_0

$$A_0 = \left(\frac{(Q_0 n)^2 x}{3600}\right)^{\frac{3}{13}}$$
(3.19)

The advance time can be calculated as follows:

1. The first step is to calculate the cross-sectional area of the flow,
2. Make a rough estimate of the empirical coefficient of advance curve (b), usually the setting, $= 0.1$ to 0.9 are good rough estimates. Then, the revised estimate of b is calculated and compared below,
3. Use Newton-Raphson technique for computing the advance time,
4. Assume an initial estimate of t as T_1, then $T_1 = \frac{5(A_0 L)}{Q_0}$, where, L is the length of the field,
5. Calculate a revised estimation of t (say T_2) as follows:

$$T_2 = T_1 - \frac{Q_0 T_1 - 0.77 A_0 L - \sigma_z L f T_1^m - \frac{T_1 L P}{1+b_1}}{Q_0 - \frac{m f \sigma_z L}{T_1^{1-m}} - \frac{P L}{1+b_1}}$$

6. Compare the initial and the revised. If it is within 0.5 minutes or less, the analysis moves to step 4. If it is not equal, let $T_1 = T_2$ and repeat steps 4 through 5. It should be noted that if the inflow is not sufficient to complete the advance phase in about 24 hours, the value of inflow is too small or the L value is too large and the design process must be redone with revised values.

The volume balance design procedure requires that the infiltration opportunity time should associated with Z_{req} be known. This time, it can be obtained with a modified Kostiakov Eq. by using the Newton-Raphson technique.

1. Make an initial estimate of T_{req} and label it T_1,
2. Calculate a revised estimation of T_{req}, T_2:

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$$T_2 = T_1 - \frac{Z_{req} - fT_1^m - PT_1}{-mfT_1^{m-1} - P}$$

3. Compare the values of the initial and revised estimates of T_{req} (T_1 and T_2) by taking their absolute difference. If they are equal to each other or within an acceptable tolerance of about 0.5 minute, the value of T_{req} is determined as the result. If they are not sufficiently equal in value, replace T_1 by T_2 and repeat steps 2 and 3.

Example 3.1

Determine the advance time for a field with following data:

m	0.568	L	200 m
f	0.00324 m³/min/m	n	0.04
P	0.000174 m³/min/m	α_1	0.55
Q_0	0.15 m³/min	α_2	1.35
S_0	0.001		

Solution

Set $b_1 = 0.6$

Use the following equation to determine the subsurface shape factor

$$\sigma_z = \frac{m + b(1-m) + 1}{(1+m)(1+b)}$$

$$\sigma_z = \frac{0.568 + 0.6(1 - 0.568) + 1}{(1 + 0.568)(1 + 0.6)} = 0.728$$

The inlet flow area can be determined as:

$$A_0 = \left(\frac{Q_0 n}{60\alpha_1\sqrt{S_0}}\right)^{\frac{1}{\alpha_2}}$$

$$A_0 = \left(\frac{0.15 \times 0.04}{60 \times 0.55\sqrt{0.001}}\right)^{\frac{1}{1.35}} = 0.0219 \, m^2$$

Assuming an initial estimate of t as T_1, then $T_1 = \frac{5(A_0 L)}{Q_0}$

$$T_1 = \frac{5(0.0219 \times 200)}{0.15} = 146 \, min$$

A revised estimation of t (say T_2) is calculated as follows:

$$T_2 = T_1 - \frac{Q_0 T_1 - 0.77 A_0 L - \sigma_z Lf T_1^m - \frac{T_1 LP}{1+b_1}}{Q_0 - \frac{mf\sigma_z L}{T_1^{1-m}} - \frac{PL}{1+b_1}}$$

$$T_2 = 146 - \frac{0.15\times146 - 0.77\times0.0219\times200 - 0.728\times200\times0.00324\times146^{0.568} - \frac{146\times200\times0.000174}{1.6}}{0.15 - \frac{0.568\times0.00342\times0.728\times200}{146^{1-0.568}} - \frac{0.000174\times200}{1.6}} = 146 -$$

$$(75.69) = 70.31 \; min$$

Take T_1 as 70.31 min,

$T_2 = 66.07$ min,

The absolute difference between the two values $(70.31 - 66.07) = 4.24$ min,

Take T_1 as 66.07 min,

$T_2 = 66.04$ min,

The absolute difference between the two values $(66.07 - 66.04) = 0.03$ min,

At the end of this iteration, the error is less than one minute and the value of t is found to be 66.07 min.

Example 3.2
Use the data of the pervious example (3.1) to determine the infiltration opportunity time associated with Z_{req}, which is equal to (60 mm).

Solution
Assuming an initial estimate of t as T_1 , which is equal to 100 min,

Calculate a revised estimation of T_{req}, T_2:
$$T_2 = T_1 - \frac{Z_{req} - fT_1^m - PT_1}{-mf T_1^{m-1} - P}$$

$$T_2 = 100 - \frac{0.06 - 0.00324\times100^{0.568} - 0.000174\times100}{-0.568\times0.00324\times100^{(0.568-1)} - 0.000174} = 100 - 4.02 = 95.97 \; min$$

Take T_1 as 95.97 min,

$T_2 = 95.99$ min,

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The absolute difference between the two values (95.99 – 95.97) = 0.02 min,

At the end of this iteration, the error is less than one minute, so the infiltration opportunity time value associated with (Z_{req}= 60 mm) is obtained to be 95.97 min.

Example 3.3

The relationship between the distance along the border length (x) in meters and the time (t) in minutes is $(x = 20t^{0.6})$, the infiltration rate is constant I (mm/hr), the inflow rate for a unit width of the border is (5.5 liter/sec/m), the depth of surface water and total infiltration values when the irrigation water reaches the end of the border are equal to (110 mm and 46 mm), respectively. Calculate the infiltration rate.

Solution

Use Lewis and Milne (1938) equation:

$$Q_0 t = \bar{y}x + \int_0^t z(t - t_s) \frac{dx}{dt}\Big|_{t=t_s} dt_s \text{ , or}$$

$$Q_0 t = \sigma_y Y_0 x + \int_0^t z(t - t_s) \frac{dx}{dt}\Big|_{t=t_s} dt_s$$

Assume the surface storage shape factor is equal to 0.75, so:

$$x = 20t^{0.6}, \quad \frac{ds}{dt_s} = 12t_s^{-0.4}, \quad I = I \times \left(\frac{10^{-3}}{60}\right) m/min, \quad Y_0 = 0.11\ m$$

$$z(t - t_s) = I.(t - t_s) \times \frac{10^{-3}}{60} m, \quad Q_0 = 0.33 \frac{m^3}{min}$$

$$0.33t = 0.75 \times 0.11 \times 20t^{0.6} + \int_0^t \left[I(t - t_s) \times \frac{10^{-3}}{60}\right] \times (12t_s^{-0.4} dt_s)$$

$$0.33t = 1.65t^{0.6} + \frac{I}{5000} \int_0^t (tt_s^{-0.4} - t_s^{0.6}) dt_s$$

$$0.33t = 1.65t^{0.6} + It^{1.6}/4800$$

The total infiltration value when the irrigation water reaches the end of the border is equal to 46 mm, so

$$I = 60 \frac{46}{t} \rightarrow t = \frac{2760}{I} \text{ , substitute in above equation, then}$$

$$0.33 \left(\frac{2760}{I}\right) = 1.65 \left(\frac{2760}{I}\right)^{0.6} + I \left(\frac{2760}{I}\right)^{1.6} / 4800$$

Solve above equation by trial and error,

$$\therefore I = 25 \ mm/hr$$

Example 3.4

Find the maximum distance that the water reaches, the relationship between the distance along the border length (x) in meters and the time (t) in minutes is $(x = 25t^{0.5})$. The inflow rate for a unit width of the border is (5 liter/sec/m), the depth of flow at the inlet is 8 cm, the infiltration rate is constant (20 mm/hr).

Solution
Use the following equation
$$Q_0 t = \sigma_y Y_0 x + \int_0^t z(t - t_s) \frac{dx}{dt}\Big|_{t=t_s} dt_s$$

$$Q_0 = 0.3 \ m^3/min \qquad \frac{ds}{dt_s} = 12.5 t_s^{-0.5}$$

Assume the surface storage shape factor is equal to 0.75, so:

$$0.3t = 0.75 \times 0.08 \times 25t^{0.5} + \int_0^t 0.0042(t - t_s)t_s^{-0.5} dt_s$$

$$0.3t = 1.5t^{0.5} + 0.0056t^{1.5}$$

Solve above equation by trial and error,

$$\therefore t = 31 \ min \rightarrow x = 25 \times 31.16^{0.5} = 139.5 \ m$$

Or, using the Newton-Raphson technique to find t, as follows

$$T_2 = T_1 - \frac{0.3T_1 - 1.5T_1^{0.5} - 0.0056T_1^{1.5}}{0.3 - 0.75T_1^{-0.5} - 0.0084T_1^{0.5}}$$

Assuming an initial estimate of t as T_1 , which is equal to 50 min,

$$T_2 = 32.06 \ min$$

Take T_1 as 32.06 min,

$T_2 = 31.16$ min,

Take T_1 as 31.16 min,

$T_2 = 31.15$ min,

The absolute difference between the two values $(31.16 - 31.15) = 0.01$ min,

3.5 Efficiencies of Surface Irrigation Systems

When furrow irrigation is designed properly, the field application efficiency can reach 65%. A lot of losses occur during deep filtration at the upper end of the field and runoff at the lower end. The field application efficiency of properly designed and managed borderstrips is up to 75%. In properly designed and managed basins, the field application efficiency reaches 80%.

Conveyance efficiency E_c and field canal efficiency E_b are sometimes combined and called distribution system efficiency E_d, expressed as:

$$E_d = E_c \times E_b \qquad (3.20)$$

Field canal efficiency E_b and field application efficiency E_a are sometimes combined and called farm irrigation efficiency E_f, expressed as:

$$E_f = E_a \times E_b \qquad (3.21)$$

Common problems that can reduce the efficiency of field application of the three surface irrigation methods can be mentioned below:

1. Over irrigation and inadequate water application occur in some places when the land leveling is poor. Water will flow to the lowest side causing over-irrigation when the cross slope is not horizontal for borderstrip irrigation.
2. The difference in the type of soil along the irrigation stream leads to a difference in the infiltration rate.
3. Small inflow rate leads to long an advance time, over irrigation may be occurred at the head of the border and furrow. In the case of basins, small inflow rate diverted into a basin will take long time to cover the area of entire basin, since the basin sections will have a very different contact time.

4. Water flows very quickly down the border or furrow when a large inflow rate is applied, when this case occurs, a cutoff taking place before the root zone has been filled with water.
5. Variable lengths of run will complicate the operation of the system and reduce efficiency.

3.6   Criteria for the Selection of the Surface Irrigation Method

Under a certain set of conditions, it is not possible to give specific guidance on which surface irrigation method to select, as each option usually has advantages and disadvantages. The selection of surface irrigation method depends mainly on the type of soil, the type of crops, the depth of irrigation, the slope of the land, the shape of the field, the availability of labor and the source of water. Table 3.2 illustrates some guidance on the most appropriate method depending on soil type, rooting depth of crops and net depth of application.

Table 3.2 Selection of an irrigation method based on soil type and net irrigation depth

Soil type	Rooting depth	Net irrigation depth	Surface irrigation method
Sand	Shallow	20-30	Short furrows
	Medium	30-40	Medium furrows, short borders
	Deep	40-50	Long furrows, medium borders, small basins
Loam	Shallow	20-30	Medium furrows, short borders
	Medium	30-40	Long furrows, medium borders, small basins
	Deep	40-50	Long borders, medium basins
Clay	Shallow	20-30	Long furrows, medium borders, small basins
	Medium	30-40	Long borders, medium basins
	Deep	40-50	Large basins

3.6.1   Soil Type

All three surface irrigation methods are preferred when used to irrigate heavy soils with lower infiltration rates. For light soils with high infiltration rates where deep percolation

losses in the upper part of the fields are high, this causes the field application to be less efficient.

3.6.2   Crop Type

Furrows irrigation is suitable for irrigation of row crops such as maize and vegetables. Furrows are used usually for crops of shallow roots. Borderstrip irrigation can also be used for row crops or for close-growing crops that do not prefer pond water for long periods, such as wheat and alfalfa. Crops that can tolerate very wet soil for up to 24 hours are best grown in basins.

3.6.3   Irrigation Application Depth

Furrow irrigation is the most appropriate method of irrigation if the depth of application is small. Large irrigation application depths can be applied more efficiently with basin irrigation.

For all three surface irrigation methods, the gross irrigation depth is much greater than the net irrigation depth, due to the lower irrigation efficiency of surface irrigation compared to pressurized systems. Basin irrigation can have higher irrigation efficiency and use less water for the same crop on the same soil compared to other surface irrigation methods where water is confined within bunds.

3.6.4   Land Slope

All surface irrigation methods prefer flat land because steep slopes require excessive land leveling to avoid erosion, which is costly and can lead to the removal of topsoil.

Flat land with a slope of 0.1% or less is preferred for basin irrigation (which needs a zero slope) because it requires minimal land leveling. Borderstrip irrigation is used on steep lands, up to 5%, depending on other limiting factors such as soil type. It is not preferable to use furrow irrigation on steep slopes, this is because the flow is confined in a small channel, which can lead to erosion.

3.6.5   Field Shape

To use the same stream size for the same furrow lengths, furrow irrigation requires regular field shapes. However, regularly shaped fields are preferred for all three types.

3.6.6   Labor Availability

In the case that there is a critical shortage of labor, this factor must be considered as important. Basin irrigation requires less labor than the other two methods.

4. Furrow Irrigation Design

4.1 Limitations of Furrow Irrigation

While furrow irrigation remains a widely adopted method for surface irrigation particularly for row crops such as maize, cotton, and sugarcane, it is not without limitations. A comprehensive understanding of these limitations is essential for engineers, agronomists, and water resource managers involved in the planning, design, and management of surface irrigation systems. Understanding these constraints is critical to ensure proper design, efficient operation, and long-term sustainability of the irrigation system. The key limitations are outlined below:

1. Non-Uniform Water Application

Furrow irrigation often suffers from non-uniform water distribution along the length of the furrow. Water infiltrates more rapidly near the upstream end due to longer ponding times, while downstream sections may receive insufficient water. This non-uniformity can lead to water stress in some areas and waterlogging in others, ultimately reducing crop yield and water use efficiency.

2. High Land Preparation and Labor Requirements

Achieving efficient furrow irrigation demands precise land leveling and careful furrow construction to ensure proper flow and infiltration. This requirement increases both the initial setup cost and ongoing labor input, particularly in uneven or sloping terrains.

3. Susceptibility to Soil Erosion

On sloped fields, furrow irrigation may cause significant surface runoff, leading to soil erosion and loss of topsoil nutrients. Without adequate erosion control measures such as shorter furrow lengths or reduced inflow rates, the system may degrade soil quality over time.

4. Poor Suitability for Sandy Soils

In sandy or highly permeable soils, the rapid infiltration rate prevents water from advancing adequately along the furrow. This results in under irrigation of downstream areas and inefficient lateral spread of water to the root zone.

5. **Deep Percolation Losses**

Prolonged application times or excessive flow rates at the furrow inlet can lead to over-irrigation and deep percolation losses. This not only wastes water but can also leach nutrients beyond the crop root zone, diminishing soil fertility and increasing environmental risks.

6. Limited Crop Compatibility

Furrow irrigation is most effective for widely spaced row crops. It is generally unsuitable for closely spaced or broadcast-seeded crops such as wheat, barley, or alfalfa, which are better served by basin or sprinkler irrigation methods.

7. Salinity and Waterlogging Risks

In regions with poor drainage or high water tables, furrow irrigation can exacerbate waterlogging and promote salt accumulation in the root zone. This is particularly problematic in arid and semi-arid areas, where careful water and salt management is essential.

8. **Surface Runoff and Environmental Concerns**

If furrow lengths are excessive or inflow rates are not well controlled, surface runoff may occur. This not only reduces application efficiency but also poses environmental concerns by transporting sediments, agrochemicals, or nutrients to adjacent water bodies.

4.2   Soil Intake Characteristics

The performance and efficiency of furrow irrigation systems are strongly influenced by the soil's intake characteristics, which describe how water infiltrates into the soil profile over time. Understanding and accurately quantifying soil intake is fundamental for designing furrow length, inflow rate, irrigation duration, and overall water application depth.

Soil intake behavior depends on several factors, including soil texture and structure, initial moisture content, compaction, and organic matter content. These factors affect both the infiltration rate and cumulative infiltration, which determine how water moves vertically and laterally into the root zone.

In furrow irrigation, water infiltrates the soil primarily through:
- **Vertical infiltration**: Water movement from the furrow into the soil below due to gravity.
- **Lateral infiltration**: Water movement horizontally from the furrow into the adjacent soil, reaching the crop root zones located on the ridges.

Because the water in furrows is concentrated in narrow channels rather than applied uniformly over the field surface, the three-dimensional wetting pattern becomes more complex than in other surface methods (e.g., basin or border irrigation).

To simulate or design furrow irrigation systems, engineers commonly use empirical infiltration models that relate the cumulative volume of intake (liter/m) (V) to the time of water application (t). These models are derived from field observations or infiltration experiments (e.g., double-ring infiltrometers, inflow-outflow methods). The most widely used models include:
1. Power Function (Kostiakov Equation)

$$V = K.t^N \tag{4.1}$$

1. Modified Kostiakov Equation (Kostiakov-Lewis)

$$V = A.t^n + B.t \tag{4.2}$$

Where
$K, N, A \text{ and } n$: Empirical parameters,
B: The basic intake rate of the soil (liter/min/m).

Soil texture is a primary determinant of infiltration behavior, influencing both the rate at which water enters the soil and the pattern of its movement within the root zone. Coarse-textured soils, such as sands, tend to have high initial infiltration rates but limited water retention capacity, while fine-textured soils, like clays, exhibit slower infiltration but greater moisture-holding ability and lateral spread. Understanding these general trends is essential when selecting appropriate furrow lengths, flow rates, and irrigation durations for different soil types. Table 4.1 summarizes typical infiltration characteristics for various soil textures, including initial and steady-state infiltration rates and qualitative descriptions of water movement behavior:

4.3 Two-point Volume Balance Method

The two-point volume balance method is a simplified yet effective analytical approach used to estimate the advance time and surface water distribution in surface irrigation systems, particularly border and furrow irrigation. This method assumes that the water front advances as a function of time and that the infiltration profile can be represented by a volume balance between inflow, surface storage, and infiltration losses.

Table 4.1 Typical infiltration behavior by soil texture

Soil Type	Initial Infiltration Rate (cm/hr)	Steady Rate (cm/hr)	Infiltration Pattern
Coarse Sand	15 – 30	10 – 20	Fast infiltration, shallow lateral spread
Loamy Sand	10 – 20	5 – 10	Moderate vertical and lateral movement
Sandy Loam	5 – 15	2 – 5	Balanced vertical/lateral movement
Loam	2 – 6	1 – 3	Moderate infiltration, good water retention
Silty Clay Loam	1 – 3	0.5 – 1.5	Slow infiltration, wide lateral spread
Clay	0.5 – 2	< 1	Very slow, high lateral movement potential

Note: These are generalized values; actual infiltration characteristics vary significantly depending on field conditions.

In its basic form, the method utilizes two known points along the field typically the time and distance when the advancing front reaches two specific locations to determine empirical coefficients in the advance function. The advance curve is usually expressed as a power function:

$$x = at^b \qquad\qquad (4.3)$$

Where

x : The distance the wetting front has advanced (m),

t : The elapsed time since irrigation started (min),

a and b : Empirical parameters determined from two field measurements.

The two-point volume balance model estimates the position of the advancing wetting front by comparing the total applied water volume with the combined surface and subsurface storage volumes. Surface storage refers to the volume of water stored above the soil surface within the furrow. It is calculated as the product of the cross-sectional flow area at the upstream end, the furrow spacing, the distance to the wetting front, and a shape factor. Subsurface storage represents the volume of water that has infiltrated into the soil and is computed as the product of the infiltrated depth at the upstream end, the distance to the wetting front, and a shape factor (see figure below).
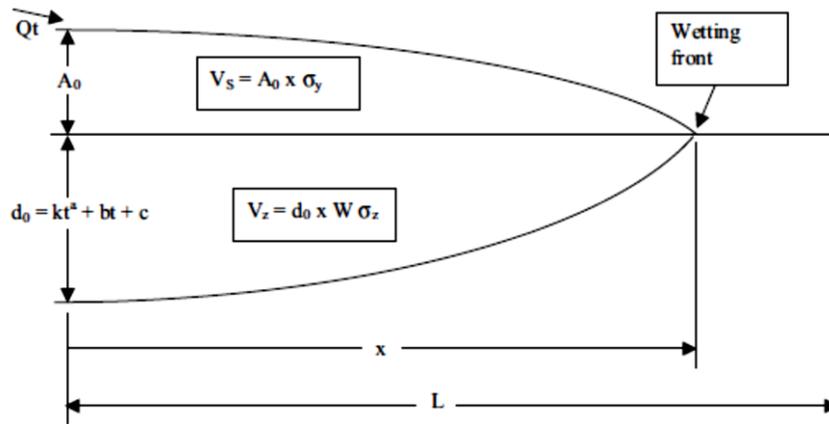
Fig. 4.1 Surface and subsurface storage volumes.

The total storage is the sum of surface and subsurface storage.

$$V_T = V_z + V_s \tag{4.4}$$

Where

V_z : Volume of subsurface storage (infiltrated volume), m³,
V_s : Volume of surface storage (water in furrow), m³,
V_T: Volume of total storage.

The inflow volume is calculated as the product of the inflow rate and the duration of application

$$V_{in} = 3.6Qt \tag{4.5}$$

Where

Q: Inflow rate, L/sec,
t: Time since irrigation water was turned on, hr,
V_{in}: Volume applied to furrow since water was turned on, m³.

The cross-sectional flow area at the upstream end of the furrow can be determined using Manning's equation. Furrow geometry is commonly characterized by either a power function or a trapezoidal shape.

The surface shape factor reflects the variation in the water profile along the furrow. If the furrow were filled uniformly to a constant depth throughout, the shape factor would equal 1.0. However, since the water depth typically decreases with distance downstream, the shape factor accounts for this variation.

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$$V_s = \sigma_y A_0 x \qquad (4.6)$$

Where

V_s : The surface storage volume,

σ_y : The surface shape factor,

A_0 : The cross-sectional flow area at the upstream end,

x : The distance to the wetting front.

The surface shape factor, σ_y, quantifies the change in water depth along the furrow. It adjusts the estimation of surface storage volume to account for the typically decreasing water depth downstream. If the furrow were filled uniformly to a constant depth, σ_y would be 1.0. In practice, since water depth decreases with distance, σ_y is less than 1 and is determined by the shape of the water profile.

To estimate σ_y, the water depth distribution along the furrow is often approximated by a known function, such as a linear or power function. By integrating the normalized depth profile over the length of the furrow, the shape factor is calculated as the ratio of the actual volume to the volume assuming uniform depth.

Mathematically, if $y(x)$ represents the normalized depth at a distance x along the furrow, then:

$$\sigma_y = \frac{1}{x_f} \int_0^{x_f} y(x)\,dx \qquad (4.7)$$

Where x_f is the distance to the wetting front.

If the water depth remains constant from the upstream end to the wetting front, the profile is uniform:

$$y(x) = 1$$

In this case, the shape factor is:

$$\sigma_y = 1$$

If the water depth decreases linearly from the upstream depth to zero at the wetting front:

$$y(x) = 1 - \frac{x}{x_f} \qquad (4.8)$$

Integrating over the furrow length:

$$\sigma_y = \frac{1}{x_f} \int_0^{x_f} \left(1 - \frac{x}{x_f}\right) dx = 0.5 \tag{4.9}$$

In furrow irrigation, subsurface storage is determined by infiltration characteristics and field geometry, as expressed in the following equation:

$$V_z = kt^a \sigma_z W x \tag{4.10}$$

Where

V_z : Subsurface storage volume (m³),
k : Empirical infiltration parameters,
t : Time (min or s),
σ_z: Subsurface shape factor,
W: Spacing between furrows (m),
x: Distance to the wetting front (m).

The total volume of infiltration along the furrow can also be expressed in terms of the infiltrated depth at the upstream end, d_0, as follows:

$$V_z = d_0 \sigma_z W x = \left(c + kt^a + \frac{h}{h+1} bt\right) \sigma_z W x \tag{4.11}$$

Where

h: Exponent in advance equation.
c: Constant or initial abstraction term (m).

The position of the wetting front during advance is modeled with the advance equation

$$t = sx^h \tag{4.12}$$

Where

s: Advance equation coefficient.

The subsurface shape factor is a function of a and h.

$$\sigma_z = \frac{h + a(h-1) + 1}{(1+a)(1+h)} \tag{4.13}$$

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The relationship between the advance exponent h, and the advance times to two points in the field can be found by taking the logarithm equation:

$$\log t = \log s + h \log x \tag{4.14}$$

Rearrange the equation and solve for log s

$$\log s = \log t - h \log x \tag{4.15}$$

For two $(\log x)$ and $(\log t)$ points, the slope of the line between them is h.

If the two distances (points x_1 and x_2) are the full length of the furrow, x_L, and half the length of the furrow, $x_{L/2}$, then x_2 is twice the length of x_1 and the following equation is used to calculate h.

$$h = \frac{\log t_2 - \log t_1}{\log x_2 - \log x_1} = \frac{\log\left(\frac{t_2}{t_1}\right)}{\log\left(\frac{x_2}{x_1}\right)} = \frac{\log\left(\frac{t_L}{t_{L/2}}\right)}{\log\left(\frac{2}{1}\right)} \tag{4.16}$$

The first step in the two-point volume balance method is to find the normal depth of flow at the given inflow rate. Next, an initial guess is made for t_L and $t_{L/2}$, and these values are used to make an initial calculation for h. Then, the subsurface shape factor is calculated, and the surface and subsurface storage are calculated. Finally, $t_{L/2}$ is adjusted based on the inflow volume and storage at time $t_{L/2}$, and t_L is adjusted based on the inflow volume and storage at time t_L. Next, the iteration procedure is repeated with the new estimates of t_L and $t_{L/2}$. During the iteration procedure, the time required for water to advance to a certain point in the furrow is adjusted by the ratio of inflow volume, V_{in}, to total storage volume, V_T.

$$t_{m+1} = t_m \left(\frac{V_T}{Q t_m}\right)^{1.4} \tag{4.17}$$

Where
   m: Iteration number.

Example 4.1
Determine the surface shape factor s and the advance exponent h, and plot the advance curve for a furrow that is 400 meters long with a spacing of 1 meter between adjacent furrows. The inflow rate is 1.0 L/s. The soil requires 10 hours to infiltrate 100 mm under

level conditions. The furrow has a bottom width of 0.1 m, side slope z = 2, Manning's roughness coefficient n = 0.05, and a longitudinal slope of 0.002 m/m. The upstream flow depth is 0.0533 m. Use an iterative approach with a convergence criterion of less than 1-minute difference between successive iterations.

Solution
Calculate cross-sectional area at the furrow inlet, A_0

$$A_0 = (b + zy)y = \left(0.1 + 2(0.0533)\right)(0.0533) = 0.011 \, m^2$$

Based on the empirical infiltration data presented in Table 4.2, if 100 mm of water infiltrates into level soil over a period of 10 hours, the corresponding parameters for the Kostiakov infiltration equation can be estimated as:

a = 0.46 and k = 34.4.

However, these values are applicable for level basin conditions where water uniformly covers the entire surface. In furrow irrigation, infiltration is concentrated along the wetted perimeter, and the infiltration rate must be adjusted to reflect the reduced contact area between water and soil.

Multiply the infiltration rate for a level field by the following ratio (wetted perimeter + 0.213 m) / (width between furrows).

$$P = b + 2y(1 + z^2)^{0.5} = 0.1 + 2(0.0533)(1 + 2^2)^{0.5} = 0.338m$$

Table 4.2 Infiltration parameters for various intake families using Merriam-Clemmens methods

Infiltration (100 mm)	Merriam and Clemmens (kt^a)		Infiltration (100 mm)	Merriam and Clemmens (kt^a)	
Time (hr)	k	a	Time (hr)	k	a
70.7	30.09	0.28	5.19	44.29	0.55
32.66	29.18	0.35	3.85	47.62	0.57
21.29	30.08	0.39	3.31	50.91	0.58
15.87	31.33	0.42	2.89	54.26	0.59
12.27	32.88	0.44	2.56	57.49	0.6
10.03	34.45	0.46	1.79	71.12	0.65
8.44	36.06	0.49	1.3	84.31	0.68
7.27	39.33	0.51	0.88	109.85	0.7
6.34	41.03	0.53	0.67	132.88	0.71

Adjust the infiltration rate for the equivalent furrow infiltration width.

$$\frac{0.338 + 0.213}{1} = 0.551$$

$$Adjusted\ k = (0.551)(34.4mm/hr) = 19.0mm/hr$$

$$d = 19.0k^{0.46}$$

This adjusted equation reflects infiltration under furrow conditions and is used for further analysis in volume balance and irrigation performance evaluations.

The design procedure starts with a guess for the advance time to ½ the field length and to the end of the field. For this example, we arbitrarily guess 100 and 250 minutes, respectively. For a flow rate of 1.0 L/s, calculate inflow volumes during these two periods.

$$V_{L/2} = Qt = 100min \left(\frac{60sec}{min}\right)\left(\frac{0.001m^3}{sec}\right) = 6m^3$$

$$V_L = Qt = 250min \left(\frac{60sec}{min}\right)\left(\frac{0.001m^3}{sec}\right) = 15m^3$$

$$h = \frac{\log\left(\frac{t_2}{t_1}\right)}{\log\left(\frac{x_2}{x_1}\right)} = \frac{\log\left(\frac{t_L}{t_{L/2}}\right)}{\log\left(\frac{2}{1}\right)} = \frac{\log\frac{250}{100}}{\log\frac{2}{1}} = 1.32$$

The next step is to calculate the subsurface shape factor.

$$\sigma_z = \frac{h + a(h-1) + 1}{(1+a)(1+h)} = \frac{1.32 + 0.46(1.32 - 1) + 1}{(1+0.46)(1+1.32)} = 0.73$$

The next step is to calculate subsurface storage. The infiltrated depths at the upper end of the field at 100 and 250 minutes are calculated.

$$d_{L/2} = 19.0 \left(\frac{100}{60}\right)^{0.46} = 24.0\ mm$$

$$d_L = 19.0 \left(\frac{250}{60}\right)^{0.46} = 36.6\ mm$$

Calculate subsurface storage at $t_{L/2}$ (time to reach L/2) and t_L

$$V_{z(L/2)} = d_0\sigma_z Wx = \left(\frac{24}{1000}\right)(0.73)(1.0)(200) = 3.50 \; m^3$$

$$V_{z(L)} = d_0\sigma_z Wx = \left(\frac{36.6}{1000}\right)(0.73)(1.0)(400) = 10.68 \; m^3$$

Calculate surface storage at $t_{L/2}$ and t_L.

$$V_{s(L/2)} = \sigma_y A_0 x = 0.75(0.011)(200) = 1.65 m^3$$

$$V_{s(L)} = \sigma_y A_0 x = 0.75(0.011)(400) = 3.29 m^3$$

Calculate total storage at $t_{L/2}$ and t_L.

$$V_{T(L/2)} = 1.65 + 3.5 = 5.15 m^3$$

$$V_{T(L)} = 3.29 + 10.68 = 13.97 m^3$$

The advance times are adjusted by following equation

$$t_{m+1} = t_m \left(\frac{V_T}{Qt_m}\right)^{1.4} = 100 \left(\frac{5.15}{6}\right)^{1.4} = 80.7 \; min$$

$$t_{m+1} = t_m \left(\frac{V_T}{Qt_m}\right)^{1.4} = 250 \left(\frac{13.97}{15}\right)^{1.4} = 226 \; min$$

The procedure is then repeated for the next iteration with $t_{L/2} = 81$ min, and $t_L = 226$ min. Subsequent iterations are shown in the following Table.

	t_1	t_2	h	σ_z	V_{in}-$t_{L/2}$	V_{in} -t_L	$V_{tL/2}$	V_{tL}
Initial guess	100	250	1.32	0.73	6	15	5.15	13.97
Iteration 1	80.7	226	1.49	0.747	4.84	13.6	4.90	13.7
Iteration 2	82.0	230	1.49	0.747	4.92	13.8	4.92	13.8
Iteration 3	82.1	231	1.49	0.747	4.92	13.8	4.93	13.8

4.4   Design Limitations

While furrow irrigation is one of the most widely used surface irrigation methods due to its simplicity, low capital cost, and adaptability to a variety of row crops, it is not without limitations. Understanding these constraints is essential to designing efficient systems and avoiding common pitfalls that reduce water application efficiency, uniformity, and crop productivity.

1. The spacing between furrows is influenced by several factors, including soil texture (which affects lateral water movement), the agronomic requirements of the crop, and the type of equipment used to construct the furrows. In general, narrower spacings such as 0.3 meters are recommended for coarse-textured soils with limited lateral water spread, while wider spacings up to 0.6 meters are suitable for fine-textured soils. In heavy clay soils, where lateral water movement is more pronounced, furrow spacings of up to 1.2 meters may be appropriate.
2. Furrows should ideally be as long as practical to minimize labor requirements and reduce overall system costs. However, excessively long furrows can compromise application efficiency and water distribution uniformity. In general, shorter furrows improve both efficiency and uniformity, making them preferable when labor is readily available or inexpensive, or when the water supply is limited. Soil texture also influences optimal furrow length, sandy (light) soils require shorter furrows due to faster infiltration rates, while clayey (heavy) soils can accommodate longer furrows.
3. The inflow rate (stream size) should not exceed the maximum non-erosive limit, which is typically established through field trials. Exceeding this limit can lead to furrow erosion and poor water distribution. A commonly used guideline for selecting safe stream sizes is given by the following empirical equation:

$$Q_{max} = \frac{K}{S_0} \tag{4.18}$$

Where

Q_{max}: Maximum non-erosive stream size (L/min)
S_0: Furrow slope in the direction of flow (%)
K: Constant.

The following table presents typical empirical values for the constant K, used to estimate the maximum non-erosive stream size in furrow irrigation systems based on soil type. These values serve as general guidelines and should be refined through local field observations and site-specific experience.

Table 4.3 Typical values of constant K for non-erosive stream size estimation

Soil Type	K (L/min per % slope)	Comments
Sand	20–30	High infiltration, low erosion risk, but shorter furrows needed.
Loamy Sand	15–25	Moderate infiltration and erosion potential.
Sandy Loam	10–20	Balance of infiltration and erosion sensitivity.
Loam	8–15	Suitable for most crops; moderate erosion control required.
Silt Loam	6–12	High erosion potential—use smaller streams.
Clay Loam	5–10	Slow infiltration; furrow erosion risk if slope is too high.
Silty Clay / Clay	3–8	High erosion sensitivity—very low stream sizes recommended.

Note:
- These values serve as starting points for design.
- Actual allowable stream sizes should be confirmed through field evaluation.
- Adjustments may be needed for surface crusting, vegetation cover, furrow compaction, or the use of surge irrigation.

4. Greater irrigation depths require longer contact times to allow water to infiltrate to the target root zone, compared to shallower irrigation depths.
5. Furrows constructed on steeper slopes can be designed longer due to the faster water advance. However, when the slope exceeds 0.5% (i.e., a 0.5 m drop over 100 m), stream sizes must typically be reduced to prevent erosion. As a result, shorter furrow lengths are often necessary under such conditions.

4.5   Design Considerations

Designing a furrow irrigation system involves evaluating the relationships between furrow length, inflow rate, inflow duration, soil characteristics, and performance metrics such as deep percolation, surface runoff, and application efficiency. These relationships are governed by a set of design equations and criteria based on soil intake behavior, furrow geometry, and irrigation requirements.

1. Opportunity time

The opportunity time (T_i), the duration water is available for infiltration at any point along the furrow is calculated as:

$$T_i = T_a - T_x \tag{4.19}$$

Where

T_a: total inflow (application) time (min)
T_x: time for the advance front to reach a specific point along the furrow (min)

For 100% adequacy, the total inflow time should equal the time required to infiltrate the net depth plus the time for water to advance the full length of the furrow:

$$T_a = T_n + T_L \tag{4.20}$$

Where

T_n: time required for the soil to absorb the net irrigation depth,
T_L: advance time to reach the end of the furrow (m).

2. Gross application depth

The gross depth of application (di) the total depth of water applied is calculated using:

$$di = 60 \times q \times T_a/(sf \times L) \tag{4.21}$$

Where

q: furrow inflow rate (m^3/s)
sf : furrow spacing (m)
L: furrow length (m)
T_a: total inflow time (min).

3. Infiltration depth at furrow ends

The equivalent infiltration depth at the furrow inlet (d_0) can be estimated from:

$$d_0 = K(T_n + T_L)^N/sf \tag{4.22}$$

Where

K: empirical infiltration constants (e.g., from Kostiakov or NRCS methods)

Similarly, the intake depth at the end of the furrow (d_L) depends on the opportunity time at that location:

- For 100% adequacy: $d_L \geq dm$
- For inadequate irrigation less than (100%): $d_L < dm$

4. Average Infiltration Depth

The average infiltration depth across the furrow (d_a) is given by:

$$d_a = \frac{(d_0 + d_L)}{2} \tag{4.23}$$

5. Runoff and deep percolation

The expected average depth of runoff (d_r) can be estimated by use of the equation:

$$d_r = di - d_a \tag{4.24}$$

Deep percolation (d_p) is the average depth of irrigation water that infiltrates the soil in excess of the design application depth. Deep percolation is determined from the equation:

$$d_p = d_a - dm \tag{4.25}$$

This framework allows the designer to predict system performance and make informed decisions to balance efficiency, uniformity, and adequacy in furrow irrigation.

4.6   Runoff Control Techniques

Runoff in furrow irrigation occurs when the inflow rate and duration exceed the soil's infiltration capacity or the crop's root zone requirements. Uncontrolled runoff can lead to water losses, soil erosion, nutrient leaching, and environmental pollution. Therefore, effective runoff control is essential for improving irrigation efficiency and sustainability. Several practical and engineering techniques are available to manage or utilize runoff.

4.6.1   Cutback Inflow

Cutback inflow is a practical and effective technique used in surface irrigation particularly in furrow system to reduce runoff and improve water application efficiency. The method involves reducing the flow rate after the advance phase is completed, i.e., once the water front has reached the end of the field.

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The goal is to minimize the time water takes to reach the end of the furrow while avoiding excessive runoff and deep percolation. Once the entire furrow is wetted, the inflow rate is cut back to a smaller value that is approximately equal to the soil's average infiltration rate.

Timing of cutback (T_a and T_c)

- Total inflow time is denoted as T_a.
- Cutback time (T_c) is when the inflow rate is reduced.

Practical guidance suggests:

$$T_c = \frac{T_a}{2}, \quad \frac{T_a}{3}, \quad \frac{T_a}{4}$$

Condition:

$$T_c \geq T_L$$

Where T_L is the advance time to the end of the furrow. This ensures that the cutback occurs only after the water has reached the entire furrow length.

The volume of runoff saved by the cutback method is estimated by:

$$V = (q_{in} - q_f)(T_a - T_c) \tag{4.26}$$

Where

  q_{in}: Initial furrow stream
  q_f: Final furrow stream (after reduction)

The average intake depth (I_f) under cutback-inflow conditions is given by:

$$I_f = \int_0^L E \left[T_c - \left(\frac{x}{a}\right)^{1/b} \right]^r . dx \tag{4.27}$$

Where

  E: Average intake when opportunity time is 1 minute
  x: Distance along the furrow (m)
  a, b: advance equation parameters
  r: slope of the intake-time curve (typically 0 to -1)
  L: total furrow length (m)

To simplify integration, divide the furrow length into four equal segments and apply the trapezoidal approximation:

$$I_f = \frac{L}{4}\left[\frac{1}{2}I_f(0) + I_f\left(\frac{L}{4}\right) + I_f\left(\frac{L}{2}\right) + I_f\left(\frac{3L}{4}\right) + \frac{1}{2}I_f(L)\right] \tag{4.28}$$

The five values within the bracket represent the intake rate at the distance zero, one quarter, one half, three quarters, and the entire length of the furrow, respectively, at the reduction time T_c.

The volume of irrigation water (V_c) for each furrow is calculated as follows:

$$V_c = q_{in}.T_c + q_f(T_a - T_c) \tag{4.29}$$

Each furrow shall be provided with an initial discharge (q_{in}) of a time T_c, then shall be provided with a final discharge (q_f) of a time of ($T_a - T_c$).

For systems with limited available discharge (Q_a), the irrigated furrows are grouped based on stream size and timing:

- Number of furrows irrigated initially:

$$N_1 = Q_a/q_{in} \tag{4.30}$$

- Number of furrows irrigated after cutback:

$$N_2 = \frac{(Q_a - N_1 \times q_f)}{q_{in}} \tag{4.31}$$

Implementation Notes

- Monitoring advance time (T_L) is essential to ensure correct timing of cutback.
- Flow control mechanisms (e.g., gated pipes, valves) must be installed to allow quick adjustment of discharge.
- The design should balance between water savings and operational simplicity, especially in large irrigation projects.

The cutback method, when properly implemented, can significantly improve water use efficiency, reduce runoff volume, and increase uniformity in furrow irrigation systems, making it a valuable tool for water saving irrigation design.

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Example 4.2

Given:

The furrow stream = 0.8 L/sec

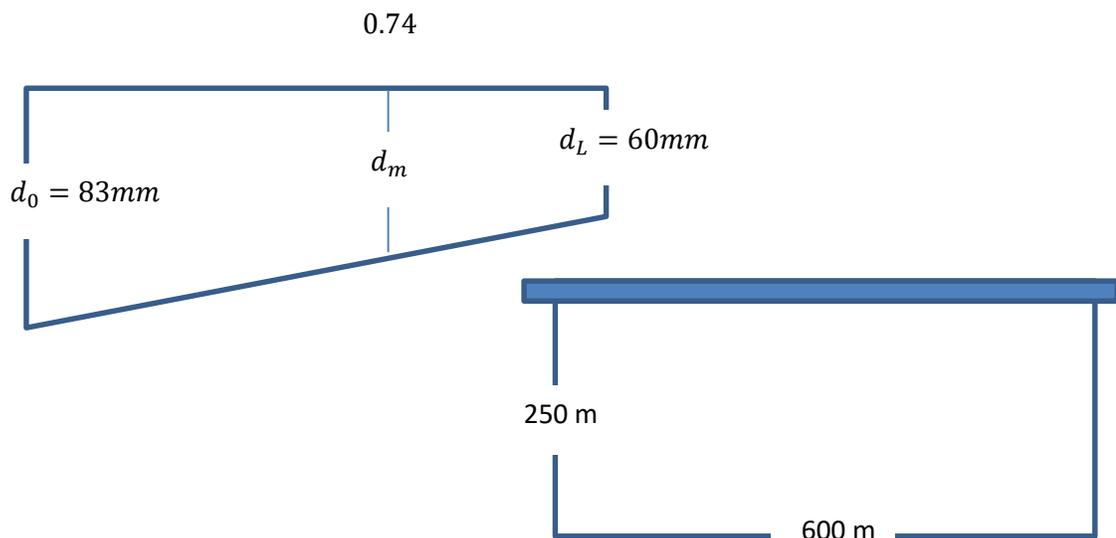The relationship between the distance along the furrow length (x) and the time (t) ($x = 9.5t^{0.555}$)

Equivalent infiltration rate $I_e = 90t^{-0.5}$, $I_e \left(\frac{mm}{hr}\right), t(min)$

Depth of water lost to consumptive use by crops = 60 mm

$sf = 1m$

Find:

1. The equation of equivalent infiltration depth
2. Find the following for two cases, the length of furrow (125m & 250m)
   a. T_a
   b. di
   c. d_0
   d. d_r
   e. d_p
   f. Efficiency of irrigation

3. Final furrow stream, reduction time, efficiency of irrigation when using the runoff control technique (cutback inflow) for furrow length is equal to 250 m.
4. The available irrigating stream ($Q_a = 24L/sec$), find the time for irrigating the field
5. If $d_L = 60mm$ and adequacy of irrigation (74%), find
   a. II, for dc (6mm/day)
   b. Efficiency of irrigation

0.74

$d_L = 60mm$

d_m

$d_0 = 83mm$

250 m

600 m

Solution

1. $d_e = \int 90t^{-0.5}.dt \quad \rightarrow 180t^{0.5} \rightarrow 3t^{0.5} \ (mm/min)$
2.
   a. To find the time required to infiltrate the net required depth:
      $60 = 3t^{0.5} \rightarrow T_n = 400min$
      From the empirical relationship between distance along the furrow length x and advance time t $\ (x = 9.5t^{0.555})$
      $T_{250} = 362 \ min$
      $T_{125} = 104 \ min$
      Thus, the total infiltration opportunity time at each location:
      $\therefore T_{a(125)} = 504 \ min$
      $\therefore T_{a(250)} = 762 \ min$
   b. Infiltrated depth $di = 60 \times q \times T_a/(sf \times L)$
      $di(125) = 60 \times 0.8 \times \dfrac{504}{125} = 194 \ mm, \ \ di(250) = 146 \ mm$
   c. $d_0 = 3 \times 504^{0.5} = 67 \ mm, \ \ d_o(250 \ m) = 83mm$
      Average of equivalent infiltration depth
      $d_a = (d_0 + d_L)/2$
      $d_a(125) = \frac{(67+60)}{2} = 63.5 \ mm, \ \ d_a(250) = \frac{(83+60)}{2} = 71.5 \ mm$
   d. $d_r = di - d_a$
      $d_r(125) = 194 - 63.5 = 130.5 \ mm$
      $d_r(250) = 146 - 71.5 = 74.5 \ mm$
   e. $d_p = d_a - dm$
      $d_p(125) = 63.5 - 60 = 3.5 \ mm$
      $d_p(250) = 71.5 - 60 = 11.5 \ mm$
   f. Application efficiency $Ea\% = \dfrac{dm}{di} \times 100\%$
      $Ea\%(125) = \dfrac{60}{194} \times 100\% = 31\%$
      $Ea\%(250) = \dfrac{60}{146} \times 100\% = 41\%$

Longer furrow length (250 m) resulted in slightly better application efficiency (41%) compared to the shorter one (31%).

3. $T_c = \frac{1}{2}T_a = 381 \ min > T_{250} = 362min$
   $I_f = 90t^{-0.5}$
   Divide the length of the furrow length into four equal parts:
   (x=0, 62.5, 125, 187.5, 250)
   Determine the opportunity time at $(T_c = 381 \ min)$ in these points

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In surface irrigation systems, opportunity time refers to the duration during which water is in contact with the soil surface at a given point along the furrow. It is a critical parameter for estimating infiltration depth. To determine the opportunity time at various points along the furrow when the cut-off time T_c is known, the following approach is used:

Divide the furrow length into equal segments. In this example, the furrow is 250 meters long and is divided into four equal sections: (x=0, 62.5, 125, 187.5, 250)
Use the advance-time relationship to estimate the advance time t to each segment:

$$T_x = \left(\frac{x}{9.5}\right)^{1/0.555} \rightarrow T_x = (0, 29.8, 103.9, 215.7, 362)m$$

This formula applies because water arrives at location x at time T_x and remains there until cut-off at T_c, resulting in an opportunity time of $T_c - T_x$.

If $T_c \leq T_x$, then water has not reached that point by the time of cut-off, and the opportunity time is zero.

$$I_f(0) = 90 \ (381)^{-0.5} = 4.61 \ L/hr$$
$$I_f(62.5) = 90 \ (381 - 29.8)^{-0.5} = 4.8 \ L/hr$$
$$I_f(125) = 5.4 \ L/hr$$
$$I_f(187.5) = 7.0 \ L/hr$$
$$I_f(250) = 20.64 \ L/hr$$
$$I_f = \frac{L}{4}\left[\frac{1}{2} \times 4.61 + 4.8 + 5.4 + 7.0 + \frac{1}{2} \times 20.64\right] = 1864\frac{L}{hr} = 0.52 \ L/sec$$
$$\therefore q_f = 0.52 \ L/sec$$
$$V_c = q_{in} \times T_c + q_f \times (T_a - T_c)$$
$$= 60 \times [0.8 \times 381 + 0.52 \times (762 - 381)] = 30175 \ liter$$
$$di = \frac{V_c}{sf \times L} = \frac{30175}{1 \times 250} = 121 \ mm$$
$$Ea\% = \frac{60}{121} \times 100\% = 50\%$$

4. In surface irrigation planning, particularly for furrow irrigation systems, it is essential to determine how many furrows can be irrigated at the same time given the available water supply. This ensures efficient use of water and uniform irrigation across the field.
   The total flow rate delivered to the field from the source ($24L/sec$), the flow rate assigned to each furrow for proper infiltration ($0.8L/sec$).
   The number of irrigated furrows at the same time
   $$Number \ of \ furrows = \frac{Q_a}{q} = \frac{24}{0.8} = 30$$

By dividing the total available discharge by the required discharge per furrow, we determine the number of furrows that can be irrigated simultaneously. This is a key step in field layout design, labor planning, and irrigation scheduling.

Once the number of furrows that can be irrigated simultaneously is known, the total time needed to irrigate the entire field can be calculated using the following steps: The total number of furrows in the field (600)

The time for irrigating the field is equal to

$$\frac{total\ number\ of\ furrows}{the\ number\ of\ irrigated\ furrows\ at\ the\ same\ time} \times T_a$$
$$\times \frac{24}{the\ number\ of\ working\ hours\ per\ day}$$
$$= \frac{600}{30} \times \left[762 \times \left(\frac{1}{24 \times 60}\right)\right] \times \frac{24}{24} = 10.58\ day$$

The field is irrigated in 20 separate sets of furrows, each requiring approximately 0.53 days to complete. With continuous 24-hour irrigation, the entire field will be irrigated in 10.58 days. This method ensures accurate scheduling of irrigation rotations based on stream availability and furrow design.

Once the volume of water applied per furrow is known, the total water volume required for the field can be calculated by multiplying by the number of furrows. Compute the total volume for the entire field:

$$= 600 \times 30175 = 18105000\ liter$$

When cutback inflow is used, water is applied to all furrows in a controlled, continuous manner. The total irrigation time is determined by dividing the total irrigation water volume by the available flow rate:

$$= \frac{the\ total\ volume\ of\ irrigation\ water}{the\ available\ irrigating\ flow} = \frac{18105000}{24} = \frac{1}{24 \times 3600}$$
$$= 8.73\ day$$

Using a continuous and efficient cutback flow system:

- The entire field (600 furrows) can be irrigated in about 8.73 days,
- Compared to 10.58 days with fixed inflow,
- This results in shorter irrigation duration and improved water use efficiency.

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5. In surface irrigation, especially furrow systems, water infiltrates unevenly along the furrow length. The infiltration depth is greatest at the upstream end and decreases toward the downstream end.

- Upstream depth at the inlet: d_0=83 mm
- Downstream depth at the end: d_L=60 mm
- Proportion of furrow length that received full irrigation: 74% (0.74 of the length)

$$dm = d_L + (d_0 - d_L) \times (1 - 0.74) = 60 + (83 - 60) \times (1 - 0.74) = 66mm$$
$$dm \times 0.74 + \frac{1}{2} \times (dm + d_L) \times (1 - 0.74) = 65.22mm$$

The efficiency of irrigation without cutback inflow

$$Ea\% = \frac{65.22}{146} \times 100\% = 44.7\%$$

The efficiency of irrigation with cutback inflow
$$E\% = \frac{65.22}{121} \times 100\% = 54\%$$

Using cutback inflow significantly improves irrigation application efficiency.

4.6.2   Irrigation Runoff Recovery System

1. Purpose of a runoff recovery system

The key objectives of an irrigation runoff recovery system in furrow irrigation are:

- To capture the excess water that leaves the field as runoff.
- To reuse this water for irrigation, either immediately or later.
- To reduce water waste, improving overall irrigation efficiency.
- To minimize soil erosion and nutrient loss at the tail end of furrows.

2. System Components (Fig. 4.2)

A typical runoff recovery system for furrow irrigation consists of:

A. Collection ditch or sump:
- Located at the tail end of the furrows.

- Captures excess water flowing out of furrows.

B. Storage pond or sump basin:
- Stores collected runoff temporarily.
- Should be lined to reduce seepage.

C. Return pump and pipeline:
- Transfers water from the storage basin back to the irrigation supply.
- Pumping may be manual, electric, or solar-powered.

D. Control structures:
- Include check gates, valves, and overflow structures to manage water levels.

E. Optional sediment trap:
- Reduces sediment transport into the storage pond or pump system.
- Protects equipment and prevents clogging.
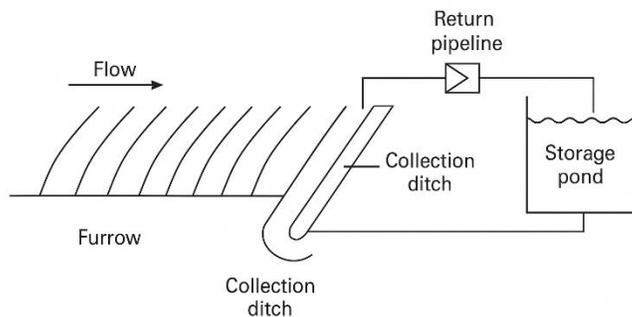
Fig. 4.2 Irrigation runoff recovery system

3. Design Considerations

Designing an effective runoff recovery system requires careful consideration of several field characteristics. Field slope and furrow length play a crucial role, as steeper slopes and longer furrows tend to generate greater runoff volumes, thereby affecting the required capacity of the recovery basin. Additionally, soil type and infiltration characteristics significantly influence runoff behavior; sandy soils typically have higher infiltration rates and thus produce less runoff, whereas clay soils have lower infiltration capacities, resulting in greater runoff accumulation. These factors must be evaluated together to ensure the recovery system is appropriately sized and efficiently integrated into the furrow irrigation design.

The number of furrows irrigated in the first group, denoted by N_1, is determined by the ratio of the available inflow rate (Q_a) to the flow rate per furrow (q):

$$N_1 = Q_a/q \tag{4.32}$$

The runoff volume generated by this first group, V_{rw}, is calculated as:
$$V_{rw} = N_1 . V_r \tag{4.33}$$

where:

V_r: volume of runoff per individual furrow (m³), calculated using a water balance approach.

The required return flow rate from the recovery system (e.g., via pump), denoted as Q_p, is:

$$Q_p = V_{rw}/T_a \tag{4.34}$$

For the second and subsequent groups, the number of furrows that can be irrigated increases due to the contribution of recovered water. The number of furrows in these groups, N_i, is given by:

$$N_i = (Q_a + Q_p)/q \tag{4.35}$$

This iterative approach allows efficient use of recovered tailwater to expand the irrigated area in subsequent cycles, enhancing overall water use efficiency.

Example 4.3
A field is irrigated using furrow irrigation, with each furrow spaced 1 meter apart. The field is 250 meters long and 60 meters wide, and the soil is clayey with low infiltration characteristics. Water is applied at a rate of 0.3 liters per second to each furrow, and the total available inflow to the field is 9 liters per second. The irrigation duration for each group of furrows is 2.5 hours, and the estimated runoff volume per furrow is 0.08 cubic meters. If the runoff is collected and returned to the system with a recovery efficiency of 90%, calculate the number of furrows in the first group, the total runoff volume generated, the required return flow rate by pump, and the number of furrows that can be irrigated in the second group using the recovered water.

Solution

Step 1: Number of furrows in the first group N_1

$$N_1 = \frac{Q_a}{q} = \frac{9}{0.3} = 30 \ furrows$$

Step 2: Total runoff volume from the first group V_{rw}

$$V_{rw} = N_1.V_r = 30 \times 0.08 = 2.4m^3$$

Step 3: Apply recovery efficiency to get recovered volume

$$V_{recovered} = \eta_r.V_{rw} = 0.9 . 2.4 = 2.16m^3$$

In a furrow irrigation system with runoff recovery, not all the water that runs off at the end of the field can be fully captured and reused. Some water may be lost due to seepage, evaporation, spillage, or inefficiencies in the pumping system. To account for these unavoidable losses, we apply a factor called recovery efficiency.

The Recovery Efficiency η_r is the fraction (or percentage) of the runoff water that can be effectively collected, stored, and reused.

$$\eta_r = \frac{\text{Volume of runoff actually recovered and reused}}{\text{Total runoff volume gerenerated}}$$

Step 4: Return flow rate from pump Q_p

$$Q_p = \frac{V_{recovered}}{T_a} = \frac{2.16}{2.5} = \frac{0.864m^3}{hr} = 0.24L/sec$$

Step 5: Number of furrows in the second group N_i

$$N_i = \frac{(Q_a + Q_p)}{q} = \frac{9 + 0.24}{0.3} = 30.8$$

Number of furrows in the second group: approximately 31 furrows.

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5. Sprinkler Irrigation Design

5.1 Basic Concept of Sprinkler Irrigation

Sprinkler irrigation is a pressurized irrigation method that simulates natural rainfall by distributing water through a network of pipes and spraying it into the air using specially designed nozzles or sprinklers. It is widely used across different soil types, terrains, and crop varieties due to its versatility and efficiency. This method allows for controlled and uniform application of water, making it suitable for both small and large-scale agricultural operations. Sprinkler systems can be permanent, semi-permanent, or portable, and are often chosen for areas where surface irrigation is impractical or where water conservation is a priority.

5.2 Advantages of Sprinkler Irrigation

Sprinkler irrigation offers several advantages that make it a popular and efficient method of water application in agriculture. One of its key benefits is the ability to distribute water uniformly across the entire field, ensuring consistent crop growth and minimizing areas of over- or under-irrigation. It is well suited for a variety of terrains, including flat, sloped, or uneven land, where traditional surface irrigation may not be practical. Sprinkler systems promote efficient use of water by reducing losses due to deep percolation and surface runoff when properly managed. They are adaptable to a wide range of soil types and crops, particularly vegetables, orchards, and turf. Additionally, sprinkler irrigation reduces the risk of soil erosion due to its gentle and controlled water application. These systems can also be automated, saving labor and allowing precise scheduling of irrigation. Moreover, fertilizers and agrochemicals can be conveniently applied through the system (a practice known as fertigation), and sprinklers can serve secondary purposes such as cooling crops and protecting them from frost during cold conditions.

5.3 Limitations of Sprinkler Irrigations

Despite its many advantages, sprinkler irrigation has several limitations that must be considered when selecting and designing a system. One of the primary drawbacks is the high initial cost of installation, which includes expenses for pumps, pipes, sprinkler heads, and control systems. This can be a significant barrier for small or resource-limited farmers.

Another limitation is the sensitivity to wind. Since water is sprayed into the air, wind can distort the spray pattern, leading to uneven water distribution and potential crop stress. Additionally, evaporation losses can be significant, especially in hot, dry, or windy conditions, reducing overall water use efficiency.

Sprinkler systems require a reliable and clean water source, as dirty or debris-laden water can clog nozzles and pipes, affecting performance and increasing maintenance needs. These systems are also energy-intensive, particularly for larger fields, due to the need for pressurized water, which raises operating costs.

In some cases, the wetting of foliage caused by sprinklers can promote fungal diseases, especially in crops that are sensitive to high humidity. Furthermore, in heavy clay soils with low infiltration rates, sprinkler application must be carefully managed to avoid surface runoff or waterlogging.

Overall, while sprinkler irrigation is an effective and modern irrigation method, its successful use depends on proper design, maintenance, and management, as well as suitability to the specific environmental and crop conditions.

5.4   Types of Sprinkler Irrigation Systems

Sprinkler irrigation systems come in various types, each designed to suit specific land conditions, crop requirements, and levels of mobility or automation. The main types include:

5.4.1   Stationary Sprinkler Irrigation Systems

Stationary sprinkler irrigation systems (Fig. 5.1), also known as solid set systems, are systems where the mainlines, laterals, and sprinkler heads are permanently or semi-permanently installed in a fixed position on the field. Once installed, the components remain in place throughout the irrigation season, and sometimes year-round.

Key Characteristics:
- Fixed location**:** Sprinklers are not moved during irrigation; they are spaced uniformly to cover the field.
- Automatic or manual operation**:** Can be controlled manually or automated using timers or sensors.
- Common in permanent crops**:** Ideal for orchards, vineyards, turf grass, and nurseries where the planting layout remains unchanged.
- Buried or surface pipelines**:** Mainlines and laterals may be buried to avoid obstruction or placed on the surface for easy access and maintenance.

Advantages:
- Labor-saving: Requires minimal labor after installation since no relocation is needed during operation.
- Suitable for automation: Easily integrated with timers, sensors, or remote-control systems.

- Uniform coverage: Designed for precise spacing and nozzle selection to ensure even water distribution.
- Low disturbance: No need to move equipment, which reduces damage to crops or soil.

Disadvantages:
- High initial cost: Due to the permanent nature and material used for pipes and sprinklers.
- Limited flexibility: Not easily adapted to crop rotation or field layout changes.
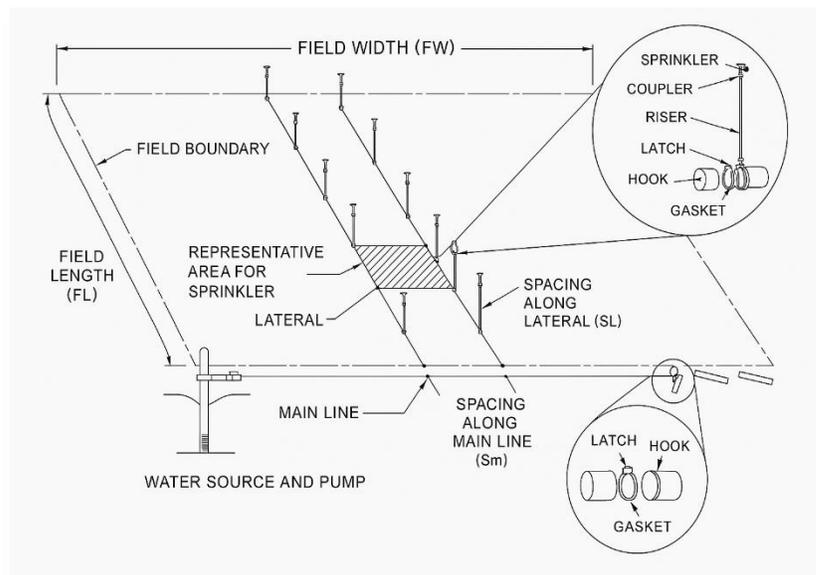- Risk of damage: Exposed parts may be damaged by machinery, animals, or weather.

Fig. 5.1 Components and general layout of sprinkler irrigation systems.

5.4.2 Continuous-Moved Sprinkler Systems

Continuous-moved sprinkler systems are irrigation systems in which the sprinkler line or the entire system moves continuously while water is being applied. This movement ensures uniform water distribution over a large area, and these systems are particularly suited to large-scale agricultural operations with relatively uniform terrain. There are two main types of continuous-moved sprinkler systems:

1. **Center Pivot Systems**

- Description: A center pivot system consists of a long sprinkler pipeline mounted on wheeled towers that pivot around a central fixed point (Fig. 5.2). Water is supplied from a central source and distributed along the pipeline.

- Movement**:** The system rotates slowly around the pivot point, irrigating a circular area.

- Coverage**:** Typically covers 50–130 hectares per system.

- Advantages**:**

a. Highly automated and labor-saving.
b. Efficient water application with low energy requirements per unit area.
c. Suitable for a wide variety of crops.

- Disadvantages**:**

a. Only irrigates circular areas—leads to under-irrigated corners in square fields.
b. Initial investment is relatively high.
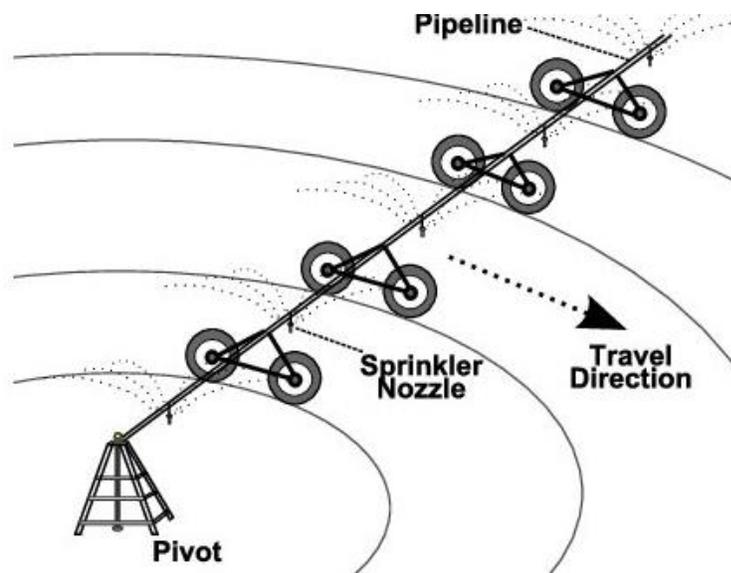c. Not ideal for irregular or hilly terrains**.**

Fig. 5.2 Basic components of a center pivot (CP) system.

2. **Lateral Move Systems (Linear Move or Continuous Move Laterals)**

- **Description**: These systems consist of a long sprinkler line mounted on wheeled or tracked towers that move laterally across the field in a straight line (Fig. 5.3).

- **Water Supply:** Water is supplied via a flexible hose or a ditch that runs parallel to the movement of the system.

- **Coverage:** Rectangular fields ideal for fields unsuited to circular coverage.

- **Advantages:**

a. Uniform water distribution over rectangular areas.
b. Can irrigate corners missed by center pivots.

- **Disadvantages**:

a. Requires a more complex water supply arrangement.
b. Terrain must be relatively flat and uniform.
c. Higher maintenance and supervision needs compared to center pivots.

Fig.5.3 Lateral move irrigation systems

5.5   Basic Parts of Sprinkler Irrigation System

A sprinkler irrigation system is composed of several key components that work together to deliver water uniformly over a crop field in the form of artificial rainfall. Understanding these basic parts is essential for designing, operating, and maintaining an efficient system.

5.5.1 Major Pipes Line

The major pipeline in a sprinkler irrigation system refers to the mainline and sometimes the sub-mainline that conveys water from the source (such as a pump or reservoir) to the lateral lines and sprinklers. This pipeline is a critical component as it forms the backbone of the entire irrigation network.

Key Characteristics:

- **Function**: Transports water under pressure from the pump to different parts of the field.
- **Material**: Typically made from PVC, HDPE, galvanized steel, or aluminum chosen based on pressure requirements, durability, and cost.
- **Layout**: Usually laid either underground (to protect from damage and UV exposure) or above ground (for easy inspection and maintenance).

Mainline Types:

1. **Permanent Mainlines**: Buried and used in permanent installations; durable and low-maintenance.
2. **Portable Mainlines**: Made of lightweight materials (e.g., aluminum) for seasonal or temporary use.

Design Considerations:

- Must withstand operating pressure without excessive head loss.
- Proper diameter selection is critical to minimize friction losses and ensure uniform water delivery.
- Fitted with valves, tees, and connectors to manage flow to laterals and sprinkler heads.

5.5.2 Lateral lines

Lateral lines are secondary pipelines in a sprinkler irrigation system that branch off from the mainline or sub-mainline and deliver water directly to the sprinkler heads. They play a crucial role in distributing water uniformly across the irrigated area.

Key Characteristics:

- **Function**: Carry pressurized water from the mainline to the individual sprinklers spaced along their length.
- **Material**: Commonly made of lightweight materials such as polyethylene (PE), PVC, or aluminum, allowing flexibility and easy handling.
- **Placement**: Can be either portable (moved between irrigations) or permanent (buried or fixed in place).

Design Features:
- **Pipe Diameter**: Selected based on flow rate, length, and allowable pressure loss to ensure uniform pressure at all sprinkler nozzles.
- **Spacing**: Sprinkler spacing along the lateral lines is carefully designed to achieve proper overlap and uniform coverage.
- **End Equipment**: Often include end plugs, drains, or valves to facilitate flushing and maintenance.

Proper design and layout of lateral lines are essential for:
- Maintaining uniform pressure distribution.
- Minimizing energy losses.
- Ensuring efficient and consistent water application over the field.

5.5.3   Sprinklers

Sprinklers are the final and most critical components in a sprinkler irrigation system. They are the devices that distribute water into the air in the form of droplets, simulating natural rainfall. Proper selection and placement of sprinklers ensure uniform water application and efficient use of water resources.

Key Characteristics:
- **Function**: Convert pressurized water into a spray or mist to irrigate crops evenly.
- **Operation**: Water is forced through nozzles under pressure, creating a spray pattern over a defined area.
- **Types**:
    1. **Fixed Spray Sprinklers** – Provide a constant spray pattern; suitable for small areas (Fig. 5.4).
    2. **Rotating Head Sprinklers** – Use rotating nozzles or arms to cover larger areas (Fig. 8.4).
    3. **Impact Sprinklers** – Use a spring-loaded arm to rotate the head; widely used in field crops (Fig. 5.5).
    4. **Gear-Driven and Pop-Up Sprinklers** – Common in turf and landscaping for precise, low-noise operation.

Design Considerations:
- **Spray Radius and Pattern**: Determined by nozzle size, pressure, and sprinkler type.
- **Spacing**: Designed to ensure proper overlap of spray patterns to avoid dry spots or overwatering.
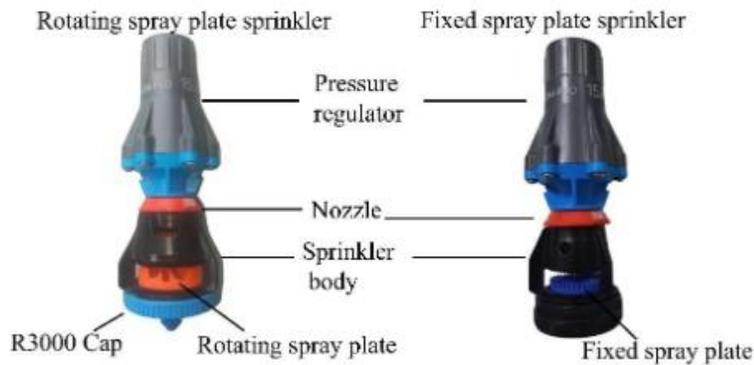- **Pressure Requirements**: Each sprinkler has an optimal operating pressure for efficient performance.

Fig.5.4 Fixed spray and rotating head sprinklers   **Fig.5.5** Impact sprinklers

5.5.4 Pump

The pump is a vital component of a sprinkler irrigation system, responsible for supplying the necessary pressure to move water through the pipelines and operate the sprinklers effectively. It ensures that water is delivered from the source whether a well, river, reservoir, or tank to the irrigation network at a consistent and adequate pressure.

Key Characteristics:

- **Function**: Lifts and pressurizes water to overcome elevation differences and friction losses in the system.
- **Types of Pumps Commonly Used**:
    1. **Centrifugal Pumps** – Most widely used; suitable for low to moderate heads.
    2. **Turbine Pumps** – Used for deep well applications; high efficiency.
    3. **Submersible Pumps** – Installed underwater, ideal for deep wells or tanks.
    4. **Booster Pumps** – Used to increase pressure within a system when the main pump is insufficient.

Selection Criteria:

- **Flow Rate (Discharge)**: Must match the system's total water requirement.
- **Total Dynamic Head (TDH)**: Includes suction lift, elevation difference, friction losses, and sprinkler pressure.
- **Power Source**: Can be electric, diesel, petrol, or solar-powered depending on availability and field conditions.

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5.6   Secondary Parts of Sprinkler Irrigation System

5.6.1   Sprinkler Riser

The sprinkler riser is a vertical pipe or extension that connects the sprinkler head to the lateral pipeline. Its primary function is to elevate the sprinkler above the crop canopy or ground surface, allowing for unobstructed and uniform water distribution across the irrigated area. Risers come in various lengths, materials (such as PVC, galvanized steel, or polyethylene), and configurations (rigid or flexible), depending on the type of crop, sprinkler system design, and field conditions.

5.6.2   Laying and Joining Fitting

Laying and joining fittings are essential components in a sprinkler irrigation system, used to assemble, connect, and secure the various segments of the pipeline network. These fittings facilitate the proper layout of mainlines, submains, and laterals, ensuring leak-proof connections and smooth water flow throughout the system.

These fittings come in various shapes, sizes, and materials commonly PVC, HDPE, or metal depending on the pipe type, pressure requirements, and field layout.

5.6.3   Valves

Valves are critical control components in a sprinkler irrigation system, used to regulate, direct, and shut off the flow of water within the pipeline network. They provide flexibility in system operation by allowing irrigation scheduling by zone, maintenance of individual sections, and protection against system damage. Valves are installed at key locations such as mainlines, submains, and lateral connections and are available in both manual and automatic types, depending on the level of control required.

5.6.4   Meters and Regulators

Meters and regulators are essential secondary components in sprinkler irrigation systems, used to monitor and control water flow and pressure to ensure efficient and uniform irrigation. These devices play a crucial role in system management, water conservation, and maintaining the operational integrity of the irrigation network.

5.7   Sprinkler Spacing

To achieve a uniform application of water over the irrigated area, the sprinklers must be spaced properly. The two types of sprinkler spacing patterns most commonly used are square spacing and triangular spacing. A rectangular pattern may also be used in some instances.

Fig. 5.6 illustrates a typical sprinkler irrigation layout showing the spacing between sprinklers and laterals. Sprinklers are arranged in a grid pattern where **S** represents the

spacing between sprinklers along the lateral, and **L** represents the spacing between laterals. A pump supplies pressurized water to the mainline, which distributes it to the laterals. Each lateral contains several sprinkler heads spaced evenly to ensure uniform water application across the irrigated area. The layout highlights the importance of proper spacing (S and L) to achieve efficient and uniform irrigation coverage. This configuration can be adapted to square, rectangular, or triangular spacing patterns based on field conditions and design requirements.
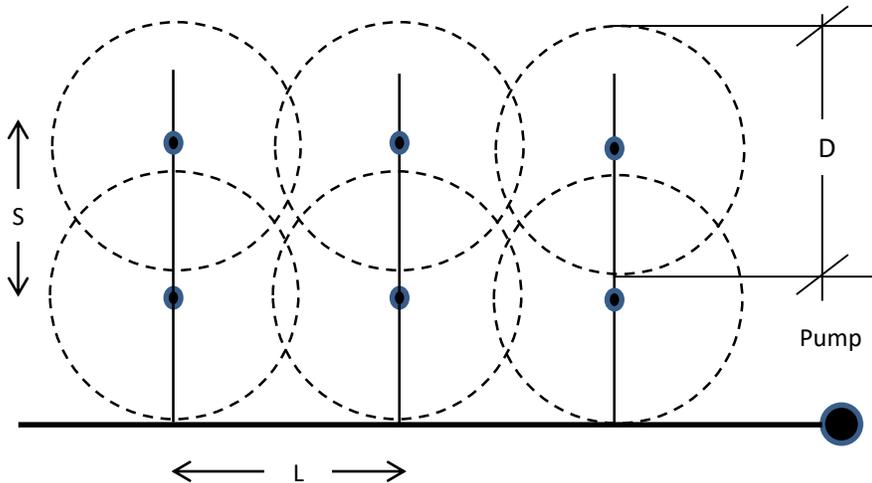

Fig. 5.6 Typical sprinkler irrigation layout showing sprinkler and lateral spacing (S and L)

Spacing Guidelines
- **Square Pattern**:
  S=L=50%−60% of the sprinkler's radius of throw.
- **Triangular Pattern**:
  S=L=60%−70% of the radius for more uniform coverage, especially in windy areas.

5.8   Sprinkler Arrangement

The arrangement of sprinklers in an irrigation system significantly influences water distribution uniformity, efficiency, and overall system performance. Proper layout ensures adequate overlap, minimizes dry spots, and reduces water losses due to wind drift and evaporation.

The primary objectives of sprinkler arrangement are to ensure uniform water application across the entire field and to optimize irrigation system efficiency. This involves minimizing water loss due to excessive overlap while also avoiding under-irrigated zones that could stress crops. A well-designed arrangement should be compatible with the field's shape, slope, and the specific crop being grown. Additionally, the layout must account for prevailing wind conditions, as wind can distort spray patterns and reduce uniformity.

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There are three primary sprinkler arrangement patterns (Fig. 5.7):

1. **Square Pattern:** Sprinklers are placed at equal distances along rows and columns forming squares.
   - **Advantages:** Simpler layout and ease of installation.
   - **Disadvantage:** Less efficient under windy conditions.
2. **Rectangular Pattern:** Sprinklers are placed in rows, with differing lateral and mainline spacing.
   - **Advantages:** Suitable for long, narrow fields.
   - **Disadvantage:** Can lead to uneven distribution if not properly designed.
3. **Triangular (Equilateral or Isosceles)** Pattern: Sprinklers are arranged at the vertices of equilateral or isosceles triangles.
   - **Advantages:** Provides the most uniform water distribution.
   - **Disadvantage:** Slightly more complex to install and design.
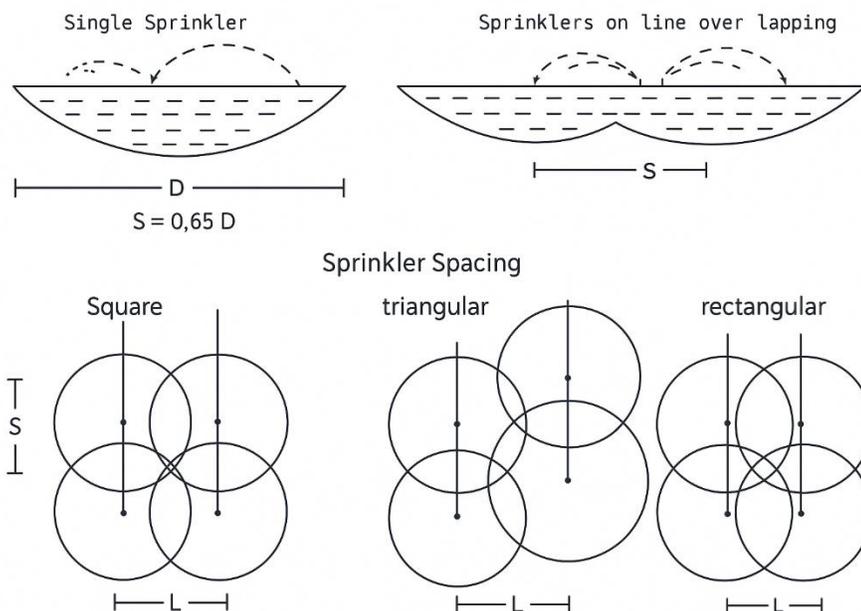

Fig. 5.7 Sprinkler water distribution patterns and spacing arrangements

In a triangular sprinkler arrangement (Fig. 5.8), sprinklers are spaced such that each sprinkler is equidistant from three adjacent units, forming an equilateral or isosceles triangle pattern in plan view. This layout is often preferred for more uniform water

distribution compared to square or rectangular arrangements. To ensure proper overlapping and coverage, the following spacing condition must be satisfied:
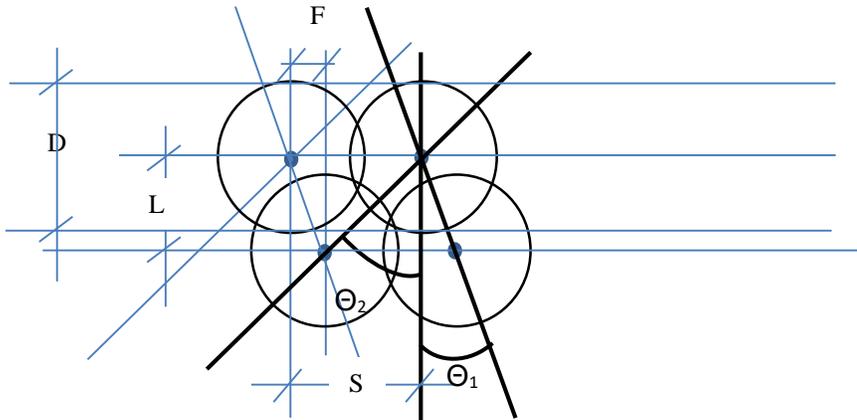

Fig. 5.8 Geometric representation of triangular sprinkler arrangement parameters (L, S, F)

$$\frac{L}{\cos\theta_1.\cos\theta_2} \le D \tag{5.1}$$

Where

       D: the diameter of the sprinkler's wetted area (i.e., the sprinkler's effective throw radius $\times$ 2),

       F: the offset distance,

The angles are given by:

$$cos\theta_1 = \frac{L}{\sqrt{F^2+L^2}} \tag{5.2}$$

$$cos\theta_2 = \frac{L}{\sqrt{(S-F)^2+L^2}} \tag{5.3}$$

By substituting these expressions into the spacing condition and simplifying, the inequality becomes:

$$[(S-F)^2 + L^2][F^2 + L^2]/L^2 \le D^2 \tag{5.4}$$

This inequality ensures that the sprinkler spacing in the triangular pattern will result in adequate overlapping of wetted areas, thereby maintaining uniform irrigation coverage across the field.

Equilateral triangle arrangement, this is a special case of the general triangular arrangement shown in Fig. 5.9, in which point F is exactly halfway between the sprinklers S. Based on this condition, and using equation (5.4) as the foundation, the simplified formula becomes:

$$L + \frac{S^2}{4L} \le D \tag{5.5}$$

For an equilateral triangle, the relationship between L and S is derived from the angle of $30°$ between the vertical and diagonal sides:
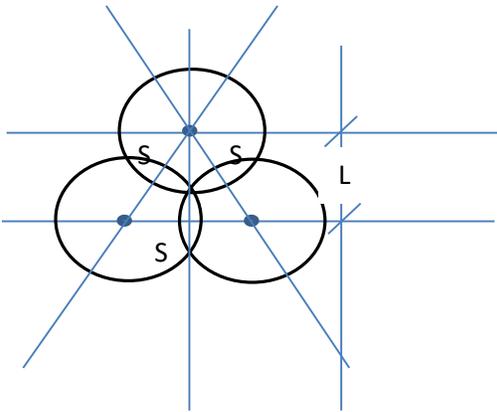
$$\frac{L}{S} = cos\ 30 \qquad (5.6)$$

Fig. 5.9 Geometric layout of equilateral triangle sprinkler arrangement

The rectangular arrangement is commonly employed in regions experiencing high wind conditions. In this layout (Fig. 5.10), the sprinkler laterals (pipes) are aligned perpendicular to the prevailing wind direction to minimize distortion of the spray pattern and ensure more uniform water distribution.

To maintain effective overlap and coverage, the sprinkler spacing must satisfy the following geometric condition:

$$S^2 + L^2 \leq D^2 \qquad (5.7)$$

This equation ensures that the diagonal distance between sprinklers does not exceed the effective throw diameter, thereby preventing dry spots even under windy conditions.
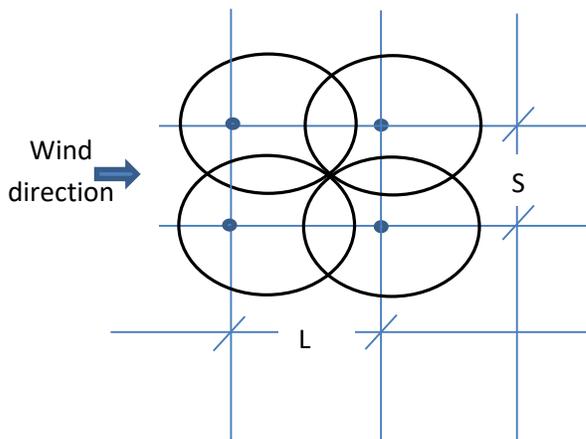

Fig. 5.10 Geometric layout of rectangular sprinkler arrangement under wind conditions

Example 5.1
For an equilateral triangle sprinkler layout, determine the maximum allowable sprinkler spacing that ensures complete field coverage without dry areas, given that the wetted diameter (D) of each sprinkler is 28 meters.

Solution:
The spacing condition for an equilateral triangle arrangement is given by the inequality:
$$L + \frac{S^2}{4L} \leq D$$
Using the geometric relationship for an equilateral triangle layout:
$$L = 0.866 \times S$$
Substitute into the spacing condition:
$$0.866 \times S + \frac{S^2}{4 \times 0.866 \times S} \leq 28$$

$\Rightarrow S \leq 24.25$ m, $L \leq 0.866 \times 24.24 \approx 21.0$ m
To ensure complete coverage and practical implementation, select the nearest smaller standard spacing values:
S= 24 m, L= 21 m.
The values of L and S are typically expressed as multiples of 3 meters for the metric system used in United States, or as multiples of 2 meters for the system commonly used in some European countries. All sprinkler spacing values will be expressed as multiples of 3 meters.

Example 5.2
For a square sprinkler layout, determine the maximum allowable spacing between sprinklers to ensure complete field coverage without dry areas, given that the wetted diameter (D) of each sprinkler is 24 meters.

Solution:
In a square arrangement, the spacing condition is based on the diagonal distance between sprinklers:
$$S^2 + L^2 \leq D^2$$
Since it's a square layout, the spacing between sprinklers in both directions is equal, i.e., S=L. Substituting into the equation:
$$2S^2 \leq 24^2$$
$$S \leq 16.97 \, m, \qquad (15 \, m \times 15 \, m)$$

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5.9 Sprinkler Application Rate

The sprinkler application rate is defined as the depth of water applied to the soil surface by a sprinkler system per unit time. It is usually expressed in millimeters per hour (mm/hr) or centimeters per hour (cm/hr). This rate plays a critical role in irrigation design, as it must be carefully matched to the soil's infiltration capacity to avoid surface runoff, water ponding, and inefficient water use.

The application rate for an individual sprinkler is given by:

$$I = \frac{q}{A} \times 1000 \tag{5.8}$$

Where

I: Application rate (mm/hr)
q: Sprinkler flow rate (m^3 /hr)
A: Area covered by the sprinkler ($S \times L$, m^2)

The discharge of an individual sprinkler in such systems is determined by three main factors: (1) the required depth of water to be applied during each irrigation event, (2) the area assigned to each sprinkler (i.e., its coverage area), and (3) the duration of water application per set.

$$q = \frac{di.S.L}{60T} \tag{5.9}$$

Where

q: Sprinkler flow rate (L /s)
T : Operational time per set for a single lateral (min)
di : Gross depth of irrigation water applied (mm)

The discharge from a sprinkler is primarily determined by the available pressure at the sprinkler head and the effective diameter of the nozzle. For circular nozzles, the discharge can be calculated using the following relationship:

$$q = 0.00111 C_d D_n^2 \sqrt{P} \tag{5.10}$$

Where

q : Discharge per nozzle (L/s)

C_d: Discharge coefficient

D_n : Inside diameter of the nozzle (mm)

P: Pressure at the base of the sprinkler device (kPa).

This equation is a modified version of the orifice flow formula:

$$q = C_d a \sqrt{2gH} \qquad\qquad (5.11)$$

Where

q : Discharge per nozzle (m³/s)

H: Pressure head at the nozzle (m)

g: Acceleration due to gravity (9.81 m/s²)

a: Cross-sectional Area of the orifice or nozzle (m²)

Example 5.3

A sprinkler irrigation system is being designed to apply water uniformly over a field using sprinklers equipped with circular nozzles. The desired discharge from each sprinkler is 0.5 (L/s). The nozzle has an inside diameter of 5.0 (mm), and the discharge coefficient of the nozzle is estimated to be 0.97. Determine the required pressure (P).

Solution:

The pressure can be calculated using the following relationship:

$$q = 0.00111 C_d D_n^2 \sqrt{P} \rightarrow \sqrt{P} = \frac{q}{0.0011 C_d D_n^2}$$

$$P = \left(\frac{0.5}{0.00111 \times\ 0.97\ \times 25} \right)^2 = 345\ kPa$$

5.10 Application Uniformity

Application uniformity refers to how evenly water is distributed over an irrigated area by a sprinkler or surface irrigation system. High uniformity ensures that all parts of the field receive nearly the same depth of water, minimizing over-irrigation in some areas and under-irrigation in others. It is a key factor in irrigation efficiency and crop yield consistency.

5.10.1 Single-Leg Distribution

The uniformity of water application in a sprinkler system is determined by how evenly water is distributed from individual sprinklers. This distribution is typically evaluated using

catch containers placed at various distances around a sprinkler, as illustrated in Fig. 5.11. The sprinkler is operated long enough to collect measurable
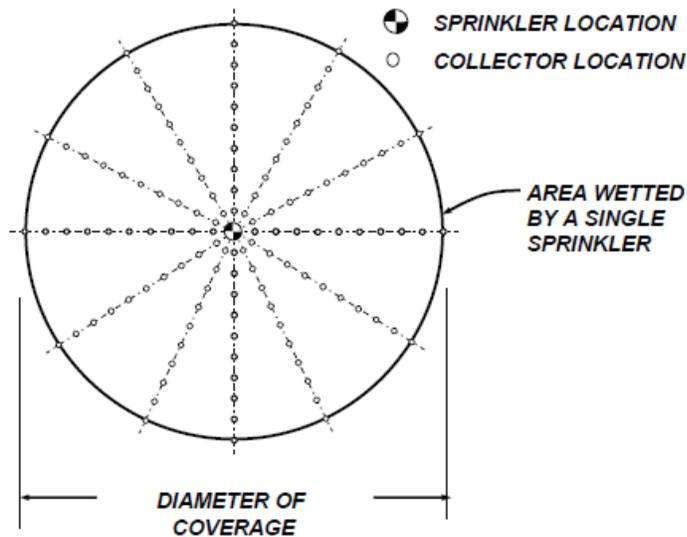

Fig. 5.11 Arrangement to measure the single-leg water distribution of a sprinkler.

The most commonly used equations are for a triangular is given by:

$$d_{(r)} = \frac{3qT}{\pi W_r^3}(W_r - r) \tag{5.12}$$

For an elliptical pattern:

$$d_{(r)} = \frac{3qT}{2\pi W_r^3}\sqrt{(W_r^2 - r^2)} \tag{5.13}$$

Where

$d_{(r)}$: Depth of water applied at a radial distance r from the sprinkler,

W_r: Radius of coverage or wetted radius of the sprinkler,

T: Duration of the sprinkler operation.

5.10.2 Overlapping

In sprinkler irrigation systems with stationary laterals, overlapping refers to the intentional intersection of water distribution patterns from adjacent sprinklers. Because a single sprinkler does not apply water uniformly across its entire wetted radius (most of the water is concentrated near the sprinkler and decreases with distance), proper overlapping is essential to ensure uniform water application across the entire field.

5.10.3 Wind Effect

Wind significantly influences the water distribution pattern of sprinklers. Studies by Christiansen (1942), Vories and von Bernuth (1986), Seginer et al. (1991a), and others have demonstrated that wind distorts the application pattern of a single sprinkler. As illustrated in Fig. 5.12, the entire distribution pattern tends to shift in the direction of the wind. Additionally, the effective diameter of coverage is reduced in the direction perpendicular to the wind. Therefore, a common design recommendation is to orient sprinkler laterals perpendicular to the prevailing wind direction.
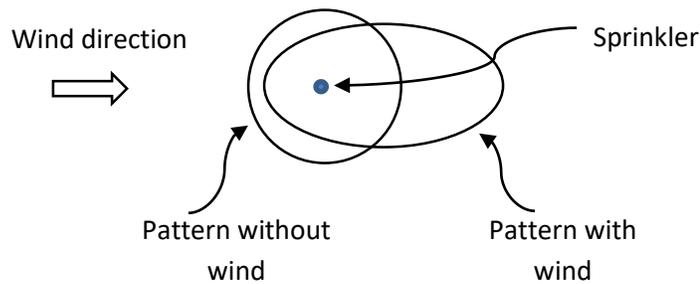

Fig. 5.12 Effect of wind on sprinkler distribution and resulting water application uniformity.

5.11 Sprinkler Irrigation Efficiency

Table 5.1 presents recommended sprinkler irrigation efficiency values for use in system design, based on average wind speed during operation and peak crop water consumption rates (in mm/day).

Table 5.1 Suggested irrigation efficiency values (%) based on wind speed and peak water consumption

Wind Speed (m/s)	Peak Consumption < 5 mm/day	Peak Consumption 5–7.5 mm/day	Peak Consumption > 7.5 mm/day
0–1.8	75%	71%	68%
1.8–4.5	71%	68%	65%
> 4.5	68%	65%	62%

Example 5.4
If the diameter of the wetted circular area for a sprinkler is 29 meters, the sprinkler discharge is 3 cubic meters per hour, and the soil infiltration rate is 10 mm/hour, what are the appropriate sprinkler spacings for this case?

Solution:
To ensure adequate overlap between sprinklers, the spacing must satisfy the geometric constraint for the wetted diameter:

$$S^2 + L^2 \leq D^2$$
$$S^2 + L^2 \leq 841 \tag{1}$$

The application rate Ar of a sprinkler system must not exceed the soil's infiltration rate to prevent runoff. The application rate is given by:

$$I = \frac{q}{A} \times 1000$$

In this case, we substitute the soil infiltration rate in place of the maximum allowable application rate, as it represents the limiting condition for water intake:

$$10 = \frac{3}{S \times L} \times 1000$$
$$S \times L > 300 \ m^2 \tag{2}$$

Assume a square layout: $S = L$
Choose $S = L = 18$ meters to satisfy the infiltration constraint.
Verify overlap condition (Eq. 1):

$$S^2 + L^2 = 18^2 + 18^2 = 324 + 324 = 648 \leq 841$$

5.12 Design Requirements
The design of a sprinkler irrigation system must ensure efficient, uniform, and sustainable water application, considering both agronomic needs and system performance. The key design requirements include:
Periodic-move and fixed sprinkler systems are collectively referred to as *set systems* because the sprinklers remain stationary during operation. These systems are relocated only after the pump (or valve) is shut off and the pipelines have been drained. The design principles for periodic-move and fixed systems are essentially the same as those used for hand-move systems, a specific type of periodic-move system.

Designing a sprinkler irrigation system would be far more straightforward if only a single type of sprinkler were available. However, in practice, a wide variety of sprinklers from

multiple manufacturers are typically available in any given market, making sprinkler selection one of the most challenging aspects of system design.

Under actual field conditions with wind speeds ranging from 0 to 3 mph, these values should be reduced by approximately 10% to estimate the effective wetted diameter, D_{eff}. For wind speeds exceeding 3 mph (5 km/h), the effective diameter should be further reduced accordingly. Therefore,

$$D_{eff} = 0.9D_{manf} \quad if \ u_2 \leq 3mph \tag{5.14}$$

and

$$D_{eff} = [0.9 - 0.025(u_2 - 3)]D_{manf} \quad if \ u_2 > 3mph \tag{5.15}$$

Where

D_{manf} : Diameter reported in manufacturer literature

u_2 : Average expected wind speed during operation at 2 m height above the ground, in mph (3 mph is equivalent to 1.3 m/sec)

A reduction of approximately 2.5% in wetted diameter for each mile per hour of wind speed above 3 mph (1.3 m/s or 5 km/h) provides a reasonable estimate under typical sprinkler operating conditions.

For set sprinkler systems, lateral spacing along the mainline should not exceed 65% of the wetted diameter under calm conditions. This spacing should be reduced to:

- 60% for wind speeds between 1 and 5 mph,
- 50% for winds between 6 and 10 mph, and
- 45% for wind speeds exceeding 10 mph.

For high-pressure and gun-type sprinklers, spacing should also be limited to a maximum of 65% of the wetted diameter in no-wind conditions. However, it should be reduced to:

- 50% for wind speeds of 5 to 10 mph, and
- 30% when wind speeds exceed 10 mph.

Distribution uniformity (DU) is a valuable metric used to quantify the uniformity of water application across a field. It is determined by comparing the average depth of water applied to the quarter of the field receiving the least amount (statistically) with the average application depth across the entire field. This provides a numerical measure of how evenly water is distributed.

$$DU = 100 \left(\frac{Average\ low-quarter\ depth\ of\ water\ recieved}{Average\ depth\ of\ water\ recieved} \right) \qquad (5.16)$$

The average low-quarter depth refers to the mean of the lowest 25% of measured water application values within a defined area, with each measurement representing an equal surface area.

Another commonly used parameter to assess sprinkler irrigation uniformity is the Uniformity Coefficient, introduced by Christiansen:

$$CU = 100 \left(1.0 - \frac{\sum x}{n.m} \right) \qquad (5.17)$$

Where

x: Absolute deviation of the individual observations from the mean, in or mm
m : Mean depth of observations, in or mm
n: Number of observations

When pressure or flow control is not implemented, pressure variations across the system can significantly reduce the overall uniformity compared to that observed in the test area. An estimate of the system's Distribution Uniformity (DU) and Christiansen Uniformity Coefficient (CU) can be calculated using the system's maximum, minimum, and average operating pressures as follows:

$$System\ DU = DU \left(1 - \frac{P_x - P_n}{5P_a} \right) \qquad (5.18)$$

$$System\ CU = CU \left(1 - \frac{P_x - P_n}{8P_a} \right) \qquad (5.19)$$

Where

P_x: Maximum sprinkler pressure in the system, kPa
P_n: Minimum sprinkler pressure in the system, kPa
P_a: Average sprinkler pressure in the system, kPa

Example 5.5
A sprinkler irrigation system is tested in a field using 16 catch cans placed evenly within a rectangular area between four adjacent sprinklers. Each can represent an equal surface area. Determine the effect of system pressure on uniformity. The measured depths of water collected (in mm) from a single irrigation event are as follows:

Measured depths (mm):
14, 13, 15, 12, 18, 16, 14, 17, 13, 15, 19, 18, 12, 14, 16, 15

Solution:

1. Sort values in ascending order:

12, 12, 13, 13, 14, 14, 14, 15, 15, 15, 16, 16, 17, 18, 18, 19

2. Select the lowest 25% of values (lowest quarter):
   There are 16 values total → 25% of 16 = 4 values
   Lowest 4 values: 12, 12, 13, 13
3. Calculate average of lowest quarter:

$$Avg\ low - quarter = \frac{12 + 12 + 13 + 13}{4} = 12.5\ mm$$

4. Calculate average depth for all measurements:

$$Average\ depth = \frac{241}{16} = 15.06\ mm$$

5. Distribution Uniformity (DU)

$$DU = 100 \times \frac{12.5}{15.06} = 83\%$$

6. Christiansen's Uniformity Coefficient (CU)

Compute absolute deviations from the mean (27.25)

$$CU = 100\left(1.0 - \frac{\Sigma x}{n.m}\right) = 100\left(1.0 - \frac{27.25}{16 \times 15.06}\right) = 88.69\%$$

7. Adjust DU and CU for Pressure Variation

Assume:

- P_x = 300kPa
- P_n = 240kPa
- P_a = 270kPa

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$$\text{System } DU = DU\left(1 - \frac{P_x - P_n}{5P_a}\right) = 83\left(1 - \frac{300 - 240}{5 \times 270}\right) = 79.3\%$$

$$\text{System } CU = CU\left(1 - \frac{P_x - P_n}{8P_a}\right) = 88.69\left(1 - \frac{300 - 240}{8 \times 270}\right) = 86.23\%$$

The maximum set time is initially determined as follows:

$$T_s' = \frac{R_e di}{I_s} + T_m \tag{5.20}$$

Where

T_s': Decimal set time (hours)

R_e: Effective portion of applied water to ground (decimal)

I_s: Maximum soil infiltration rate expected at the end of the set, typically the long-term rate

T_m: Time required to move the lateral (while off), hours

To round T_s' down:

The initial value for maximum set time typically ranges from 0.5 hours for side-roll (wheel line) systems to 1.0 hour for quarter-mile (400 m) hand lines. The prime symbol (') on T_s' denotes that it is a calculated decimal value, which will later be adjusted usually rounded to a time that divides evenly into 24 hours.

$$T_s = \max(2,3,4,6,8,12,24) \leq T_s' \tag{5.21}$$

The gross application depth to adhere to the I_s limitation is:

$$di = \frac{(T_s - T_m)I_s}{R_e} \tag{5.22}$$

The revised value for di will be less than or equal to the initially calculated di . When T_s' is rounded up, it is calculated as:

$$T_s = \min(2,3,4,6,8,12,24) \geq T_s' \tag{5.23}$$

The average gross application rate decreases as the set time increases. The mean design application rate is calculated using the following expression:

$$I = \frac{di}{(T_s - T_m)} \leq \frac{I_s}{R_e} \tag{5.24}$$

The number of sets per day is given by:

$$n_s = \frac{24}{T_s} \qquad (5.25)$$

If T_s is calculated using Eq. 5.21, the resulting value of n_s will be an integer. However, if T_s is not restricted to a duration that divides evenly into 24 hours, n_s may not be a whole number. In such cases, the designer may choose to round n_s to the nearest integer to simplify and standardize system operation. To accommodate this adjustment, T_s can be recalculated (increased) by rearranging Eq. 5.24 as follows:

$$T_s = \frac{24}{n_s} \qquad (5.26)$$

Example 5.6
Determine an appropriate set time, adjusted gross depth, and system parameters such that the application rate remains within the soil's long-term infiltration limit.

Parameter	Value	Unit
R_e	0.85	--
I_s	1.2	cm/hr
T_m	0.5	hr
di	10.0	cm

Solution:
Using Equation (5.20):

$$T_s' = \frac{R_e di}{I_s} + T_m = \frac{0.85 \times 10}{1.2} + 0.5 = 7.58 \; hr$$

From standard design values (Eq. 5.21):

$$T_s = \max(2,3,4,6,8,12,24) \leq 7.58 \Rightarrow T_s = 6hr$$

Using Equation (5.22):

$$di = \frac{(T_s - T_m)I_s}{R_e} = \frac{(6-0.5). \; 1.2}{0.85} \Rightarrow di = 7.76 \; cm$$

Calculate mean application rate I

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$$I = \frac{di}{(T_s - T_m)} = \frac{7.76}{5.5} = 1.41 \; cm/hr$$

$$\frac{I_s}{R_e} = \frac{1.2}{0.85} = 1.41 cm/hr \quad \text{acceptable.}$$

Using Equation (5.25):

$$n_s = \frac{24}{T_s} = \frac{24}{6} = 4 \; sets/day$$

The required inflow rate for the system is the amount of water that must be delivered to meet crop water needs and prevent water stress. It is calculated using the following system flow rate formula:

$$Q_s = 0.116 \left(\frac{di \cdot A_i}{T_a} \right) \tag{5.27}$$

Where

Q_s: Inflow to the sprinkler irrigation system (i.e. gross system capacity), L s^{-1}

A_i: Area irrigated, ha,

T_a: Time of operation per irrigation, days

di: Gross depth of irrigation water applied, mm.

Table (5.2) provides guidance for selecting the appropriate operating pressure head (H) in meters for single-nozzle sprinkler heads based on the internal diameter of the nozzle opening (d) in millimeters. Selecting the correct pressure is essential to ensure that the water droplets produced are of acceptable size minimizing evaporation and wind drift while achieving uniform distribution.

Table 5.2 Recommended operating pressure ranges for single-nozzle sprinklers based on nozzle diameter

Nozzle Diameter (mm)	Recommended Pressure Head (m)
$3 < d \leq 4.8$	25 – 35
$4.8 < d \leq 6.4$	30 – 40
$6.4 < d \leq 9.6$	35 – 50

The Darcy-Weisbach and Hazen-Williams equations are the most widely used formulas for estimating pipe friction losses in sprinkler irrigation design. These equations, along

with definitions for key associated terms, are fundamental tools in hydraulic analysis. Accurately estimating friction loss is essential, as it represents one of the two primary factors influencing pressure variation within pipelines, the other being elevation change. Pressure variations directly impact the uniformity of water application across a sprinkler system, which in turn affects overall irrigation efficiency.

1. Darcy-Weisbach Equation

This equation is formulated for circular pipe cross-sections, which encompass nearly all pipes used in sprinkler irrigation systems.

$$H_f = f\frac{L}{D}\cdot\frac{V^2}{2g} \qquad (5.28)$$

Where

H_f: Friction loss (head of water), ft or m
f: Friction factor
L: Pipe length, ft or m
D: Pipe inside diameter, ft or m
V: Average velocity at a cross-section

The Darcy-Weisbach equation is generally considered to provide more accurate estimates of pipe friction loss compared to alternative methods.

The Moody diagram, commonly found in hydraulics textbooks, can be used with the Darcy-Weisbach equation to determine the friction factor f. However, with the availability of calculators and computer tools, it is often more convenient to calculate f using the Swamee-Jain equation.

$$f = \frac{0.25}{\left[\log_{10}\left(\frac{\varepsilon}{3.75D}+\frac{5.74}{R_e^{0.9}}\right)\right]^2} \qquad (5.29)$$

Where

ε: Roughness height of the pipe material, ft or m
D: Inside diameter of the pipe, ft or m
R_e: Reynolds number

The Swamee-Jain equation is applicable for turbulent flow within the Reynolds number range of 4,000 to 1.0×10^8. In sprinkler irrigation systems, the flow within pipes is almost always turbulent, making this equation well-suited for such applications.

The ratio ε/D is known as the relative roughness. The roughness height ε can vary significantly depending on the pipe's material (Table 5.3), condition, and age.

The Blasius equation can be used in certain cases to estimate the friction factor f for smooth pipes, such as those made of plastic.

$$f = 0.32 R_e^{-0.25} \tag{5.30}$$

Table 5.3 Typical values of roughness (ε) for various pipe materials commonly used in irrigation and water conveyance systems

Pipe Material	Roughness Height (ε, mm)	Pipe Material	Roughness Height (ε, mm)
Concrete	0.3-3.0	Asphalted Cast iron	0.12
Cast iron	0.26	Commercial or welded steel	0.045
Galvanized iron	0.15	PVC	0.0015

2. Hazen-Williams equation

One of the most commonly used flow equations for pipes, especially in irrigation engineering, is the Hazen-Williams equation, given by:

$$H_f = 1.14 \times 10^9 \left(\frac{Q}{C}\right)^{1.852} L/D^{4.87} \tag{5.31}$$

Where

Q: Pipe discharge (m³/hour)
L: Length of pipe (m)
D: Internal diameter of the pipe (mm)
C: Roughness coefficient (dimensionless), with typical values for selected pipe types listed in Table 5.4.

Table 5.4 Roughness coefficient C in the Hazen-Williams equation

Pipe Type	Roughness Coefficient C
Concrete	100
Aluminum	120
Steel	130
Asbestos Cement	140
Plastic (e.g., PVC)	150

Valves, elbows, couplings, screens, and other components commonly found in sprinkler irrigation systems cause localized or "minor" hydraulic losses. Depending on the system configuration, these losses can represent a significant portion of the total head loss particularly in systems with short pipe lengths or may be relatively negligible. Local losses are typically estimated using a dimensionless coefficient, K_r, which is multiplied by the velocity head, $\frac{V^2}{2g}$, to calculate the head loss due to each fitting. This approach is consistent with methods used in both the Darcy-Weisbach and Hazen-Williams equations.

$$\left(H_f\right)_{local} = K_r \frac{V^2}{2g} \tag{5.32}$$

Friction loss in a pipeline with multiple outlets is lower than in a pipeline carrying a constant discharge along its entire length, as the flow rate decreases progressively after each outlet. To account for this, Christiansen developed a widely accepted method for estimating pressure losses in pipelines with multiple outlets. The approach involves first calculating the friction loss as if the pipeline had a constant flow throughout, and then adjusting the result by multiplying it with a factor F, which depends on the number of outlets (such as sprinklers), denoted by N, along the lateral.

The value of the factor F approaches approximately 0.36 when the number of outlets N exceeds 35, which is commonly the case in sprinkler laterals (see Table 5.5). It is important to note that Equations 5.33 and 5.34, used to calculate F, apply specifically to pipelines with no flow beyond the last outlet (i.e., the last sprinkler). Therefore, they are not suitable for estimating friction losses at intermediate points along the lateral. Additionally, the calculation of F assumes uniform discharge from each outlet, a condition that is only approximate due to inevitable pressure variations along the lateral. These equations are intended for use in systems where outlet discharge is nearly constant, such as hand-move, wheel-line, solid-set (fixed), and linear-move sprinkler systems.

Table 5.5 Friction loss reduction coefficients (F) for pipes with multiple outlets of nearly uniform discharge

Number of Outlets	$F^{(1)}$ (end)	$F^{(2)}$ (mid)
1	1.00	1.00
2	0.64	0.52
3	0.53	0.44
4	0.49	0.41
5	0.46	0.40
6	0.44	0.39
7	0.43	0.38

8	0.42	0.38
9	0.41	0.37
10–11	0.40	0.37
12–14	0.39	0.37
15–20	0.38	0.36
21–35	0.37	0.36
>35	0.36	0.36

$F^{(1)}$ (end): For laterals with the first sprinkler located at the **end** of the pipe (end-riser design).
$F^{(2)}$ (mid): For laterals with the first sprinkler located at the **middle** of the pipe (mid-riser design).

Christiansen's equation for calculating the reduction coefficient F in pipes with multiple, equally spaced outlets, where the first outlet is located at position S from the mainline is given by:

$$F = \frac{1}{m+1} + \frac{1}{2N} + \frac{\sqrt{m-1}}{6N^2} \tag{5.33}$$

and where the first outlet is located at S/2 from the mainline, for example, when laterals extend in both directions from the mainline:

$$F = \left(\frac{2N}{2N-1}\right)\left(\frac{1}{m+1} + \frac{\sqrt{m-1}}{6N^2}\right) \tag{5.34}$$

Where

m: 1.852, 2.0 for the Hazen-Williams equation and Darcy-Weisbach equation
, receptively

Example 5.7
A PVC pipe is used in a sprinkler irrigation lateral with a total length of L= 90 m and an internal diameter of D = 50 mm. The total discharge entering the pipe is Q = 6 L/s, and the system includes N = 9 evenly spaced sprinklers, each assumed to deliver a uniform discharge. The flow is turbulent, which is typical for pressurized sprinkler systems. The kinematic viscosity of water is $\nu = 1.01 \times 10^{-6}$ m^2/s, and the roughness height of the PVC pipe is $\varepsilon = 0.0015$ mm. Find the friction head loss.

Solution
Calculate cross-sectional area A
$$A = \frac{\pi D^2}{4} = \frac{\pi}{4} 0.05^2 = 0.00196 m^2$$
Velocity V
$$V = \frac{Q}{A} = \frac{0.006}{0.00196} = 3.06 m/s$$

Reynolds Number Re

$$R_e = \frac{VD}{\nu} = \frac{3.06 \times 0.05}{1.01 \times 10^{-6}} = 151485$$ Turbulent flow confirmed.

Swamee-Jain friction factor f

$$f = \frac{0.25}{\left[\log_{10}\left(\frac{\varepsilon}{3.75D} + \frac{5.74}{R_e^{0.9}}\right)\right]^2} = 0.0166$$

Darcy-Weisbach head loss

$$H_f = f\frac{L}{D}.\frac{V^2}{2g} = 14.26 \ m$$

Apply Christiansen's F, From Table 5.5 for N = 9, end-riser:

F = 0.41⇒H_f,actual = 0.41×14.26 = 5.85m.

The allowable pressure variation inside the lateral is limited to 20% of the average operating pressure of the sprinkler lateral pipe. In most cases, the difference in pressure head inside the pipe is not more than 20%, especially if the pipe is laid level or nearly level. Referring to Fig. 5.13, the following equation can be used to express the allowable pressure variation along the lateral:
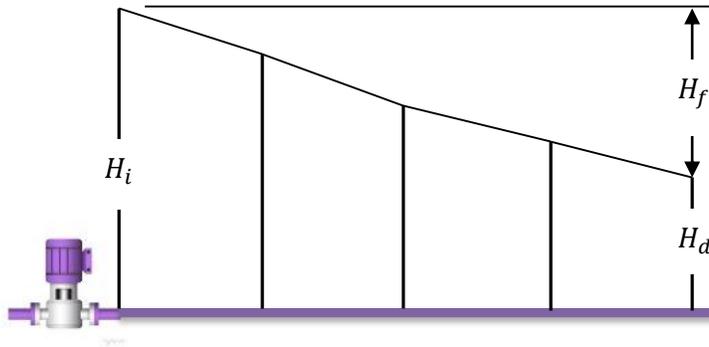

Fig. 5.13 Pressure head distribution along a sprinkler lateral pipe

$$H_i - H_d \leq 20\%H_a \tag{5.35}$$

Where

H_i: Pressure head at the inlet of the lateral (m)

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H_d: Pressure head at the end of the lateral (m)

H_a: Average operating pressure head in the lateral (m), calculated using Equation (8.11)

When the pipe is installed on a slope, the pressure loss due to friction H_f must be considered. If the slope is significant, the allowable friction loss is adjusted based on the elevation difference, using the following relationships:

$$H_f \leq 20\%H_a + \Delta z \tag{5.36}$$
$$H_f \leq 20\%H_a - \Delta z \tag{5.37}$$

Where

Δz: Elevation difference between the beginning and end of the lateral pipe, calculated from the pipe's length and slope

- For a pipe sloping downward, use Equation (5.36)
- For a pipe sloping upward, use Equation (5.37)

5.13 Hydraulic Design of Pipes

If the allowable friction loss is known, the diameter of the sprinkler pipe can be calculated using equations (5.28) or (5.31) along with the basic forms of equations (5.33) or (5.34).

If the value of D is unknown, it must be determined through a process of trial and error or by using standard pipe diameter charts. If the calculated value of D does not match available standard pipe sizes, a slightly larger pipe diameter should be chosen to avoid exceeding the allowable friction loss. However, in many cases, it is necessary to choose pipe diameters based on what is available, and we may not want to exceed the specified allowable friction loss. To assist in selecting among available pipe diameters, we can use the relative sizing method by comparing the allowable friction losses for different pipe sizes using the following proportional formula:

$$H_{f2} = H_{f1} \left(\frac{D_1}{D_2} \right)^n \tag{5.38}$$

Where

H_{f1}: Friction loss in a pipe of diameter D_1

H_{f2} : Friction loss in a pipe of diameter D_2

n : Exponent depending on the equation used to calculate friction loss (e.g., 4.87 in Hazen-Williams)

5.14 Pressure Head Calculation

The design of sprinkler pipes requires determining the necessary pressure head at the pipe inlet and at key locations along its length, especially near the end plug positioned just beyond the last sprinkler outlet. Studies and hydraulic analyses of pipelines with multiple outlets under varying conditions have shown that the average pressure head typically occurs about 40% of the distance from the inlet (Fig 5.14). In other words, approximately 75% of the total friction loss takes place between the pipe inlet and this point, where the pressure head equals the system's average operating pressure. Based on this principle, the required inlet pressure can be estimated using the following equation:

$$H_i = H_a + 0.75H_f \pm 0.5\Delta z \tag{5.39}$$

The sign of the last term in Equation (5.39) is:
- Negative (–) if the pipe slopes downward
- Positive (+) if the pipe slopes upward
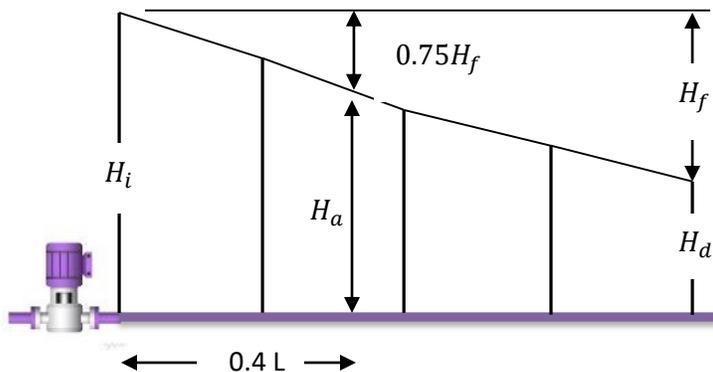
If the lateral pipe is laid level, then $\Delta z = 0$

Fig. 5.14 Pressure head distribution along a sprinkler lateral with multiple outlets

Example 5.8
A sprinkler lateral pipe is 120 meters long and laid level across a field. The system is designed to operate at an average pressure head of $H_a = 25$ m, and the average discharge along the lateral is $Q = 3.06 \text{m}^3/\text{hr}$, length of pipe is 120m, C=150 (PVC pipe). The designer wants to ensure that the pressure variation along the lateral does not exceed the allowable limit of 20%.

Determine:

1. The maximum allowable pressure loss (friction loss H_f) along the lateral.
2. The required pressure head at the inlet H_i
3. Check if a selected pipe of diameter D = 40 mm satisfies the allowable criteria.

Solution

Calculate the maximum allowable pressure variation:

$H_i - H_d \leq 20\% H_a \Rightarrow H_f \leq 5.0\ m$

Estimate inlet pressure head, since the pipe is laid level, we apply Equation (5.39) with $\Delta z = 0$:

$H_i = H_a + 0.75 H_f \pm 0.5 \Delta z \Rightarrow H_i = 28.75\ m$

So, the required inlet pressure head is 28.75 m.

Compute friction loss for 40 mm pipe:

$$H_f = 1.14 \times \frac{10^9 \left(\frac{Q}{C}\right)^{1.852} L}{D^{4.87}} = 1.14 \times 10^9 \left(\frac{3.06}{150}\right)^{1.852} \times \frac{120}{(40)^{4.87}} = 1.6m$$

Apply Christiansen's F, From Table 5.5 for N = 10, end-riser:

F = 0.41 $\Rightarrow H_f$,actual = 0.40×1.6 = 0.64m, Acceptable, below 5 m

5.15   Moving and Operation Sprinkler Pipes

In portable sprinkler irrigation systems, **the relocation and operation of lateral pipes** is a key factor influencing labor requirements, system efficiency, and field coverage. These systems are typically designed for flexibility and ease of handling to accommodate various crop layouts and irrigation schedules.

Sprinkler laterals in portable systems are usually lightweight aluminum or PVC pipes equipped with quick-coupling joints. The standard operation involves the following steps:

1. Shut off the main valve to stop water flow and relieve pressure within the system.
2. Disconnect the lateral pipe sections from the risers or mainline, typically using lever-lock or cam-lock couplings.
3. Move the laterals laterally or longitudinally across the field to the next irrigation strip or zone. This is often done manually by workers or using pipe carts for longer lines.
4. Re-connect and align the lateral ensuring proper spacing and riser alignment.
5. Open the valve and re-pressurize the system for the next irrigation cycle.

Depending on the layout and spacing of the sprinklers, the moving interval may range from 6 to 24 hours.

Several factors influence the ease and time required to move sprinkler laterals:

1. Pipe material and diameter: Lightweight pipes (e.g., 40–75 mm PVC or aluminum) are easier to handle manually.
2. Joint type: Quick-coupling systems significantly reduce labor and time.
3. Field topography: Steep or uneven terrain may complicate movement and alignment.

4. Crop height and stage: Tall or dense crops (e.g., maize) may require more careful handling to avoid damage.
5. Labor availability: Systems with longer or heavier pipes require more labor or mechanical assistance.

Efficient operation of sprinkler pipes involves maintaining proper:

1. Pressure: Each sprinkler must operate within its designed pressure range to ensure uniform distribution.
2. Nozzle condition: Clogged or worn nozzles reduce performance and distribution uniformity.
3. Sprinkler spacing: Must be consistent across moves to avoid over- or under-irrigated zones.
4. Overlap: Adequate overlap is required between sprinkler patterns to ensure even water application.

The total number of sprinkler pipe moves required depends on the layout plan of the irrigation network, which includes the total length of the main distribution line and the length of the lateral pipes, separated by the distance (L) between each sprinkler pipe. If the length of the sprinkler pipe is fixed and standard (which is usually the case), then the total number of sprinkler pipe moves required in the field can be calculated using the following formula:

$$NV = \frac{AF}{P.L} \tag{5.40}$$

Where

NV: Total number of sprinkler pipe moves per irrigation cycle
AF: Area of the field (m²)
P: Length of the sprinkler pipe (m)
L: Move distance or spacing between sprinkler pipe lines (m)

The required number of sprinkler pipes (n) for the entire field can be calculated using the following formula:

$$\frac{NV}{M.n} \leq II \tag{5.41}$$

By combining equations (5.40) and (5.41) and simplifying, we obtain:

$$n \geq \frac{AF}{M.P.L.II} \tag{5.42}$$

Where

M: Number of sprinkler pipe moves per day

II: Irrigation interval (days)

In Equation (5.41), the value of M is equal to 1 when the irrigation system operates for 23 hours per day. If the system operates for only 11 hours per day, then M is equal to 2.

If the shape of the field is rectangular, then the field area is calculated as:

$$AF = P.T \tag{5.43}$$

Where

T: Total length of the field in the direction of sprinkler pipe movement

Substituting Equation (5.43) into Equation (5.42), we get:

$$n \geq \frac{T}{M.L.II} \tag{5.44}$$

If the main pipe is located in the center of the field, Equation (5.44) becomes:

$$n \geq \frac{2T}{M.L.II} \tag{5.45}$$

The value of n should be rounded up to the nearest whole number when calculated from Equations (5.42).

Example 5.9
Determining a consistent sprinkler set time and number of moves per day.
Given Data:

Parameter	Value
Required gross depth di	38 mm = 0.038 m
Soil long-term infiltration rate Is	10 mm/hr
Application efficiency Re	75% = 0.75
Lateral move time Tm	0.25 h
Sprinkler pipe length P	60 m
Lateral move spacing L	18 m
Field area AF	18,000 m²
Irrigation interval II	6 days

Solution:
Using Equation (5.20):

$$T'_s = \frac{R_e di}{I_s} + T_m = \frac{0.75 . 0.038}{0.010} + 0.25 = 3.10 \; hr$$

Using Equation (5.23) (round up to nearest standard set time):
$$T_s = \min(2,3,4,6,8,12,24) \geq T_s' \Rightarrow T_s = 4\ hr$$
Using Equation (8.25):
$$I = \frac{di}{(T_s - T_m)} = \frac{0.038}{4 - 0.25} = 0.01013\ \frac{m}{hr} = 10.13\ mm/hr$$
Compare to allowable:
$$\frac{I_s}{R_e} = \frac{10}{0.75} = 13.33\ mm/hr \qquad \text{Acceptable}$$
Using Equation (8.26):
$$n_s = \frac{24}{T_s} = \frac{24}{6} = 6 \Rightarrow M = 6\ \text{moves/day}$$
Using Equation (5.42):
$$n \geq \frac{AF}{M.P.L.II} = \frac{18000}{6.\,60.\,18.\,6} = 0.463 \Rightarrow n = 1(\text{rounded up})$$
With 1 lateral pipe and 6 moves per day, the full field can be irrigated in 2.8 days.

6. Drip Irrigation Design

6.1 Advantages of Drip Irrigation

Drip irrigation, also known as trickle irrigation, offers several agronomic, economic, and environmental benefits. These advantages make it particularly suitable for arid and semi-arid regions, as well as for high-value crops. The main advantages of drip irrigation include:

1. Water Use Efficiency
- Water is applied directly to the root zone, minimizing losses due to evaporation, runoff, and deep percolation.
- Efficiency can reach up to 90–95%, compared to 50–70% in surface irrigation systems.

2. Improved Crop Yield and Quality
- Consistent soil moisture levels reduce plant stress and promote uniform growth.
- Enables higher yields and better crop quality, especially in fruits and vegetables.

3. Reduced Weed Growth
- Water is delivered only to the root zone, leaving the inter-row areas dry, thereby suppressing weed development.

4. Fertilizer Efficiency (Fertigation)
- Fertilizers can be injected directly into the irrigation system, delivering nutrients precisely where needed.
- Reduces fertilizer waste and leaching.

5. Adaptability to Various Terrains
- Suitable for undulating lands, slopes, and irregular field shapes without requiring land leveling.
- Minimizes erosion on sloped fields.

6. Labor and Energy Savings
- Automation is easier compared to other irrigation systems.
- Lower operating pressure reduces energy consumption compared to sprinkler systems.

7. Minimized Disease Pressure
- Foliage remains dry, reducing the incidence of fungal and bacterial diseases common in wet canopy conditions.

8. Flexibility in Operation
- Can be operated at any time of day without interfering with farm labor or field activities.
- Allows irrigation during windy conditions or in greenhouses.

9. Suitability for Marginal Water
- Tolerant of moderately saline water when proper management is applied, as water is applied frequently in small amounts.

10. Scalability and Control
- Ideal for small-scale gardens to large commercial farms.
- Allows for precise control over water application rates and scheduling.

6.2 Limitations of Drip Irrigation

Despite its many advantages, drip irrigation is not universally applicable in all situations. Several technical, economic, and operational limitations may affect its suitability, especially in resource-limited or large-scale field crop systems. Understanding these limitations is crucial for making informed irrigation design and management decisions.

1. High Initial Investment Cost
- Drip systems require a relatively high initial capital investment in pipes, emitters (Fig. 6.1), filters, pressure regulators, and automation components.
- Cost per hectare is typically higher than for surface or sprinkler systems, especially for smallholder farmers.

Fig. 6.1 Adjustable drip emitter

2. System Clogging
- Emitters are prone to clogging due to:
  - Sediment or organic matter in the water.
  - Chemical precipitation (e.g., calcium or iron).
  - Algae or bacterial slime growth in the pipelines.
- Requires efficient filtration, periodic flushing, and chemical treatment (e.g., acid or chlorine injection).

3. Maintenance Requirements
- The system requires regular inspection, cleaning, and maintenance to prevent malfunction.
- Skilled labor is needed to manage pressure control, leak detection, and clogged emitter repair.

4. Limited Suitability for Certain Crops or Field Conditions
- Not well-suited for closely spaced cereal crops or large-scale field crops (e.g., wheat, maize) due to high material and labor costs.
- Challenging to install in stony soils or where rodents and insects can damage pipes.

5. Uneven Distribution Due to Pressure Loss
- Variations in pressure along the lateral lines can lead to non-uniform water application, especially in long or sloped fields.
- Requires careful hydraulic design to maintain acceptable emission uniformity (EU).

6. Vulnerability to Physical Damage
- Pipes and emitters laid on the soil surface are exposed to:
  - Animal damage (e.g., rodents, livestock).
  - UV degradation over time.
  - Mechanical damage from farm operations or foot traffic.

7. Limited Leaching of Salts
- Unlike surface irrigation, drip irrigation provides no excess water for leaching accumulated salts from the root zone.
- In saline soils, periodic supplemental irrigation may be required to flush salts downward.

8. Dependency on Electricity or Fuel
- Pumps and fertigation units often require a reliable power supply.
- In remote or undeveloped areas, lack of access to electricity may limit system operation.

9. Shorter Lifespan of Components
- Emitters and thin-walled drip tapes have a limited service life and may require frequent replacement.
- Plastic materials degrade over time due to sunlight, heat, and pressure fluctuations.

10. Complexity of Design and Operation
- Proper system design requires technical expertise in hydraulic calculations, emitter selection, and crop water requirements.
- Poorly designed systems can result in non-uniform application, crop stress, and economic losses**.**

6.3 Basic Parts of Drip Irrigation

A drip irrigation system consists of several key components that work together to deliver water efficiently and precisely to the plant root zone. The system typically begins with a pump, which draws water from the source and provides the necessary pressure to move it through the network. An optional pre-filter may be installed before the pump to remove large debris and protect the system components. Immediately after the pump, a pressure relief valve ensures safety by releasing water when pressure exceeds safe limits. Water then passes through filters, which are essential for removing fine particles and preventing emitter clogging. These filters are often equipped with a backwash system that allows for easy cleaning by reversing the flow of water.

Next, the water flows through a pressure control device that maintains the optimal operating pressure for the system. A water meter is installed to measure the volume of water applied, which is useful for monitoring usage and managing fertigation. An air vent is included to release trapped air from the pipelines, ensuring uninterrupted flow. The system also incorporates a chemical injection unit, which allows for the precise delivery of fertilizers and other chemicals directly into the irrigation stream a process known as fertigation. To protect the water source, a backflow prevention device is placed after the injection point to stop any reverse flow of potentially contaminated water.

A pressure reducing solenoid valve, which can be automated, controls the flow of water into different irrigation zones while also reducing pressure to safe levels. The water then travels through the main line, a large-diameter pipe that distributes water from the filtration unit to various parts of the field. From the main line, water enters sub-main lines, which further divide the flow to specific irrigation zones, such as Zone 1 shown in the Fig. 6.2. Connected to the sub-mains are the drip lines, which are flexible tubes laid along crop rows and equipped with emitters that release water directly to the plant roots. At the ends of these lines, flush valves are installed to periodically clean the system by flushing out any accumulated sediments. Together, these components ensure that water and nutrients are applied uniformly, efficiently, and with minimal waste, making drip irrigation one of the most precise and sustainable irrigation methods available.

6.4 Secondary Parts of Drip Irrigation

In addition to the primary components, a well-designed drip irrigation system incorporates a range of secondary parts that enhance its functionality, reliability, and ease of maintenance. Among these are **air release valves**, which automatically expel trapped air from the pipelines during system start-up and operation, thereby preventing air blockages that can disrupt uniform water distribution. **Flush valves**, typically located at the ends of sub-mains and lateral lines, are used to discharge accumulated sediments and debris, helping to maintain emitter performance and prevent clogging.
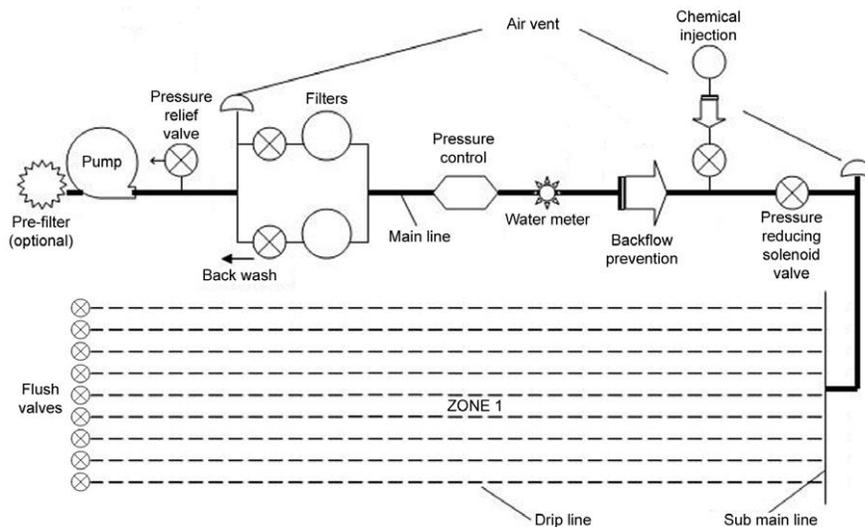
Fig. 6.2 Typical layout and components of a drip irrigation system

End caps or **end plugs** are essential for sealing the terminal points of drip lines and sub-mains. These closures can be easily removed for periodic flushing and system inspection. **Pressure gauges** are installed at strategic locations to monitor pressure levels throughout the system, ensuring optimal operating conditions and enabling early detection of issues such as blockages or leaks.

To support filter maintenance, **filter flush valves** may be installed, allowing for quick manual flushing of filters, especially in systems not equipped with automatic backwashing. **Drain valves** are often included to facilitate complete drainage of the system during shutdowns or maintenance, thereby preventing water stagnation and potential microbial growth.

Additionally, the system includes various **connectors**, **grommets**, **joiners**, and **fittings**, which are used to link pipes and components securely while maintaining leak-proof connections. Though considered secondary, these components play a critical role in maintaining the efficiency, durability, and operational flexibility of the drip irrigation system, particularly in complex or large-scale agricultural applications.

6.5   Filtration

Drip emitters contain small orifices that are highly susceptible to clogging; therefore, effective filtration is essential to remove suspended sediments and biological contaminants from the irrigation water. The three main types of filters commonly used in drip irrigation systems are sand filters, mesh (screen) filters, and disc filters.

6.5.1 Perfiltration

When irrigation water contains high levels of algae or sediment, sand filters alone may become overwhelmed and prone to clogging. To address this, a prefiltration step is necessary and should be installed upstream of the main filtration unit. For sediment-laden water, a settling pond is effective, while stainless steel screen filters are recommended for algae removal. Algae can be filtered using cylindrical stainless steel screen filters (as shown in Fig. 6.3) or flat round screen filters. Flat screen filters offer the advantage of self-cleaning, as the flow of water naturally displaces debris off the edge of the screen. These filters can also be designed with a linear flow path, where water enters from one side and exits from the other, pushing debris away from the filtration surface and reducing the need for frequent maintenance.

For laminar flow, the settling velocity is determining by Stokes law and can be calculated with the following equation.

$$V_s = 3.43 \times 10^{-5} d^2 SG \tag{6.1}$$

Where

V_s: Settling velocity of particle, m/min,
d: Particle diameter, microns,
SG: Specific gravity of particle, 2.65 for mineral (soil) particles.

Fig. 6.3 Cylindrical stainless steel ring filters.

If the settling basin has a depth of 0.6 meters and side slopes of 2:1 (horizontal to vertical), the water surface area can be calculated using the following equation:

$$A = 0.001F \left(\frac{Q}{V_s} \right) \qquad\qquad (6.2)$$

Where

  A: Water surface area, m^2,

  F: Safety factor,

  Q: Irrigation system flow rate, LPM

The length of the settling basin is determined as a function of its width, typically expressed as: ($L = 5W$), where the length is five times the width to ensure effective sedimentation and prevent short-circuiting.

6.5.2 Filter Mish Size

Sand, screen, and disc filters used in drip irrigation systems are typically classified by their mesh size, which indicates the fineness of the filtration. The pore size of the filter should be approximately one-quarter the size of the emitter openings to effectively prevent clogging. Drip irrigation manufacturers usually specify the required mesh size for each type of emitter. In screen filters, mesh size refers to the number of openings per linear inch (25.4 mm); for example, a 200-mesh filter contains 200 openings per inch in both directions. Commonly recommended mesh sizes for drip emitters range between 100 and 200, corresponding to 150 and 75 microns, respectively. Although sand filters do not have a physical mesh like screen filters, they are still classified by an equivalent mesh size based on their filtration capability.

6.5.3 Sand Filter

Pressurized tank sand filters operate by utilizing a pressure differential to force water through a sand media bed. As water passes through, particulate matter such as sediment and algae becomes trapped in the spaces between the sand particles. Over time, as the filter accumulates debris, the pressure differential across the filter increases. When this differential reaches a specified threshold, an automatic backflush cycle is initiated. During backflushing, the backflush valve closes off flow from the inlet manifold, and water is redirected to flow upward through the sand media from the outlet manifold, effectively dislodging and removing the accumulated contaminants from the filter.

Example 6.1
An irrigation system delivers 400 GPM (1514 LPM) to a field. To maintain adequate pressure at the emitters, the required outlet pressure from the sand filter system is 35 PSI (240 kPa). The system uses two parallel sand filters, and available filter diameters are 30, 34, and 38 inches. When clean, each filter has a pressure loss of 2 PSI (14 kPa). During normal operation, the pressure loss is expected to increase by an additional 6 PSI. Determine the appropriate filter diameter and calculate the required inlet manifold pressure.

Solution
Since the system uses two filters, each will carry half the total flow:

$$\frac{400GPM}{2} = 200GPM = 757LPM$$

Convert to LPS: $\frac{757LPM}{60} = 12.62LPS$

Using the design guideline of 18 LPS/m² maximum flow rate:

Required area $= \frac{12.62}{18} = 0.7m^2$

Now calculate the required diameter:

$$A = \frac{\pi D^2}{4} \Rightarrow D = \sqrt{\frac{4 \times 0.7}{\pi}} = 0.944 \, m = 37.2 \, in$$

Use two filters (38 in)

Expected pressure loss:

2 PSI (clean) + 6 PSI (operating) = 8 PSI = 55 kPa

Required inlet pressure:

35 PSI (outlet) + 8 PSI (loss) = 43 PSI = 296 kPa (approx.)

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6.5.4 Disc Filter

Disc filters employ a stack of grooved, disc-shaped elements designed to capture sediment between their surfaces. During normal operation, the discs are tightly compressed, allowing water to flow through while retaining debris within the grooves. When a backflush cycle is triggered, the disc stack is released, and water is forced in the reverse direction, effectively dislodging and flushing out accumulated sediment. The resulting backflush water and debris are then discharged through a dedicated outlet.

6.5.5 Screen Filter

Screen filters are commonly used in drip irrigation systems supplied by relatively clean water sources, such as wells or municipal supplies. Due to their limited filtration surface area, they are prone to rapid clogging when the water contains substantial amounts of algae or sediment. The fine mesh used in screen filters can be made from various materials; however, nylon is generally unsuitable for high-temperature environments, as it tends to stretch and lose its shape under heat.

6.6   Design Requirements

Drip irrigation is a highly efficient method of applying water directly to the root zone of plants, minimizing losses due to evaporation, runoff, and deep percolation. To achieve maximum efficiency, uniformity, and system longevity, the design of a drip irrigation system must account for a variety of technical, agronomic, and environmental factors. These include water quality, pressure regulation, emitter selection, system hydraulics, filtration, and maintenance needs. Proper system design ensures reliable operation, supports crop productivity, and reduces operational costs over time.

The hydraulic performance of an emitter is governed by the nature of fluid motion within its flow path, typically characterized by the Reynolds number (R_e). All emission devices control water discharge by dissipating energy through frictional resistance along the flow path, as described by the appropriate flow equation.

$$q = K_d \,.\, H^x \tag{6.3}$$

Where
   q: Emitter flow rate, (L/h)
   K_d: Flow coefficient, a proportionality factor that characterizes the dimensions of the emitter flow path
   H: Operating pressure head, (m)
   x: Emitter flow rate exponent, which characterizes the flow regime

In general, the values of the discharge coefficient K_d and the exponent x are provided by the manufacturer. Alternatively, they can be determined experimentally by plotting the emitter discharge (q) against pressure head (H) on a log-log scale. The slope of the resulting straight line represents the exponent x, while the intercept at $H=1$ corresponds to the coefficient K_d.

In drip irrigation systems, water is discharged through a small-diameter opening or a series of openings, where the majority of the pressure head loss occurs. The discharge rate (q) of an orifice emitter can be calculated using Equation 6.4.

$$q = C_d a \sqrt{2gH} \tag{6.4}$$

Where

q : Discharge per nozzle (m³/s)

H: Pressure head at the nozzle (m)

g: Acceleration due to gravity (9.81 m/s²)

a: Cross-sectional area of the orifice or nozzle (m²)

c_d: Coefficient that depends on the characteristics of the nozzle (0.6 to 1.0)

The manufacturer's coefficient of variation (Cv) is a key parameter used to evaluate the uniformity of emitter discharge rates under controlled conditions. As defined by the American Society of Agricultural Engineers (ASAE, 1990), Cv quantifies the inherent variability in the discharge of emitters of the same make, model, and size, as provided by the manufacturer. This assessment is conducted prior to field installation or any exposure to operational aging, ensuring that the results reflect the manufacturing quality alone.

To determine Cv, a discharge test is typically performed on a sample of 50 emitters, operating under a specified pressure and temperature (20°C). The resulting discharge data are then analyzed using the following statistical relationship:

$$Cv = \frac{\sqrt{(q_1^2 + q_2^2 \ldots + q_n^2 - nq_a^2)/(n-1)}}{q_a} \tag{6.5}$$

Where

Cv: Manufacturer's coefficient of variation (dimensionless)

q_1, q_2, ..., q_n : Individual emitter discharge rates (liters per hour, lph)

q_a = Average emitter discharge rate, calculated as: $q_a = (q_1 + q_2 + \ldots + q_n) / n$

n = Number of emitters tested (typically 50)

or

$$Cv = \frac{S_d}{q_a} \tag{6.6}$$

S_d: Estimated standard deviation of the emitter discharge rates (lph)

A lower Cv value indicates higher manufacturing uniformity and better potential for system performance in the field. Typically, Cv values below 0.05 indicate excellent manufacturing quality, whereas values above 0.10 may result in significant variations in water application uniformity.

However, when more than one emitter is used per plant, the variability in the volume of water delivered to each plant is reduced. This is because one emitter with a higher discharge rate may compensate for another with a lower rate, thereby balancing the total application. To account for this effect, an expression for the system coefficient of manufacturing variation was developed, as presented in Equation 6.7.

$$Cv_s = \frac{Cv}{\sqrt{N_p}} \tag{6.7}$$

Where

Cv_s: System coefficient of manufacturing variation

N_p: Number of emitters per plant

Table 6.1 presents the recommended classification of the manufacturer's coefficient of variation (Cv), as specified by ASAE (1990). This classification helps evaluate the manufacturing uniformity of both point-source and line-source emitters based on their Cv values.

Table 6.1 Recommended classification of manufacturer's coefficient of variation

Emitter Type	Cv Range	Classification
Point-source	< 0.05	Excellent
	0.05 to 0.07	Average
	0.07 to 0.11	Marginal
	0.11 to 0.15	Poor
	> 0.15	Unacceptable
Line-source	< 0.10	Good
	0.10 to 0.20	Average
	> 0.20	Marginal to Unacceptable

Example 6.2

A sample of 10 emitters was tested at 20°C under constant pressure. The discharge rates (in liters per hour, L/h) were recorded as follows:
(3.9, 4.0, 3.8, 4.1, 3.9, 4.0, 4.2, 3.7, 4.0, 3.8)
Determine the coefficient of variation

Solution

$$q_a = \frac{(3.9 + 4.0 + 3.8 + 4.1 + 3.9 + 4.0 + 4.2 + 3.7 + 4.0 + 3.8)}{10} = 3.94$$

$$Cv = \frac{\sqrt{(q_1^2 + q_2^2 \dots + q_n^2 - nq_a^2)/(n-1)}}{q_a} = \frac{\sqrt{\frac{155.44 - 10 \times 3.94^2}{10-1}}}{3.94} = 0.038$$

Calculate System Coefficient of Variation Cv_s
Assuming 2 emitters per plant:

$$Cv_s = \frac{Cv}{\sqrt{N_p}} = \frac{0.038}{\sqrt{2}} = 0.027$$

According to ASAE (1990), the design emission uniformity (EU) of a drip irrigation system can be estimated using the following equation, which incorporates the effects of manufacturing variation and pressure differences at the emitters:

$$EU = 100 \times \left(1.0 - \frac{1.27 C_v}{\sqrt{N_p}}\right) \times \left(\frac{q_m}{q_a}\right) \tag{6.8}$$

Where

EU: Design emission uniformity (%)

q_m: Minimum emitter discharge under the lowest operating pressure in the sub-unit (L/h)

q_a: Average or design emitter discharge in the sub-unit (L/h)

A sub-unit is a hydraulic section of an irrigation system in which the variation in emitter operating pressure does not exceed the allowable pressure limit. It is defined based on uniform pressure conditions to ensure consistent emitter performance. To support this requirement, Keller and Bliesner (1990) proposed the following equation to determine the allowable pressure variation:

$$\Delta H_s = 2.5(H_a - H_m) \tag{6.9}$$

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Where

ΔH_s: Allowable pressure variation (m), required to maintain a design EU

H_a: Pressure head required to produce the average emitter discharge q_a, consistent with the design EU (Fig. 6.4)

H_m: Pressure head required to produce the minimum emitter discharge q_m, needed to achieve the same EU

Rearranged EU Equation

$$q_m = \frac{EU.q_a}{100\left[1.0 - \frac{1.27C_v}{\sqrt{N_p}}\right]}$$

Once q_m is determined, the corresponding pressure head H_m can be calculated using the following expression, adapted from the emitter flow-pressure relationship:

$$H_m = H_a \left(\frac{q_m}{q_a}\right)^{1/x} \tag{6.10}$$

This method ensures that the pressure range across the irrigation sub-unit is sufficiently controlled to maintain the required emission uniformity, thereby supporting efficient and uniform water application.
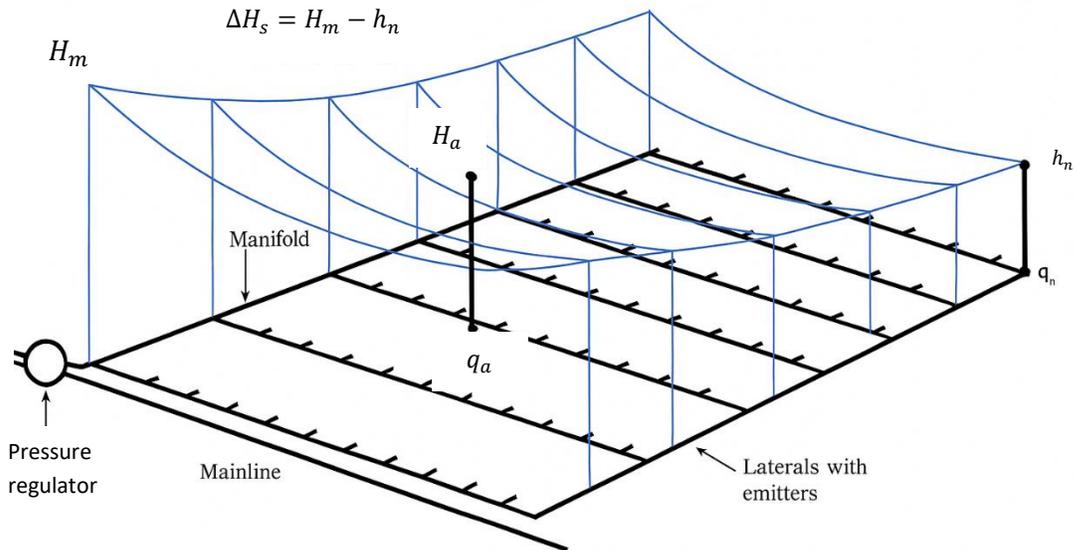

Fig. 6.4 Distribution of a pressure head in a subunit

Example 6.3

An emitter with the following characteristics, as specified by the manufacturer, is being considered for use in a drip irrigation system: a nominal discharge rate of 4 L/hr at a pressure head of 10 meters, a manufacturer's coefficient of variation $C_v = 0.07$, and a flow exponent x = 0.42. If the average emitter discharge q_a at a pressure head $H_a = 12$ meters is found to be 4.32 L/hr, and six emitters are used per tree, determine the allowable pressure variation ΔH_s when the desired design emission uniformity (EU) is 90%. Additionally, evaluate how the allowable pressure variation would change if a higher EU value of 95% were adopted.

Solution

Using the rearranged EU equation:

$$q_m = \frac{EU \times q_a}{100\left[1.0 - \frac{1.27 C_v}{\sqrt{N_p}}\right]} = \frac{90 \times 4.32}{100\left[1.0 - \frac{1.27 \times 0.07}{\sqrt{6}}\right]} = 4.03 L/hr$$

Calculate H_m using Equation 9.10:

$$H_m = H_a \left(\frac{q_m}{q_a}\right)^{1/x} = 12 \left(\frac{4.03}{4.32}\right)^{1/0.42} = 10.2\ m$$

Calculate Allowable Pressure Variation ΔH_s

$$\Delta H_s = 2.5(H_a - H_m) = 2.5(12 - 10.2) = 4.5\ m$$

During the system design process, provisions must be made to ensure that head losses and elevation differences within each sub-unit do not exceed 4.5 meters to maintain the desired uniformity.

Optional Case: Higher EU = 95%

$$q_m = \frac{95 \cdot 4.32}{100\left[1.0 - \frac{1.27 \times 0.07}{\sqrt{6}}\right]} = 4.26\ L/hr$$

$$H_m = 12 \left(\frac{4.26}{4.32}\right)^{1/0.42} = 11.6\ m$$

This leads to a smaller pressure variation:

$$\Delta H_s = 2.5(12 - 11.6) = 1.0\ m$$

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The application time (T_a) for delivering the gross volume of water required per plant during the peak use period can be calculated using Equation 6.11.

$$T_a = \frac{V}{N_p . q_a}$$ (6.11)

Where

T_a: Set time, hour

V: Average gross volume of water required per plant per day during the peak use period, L

N_p: Number of emitters per plant

q_a: Average emitter discharge, in (L/hour)

The maximum daily operating time should be limited to 90 percent of the total available time, equivalent to 21.6 hours per day. The remaining non-operating period serves as a safety margin to accommodate potential system failures or other unforeseen downtime. To determine the average emitter pressure head (H_a) corresponding to a desired average discharge (q_a), the basic emitter discharge equation is modified accordingly. The value of H_a for a specified discharge can be calculated using Equation 6.12.

$$H_a = \left(\frac{q_a}{K_d}\right)^{1/x}$$ (6.12)

The percentage wetted area (P_w) is defined as the average horizontal area within the top 30 cm of the crop root zone depth that is wetted, expressed as a percentage of the total cropped area. This parameter is important in determining the number of emitters required per plant, as it depends on both the desired percentage wetted area (P_w) and the wetted area produced by a single emitter.

The desirable value of P_w varies with crop spacing and system design. For widely spaced crops, most engineers recommend a minimum of 33% and a maximum of 67%. Keller and Bliesner (1990) describe a potential relationship between P_w and crop production, noting that P_w often approaches 100% for closely spaced crops when drip laterals are spaced less than 1.8 m apart. However, they provide no specific recommendations for widely spaced crops with rows and emitters spaced more than 1.8 m apart.

Table 6.2 presents the typical horizontal area wetted by a single emitter (A_w) for different soil types, based on data from Rainbird International (1980). The wetted area varies significantly with soil texture due to differences in water infiltration and lateral movement within the soil profile.

Table 6.2 Typical area wetted by a single emitter for different soil types

Soil Type	Area Wetted per Emitter, A_w (m²)
Sandy soils	0.5 – 2.0
Loam soils	2.0 – 6.0
Clay soils	6.0 – 15.0

Table 6.3 presents the approximate horizontal wetted dimensions (short dimension × long dimension, in meters) produced by a 4 liters-per-hour (L/h) drip emitter, considering different soil textures, root depths, and degrees of soil stratification. The wetted dimensions are influenced by the infiltration characteristics of the soil, which depend on its homogeneous, stratified, and layered.

Table 6.3 Estimated wetted areas for a 4 L/h drip emitter under various soil textures, root depths, and stratification conditions (Source: Keller and Bliesner, 1990)

Root Depth & Soil Texture	Degree of Stratification and Equivalent Wetted Area (m×m)		
Depth 0.75 m	**Homogeneous** $(S \times W)$	**Stratified** $(S \times W)$	**Layered** $(S \times W)$
Coarse (coarse–medium sands)	0.4×0.5	0.6×0.8	0.9×1.1
Medium (loamy sand–loam)	0.7×0.9	1.0×1.2	1.2×1.5
Fine (sandy clay–clay)	0.9×1.1	1.2×1.5	1.5×1.8
Depth 1.50 m			
Coarse (coarse–medium sands)	0.6×0.8	1.1×1.4	1.4×1.8
Medium (loamy sand–loam)	1.0×1.2	1.7×2.1	2.2×2.7
Fine (sandy clay–clay)	1.2×1.5	1.6×2.0	2.0×2.4

Wetted Area Dimensions:

- W = long dimension (m) ≈ wetted diameter.
- S = short dimension (m) ≈ 0.8 × W.

The required number of emitters for each plant is determined using the following relationship:

$$Emitters\ per\ plant = \frac{Area\ per\ plant \times P_w}{A_w} \tag{6.13}$$

Where

Area per plant (m²): The ground area allocated to an individual plant.

P_w: Percentage of the plant's area to be wetted, expressed as a fraction (% / 100).

A_w: Area wetted by a single emitter (m²).

Example 6.4

A medium-textured soil (loamy sand–loam) supports fruit trees planted at a spacing of 3.0 m × 3.0 m. The desired percentage wetted area (P_w) is 60%. Each tree will be irrigated using 4 L/h emitters. The soil profile is stratified to a root depth of 1.50 m. Determine the required number of emitters per plant.

Solution

Plant spacing: 3.0 m×3.0 m = 9.0 m²

Desired wetted percentage (P_w): 0.6

- Emitter discharge: 4 L/h
- Soil texture: Medium (loamy sand–loam)
- Root depth: 1.50 m
- Soil stratification: Stratified

From Table 6.3 (Keller and Bliesner, 1990), for a medium-textured soil at 1.50 m depth under stratified conditions:

- Wetted dimensions = 1.7 m (short) × 2.1 m (long) = 3.57 m²

Apply the emitter requirement equation:

$$Emitters\ per\ plant = \frac{Area\ per\ plant \times P_w}{A_w} = \frac{9 \times 0.6}{3.57} = 1.5$$

Round up to the nearest whole emitter: Since fractional emitters are not practical,

Required emitters per plant = 2

When a single lateral is used, with emitters placed at equal intervals along its length, the emitter spacing (S_e) can be determined using the following equation:

$$S_e = \frac{S_p}{N_p} \qquad (6.14)$$

Where

S_e: Emitter spacing, i.e., the distance between adjacent emitters (m)

S_p: Spacing between plants within a row (m)

Example 6.5

Given $N_p = 5$ and $S_p = 6$ m, the emitter spacing is:

$$S_e = \frac{S_p}{N_p} = \frac{6}{5} = 1.2$$

Once N_p, S_p, and S_e are determined, the percentage wetted area (P_w) should be checked to ensure it remains within the recommended range. This verification can be carried out using Equation 6.15.

$$P_w = \frac{100 \times N_p \times S_e \times W}{S_p \times S_r} \qquad (6.15)$$

Where

      W: Width of wetted strip along lateral with emitters (m)

      S_r: Distance between plant rows or row spacing (m)

The diameter of the wetted area can be calculated using the following relationship:

$$A_w = \frac{\pi . D^2}{4} \qquad (6.16)$$

Where

      D : Diameter of the wetted area (m)

If the area wetted by one emitter is estimated at $A_w = 4$ m^2, rearranging Equation 9.16 to solve for D gives:

$$D = \sqrt{\frac{4 \times A_w}{\pi}} = \sqrt{\frac{4 \times 4}{\pi}} = 2.26 \ m$$

$$P_w = \frac{100 \times N_p \times S_e \times W}{S_p \times S_r} = \frac{100 \times 5 \times 1.2 \times 2.26}{6 \times 6} = 38\%$$

In this case, the percentage wetted area (P_w) achievable using a single lateral is unsatisfactory. Increasing the number of emitters along the same lateral will not improve P_w, because according to Equation 6.14, the product ($N_p \times S_e$) remains constant for a fixed plant spacing (S_p). Consequently, P_w in Equation 6.15, also remains unchanged. Therefore, to achieve the desired wetted area, two emitter lines (laterals) must be used. For uniform wetting between the two laterals, their spacing should not exceed 80% of the wetted diameter, calculated as:

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$0.8 \times 2.26 = 1.81$ m

The total number of emitters per plant should be approximately the same as that for a single-line arrangement. For uniformity, the emitters should be equally divided between the two lines. Since Equation 6.13 resulted in five emitters per plant, the nearest even total would be six emitters or three emitters per lateral.

The percentage wetted area is then:

$$P_w = \frac{100 \times 6 \times 2 \times 1.81}{6 \times 6} = 60\%$$

Although this value slightly exceeds the target P_w of 50%, it is preferable to adopt it rather than accept the much lower 38% achieved with a single line.
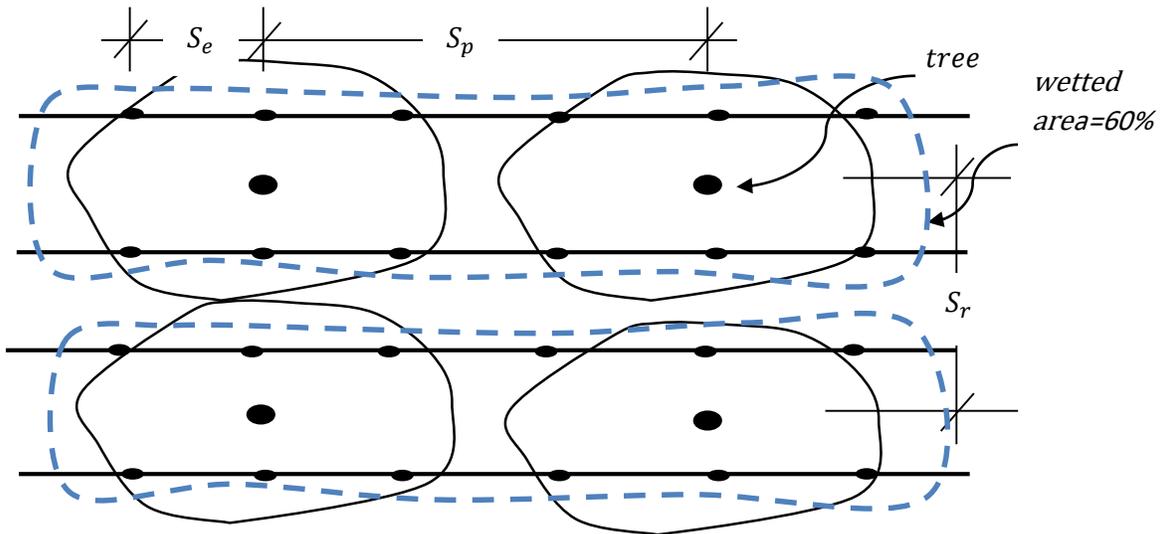

Fig. 6.5 Plant and emitter distance

The head loss due to pipe friction (H_f) in laterals and manifolds with evenly spaced outlets and uniform discharge from each outlet can be estimated using Equation 6.17.

$$H_f = F.H_{fno\ outlets} \tag{6.17}$$

The head loss in a multiple-outlet pipeline with uniform outlet spacing and discharge can be calculated using Equation 6.18:

$$H_x = F.H_f.\left(\frac{X}{L}\right)^K \tag{6.18}$$

Where

K : 2.852 for the Hazen–Williams equation

H_x: Head loss from position X to the closed end, ft (m)

H_f: Total head loss of the pipe with emitters, ft (m)

F: Reduction coefficient accounting for the discharge along the pipe

X: Distance from the closed end, ft (m)

The mathematical derivation of Equation 9.18 assumes that F remains constant between the closed end and any point along the multiple-outlet pipeline. While this assumption is not strictly accurate, for pipelines with 12 or more outlets, the resulting error is typically less than 5 percent.

Table 6.4 presents F values for different numbers of openings along the pipe, provided for use with the Hazen–Williams formula (flow rate exponent 1.852).

Table 6.4 Reduction coefficient (F) for multiple-outlet pipelines using the Hazen–Williams formula

Number of Outlets	F (Exponent = 1.852)	Number of Outlets	F (Exponent = 1.852)
1	1.00	8	0.42
2	0.64	9	0.41
3	0.54	10–11	0.40
4	0.49	12–15	0.39
5	0.46	16–20	0.38
6	0.44	21–30	0.37
7	0.43	31–70	0.36

Example 6.6

A drip irrigation lateral is made of smooth PVC pipe (C = 150), with a total length of L= 60 m. The lateral has 80 emitters evenly spaced along the pipe, each discharging q =2 L/hr. The inlet flow rate is therefore the sum of all emitters.

Determine:

1. The total head loss due to friction in the lateral.
2. The head loss between the midpoint and the closed end.
3. The additional local loss due to one inline filter (K_r = 2.5).

Solution:

$Q_{in} = N \times q = 80 \times 2/1000 = 0.16 \ m^3/hr$

Hazen–Williams formula, (Use pipe diameter D =16 mm).

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$$H_f = 1.14 \times \frac{10^9 \left(\frac{Q}{C}\right)^{1.852} L}{D^{4.87}} = 1.14 \times 10^9 \left(\frac{0.16}{150}\right)^{1.852} \times \frac{60}{16^{4.87}} = 0.293\ m$$

F≈ 0.36 (closest table value for ≥ 70 outlets), F = 0.36 ⇒ H_f = 0.36×0.293 ≈ 0.1055 m

Head loss from midpoint to closed end, from Eq. (6.18):

$$H_x = F.H_f.\left(\frac{X}{L}\right)^K = 0.1055(0.5)^{2.852} = 0.0146\ m$$

Local loss from filter

$$V = \frac{Q_{in}}{A} = \frac{4.44 \times 10^{-5}}{\frac{\pi \times 0.016^2}{4}} = 0.22 m/sec, \quad \text{Velocity head: } \frac{V^2}{2g} = \frac{0.22^2}{2 \times 9.81} = 2.47 \times 10^{-3}$$

Local loss: $K_r \frac{V^2}{2g} = 2.5 \times 2.47 \times 10^{-3} = 6.167 \times 10^{-3} m,$

$$H_{ftotal} = 0.1055 + 6.167 \times 10^{-3} = 0.1117 m$$

The most commonly used dripline sizes are polyethylene pipes with inside diameters of 12 mm and 16 mm. Other available sizes include 20 mm, 25 mm, 32 mm, and 40 mm. A number of fundamental characteristics of laterals must be carefully considered during the design process.

Length — A pair of laterals extends in opposite directions from the same manifold inlet, with length *l* equal to half the manifold spacing (*Sm*/2). For a single lateral extending in one direction, *l* represents its actual length (Fig. 6.6).
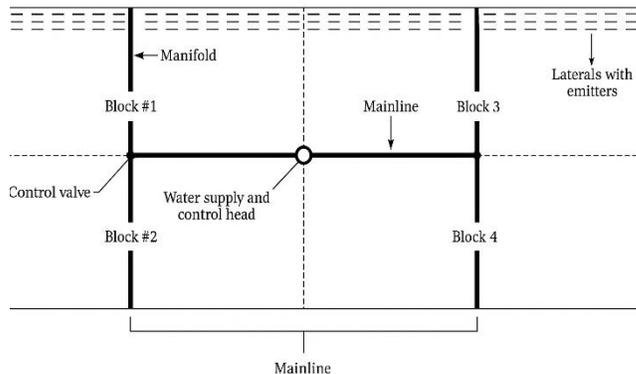

Fig. 6.6 Typical two-station split-flow layout of a drip irrigation system, with Blocks I and III or Blocks II and IV operating simultaneously.

The figure illustrates a typical drip irrigation system layout, where the mainline delivers water from the water supply and control head to multiple irrigation blocks. Each block is supplied through a manifold connected to laterals with emitters, which distribute water

directly to the plants. Control valves are installed at the entry of each block to regulate water flow and isolate sections when needed. The system is divided into four blocks (Block #1, Block #2, Block #3, and Block #4) to allow for efficient irrigation scheduling and pressure management.

Flow Rate — The flow rate of a lateral (q_L) is determined using Equation 6.19:

$$q_L = \frac{n_e \times q_a}{60} \tag{6.19}$$

Where

q_L: Lateral flow rate, (L/min)
q_a: Average emitter discharge, in (L/hour)
n_e: Number of emitters along the lateral

Inlet Pressure — In some cases, it is useful to determine the inlet pressure required for the *average lateral* in a system. The average emitter pressure head (H_a) is defined as the head that produces the corresponding average emitter discharge (q_a). For constant-diameter laterals, the average emitter position that yields q_a at H_a is generally located between $x/L = 0.60$ and $x/L = 0.62$, measured from the downstream end of the lateral. Approximately three-fourths of the total head loss occurs between the average emitter and the inlet, where the flow rate is highest. As flow decreases along the lateral due to water discharge through the emitters, the head-loss curve flattens, resulting in only about one-fourth of the total loss occurring between the average emitter and the downstream end.
For a pair of constant-diameter laterals, each with length L equal to the manifold spacing (S_m), laid on a uniform slope, the inlet pressure head (H_i) required to produce the average emitter head (q_a) can be calculated using Equations 6.20 and 6.21:

$$H_i = H_a + 0.75 H_f [z^k + (1-z)^k] - \left(\frac{\Delta E}{2}\right)(2z - 1) \tag{6.20}$$

Where

H_i: Lateral inlet pressure head, (m)
k: 3.852 for Hazen–Williams equation;
H_f: Friction loss in a lateral of length L, (m)
z: Location ratio of the inlet point to the pair of laterals that ensures equal minimum pressures in both uphill and downhill laterals (expressed as the length of the downhill lateral divided by L)
ΔE: Absolute elevation difference between the two ends of the paired laterals, (m)
For level fields this reduces to:

$$H_i = H_a + 0.75H_f(0.5)^k \tag{6.21}$$

Where

k: 2.852 for Hazen–Williams equation.

For a single, non-paired, constant-diameter lateral installed on a uniform slope, H_i can be determined using Equation 6.22.

$$H_i = H_a + \frac{3H_f}{4} + \frac{\Delta EI}{2} \tag{6.22}$$

Where

ΔEI : Change in elevation, positive for laterals running uphill from the inlet and negative for laterals running downhill, expressed in (m).

The pressure head at the closed end of the lateral (H_d) is calculated using Equation 6.23.

$$H_d = H_a - \left(\frac{H_f}{4} + \frac{\Delta EI}{2} \right) \tag{6.23}$$

Or

$$H_d = H_i - \left(H_f + \Delta EI \right) \tag{6.24}$$

After calculating the friction loss for a given lateral length, the friction loss for any other lateral length can be determined using Equation 6.25.

$$\left(H_f \right)_b = \left(H_f \right)_a \left(\frac{L_b}{L_a} \right)^k \tag{6.25}$$

Where

k : 2.852 for Hazen-Williams equation
L_a, L_b: Original and revised lateral pipe length (m)
$\left(H_f \right)_a, \left(H_f \right)_b$: Original and revised lateral pipe friction losses (m)

Conversely, the lateral length (L_b) required to produce a specified $\left(H_f \right)_b$ can be calculated using the following equation.

$$L_b = L_a \left(\frac{\left(H_f \right)_b}{\left(H_f \right)_a} \right)^k \tag{6.26}$$

Where

       k : 0.35 for Hazen-Williams equation

Example 6.7

A drip irrigation system consists of a pair of constant-diameter laterals extending in opposite directions from a common inlet point on a manifold. The laterals have the following characteristics:

- Inside diameter: 16 mm
- Manifold spacing (Sm): 160 m
- Emitter spacing: 1 m
- Average emitter discharge (q_a): 4 L/h
- Number of emitters per lateral (n_e): 80
- Friction loss (H_f) for 80 m length: 4.0 m
- Average emitter head (H_a): 10 m
- Field slope: Level ($\Delta E = 0$)

Determine:

1. Length definition for paired laterals.
2. Flow rate of the lateral.
3. Required inlet pressure head.
4. Pressure head at the downstream closed end.
5. Friction loss for a shorter lateral of 50 m.

Solution

For a pair of laterals: L = $Sm/2$ = 80 m

Each lateral extends 80 m from the inlet, one uphill and one downhill (in this case, level field).

From Equation 6.19: $q_L = \dfrac{n_e \times q_a}{60}$

$q_L = \dfrac{80 \times 4}{60} = 5.33\ L/min$

From Equation 9.21 for level fields: $H_i = H_a + 0.75 H_f (0.5)^k$

$H_i = 10 + 0.75 \times 4 \times (0.5)^{2.852} = 10.42\ m$

Pressure Head at the Downstream End: From Equation 6.23

$H_d = H_a - \left(\dfrac{H_f}{4} + \dfrac{\Delta EI}{2}\right) = 10 - \dfrac{4.0}{4} = 9.0\ m$

Friction Loss for Shorter Lateral: From Equation 6.25

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$$\left(H_f\right)_b = \left(H_f\right)_a \left(\frac{L_b}{L_a}\right)^k = 4.0 \left(\frac{50}{80}\right)^{2.852} \approx 1.0 \text{ m}$$

Now, when using location ratio of downhill lateral length to L: $z = 0.55$,

Calculation of $z^k + (1-z)^k = 0.55^{3.852} + (1-z)^{3.852} = 0.146$

Case 1: Level field ($\Delta E = 0.0$ m)

$$H_i = H_a + 0.75H_f[z^k + (1-z)^k] - \left(\frac{\Delta E}{2}\right)(2z-1) = 10 + 0.75(4) \times 0.146$$

$H_i = 10.438m$

Case 2: Uniform slope ($\Delta E = 2.0$ m)

$$\left(\frac{\Delta E}{2}\right)(2z-1) = \frac{2}{2}(2 \times 0.55 - 1) = 0.1$$

Substitute: $10 + 0.75(4) \times 0.146 - 0.1 = 10.34 \ m$

Interpretation: The inlet pressure head is slightly lower on the sloped field because the downhill side benefits from a small gravitational head. The term $z^k + (1-z)^k$ accounts for the inlet position, ensuring that both uphill and downhill laterals operate with equal minimum pressures. The factor $0.75H_f$ represents the proportion of friction loss occurring between the inlet and the average emitter position that yields q_a, which is typically located about 60–62% of the lateral length from the downstream end.

Table 6.5 presents the recommended manifold location ratio z for paired laterals on sloping terrain, where one lateral extends uphill and the other downhill from the manifold. The ratio z is defined as the length of the downhill lateral divided by the total lateral length L. Values are provided as a function of the dimensionless slope parameter $\frac{\Delta E}{H_f}$.

Table 6.5 Recommended manifold position ratio (z) for paired laterals on sloping fields

$\dfrac{\Delta E}{H_f}$	z	$\dfrac{\Delta E}{H_f}$	z
0.0	0.50	1.0	0.85
0.1	0.56	1.2	0.89
0.2	0.60	1.4	0.92
0.3	0.65	1.6	0.94
0.4	0.69	1.8	0.96
0.5	0.72	2.0	0.98
0.6	0.75	2.2	0.99
0.7	0.78	2.4	1.00
0.8	0.81	2.6	1.00
0.9	0.83	2.75	1.00

The table shows how the optimal manifold position ratio (z) for paired laterals on sloping fields changes with the ratio of elevation difference (ΔE) to friction loss (H_f). When $\frac{\Delta E}{H_f}$ is small (0.0–0.4), meaning the slope effect is minor compared to friction loss (see Fig. 6.7),

the manifold should be located near the midpoint between the two laterals ($z \approx 0.50$–0.69) to maintain balanced minimum pressures. As $\frac{\Delta E}{H_f}$ increases to moderate values (0.5–1.0), the manifold is shifted progressively uphill (increasing z from ≈ 0.72 to ≈ 0.85) so that the downhill lateral becomes longer, compensating for the greater gravitational head it receives. For steep slopes ($\frac{\Delta E}{H_f} > 1.0$), the optimal position moves even further uphill ($z \approx 0.89$–1.00), and in extreme cases ($\frac{\Delta E}{H_f} \geq 2.4$) the manifold is placed at the very uphill end ($z = 1.00$), with the entire lateral length extending downhill to take maximum advantage of gravity.
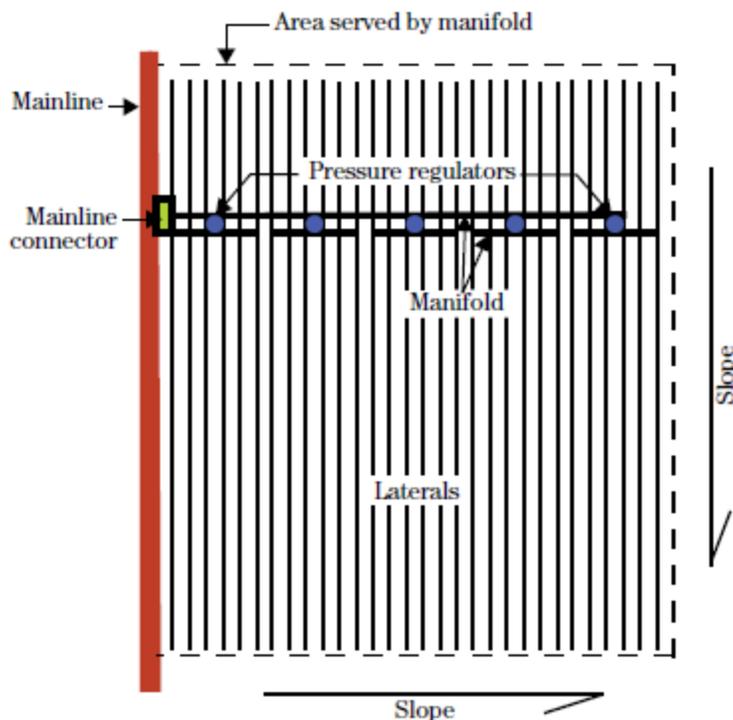

Fig. 6.7 Manifold layout with the inlet connection positioned uphill from the center, featuring pressure-regulated manifolds.

On fields where the average slope along the manifolds is less than 3%, it is generally more economical to install manifolds extending both uphill and downhill from a common mainline connection. The inlet from the mainline should be positioned so that the minimum operating pressures in both manifolds are equal.

On level ground, this condition is achieved by making the uphill and downhill manifolds equal in length. However, when the ground slopes along the manifold alignment (across the crop rows), the inlet should be shifted uphill from the midpoint. This adjustment

shortens the uphill manifold and lengthens the downhill manifold, balancing the combined effects of friction loss and elevation difference.

The allowable variation in manifold pressure head can be calculated using Equation 6.27.

$$(\Delta H_m)_a = \Delta H_s - \Delta h' \tag{6.27}$$

Where

$(\Delta H_m)_a$: Allowable manifold pressure head variation

ΔH_s: The allowable subunit pressure variation, (m)

$\Delta h'$: The greater of Δh or Δhc, the lateral line pressure variation, (m).

For simplification, the design procedure assumes laterals operate at the average emitter flow rate (q_a). Accordingly, for manifolds serving rectangular subunits, the lateral flow rate (q_L) is treated as constant.

On level fields, laterals should extend equally to both sides of the manifold, with each length (L) equal to half the manifold spacing (Sm/2). On sloping fields, the manifold should be positioned uphill from the subunit centerline, as illustrated in Fig. 6.7. The optimal manifold location providing equal minimum and maximum pressures in both uphill and downhill laterals can then be determined.

When designing manifold and lateral layouts for sloped fields, it is essential to position the manifold such that pressure distribution in both uphill and downhill laterals remains balanced. The optimum location can be determined using dimensionless relationships between elevation difference, friction loss, and manifold spacing. Fig. 6.8 illustrates the dimensionless parameters used in this procedure.

1. Determine the friction head loss (H_f) and the multiple-outlet factor (F) for a single lateral whose length is equal to half the manifold spacing (*Sm*/2).
2. When the absolute elevation difference along the lateral (ΔE) is less than the friction loss (H_f), compute the tangent location ratio (Y) using Eq. 9.28. If $\Delta E > H_f$, then set $Y = 1$. The value of Y represents the ratio *x/L* at which the friction curve is tangent to the ground profile (see Fig. 6.8).

$$Y = \left(F\frac{\Delta E}{H_f}\right)^k \tag{6.28}$$

Where

Y: Ratio *x/L* at the tangent point between the friction curve and the ground

F : Multiple-outlet factor

k : 0.54 for the Hazen–Williams equation.

3. Calculate the optimum x/L value (z) that satisfies equation (6.29) (Keller and Bliesner, 1990):

$$\frac{\Delta E}{H_f} - 0.36 \left(\frac{\Delta E}{H_f}\right)^{k1} = (z)^{k2} - (1-z)^{k2} \tag{6.29}$$

Where

z: Optimum manifold location that will give the equal minimum and maximum pressures in both uphill and downhill laterals

$k1$: 1.54 for the Hazen–Williams equation

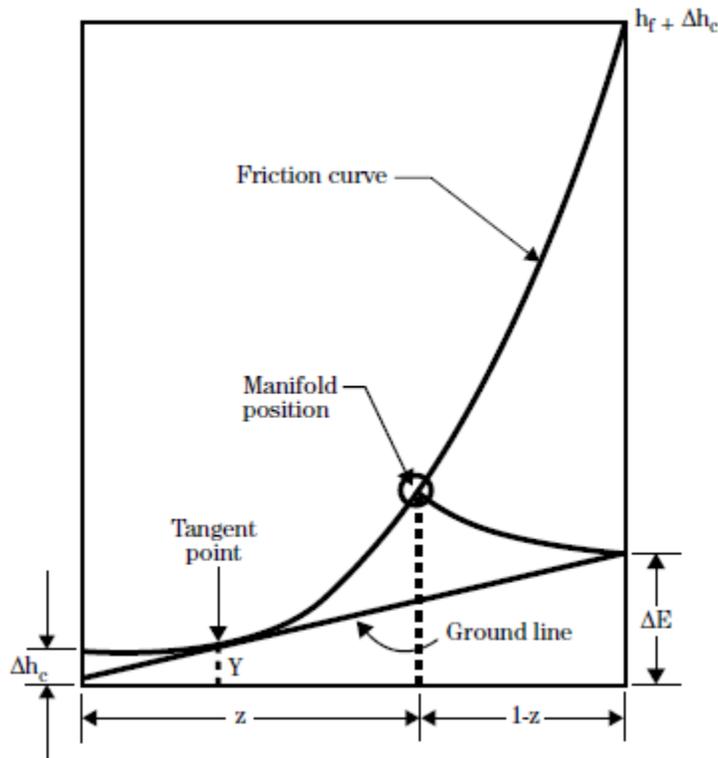$k2$: 2.75 for the Hazen–Williams equation

Fig. 6.8 Illustration depicting the relationship between manifold placement and lateral hydraulics in a paired-lateral configuration.

To apply the equation, first calculate the term on the left-hand side. Then, using a trial-and-error approach, identify the x/L value that satisfies the equation. Alternatively, Table 6.5 offers a non-iterative method for estimating the optimum manifold position, providing a reasonably accurate solution (Keller and Bliesner, 2000).

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4. For laterals installed on relatively mild slopes, the maximum pressure head variation (Δh) along a paired-lateral system can be determined from the x/L (or z) value representing the selected manifold location, using equation (6.30) (Keller and Bliesner, 1990).

$$\Delta h = \Delta E(1-z) + H_f(1-z)^k \tag{6.30}$$

Where

k : 2.852 for Hazen-Williams equation

On steep slopes, the maximum pressure head variation (Δh) may occur at the closed end of the lateral. To evaluate this, calculate the difference (Δh_c) between the downstream-end pressure head and the minimum pressure head using equation (6.31) (Keller and Bliesner, 2000).

$$\Delta h_c = \Delta E(Y) - H_f(Y)^k \tag{6.31}$$

Where

k : 2.852 for Hazen-Williams equation

For relatively mild slopes, the maximum pressure head difference (Δh) along the lateral can be calculated using equation (6.32)

$$\Delta h = \Delta h_c = \Delta E H_f \tag{6.32}$$

Manifolds are typically tapered and constructed using two, three, or four different pipe sizes. To ensure proper flushing, the diameter of the smallest pipe should be at least half the diameter of the largest pipe. Flow velocity in manifolds should generally be limited to about (2.13 m/s), which is higher than the (1.52 m/s) commonly used in mainlines. This higher limit is acceptable because the outlets along a manifold are always open, which helps dampen water-hammer effects.

The length of a paired manifold system (L) can be determined using equation (6.33).

$$L_p = \left[(n_r)_p - 1\right]S_r \tag{6.33}$$

Where

L_p: Length of a paired manifold system (m)

$(n_r)_p$: Number of row (or lateral) spacings served from a common inlet point

S_r: Row spacing (m)

The length of a single manifold (Lm) is typically determined using equation (6.34)

$$L_m = \left(n_r - \frac{1}{2}\right) S_r \tag{6.34}$$

The manifold flow rate is calculated by multiplying the number of lateral pairs along the manifold by the flow rate of each pair. The flow rate of a lateral (q_L) is determined using Equation 6.19.

Example 6.8
A paired-lateral manifold system is to be installed in a rectangular subunit of a drip irrigation system. The manifold alignment runs perpendicular to the crop rows and follows a uniform slope of **1.5%**. The design data are as follows:

Parameter	Value
Row (lateral) spacing S_r	1.5 m
Manifold spacing Sm	144 m
Average emitter discharge (q_a)	4 L/h
Number of emitters per lateral	24
Hazen–Williams coefficient C	150
Allowable subunit pressure variation (ΔH_s)	3.0 m
Average lateral length for hydraulic calc.	72 m
Elevation slope along manifold alignment	1.5% uphill from inlet

The objective is to:

1. Determine the optimum inlet location of the manifold so that the minimum pressures in the uphill and downhill laterals are equal.
2. Calculate the allowable manifold pressure head variation $(\Delta H_m)_a$.
3. Determine the maximum pressure variation Δh along the laterals.
4. Find the length and flow rate of the manifold.

Solution:
Determine lateral flow rate (q_L)
$$q_L = \frac{n_e \times q_a}{60} = \frac{24 \times 4}{60} = 1.6 L/min$$
Friction head loss in one lateral: We use the Hazen–Williams equation for friction loss, assume D = 12 mm.

$$H_f = 1.14 \times \frac{10^9 \left(\frac{Q}{C}\right)^{1.852} L}{D^{4.87}} = 1.14 \times 10^9 \left(\frac{0.096}{150}\right)^{1.852} \times \frac{72}{12^{4.87}} = 0.554m$$

From Table 6.4 (Christiansen's method, Hazen–Williams), for laterals with uniform spacing and continuous outlets: F≈ 0.37

Absolute elevation difference along the lateral: Slope = 1.5% = 0.015 m/m, Length = 72 m.

$\Delta E = 0.015{\times}72 = 1.08$ m

Since ΔE, per the stated rule set Y=1 (Eq. 9.28 bypassed).

Optimum manifold location z = x/L (Eq. 9.29)

$$\frac{\Delta E}{H_f} - 0.36\left(\frac{\Delta E}{H_f}\right)^{k1} = (z)^{k2} - (1-z)^{k2}$$

$$\frac{1.08}{0.554} - 0.36\left(\frac{1.08}{0.554}\right)^{1.54} = 0.943, \qquad \text{solve } (z)^{k2} - (1-z)^{k2} = 0.943$$

$z \approx 0.980$

Interpretation: place the inlet at (z. Sm) from the downhill end: $x = 0.98 \times 144 = 141.12m$, the slope dominates, so the inlet shifts very near the uphill side.

Maximum lateral pressure variation, Δh (Eq. 6.30)

$\Delta h = \Delta E(1-z) + H_f(1-z)^k, \qquad \Delta h = 1.08(1-0.98) + 0.554(1-0.98)^{2.852} = 0.0216m$

Closed-end check (steeper-slope condition), Δh_c (Eq. 6.31)

$\Delta h_c = \Delta E(Y) - H_f(Y)^k$, with Y= 1:

$\Delta h_c = 1.08 - 0.554 = 0.526m$

Thus $\Delta h' = $ max (Δh or Δh_c) = 0.526 m

Allowable manifold pressure variation $(\Delta H_m)_a$ (Eq. 6.27)

$(\Delta H_m)_a = \Delta H_s - \Delta h'$

$(\Delta H_m)_a = 3.0 - 0.526 = 2.474m$

Manifold length(s) and flow

If 24 laterals (12 pairs) are served from a common inlet:

Paired-manifold length (Eq. 6.33):

$L_p = \left[(n_r)_p - 1\right]S_r = [24 - 1] \times 1.5 = 34.5m$

Single-manifold length (Eq. 6.34), if one side has $n_r=12$:

$$L_m = \left(n_r - \frac{1}{2}\right)S_r = (12 - 0.5) \times 1.5 = 17.25m$$

$q_L=$ 96 L/h

12 lateral "pairs" along the manifold, in a paired layout each outlet station feeds two laterals, so the inlet flow is based on pair flow = $2 \times 96 = 192$ L/hr.

Manifold inlet flow:

$Q_m = 12 \times 192 = 2304\ L/hr.$